BEHAVIOUR DURING CONSTRUCTION OF A PROPPED CONTIGUOUS BORED PILE WALL IN STIFF CLAY AT RAYLEIGH WEIR

by P Darley, D R Carder and G H Alderman

Prepared for: Project Record: E468A/BG Behaviour of Bored Pile Retaining Structures during Construction
Customer: Bridges Engineering Division, DOT

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EXECUTIVE SUMMARY

For urban roads, environmental considerations and pressures to minimise landtake are resulting in increased construction below ground level in retained cuttings and cut and cover tunnels. Such structures are often formed from embedded retaining walls constructed using bored piles or diaphragm walling techniques. Where embedded retaining walls are founded in stiff over-consolidated clays it is difficult to reliably predict the lateral stresses acting on the wall and the resulting movements of the wall and surrounding ground.

This report gives the results of a study of the behaviour of a contiguous bored pile retaining wall during and immediately after its construction as part of an underpass at Rayleigh Weir in Essex. The construction sequence involved pile installation followed by partial excavation in front of the wall, temporary props were then installed and the bulk excavation completed. The permanent prop slab below the final carriageway level was then constructed and the temporary props removed.

During installation of the piles measurements of total lateral stresses, porewater pressures, and ground movements were made. Instrumentation was also installed in the wall and both the temporary props and permanent prop slab to monitor wall movements and bending moments together with loads in props during and after construction.

The initial horizontal stresses in the ground were consistent with a K-value of 2.1, where K is the ratio of horizontal to effective vertical stress. As a result of pile installation reductions in total horizontal stress were recorded with the largest reductions occurring at depth. Twelve days following piling across the instrumented section a mean K-value of 1.9 was measured. Porewater pressures during installation fell by up to 150 kPa but subsequently returned to their pre-construction values. Ground movements measured 11.5 metres from the wall showed 5 mm of lateral movement and 3 mm of settlement.

During underpass construction total lateral stresses fell to a K-value of just over 1. When the reductions due to pile installation are included an approximately uniform reduction of about 100 kPa from the initial in-situ stresses was observed over the depth of the wall. Over this period the top of the wall cantilevered towards the underpass by about 25 mm and a vertical heave of 9 mm was recorded. The surface of the retained ground at 11.5 m from the wall moved towards the wall by 10 mm and settlement of 4 mm was observed.

Loads in the temporary props varied significantly with changing temperature and a seasonal variation in permanent prop loads consistent with thermal expansion of the slab was recorded. Bending moments developed in the wall were much smaller than those calculated from measured earth pressures at 1.5 m from the wall face and prop loads suggesting that further stress relief occurred close to the wall.
BEHAVIOUR DURING CONSTRUCTION OF A PROPPED CONTIGUOUS BORED PILE WALL IN STIFF CLAY AT RAYLEIGH WEIR

ABSTRACT

Field instrumentation has been installed to investigate the performance of a contiguous bored pile wall propped at carriageway level founded in over-consolidated clay during and immediately after its construction to form part of an underpass at Rayleigh Weir, Essex. The construction sequence involved installation of bored piles followed by partial excavation, installation of temporary props, completion of excavation, construction of the permanent prop slab and removal of the temporary props.

Measurements of ground movements, total lateral stresses and porewater pressures were made both during pile installation and underpass construction. The development of movements and bending moments in the wall, and loads in both the temporary props and permanent prop slab were monitored.

Frith (Darley, Symons and Carder, 1990) and the A406/A10 junction (Carder, Ryley and Symons, 1991) have been reported previously.

2. SITE LOCATION

The Rayleigh Weir underpass is located at the junction of the A127 London-Southend road and the A129 about 6 miles to the west of Southend-on-Sea. The instrumented section of wall forms part of the north eastern retaining wall between chainage 1740 and 1760 (Figure 1). Due to failure of some instruments in the north eastern wall and in the retained ground behind it, additional instruments were installed on the south side of the underpass between the same chainages.

3. INSTRUMENTATION

Instrumentation was installed in January 1990, to monitor surface and sub-surface ground movements in the retained ground, and earth and porewater pressures on both sides of the wall. During late April and early May 1990 whilst pile installation was being carried out, instruments were installed to measure movement of, and strain and tilt within, the piles. A pile in the south wall (S210) was instrumented with strain gauges in October '90, and additional earth pressure cells were installed behind the south eastern wall in February 1991 to replace malfunctioning instrumentation in the north eastern section. Strain gauges were installed in August 1991 in three of the thrust blocks to determine the load being transmitted to the permanent prop slab constructed at carriageway level, a vibrating wire piezometer (VP5) was installed beneath the carriageway at the same time.

Figures 2 and 3 show a plan and section giving the types of instrument and their locations. The sequence of construction of the piles in the instrumented area is given in Table 1, together with details of the additional instrumentation in the south eastern wall.

3.1 MEASUREMENT OF EARTH AND WATER PRESSURES

Push-in spade shaped pressure cells (SC), each fitted with an integral pneumatic piezometer, were installed at 2.25 m both behind and in front of the line of the proposed retaining wall at the locations shown in Figures 2 and 3. Each cell was lowered to the bottom of a borehole
Fig. 1 Plan of the site

Fig. 2 Layout of instrumentation (not to scale)
and then pushed a further 0.5 m into the undisturbed soil using a lorry mounted drilling rig. During this operation care was taken to maintain the orientation of the active face of the cell so as to measure total lateral stress acting towards the retaining wall. The boreholes were then backfilled with bentonite pellets to form an impermeable seal above the pressure cell.

In four of the spade cell boreholes (Figures 2 and 3) high air entry vibrating wire piezometers (VP) were also installed to give further data on porewater pressures on the retained side of the wall. Each piezometer was installed in a nominal 100 mm long sand cell with a bentonite seal above and below.

Following failure of spade cell SC1, four additional cells (SC13 - 16) were installed behind the south eastern retaining wall at depths between 3 m and 4.7 m.

3.2 MEASUREMENT OF SURFACE MOVEMENT

Horizontal movements of the ground surface at I1, X4, and M5-M8 were measured using a proprietary tensioned tape extensometer system and a high precision electronic distance meter (Geomensor CR 204). The measurements were made relative to a reinforced concrete pillar, PP, sited 30.5 m from the line of the wall, close to the Weir public house (Figure 1). In addition Geomensor readings were also taken from a second pillar, PA, sited some 59.5 m from the wall in the grounds of the ambulance station on the opposite side of the underpass to pillar PP (Figure 1). Geomensor readings of the distance between the two pillars were used to check for any movement of the reference pillars: no movement was detected during pile construction but movements of pillar PA occurred subsequently.

Each surface station consisted of a concrete block founded about 0.5 m below ground level and fitted with a precision machined stainless steel socket. These sockets were designed to accept stainless steel pillars to which the tensioned tape and Geomensor reflectors were attached. All tape readings were corrected for thermal effects on the steel tape. Vertical movements of the surface stations were monitored using precise levelling relative to temporary benchmarks cast into the bases of pillars (PP, PA) and located in the ambulance station car park.
### TABLE 1

**Pile installation sequence and instrumentation**

<table>
<thead>
<tr>
<th>Pile Number</th>
<th>143</th>
<th>144</th>
<th>147</th>
<th>148</th>
<th>149</th>
<th>150</th>
<th>151</th>
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<td></td>
</tr>
<tr>
<td>N 151</td>
<td></td>
<td>I3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N 152</td>
<td>X1</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>N 153</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N 154</td>
<td></td>
<td></td>
<td>EL X2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N 155</td>
<td></td>
<td>I2</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>N 156</td>
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<td></td>
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<tr>
<td>N 159</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>N 160</td>
<td></td>
<td>I5</td>
<td></td>
<td></td>
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<td>N 161</td>
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</tr>
<tr>
<td>N 162</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S 210</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>VW</td>
</tr>
</tbody>
</table>

* Pile constructed, I Inclinometer & Geomensor, EL Electrolevels, VW Vibrating wire strain gauges, X Geomensor

Day 1 is 5th December 1989, Days 145 & 146 Weekend

Surface movements measured using the Geomensor were determined to better than +/- 1 mm. This accuracy was not as high as would normally be expected from the Geomensor possibly because the light beam passed through areas of high air temperature gradients caused by the blacktop surfacing of the car park. The potential accuracy of the tensioned tape was similar to the Geomensor but was not achieved at this site for the reasons discussed in Section 6.1.3.

### 3.3 MEASUREMENT OF SUB-SURFACE MOVEMENT

A proprietary sleeve jointed aluminium inclinometer tube was installed in a 26 m deep borehole at location II, 11.5 m from the centreline of the wall. Horizontal deflections of this tube in a direction normal to the line of the wall were then calculated from measurements using a uniaxial inclinometer probe. Tests on the reproducibility of the measurements using two separate inclinometer probes showed that lateral movements of the top of the tube relative to the base could be determined to better than +/- 2 mm.

As the inclinometer tube was neither founded in bedrock nor founded at a significant depth below the toe level of the bored piles, fixity of the base of the tube could not be guaranteed. The top of the inclinometer tube was therefore incorporated into the same concrete block as a surface movement station, so that any movements of the top of the tube could be measured by the tape extensometer and the Geomensor systems, and hence any movement of the base of the tube determined.

Vertical sub-surface movements were determined using four magnetic settlement rings installed in a borehole at locations M5-M8. The positions of the rings were determined using a single reed switch probe attached to a fibreglass measuring tape that was lowered down a 24 mm diameter plastic access tube. Any change in the level of the top of the access tube was determined by precise levelling with respect to the temporary benchmark, PP.

### 3.4 PILE INSTRUMENTATION

Aluminium inclinometer tubes were installed in Pile N 155 and N 160 and 32 electrolevels in pile N 154 to measure the deflected shape and bending of the piles. Geomensor targets were installed in these three piles to
enable absolute lateral movements to be established without having to assume base fixity of the piles. Additional targets were also located in piles N 151 and 152 and in the south eastern wall to measure movement of the top of the wall and any changes in underpass span. Vibrating wire strain gauges were installed in pile N 152 to measure bending of the pile but after placement of the concrete too few survived to provide useful information and a further 16 gauges were therefore installed in pairs at depths between 3.5 m and 11.5 m during construction of a similar pile (S210) in the south eastern wall.

3.5 TEMPORARY AND PERMANENT PROP INSTRUMENTATION

Three of the temporary steel props were instrumented with strain gauges and thermocouples in order to measure loads in the props and to investigate changes in prop load with temperature. One prop was fitted with six weldable gauges, one with six vibrating wire gauges, and one with three of each type. A thermocouple was installed close to each gauge.

Vibrating wire strain gauges were installed in three of the permanent reinforced concrete thrust blocks to measure the horizontal loads being transmitted to the permanent prop slab by the wall.

4. SOIL PROPERTIES

The Geological Survey map indicates that the soil consists of Made Ground and Head Deposits overlying Claygate Beds and London Clay. Site investigations were carried out in 1971 and 1984 on behalf of the Department of Transport in preparation for the construction works. In addition soil sampling was carried out by TRL during instrumentation of the ground near the line of the wall in January 1990 prior to any construction works. A summary of the borehole records obtained during these investigations is shown in Figure 4.

Figure 5 shows the plasticity and moisture content data from undisturbed samples taken by TRL. Undrained shear strengths determined from triaxial compression tests carried out on 100 mm diameter specimens obtained from thin walled tube samples are shown in Figure 6. Also shown are the values determined from Marchetti flat bladed dilatometer tests carried out to a depth of 15 m at approximately 2 m from the proposed line of the wall. These undrained strengths were calculated using the empirical correlations developed by Marchetti (1980). Reasonable agreement was obtained between the triaxial results and those from the dilatometer.

A series of consolidated undrained triaxial tests was carried out on 38 mm diameter specimens taken from

<table>
<thead>
<tr>
<th>TRL BOREHOLES</th>
<th>SITE INVESTIGATION</th>
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</thead>
<tbody>
<tr>
<td>Made ground</td>
<td>Made ground</td>
</tr>
<tr>
<td>Grey/brown laminated silty clay with lenses of silt</td>
<td>Stiff brown/grey mottled silty clay with rust &amp; yellow fine sand &amp; silt (CLAYGATE BEDS)</td>
</tr>
<tr>
<td>Stiff grey silty clay</td>
<td>Stiff dark grey laminated silty clay with occasional silt &amp; fine sand becoming very stiff below about 12 m and becoming slightly sandy below about 16 m (LONDON CLAY)</td>
</tr>
<tr>
<td>Very silty grey clay</td>
<td></td>
</tr>
<tr>
<td>Stiff to very stiff grey silty clay with occasional silt lenses</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 4 Soil profile
thin walled tube samples of the London Clay between depths of 5 m and 26m. The test results are summarised in Figure 7 together with a best fit and lower bound lines corresponding to values of effective cohesion ($c'$) and effective angle of internal shearing resistance ($\phi'$) of $c'$= 8 kPa , $\phi'$= 26° and $c'$= 0 , $\phi'$=24° respectively in the mean effective stress range of up to 550 kPa.

4.1 IN SITU GROUND CONDITIONS

Figure 8 shows the total lateral stress measurements obtained from the spade cells together with the values determined from the Marchetti dilatometer tests and from measurements of initial suction on 100 mm diameter triaxial specimens from undisturbed samples of the clay. The spade cell results have been reduced by 0.5 $c'$ to correct for overread as suggested by Tedd and Charles (1983). The lateral stresses from the dilatometer have been calculated employing the empirical relations of Marchetti (1980) and the measured porewater pressures. The distribution of porewater pressure with depth was approximately hydrostatic from the surface and these results are discussed later in section 6.1.2. The lateral stresses obtained from the initial suction ($\sigma^*_{\text{h}}$) measurements were calculated using the formula:

$$\sigma^*_{\text{h}} = (\rho_{\text{w}} - \sigma^*_{\text{w}} \times A_\rho) + (1 - A_\rho)$$

where $A_\rho$ is the pore pressure coefficient during sampling for which a value of 0.5 has been assumed as suggested by Burland and Maswoswe (1982) for an over consolidated clay. $\sigma^*_{\text{h}}$ and $\sigma^*_{\text{v}}$ are the effective horizontal and vertical stresses at the sampling depth.

The total lateral stress results in Figure 8 show reasonable agreement between the three measurement techniques. In general the dilatometer stresses are slightly lower than those of the spade cells and this can be accounted for because the empirical relations used in interpreting the dilatometer data may under-estimate the over-consolidation ratio and hence the lateral stresses in stiff clays (Powell and Uglow, 1988). On the basis of the spade cell results and porewater pressure measurements the initial effective earth pressure coefficient $K$ in the London Clay was found to lie between 1.3 and 2.3 with a mean value of 2.1, no trend with depth being apparent.

5. CONSTRUCTION

The underpass was designed by Ove Arup and Partners who also supervised construction for the Eastern Regional Office of the Department of Transport. The main contractor was Christiani and Nielsen who subcontracted the piling to Simplex Piling Ltd. Bulk earthmoving was carried out by Cosgrove and Son Ltd.
Fig. 6 Undrained shear strength profile

Fig. 7 Strength parameters of the London Clay

Best fit $c' = 8\text{kPa}$, $\phi' = 26^\circ$

Lower bound $c' = 0$, $\phi' = 24^\circ$
5.1 BORED PILE WALL INSTALLATION

The alignment of the wall was set out and ladder frames anchored to the ground over the pile locations. Steel guide tubes were then driven approximately 3 to 3.5 m into the ground. A crane mounted rig (Figure 9) was used to auger to the required depth for each pile prior to lowering the reinforcing cage into position (Figure 10). Concrete was then poured into the hole and the guide tube extracted. The piles were subsequently trimmed and a reinforced concrete capping beam cast along the tops of the piles (Figure 11).

The contiguous bored piles in the instrumented area were 1500 mm in diameter and spaced at 1700 mm centres. The toe level of the piles was at 35.5 m A.O.D. corresponding to pile depths of approximately 24 m. The piles were constructed alternately and a rate of construction of 2 to 3 piles per day was achieved through the instrumented area (Table 1).

5.2 UNDERPASS CONSTRUCTION

Bulk excavation of the material between the retaining walls was carried out in two stages using an hydraulic excavator and tipper lorries. During stage 1 the soil was removed to a depth of 3 m above the final excavation level some time before the temporary steel props were installed. The temporary props used during excavation each comprised two parallel universal beams, nominally 914 mm by 305 mm joined at regular intervals by 12.7 mm thick plates to form a box structure. Each prop was further reinforced by 19 mm thick webs at 3 m spacings along its 22 m length. The props were fitted with cradles at each end to enable them to rest on, and be jacked against, the capping beams using hydraulic jacks at one end of each prop. The props were spaced at about 7.2 m intervals along the wall and each prop was prestressed to a nominal load of 300 kN. Following placement of the temporary props the remaining soil was then removed down to final excavation level (stage 2). The sequence of excavation and temporary prop placement is given in Table 2.
Fig. 9 Augering pile hole

Fig. 10 Placing reinforcing cage
The permanent prop slab comprised reinforced concrete walings cast against, but not tied to, each pile wall. Reinforced concrete thrust blocks 600 mm thick and 2.2 m long were constructed, at 5.1 m centres, between the waling beam and the 500 mm thick, unreinforced concrete slab on which, following removal of the temporary props, the road was constructed. Figure 12 shows the bulk excavation, the temporary props, and the permanent prop slab. PTFE sliding bearings were installed between the thrust blocks and the waling (Figure 13) to allow the clay beneath the slab to heave. The completed underpass was opened to traffic in December 1992.

6. OBSERVATIONS

6.1 PILE INSTALLATION

6.1.1 Earth pressures

Detailed plots of uncorrected total lateral stress measured by the spade cells located closest to the wall are given in Figures 14 and 15. The graphs show that the total lateral stress measured by each spade cell fell rapidly by between 40 and 350 kPa during construction of the three piles closest to the cell, although a partial recovery to a value lower than the initial lateral stress then occurred.

Profiles of corrected total lateral stress with depth are shown in Figure 16 before pile construction (Day 127), and about 12 days after completion of construction of the piles across the instrumented section (Day 162). In general the net reduction in total lateral stress at 2.25 m from the centreline of the wall over this period increased with depth as shown in Figure 16. Calculations using the corrected total lateral stresses and porewater pressures measured on individual spade cells gave a mean value for the in situ effective earth pressure coefficient (K) of 2.1 before construction of the piles and a value of 1.9 twelve days following piling across the instrumented section.

Figure 17 shows the variation in uncorrected total lateral stress at about 6 m below ground level measured by spade cells located at 2.25, 5, and 8 m from the line of the retaining wall. When the three piles closest to the spade cells were being installed, the results indicated that a larger reduction in lateral stress occurred at 2.25 m.
<table>
<thead>
<tr>
<th>Date</th>
<th>Excavation Between Pile Numbers</th>
<th>Depth excavated (Metres)</th>
<th>Details of excavation and prop placement &amp; loading</th>
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<td>18 March 91 (500)</td>
<td>158 to 163</td>
<td>2.5 Metres</td>
<td>Full width of underpass excavated</td>
</tr>
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<td>20 June 91 (563)</td>
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<td>5 Metres</td>
<td>Full width of underpass excavated</td>
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<td></td>
<td>160 Eastwards</td>
<td>7 Metres</td>
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<td>147 to 154</td>
<td>3.5 Metres</td>
<td>Southern third of width excavated</td>
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<td>29 July 91 (602)</td>
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<td>4.5 Metres</td>
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<td>31 July 91 (604)</td>
<td>147 to 160</td>
<td>5 Metres</td>
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<td>6 August 91 (610)</td>
<td>138 to 143</td>
<td>8.7 Metres</td>
<td>Full width excavated</td>
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<td>143 to 146</td>
<td>&quot;</td>
<td>Prop at pile 142 under load</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>South side excavated</td>
</tr>
<tr>
<td>7 August 91 (611)</td>
<td>143 to 149</td>
<td>8.7 Metres</td>
<td>Full width excavated</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Props at 142 and 145 under load</td>
</tr>
<tr>
<td>8 August 91 (612)</td>
<td>149 to 152</td>
<td>8.6 Metres</td>
<td>Full width excavated</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Props at 142,145,150 under load</td>
</tr>
<tr>
<td>9 August 91 (613)</td>
<td>152 to 155</td>
<td>8.6 Metres</td>
<td>Full width excavated</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Props at 142,145,150,152 under load</td>
</tr>
<tr>
<td>12 August 91 (616)</td>
<td>155 to 162</td>
<td>8 Metres</td>
<td>props at 142,146,150,154,158 under load</td>
</tr>
<tr>
<td>29 August 91 (633)</td>
<td></td>
<td></td>
<td>All props in instrumented area released after permanent prop construction</td>
</tr>
<tr>
<td>2 Sept 91 (637)</td>
<td></td>
<td></td>
<td>All props moved out of instrumented area</td>
</tr>
</tbody>
</table>
Fig. 12 Underpass construction
from the wall than at distances of 5 m and 8 m from the wall. The range of initial in situ stresses measured by the three cells at this depth was probably due to the variable nature of the clay.

6.1.2 Porewater pressures

Detailed plots of porewater pressure measured by the spade cell piezometers and vibrating wire piezometers are given in Figures 18 to 20. The porewater pressures generally show a slight rise as pile construction approached each piezometer location, followed by a rapid fall of up to 150 kPa as the three piles closest to the piezometer position were constructed. The porewater pressures then rose steadily and by Day 162 were close to the values measured before piling took place. The piezometers in front of the wall (SC8 to 11) were still showing some increase in pressure on Day 162 but measurements made on Day 185 showed no further significant rise. The spade cell piezometers (SC6 and SC7) at 5 m and 8 m from the wall showed similar but smaller changes in porewater pressures than SC2 at 2.25 m from the wall at the same depth of 6 metres.

Profiles of the porewater pressure before and after pile installation given in Figure 21 include the values measured by the vibrating wire piezometers and indicate only small changes which could be due to seasonal effects rather than the construction operations.

6.1.3 Surface and subsurface ground movements

The horizontal deflections measured on inclinometer tube 11, 11.5 m away from the retaining wall are shown in Figure 22 for several dates spanning the period of pile construction. A surface lateral movement of about 5 mm towards the wall had occurred by the end of pile installation. The deflections have been calculated assuming base fixity of the inclinometer tube. The agreement between the movements of the top of the tube as determined from the inclinometer and from the Geomensor measurements shown in Table 3 confirmed that little movement of the ground occurred near the base of the tube. The horizontal movements of station 11 as measured by the tensioned tape over the period when the piles were installed are also given in Table 3. By contrast the tensioned tape measurements do not agree with the inclinometer and Geomensor results and show little movement at the ground surface. This discrepancy was thought to be due to the high extension pillar needed to raise the tape above the car park level, which would serve to exaggerate errors caused by seating of the pillar and any tilting of the ground station itself.

Precise levelling on the surface stations 11, X4, and M5-M8 (Figure 23) showed settlements of about 3 mm over the period (Day 140 to 162). Measurements on the magnetic settlement rings M5, M6, and M7 all showed settlements of about 2 to 3 mm with less than 1 mm being recorded on the deepest ring M8 (Figure 24).
Fig. 14 Total lateral stress in the retained ground at 2.25m from the centreline of the wall
Fig. 15 Total lateral stress on underpass side of wall at 2.25m from the centreline of the wall
Fig. 16 Total lateral stress during pile installation
Fig. 17 Total lateral stress in the retained ground at 6m depth for three distances from the wall
Fig 18 Spade cell piezometers in the retained ground at 2.25m from the centreline of the wall
Fig. 19 Vibrating wire piezometers in the retained ground at 2.25m from the centreline of the wall
Fig. 20 Spade cell piezometers on the underpass side of the wall at 2.25m from the centreline of the wall
Fig. 21 Porewater pressure profiles in the retained ground at 2.25m from the centreline of the wall

TABLE 3

Lateral movement towards wall of top of tube I1 before, during, and after pile construction

<table>
<thead>
<tr>
<th>Day</th>
<th>Inclinometer I1 (mm)</th>
<th>Geomensor (mm)</th>
<th>Tensioned tape (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18th April '90</td>
<td>135</td>
<td>0.2</td>
<td>0.2 *</td>
</tr>
<tr>
<td>1st May '90</td>
<td>148</td>
<td>3.3</td>
<td>2.1</td>
</tr>
<tr>
<td>3rd May '90</td>
<td>150</td>
<td>4.8</td>
<td>6.2</td>
</tr>
<tr>
<td>4th May '90</td>
<td>151</td>
<td>4.8</td>
<td>4.9</td>
</tr>
<tr>
<td>15th May '90</td>
<td>162</td>
<td>4.7</td>
<td>3.6</td>
</tr>
</tbody>
</table>

* Adjusted to give common datum
Fig. 22 Deformation profiles measured in the retained ground at 11.5m from the centreline of the wall (I1)
Fig. 23 Vertical ground surface movements (at 11.5m behind the wall) and vertical wall movements
Fig. 24 Vertical subsurface movements in the retained ground at 11.5m behind the wall
6.2 UNDERPASS CONSTRUCTION

6.2.1 Earth pressures

During stage 1 of the bulk excavation when the walls were unpropped (from about Day 597 to 618) the spade cells behind the north eastern wall showed reductions in total lateral stress of up to 72 kPa with the largest reductions being measured by the shallower spade cells (Figure 14). The two cells still functioning behind the south eastern wall at depths of 3 m and 4.7 m showed reductions in total lateral stress of between 53 kPa and 95 kPa respectively. During the time the temporary props were in place (Day 618 to Day 632) increases in total lateral stress occurred behind both walls of between 7 kPa and 24 kPa, probably associated with thermal expansion of the steel temporary props. Following removal of the temporary props the total lateral stress fell to give a maximum overall decrease during the underpass construction period (Day 492 to 674) of 116 kPa at cell SC2 (at a depth of 6.1 m), and 124 kPa at cell SC16 (at a depth of 4.7 m) behind the south eastern wall.

The total lateral stresses measured in front of the wall prior to and following excavation are shown in Figure 15. With the exception of spade cell SC11, the cells in front of the wall were disconnected below dredge level some weeks before excavation occurred and reconnected afterwards: no data were therefore obtained during excavation. After excavation the results obtained from SC9 and SC10 showed that the total lateral stress in front of the wall fell between Day 477 and 684 by 56 kPa and 65 kPa respectively.

Figure 25 shows profiles of total lateral stress before and shortly after the construction of the underpass. When the overall reductions in total lateral stress measured by the cells behind the north eastern wall during pile installation and underpass construction are calculated, Figure 26 shows that an approximately uniform reduction of about 100 kPa occurred in the retained ground behind the wall.

Figure 17 shows the total lateral stresses measured at a depth of 6 m by cells SC2, SC6 and SC7 at 2.25 m, 5 m, and 8 m from the wall. All three cells show reductions in total lateral stress during underpass construction with the cell SC2 closest to the wall indicating the largest fall. Small increases in total lateral stress were once again observed while the temporary props were in place.

6.2.2 Porewater pressures

The porewater pressures behind the north eastern wall (Figures 18 and 19) decreased as underpass construction took place, with reductions of between 4 kPa and 26 kPa being measured. Several piezometers (SC2, SC3, VP3, VP4) showed small increases of up to 7 kPa in porewater pressure whilst the temporary props were in place. Similar results were obtained on the piezometers behind the south eastern wall. The piezometers (SC9 and SC10) below the carriageway (Figure 20) showed reductions in porewater pressure of 93 and 72 kPa respectively over the period of underpass construction. Profiles of porewater pressure with depth behind the north eastern wall, before and after underpass construction, are shown in Figure 27.

6.2.3 Wall movements

Figure 28 shows the movement profiles for inclinometer tube I2 in pile N155 for five stages of underpass construction. The results have been corrected for movement of the top of the tube measured by the Geomensor. During stage 1 of the excavation the top of the wall cantilevered towards the underpass by about 12 mm, whilst the toe moved back 1.5 mm. The movement of the top of the wall towards the underpass reached 18 mm immediately before release of the temporary props. Following their release the top of the wall moved about 5 mm towards the underpass whilst the toe moved by about 3 mm in the same direction. Some two weeks later the top of the wall had moved a further 2 mm towards the underpass with little movement of the toe being observed.

A comparison of movements measured using the inclinometer tube I2 (pile N155) and the electrolevels (pile N154) is given in Figure 29 for two days during the study. The overall lateral movements of the top of the piles were in reasonable agreement, however the profile indicated by the electrolevels was less smooth. This behaviour was explained when the unprocessed electrolevel outputs were examined: this showed that about one third of the devices were drifting during the study. Figure 30 gives typical unprocessed outputs for four electrolevels that showed drift together with four that did not.

Measurements to the top of the wall using the Geomensor showed that, during the first stage of excavation, lateral movements at X1 and X2 were 21 mm and 23 mm respectively. During the time the temporary props were in place, lateral movements of 2 mm and 5 mm away from the underpass were recorded at X1 and X2. Following release of the temporary props immediate movements towards the underpass of between about 3 mm and 5 mm were observed. Overall movements measured from the start of underpass construction of 29 mm and 27 mm were recorded at X1, and X2 respectively some two weeks after completion.

Vertical movements of the precise levelling stations on the wall are given in Figure 23. The four stations (X1, X2, I2, I5) all indicated an upward heave of the wall of about 9 mm starting at about Day 500 and ceasing around Day 640. Checks on the stability of the TBM mounted on pillar PP, by levelling to TBMs maintained by the Contractor showed changes of less than 2 mm over the construction period of the underpass.

6.2.4 Surface and subsurface ground movements

During the period from February 1991 (Day 441) up to the start of underpass construction the top of inclinometer I1 moved about 6 mm towards the wall possibly due to an excavation on the south side of the underpass for ground anchor trials around piles S215 to S220 and bulk excavation of the underpass approaching from an easterly
Fig. 25 Total lateral stress during underpass construction measured at 2.25m from the centreline of the wall.

- **Retained Side**
  - Before
  - After
  - K=1

- **Underpass Side**
  - Before
  - After

**Corrected total lateral stress (kPa)**

Depth (m)
Fig. 26 Reductions in total lateral stress in the retained ground
Fig. 27 Porewater pressure profiles during underpass construction measured at 2.25m from the centreline of the wall
Fig. 28 Lateral movement of pile N155 during underpass construction
Fig. 29 Comparison of deflected shapes of the wall measured by inclinometer and electrolevels
direction. Measurements of the movement of the inclinometer tube I2 in pile N155 for the same period showed a similar amount of movement towards the underpass. During underpass construction (Figure 22) the top of the inclinometer tube I1 moved, assuming base fixity of the tube, by an additional 4 mm towards the underpass, with only a small further movement of 1 mm being observed some two weeks later. These values can be compared with those from the Geomensor measurements which showed a total surface movement at I1 of 8.5 mm and at X4 of 8 mm over the same period suggesting that very little, if any, movement was occurring at the bottom of tube I1.

The three levelling stations located on the retained ground (I1, X4, M5-M8) all show a similar pattern of settlement during underpass construction (Figure 23) with settlements of about 4 mm occurring from before any excavation (Day 500) until release of the temporary props (Day 633).

Figure 24 shows the vertical movements of the magnetic rings M5 to M8 in the retained ground at a distance of 11.5 m from the wall. The shallowest ring M5 settled by about 6 mm in the period from the start of excavation up to prop release (Day 500 to 633) whilst M6 indicated a settlement of around 5 mm. Rings M7 and M8, the two deepest rings, showed settlements of between 5 and 6 mm up to prop release followed by heaves of 3.5 and 7 mm respectively, when measurements were made several days later. Although heave of a few millimetres might be expected at this depth and distance from the wall (Carswell et al., 1993), the measured magnitudes were higher than anticipated. Despite thorough investigation, no malfunction of the system could be found and the cause of the indicated heave is unclear.

6.2.5 Prop loads and wall bending moments

6.2.5.1 Prop loads

The strain gauges indicated that the temporary prop loads were generally in the range 310 kN to 575 kN during completion of excavation and construction of the permanent prop slab giving a load per metre run of wall of about 43 to 80 kN. Temporary prop loads were not high at this site as the major part of the excavation was carried out before prop installation. Figure 31 shows the loads calculated from the hydraulic jack pressures on one prop over a 27 hour period in which the prop temperature changed by over 9°C. The load in the prop rose with increasing temperature from 235 kN to 430 kN an increase of some 83%.

The initial loads measured in the concrete thrust blocks of the permanent prop slab (Figure 32) after release of the temporary props ranged between about 450 kN and 700 kN giving a mean thrust of approximately 112 kN per

Fig. 31 Variation of temporary prop load with temperature
metre run of wall. However it must be noted that because only a few days elapsed between casting of the thrust blocks and casting the permanent slab, the datum zero values of the strain gauges taken on Day 627 may be significantly in error due to thermal and curing effects in the concrete. An indication of the possible magnitude of these errors was that measurements taken on block 3 on 28th August (Day 632) showed the block to be in tension by 346 kN. The results for this block have therefore been adjusted to a datum zero for Day 632. The prop load per metre run of wall derived from force balance calculations using the measured earth pressures at 1.5 m away from the wall face was about 300 kN. This is three times the measured value and hence the pressures on the wall may be lower than the values 1.5 m away.

6.2.5.2 Wall bending moments

Figure 33 shows the measured and calculated bending moments in pile S210 for five stages of underpass construction. Moments were determined from strains measured on each pair of gauges based on the flexural rigidity (EI) for each pile of 7.26x10^6 MN/m², assuming that the concrete would remain uncracked at the small strain levels involved. On this basis the measured moments at each stage of construction showed a similar pattern but were much smaller than those calculated from the measured earth pressures and prop loads. The bending moments calculated from the measured permanent prop load and earth pressures at 14 days after temporary prop removal and those calculated using the design working load for the permanent prop of 650 kN/m are also shown for comparison.

The values of calculated bending moment would be expected to be an upper bound as additional stress relief would occur much closer to the wall than was measured by the spade cells located at 1.5 m away from the wall face. Bending moments calculated from the electrolves in pile N154 were unreliable due to the drift of some of the electrolves as discussed in Section 6.2.3. Therefore, they have not been presented here.

6.3 POST CONSTRUCTION

6.3.1 Earth pressures

The distributions of total lateral stress measured in July 1992 (Day 958), some 10 months after completion of the underpass construction, are shown in Figure 34. The four cells in the retained ground at 2.25 m from the centreline of the wall recorded stresses corresponding reasonably
Fig. 33 Bending moments for pile S210 at 5 stages of underpass construction
Fig. 34 Total lateral stresses measured 10 months after underpass completion at 2.25m from the centreline of the wall
with a K value of just over 1. The total lateral stresses measured by the spade cells at the same distance in front of the wall, with the exception of the shallowest spade cell, were close to the passive pressures calculated using the lower bound parameters \( c' = 0 \) kPa, \( \phi' = 24^\circ \) obtained from triaxial compression tests. The passive earth pressures were calculated assuming a wall friction angle of \( \phi' / 2 \) and zero wall cohesion from Caquot and Kenisiel (1948) in accordance with the recommendations of Padfield and Mair (1984).

### 6.3.2 Porewater pressures

The porewater pressure distributions for July 1992 are shown in Figure 35. The pressures recorded on the retained side at 2.25 m from the centrelne of the wall are approximately hydrostatic with depth, from a ground water level about 4 m below the surface. Beneath the carriageway on the underpass side the limited porewater pressure data were consistent with a hydrostatic variation with depth from a ground water level immediately below the prop slab. The geophones (SC9, SC10, VP5) beneath the underpass carriageway showed increases in porewater pressure during June and July 1992, and further measurements will be required to confirm whether this is an upward trend or a seasonal variation.

### 6.3.3 Wall movements

Figure 36 shows the lateral movement profiles for wall inclinometer tube I2 for four dates between September 1991 (Day 658) and July 1992 (Day 959). The values have been corrected for movement of the top of the tube measured by the Geomensor. The wall rotated about the permanent prop between September and December 1991 with about 3 mm of outward lateral movement occurring at the top and a similar movement in the opposite direction at the toe, virtually no movement being observed at prop slab level. No significant further movement occurred up to July 1992 (Day 959).

The overall lateral movement of the top of the wall from pile installation to July 1992 (Day 959), measured by the Geomensor at stations X1, X2, I2 and I5 was between 22 mm and 25 mm towards the underpass. Vertical movement of the top of the wall measured at the same stations showed settlement of about 2 mm up to about Feb 1992 (Day 800) followed by a gradual heave of up to 3 mm.

### 6.3.4 Surface and subsurface ground movements

Horizontal movements of the ground surface behind the wall measured at the Geomensor stations I1, X4, and M6-M8 following underpass completion showed an overall movement of about 12 mm up to July 1992 (Day 959), with a superimposed fluctuation of +/- 3 mm. This was consistent with the results from inclinometer I1 (Figure 22), at the same distance of 11.5 m from the wall, which showed an overall horizontal movement of about 14 mm up to July 1992.

The deepest magnetic settlement ring M8 at a depth of 15.1 m, and at 11.5 m from the wall indicated little or no movement between September 1991 (Day 658) and January 1992 (Day 784) but subsequently indicated a steady heave reaching about 6 mm up to mid July 1992 (Day 959). Heave at depth was also observed in the retained ground at a bored pile retaining wall at Walthamstow (Carswell, Carder and Gent, 1993). The two shallowest rings (M5 and M6) settled gradually by about 2 mm up to April 1992 (Day 855) and then recovered to their post underpass completion values by July 1992 (Day 959).

### 6.3.5 Permanent prop loads and wall bending moments

The results in Figure 32 show that the loads being transmitted through the three instrumented thrust blocks to the prop slab fell following underpass construction and reached a minimum in December 1991 (about Day 780). The loads then increased until by July 1992 (Day 959) they had risen to between 475 and 800 kN, equivalent to about 93 and 157 kN/m run of wall. This behaviour is consistent with a thermal expansion of the prop slab in summer. This same mechanism was considered to account for seasonal variations in measured lateral stress recorded behind a propped retaining wall at Maiden Way (Carder and Symons, 1989). Further observations will be made to confirm this apparent seasonal variation.

The bending moments measured using the vibrating wire strain gauges in pile S210 are given in Figure 37 for three dates between underpass completion and June 1992 (Day 910). The bending moments showed small increases over the period.

### 7. CONCLUSIONS

The performance of a contiguous bored pile retaining wall founded in over-consolidated clay and propped at carriageway level was monitored during its construction at Rayleigh Weir. The following conclusions were reached.

1. The initial horizontal stresses in the ground measured using spade pressure cells were consistent with a K value of approximately 2.1. Piezometer measurements indicated that the water table was about 2 m below the ground surface and that the profile with depth was slightly below that of a hydrostatic distribution.

2. As a result of installation of the pile wall, reductions in lateral stress were recorded with the largest reductions being measured at depth. Twelve days following piling across the instrumented area, a mean K-value of 1.9 was measured. During installation of the piles nearest to each spade cell, an immediate decrease in porewater pressure of up to 150 kPa was measured, although the pressures quickly recovered to values close to those existing prior to construction. During installation, a horizontal surface movement and settlement of about 5 mm and 3 mm respectively were recorded at 11.5 m away from the wall; subsurface movements at this location were of smaller magnitude.
Fig. 35 Porewater pressures measured 10 months after underpass completion at 2.25m from the centreline of the wall
3. During underpass construction the total lateral stress in the retained ground decreased, the largest reductions being measured at shallow depths. When the effect of pile installation is included, these changes correspond to an overall reduction of about 100 kPa from the initial in situ stress, the change being fairly uniform with depth. During the unproped stage of underpass excavation, the top of the wall cantilevered towards the excavation by about 12 mm: a further 6 mm of outward lateral movement occurred whilst the temporary props were in place. Following release of the props an additional 7 mm of movement occurred over the following 2 weeks. Precise levelling showed that the wall heaved about 9 mm during underpass construction. Surface lateral movements and settlements of the retained ground at 11.5 m from the wall were 10 mm and 4 mm respectively during underpass construction.

4. Loads calculated on the temporary steel props were in the range 43 to 80 kN per metre run of wall and varied considerably with temperature. Temporary prop loads were not high, as the major part of the excavation was carried out before prop installation. Following release of the temporary props, loads in the thrust blocks of the permanent prop slab were between 450 and 700 kN, i.e. a mean load of 113 kN per metre run of wall. The maximum bending moment measured over the upper 12 m of the wall, using strain gauges, was of the order of 300 kNm/m: this was considerably smaller than that determined from spade cell and prop load measurements. This might be anticipated as further stress relief may have occurred between the spade cell locations at 1.5 m from the wall and the wall face. Long term drift of some of the electrolevels prevented the determination of bending moments using these devices.

5. By July 1992 (Day 959), 10 months after underpass completion, total lateral earth pressures measured in the retained ground were consistent with a K-value of just over 1. With the exception of the shallowest spade cell measurements beneath the carriageway were close to the passive pressures calculated from the "lower bound" values of soil strength obtained from triaxial tests following the recommendations of Padfield and Mair (1984). The porewater pressures on both sides of the wall followed an approximately hydrostatic distribution with depth from ground water levels of 4 m below ground level on the retained side and immediately below the carriageway prop slab on the underpass side.
6. Following completion of the instrumented section of the underpass in Sept 1991 the top of the wall rotated about 3 mm towards the underpass by December 1991 with no movement at prop slab level. The surface of the retained ground at 11.5 m from the wall moved a further 4 mm towards the underpass up to July 1992 (Day 959). Loads in the thrust blocks of the prop slab followed a seasonal variation consistent with thermal expansion during the summer.

8. ACKNOWLEDGMENTS

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9. REFERENCES


MORE INFORMATION FROM TRL

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PR10 Behaviour during construction of a propped contiguous bored pile wall in stiff clay at Walthamstow, I Carswell, D R Carder and A J C Gent, Price Code H

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