Behaviour during construction of a propped secant pile wall in stiff clay at Hackney to M11 link

by S N Bennett, D R Carder
and M D Ryley
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BEHAVIOUR DURING CONSTRUCTION OF A PROPPED SECANT PILE WALL IN STIFF CLAY AT HACKNEY TO M11 LINK

by S N Bennett, D R Carder and M D Ryley

This report describes work commissioned by the Bridges Engineering Division of the Highways Agency under E468A/BG, Behaviour of Bored Pile Retaining Structures during Construction.

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EXECUTIVE SUMMARY

For environmental reasons highways within built-up areas are increasingly being constructed below ground in retained cutting using bored pile or diaphragm retaining walls. The use of permanent reinforced concrete prop slabs at carriageway level to support the walls can be advantageous in terms of reducing the depth of wall penetration required for stability and hence the overall cost of the construction. However, little guidance is available on the design of such structures particularly when they are founded in stiff clays where high in situ lateral stresses can produce high structural loadings on both the wall and propping system.

This report describes the field instrumentation and monitoring carried out to establish the behaviour of a propped secant bored pile wall under construction at Hackney. Measurements of movements, total lateral stress and porewater pressures were carried out in the ground. The movements and bending moments developed in the wall and the loads in the permanent prop were also monitored.

During pile installation, surface lateral movements away from the pile were measured with a maximum of about 7mm being recorded at 1m distance. This behaviour was probably caused by a ‘displacement effect’ in the 5.8m surficial sandy gravel as a consequence of using a heavy oscillator to drive full depth casing for installation of the male piles. Measurements at other sites confirm that normally surface movements are towards the pile. A small amount of lateral stress relief was recorded on the deep spade cells and this indicated that subsurface lateral movement was probably towards the pile installation.

During bulk excavation, the temporary props were effective in restricting movement at the top of the wall. After construction of the permanent prop slab and release of the temporary props, the wall translated out by about 12mm as load was transferred to the permanent slab. After a further 5 months some additional outward rotation of the top of the wall towards the excavation occurred with an overall movement of 20mm being measured. This agreed closely with the upper bound limit of 0.2% of the excavation depth established from previous studies. An overall heave of 19mm was recorded at 3m depth in the clay beneath the carriageway slab as a consequence of the unloading due to bulk excavation.

Initial lateral stresses prior to construction corresponded to a K-value of slightly above 2 in the London Clay which was overlain by nearly 6m of Boynt Hill Gravel. Final stresses in the retained clay at 5 months after construction were generally just below a K of 2. Measured stresses in front of the wall were close to those calculated using best fit soil parameters from triaxial compression tests assuming zero wall adhesion and a wall friction angle of $\phi/2$. 


BEHAVIOUR DURING CONSTRUCTION OF A PROPPED SECANT PILE WALL IN STIFF CLAY AT HACKNEY TO M11 LINK

ABSTRACT

For environmental reasons highways within built-up areas are increasingly being constructed below ground in retained cutting using bored pile or diaphragm retaining walls. Wall penetration, and hence costs, can often be reduced by using permanent prop slabs for support at carriageway level. However little guidance is available on the design of such structures particularly when they are founded in stiff clays where high insitu lateral stresses can produce high structural loadings on both the wall and propping system.

This report describes the field instrumentation and monitoring carried out to establish the behaviour of a propped secant bored pile wall under construction at Hackney. Measurements of movements, total lateral stress and porewater pressures were carried out in the ground. The movements and bending moments developed in the wall and the loads in the permanent prop were also monitored.

1. INTRODUCTION

The use of permanent reinforced concrete prop slabs at carriageway level to support embedded retaining walls can be advantageous in terms of reducing the depth of wall penetration required for stability. As this generally leads to savings in the construction costs, this structural form is being increasingly used for below ground construction in retained cutting. However, little guidance is available on the design of walls propped at carriageway level. The design of such structures is not straightforward and currently available advice such as that given by Padfield and Mair (1984) is limited to cantilever walls or walls propped near the top.

Previous findings have indicated that particular problems exist with walls founded in heavily over-consolidated clays because of the presence of high insitu lateral stresses. During construction, lateral stresses are likely to reduce but the magnitude of the reductions will depend almost entirely on the method of wall installation and the construction sequence adopted during excavation in front of the wall, i.e. whether movements are allowed to occur. The design of the wall and propping system in terms of the developed loads and bending moments is related to the soil stresses and thus also construction sequence dependent.

An essential element in giving design guidance for walls of this type is therefore the understanding of behaviour during construction as well as in-service. This field study of the performance of a propped secant bored pile wall founded in stiff clay at Hackney is one of a series being undertaken by TRL on behalf of the Department of Transport. Results from previous studies have been summarised by Symons (1992) and Carder (1995).

A section of the wall was extensively instrumented to measure movements, total lateral stresses and porewater pressures in the ground behind and in front of the wall. The performance of the structural members was also monitored by measuring wall movements and bending moments together with loads in the permanent prop at carriageway level. The report gives detailed results obtained during the wall installation stage, construction of the retained cutting and the 5 months following its completion.

2. SITE LOCATION

The secant pile wall being investigated forms part of the south wall on the George Green tunnel approach which is located on the new alignment of the A12 to M11 link road. The instrumented section lies between Wanstead underground station and Blake Hall Road and is centred on chainage 5020 of the Contract.

The wall at the instrumented section has a nominal retained height of 7.5m and an overall penetration of 18m.

3. SOIL PROPERTIES

3.1 SOIL PROFILE AND PLASTICITY DATA

The soil plasticity data established during the TRL investigation are shown in Fig 1. Made ground was encountered to a depth of 1m which overlaid a 4.8m band of sandy gravel (Boyn Hill Gravel). Below this was grey clay typical of London Clay which became stiffer with depth. There was some evidence of weathering in the upper 0.3m of the London Clay.

The plastic limit of the clay was fairly constant with depth around a mean value of 23%. The liquid limit was in the range of 70-80% at depths of up to 12m, although below this lower liquid limits of 60-70% were measured and this was probably due to the presence of sand lenses.
GL = 30m AOD

TRL borehole log

Moisture content (%)

Undrained shear strength (kN/m²)

Fig. 1 Properties of soil
3.2 LABORATORY TESTS

Fig 1 shows the variation of undrained shear strength with depth. Strength values were determined from triaxial tests on 100mm diameter specimens from thin walled tube samples. Strength increased with depth as indicated by the best fit line although some scatter in the results was obtained.

A summary of triaxial test results on 38mm diameter specimens cut from the high quality 100mm diameter samples to determine effective stress strength parameters is shown in Fig 2. This shows that overall best fit parameters of $c' = 12.7$ kN/m$^2$ and $\phi' = 25.1^\circ$ and lower bound parameters of $c' = 0$ kN/m$^2$ and $\phi' = 25.1^\circ$ are obtained at mean effective stress levels up to 275 kN/m$^2$.

4. DETAILS OF CONSTRUCTION SEQUENCE

Dates for the main stages of construction in the instrumented area are given in Table 1.

![Fig. 2 Evaluation of clay strength parameters from laboratory triaxial tests](image)

**TABLE 1**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
<th>Period</th>
<th>Day Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Installation of piles S209-S217</td>
<td>8/7/94 - 21/7/94</td>
<td>100-113</td>
</tr>
<tr>
<td>2</td>
<td>Capping beam constructed</td>
<td>30/9/94-17/11/94</td>
<td>184-232</td>
</tr>
<tr>
<td>3</td>
<td>Excavation to 4m depth</td>
<td>16/12/94-19/12/94</td>
<td>261-264</td>
</tr>
<tr>
<td></td>
<td>Temporary props installed</td>
<td>23/1/95-10/2/95</td>
<td>299-317</td>
</tr>
<tr>
<td>4</td>
<td>Excavation to 6.5m depth</td>
<td>21/3/95-28/3/95</td>
<td>356-363</td>
</tr>
<tr>
<td></td>
<td>Excavation to formation (8m depth)</td>
<td>11/4/95</td>
<td>377</td>
</tr>
<tr>
<td>5</td>
<td>Reinforced concrete slab cast</td>
<td>19/5/95-25/5/95</td>
<td>415-421</td>
</tr>
<tr>
<td>6</td>
<td>Jacks inserted in props</td>
<td>10/7/95-21/7/95</td>
<td>467-478</td>
</tr>
<tr>
<td>7</td>
<td>Temporary props released</td>
<td>21/7/95</td>
<td>478</td>
</tr>
</tbody>
</table>
In stage 1 the secant bored piles of 1.2m diameter were installed at 1m centres. Excavation for each female pile was carried out using a rotary auger rig (Fig 3) operating through approximately 7m of casing. Once the excavation was completed a universal I-beam (914x305mm, 224kg/m) was lowered into the excavation and the concrete placed using a hopper. Excavation for each male pile was carried out using a heavy oscillator which drove a casing with a cutting edge and an auger to remove material from inside the excavation. The casing extended the full depth of the pile and was progressively removed as the concrete was poured. The dates of installation of each pile in the instrumented area are given in Table 2. Each of the piles in the instrumented area was excavated and concreted on the same day.

The pile tops were then reduced to their cut-off level and a reinforced concrete capping beam constructed. Shortly after this, excavation to 4m depth below ground level took place to allow access for installation of temporary props. The steel temporary props were 1100mm extended diameter and had a 21mm wall thickness. The props were lowered into place at the top of the wall as shown in Fig 4 and then grouted into position against the capping beam face. In the instrumented area the spacing between centres of adjacent props varied between 2.7m and 3.87m.

Bulk excavation was then carried out beneath the props using a bucket excavator and dozer with spoil being transported away by lorry. Excavation was completed to a depth of 8m throughout the instrumented area during April 1995 ready for construction of the permanent prop slab at carriageway level (Fig 5). A 300mm drainage layer and 75mm layer of blinding concrete were placed before construction of the permanent prop slab.

The prop was designed as a horizontal reinforced concrete slab and its thickness varied from 0.9m at the wall to 1.1m at the carriageway centre. The slab was constructed in nominal 16m length bays with 25mm expansion joints between adjacent bays (Fig 6). The joint between the slab and the wall was constructed using a shear connection which allowed some rotation to occur, thus accommodating long term heave of the underlying clay.

After construction of the permanent prop slab the temporary props were destressed and removed. This was carried out by installing a hydraulic jack between one end of each
Fig 4.  Temporary props installed before excavation

Fig 5.  Excavation to formation level
TABLE 2
Pile installation in instrumented area

<table>
<thead>
<tr>
<th>Pile Number</th>
<th>Description</th>
<th>Date</th>
<th>Day Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>214</td>
<td>Casing installed</td>
<td>8/7/94</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Excavation and concreting</td>
<td>14/7/94</td>
<td>106</td>
</tr>
<tr>
<td>216</td>
<td>Casing installed</td>
<td>12/7/94</td>
<td>104</td>
</tr>
<tr>
<td></td>
<td>Excavation and concreting</td>
<td>12/7/94</td>
<td>104</td>
</tr>
<tr>
<td>217</td>
<td>Casing installed</td>
<td>15/7/94</td>
<td>107</td>
</tr>
<tr>
<td></td>
<td>Excavation and concreting</td>
<td>15/7/94</td>
<td>107</td>
</tr>
<tr>
<td>215</td>
<td>Casing installed</td>
<td>16/7/94</td>
<td>108</td>
</tr>
<tr>
<td></td>
<td>Excavation and concreting</td>
<td>16/7/94</td>
<td>108</td>
</tr>
<tr>
<td>212</td>
<td>Casing installed</td>
<td>14/7/94</td>
<td>106</td>
</tr>
<tr>
<td></td>
<td>Excavation and concreting</td>
<td>18/7/94</td>
<td>110</td>
</tr>
<tr>
<td>213</td>
<td>Casing installed</td>
<td>19/7/94</td>
<td>111</td>
</tr>
<tr>
<td></td>
<td>Excavation and concreting</td>
<td>19/7/94</td>
<td>111</td>
</tr>
<tr>
<td>210</td>
<td>Casing installed</td>
<td>18/7/94</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td>Excavation and concreting</td>
<td>19/7/94</td>
<td>111</td>
</tr>
<tr>
<td>211</td>
<td>Casing installed</td>
<td>20/7/94</td>
<td>112</td>
</tr>
<tr>
<td></td>
<td>Excavation and concreting</td>
<td>20/7/94</td>
<td>112</td>
</tr>
<tr>
<td>209</td>
<td>Casing installed</td>
<td>21/7/94</td>
<td>113</td>
</tr>
<tr>
<td></td>
<td>Excavation and concreting</td>
<td>21/7/94</td>
<td>113</td>
</tr>
</tbody>
</table>

prop and the wall, breaking out the grout and then using the jack to destress the prop before final removal. Following this the final road surface was constructed.

5. FIELD INSTRUMENTATION

The layout of the field instrumentation to monitor the behaviour of the secant pile wall and construction of the tunnel approach is shown in section and plan in Figs 7 and 8 respectively. All ground instrumentation was installed in April 1994, about three months before construction began in the research area. Instruments in the piles and permanent prop slab were installed at the appropriate stage during the construction works.

5.1 EARTH AND WATER PRESSURES

Nine spade shaped pressure cells (SC) incorporating pneumatic piezometers were installed to monitor total lateral stresses and porewater pressures in the ground. Five of the spade cells were located 1.25m behind the wall as shown in Figs 7 and 8, the remaining four cells were located 1.25m in front of the wall and below the formation level of the permanent prop slab. Each cell was installed by boring a hole and jacking the instrument a further 0.5m into the undisturbed ground at the bottom of the borehole. Boreholes were then backfilled with bentonite pellets to provide an impermeable seal. In some boreholes separate pneumatic piezometers (PP) were installed to provide additional data: for example three piezometers were installed in the centre of the carriageway to measure the groundwater regime in this area.

Throughout this study, spade cell readings have been empirically corrected by subtracting one half of the undrained shear strength as recommended by Tedd and Charles (1983). A more refined calibration was presented by Ryley and Carder (1995) which confirmed that this correction was appropriate for design purposes.

5.2 GROUND MOVEMENTS

Various measurement techniques were employed to assess the extent and magnitude of ground surface and subsurface movement. Surface movements were monitored using stations L1-L5 as shown in Fig 7. These stations comprised machined stainless steel inserts cast into small concrete blocks founded at 0.5m depth. The inserts were designed to accept the invar staff for precise levelling measurements.
Fig 6. Reinforced steel cage for permanent prop slab

Inclinometer tubes in piles S211 and S215

Vibrating wire gauges in prop slab

Vibrating wire gauges in pile S214

Fig 7 Composite section showing instrumentation
and also for the attachment of pillars for lateral movement measurements using a tensioned tape extensometer. All levelling data were referred to a benchmark on Dangan Road over 30m from the construction zone.

Two magnetic ring extensometer systems were installed in the area to be excavated to measure subsurface ground heave beneath the prop slab: one was along the centre of the carriageway and one positioned 1.5m away from the bored pile wall. During bulk excavation the access tubes to these instruments were progressively removed. From precise levelling on the tops of the access tubes and the readings on the magnetic rings the magnitude of heave caused by excavation was calculated.

5.3 WALL MOVEMENTS AND BENDING MOMENTS

During installation of the bored pile wall two 140mm square steel ducts were cast vertically into piles S215 and S211. When wall installation was complete, inclinometer access tubes were grouted into the steel ducts using a deep tremie to ensure that the grout (containing a non-shrink additive) reached the bottom of the ducts.

The two inclinometer tubes 11 and 12 were used for surveys of lateral wall movement. Absolute movements of the tops of the inclinometer tubes were monitored using a high precision electronic distance meter called a Geomensor mounted on a reference pillar installed 14m behind the wall. This pillar was constructed from reinforced concrete with a surrounding plastic sheath to prevent bending caused by differential temperatures.

Ten pairs of vibrating wire strain gauges were installed in pile S214 along the flanges of the universal I-beam so that one gauge of each pair was positioned at the back and one towards the front. The depth interval between the gauges varied as shown in Fig 7 with the gauges being more closely spaced at the level of the permanent prop slab as the peak bending moment was anticipated in this area.

5.4 PERMANENT PROP LOADS

Axial strains and hence loads in the permanent prop slab were monitored using eight vibrating wire strain gauges. These were installed in pairs on the top and bottom of the reinforcing cage at distances of 1.5m and 3m from the pile wall. The gauges were installed in two lines spaced 3m apart as shown in the instrument layouts in Figs 7 and 8.
6. DISCUSSION OF RESULTS

6.1 INSTALLATION OF BORED PILE WALL

6.1.1 Earth and water pressures

The changes in total lateral stress in the ground as measured by the spade cells behind the wall during pile installation are shown in Fig 9. Similar changes were recorded by the cells in front of the wall. The measured stresses have been corrected throughout by deducting 0.5\(c\) as discussed in Section 5.1. Fig 9 shows that a significant drop in lateral stress was detected at each spade cell location during excavation of the adjoining pile. The largest reductions of 120kN/m and 80kN/m were measured behind and in front of the wall respectively at 14.5m depth. After the piles were concreted the lateral stress steadily increased, Fig 10 shows the overall changes caused by wall installation.

The results in Fig 10 indicate that stresses in the clay prior to construction generally corresponded to a K-value (ratio of effective horizontal to vertical stress) of slightly above 2. This value is typical of that expected in London Clay (Symons, 1992; Burland et al, 1979). The K lines in Fig 10 have been calculated on the basis of the measured porewater pressure distribution. The effect of pile installation on the stresses was such that a small amount of stress relief was recorded on the deep spade cells, whilst stresses on the shallow spade cells generally returned to similar values to those measured before construction. Normally lateral stress reductions due to bored pile installation (Symons and Carder, 1992) would be expected to be larger than measured at this site.

The distribution of porewater pressure with depth determined from the spade cell and pneumatic piezometers prior to construction is shown in Fig 11. The results indicated that the ground water level was about 4.5m below the surface (i.e. at 25.5m AOD) and conformed to a hydrostatic distribution with depth. During pile installation the porewater pressure changes measured in the ground behind the wall are shown in Fig 12. Generally a reduction in porewater pressure occurred during excavation for the piles in the instrumented area although within 20 days of completion of this section of wall, porewater pressures had generally returned to their equilibrium values prior to wall installation.

6.1.2 Ground movement

Surface lateral ground movements were monitored during the pile installation phase by tensioned tape extensometer and the results are shown in Fig 13. Movement measurements were referred to surface station L5 at a distance of 15m from the wall. Surface stations L1, L2 and L3 consistently moved away from the bored pile wall during construction: this was probably caused by a 'displacement effect' in the 5.8m surficial sandy gravel as a consequence of using a heavy oscillator to drive full depth casing for installation of the male piles. A sharp increase in lateral movement was
detected at day 108 which was due to the installation of the nearest pile S215. A maximum surface lateral movement of about 7mm was recorded 1m away from the wall on station L1. Precise levelling on the same surface stations during wall installation showed no measurable vertical movement.

It should be noted that surface ground movements normally occur towards a wall installed in stiff clay as reported in a number of cases by Carder (1995). The results at this site demonstrate the dependence of the magnitude and direction of ground movements upon the construction technique employed and the ground conditions.

6.1.3 Pile temperature

The temperatures in pile S214 were monitored by thermistors which were incorporated in the strain gauges attached to the pile I-beam. Fig 14 shows the temperatures recorded by every alternate thermistor on the front face of the I-beam,
Fig 12. Porewater pressure changes at 1.25m away during wall installation

Fig 13. Surface lateral ground movement during installation of bored pile wall
thermistors on the backface gave near identical results. The variations up to day 111 were caused by construction of piles S215 and S216 on either side of the instrumented pile. Peak temperatures of about 35°C due to the heat of hydration given off during concrete curing were recorded about 9 days after the pour and a further 50 days later the temperatures had dropped to about 22°C. After day 170, thermistors 1 and 5 at the exposed top of the pile showed variations caused by changes in atmospheric temperatures.

Fig 15 shows the same temperature data plotted against depth at four different times after installation. Generally, with the exception of the exposed pile top, the temperature remained fairly constant with depth during the concrete curing probably because the I-beam tended to act as a heat conductor.

For the purpose of interpreting the strain gauge results, day 232 was selected as a suitable datum date when concrete shrinkage during curing was complete and before any excavation had taken place in front of the wall.

6.2 CONSTRUCTION OF THE TUNNEL APPROACH

6.2.1 Earth and water pressures

The corrected total lateral stresses and porewater pressures measured on the retained and excavated sides of the wall during the various stages of construction of the tunnel approach are shown in Figs 16, 17, 18 and 19.

Fig 16 indicates that a gradual reduction in the total lateral stresses in the retained ground began after excavation to 4m
Fig 16. Total lateral stress changes behind the wall throughout the construction period

Fig 17. Porewater pressure changes behind the wall throughout the construction period
Fig 18. Total lateral stress changes in front of the wall throughout the construction period

Fig 19. Porewater pressure changes in front of the wall throughout the construction period
depth to provide access for installation of the temporary prop spanning the tunnel approach. Further lateral stress reduction occurred behind the pile wall as excavation continued to formation level depth and overall changes in stress of between 60 kN/m² and 100 kN/m² were measured throughout the construction period. Porewater pressures measured in the retained ground close to the wall (Fig 17) showed a reduction of up to 12 kN/m² when excavation to formation level was carried out, although a partial recovery in values occurred after the following few months.

Some fluctuations in both lateral stress and porewater pressure in the retained ground were observed around day 425 as a consequence of the haul road through the instrumented area being heavily used for plant operations.

The measurements from spade pressure cells and piezometers in front of the wall are shown in Figs 18 and 19 respectively. These instruments were not read as frequently as those behind the wall, because they had to be disconnected during bulk excavation. Lateral stress changes of up to 80 kN/m² were recorded during bulk excavation (Fig 18) although very little change was observed on temporary prop release. The results in Fig 19 show a marked reduction in porewater pressures during both the initial excavation and final excavation to formation level as the water table fell to below the respective excavation levels.

The results in Fig 20 summarise the total lateral stress changes on both sides of the wall which occurred during construction of the tunnel approach. Also shown on the retained side of the wall are the lines corresponding to K-values of 1 and 2 calculated assuming the porewater pressure distribution is hydrostatic from a depth of 4.5 m (i.e. 25.5 m AOD). On this basis the initial stresses prior to construction were all slightly above a K of 2 whereas, 5 months after construction, stresses were generally just below a K of 2.

The measured stresses in front of the wall are compared in Fig 20 with the passive values calculated using the observed porewater pressures and the best fit soil parameters from triaxial compression tests of $c' = 13$ kN/m² and $\phi' = 25^\circ$. In one case the values of the passive earth pressure coefficient $K_p$ have been determined assuming a wall friction angle of $\phi'/2$ and zero wall adhesion from Caquot and Kerisel (1948) in accordance with the recommendations of Padfield and Mair (1984). In the other case, an assumption of full wall friction has been used. Generally measured values were below the line calculated assuming full wall friction and in closer agreement with that using wall friction of $\phi'/2$.

A summary of the porewater pressure distributions on both sides of the wall measured 5 months after completion of construction in the instrumented area is given in Fig 21. Also shown are the hydrostatic and linear seepage relations determined from the measured water table levels. The ground water level on the retained side of the wall remained similar to that existing before construction although the distribution with depth was between that calculated using hydrostatic and linear seepage assumptions. On the excavated side of the wall the ground water level was at 18.1 m AOD and the distribution with depth was closer to the hydrostatic condition. In the longer term, the ground water level would be expected to rise further as seepage
30
= 25
317 x 711
156 x 739
100 x 729
London Clay
Fig 21. Porewater pressure distribution 5 months after prop release

around the wall occurs until it is controlled at about 21 mAOD by the sand drainage blanket beneath the permanent prop.

6.2.2 Ground movements

Although surface ground movement data were obtained during wall installation, subsequent measurements were not possible at later stages of the construction (after day 127) as the surface stations were disturbed by the trafficking of construction plant.

The subsurface heave of the ground below the permanent prop slab was monitored using the magnetic extensometers whose location are shown in Fig 7. During excavation to formation and construction of the slab measurements were not possible because of extensive site traffic and limited access. However, 5 months after completion of the prop slab and release of the temporary props a mean heave of 19mm was recorded at 3m depth below the top of the slab. The heave reduced with depth, mean values of 12mm and 7mm being measured at depths of 7m and 13m respectively.

6.2.3 Wall movements

The measured lateral movements at the top of inclinometer tube 12 in the wall are shown in Fig 22 during construction of the tunnel approach. Readings using the electronic distance measuring system (Geomensor) were only available from day 366 onwards. The inclinometer results assume base fixity, which takes no account of any movement of the toe of the wall. This was assessed using the Geomensor in two ways, firstly by assuming the Geomensor reference pillar at 14m from the wall remained stationary, and secondly by measuring the changes in span between the opposing walls and assuming they moved identically.

Fig 22 shows a comparison of the lateral movements at the top of the wall obtained using these three techniques. The difference between the semi-span and the movement of the instrumented wall measured using the Geomensor was minimal during the construction period up until the release of the temporary props. After this time there were small differences of up to 3mm which indicated that some movement of the reference pillar at 14m from the wall was occurring. For this reason the readings of semi-span were used in conjunction with the inclinometer results to establish absolute wall movement profiles. The profiles of lateral wall movement determined in this manner are shown in Fig 23.

No significant lateral movement of the wall occurred during excavation to 4m depth for temporary prop installation as is shown in Fig 23a. Bulk excavation then took place below the props which were effective in supporting the wall and preventing any excessive lateral movement although thermal expansion and contraction of the props caused small fluctuations of ±3mm at the wall top as indicated in Fig 22 and Fig 23b. It must be noted that wall movement measurements were not taken at the extremes of temperature and larger movements than this may have actually occurred. During the period the temporary props were in place supporting the full excavation, prop temperatures were logged at 3 hour intervals and varied between -1°C and 40°C. Associated loads measured in the props in the instrumented area were 700kN and 1360kN respectively.
On temporary prop release, the wall translated out by about 12mm as load was transferred to the permanent prop slab which had been cast at carriageway level (Fig 23c). After a further 5 months some additional outward rotation of the top of the wall towards the excavation occurred with an overall movement of 20mm being measured (Fig 23d). If this movement is expressed non-dimensionally as a percentage of the excavation depth a value of 0.24% is obtained: this is slightly above the upper limit of 0.2% for similar walls propped at carriageway level determined by Carder (1995). However, if the overall effect of wall installation and bulk excavation is compared with that at other sites, lateral movements are similar.

6.2.4 Wall bending moments

The bending moments were calculated from the strains given by the ten pairs of strain gauges installed into pile S214. The position of the gauges relative to the temporary prop and permanent prop slab can be seen in Fig 7. Moments were calculated from the bending strains determined from each pair of gauges based on the flexural rigidity (EI) per metre run of the secant pile wall of 2.87 x 10^6 kNm^2, which assumes that the concrete would remain uncracked at the small strain levels involved.

The bending moment distribution at different construction stages is shown in Fig 24. After excavation to 4m depth for installation of the temporary props, virtually no change in the bending moment profile was recorded (Fig 24a). Once excavation had been completed to formation level and temporary props installed anticlockwise bending moments developed between prop and dredge levels. Initially the moments peaked at 600kNm/m just above dredge level as indicated in Fig 24b. Two weeks after the prop slab was cast a further increase in the peak bending moment occurred with 940kNm/m being measured at 7m depth. Fig 24c shows the bending moment distribution shortly after the temporary props were released as the bending moments started to reverse, 5 months later (Fig 24d) a maximum clockwise moment of approaching 500kNm/m was measured near permanent prop level.

If the bending moments are calculated on the basis of active pressures on the retained side of the wall assuming zero cohesion and ϕ of 36° for the overlying gravel, a peak bending moment of 400kNm/m is determined at carriageway level. This agrees closely with the measured value of 500kNm/m shown in Fig 24d.

6.2.5 Permanent prop loads

Axial loads in the permanent prop slab were calculated from the strain measurements on pairs of vibrating wire gauges installed on the top and bottom of the slab reinforcing cage. The mean load from three pairs of strain gauges is shown in Fig 25: results from the other pair of gauges were not available because of failure of one of the gauges. The results demonstrated that some load (about 250kN/m) developed in the slab before temporary prop release. After their release further load increases occurred with a peak load of 503kN/m being recorded about 3 months later.
Fig 23. Development of wall movements

(a) Excavation to 4m prior to installation of temporary props
(b) After excavation to formation (8m)
(c) Temporary prop release
(d) 5 Months after temporary prop release
Fig 24. Development of wall bending moment

(a) Excavation to 4m prior to installation of temporary props
(b) Excavation to formation
(c) Temporary prop release
(d) 5 Months after temporary prop release
7. SUMMARY AND CONCLUSIONS

The behaviour of a secant bored pile wall founded in stiff over-consolidated clay and permanently propped at carriageway level has been monitored during its construction. The construction procedure involved excavation beneath high level temporary props. The following conclusions were reached.

(i) During pile installation, surface lateral movements away from the pile were measured with a maximum of about 7mm being recorded at 1m distance. This behaviour was probably caused by a 'displacement effect' in the 5.8m surficial sandy gravel as a consequence of using a heavy oscillator to drive full depth casing for installation of the male piles. Measurements at other sites confirm that surface movements normally occur towards the pile. A small amount of lateral stress relief was recorded on the deep spade cells and this indicated that subsurface lateral movement was probably towards the pile installation. Generally a reduction in porewater pressures occurred close to the excavation for each pile, although porewater pressures returned to their initial values within about 20 days of completion of this section of wall.

(ii) During bulk excavation, the temporary props were effective in restricting movement at the top of the wall. After construction of the permanent prop slab and release of the temporary props, the wall translated out by about 12mm as load was transferred to the permanent slab. After a further 5 months some additional outward rotation of the top of the wall towards the excavation occurred with an overall movement of 20mm being measured. If this movement is expressed non-dimensionally as a percentage of excavation depth a value of 0.24% is obtained: this is slightly above the upper limit of 0.2% established from previous studies. An overall heave of 19mm was recorded at 3m depth in the clay beneath the carriageway slab as a consequence of the unloading due to bulk excavation.

(iii) Initial lateral stresses prior to construction corresponded to a K-value of slightly above 2 in the London Clay which was overlain by nearly 6m of Boyne Hill Gravel. Final stresses in the retained clay at 5 months after construction were generally just below a K of 2. Measured stresses in front of the wall were close to those calculated using best fit soil parameters from triaxial compression tests assuming zero wall adhesion and a wall friction angle of $\phi/2$.

(iv) At the end of construction a wall bending moment of approaching 500kN/m was measured near permanent prop level. This value agreed closely with that determined from active pressures in the gravel on the retained side of the wall. A permanent prop load of 500kN/m was recorded about 3 months after construction, although loads reduced to 432kN/m after a further 2 months probably as a result of a seasonal thermal contraction of the permanent prop slab.

Fig 25. Development of permanent prop loads
(v) Further measurements are required to establish the longer term performance of the wall.

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9. REFERENCES


MORE INFORMATION FROM TRL

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

PR 10 Behaviour during construction of a propped contiguous bored pile wall in stiff clay at Walthamstow. IG Carswell, DR Carder and AJC Gent. Price Code H.

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