BEHAVIOUR OF A TEMPORARY ANCHORED SHEET PILE WALL ON A1(M) AT HATFIELD

by I F Symons, J A Little, T A McNulty, D R Carder and S G O Williams

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FOREWORD

A major part of the A1(M) Improvement project at Hatfield, Hertfordshire, consists of a dual three lane motorway within a 1150 metres long tunnel. The tunnel has been constructed by the ‘cut and cover’ technique, the temporary works comprising over 2 km of sheet pile retaining walls restrained by ground anchors in predominantly granular soils.

During the design of the sheet pile walls, the Main Civil Contractor, Tarmac Construction Ltd, recognised that the project provided a rare opportunity to monitor the performance of a temporary anchored wall at full scale. The design and installation of the ground anchors was subcontracted to Cementation Piling and Foundation Ltd, who also recognised the opportunity to study anchor behaviour.

The Civil Engineering Task Force report (1981) identified the need to refine understanding of retaining wall behaviour as one of the selected programmes requiring geotechnical research. In response to the latter SERC and CIRIA are funding research studies of retaining walls. TRRL are also undertaking a programme of research on comparatively rigid insitu concrete retaining walls on behalf of the Department of Transport. Monitoring the performance of a temporary anchored wall complemented this strategy.

The specific aims of the research were to provide further understanding of soil-structure interaction of temporary retaining walls, to assess the relevance of factors applied in current design procedures, and to increase knowledge of ground anchor behaviour. The aims were achieved through measurement of ground movement, pile bending strains, ground anchor strains and head loads, and by a comparison of observed behaviour with the design predictions which, of necessity, incorporated simplifying assumptions.

From inception, it was recognised that for a field measurement project to succeed there were three essential ingredients; all parties must be willing to fully co-operate; very experienced personnel were needed to install and use the instrumentation; and finance must be available. Through extensive collaboration all these ingredients were achieved at Hatfield.

The research was initiated by Tarmac and jointly organised with the other key participants, Cementation Research Ltd, the TRRL and Hatfield Polytechnic. Funding was provided by the first three organisations and by the SERC through a research grant to Hatfield Polytechnic for instrumentation and research personnel. * A Steering Group was set-up to produce overall coordination of the work. The project was fortunate in that the research institutions were located either in or relatively near Hatfield, enabling rapid communication on a busy working site.

Additionally the TRRL provided the personnel experienced in installing and using an extensive range of geotechnical instrumentation. Cementation Research Ltd also provided personnel similarly experienced and developed equipment specifically for the project.

The timescale available to organise the project and place orders for vital instrumentation was short by comparison with normal research activities, so it is to the credit of SERC that funds were made available quickly to fit in with the programme constraints of a ‘live’ contract. Acknowledgement is also made to the Engineer for the A1(M) Improvement Scheme, Hertfordshire County Council, who appreciated the research initiative and gave permission for it to proceed.

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ABSTRACT

For road improvement schemes in urban areas the restricted space available has led to an increase in the use of earth retaining structures for both permanent and temporary works. Such structures involve complex problems of soil-structure interaction which limit the accuracy of design predictions.

The instrumentation and monitoring of full scale structures provides an essential element in research to improve understanding of retaining wall behaviour. This report describes a collaborative study carried out on a section of anchored sheet pile wall in granular soil. The wall formed part of the temporary works on the A1 trunk road improvement scheme at Hatfield in Hertfordshire.

Information is provided on the pattern of ground movements, deflections and strains in the sheet piles and behaviour of the prestressed ground anchors during construction and over the ten month period when the wall was in service.

1 INTRODUCTION

In urban and semi-urban areas improvements to the existing road network frequently necessitate the construction of new roads in retained cutting, in cut and cover tunnel or on retained embankment. This stems from the higher standards of alignment required for modern roads and the restricted space available. Greater use is therefore being made of earth retaining structures both for permanent and temporary works.

The improvement scheme to the A1 trunk road at Hatfield in Hertfordshire provides an example of one such project and involved the construction of an 1150 m long cut and cover tunnel together with a 350 m length of retained cut for the tunnel northern approach. At this site the ground conditions over the tunnel depth consist of predominantly granular soils. Temporary anchored sheet pile walls were used to retain the ground during construction of the permanent works, which comprised reinforced concrete retaining walls supporting the tunnel roof to form a two bay portal. On completion of backfilling behind the permanent retaining walls the sheet pile walls were withdrawn.

Retaining walls of the type used as temporary works at Hatfield involve complex problems of soil-structure interaction which limit the accuracy of predictions of behaviour made at the design stage. For example uncertainties concerning the effect of wall and anchor installation on the in situ stresses and soil properties make it difficult to reliably assess the bending moments developed in the wall, the loads imposed on the anchors and the ground movements which will occur during and after excavation. To provide more data on these topics a section of the anchored sheet pile wall on the northern approach to the Hatfield tunnel has been extensively instrumented. This report describes the field observations and compares the measured anchor loads and bending moments in the wall with the values determined in design.

2 THE SITE

2.1 SITE LOCATION

The site was located approximately 250 m north of the tunnel portal at Green Lanes roundabout, Hatfield (Figure 1a) and the research area was situated at chainage 4700 immediately adjacent to the new southbound carriageway of the A1(M). The research area was positioned within a 250 m continuous length of sheet piling and offered sufficient space in which to install instrumentation in open ground to the east.

The 20 m long instrumented section of sheet piling was located in the central reservation of the original A1 carriageways, Figure 1b. The research area extended from approximately 4 m west, to approximately 50 m east of the sheet piling across the former A1 southbound carriageway and beyond into a wide grassed area.

2.2 GROUND CONDITIONS

2.2.1 GEOLOGY

The site lies within an area of the Vale of St Albans where Pleistocene Anglian deposits overlie Chalk bedrock. The sequences of sediments in the Vale have been described by various authors. Whilst locally variations do exist, generally a lower sequence of fluviatile (Proto-Thames) sands and gravels (Westmill Lower Gravel) is separated from an upper sequence (Westmill Upper Gravel) of fluvioglacial sands and gravels by a lodgement till deposit (Ware Till) of variable thickness (Little and Atkinson, 1985). An extensive site investigation carried out in 1979 on behalf of the Hertfordshire Sub-Unit of the Eastern Road Construction Unit in preparation for the A1(M) Improvement Scheme, confirmed this general sequence.
An additional smaller investigation at the research site was carried out in January 1985, to obtain specific information on the deposits. Four boreholes were put down to depths ranging from 15.5 m to 25 m at the positions shown in Figure 1b, using a light cable percussion boring rig. The soil and groundwater profiles obtained from the boreholes which were situated very close to the line of the piling showed little variation. These profiles (based on visual inspection and soil grading tests) have been summarised in the composite log shown in Figure 2a. The level of the till in borehole 2 occurred approximately 1 m higher than that shown.

In boreholes 1 to 3 continuous Standard Penetration Testing (SPT) and bulk sampling of all granular soils was carried out together with continuous U100 sampling of the till. Borehole 4 was put down solely to confirm the level of the upper surface of the till.

The results from the SPT's showed some variation between similar soils in different boreholes; comparisons between SPT 'N' values proved particularly difficult to summarise. The SPT profile shown in Figure 2a is obtained from borehole 1, located closest to the sheet pile wall.

2.2.2 Laboratory soil testing
The granular soil above the till in boreholes 1 to 3 was tested to determine representative values of the drained shear strength. These were compared with the values of $\phi'$ derived from the SPT results obtained in the main site investigation (see Section 3.1) and used for the design of the temporary wall.

In addition, wet sieving of all the granular soils, particle specific gravity and compaction testing of selected samples was carried out. The grading test
results confirmed the findings of a visual examination of the granular soils in the identification of three distinct layers (designated A, B, C in Figure 2a) at approximately 3 m, 5 m, and 7 m below ground level. Grading curves for these three layers are shown in Figure 2b.

The coarse nature of soils A and B required a 300 mm shear box for testing; the sand, however, was satisfactorily tested in a standard 60 mm shear box.

Samples were tested at normal stresses corresponding to $\frac{1}{2}$, 1 and 2 times overburden pressure at strain rates compatible with a condition of full drainage. The range of values for $\phi'$ obtained, shown in Figure 2c, reflects in part the initial density at which the soils were tested. The largest values for $\phi'$ were obtained at the maximum soil density and optimum water content determined from compaction tests carried out in accordance with Test 14 of BS1377 (British Standards Institution, 1975). The

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Fig. 2 Soil properties

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lower values were those obtained from soil tested at lower densities corresponding to a medium dense state which the relation between SPT 'N' value and relative density (Thorburn, 1963) indicated were more representative of the insitu states. The maximum and minimum voids ratios required for relative density determination were obtained using the method outlined by Kolbuszewski, 1948.

3 DESIGN CONSIDERATIONS

The structural design of the sheet pile wall at Hatfield was based on the conventional approach for this type of structure. This consists of determining the depth of penetration necessary to ensure that overall collapse of the wall and adjoining ground does not take place and that the structural elements are capable of carrying the imposed load. For driven walls an additional consideration is that the wall components are capable of resisting the stresses induced by the driving operations.

The depth of penetration is normally established from a stability assessment based on limiting equilibrium methods of analysis in which conditions at failure are examined and a suitable factor of safety then introduced to ensure that such a collapse state will not develop. The introduction of the factor of safety can be done in several ways: by using a multiplying factor on the depth of penetration at limiting equilibrium, by factoring soil strength or by various methods of factoring the net or gross forces imposed on the structure (Symons, 1983). The magnitude of the factor of safety adopted in design is dependent on which of these approaches is adopted (Potts and Burland, 1983) but must be of sufficient magnitude in all cases to cater for inherent uncertainties in the input parameters. In general the determination of representative soil strength parameters constitutes a major source of this uncertainty. For granular soils this can present particular problems since undisturbed samples cannot be obtained and strength parameters are therefore often derived from empirical correlations with insitu tests (Peck, Hanson and Thornburn, 1974).

The structural design involves the assessment of the loads in the anchors, the maximum bending moments, and shear forces applied to the structural elements under working conditions. In practice the magnitude and distribution of earth pressures acting on the structure will depend on factors such as the initial stress state in the ground, the changes in soil stress produced by wall construction and the relative stiffness of the wall and ground (Tedd, Chard, Charles and Symons, 1984; Potts and Fourie, 1984). Although these soil-structure interaction effects can be examined using numerical techniques such as the finite element method, their application in design is limited by the difficulty of determining reliable values for the required input parameters. For design purposes an estimate of bending moments and shear forces is therefore normally obtained using simplified assumptions concerning the magnitudes and distributions of earth pressures acting on the structure in service.

The design approach outlined above provides no information on the ground movements caused by the construction of the wall, although this is a prime concern at Hatfield where roads, buildings and buried services are located in close proximity. The complexity of the construction sequence and effects of soil-structure interaction generally preclude accurate predictions of ground movements although an assessment of their upper and lower bounds can be obtained from case records (Draft Code of Practice DD81, 1982) and from empirical relations based on previous field measurements (Peck, 1969).

3.1 DESIGN OF THE SHEET PILE WALL

The detailed design of the sheet pile wall was carried out at the tender stage and in the period prior to the start of construction using information on the ground conditions and soil properties obtained during the main site investigation. This involved a comprehensive assessment of wall stability using a number of different design methods to determine the depths of pile penetration required for the range of retained heights encountered along the scheme. This assessment also provided design values of maximum bending moment and propping force used for the selection of the type of sheet pile and for the design of the walings and ground anchors. Both free and fixed earth support conditions were considered in the calculations and an examination made of the effect of variations in position of the support system. In addition an appraisal was made of the likely driving conditions at the site based on empirical relations and the Contractor's previous experience of piling in this type of ground.

The principal method of analysis used for the design of the wall was based on the recommendations contained in the British Steel Corporation Piling Handbook (1984). The BSC method considers only the net total pressure distribution which is obtained as a difference between the total active pressures and the total passive pressures acting on the wall. For a free earth support condition the factor of safety against rotation of the wall about the prop position is then equal to the net resisting moment divided by the net overturning moment. The prop force is determined by considering the equilibrium of forces acting over a height of wall having a factor of safety of unity. The BSC method permits a reduction in the maximum value of the bending moment of up to 25 per cent at pile deflections in excess of 0.5 per cent of the span, where a uniform homogenous granular soil exists for the entire depth of the structure. In view of the ground conditions at the site no such allowance was made for moment reduction in the design calculations.
A representative cross-section adopted for the design of the wall at the instrumented section is shown in Figure 3 together with the corresponding distribution of net total pressures. The design values of soil properties were derived from the results of Standard Penetration Tests carried out during the main site investigation for the scheme. The calculations using the BSC method gave a horizontal prop force of 167 kN/m and a maximum bending moment of 422 kNm/m acting at a depth of 7.3 m below the top of the piles. Using a factor of safety of 2 gave a required depth of penetration below the 9 m excavation level of 3.92 m and a pile length of 13 m was therefore adopted.

For this length of pile the factor of safety given by the BSC method is shown in Table 1 together with the values obtained by a number of other methods of assessing wall stability. The method given in the Code of Practice on Earth Retaining Structures (CP2, 1951) factors the total passive soil resistance. The factor of safety given by the Revised Method (Burland, Potts and Walsh, 1981) is the ratio of the moment of the net available passive resistance to the moment activated by the retained material. The factor on embedment is the ratio of the actual depth of penetration below excavation level to the depth required at limiting moment equilibrium. Fuller descriptions of these methods are given by Padfield and Mair (1984). The results are compatible with previous studies (Potts and Burland, 1983) which show that the BSC method gives consistently higher factors of safety than other design methods.

On the basis of the general ground conditions and test results available prior to the start of construction it was assessed that driving sheet piles would be difficult if they penetrated the clay by a significant amount. A Larssen no 3 section would have been a reasonable choice for driving piles in panels. However noise restrictions at the site dictated the use of a Hushrig and with this technique pairs of piles have to be driven to full depth in one operation and an increase of pile stiffness was therefore considered advisable. Larssen 4/20 sheet piles on Grade 50B steel were therefore chosen as giving reasonable assurance of reaching the depths required to satisfy the stability criterion, and of meeting the structural requirements. The driving operations provided some check on the ground conditions at the site. Thus if general refusal had occurred at appreciably shallower depth than the driving appraisal had suggested, it would have indicated that the ground was different from that anticipated, and led to a reappraisal of the wall stability.

### 3.2 ANCHOR DESIGN

The wall was supported at the propping position by a system of walings and anchors. The anchors were designed in accordance with generally recognised methods described in the Draft Code of Practice DD81 (British Standards Institution, 1982). The position of the fixed anchor zone was selected with due consideration to the overall stability of the wall/anchorage system. The draft code describes an empirical method which may be used to examine the

<table>
<thead>
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<th>Method</th>
<th>BSC</th>
<th>CP2</th>
<th>Revised</th>
<th>Embedment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor of safety</td>
<td>2.13</td>
<td>1.47</td>
<td>1.50</td>
<td>1.21</td>
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</tbody>
</table>
stability of walls supported by a single level of anchorages. Figure 4 shows how this method was applied to the sheet pile wall. Using this, the fixed anchor length is required to lie outside the cross hatched area shown in the Figure. The free length and inclination of the anchor, required to satisfy stability were 9.0 m and 25 degrees to the horizontal respectively. The inclined anchors were installed at 3.048 m centres and were prestressed to 560 kN, in order to achieve the required horizontal load of 167 kN/m run (Section 3.1). The design of pressure grouted anchors is based primarily upon principles similar to those applied to pile design. The fixed anchor length (L) was determined according to the draft code from the following empirical equation for the ultimate pullout capacity (P) of pressure grouted anchors:

\[ P = L \cdot n \cdot \tan \phi' \]

where \( n \) is an empirical factor to take account of the drilling technique, depth of overburden, anchor diameter, grout pressure, insitu stress field and dilative characteristics. The value of \( n \) is normally determined from test anchors. Two test anchors were constructed at Hatfield and the value of \( n \) was found to be 200 kN/m. Using a value for \( \phi' \) of 35 degrees in conjunction with a factor of safety of 2 against failure of the ground gave a fixed anchor length of 8 m.

The draft code recommends minimum factors of safety to be applied to the characteristic strength of the prestressing strand. For temporary anchors these factors are 1.6 and 1.25 for the working load and test load conditions respectively. The anchors used at the instrumented section were manufactured using four 15.4 mm ‘Supa’ type prestressing strands which, when stressed simultaneously, have an ultimate capacity of 1000 kN based on their characteristic strength. The factor of safety against failure of the strand was therefore 1.8 at the working load of 560 kN and 1.4 at the test load of 700 kN.

4 CONSTRUCTION OF THE WALL

In Table 2 a brief description of the stages in construction of the temporary anchored sheet pile wall is given together with the dates when each phase was carried out at the instrumented section. The construction procedure was generally typical of that used in other areas of the site.

4.1 STAGE 1: PILE DRIVING

Pile driving was carried out percussively using a Hushrig so as to conform to the noise restrictions for the site. Prior to driving, a 0.5 m deep and 2 m wide trench was excavated along the projected line of the sheet pile wall to accommodate the runners for the piling rig. Larssen 4/20 steel sheet piles of 13 m length were then driven in clutched pairs sequentially.
<table>
<thead>
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<th>STAGE</th>
<th>DESCRIPTION</th>
<th>PERIOD</th>
<th>SCHEMATIC</th>
</tr>
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<tbody>
<tr>
<td>1.</td>
<td>Pile driving</td>
<td>25/7/85-31/7/85</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Excavation to 3.2 m depth</td>
<td>14/8/85-28/8/85</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Anchor installation and tensioning</td>
<td>10/9/85-20/9/85</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>a. Bulk excavation to 7.1 m depth in central part of tunnel approach</td>
<td>28/8/85-17/9/85</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b. Excavation to 7.1 m depth in front of sheet pile wall</td>
<td>20/9/85-1/10/85</td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>a. Dewatering operational</td>
<td>11/10/85-5/12/85</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b. Excavation to 9.3 m depth for foundations</td>
<td>11/10/85-26/11/85</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b. Backfilling and release of anchors</td>
<td>28/4/86-21/5/86</td>
<td></td>
</tr>
<tr>
<td></td>
<td>c. Carriageway construction, battering behind wall and pile extraction</td>
<td>1/5/86-3/7/86</td>
<td></td>
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</tbody>
</table>
to full depth and not by the normal panel method. Dimensions and properties of the piles are given in the British Steel Corporation Piling Handbook (1984). Plate 1 shows the rig in position prior to driving one of the instrumented piles.

Plate 1 Hushrig driving instrumented pile

4.2 STAGE 2: EXCAVATION TO 3.2 m DEPTH

Before excavation was possible in the region of the instrumented section, breakout of the concrete of the northbound carriageway of the old A1 took place across the tunnel approach. Excavation was then carried out by backhoe with the spoil being transported away by lorry; the excavation operation is shown in Plate 2. Near to the instrumented sheet pile wall this procedure took a few days with the top 1.5 m of spoil being stripped first, followed by further excavation to 3.2 m depth to permit access for anchor installation.

Plate 2 Excavation

and secured by three ground anchors; metal plates of varying thicknesses were welded to the sheet piles to provide packing between the piles and the walings as required.

Details of the ground anchor used are shown in Figure 5. During installation a full length of 76 mm ID by 90 mm OD heavy duty casing, closed at the leading end by a sacrificial point, was vibrator driven to full depth using the rig shown in Plate 3. A neat cement grout was then tremied into the casing and the assembled anchor installed. During withdrawal of the casing over the fixed anchor length the grout was pressurised to between 1500 and 2000 kN/m². Once the casing had cleared the fixed anchor length withdrawal continued without pressurisation of the grout. The grout was then allowed to cure for about 7 days before the anchors were stressed. Stressing of the anchors was carried out using an hydraulic jack fitted with a calibrated pressure gauge and the movement of the ram of the jack was recorded. The anchors were initially test loaded to 700 kN, then destressed, after which the permanent wedges were installed and the working load of 560 kN applied. A typical stressing record is shown in Figure 6.

4.4 STAGE 4: BULK EXCAVATION TO 7.1 m DEPTH

While anchor installation was in progress, bulk excavation by backhoe took place initially to about 7 m depth below original ground level over the central portion of the tunnel approach, leaving a 6 m wide berm in front of the instrumented section of temporary works. Subsequently this berm was excavated to complete the bulk excavation to a measured depth of 7.1 m near to the sheet pile wall.
Plate 3 Anchor installation

4.5 STAGE 5: DEWATERING AND EXCAVATION TO FOUNDATION LEVEL (9.3 m)

Before excavating to foundation level, the water table in the cut was controlled by pumping from well points installed in the pan of the sheet piles at 2 m spacings along the wall. Dewatering occurred to a depth of about 10 m below original ground level and resulted in a drawdown of about 2 m from the water level existing prior to the construction works.

A strip about 8 m wide in front of the sheet pile was then excavated to a depth of 9.3 m prior to placing a 75 mm thick layer of blinding concrete ready for the construction of the permanent works.

Fig. 5 Typical ground anchor

Fig. 6 Typical anchor stressing record
4.6 STAGE 6: POST-EXCAVATION OPERATIONS

The base slab of the permanent reinforced concrete wall was cast full width (Plate 4) and the wall stem constructed in panels. Backfilling of the 3 m wide gap between the permanent and temporary walls commenced about 6 weeks later using selected granular material obtained from the tunnel excavation. When fill had been placed up to the base of the walings, the anchors were destressed, the anchor cables cut and the walings removed. The retained ground was then cut back to a slope before the sheet piles were extracted in pairs. In front of the permanent retaining wall additional trimming to road formation level was carried out and the pavement for the southbound carriageway of the new A1(M) constructed.

Instruments to measure ground behaviour were installed as early in the construction timetable as possible to give them the maximum time to stabilise. This work was hampered by the presence of the heavily trafficked southbound carriageway of the old A1 which closely followed the line of the new local road shown in Figure 7. The major part of this work took place in March 1985 when subsurface ground movement stations and piezometers were installed in boreholes drilled using a light cable percussion boring rig. Pressure cells and the remaining instrumentation beneath or near to the southbound carriageway of the old A1 were installed in May when traffic had been diverted prior to breakout of the existing road surface and construction of the new local road.

The surface stations and the instruments in trench beneath the new local road were installed during its construction early in June in advance of this road opening to temporarily carry southbound A1 traffic in mid-July.

Datums for the instruments monitoring ground behaviour were established prior to driving the instrumented sheet piles at the end of July.

Details of the various measurement systems are given below:

5.1 MEASUREMENT OF SURFACE MOVEMENTS

Horizontal displacements of the two lines of surface movement stations (LL) and of the tops of the inclinometer tubes (I) were determined using a proprietary tensioned tape system with readings referred to tape stations LL6 and LL12 located at 40 m distance from the sheet pile wall. Vertical movements were determined by precise levelling using a temporary benchmark founded in the till at a depth of 13.2 m and remote from the influence of the construction works.

Each surface movement station consisted of a 0.5 m square concrete block founded at 0.5 m depth with a machined stainless steel insert designed to receive both the levelling staff and the attachment used for the tape measurements.

Continuity of surface movement stations across the new local road carrying the A1 southbound traffic was not feasible as lane closures would have been required to take measurements. For this reason a magnetic extensometer system with six plates (H1 to H6) and five overflow settlement cells (S1 to S5) were installed in a trench crossing the line of the new local road at a depth of about 1.5 m below the final road surface (Plate 5). A sleeved steel rod was extended from cell S1 to ground surface, so that the vertical movements of the overflow settlement gauge system could be checked by precise levelling. Plates H1 and H6 were positioned so that measurements of their horizontal movements could be related to those on the surface stations at the same distance from the wall using the inclinometer survey results (see Figure 8).
Fig. 7 Plan showing instrumentation
Fig. 8 Section showing instrumentation between chainages 4690 and 4710
To supplement the tape measurements on the tops of the inclinometer tubes attached to the piles, the horizontal movement of the top of the sheet pile wall was determined using a Mekometer electronic distance measuring system. For this, two replaceable reference pillars were positioned at approximately 50 m behind the wall and outside the area affected by the construction activities and five targets, M1 to M5 in Figure 7, were positioned on the tops of the sheet piles. The distances measured by the Mekometer from the reference pillars to these targets were resolved perpendicular to the line of the wall using angles obtained from a theodolite survey. Datum readings were established after completion of pile driving.

The accuracy of each of the techniques for measurement was such that the horizontal and vertical movement of each surface station was considered to be known to within about 0.5 mm.

**5.2 MEASUREMENT OF SUBSURFACE MOVEMENTS**

Proprietary sleeve-jointed aluminium inclinometer tubing was installed to depths of 15 m in boreholes 12, 13, 14, 16, 17 and 18. During the installation of tube 13, distortion of a joint occurred at 11 m depth with the result that the inclinometer probe could only be operated to this depth. Boreholes were backfilled with bentonite pellets below 13 m depth and with peagravel above so that the backfill had similar characteristics to that of the surrounding ground. Measurements of subsurface lateral movement in directions normal and parallel to the line of the sheet pile wall were made at 0.5 m intervals of depth using a conventional inclinometer probe. Repeatability of inclinometer readings was such that lateral movements at the tops of the tubes were considered to be known within about 1 mm with improved accuracy at depth. Long term accuracy of lateral movement determination using the inclinometer system was estimated at about double this value.

Vertical ground movements were monitored using subsurface levelling stations (LI to L4) installed in boreholes put down to two different levels in front of the wall to measure the response of the ground to excavation. These stations were similar to the other subsurface levelling stations except that after datum levels had been taken, measured lengths of rod were removed from them. After excavation to 3.2 m depth (Stage 2), L3 and L4 were then relocated and from precise levelling, the vertical movement was calculated. A similar procedure for stations L1 and L2 was carried out after completion of excavation to 9.3 m depth (Stage 5). The accuracy of measurements on stations L1 and L2 was not as good as on the other subsurface stations, as the large differences in level between the stations and the temporary benchmark led to transfer errors of the order of a few millimetres.

**5.3 MEASUREMENT ON SHEET PILES**

As hard driving had been experienced by the Contractor, it was recognised that insufficient knowledge was available to guarantee success of instrumentation attached to the pile. For this reason, a trial drive was carried out in another area of the site which will be reported separately. This proved of great value in establishing the method of fixing and protecting the inclinometer tubes and strain gauges which were attached to the sheet piles at the instrumented section.

Three inclinometer tubes 10, 11 and 15 were attached to the sheet piles driven in the instrumented section. Tubes 11 and 15 were positioned in the pan of the piles, whilst tube 10 was positioned near to the clutch between adjoining piles (Figure 7). The criteria adopted were that the inclinometer tubes should remain undamaged by the high dynamic shear forces developed during driving, whilst their method of attachment should ensure a low flexural stiffness to avoid local stiffening of the pile. Based on the experience gained from the trial drive, the attachment procedure incorporated the following features:

(a) An oversize wedge-shaped steel shoe welded at the base of each pile to protect the inclinometer
(a) A tube was fitted to the pile during driving. The tube was attached to this shoe at its lower end (see Plate 6).

(b) The inclinometer tube passed through steel retaining clips welded at 0.9 m intervals along the length of the pile.

(c) Fixed sleeves were located on the tube beneath each retaining clip. These served to transfer dynamic loading from the tubes onto the retaining clips. Elsewhere floating sleeves prevented loading on the inclinometer tubes from ground friction. These sleeves transmitted any such forces directly to the pile via the fixed sleeves and retaining clips.

(d) Steel rivets were used at 15 mm spacings on all four sides of each fixed sleeve and at each joint between adjacent lengths of tubing.

(e) A steel block was bolted onto the pile over the top of the inclinometer tube to provide an additional restraint during driving.

(f) Further protection to the inclinometer tube 10 was provided by steel ducting placed over the entire length of the tube and welded at intervals to the pile. As this inclinometer tube was near to the clutch and hence the neutral axis of the pair of piles any additional stiffening due to the ducting would have minimal effect on pile deflections.

Two pairs of piles, numbered 9 and 18 on Figure 7, were instrumented with strain gauges. From the results of the driving trial two types of gauges were selected for the study: arc welded vibrating wire gauges and spot welded electrical resistance strain gauges. A total of one hundred and twenty gauges were attached to the piles, seventy two on pile pair 18 and forty eight on pile pair 9. The gauges were fixed at 1 m intervals along the length of the pile in profiles of twelve gauges. On pile pair 18 there were four such profiles of vibrating wire gauges located on the insides of both pans and on either side of the clutch, together with two profiles of electrical resistance gauges on the outside of each pan. On pile pair 9 there were four profiles of electrical resistance gauges, one on the inside and one on the outside of both pans.

Each profile of vibrating wire gauges was protected by a mild steel channel running the length of the pile and welded at intervals along its length. These channels also served as ducts to house the cabling from the gauges. An oversize shoe was welded to the toe of the piles at the gauge positions to provide additional protection during driving (see Plate 6). The electrical resistance gauges were mounted flush with the surface of the pile and were therefore less liable to damage during driving although a protective channel was still required for the cabling. However channels could not be connected to the outside of the pans of the piles due to the constraints of the piling guide rails. For those electrical resistance gauges located on the outside of the pans, the cables were taken through holes drilled in the piles and in channels fixed to the insides of the pile pans.

Of the original one hundred and twenty gauges, 78 per cent of the vibrating wire and 71 per cent of the electrical resistance type functioned satisfactorily after pile driving. Generally the greatest losses were of gauges positioned near the tops of the piles particularly in the profiles positioned close to the clutch. Datum readings on the gauges were established after completion of driving, which meant that any locked-in driving strains (and hence stresses) were not known.

5.4 MEASUREMENT OF ANCHOR BEHAVIOUR

All five anchors within the instrumented section, numbered A3 to A7 in Figure 7, were equipped with load cells located between the anchor heads and bearing plates. On completion of the monitoring exercise the load cells were returned to the laboratory for recalibration. With the exception of No 4, which is discussed separately in Section 6.3.1.2, the load cells were found to be within 3 per cent of their original calibration. In addition anchors 4 and 6 had 'sonic probe' multipoint extensometers installed in their fixed lengths. These extensometers monitored changes in the overall dimensions of the fixed anchor length and also provided a measure of the distribution of strain in the grout. The accuracy of the sonic probe extensometer is discussed in Appendix 1. Anchor 6 was also equipped with a single point extensometer of the tensioned wire type, to measure changes in the free length. This instrument enabled measurements from the multipoint extensometer in the fixed anchor length to be translated to the anchor head. A more detailed description of the ground anchor instrumentation is given in Appendix 1.
To measure the horizontal movement of the heads of the instrumented anchors 4 and 6, a tensioned tape was used between the surface stations in the retained ground and an inclinometer ‘null system’ attached to the front faces of the sheet piles. This system comprised a specially stiffened length of inclinometer tubing pin-jointed at its lower end to the waling behind the anchor head and attached to the sheet piles via adjustable screw connectors. Prior to each set of measurements, the tube was adjusted to a vertical position using an inclinometer probe lowered into the tubing.

5.5 MEASUREMENT OF EARTH AND WATER PRESSURES

Pneumatic piezometers (PP1 to PP4) and standpipe piezometers (SP1 to SP4) were installed to monitor changes in porewater pressure during construction. The piezometer tips were surrounded by sand and the boreholes backfilled with peagravel. In cases where two piezometers were installed in the same borehole, a plug of bentonite pellets was used to seal the borehole above the lower piezometer tip. Standpipe piezometers SP2 and SP3 were installed with only short lengths of tubing attached. On completion of excavation in front of the wall these tubes were relocated and porewater pressure measurements then commenced.

Spade pressure cells SC1 and SC2 (Tedd and Charles, 1981) and pneumatic type pressure cells PC1 and PC2 (Carder and Krawczyk, 1975) were installed at depths of 4.4 m and 3.1 m respectively in boreholes located at 1.2 m from the sheet pile wall. The instruments were lowered into the boreholes, and then surrounded by sand which was hand tamped whilst adding water in an attempt to achieve good compaction. The remainder of the boreholes were backfilled with peagravel. Although it is suspected that pressure cells installed in granular soils in this manner are subject to significant registration errors, it was nevertheless considered that readings might reflect the change in total stress occurring in the ground due to the tensioning of the ground anchors.

6 OBSERVATIONS AND DISCUSSION

6.1 GROUND MOVEMENTS

6.1.1 During pile driving

Significant ground movements were observed when the sheet piles were driven into the ground by the Hushrig. The measured movements near the ground surface are shown in Figure 9. The discontinuities in the plots of lateral movement occurred because movements near to the sheet pile wall were measured at the ground surface by tape, whilst movements further away were measured using the horizontal plate gauge system located beneath the local road at a depth of 2 m below original ground level. Lateral movements were calculated assuming fixity of the horizontal plate gauge (H6) at a distance of about 20 m from the wall as tape measurements indicated no movement beyond this point. On this basis, lateral movements of the ground surface of between 15 and 18 mm were recorded by both lines of horizontal movement stations at distances within 4.5 m of the sheet pile wall. The similarity in the readings over this distance may have resulted from the presence in this area of a concrete pavement from a previous alignment of the A1 which remained partly intact at a depth of 0.5 m below ground level.

Also shown plotted in Figure 9a are the lateral surface movements calculated from the inclinometer surveys assuming base fixity of each inclinometer tube; reasonable agreement was obtained confirming the fixity condition during this stage.

In Figure 9b, ground surface settlements due to pile driving of the order of 50 mm were measured close to the piles. The amount of settlement decreased rapidly with distance from the wall until at 12 m, as in the case of the lateral movements, no further settlements were recorded.

The subsurface lateral movements measured by inclinometer during the pile driving and calculated assuming base fixity of the tubes in the till are shown in Figure 10. At each distance from the sheet pile wall, readings from the pair of inclinometer tubes showed close agreement. At a distance of 1.5 m, lateral movements towards the wall of nearly 20 mm were recorded at the ground surface. These decreased to zero over a depth of 7.5 m. Between this depth and a depth of 13 m, which corresponded to the toe of the piles, soil movements of up to 13 mm away from the sheet pile wall were measured. Lateral movements measured on the inclinometer tubes at 4.5 m from the piles were also large. Movements near the surface of up to 18 mm towards the piles were recorded, although the magnitude of these movements decreased more rapidly with depth. Below about 7.5 m depth, the movements were small and within the accuracy of measurements. As expected inclinometer tubes remote from the sheet pile wall at a distance of 20 m showed no significant movements at this stage.

Table 3 gives the ground subsurface settlements which occurred during pile driving at distances of 1.5 m and 4.5 m from the line of piles. Comparison of these data with the plot of surface settlement shown in Figure 9b shows that the ground settlements generally decreased with depth and distance from the wall. However at 1.5 m from the piles, the measured settlement at 4 m depth appears to be slightly greater than at the ground surface. Since the subsurface and surface stations were not located at an identical chainage, this difference may reflect some small variation in behaviour along the length of the wall.
Fig. 9 Ground movements near the surface caused by pile driving
TABLE 3
Subsurface settlements caused by pile driving and excavation to 9.3 m

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Settlement caused by pile driving (mm)</th>
<th>Additional settlement during excavation to 9.3 m (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At 1.5 m from wall</td>
<td>At 4.5 m from wall</td>
</tr>
<tr>
<td>2.0</td>
<td>–</td>
<td>14.7</td>
</tr>
<tr>
<td>4.0</td>
<td>49.0</td>
<td>6.0</td>
</tr>
<tr>
<td>8.5</td>
<td>12.7</td>
<td>4.9</td>
</tr>
<tr>
<td>12.0</td>
<td>2.9</td>
<td>–</td>
</tr>
</tbody>
</table>

The concrete pavement of an earlier A1 alignment ran obliquely across the instrumented section and may also have contributed to this effect. Measurements on the subsurface levelling stations at 20 m from the piles showed no movements.

The ground movements caused by pile driving are summarised in the vector diagram given in Figure 11. The pattern of movement suggests that vibration and downward drag during driving caused significant densification of the granular soils close to the piles together with an outward displacement of the soil above the pile toe.
6.1.2 During excavation

Only small ground movements were recorded during each stage of excavation to foundation level (Stages 2 to 5). For this reason only the total ground movements measured from completion of pile driving until the completion of excavation to foundation level are plotted in Figures 12 and 13.

Figure 12 shows the movements measured near the ground surface. As for the piling stage, the values of lateral movement have been calculated on the basis of fixity of horizontal plate gauge (H6). Towards the end of the excavation, a discrepancy of a few millimetres was observed between the reading on plate gauge H1 and those on the adjacent inclinometer tubes at the same depth. This has been accounted for by contraction of the aluminium rods of the plate gauge system due to a sudden fall in temperature which occurred at this time. A small correction has therefore been incorporated in the calculation of lateral movement to allow for this.

Good agreement was recorded along the two lines of horizontal movement stations. In both cases peak movements of about 8 mm towards the excavation were recorded at about 5 m distance from the sheet pile wall, with some movement being measurable up to 15 m away. Lateral movements of about 6 mm developed at 0.75 m from the wall during excavation to 3.2 m (Stage 2); however, tensioning of the anchors pulled the ground in (ie away from the excavation) by about 3 mm and thereafter the anchors acted to restrain further movements of the ground close to the piles. Subsequent to anchoring of the wall, the major part of the lateral movement occurred during excavation for foundations (Stage 5).

The movement of the tops of the inclinometer tubes calculated assuming base fixity are also shown in Figure 12a. Reasonable agreement was generally obtained, except in the case of tube 13 in line LL1 to LL3 at 4.5 m from the wall. The smaller movement recorded on this tube indicated some lack of base fixity which was not unexpected as the tube was only operational to a depth of 11 m rather than the normal 14.5 m (see Section 5.2).

The ground surface settlement plots in Figure 12b show that magnitudes generally increased with proximity to the sheet pile wall. A settlement of 9 mm was recorded at a distance of 0.75 m away from the sheet piles, although precise levelling on the tops of the piles showed no corresponding movement. One third of this 9 mm settlement occurred during Stage 2 excavation, small additional settlements then took place during anchor tensioning and bulk excavation with most of the remaining settlement being recorded during excavation to 9.3 m.
The changes in subsurface lateral movement measured using the inclinometer during the excavation phase are shown in Figure 13. Readings from inclinometer tubes I2 and I6 at 1.5 m from the sheet pile wall showed similar results with a movement of up to 12 mm being recorded at about mid-height between the anchor and final excavation levels. At the surface, lateral movements towards the excavation of 7 mm and 9 mm were measured on tubes I2 and I6. In both cases about 6 mm of outward surface lateral movement developed during excavation to 3.2 m (Stage 2) prior to the anchor being installed. Surface inward movements of 4 mm and 2 mm respectively were measured on these tubes as a result of tensioning of the anchors. Subsequent movements between the ground surface and anchor level were restrained by the anchor, with only an additional 5 mm of movement towards the excavation being
measured at the tops of tubes I2 and I6 during the main stages of the excavation. At 4.5 m away from the wall, measurements on tube I3 showed movements about 3 mm less than on I7. This occurred because, as mentioned previously, fixity at 11 m depth for tube I3 cannot be assumed. If suitable correction is made, both inclinometer tubes I3 and I7 show a uniform lateral ground movement towards the excavation of up to about 8 mm over a 5 m depth.

The magnitudes of the subsurface settlement measured during the excavation are given in Table 3. As in Stage 1 (pile driving), settlements were generally greater close to the ground surface. The general pattern of the subsurface movements measured over the 20 m length of instrumented wall and representative of the behaviour midway between the anchor positions is illustrated in the vector diagram in Figure 14a. The values in this Figure have been calculated for the period from immediately after tensioning of the ground anchors (end of Stage 3) until the end of excavation to 9.3 m (Stage 5). This has been done to eliminate the ground movements induced by the tensioning procedure. Also included in this Figure are the deflections of the sheet pile wall measured on the inclinometers attached to the piles. More detailed results on wall movement are given in Section 6.2. Most movement of the piles tended to occur about midway between anchor and excavation levels. The lateral pile movements therefore showed good correlation with the pattern of horizontal movement developed in the adjacent ground. The results indicate significant differential vertical movement between the pile and adjacent ground.
When excavation was completed to 3.2 m depth, subsurface levelling stations L3 and L4 at about 4 m depth and at 4 m and 1 m respectively from the front face of the sheet pile wall were relocated and relevelled. The measurements showed that a total ground heave of 3 mm (L3) and a settlement of 38 mm (L4) had occurred in the period from before the start of pile driving until the completion of excavation to 3.2 m depth. The response of the ground to excavation alone was assessed by deducting estimates of the subsurface settlement which had occurred during pile driving. These were obtained from the pattern of settlements measured on the retained side. On this basis heaves during this stage of the excavation of 9 and 12 mm were calculated at about 4 m depth for stations L3 and L4 respectively. The same procedure was carried out after excavation to 9.3 m for stations L1 and L2 at about 10.5 m depth at similar distances in front of the wall. These indicated total settlements of 15 mm and 13 mm respectively measured from before pile driving until completion of excavation. Deducting the subsurface settlements measured on the retained side during pile driving leads to estimated net settlements on these stations of 13 mm and 7 mm respectively for the excavation alone. Since some heave would also have been expected on these stations in response to the vertical unloading, it is possible that the settlements were a result of densification or loss of soil fines due to pumping at the nearby dewatering wellpoints.

6.1.3 Post-excavation

After excavation to 9.3 m was complete, there was a period of about 5 months during which the permanent works were being constructed. The ground movements measured during this period are shown in Figure 14b. Generally movements at this stage were small. The apparent discrepancy between the lateral movements of the top of the pile and the nearest surface station is possibly a result of ground disturbance caused by the trench for the piling rig mentioned in Section 4.1.

Backfilling between the completed reinforced concrete wall and the sheet pile wall produced no significant changes in ground movement. However large lateral movements of up to 25 mm were measured when the anchors were subsequently released and the sheet pile wall cantilevered towards the excavation. These movements are summarised in the vector diagram in Figure 14c.

6.2 BEHAVIOUR OF SHEET PILES

6.2.1 Movements

The development of lateral movement of the sheet pile wall is shown in Figure 15. Similar results were obtained from each of the three inclinometer tubes (10, 11, 15) fixed to the piles at the locations shown in Figure 7. Movement profiles have been calculated from the top of the tube where the movement was known from tape measurements, as the toe of the pile may have undergone some movement during the latter stages of excavation and base fixity could not therefore be assumed throughout the construction period.

The behaviour of the sheet pile wall during the initial excavation to 3.2 m is shown in Figure 15a. The wall
tended to behave as a free cantilever with maximum lateral movements of 3.5 mm being recorded at the top of the piles. Although some small movements into the retained ground were measured below excavation level which are compatible with this mode of behaviour, the movements measured below 5 m are close to the accuracy of measurement with the inclinometer system. If base fixity is assumed for this stage the maximum movement at the pile top would increase to 5.2 mm.

The tensioning of the ground anchors (Figure 15b) caused significant compression of the retained ground which extended to a distance of about 5 m at ground surface behind the sheet piles. This was accompanied by an inward movement of the piles of up to 6 mm at anchor level. This value can be compared with a waling movement of about 12 mm recorded by the inclinometer null system as the waling seated itself against the sheet pile wall during tensioning of the anchors. The restraining influence of the ground anchors is apparent in Figure 15c where no significant movement of the pile above anchor level was monitored during the bulk excavation (Stage 4). Movements of up to 6 mm were however recorded approximately midway between anchor and excavation levels during this stage.

During the further excavation to 9.3 m (Stage 5), the maximum lateral movement measured on the pan of the piles increased to 11.5 mm. Slightly larger movements of up to 14.5 mm were measured on the clutch of the piles. This difference may have been caused by difficulties in accurately determining movements of inclinometer tube 10 on the clutch. As the grooves on this particular inclinometer tube were not correctly orientated, movements towards the excavation were calculated by resolving inclinometer readings taken from both sets of grooves. Furthermore tensioned tape measurements to determine lateral movement of the top of this tube were taken at an angle to the main taping lines resulting in reduced accuracy. During this stage of the excavation movements of between 4 mm and 2.5 mm were measured at waling level using the inclinometer null system described earlier together with tape measurements.

Over the next 5 months whilst the reinforced concrete wall was being constructed in front of the sheet pile wall, additional lateral movements of between 2.5 and 4 mm were recorded at mid-height between anchor and foundation level (Figure 15e). The maximum deflection of the piles was then less than 0.2 per cent of the span. On backfilling between the temporary and permanent works, a small reduction of up to 2.5 mm in the maximum lateral movement was observed. Figure 15f shows the effect of cutting the cables to the ground anchors at the anchor heads. Immediately the anchors were released, significant lateral movements were measured over the top 5 m of the piles as the wall deflected as a free cantilever.
The results from the Mekometer survey, whilst not presented in detail in this report, did nevertheless show good agreement with the values of pile top deflection measured by the tensioned tape method. Thus, for example, following excavation to 3.2 m measured pile top deflections (towards the excavation) using the two independent systems were 4.0 mm (Mekometer) and 3.5 mm (tensioned tape); stressing of the ground anchors resulted in measured movements away from the excavation of 7 mm (Mekometer) and 8 mm (tensioned tape).

6.2.2 Strains and bending moments
The bending and axial strains developed in the piles at various stages during excavation were determined from the measured changes in total strains at each position down the pile length. For this the assumptions of no slippage at the clutch and a linear distribution of strain across the section of each pile pair were made. In this evaluation only the strains measured on the gauges fixed to the pile pans were used. The bending moments in the pile were then calculated from the bending strains using a section modulus of 2266 cm³/m (British Steel Corporation, 1984) and assuming a Young's modulus for the steel of 210 x 10⁶ kN/m² as adopted in the original design. The Young's modulus was confirmed by tensile tests carried out on four samples cut from a sheet pile. These tests give a mean value of 205 x 10⁶ kN/m² and a range of 200 to 210 x 10⁶ kN/m². The value of the section modulus was checked by the measurement of the dimensions of the instrumented piles. The bending moments are shown in Figure 16.

An estimate of the reproducibility of the readings from the two types of gauges was determined from a comparison of sets of data obtained on a number of occasions during the relatively stable periods between the main construction stages. This analysis indicated that an error of no more than ±5 kNm/m in the calculated bending moment should result from the use of this type of instrumentation. The overall accuracy in assessing the maximum bending moments is therefore considered to be within the 15 kNm/m range shown in Figure 16 from the measurements obtained on pile pairs 9 and 18.

Figure 16a shows that the peak bending moments in the piles induced by excavation to a depth of 3.2 m in Stage 2 were of the order of 23 kNm/m for pile pair 9 and 30 kNm/m for pile pair 18. These maxima occurred at approximately 0.8 m below excavation level. Only small tensile axial strains of up to 20 x 10⁻⁶ were measured in the piling following Stage 2 excavation and these may represent some relaxation in the compressive strains which would have been expected in the piles due to the driving operation.

It was anticipated that localised effects introduced by the placing of walings and stressing of the anchors during Stage 3 would hamper the interpretation of strain data in this vicinity. This was further exacerbated by the loss of gauges which occurred near the tops of the piles during driving. Great reliance cannot therefore be placed on the magnitudes of the bending moments and axial strains calculated at, and above, anchor level. However the results do indicate the expected reversal in bending...
moment just below anchor level as a consequence of anchor stressing (Figure 16b).

The effect of further excavation in front of the wall during Stages 4 and 5 was to progressively increase the bending moments developed below anchor level and also the depths at which the maximum moments occurred. By completion of excavation to foundation level in Stage 5 the strain measurements indicated a maximum moment of 65 kNm/m at a depth below original ground level of about 6 m on pile pair 18 (Figure 16d). If an allowance is made for the effect of the protective channels on the pile stiffness, these bending moments may be increased by about 15 per cent suggesting a maximum moment of about 75 kNm/m. No changes in axial strain were measured in the piles during excavation in Stages 4 and 5.

After 5 months had elapsed, the permanent retaining wall in front of the temporary piling had been constructed but at this stage the backfill had still to be placed between the wall and the piling. A set of readings at this time showed that the maximum bending moment had increased to about 95 kNm/m (Figure 16e), although the general pattern of the bending moment distribution had remained unchanged. This value corresponds to a maximum moment of about 110 kNm/m if allowance is made for the stiffening effect of the protective channels. Backfilling to waling level induced only a very small reduction in the maximum bending moment derived from the measured strains. After backfilling was complete and the anchors were released, the bending moments in the upper part of the piling then reduced considerably with the wall deflecting as a free cantilever (Figure 16f). Although it is understood that extensive damage to the piles had occurred elsewhere on the site as a result of the driving operations, a visual inspection after extraction indicated only slight distortions confined near to the toes of the instrumented piles. Also the readings from the limited number of strain gauges which survived the extraction showed reasonable agreement with the initial readings taken prior to driving.

Calculated values for pile deflection obtained by a technique of graphical integration of the bending strain profiles demonstrated compatibility with pile deflections measured by inclinometer. Thus, the maximum pile deflection at the end of Stage 2 excavation computed from the strain gauge data is 5 mm; this compares with a measured value of 3.5 mm from the inclinometer. Similarly, at the end of Stage 5 excavation a computed value of 9 mm may be compared with the measured value of 11.5 mm.

6.3 ANCHOR BEHAVIOUR

6.3.1 Anchor loads

6.3.1.1 Anchor stressing

Comparisons between the loads indicated by the pressure gauge on the stressing jack and those given by the load cells on anchors 4 and 5 are shown in Figure 17. The behaviour illustrated for anchor 4 is typical of that observed on four of the instrumented anchors. The effect of friction in the stressing jack is illustrated by the difference between the loads determined from the pressure gauge during loading and unloading. These results also show that at loads below 550 kN the pressure gauge on the hydraulic jack was underestimating the true load applied to the anchor head by up to 40 kN during stressing and 110 kN during unloading. A contrary effect is demonstrated by the results from anchor 5 where the discrepancy between the two measurement systems was up to 130 kN. Subsequent recalibration of the load cell did not reveal any significant deviation from the original calibration and it is therefore concluded that the load indicated by the jack was in error. It is suspected that the apparent overread is due to a misalignment of the jack which resulted in the anchor load acting eccentrically to the jack’s axis.

6.3.1.2 Post stressing measurements

Figure 18 shows the variation in measured anchor loads with time. Immediately following stressing the load cells indicated loads of between 469 kN and 554 kN. The loads measured on anchors 3, 5, 6 and 7 show similar variations with time. Twenty four hours after stressing these load cells had recorded a reduction in load of between 1 and 3 per cent. These losses were attributed to a redistribution of waling load as adjacent anchors were stressed. The equivalent horizontal prop force, calculated from the anchor loads, was 153 kN/m. Excavation of the berm in front of the wall during Stage 4 resulted in small increases in load at the anchor heads so that by completion of excavation to 7.1 m depth the measured values had increased by approximately
10 kN. In the period between completion of excavation in Stage 4 and commencement of excavation to foundation level in Stage 5 small decreases in anchor load were registered and these are possibly associated with slight creep of the anchors. Further increases in anchor load were measured on completion of excavation in front of the wall in Stage 5. At this stage the equivalent prop force was 157 kN/m. Six weeks later the anchor loads had decreased by approximately 8 kN. Readings taken during the first quarter of 1986 have been discounted because of a readout unit malfunction. Immediately prior to backfilling between the completed permanent structure and the temporary wall the load cells indicated that the average anchor load had decreased by approaching 6 per cent since the completion of the excavation to the foundation level. This reduction is again believed to be associated with creep of the fixed anchor length. The equivalent prop force at this stage was then 148 kN/m. The effect of the backfilling on the anchor loads was negligible.

The loads measured on anchor 4 have not been included in the calculation of equivalent prop force, since they are at variance with the loads obtained on the other anchors (Figure 18). An inspection of this load cell on completion of excavation revealed the presence of water in an electrical connection. Although remedial action was taken subsequent recalibration of the cell showed that the zero point had shifted by 120 kN.

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Figure 18: Variation of anchor load with time
6.3.2 Fixed anchor length

6.3.2.1 Anchor stressing

The effect of the stressing operation on the fixed anchor length of anchors 4 and 6 is shown in Figure 19. The anchors gave very similar results and showed an extension of the fixed anchor lengths of approximately 6.5 mm under the test load of 700 kN with a permanent set of approximately 2.8 mm on unloading. This permanent set is attributed to irrecoverable deformation of the ground surrounding the fixed anchor length. When the anchors were locked off at the working load the overall extensions recorded were 6.4 mm for anchor 4 and 7.0 mm for anchor 6.

Figures 20a and 21a show the strain distributions over the fixed lengths of anchors 4 and 6 during initial stressing of the anchors. At lower load levels the distributions are of classic form with a peak occurring at the upper end of the fixed anchor length and with a steady decrease towards the lower end as load is progressively transferred to the surrounding ground. At the higher loads for anchor 4, and under the test load for anchor 6, some flattening of the strain distributions is apparent at the upper end of the fixed anchor lengths. For anchor 4 the form of the strain distribution under the test load suggests that very little load was being transferred to the ground over the upper 1.5 to 2 m indicating that the strands had begun to debond from the grout. The corresponding distribution for anchor 6 suggests that any debonding is confined nearer to the top of the fixed length. This interpretation appears to be supported by the distribution measured during subsequent destressing and restressing of the anchors. During destressing of anchor 4, Figure 20b, a rapid drop in tensile strain occurred over the upper part of the fixed length with a small amount of compression being measured when fully unloaded. It appears that the grout may have cracked at a point between 1 and 1.5 m from the top of the fixed anchor length thus effectively isolating a section of debonded grout. As the load was reduced any residual bond would be opposed by the reverse friction imposed by the deformed ground on the length of isolated grout. When the anchor was restressed to the working load, Figure 20c, the majority of the load was transmitted through the partially debonded section so that peak tensile strains in the grout occurred 2 m from the top of the fixed anchor length. The distribution obtained during destressing and restressing of anchor 6, Figures 21b and 21c, suggest that the degree of debonding was less than for anchor 4 and that cracking did not occur since during restressing the distributions were similar to those during initial stressing to the test load.

6.3.2.2 Post stressing measurements

The variations in the fixed anchor lengths with time is shown in Figure 22. The data for the overall dimension of this length suggest that the anchors behaved very differently from each other. Following stressing the extensometer in anchor 4 began to
indicate a large and increasingly compressive strain over the lower 3 m of the fixed anchor length which did not appear compatible with the distribution of strain measured in the upper 5.2 m. The measured changes in the upper 5.2 m of the fixed anchor lengths are also shown in Figure 22 and these indicate much better agreement between the two anchors. It is therefore suspected that the instrument in anchor 4 was not operating correctly over the lower 3 m and that the true variation in the fixed anchor length was similar to that recorded for anchor 6.

The response of the fixed anchor length to construction activities is broadly similar to that
Fig. 21 Distribution of strain in grout along fixed anchor length (anchor 6)
described previously for anchor loads. Thus reductions in the fixed anchor lengths occurred due to the redistribution of the waling load as adjacent anchors were stressed, and increases in the fixed anchor lengths took place in response to excavation in front of the wall during stages 4 and 5. However, in the periods between these excavation stages and following completion of excavation to foundation level, the extensometer measurements indicate that the fixed anchor lengths were increasing at a rate of about 0.02 mm per day. During construction of the permanent structure the fixed anchor length continued to increase though the rate eventually declined to less than 0.01 mm per day. This behaviour is almost certainly due to creep.

The distribution of strain measured at several stages during construction of the wall are shown in Figures 20c and 21c. Over this period the magnitudes of the tensile strains were observed to increase in response to excavation in front of the wall, although the shapes of the strain distributions remained substantially the same. Excavation to 7.1 m depth in Stage 4 caused a fairly uniform rise in strain of about $200 \times 10^{-6}$ to $300 \times 10^{-6}$ on anchor 4 while on anchor 6 similar increases in strain were confined to the lower 6 m of the fixed length. There was very little change in strain on either anchor in the period between excavation in Stages 4 and 5. Significantly larger strains were then measured on both anchors following completion of excavation to foundation level in Stage 5.

Although the sonic probe extensometer in anchor 4 eventually failed the instrument in anchor 6 continued to operate satisfactorily until the lead wires were damaged during destressing of the anchor. Throughout this period the distribution of strain, shown in Figure 21c, remained essentially unchanged. The most significant changes in strain occurred in the top 1.5 m where debonding and, probably, tensile cracks render interpretation difficult.

![Fig.22 Variation in fixed anchor length with time](image-url)
6.3.3 Anchor stressing

During initial stressing to the test load the anchor strands interfered with the measuring head mechanism and therefore no record was obtained from the single point extensometer on anchor 6. Following modification of the head, the anchor was stressed to working load and the record obtained from the extensometer is shown in Figure 23. This indicates a linear relation between anchor load and relative movement between the waling and top of the fixed anchor length. The value of 11 mm recorded when the anchor was locked off departs from this relation and is believed to be a result of friction in the extensometer system. For this reason a figure of 8.5 mm taken from the linear relation is considered to be a more representative value at this stage.

Assuming no movement at the bottom of the fixed anchor length this figure compares with a value of 11.8 mm derived from the measured extensions of the ram of the stressing jack (62 mm) and of the fixed anchor length (4.2 mm) during application of the working load together with an allowance for the elastic extension of the anchor strands (46 mm). A similar comparison incorporating the waling movement of 12 mm measured on the inclinometer null system over the entire stressing operation gave a value of 6.2 mm for the relative movement between the waling and the top of the fixed anchor length. This assessment was based on the permanent set of the ram at the end of the test load cycle (14 mm) less the extension of the fixed anchor length at working load (7 mm).

6.3.3.2 Post stressing measurements

Readings obtained from the extensometer suggested that the free length of anchor 6 was unchanged by the excavation, and subsequent operations. This is not consistent with the load cell results which indicate that the anchor load increased 12 kN during the excavation stages and then declined by 41 kN before backfilling operations commenced. This implies changes in the free anchor length, due to elastic extension/contraction, of +1.3 mm and −1.6 mm respectively. These changes are supported by the independent measurements of waling movement, recorded by the inclinometer null system and changes in the fixed anchor length, recorded by the sonic probe. If it is assumed that there was no significant movement at the bottom of the fixed anchor length then the equivalent changes in the free anchor length were approximately + 1.3 mm and −1.6 mm respectively. The discrepancies between the measurements recorded by the single point extensometer and those recorded by the other instruments is attributed to excessive friction between the tensioned wire and protective duct.

6.4 WATER AND EARTH PRESSURES

The variation in ground water levels during the construction of the temporary works is shown in Figure 24. Measurements on the piezometers installed at two depths at each location on the retained side confirmed that the porewater pressure distribution was initially hydrostatic. Prior to construction the water table across the site was fairly constant at a depth of about 8 m below natural ground level. Although a gradual fall of 0.5 m was recorded during Stages 1 to 4 of the construction (ie excavation to 7.1 m depth), this is likely to be a result of seasonal changes rather than the direct effect of the construction works at the instrumented section.

Before Stage 5 construction commenced, dewatering of the cut from wellpoints installed in the pans of the sheet piles was initiated. The effects of this dewatering was to induce an immediate further fall of up to about 1 m in the water table at all the measuring points across the section. The speed of the response to dewatering on the retained side, confirms that the sheet piles did not penetrate into the underlying impermeable till. A Mackintosh probe test, carried out in front of the wall after excavation to foundation level, also indicated that the toes of the piles were above the top of the till at the centre of the instrumented section.

After cessation of pumping, the water table across the section slowly rose by up to about 0.6 m over the following 6 month period. It was then about 0.8 m below the level of the water table before the start of construction.

The total lateral pressures measured on the four earth pressure cells are shown in Figure 25. These were located 1.2 m behind the wall at positions midway between anchors (Figure 7). As a result of the ground disturbance caused by their installation only small pressures were recorded over the period prior to driving of the piles (Stage 1). The effect of installation of the piles was to cause an increase in total pressure of about 20 kN/m² on the pneumatic cells installed at 3.1 m depth and a corresponding increase of about twice this amount on the spade cells at 4.4 m depth. Apart from an average increase of about 10 kN/m² during anchor tensioning, the pneumatic cells which were located very close to the anchor level showed only small variations in readings during the
subsequent stages of construction. The trend of these fluctuations in pressure was generally for a small decrease to occur in response to excavation in front of the wall. The spade cells located at about 1.6 m below the ground anchor level showed a smaller response to tensioning of the anchors, but this was followed by reductions in pressure of about 30 kN/m² and 12 kN/m² during excavation to 7.1 m in Stage 4 and excavation to 9.3 m in Stage 5 respectively.

To provide a framework for assessing the measured pressures, estimates of the active and at rest pressures are also shown in Figure 25. These were calculated neglecting the effects of wall friction and any surcharge loading from the local road, using the following simplified relations:

\[ K_a = 1 - \sin \phi' \]  
\[ K_a = \frac{1 - \sin \phi'}{1 + \sin \phi'} \]

(After Jaky, 1944)  
(After Rankine 1857)
The measured pressures after pile driving lie close to the at rest condition at 3.1 m and appreciably above this condition at 4.4 m depth. At 3.1 m depth no further significant changes occurred while at 4.4 m the pressure reduced to the lower limit of the active value during excavation in front of the wall.

For an anchored flexible wall in stiff ground, soil structure interaction effects are likely to lead to pressures lying below the theoretical active values between support levels, and above the active values close to the supports, resulting in a reduction in maximum bending moment on the wall (Pappin, Simpson, Felton and Raison, 1985). The limited earth pressure measurements are consistent with the bending moments determined from measured strains in suggesting this mode of behaviour at the instrumented section.

7 COMPARISONS WITH DESIGN

In this Section general comparisons are made between the values of maximum bending moment and prop force obtained from the field observations and the values given using different design methods and assumptions. The measured settlements of the ground are also compared with empirical relations based on previous field measurements.

7.1 PROP FORCE AND BENDING MOMENTS IN THE WALL

The design values of prop force and maximum bending moment given in Section 3.1 were based on the assumption of unfactored limiting active and passive earth pressures acting on the minimum height of wall required for a condition of limiting equilibrium. By contrast the values derived from the field measurements correspond to a working condition in which the wall is neither on the point of failure nor are the earth pressures necessarily at their limiting values. In addition the actual pressure distributions are unlikely to follow the simple linear increases with depth assumed in design. For these reasons comparisons between the design and measured values can only be used to obtain a general assessment of the design approach.

At this site the measurements of anchor load showed only small changes over the period of observation. The differences between the measured and design value of anchor load therefore largely reflect the

<table>
<thead>
<tr>
<th>CASE</th>
<th>MAXIMUM BENDING MOMENT (\text{kNm/m})</th>
<th>HORIZONTAL PROP FORCE (\text{kN/m})</th>
<th>ASSUMED ANGLES OF WALL FRICTION (\delta)</th>
<th>METHOD AND FACTOR OF SAFETY</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS CONSTRUCTED GEOMETRY AND REVISED SOIL PROFILE (Figure 26)</td>
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</tr>
<tr>
<td>1</td>
<td>302</td>
<td>132</td>
<td>(\delta a = 0), (\delta p = \frac{3}{2} \phi^')</td>
<td>BSC, (F = 2.33)</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>232</td>
<td>106</td>
<td>(\delta a = \delta p = \frac{3}{2} \phi^')</td>
<td>BSC, (F = 3.28)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>140-200t</td>
<td>185</td>
<td>(\delta a = \delta p = \frac{3}{2} \phi^')</td>
<td>ROWE, (F = 1.5) applied to (\tan \phi^p) and (\tan \phi^p)</td>
<td>Min required pile length = 13.4 m</td>
</tr>
<tr>
<td>DESIGN GEOMETRY AND SOIL PROFILE (Figure 3)</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>190-240t</td>
<td>210</td>
<td>(\delta a = \delta p = \frac{3}{2} \phi^')</td>
<td>BSC, (F = 2.13)</td>
<td>Min required pile length = 12.3 m</td>
</tr>
<tr>
<td>5</td>
<td>422</td>
<td>167</td>
<td>(\delta a = 0), (\delta p = 0) and (0.25 \phi^') (Figure 3)</td>
<td>BSC, (F = 2.13)</td>
<td>ORIGINAL DESIGN VALUES</td>
</tr>
</tbody>
</table>

| | 60-75t | 157 | DERIVED FROM MEASUREMENTS, ON COMPLETION OF EXCAVATION (STAGE 5) | | |
| | 80-110t | 148 | DERIVED FROM MEASUREMENTS PRIOR TO BACKFILLING (STAGE 6) | | |

\(\dagger\)Based on the use of Larssen 4/20 Piles
difficulty of achieving the precise design value in the field. For this reason the observations provide no direct evidence of the extent of any over-design in the calculated value of prop force. Twenty-four hours after prestressing the load measured on anchors 3, 5, 6 and 7 (Figure 18) corresponds to an average horizontal prop force of 153 kN/m. On completion of excavation in front of the wall this had increased to 157 kN/m. At this stage the strain measurements on the sheet piles indicated a maximum bending moment in the wall of about 75 kNm/m run which is substantially below the design value of 422 kNm/m run (Section 3.1). Five months later immediately prior to backfilling the average prop force had decreased to 148 kN/m while the maximum bending moment derived from the strain measurements had increased to about 110 kNm/m run.

To examine the sensitivity of the maximum bending moment and prop force to the assumed input parameters and method of design further calculations have been carried out and the key results are summarised in Table 4 together with the original design values (case 5) and those derived from the field measurements. As in the original design (Section 3.1) no allowance has been made for moment reduction when using the BSC method. In this context the measured pile deflections (Section 6.2.1) would correspond to a reduction in maximum bending moment of less than 10 per cent in uniform granular soil. The as constructed geometry is shown in Figure 26a and shows some small differences in the depth of excavation, anchor level and ground water conditions from the assumptions made in the original design (Figure 3). An assessment of the effect of these changes on the original design showed that they would increase the wall stability and reduce the maximum bending moment to 375 kNm/m run.

The additional site investigation and soil testing enabled a reappraisal of the soil profile and strength properties at the instrumented section. The amended profile is also shown in Figure 26a and was based on the composite borehole log and mean soil strengths determined from the laboratory tests (Figure 2). Since no additional test data were available for the top 3 m of made ground, the same strength properties were used as in the original design. The values of wall friction adopted for these subsequent calculations with the amended profile are consistent with the recommendations given in the British Steel Corporation Handbook (BSC, 1984). The results using the amended profile and strength data are given in the first case of Table 4 and show factors of safety similar to the original design although the values of maximum bending moment and prop force are significantly smaller. The distribution of bending moment down the wall for this case is shown in Figure 26b and indicates that the maximum moment occurred at a similar level to the measured value. The values of propping force and bending moments are sensitive to the magnitude of the limiting earth pressures assumed in design. This is illustrated in case 2 of Table 4 which shows the increased factors of safety and reduced maximum bending moment and prop force which results from an increase in wall friction acting on the back of the sheet piles. In this context the observations of ground movement indicated substantial differential movement between the sheet piles and retained ground (Figure 13) which suggests that high values of wall friction may well have been mobilised on the back of the wall.

The simplifying assumptions concerning the distribution of limiting earth pressures used in the design takes no account of the stiffness of the wall or of the propping system, although numerical studies (Potts and Fourie, 1985) and model tests (Rowe, 1952) have indicated that these can have a major influence on the magnitude of the bending moments and prop forces. For a relatively flexible and rigidly propped wall the effect of soil structure interaction is likely to result in enhanced earth pressures above the prop position with reduced pressures at lower levels where the greatest wall deflections occur. This leads to reduced bending moments compared with those calculated using the linear distributions of limiting earth pressure assumed in design.

Rowe’s method of design, which resulted from an extensive series of model tests, enables account to be taken of the relative stiffness of the wall and the propping system, through the application of bending moment and prop load factors. This design approach is described by Barden (1974) and calculations have been carried out for both the as constructed geometry and amended soil profile (Figure 26a and case 3 of Table 4) and the original design geometry and soil profile (Figure 3 and case 4 of Table 4). In these calculations the maximum recommended values of wall friction of 2/3 \( \phi' \) were assumed on both sides of the wall and the allowances for moment reduction relate to Larssen 4/20 sheet piles as used at the site. The method of calculating bending moments differs from that used in the original design in that factored values of passive pressure are used over the minimum required depth of embedment. For the amended soil profile and strength properties this design method gave a minimum pile length of 13.4 m compared with the 13 m length used at the site. A range of maximum bending moments is shown in Table 4 for each case; the lower values take account of moment reduction factors for fixity above prop level and below excavation level and the higher values for fixity below excavation level only. The values of prop force relate to an unyielding anchor for which a prop force enhancement factor of 1.5 has been employed.

From Table 4 Rowe’s method is seen to give smaller values of maximum bending moment and higher prop forces than the method used for the original design. Even with the maximum allowance for moment reduction together with high values of wall friction of 2/3 \( \phi' \), the calculated maximum bending moments for the sheet piles used at the site still exceed the
values derived from the measured pile strains. If the sheet pile section for the original design geometry (Figure 3) had been selected using Rowe's method, the range of design moments would have reduced from 190–240 kNm/m run (case 4 of Table 4) to 80–110 kNm/m run. This suggests that Larssen 2 or 10B/20 piles in grade 50B steel would have been acceptable. The economic advantage offered by the use of a lighter section could have been obtained if the piles had been driven in panels or if an installation technique such as predrilling, jetting or vibrodriving had been employed. However as indicated in Section 3, the selection of the Larssen 4/20 piles was governed by the particular driving technique used at the site to meet noise restrictions.

7.2 GROUND ANCHORS

The measurements obtained from the instrumentation indicated that the ground anchors supported the wall satisfactorily and that at no time was there any evidence of distress. The prestressed anchors behaved as a very stiff element in the temporary retaining structure. Excavation below waling level resulted in only very marginal increases in prop force and absolute movement of the waling towards the excavation was less than 5 mm, the majority of which can be attributed to an extension of the fixed anchor length. Following excavation the behaviour of the anchorage system was governed largely by an extension of the fixed anchor length which appeared to be associated with creep. This extension was accommodated mainly by a relaxation of the strand, leading to a small reduction in anchor load, rather than movement of the waling towards the excavation. The equivalent prop force only varied between 148 kN/m and 157 kN/m and remained close to the initial prestress value throughout the period of observation. The maximum individual anchor load was 550 kN with a maximum load on any one strand of 138 kN or 55 per cent of the strand’s characteristic strength. This easily satisfies the recommendations of the draft code (DD81, 1982) which suggests that the strand load should not exceed 62.5 per cent of the characteristic strength.

As indicated in the preceding Section, Rowe's method gives higher propping forces than the original design calculations if a prop load enhancement factor of 1.5 is assumed for a prestressed anchor (Barden, 1974). For the original design geometry and soil profile (case 4 of Table 4) this would have increased the anchor load by 140 kN to a design value of 700 kN. To accommodate this additional load while maintaining the required factor of safety against failure of the steel or ground would have necessitated the inclusion of an extra prestressing strand together with an increase in the effective grout to ground contact area of about 25 per cent. It was not possible to attempt to fail a working ground anchor on this site and, therefore, an assessment of the actual factor of safety against failure of the ground around the anchor could not be made. However, following excavation, the distribution of strain along the fixed anchor length, which showed that the load was effectively transferred to the ground, remained almost unchanged. There was therefore no indications of any distress of the anchor and it would seem reasonable to assume that there was an adequate factor of safety against failure in this mode with no evidence from the field observations that anchors of increased load capacity were required at this site.
7.3 COMPARISON OF GROUND SETTLEMENT

The total settlements of the ground surface measured behind the sheet piles on completion of excavation to foundation level and immediately prior to backfilling are shown plotted in dimensionless form in Figure 27. Also shown on this figure are the zones suggested by Peck (1969) based on field data from deep excavations in several types of material. As indicated in Section 6.1 the major part of these settlements occurred as a result of the pile driving operation. The magnitude of these settlements are nevertheless small with the maximum lying close to the value of 0.5 per cent of the depth of cut suggested by Terzaghi and Peck (1967) for loose sands and gravels where the groundwater is adequately controlled. Similar small settlements have also been observed in studies of ground movements caused by deep trenching in granular soils where the ground has not been subjected to trafficking by heavy construction plant (Toombs, McCaul and Symons, 1982).

Smaller movements of the retained ground were measured as deflections of the sheet piles took place during the initial excavation, anchor installation and tensioning, and bulk excavation in front of the wall. On completion of excavation, the measured settlements of the ground surface behind the wall fell well within the zone suggested by Peck (1969). Further movements of the retained ground close to the wall of up to 25 mm were then recorded when the anchor cables were cut after filling in front of the piles to waling level.

(2) The wall deflected as a free cantilever during the initial excavation to about 3 m depth with small outward movements of 3.5 mm measured near the top of the piles. A reversal of these movements then took place during installation and tensioning of the ground anchors. The anchors served to limit movements of the piles above waling level during the subsequent stages of construction. During bulk excavation in front of the wall outward deflection of the piles occurred with a maximum movement measured at about the midpoint between excavation and anchor level. Wall deflections in this mode continued over the period until commencement of filling to give a maximum midpoint movement of about 15 mm. Larger outward deflections of the tops of the piles were then measured after cutting of the anchor cables. No significant vertical movements of the piles were observed during construction and the settlements measured in the retained ground suggest that some wall friction was mobilised during the main part of the excavation.

(3) The bending moments determined from the measurements of pile strain generally reflected the pattern of wall deformations. Thus a reversal in bending moment occurred just below anchor level as a consequence of anchor stressing and the magnitude and depth of the maximum bending moment increased during bulk excavation in front of the wall. Over the five month period between completion of excavation and commencement of backfilling behind the permanent works, the measurements indicate an increase in maximum bending moment of about 45 per cent to a value of about 110 kNm/m.

(4) Measurement of anchor load showed a small decrease immediately following anchor stressing which is attributed to a redistribution of load in the walings as adjacent anchors were stressed. This was followed by an increase in anchor load in response to further excavation in front of the wall together with a small decrease in load during pauses in excavation. The net effect was that on completion of excavation the average load in the anchors was within 1 per cent of the 525 kN load applied during stressing. By commencement of filling in front of the piles the anchor load had decreased by approximately 6 per cent. Measurements indicated an ongoing extension of the fixed anchor length during pauses in excavation and following completion of excavation to foundation level, suggesting that the reductions in anchor loads are a result of creep of the fixed anchor length.

8 SUMMARY AND CONCLUSIONS

(1) Ground movements of about 50 mm were measured close to the 13 m long Larssen piles during driving. The pattern of these movements suggest that densification of the granular soils had occurred close to the piles as a result of vibration and downdrag.
generally reliable over the 12 month period of the study. The trial drive formed an essential component in establishing a viable method of fixing and protecting the inclinometer tubes and strain gauges on the sheet piles so as to minimise damage sustained during the driving operations.

(8) The study provided useful data on the behaviour of the wall during construction and in service and illustrated the effect of soil-structure interaction on the magnitude of the bending moments. Further studies of this type should be undertaken to provide information on the pile strains developed during driving and to examine the adequacy of existing design methods for sheet pile walls in clay (Burland et al, 1981). In this context the value of such studies could be greatly enhanced if local failures could be initiated by over-excavation in front of the wall or by reducing the available propping force.

9 ACKNOWLEDGEMENTS

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10 REFERENCES


APPENDIX A. GROUND ANCHOR INSTRUMENTATION

A fully instrumented ground anchor is shown diagrammatically in Figure 28. The instrumentation was designed to monitor the anchor load, changes in the free anchor length, and the behaviour of the fixed anchor length.

A1 ANCHOR LOAD—LOAD CELL

The anchor load was monitored by a load cell inserted between a bearing plate and the stressing head. The cell was manufactured from a cylinder of EN24T steel with eight 350 ohm electrical resistance strain gauges arranged so as to give a full bridge configuration. The output from the cell was read by a portable transducer indicator which included reference circuitry to correct any zero and span drift. A calibration curve provided with the cell converted the indicated reading to anchor load in kN.

A2 FREE ANCHOR LENGTH—SINGLE POINT EXTENSOMETER

A single point extensometer was installed to measure changes in the free anchor length. The instrument consisted of a tensioned wire anchored at its lower end to the top of the fixed anchor length and wound around a tensioning drum, which was free to rotate, at the head. The wire was centralised within a PVC tube by ‘knife edge’ spacers located at 3 m intervals. A counterweight maintained tension in the wire and any change in the free anchor length caused the tensioning drum to rotate. An optical rotary encoder connected to the drum’s shaft measured this rotation with the output being scaled to give a direct indication of changes in the free anchor length. The instrument had a resolution of 0.1 mm.

A3 FIXED ANCHOR LENGTH—SONIC PROBE EXTENSOMETER

The sonic probe extensometer system is based upon the measurement of distances between magnets positioned along a probe. The measurements are made by timing the interval required for a stress wave to travel between two or more magnets located close to a probe manufactured from a tube of magnetostrictive material. A current pulse passed through the probe from a readout unit creates an instantaneous magnetic field along the length of the probe which interacts with the fields around the individual magnets. The resulting stress waves generated at each magnet position travel at a known velocity.
towards a sensor at the probe head. By timing the arrival of the second and subsequent waves relative to the first the readout unit computes the distance between the magnets. The measurement is displayed on a liquid crystal display and was found to have an effective resolution of 0.06 mm. It was therefore decided that strains should not be computed for magnet assemblies positioned closer than 600 mm apart. The accuracy of the strains reported is therefore ±100 microstrain at the head of the fixed anchor improving to ±30 microstrain as the magnet spacing increased towards the lower end. Ten individual magnet positions can be measured by a single probe up to 8 m in length.

The instrument was selected because only minor modifications to the standard anchor were required and, furthermore, it could be easily accommodated within the 75 mm internal diameter of the temporary casing. Another desirable feature is that the signals between the readout and the head of the sonic probe are digital rather than analogue and are, therefore, less prone to external interference or factors effecting the impedance of long lead wires such as temperature.

The probe and head were encapsulated in brass which provided additional protection against damage during the installation and grouting operations. The magnet assemblies comprised four Alinco 8 magnets (6 mm diameter and 25 mm long) mounted in PVC and arranged so that the magnets surrounded the probe on a 26 mm pitch circle diameter. A total of ten magnet assemblies were positioned along the 8 m long probe. Details of the sonic probe and magnet assemblies together with the positions of the magnets along the fixed anchor length are shown in Figure 29.

The probe and magnet assemblies were attached to the stressing strands prior to homing of the anchor. Measurements obtained from the instrument were used to determine changes in the fixed anchor length and the distribution of axial strain in the grout along its length. The distribution of strain was determined by computing the average strain at the midpoint between two magnet assemblies. As it was anticipated that the greatest axial strain would be at the top of the fixed anchor length and decrease with distance as load is transferred to the ground, smaller spacings between the magnets were used towards the top of the fixed anchor length.

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**Fig. 29** Instrumentation of fixed anchor length