# CONTENTS

<table>
<thead>
<tr>
<th>Abstract</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Introduction</td>
<td>1</td>
</tr>
<tr>
<td>2. Soil properties</td>
<td>1</td>
</tr>
<tr>
<td>3. Details of wall construction</td>
<td>1</td>
</tr>
<tr>
<td>4. Field instrumentation</td>
<td>6</td>
</tr>
<tr>
<td>4.1 Earth and water pressures</td>
<td>6</td>
</tr>
<tr>
<td>4.2 Ground movements</td>
<td>6</td>
</tr>
<tr>
<td>4.3 Wall movements and bending moments</td>
<td>9</td>
</tr>
<tr>
<td>5. Discussion of results</td>
<td>9</td>
</tr>
<tr>
<td>5.1 In situ ground conditions prior to construction</td>
<td>9</td>
</tr>
<tr>
<td>5.2 Installation of diaphragm wall</td>
<td>9</td>
</tr>
<tr>
<td>5.2.1 Earth and water pressures</td>
<td>9</td>
</tr>
<tr>
<td>5.2.2 Ground movements</td>
<td>12</td>
</tr>
<tr>
<td>5.3 Underpass construction</td>
<td>12</td>
</tr>
<tr>
<td>5.3.1 Earth and water pressures</td>
<td>12</td>
</tr>
<tr>
<td>5.3.2 Wall movements</td>
<td>17</td>
</tr>
<tr>
<td>5.3.3 Ground movements</td>
<td>18</td>
</tr>
<tr>
<td>5.3.4 Wall bending moments</td>
<td>22</td>
</tr>
<tr>
<td>5.4 Post-construction</td>
<td>23</td>
</tr>
<tr>
<td>5.4.1 Earth and water pressures</td>
<td>23</td>
</tr>
<tr>
<td>5.4.2 Wall and ground movements</td>
<td>23</td>
</tr>
<tr>
<td>6. Summary and conclusions</td>
<td>27</td>
</tr>
<tr>
<td>7. Acknowledgements</td>
<td>27</td>
</tr>
<tr>
<td>8. References</td>
<td>27</td>
</tr>
</tbody>
</table>
Ownership of the Transport Research Laboratory was transferred from the Department of Transport to a subsidiary of the Transport Research Foundation on 1st April 1996.

This report has been reproduced by permission of the Controller of HMSO. Extracts from the text may be reproduced, except for commercial purposes, provided the source is acknowledged.
BEHAVIOUR DURING CONSTRUCTION OF A PROPPED DIAPHRAGM WALL IN STIFF CLAY AT THE A406/A10 JUNCTION

ABSTRACT

Field instrumentation has been installed to monitor the behaviour of a counterforted diaphragm retaining wall founded in over-consolidated clay during and immediately after its construction as part of the A406 North Circular Road, Great Cambridge Road Junction Improvement Scheme in North London. The construction sequence involved installation of the wall under bentonite followed by excavation below temporary props and casting of a permanent reinforced concrete prop slab below the final carriageway level.

Measurements of ground movements, total lateral stresses and porewater pressures were made both during installation of the wall and construction of the underpass. The wall itself was instrumented to monitor the development of lateral movements and bending moments.

1. INTRODUCTION

For urban roads, requirements of landtake and environmental considerations are necessitating increased construction below ground level in retained cuttings and in cut-and-cover tunnels. Such structures are often formed from embedded retaining walls installed from ground level using techniques of diaphragm or bored pile construction. Improvements in methods for predicting the behaviour of embedded walls and of the adjoining ground are required, particularly for stiff over-consolidated clays on which many urban areas are founded. Uncertainties in the magnitude of the in-situ lateral stress and the stress changes caused by the construction make it difficult to predict reliably the bending moments developed in the wall and the ground and wall movements which will take place during construction and in the longer term.

Data on the performance during construction of the walls of the Bell Common Tunnel (M25) and the Chapel-en-le-Frith Bypass (A6) have already been reported (Tedd et al, 1984; Darley et al, 1990) as have studies on the long term performance of walls on the A329(M) and the A3 that have been in service for over 15 years (Symons and Carder, 1990). This report provides a further case study of the performance of a counterfort diaphragm wall during and immediately after its construction as part of the A406/A10 Junction Improvement Scheme in North London.

Field instrumentation was installed prior to the start of any construction works to determine the initial stress state in the ground and to establish datum readings for the measurement of ground movements during construction. During wall installation, instruments were installed in two of the wall panels to measure their deflection and the bending moments which developed during subsequent stages of construction.

2. SOIL PROPERTIES

The soil profile established at the instrumented section is shown in Fig 1, together with data on the plasticity, natural moisture content and undrained shear strength. Made ground at the surface overlies a 1.3m thick deposit of firm sand and gravel. Below a depth of about 2.4m (corresponding to 15.7m above the Newlyn ordnance datum) firm silty clay characteristic of the London clay formation was encountered. The clay became stiffer with depth as shown by the undrained strength data in Fig 1 obtained from triaxial tests on 100mm diameter specimens from high quality thin-walled tube samples. Index tests on the London clay gave average values of plastic and liquid limit of 24% and 73% respectively. The soil profile agreed closely with that obtained during the original site investigation for the scheme which also showed that the London clay extended to a depth well beyond the toe of the retaining walls.

Consolidated undrained triaxial compression tests were carried out on sets of four 38mm diameter specimens cut from 100mm diameter thin-walled tube samples of the London clay taken at depths of up to 12m in the instrumented area. These results are compared with those obtained from nearby boreholes 111 and 112 of the original site investigation in Fig 2. Reasonable agreement between the TRRL and site investigation results was obtained at mean effective stress levels of less than 500 kN/m². In this stress range, best fit parameters of c' = 13 kN/m² and φ' = 27° and lower bound parameters of c' = 0 and φ' = 24° were calculated for the clay.

3. DETAILS OF WALL CONSTRUCTION

Construction of the underpass for the North Circular Road was commenced in 1987 and the road opened to traffic early in 1990. The dates for each of the main stages of construction at the instrumented section are given in Table 1.
Figure 1 Properties of the soil

Figure 2 Evaluation of strength parameters of London clay
Table 1
Construction sequence at instrumented section of south wall

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
<th>Period</th>
<th>Schematic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Installation of guide walls</td>
<td>14-7-87</td>
<td><img src="image1" alt="End of stage 1" /></td>
</tr>
<tr>
<td></td>
<td>Installation of wall panels under bentonite</td>
<td>11-9-87 to 11-4-88</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Temporary excavation to 2 m on both sides of wall; panels reduced to cut off levels.</td>
<td>5-2-88 to 9-2-88</td>
<td><img src="image2" alt="End of stage 3" /></td>
</tr>
<tr>
<td></td>
<td>Retained side backfilled</td>
<td>12-4-88</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Capping beam and parapet built</td>
<td>16-5-88 to 14-8-88</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Construction of slip road on retained side</td>
<td>30-9-88 to 11-10-88</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slip road opened</td>
<td>16-10-88</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Excavation to 1.7 m</td>
<td>21-2-89</td>
<td><img src="image3" alt="End of stage 4" /></td>
</tr>
<tr>
<td></td>
<td>Temporary props installed and concreted to wall</td>
<td>10-3-89 to 17-3-89</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Excavation to 5 m</td>
<td>20-3-89</td>
<td><img src="image4" alt="End of stage 5" /></td>
</tr>
<tr>
<td>5</td>
<td>Excavation to 6 m</td>
<td>21-3-89</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Excavation to 6.3 m</td>
<td>22-3-89</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Excavation completed to north wall</td>
<td>5-4-89</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Prop slab with hinges cast</td>
<td>12-7-89</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Temporary props released</td>
<td>2-8-89 to 3-8-89</td>
<td><img src="image5" alt="End of stage 6" /></td>
</tr>
<tr>
<td>7</td>
<td>Carriageway construction</td>
<td>14-12-89</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Road opened</td>
<td>25-2-90</td>
<td></td>
</tr>
</tbody>
</table>
During stage 1 the excavation for each wall panel was carried out using a mechanical grab operating between concrete guide walls (Fig 3). Throughout the period when the trench for each panel was open, a bentonite slurry was used to provide support. Detailed information on the panel installation sequence in the instrumented area is given in Table 2. Generally between one and three days were required for the excavation of each T-shaped panel with installation of the reinforcing cage (Fig 4) and pouring of the concrete taking place on the following day. In plan each T-panel consisted of a 4.0m by 0.8m front section with a 2.7m by 0.8m counterfort. T-panels 81 to 83 at the instrumented section penetrated from original ground level to a depth of about 13.5m, i.e. from 18m to 4.2m.

**Fig.3 Excavation for the T-shaped wall panels**

**TABLE 2**

<table>
<thead>
<tr>
<th>Panel Number</th>
<th>Start of excavation</th>
<th>End of excavation</th>
<th>Concreting of panel</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>29-3-88</td>
<td>29-3-88</td>
<td>30-3-88</td>
</tr>
<tr>
<td>81</td>
<td>8-4-88</td>
<td>9-4-88</td>
<td>11-4-88</td>
</tr>
<tr>
<td>82</td>
<td>17-9-87</td>
<td>18-9-87</td>
<td>19-9-87</td>
</tr>
<tr>
<td>83</td>
<td>11-9-87</td>
<td>14-9-87</td>
<td>15-9-87</td>
</tr>
<tr>
<td>84</td>
<td>4-9-87</td>
<td>7-9-87</td>
<td>8-9-87</td>
</tr>
<tr>
<td>85</td>
<td>14-9-87</td>
<td>15-9-87</td>
<td>17-9-87</td>
</tr>
</tbody>
</table>

Note: Excavation to 2m depth in front of the wall took place on 4-2-88.
Fig. 4 Installing the reinforcing cage for a panel

4.5m above the Newlyn ordnance datum. Panel 84 was constructed without the counterfort and penetrated from 18m to 5.4m above the ordnance datum.

Following installation of the wall, an excavation to about 2m depth was carried out close to and on both sides of the wall to provide access to trim each panel to a cut-off level of 17.5m AOD. Construction of the reinforced concrete capping and parapet unit then took place, followed by construction of the slip road on the retained side of the wall (stage 2 in Table 1). The slip road was opened to the diverted North Circular Road traffic in October 1988, so that work could then commence within the underpass.

Excavation was then carried out across the width of the underpass to a depth of about 2m in preparation for the installation of temporary props (stage 3). The propping was achieved by using nominal 800mm diameter steel tubes, acting in compression, which spanned between the two walls as shown in Fig 5. These props were positioned at a level of 17.3 m AOD at the centres of each panel within the instrumented area. The ends of the props were plated and concreted to the surface of the diaphragm walls.

During March and April 1989 bulk excavation to a depth of 6.3m from original ground level (stage 4 in Table 1) took place beneath the temporary props. Excavation of the London clay was carried out by bucket excavator and dozer with the spoil being lifted over the north wall in the instrumented area and transported away by lorry (Fig 6).

A permanent reinforced concrete prop slab of 600mm thickness was then cast in the form of a shallow V-shape below the final carriageway level (Fig 7). Precast concrete hinges were used at the joints between the prop and each panel of the diaphragm wall to accommodate heave of the underlying clay. The bearing surface of the hinges consisted of a stainless steel tube and a stainless steel sheet of similar curvature with PTFE between. In deeper sections of the underpass outside of the instrumented area a further hinge was built into the centre line of the slab to provide additional articulation.

After construction of the permanent prop slab, the temporary props were released and removed, and the road pavement constructed. At the instrumented section the mean level of the westbound carriageway was 12.7m AOD giving a final retained height of 6m.
4. FIELD INSTRUMENTATION

Instrumentation was installed to measure earth and water pressures near to the retaining wall, movements of both the ground and the wall, and the bending moments developed in the wall. A plan and section showing the instrumentation layout are given in Figs 8 and 9 respectively.

4.1 EARTH AND WATER PRESSURES

Spade shaped pressure cells (SC1 to SC10) were installed at the locations and depths shown in Figs 8 and 9 during June 1987, some three months before the start of wall installation in the area. Each cell was installed by boring a hole and pushing the instrument a further 0.5m into the intact ground beyond the bottom of the borehole. The spade cells recorded total lateral earth pressure and incorporated piezometers to measure porewater pressure. Where suitable, separate pneumatic piezometers (PP1 to PP5) were installed within 100mm long sand cells in the same borehole as a spade cell to provide additional data on porewater pressures. Boreholes were backfilled with bentonite to provide an impermeable seal above and below each piezometer position. Spade cells were installed on both the excavated and retained side of the wall; those on the excavated side incorporated a quick release mechanism on their pneumatic lines at just below formation level. This enabled the cells to be disconnected prior to bulk excavation and reconnected following its completion.

Additional spade cells (SC11 to SC13) were installed on the retained side at 2.3m from the front face during July 1988: access was not possible at an earlier date because of the intensity of the construction work in this area.

4.2 GROUND MOVEMENTS

To measure the surface movement of the ground during diaphragm wall installation a line of three stations (L1 to L3), at distances of 2m, 5m and 12m from the front face of the T-panel, was established. Each station consisted of a 0.5m square concrete block founded at 0.5m depth with a machined stainless steel insert to receive the levelling staff and the attachments for tape extensometer and electronic distance measurements (EDM) using a Geomensor. Lateral movements using the latter two techniques were measured from a datum pillar installed outside the construction zone. Vertical movement measurements obtained by precise levelling were referred to a temporary benchmark founded at depth and positioned remote from the influence of any construction works.
Fig. 6 Excavation below the temporary props

Fig. 7 Construction of the permanent prop slab
Front face of wall

New slip road

Inclinometer
L — Surface levelling station
SC — Spade cell incorporating piezometer
PP — Pneumatic piezometer
H — Horizontal plate gauges

Figure 8 Plan showing instrumentation

Portastore

Figure 9 Section showing instrumentation between chainages 1245 and 1265
The line of surface stations (L1 to L3) could only be used to measure the movements of the ground surface during wall installation as a slip road was due to be constructed on the retained ground prior to the start of excavation in front of the wall. A magnetic extensometer system with six plates (H1 to H6 in Fig 9) was therefore installed at about 2m depth across the line of the proposed slip road. Lateral movement of the plate magnets was then measured by reed switches permanently installed on a rod which could be adjusted by a micrometer from within a manhole located adjacent to the datum pillar. Thermocouples installed within the access tube enabled the readings to be corrected for the effects of thermal expansion of the rod.

After the surface movement stations (L1 to L3) were removed during slip road construction, settlements were monitored by precise levelling on studs (S1 to S6 in Fig 9) installed over the line of the horizontal plate gauges in the surface of the carriageway and footpath. Measurements of the lateral movements of the ground across the slip road and of the wall were then only possible using the horizontal plate system and the Geomensor.

4.3 WALL MOVEMENTS AND BENDING MOMENTS

Two inclinometer tubes (I1 and I2 in Fig 8) were installed in panel 83 to monitor the deflected shape of the wall. Inclinometer surveys were taken at regular intervals and before and after particular construction operations. As base fixity of the wall and hence the toe of the inclinometer tubes could not be assumed, the absolute lateral movement of the top of each tube was monitored using the Geomensor EDM.

Additional tubes I3 and I4 were installed close together in panel 82 with 9 electolevels in each tube to monitor tilt. From these two tubes the change in tilt at 0.75m intervals was determined over the wall height. The deflected shape of the wall panel was then determined by accumulating the deflections calculated from the change in tilt at each position.

The bending moments in panel 82 were determined using vibrating wire strain gauges. For this purpose, five pairs of gauges were clamped at 1m intervals of depth to the vertically running reinforcement bars above and below prop slab level. One gauge of each pair was positioned on the back and the other on the front of the reinforcement cage.

When the parapet of the wall had been completed additional sockets for the Geomensor target reflector were installed on the top face, one in the north and two in the south wall. This enabled the relative movement of the underpass walls to be determined.

5. DISCUSSION OF RESULTS

5.1 INSITU GROUND CONDITIONS PRIOR TO CONSTRUCTION

The total lateral stresses determined from the spade cells at various depths prior to any construction activities are shown in Fig 10. The measured values have been corrected by subtracting one half of the undrained shear strength (Fig 1) as recommended by Tedd and Charles (1983) to allow for the over-registration error expected with this type of push-in cell. Also plotted are the total lateral stresses from self-boring pressuremeter tests carried out as part of the site investigation for the scheme at a location about 20m to the west of the instrumented section. Agreement between the spade cells and pressuremeter tests was reasonably close and generally indicated an initial stress state in the clay corresponding to an average earth pressure coefficient (K) of between about 1.5 and 2 over the depth of the wall. The calculated K lines shown in Fig 10 have been based on the measured porewater pressure distribution.

Also shown in Fig 10 are the total lateral stresses calculated from soil capillary pressure measurements on 100mm diameter clay specimens carried out in the laboratory triaxial apparatus. The specimens were taken from high quality thin-walled tube samples and the testing procedure followed that reported by Burland and Maswowsie (1982). From the values of capillary pressure ($p_c$) the insitu effective horizontal stress ($\sigma_{eh}$) at sample depth was derived from the relation:

$$\sigma_{eh} = \frac{(p_c - A_s \cdot \sigma_v)}{(1 - A_s)} \quad \text{...... Skempton,1961.}$$

where $A_s$ is the pore pressure coefficient during sampling and $\sigma_v$ is the effective vertical stress. A value of 0.5 was assumed for the $A_s$ of the London clay as recommended by Burland and Maswowsie. The total lateral stresses, calculated by addition of the measured porewater pressures, tended to be lower than those measured more directly by the spade cells and pressuremeter (Fig 10), possibly because of a loss of suction during the 2 to 5 week period between sampling and testing.

The porewater pressure profile with depth measured by the piezometers incorporated in the spade cells and the separate pneumatic piezometers is given in Fig 11. This shows a linear increase with depth below the surface of the London clay which is slightly below that of a hydrostatic distribution and suggests that under-drainage may be taking place in this area.

5.2 INSTALLATION OF DIAPHRAGM WALL

5.2.1 Earth and water pressures

The corrected lateral stress and porewater pressure changes during excavation and concreting of the wall panels in the instrumented section are shown in Fig 12 for the spade cells located at 1.5m away from the front face of the wall (Fig 8). Spade cell SC8 was directly in
front of panel 83 and falls of 80 kN/m² in total lateral stress and 20 kN/m² in porewater pressure were observed during panel excavation. Concreting of this panel resulted in increases in porewater pressure and total lateral stress with further rises during construction of panel 82. Spade cell SC9 showed little response to the installation of panel 83 while SC10 at greater depth recorded an increase in both total lateral stress and porewater pressure. The installation of panel 82 adjacent to these cells resulted in reductions in total lateral stress of about 50 kN/m² on spade cell SC9 and by about 160 kN/m² on SC10 with smaller changes in porewater pressures.

Panel 81 was not installed until April 1988 which was some six months after the installation of the other panels in the instrumented area. Only small net reductions in total lateral stress of less than 10 kN/m² were recorded on spade cells SC8 to SC10 during this operation. Porewater pressure changes followed a similar pattern to those shown in Fig 12.

Spade cells at 1.5m behind the back counterforts of the T-panels (Fig 8) showed a similar pattern of behaviour during wall installation, however lateral stress changes were much smaller than those measured in front of the wall face. The magnitude of the effective lateral stress changes for cells in front of and behind the T-panels are summarised in Fig 13. The changes are compared with the theoretical K-lines calculated using the measured porewater pressures prior to the construction works. Although some stress relief occurred during wall installation, the lateral stresses in the ground at 1.5m away remained in excess of a K-value of unity.

Spade cell SC5 at the same depth as SC2 but a further 2.5m back into the retained ground showed the same effective stress change during wall installation, although SC6 at 6.5m behind SC2 showed a reduction of only 5 kN/m².
Figure 12 Pressure changes at 1.5 m away during diaphragm wall installation
5.2.2 Ground movements

The ground surface movements measured during diaphragm wall installation in the instrumented area are shown in Fig 14. Results were not available from surface station L3 as it was damaged by construction plant. Lateral movements of the remaining two stations L1 and L2 were measured by both tape extensometer and Geomensor EDM, and readings using both methods agreed to better than 1mm during the installation of wall panels throughout September 1987. Consistency between the two techniques was not as good during October since measurements could not necessarily be taken on the same day due to construction plant operating inside the instrumented area. Lateral movements and settlements of about 5mm were recorded during installation of panels 82 and 83 at station L1 located within the bay formed by these two panels. At 5m behind the front face of the wall (station L2 in Fig 9) a settlement of about 3mm and a surface lateral movement towards the wall of about 7mm were monitored. The latter value can be compared with the 4mm measured at H2 in a similar location but at 1.5m depth using the horizontal plate system as is shown in Fig 15. The plate gauges at 7m and 9m back from the wall face showed similar lateral movements of up to 4mm, although no significant movements were detected beyond a distance of 12m.

5.3 UNDERPASS CONSTRUCTION

5.3.1 Earth and water pressures

Figs 16 and 17 show the corrected lateral stress and porewater pressure changes observed from before the temporary props were installed until after their release following bulk excavation (stages 2 to 6 in Table 1). As indicated, the final retained ground level was at 18.8m AOD, i.e. 0.8m above the ground level which existed prior to construction.

In Fig 16 the results are given for spade cells and piezometers located in the retained ground at a distance of 2.3m from the front face of the wall. Little or no change in either lateral stress or porewater pressure was recorded during excavation for and installation of the temporary props. However a fall in porewater pressures of up to about 15kN/m² accompanied by a similar reduction in total lateral stress occurred during the bulk excavation of the ground in front of the wall; as would therefore be expected little change in effective stress was
Figure 14 Ground surface movements during wall installation
Figure 15 Lateral subsurface movement at 1.5 m depth during wall installation
Figure 16 Pressure changes in the retained ground during temporary propping and bulk excavation (stages 2 to 6)
Figure 17 Pressure changes at 1.5 m in front of the wall during temporary propping and bulk excavation (stages 2 to 6)
measured during this period. No measurements were obtained during prop installation and bulk excavation on the spade cells in front of the wall as these instruments were temporarily disconnected so as not to impede construction operations. On reconnection, the results in Fig 17 indicate that significant changes in both lateral stresses and porewater pressures had occurred during the excavation of the soil above the instruments. Porewater pressures on all three cells had fallen by up to about 60kN/m² and considerable redistribution of the total and effective lateral stresses had taken place.

On release of the temporary props, spade cells SC11 and SC12 at depths of 4m and 6m in the retained ground showed a decrease in total lateral stress, although a stress increase was measured on SC13 at 12m depth (Fig 16). These changes were accompanied by an increase in total lateral stress on the upper two spade cells (SC8 and SC9) immediately in front of the wall (Fig 17). The results during release of the temporary props, including measurements on the profile of spade cells in the retained ground at 5m behind the front face of the wall, are summarised in Fig 18. A similar decrease to that shown for spade cell SC2 was recorded on SC5 at the same depth but at 7.5m behind the front face of the wall. Further back at 11.5m behind the front face, cell SC6 showed a change of only half that on cells SC2 and SC5. Above a level between 6m and 8m AOD, cells on the retained side in both profiles showed small decreases in total lateral stress whilst cells in front of the wall showed small increases. The converse was true below this level for pressure cells SC10 and SC13 located close to and on opposite sides of the wall. This pattern of stress change is consistent with an outward rotation of the wall occurring about a point located at 2 to 3m above the toe, with compression of the permanent prop slab taking place as additional load is transferred from the temporary props.

A summary of the porewater pressure distributions with depth at various distances from the wall is given in Fig 19 with the measured values after release of the temporary props in August 1989 being compared with the initial values prior to construction. On the retained side, the results indicated an average reduction in porewater pressure of 12kN/m² at 5m behind the front face of the wall and a slightly larger decrease of about 18kN/m² at only 2.3m behind the front face. As would be anticipated the porewater pressures in front of the wall were much reduced after excavation and slightly below those that would be calculated for a hydrostatic distribution from a phreatic surface at 11.5m AOD. The values measured after opening the road to traffic are discussed in Section 5.4.1.

### 5.3.2 Wall movements

Wall movements were monitored from a datum established in mid February 1989 after completion of the diaphragm wall and its associated capping beam and parapet unit, but prior to any excavation in front of the wall. The lateral movements of the top and toe of the wall from this time and up to and including stage 6 of the construction are shown in Fig 20. These movements were determined from the Geomensor measurements to the top of the wall and the deflection profiles established from the inclinometer and electrolevel data.

![Figure 18 Stresses measured before and after temporary prop release](image-url)
While the temporary props were in place the lateral movements were found to vary with air temperature (Fig 20), probably because of thermal expansion and contraction of the steel props. In this context the Contractor’s records of load in the temporary prop for panel 82 showed an average change of about 70 kN/°C during May 1989. Examples of the wall movement profiles at various stages during the construction are given in Fig 21 together with the corresponding prop loads measured in panel 82. The initial excavation, installation of the temporary props and bulk excavation produced small outward rotations of the tops of the wall panels relative to their bases (Fig 21a). Shortly after completion of excavation the direction of this rotation was reversed with larger forward movements developing near the toe of the wall. During the next four months as air temperatures rose (Fig 20) small movements of the top of the wall towards the retained ground were measured probably due to expansion of the temporary props. The forward movement of the toe of the wall and the load in the temporary props were near their maximum values in July 1989 when the permanent prop slab was cast. The movement profiles at this time are given in Fig 21b based on the average measured forward movement of the toe of the wall of 3mm (Fig 20).

Release of the temporary props resulted in an immediate reversal in the direction of wall rotation with the top of the wall moving towards the excavation and the toe moving back towards the retained ground (Fig 20). The changes in the movement profiles from before release of the temporary props (Fig 21b) until after (Fig 21c) are generally consistent with the measured changes in total lateral stress on the spade cells close to the wall, which were described in the preceding section.

The differences between the electrolevel and inclinometer results in Fig 21c may be because the former were in diaphragm wall panel 82 while the latter were both in adjoining panel 83 (Fig 8). It is possible that there were small differences in the deflections of these two panels, particularly as remedial work had been undertaken behind panel 82 with additional concrete being placed to about 4m depth below cut-off level.

5.3.3 Ground movements

The development of ground movements near the surface on the retained side of the wall calculated from the datum established in February 1989 is given in Fig 22. Lateral movements were measured using the horizontal plate gauges at about 2m depth and calculated assuming no movement on the gauge located 15m from the wall as no readings at the manhole location were available at this stage. Vertical movements of road surface studs were monitored by precise levelling with respect to the temporary benchmark founded at depth and remote from the influence of construction. The levelling station at 0.8m from the front face of the wall was installed at the location of inclinometer tube H1 in the ground immediately above the wall panel and its movement was therefore expected to follow that of the wall.

After excavation to 6.3m depth below the temporary props a small movement towards the excavation of a few millimetres was recorded on the plate gauge H1 fixed to the back of the wall counterfort and this is consistent with the results given in Fig 21a. Ground movements on gauge H2 at 1.5m behind the counterfort showed an outward lateral movement of only 0.5mm.

Figure 19 Porewater pressure distributions after release of temporary props
Figure 20 Lateral movements during temporary propping and bulk excavation (stages 3 to 6)
Figure 21 Development of lateral wall movement
Horizontal plate gauges at 2 m depth

Precise levelling on surface studs

Figure 22 Development of surface movements
Thermal expansion of the temporary props was at a maximum after construction of the permanent prop slab causing the top of the wall to be pushed into the retained ground. Plate gauge H1 on the wall counterfort and H2 in the ground both showed a movement of nearly 2mm at this stage and this again conformed with the measured pattern of wall movement (Fig 21b). Further back in the retained ground the lateral movement fell to a value of 0.8mm at 7m behind the front face of the wall.

On release of the temporary props, a lateral movement towards the excavation again occurred. At the back of the counterfort and in the ground close to it, a movement of about 2mm was then measured. The magnitude of this movement reduced to less than 0.5mm at a distance of about 8m behind the front face of the wall as is shown in Fig 22.

A heave of about 1mm was recorded on the station immediately over the front face of the wall panel between stages 4 and 5 of the construction. Elsewhere the changes in vertical movement measured at the ground surface were probably dominated by the effects of trafficking of the slip road on the retained side of the wall. The movement results taken at the main stages of construction together with the extent of the slip road are indicated in Fig 22. The magnitude of the settlements tended to increase with time with a maximum value of about 9mm being recorded at the centre of the slip road during construction of the underpass. Further increases in settlement across the slip road were recorded in the longer term and these are discussed in section 5.4.

5.3.4 Wall bending moments

The bending moments in the wall determined from measurements on the vibrating wire strain gauges are shown in Fig 23. The five pairs of vibrating wire strain gauges installed in panel 82 only functioned up to stage 5 of the construction and thereafter readings were only available on the uppermost pair of gauges.

Moments were determined from the strains measured on each pair of gauges based on a calculated flexural rigidity (EI) for each T-panel of $1.53 \times 10^5$ MN.m², assuming that the concrete would remain uncracked at the small strain levels involved.

To provide a basis of comparison with the measurements, Fig 23 also shows the bending moments in the upper part of the wall estimated assuming a linear distribution of earth pressure corresponding to an earth pressure coefficient of unity in the soil on the retained side together with the temporary prop loads measured by the Contractor. After bulk excavation a maximum anticlockwise bending moment of 315 kNm/m is calculated at 14m AOD on this basis (Fig 23a). The bending moments determined from the strain gauge measurements are in reasonable accord with the calculated values except for the top pair of gauges which indicate higher bending moments than would be obtained with no soil or water pressure acting on the wall. The results from this uppermost pair of gauges is therefore considered to be suspect.

![Figure 23 Bending moments over the upper part of the wall](image-url)
At the end of stage 5 the temporary prop load rose to 466 kN/m with the estimated anti-clockwise bending moment at 14m AOD rising accordingly to 1180 kNm/m and with a value of about 1480 kNm/m at the position of the permanent prop. The bending moments from the strain measurements are compared with these values in Fig 23b, but indicate little correlation.

On release of the temporary prop the sign of the bending moment was expected to change to give clockwise moments above the prop slab level and a value of about 890 kNm/m at the prop position assuming an earth pressure coefficient of unity and zero wall friction. No such change was recorded on the uppermost pair of vibrating wire gauges which again suggests that this pair of gauges is in error.

5.4 POST-CONSTRUCTION

5.4.1 Earth and water pressures

The variations in corrected total and effective lateral stress and porewater pressure measured from September 1989 until about a year after the opening of the underpass to traffic in February 1990 are shown in Figs 24 and 25. Fig 24 shows the measurements obtained from the cells located below the new carriageway at 1.5m in front of the wall, whilst Fig 25 shows the results from instruments located in the retained ground at 2.3m behind the front face of the wall. Changes further back in the retained ground were of reduced magnitudes from those in Fig 25 and virtually insignificant.

Fig 24a shows that the total and effective lateral stresses measured on cells below the carriageway were at a minimum in early summer 1990 with the stresses rising again as winter approached. The minima in the porewater pressures were slightly out of phase with the lateral stress changes and occurred a few months later. In general the seasonal fluctuations measured on the retained side were small (Fig 25), however slight increases in both lateral stress and porewater pressure were apparent in the middle to late summer. This pattern of seasonal fluctuation is similar to that reported for a bored pile retaining wall propped at carriageway level at Malden Way Underpass (Garder and Symons, 1989). The results are broadly compatible with an increase in loading caused by thermal expansion of the prop slab in the summer causing a build-up of lateral stress and porewater pressure on the retained side and a corresponding reduction in ground and water pressures on the carriageway side.

The distributions of total lateral stress with depth determined from the spade cells in March 1991, just over a year after opening the underpass to traffic, are shown in Fig 26. Also shown are the lines of constant earth pressure coefficient (K) calculated using the measured porewater pressure distribution at that time. In the retained ground at 5m behind the front face of the wall the three uppermost spade cells indicated that the overall effect of the construction had caused some lateral stress relief with the K-value reducing to just below unity from the higher values of 1.5 to 2 which existed before construction started. The deepest cell at 8m AOD was not so affected and still indicated an earth pressure coefficient of nearly 2. Similar results were obtained from the cells installed at 2.3m behind the wall face in the area between the panel counterforts with the two shallowest spade cells showing the most reduction. The measured total lateral stresses on the two spade cells in front of the wall which continued to function are compared in Fig 26 with the passive pressures calculated using both the average (c' = 13 kN/m², φ' = 27°) and lower bound (c' = 0, φ' = 24°) strength parameters from triaxial compression tests (Section 2). The values of the passive earth pressure coefficient K, have been determined from Caquot and Kerisel (1948) assuming a wall friction angle of φ' /2 and a wall cohesion of zero in accordance with the recommendations of Padfield and Mair (1984). The measured cell pressure at 8m AOD lies close to the lower bound passive pressure line while the lateral stress recorded on the cell at 6m AOD is substantially below the calculated values.

The porewater pressure distributions in March 1991 were not very different from those recorded after the release of the temporary props (Fig 19) and are given in Fig 27. Also shown are the hydrostatic and linear seepage relations determined from the measured water table levels. The ground water on the retained side and 5m behind the wall face was at a reasonably steady level at about a metre below that which existed prior to any construction. A reduction of about 2m was measured close to the wall in the bay between the counterforts and the distribution of porewater pressure with depth was between the hydrostatic and linear seepage relations. On the carriageway side the ground water level had stabilised at about 1.4m below the surface and the porewater pressure distribution was below that calculated from both hydrostatic and linear seepage assumptions.

5.4.2 Wall and ground movements

The wall movement profile 9 months after opening the underpass to traffic is given in Fig 21d calculated from the inclinometer and electrolevel data assuming base fixity of the wall. The results indicate a lateral movement at the top of the wall towards the excavation of between 5 and 6.5mm from the datum established shortly after diaphragm wall installation. As with the previous results during construction of the underpass, the deflections measured using the electrolevels in wall panel 82 were slightly less than those recorded on inclinometer tubes 11 and 12 in panel 83.

Direct readings on the tops of the inclinometer tubes using the Geomensor were not possible at this time as the sight lines were obstructed by the construction of a brick wall. However measurements were made to the parapet tops of the north and south wall and these indicated a closure of 16mm. Assuming symmetry about the centre of the underpass, this implied an outward movement of the parapet top of about 8mm. If account is taken of the extra height of the parapets, the absolute lateral movement of the tops of the inclinometer tubes at
Figure 24 Pressure changes at 1.5 m in front of the wall following completion of construction.
Figure 25 Pressure changes in the retained ground following completion of construction

(a) Lateral stresses

(b) Porewater pressures

Depth of spade cells and piezometers
SC11: 4 m
SC12: 6 m
SC13: 12 m
PP6: 7.8 m
Figure 26 Stresses measured at 13 months after opening of the underpass

Figure 27 Porewater pressure distributions at 13 months after opening of the underpass
this stage is then about 7mm. Any outward movement of the toe of the wall was therefore likely to be less than about 1.5mm.

Measurements of horizontal and vertical ground movements on the plate gauges and road studs respectively taken 9 months after the opening of the underpass are given in Fig 22. Small additional lateral movements had gradually developed since the release of the temporary props with gauge H3 at 7m behind the wall face showing the largest increase of 2mm to give a total movement of 2.5mm. Over the same period the peak settlement at the centre of the slip road increased to 11mm.

6. SUMMARY AND CONCLUSIONS

The behaviour of a section of diaphragm wall founded in stiff clay and propped at carriageway level has been monitored during its construction as part of the Underpass for the A406/A10 Junction Improvement Scheme. The following conclusions were reached:

1. During installation of the diaphragm wall reductions in total lateral stress ranging from 50 to 160kN/m² were recorded by the spade cells at 1.5m away from the front face of the wall. Porewater pressures fell during excavation for each panel but recovered almost immediately after the concrete was placed.

A maximum lateral movement towards the excavation of 7mm was measured at 5m behind the wall face (i.e. 1.5m behind the back of the counterfort) and this was accompanied by a settlement of 3mm. No measurable movements were detectable beyond a distance of 12m from the front face of the wall during its installation.

2. The initial excavation, installation of temporary props and bulk excavation produced small outward rotations of the top of the wall panels relative to their base. Shortly after completion of excavation to full depth the direction of this rotation reversed with larger forward movements occurring at the toe of the wall. During the following four month period small movements of the top of the wall towards the retained ground took place probably as a result of thermal expansion of the temporary props. The loads in the temporary props were near a maximum when the permanent prop slab was cast and by this stage the forward movement of the toe of the wall had increased to about 3mm.

3. Release of the temporary props resulted in an immediate reversal in the direction of wall rotation with the toe moving towards the retained ground and the top of the wall moving towards the excavation to give a nett forward movement of up to about 4mm relative to the start of construction. Small changes in total lateral stress consistent with this pattern of wall movement were measured by the spade cells.

4. During the first year after construction, a further outward lateral movement of the top of the wall of about 2mm was measured. Small seasonal changes in both lateral stress and porewater pressure behind and in front of the wall were also observed which were consistent with the thermal expansion and contraction of the permanent prop slab beneath the carriageway.

5. In the retained ground close to the wall the overall effect of construction caused some stress relief, with the earth pressure coefficient reducing to just below unity from the higher values of between 1.5 to 2 which existed before construction started. In front of the wall the measured lateral stresses were below current design recommendations for the passive pressure based on soil strength parameters determined from triaxial compresion tests. A lowering of the water table of up to 2m was measured close to the retained side of the wall in the bay between the counterforts.

7. ACKNOWLEDGEMENTS

The work described in this report forms part of the research programme of the Ground Engineering Division (Division Head : Dr. J. Temporal) of the Structures Group of TRRL. In addition to the authors, the research team consisted of Mr G H Alderman, Mr I G Carswell, Mr P Darley, Mr A J Gent and Mr P E Johnson.

Thanks are due to the London Regional Office of DTp for permission to carry out this study. The co-operation of Bulien and Partners Consulting Engineers (Senior Partner: Mr D Dennington), in particular their site staff Mr A W Cook, Mr G Gallagher and Mr R K Postill, and the Main Contractor (Alfred McAlpine Ltd) is gratefully acknowledged.

8. REFERENCES


MORE INFORMATION FROM TRRL

TRRL has published the following other reports on this area of research:

1. RR273 Long term performance of a propped retaining wall embedded in stiff clay, D R Carder and I F Symons, Code B
2. RR288 Behaviour of an embedded retaining wall on the A6 Chapel-en-le-Frith bypass, P Darley, I F Symons and D R Carder, Code C
3. RR313 Long term performance of an anchored diaphragm wall embedded in stiff clay, I G Carswell, D R Carder and I F Symons, Code B

If you would like copies, photocopy and fill in the slip below. There is a 20% discount if you take all the reports listed above. Prices include postage and are correct at the time of publication. Please note that reports produced by TRRL Overseas Unit are available free of charge. Enquiries to TRRL Library Services, 0344 770203, or Overseas Unit, 0344 770187.

To: TRRL Library Services, Old Wokingham Road, Crowthorne Berks RG11 6AU.
Please send me ........ copies of the following TRRL reports (state report Nos)

Name ..............................................................
Address ............................................................
Postcode ...........................................................
Telephone ..........................................................

PAYMENT:
I enclose a cheque for £ ............... payable to TRRL Library Sales/Please debit my Deposit Account

USE OUR EXPERTISE

TRRL’s researchers and Laboratory facilities are available at competitive rates.

Our 300 scientists and engineers include many world-class experts on highways design and maintenance, transport structures, traffic systems, vehicle safety and road safety.

TRRL facilities include a 2.3 mile test track, large structures test halls for static and fatigue testing, a dynamic pavement test facility, advanced computer systems and a large specialist library with on-line access to worldwide information.

If you are planning a project where we may be able to help, contact TRRL (TTU) at Crowthorne, Berkshire RG11 6AU, telephone 0344 770004, fax 0344 770356.