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BEHAVIOUR DURING CONSTRUCTION OF A PROPPED DIAPHRAGM WALL FOUNDED IN LONDON CLAY AT ALDERSHOT ROAD UNDERPASS

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This report describes work commissioned by the Bridges Engineering Division of the Highways Agency under E467A/BG. Behaviour of Diaphragm Retaining Structures during Construction.

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CONTENTS

EXECUTIVE SUMMARY

New roads through urban areas are increasingly making use of underpasses to form grade separated interchanges with existing roads. The principal structural feature of an underpass involves the installation of embedded retaining walls constructed from existing ground level using either bored pile or diaphragm wall techniques to minimise landtake. Embedded walls can be free standing and act as cantilevers, or supported by anchoring back into the retained ground or by provision of structural props. For underpasses, support is often provided by a structural slab spanning between the walls on each side and cast just below carriageway level. The design of such structures is not straightforward and currently available advice such as that given by CIRIA 104 (Padfield and Mair, 1984) does not cover this support category. The structural forces in the prop slab can be affected by the nature of the wall connection, the magnitude of the swelling pressures (particularly if the slab is underlain by overconsolidated clay), and the efficiency of the under-slab drainage.

This report describes the field instrumentation and monitoring carried out to establish the behaviour of a T-shaped diaphragm wall founded in overconsolidated clay during its construction as part of the Aldershot Road underpass. The construction sequence involved excavation below two levels of temporary props and casting of a permanent prop slab below carriageway level. At this site, the potential for uplift from groundwater and long term swelling pressures in the clay made the permanent prop slab a particularly critical feature of the design. For this reason, the slab was designed as a shallow V-shape with hinges incorporated both at the wall connections and in the centre of the slab. This design was aimed at minimising moments in the slab whilst restricting the heave. Measurements of ground movements, wall movements and bending moments, temporary and permanent prop loads, and heave pressures and porewater pressures below the carriageway slab were made during the various construction stages.

Overall wall and ground movements were within the upper bound of 0.2% of the excavation depth proposed by Carder (1995) on the basis of measurements at four other sites where underpass walls were propped at carriageway level. On completion of construction, overall heaves of about 45mm were recorded at depths of a few metres below final excavation level. Measurements from pressure cells in the drainage blanket below the prop slab currently indicate that vertical stresses are no more than expected from overburden calculations, although further monitoring is continuing to evaluate longer term effects.

The project was a collaborative study with the University of Surrey (funded by EPSRC) and Surrey County Council.

BEHAVIOUR DURING CONSTRUCTION OF A PROPPED DIAPHRAGM WALL FOUNDED IN LONDON CLAY AT ALDERSHOT ROAD UNDERPASS

ABSTRACT

Field instrumentation has been installed to monitor the behaviour of a T-shaped diaphragm retaining wall founded in overconsolidated clay during and immediately after its construction as part of the Aldershot Road underpass. The construction sequence involved installation of the wall under bentonite followed by excavation below two levels of temporary props and casting of a permanent reinforced concrete prop slab below the final carriageway level. The permanent prop slab was designed to restrict long term swelling pressures of the clay and constructed as a shallow V-shape with hinges at each wall and in the centre.

Measurements of ground movements, wall movements and bending moments, temporary and permanent prop loads, and heave pressures and porewater pressures below the carriageway slab were made during the various construction stages.

1. INTRODUCTION

New roads through urban areas are increasingly making use of underpasses to form grade separated interchanges with existing roads. The principal structural feature of an underpass involves the installation of embedded retaining walls constructed from existing ground level using either bored pile or diaphragm wall techniques to minimise landtake. Embedded walls can be free standing and act as cantilevers, or supported by anchoring back into the retained ground or by provision of structural props. For underpasses, support is often provided by a structural slab spanning between the opposing walls and cast just below carriageway level. The design of such structures is not straightforward and currently available advice such as that given by CIRIA 104 (Padfield and Mair, 1984) does not cover this support category. The structural forces in the prop slab can be affected by the nature of the wall connection (Powrie and Li, 1991; Potts, 1992), the magnitude of the swelling pressures (particularly if the slab is underlain by overconsolidated clay), and the efficiency of the under-slab drainage.

A collaborative study has been undertaken by the Transport Research Laboratory, the University of Surrey, and Surrey County Council to monitor the performance during and immediately after the construction of a diaphragm wall singly propped at carriageway level for Aldershot Road underpass. The soil profile consists of approximately 4m of made ground and river gravels overlying London Clay. Owing to the proximity of the River Blackwater, local groundwater levels are high. The design therefore employed very stiff T-shaped wall panels with substantial temporary and permanent propping. The potential for uplift from groundwater and long term swelling pressures in the clay made the permanent prop slab a particularly critical feature of the design. For this reason, the slab was designed as a shallow V-shape with hinges incorporated both at the wall connections and in the centre of the slab. This design was aimed at minimising moments in the slab whilst restricting the heave.

Field instrumentation was installed prior to the start of construction to determine lateral subsurface movements of the retained ground together with ground heave and porewater pressures below the new carriageway. During wall installation, instruments were incorporated in the diaphragm panels to determine wall movement and bending moments developed at the various construction stages. Loads developed in the temporary and permanent props were also monitored together with the heave pressures induced in the ground below the carriageway prop.

The implications of the findings on the design of future structures of a similar type are discussed.

2. SITE DESCRIPTION

The diaphragm wall being investigated forms part of the underpass at the junction of the A323 Aldershot Road with the A331 Blackwater Valley Route. The instrumented area of wall is located within the central roundabout area of the underpass at a contract chainage of about 4385 as shown in Fig 1. The research area is also known as instrumented area 6 to distinguish it from areas where field monitoring was carried out to control the construction procedure under the Main Contract.

The underpass at the instrumented section is formed from T-shaped diaphragm wall panels with an overall penetration of 14.6m and the panels are capped with a 4m high reinforced concrete capping beam. The instrumented area is situated at the deepest part of the underpass where the final retained wall height is about 10m.

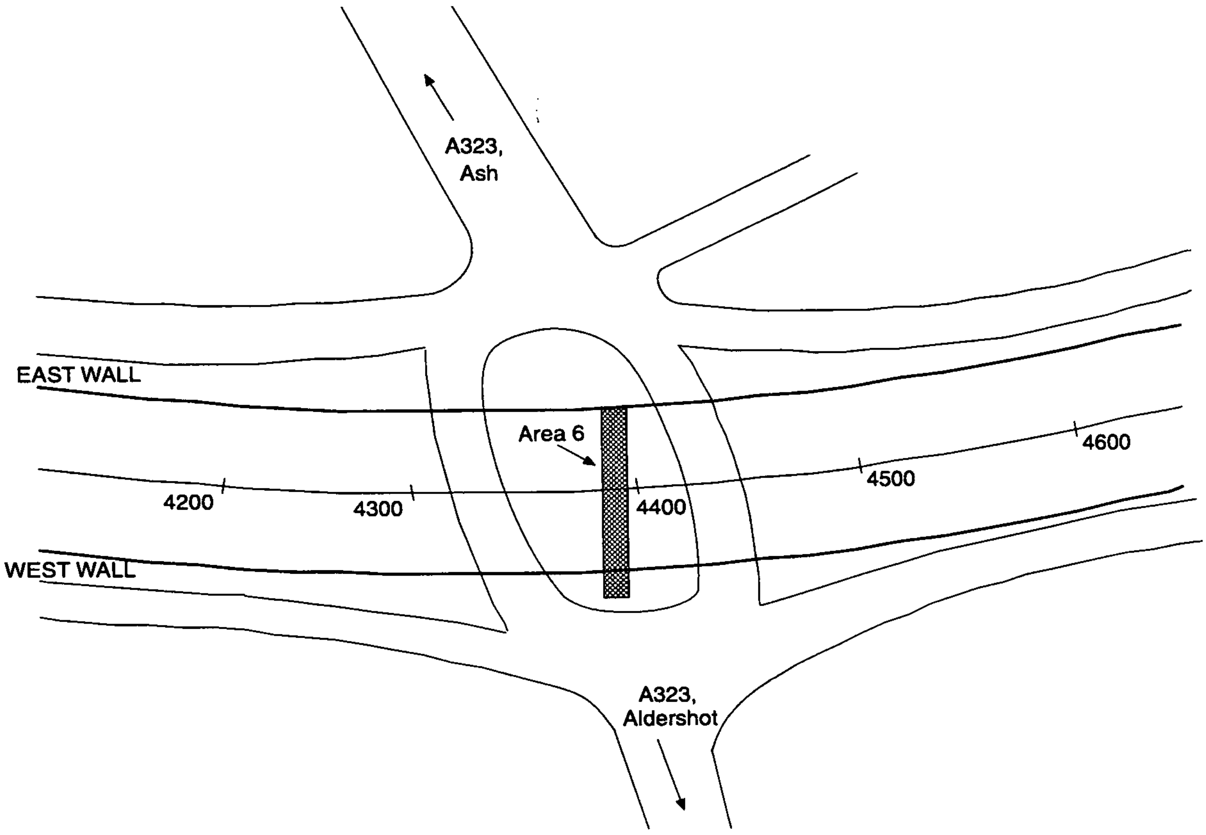


Fig 1. Location of the site

3. SOIL PROPERTIES

3.1 SOIL PROFILE AND PLASTICITY DATA

Three site investigations were carried out for Surrey County Council between April 1987 and December 1992 for the Aldershot Road Underpass by Foundation and Exploration Services Ltd. The borehole logs in the vicinity of the instrumented area indicate that the ground conditions consist of approximately 2m of made ground and silty sandy clay overlying a 2m band of sand and gravel, which overlies the London Clay to depth. Fig 2 shows the soil profile for borehole BH246.

The soil plasticity data established during the investigations are also shown in Fig 2. The plastic limit of the clay was fairly constant with depth around a mean moisture content of 27%. The natural moisture content was about 29% near the top of the London Clay falling to 24% at 20m depth. The majority of the liquid limits were in the range 60-70% between depths of 3m and 28m, with some values between 70-80%. Lower liquid limits of between 30-50% were measured within the upper sandy silty clay.

3.2 LABORATORY STRENGTH TESTS

Fig 2 shows the variation of undrained shear strength with depth. The strength values were determined from quick undrained triaxial compression tests on 38mm diameter specimens as part of the site investigations. Some scatter in the results was obtained, but generally the strength showed an increase with depth as indicated by the best fit line in Fig 2.

A summary of peak strength results from consolidated undrained triaxial compression tests on 38mm diameter specimens extruded from undisturbed samples of London Clay is shown in Fig 3. This shows that the overall best fit parameters of $c' = 19 kN/m^2$ and $\phi' = 24.5°$ and lower bound parameters of $c' = 0$ and $\phi' = 24.5°$ are obtained at mean effective stress levels up to $365 kN/m^2$.

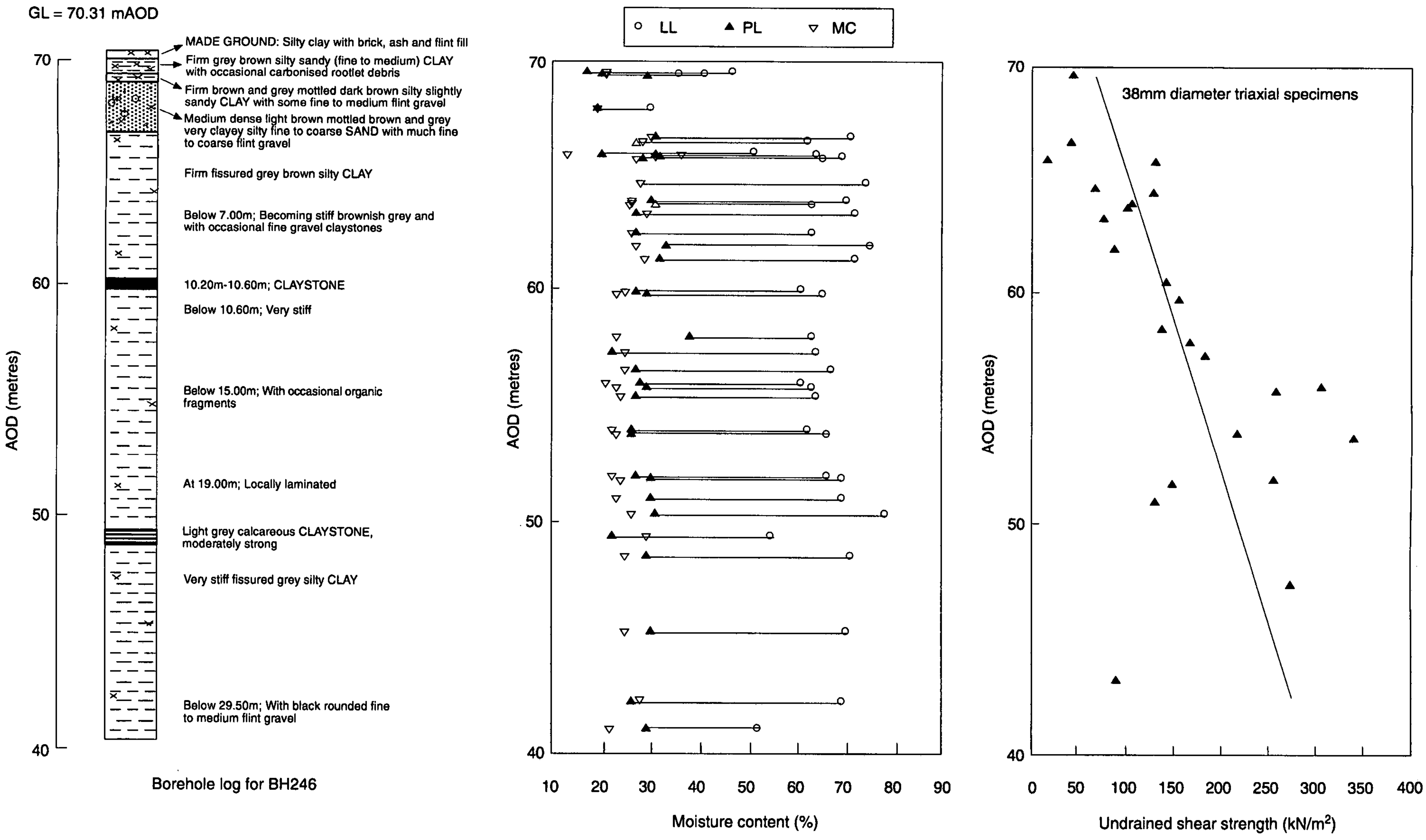


Fig 2. Soil properties

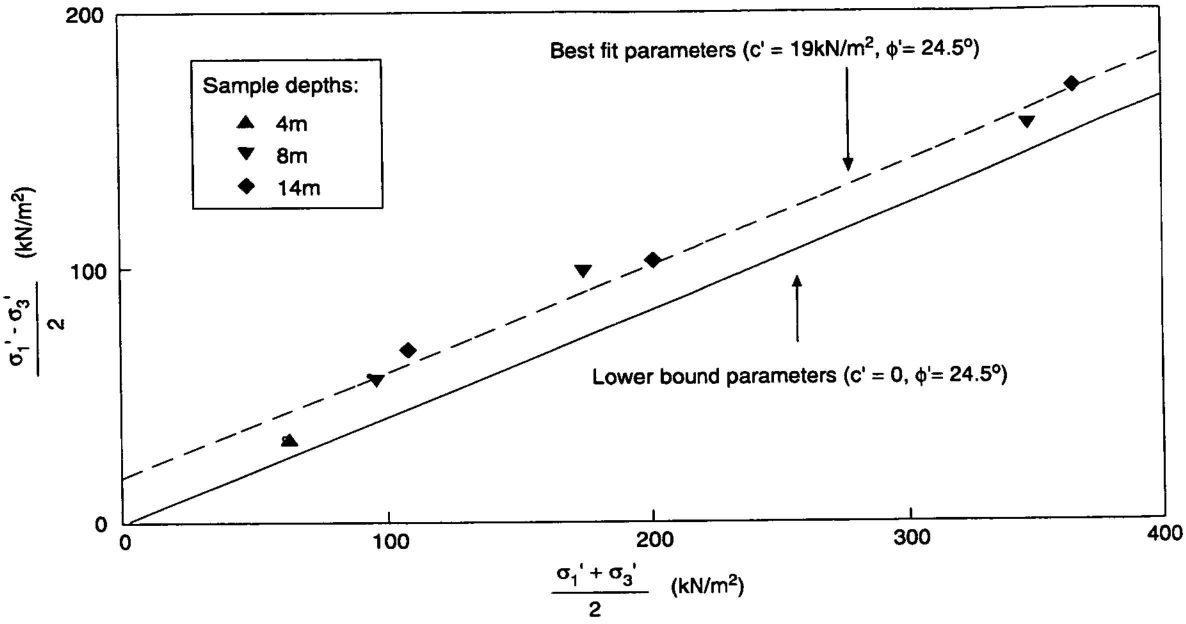


Fig 3. Peak strength parameters of the clay from laboratory triaxial tests

4. DETAILS OF WALL AND UNDERPASS CONSTRUCTION

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

Stage 1 of construction was the placing of 1m of fill followed by the installation of the T-shaped diaphragm wall panels. Each panel in the instrumented area consisted of a 4.5m by 1.2m front section with a 3.3m by 1.2m counterfort. Excavation for each panel was carried out using a mechanical grab operating between concrete guide walls (Fig 4a). Throughout the period when the panel excavation was open, a bentonite slurry was used to provide support. Following completion of excavation, the reinforcing cage was lowered into the excavation (Fig 4b) and the concrete placed using a tremie. The dates of the excavation and concreting of each panel in the instrumented area are given in Table 2. Excavation generally took about 2 days with concreting occurring on the following day. Surface ground conditions were poor during this period because of plant trafficking and some bentonite spillage.

After completion of wall installation, the diaphragm panels were then reduced to their cut-off levels of 68.6mAOD and a 4m high reinforced concrete capping beam constructed on top of the panels (Stage 2). While this work was under way, excavation to a depth of about 3m below original ground level in front of the wall was taking place to provide access for installation of the upper levels of temporary props. The 30m long temporary props spanned the underpass and consisted of 1.2m diameter x 11mm thick steel tubes at 4.5m centres bearing on the capping beam at 69.1mAOD, i.e. about 0.5m above diaphragm panel cut-off levels. Installation of the props was carried out by lowering them onto support brackets and concrete packers were then cast between the prop end plates and the wall. This was carried out prior to backfilling behind the capping beam (Stage 6).

Immediately after backfilling was complete, further excavation was carried out to about 7m depth (63.6mAOD) so that the lower level of props could be installed. These props were packed out with concrete between end plates and steel walings which were fixed to the diaphragm walls. Lower props were installed at a similar spacing of 4.5m between centres and at 64.3mAOD, i.e. a depth of 4.3m below panel cut-off levels (Stage 8).

Bulk excavation in front of the walls was carried out using tracked loading shovels and dozers with the spoil being transported away by lorry. This excavation in the instrumented area was completed to a final depth of 10.5m (60mAOD) in the centre of the underpass during August 1995 (Stage 9). A 200mm thick sand filter layer was then placed on the excavated clay surface. This filter layer was covered with a geocomposite drainage sheet system (about 10mm thick) and steel mesh reinforced blinding concrete placed in preparation for installing the reinforcing cage for the permanent prop slab (Fig 5a).

The reinforced concrete prop slab was 700mm thick with a shallow V-shaped profile to restrict the uplift due to heave (Fig 5b). Hinges were formed at the joints between the slab and each wall and also at the centre of the underpass to reduce bending moments in the slab. A neoprene rubber lining was placed between the concrete bearing surfaces at each hinge.

TABLE 1

Underpass construction sequence at instrumented area 6

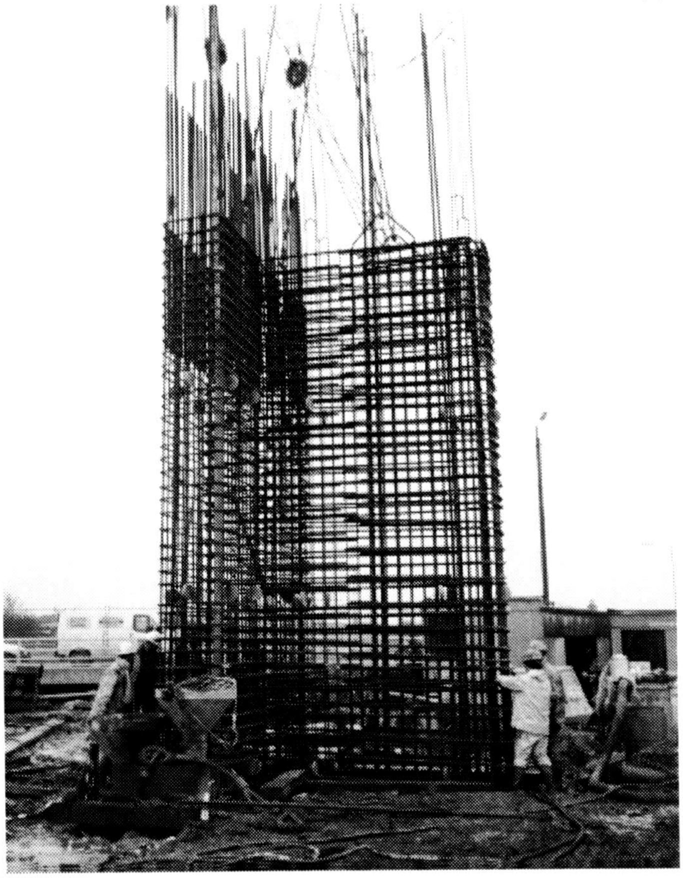
Stage	Construction operation	Period
1	Installation of diaphragm wall panels under bentonite	14/10/94 - 14/2/95
2	Reduce ground level to capping beam formation level of 68.6mAOD (1.9m below OGL). Construct 4m high reinforced concrete capping beam	15/2/95 - 3/6/95
3	Excavate in front of the wall to 67.5mAOD (3m below OGL)	7/6/95 - 9/6/95
4	Excavate central haul road to 63.5mAOD (7m below OGL)	18/7/95 - 25/7/95
5	Upper temporary props concreted in	29/7/95
6	Backfill placed behind capping beam to 71.7mAOD	2/8/95
7	Excavate in front of the wall to 63.6mAOD (6.9m below OGL) with central haul road to 60mAOD	7/8/95 - 10/8/95
8	Lower temporary props concreted in	22/8/95
9	Complete bulk excavation to formation level, minimum 60mAOD (10.5m below OGL)	25/8/95
10	Reinforced concrete prop slab installed in area 6 west	15/11/95
11	Reinforced concrete prop slab installed in area 6 east	13/12/95
12	Lower temporary props removed	15/12/95
13	Upper temporary props removed	15/1/96
14	Backfill placed behind capping beam to 72.4mAOD.	
	Carriageway construction completed	15/7/96
15	Road opened to traffic	19/7/96

TABLE 2

Diaphragm wall panel installation in instrumented area 6

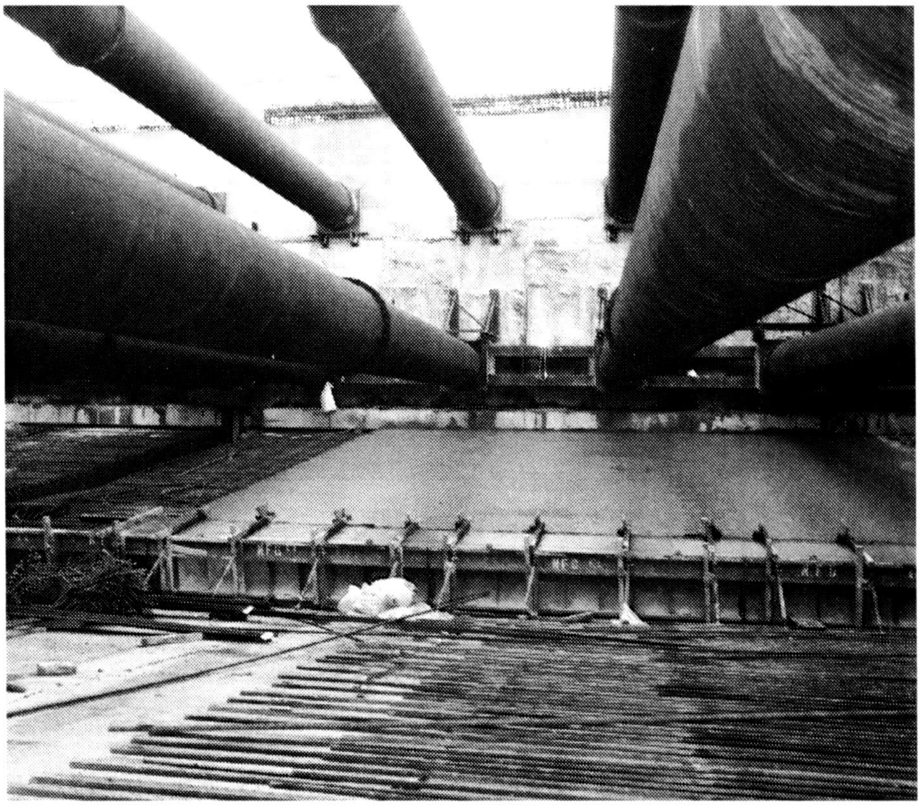
Panel number	Operation	Period
W33	Excavation	9/2/95 - 14/2/95
	Concreting	14/2/95
W34	Excavation	1/2/95 - 2/2/95
	Concreting	3/2/95
W35	Excavation	25/1/95 - 26/1/95
	Concreting	27/1/95
W36	Excavation	14/11/94 - 16/11/94
	Concreting	17/11/94
W37	Excavation	26/10/94 - 27/10/94
	Concreting	27/10/94

(a) Panel excavation

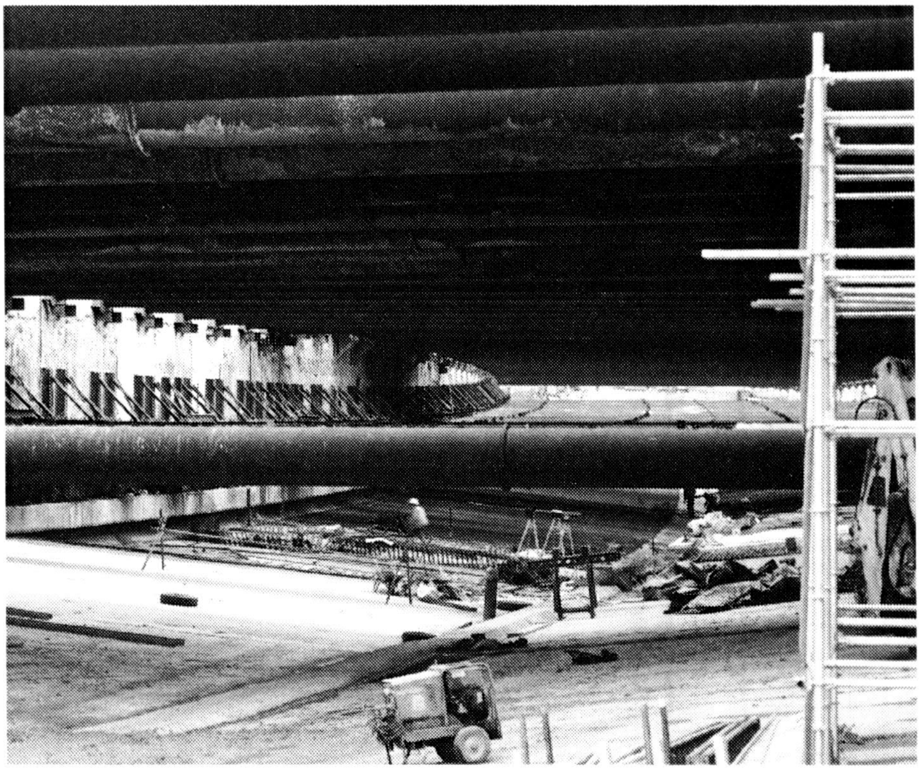


(b) Installation of reinforcing cage

Fig 4. Diaphragm wall installation



(a) Casting the prop slab in bays



(b) View showing the V-shaped profile

Fig 5. Permanent prop construction

After construction of the permanent prop slab, the temporary props were removed, and the final road surface constructed (Fig 6). The underpass was opened to traffic in July 1996.

5. FIELD INSTRUMENTATION

The layout of the field instrumentation to monitor the behaviour during installation of the diaphragm wall and construction of the underpass is shown in section and plan in Figs 7 and 8 respectively. All instrumentation in the ground was installed in early October 1994 in advance of any construction work in the research area. Instruments in the wall panels and permanent prop slab were installed at the appropriate stage during the construction works.

5.1 GROUND MOVEMENTS

Subsurface lateral movements of the ground on the retained side of the wall were monitored using inclinometer tubes I1 and I2 installed at distances of 1.5m and 4m behind the counterfort of diaphragm wall panel W35 (Fig 7). The tubes were grouted into boreholes sunk to a depth of 21m using a cable tool percussion drilling rig. This depth was such that the bottom of each tube was about 4m below the toe of the retaining wall and founded in the stiff underlying London Clay. Inclinometer surveys during wall installation and underpass construction were then used to calculate movements assuming base fixity of the inclinometer tubes.

The drilling rig was also used to install three magnetic ring extensometer systems (MR9 to MR11 in Fig 8a) in the area to be excavated to measure subsurface heave beneath the permanent prop slab. One of these was located at a distance of 6.3m in front of the edge beam of the west wall, another at the same distance in front of the east wall, and the third towards the centre of the underpass. The third system was damaged during construction and subsequently reinstated at contract chainage 4367m, approximately 18m north of the main instrumented area. During bulk excavation the access tubes to these instruments were progressively removed. From precise levelling on the tops of the access tubes and the readings on the magnetic rings, the magnitude of heave caused by the excavation was calculated at the various ring depths indicated in Fig 7.

5.2 WALL MOVEMENTS AND BENDING MOMENTS

During installation of the diaphragm wall, steel tubes of nominally 100mm diameter were cast vertically into both the west and east wall in the research area. When wall installation was complete, inclinometer access tubes (W6 and E6 in Fig 7) were grouted into the steel tubes using a deep tremie to ensure that the cement grout (containing a non-shrink additive) reached the bottom of the steel tube. During construction, an inclinometer torpedo was used to carry out surveys of the lateral deflected shape of the walls. Absolute wall movements were obtained by correlating the inclinometer results with measurements of change in underpass span made using a high precision electronic distance measuring system (Geomensor). For the purpose of these calculations, the movements of the top of the west and east walls were assumed to be identical.

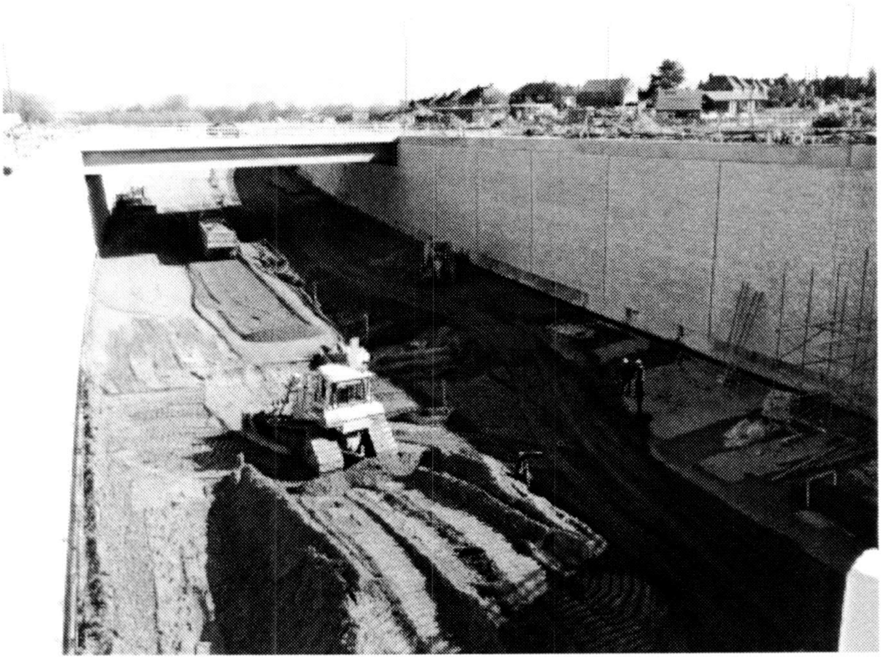


Fig 6. Cariageway construction

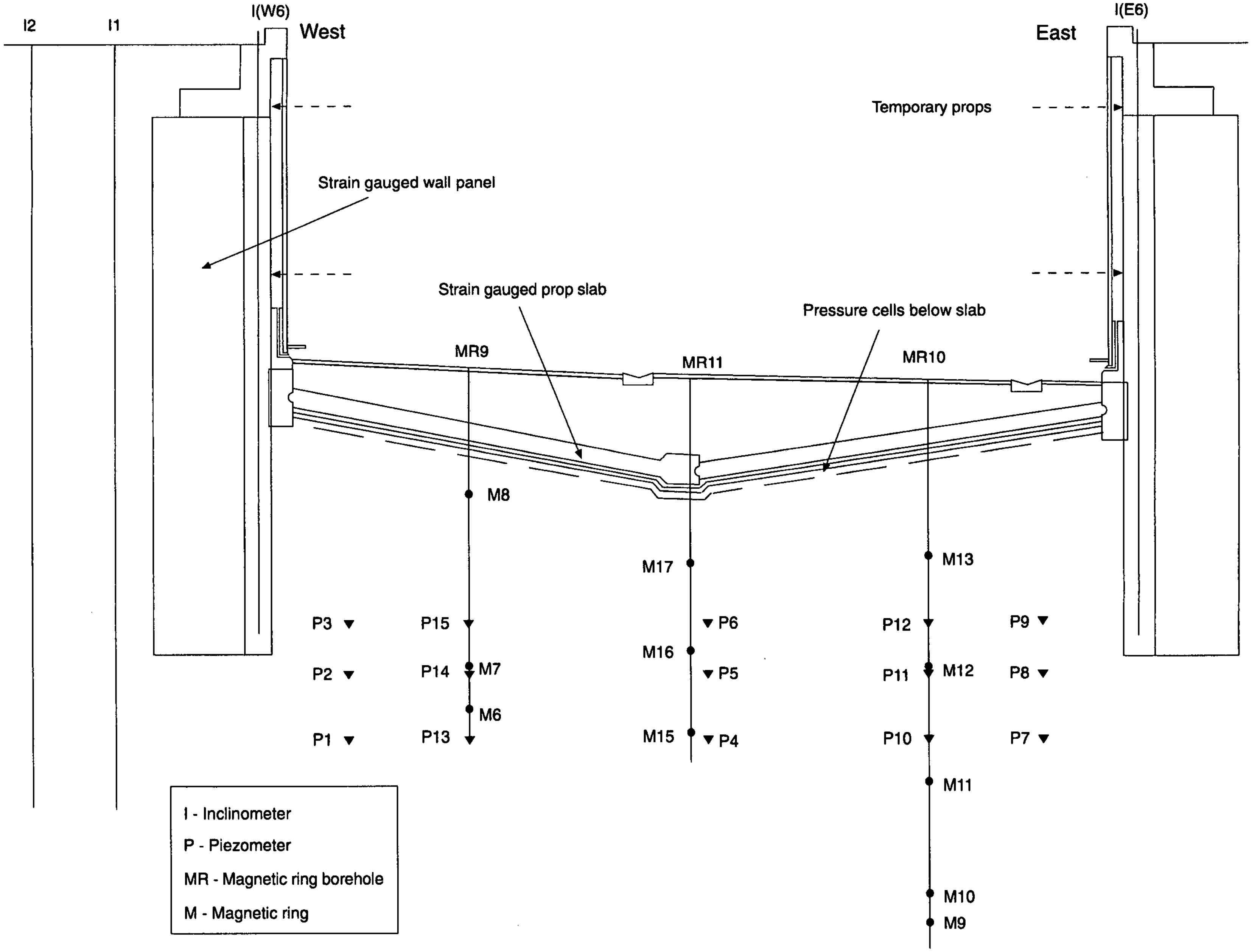
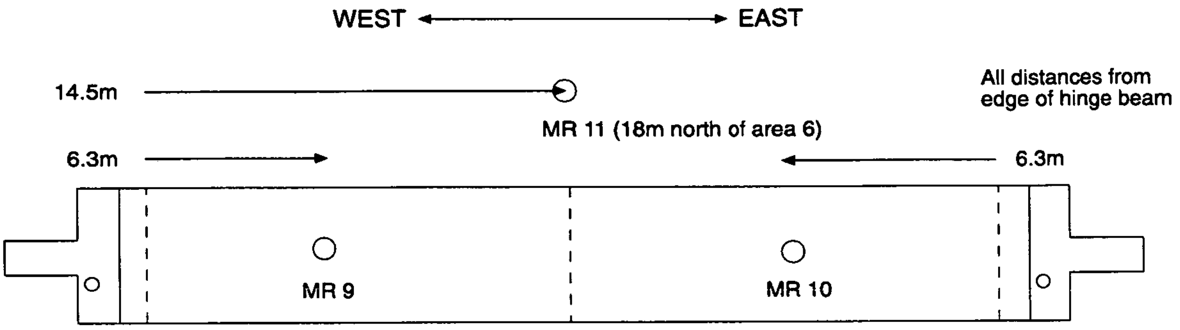
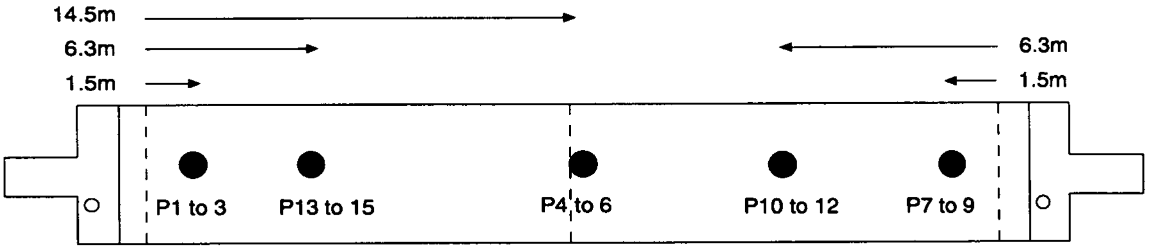


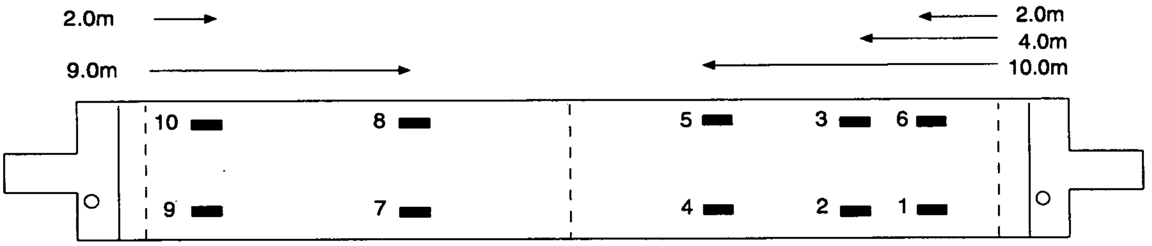
Fig.7 Schematic section in instrumented area 6



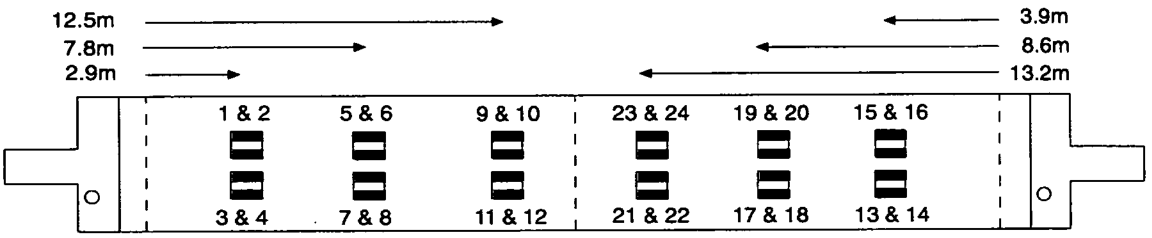
(a) Magnet settlement rings



(b) Piezometers



(c) Pressure cells



* Gauges in pairs

(d) Strain gauges

Fig.8 Schematic plan of instrumented area 6

The bending moments in diaphragm wall panel W35 were determined using vibrating wire strain gauges. For this purpose, twelve pairs of gauges were clamped to the vertical reinforcement bars so that one gauge of each pair was positioned at the back of the panel counterfort and one towards the front. An additional two strain gauges were installed at permanent prop level and 230mm behind the front gauges in case these gauges were damaged during construction of the edge beam for the permanent prop. The longitudinal distance between pairs of gauges varied between 1m and 2m with the gauges being more frequent in the region of the permanent prop slab where the maximum bending moment was expected to occur. Wall bending moments were calculated from the strains measured on the gauge pairs at each depth based on a calculated flexural rigidity (EI) for each T-panel of 4.53 x 10^5 MNm^2, assuming that the concrete would remain uncracked at the small strain levels involved.

5.3 TEMPORARY AND PERMANENT PROP LOADS

Four vibrating wire strain gauges were fitted to both the upper and lower level steel temporary props supporting wall panel W34 in the instrumented research area. The gauges were secured between pairs of steel blocks welded equidistantly around the prop circumference and were protected from direct sunlight by aluminium covers. Each gauge incorporated a thermistor for monitoring temperature changes.

Axial loads and bending moments in the permanent prop slab below the carriageway were monitored using twenty-four vibrating wire strain gauges. These were installed in pairs on the top and bottom of the reinforcing cage at distances of 2.9m, 7.8m and 12.5m from the west wall and at distances of 3.9m, 8.6m and 13.2m from the east wall. The locations of the gauge pairs are shown in Fig 8. Each gauge incorporated a thermistor for temperature measurement.

5.4 EARTH AND POREWATER PRESSURES BELOW THE CARRIAGEWAY PROP SLAB

Piezometers were installed in five boreholes sunk in the area to be excavated to measure porewater pressures developed beneath the permanent prop slab. Each borehole accommodated three pneumatic piezometers with high air entry tips located at approximate depths of 15.5m, 17.5m and 20m below original ground level. Each tip was encased in a nominal 100mm long sand cell with the remainder of the borehole sealed with bentonite pellets. The various borehole locations are shown in Fig 8.

The measurement of pressures caused by heave associated with swelling of the clay beneath the permanent prop slab was carried out using ten vibrating wire pressure cells. These cells were installed in the sand filter layer immediately beneath the reinforced concrete slab. Pairs of cells were installed at 2m and 9m from the edge beam of the west wall and at 2m, 4m and 10m from the edge beam of the east wall as indicated in Fig 8.

6. OBSERVATIONS

6.1 DIAPHRAGM WALL INSTALLATION

Measurements of ground subsurface movement using inclinometer tubes I1 and I2 positioned at 1.5m and 4m behind the counterfort of diaphragm panel W35 are shown in Fig 9. Lateral movements at the surface of 6.2mm and 5.5mm were recorded on I1 and I2 respectively on completion of wall installation, with movements generally decreasing with depth. In Fig 9 slightly less movement was recorded between about 65mAOD and 70mAOD on tube I1, which was closer to the construction and may have been disturbed by the construction plant.

6.2 UNDERPASS CONSTRUCTION

6.2.1 Wall and ground movements

The development of lateral movement at the top of the wall during the various stages of underpass construction is shown in Fig 10. Movements determined by halving the changes of underpass span measured using the Geomensor are compared with those recorded by inclinometer surveys assuming base fixity of the wall. The difference between the two sets of results provides an estimate of the lateral movement of the wall toe. Generally, up until the start of excavation to 6.9m depth (Stage 7 in Table 1), movements of the top of the wall measured using the Geomensor were no more than 5mm towards the excavation whilst the inclinometers recorded up to 3mm. This indicated that toe movements of the wall were of the order of 2mm. During this period, some fluctuations in the results were observed and these were related to thermal expansion and contraction of the upper level of steel props. This effect is discussed in more detail in Section 6.2.4.

The results in Fig 10 also indicate that the lateral movement of the wall toe towards the excavation increased to about 5mm during excavation to 6.9m depth. Installation of the lower temporary props was effective in minimising further toe movements during the following stages of construction. Lateral movement of the top of the wall increased to about 7mm after completion of bulk excavation and a further 4mm occurred over the following 2 months whilst the permanent prop beneath the carriageway was being installed. Movements of the top of the wall increased to 16mm immediately after temporary prop removal and reached 20mm one month later.

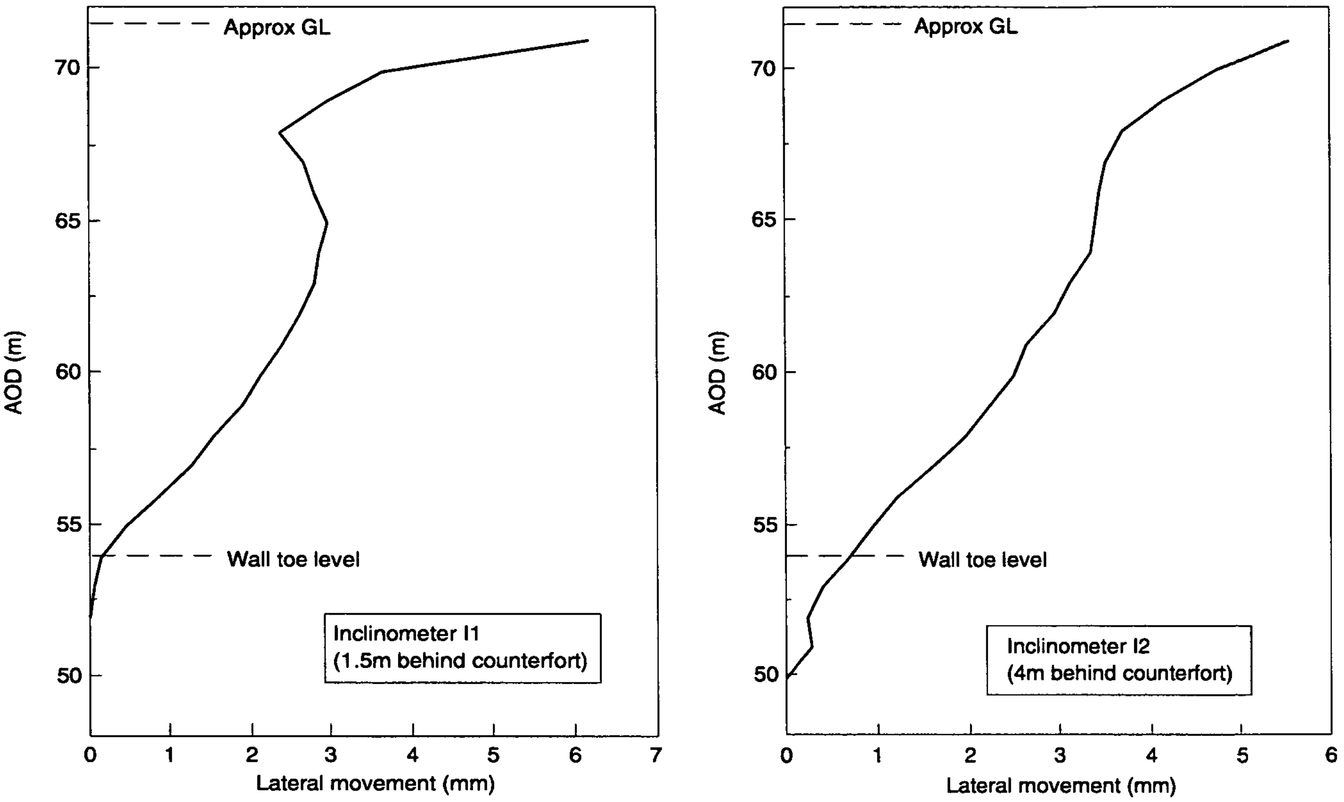


Fig.9 Subsurface ground movements during diaphragm wall installation

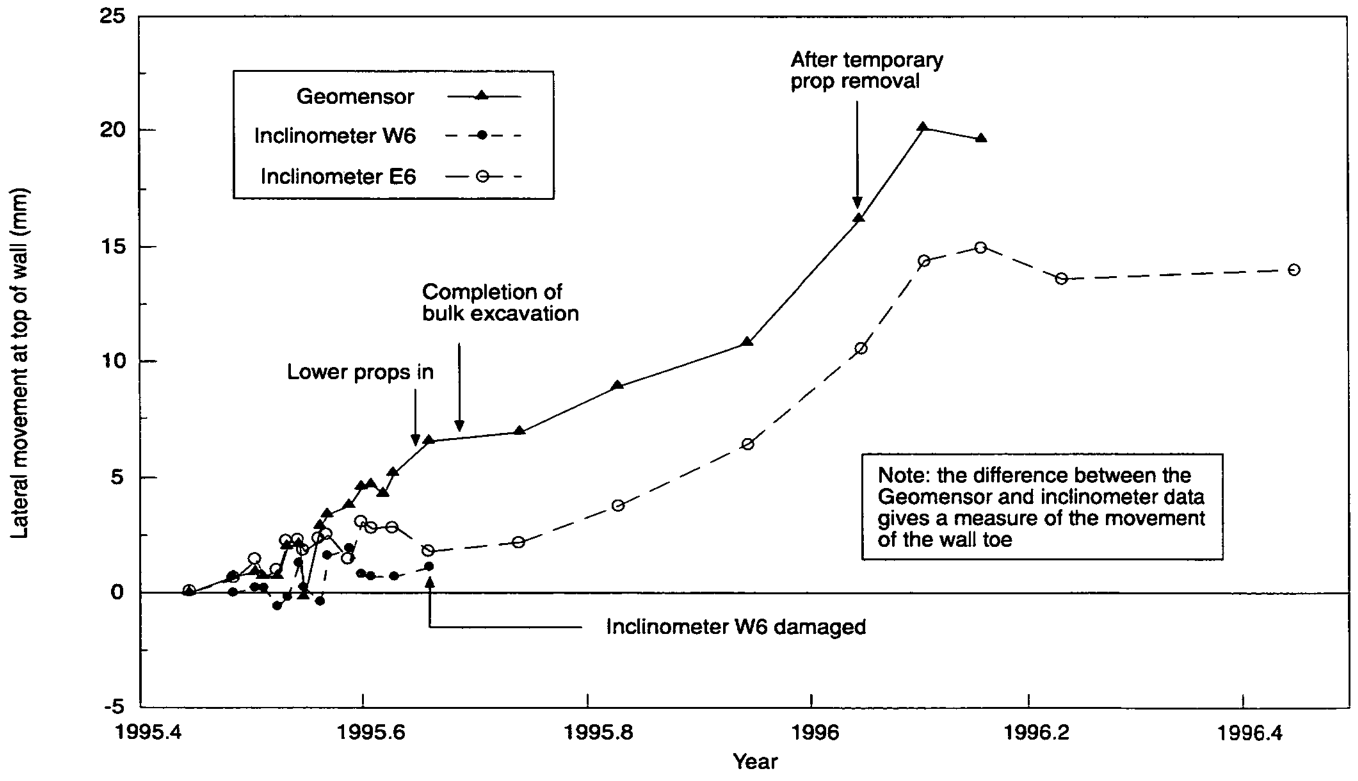


Fig.10 Development of lateral movement of the top of the wall

The profiles of wall movement with depth established from the inclinometer and Geomensor data at various key construction stages are shown in Fig 11. Results over the full depth of wall inclinometer tube W6 were only available for Fig 11a as the tube was subsequently damaged during construction of the wall edge beam at carriageway level. Generally the temporary props were effective in restricting movement of the top of the wall to about 5mm, although movement towards the excavation increased to about 11mm in December 1995 (Fig 11b). This change was probably associated with thermal contraction of the steel temporary props as sub-zero temperatures were experienced and thermal loads in the temporary props reduced as a consequence (see Section 6.2.4). On release of the temporary props, the wall cantilevered towards the excavation and by February 1996 movements of about 20mm were measured at the top of the wall. Movements of a few millimetres occurred at carriageway level as load was taken up by the permanent prop slab. Fig 11d shows that very little change in the movement profile with depth was recorded over the following 4 months.

The wall movement results from Fig 11 are summarised in Fig 12 and compared with lateral movements measured on the ground inclinometer tubes I1 and I2 at 1.5m and 4m respectively behind the counterfort of the diaphragm wall. Both ground inclinometer tubes were extended during the construction works and lateral movements subsequently developed in the backfill have been included for completeness. During backfilling, tube I1 suffered some damage at the top and the apparent reduction in movement measured at about 67mAOD may not be reliable. Generally the results were reasonably consistent and indicate that, at distances of up to 4m behind the counterfort, ground movements approached but were not quite as large as the wall movements.

6.2.2 Wall bending moments

Readings from the thermistors incorporated in the strain gauges embedded in diaphragm wall panel W35 showed that temperatures of up to about 58°C were measured 5 days after the concrete pour. Fig 13 shows the variation of temperature with depth at different times for thermistors installed 250mm in from the front face of the T-panel and the same distance in from the back of the panel counterfort. Higher temperatures were recorded throughout the hydration process on the thermistors towards the front face of the panel because of the greater mass of concrete in this area. Two of these thermistors, in the region where the permanent prop acted, were installed at a slightly greater depth of 480mm behind the front face and showed higher temperatures than the others (Fig 13). Generally temperatures on the thermistors had dropped to below 32°C after 46 days and were considered to have virtually stabilised by 126 days. Datum readings for the strain gauges were therefore established at this time when subsequent effects of shrinkage due to concrete hydration were considered likely to be minimal.

Fig 14 shows the development of wall bending moments during key stages of underpass construction as calculated from the vibrating wire strain gauge data. Fig 14a shows that, immediately before the lower temporary props were installed, the moments (negative) increased with depth to a value of about -1400kNm/m at 62mAOD just below excavation level. This can be compared with a value of about -600kNm/m calculated at 65mAOD using a load of 270kN/m in the upper temporary prop (Section 6.2.4) and assuming a K value (ratio of effective horizontal to vertical stress) of 1.5 for the London Clay. Some variation in the bending moment profile was measured in the period following excavation to formation level and this was probably related to the thermal loads developed in the temporary props. Shortly after temporary prop removal (Fig 14c), bending moments started to increase. After a further 5 months had elapsed Fig 14d shows that bending moments generally were in the range -630kNm/m to +500kNm/m. Further changes are expected in the longer term as any remaining excess porewater pressures in the clay slowly dissipate. Calculations based on K values of 1 and 1.5 in the clay predict bending moments of the order of +1700kNm/m and +2000kNm/m respectively at prop slab level.

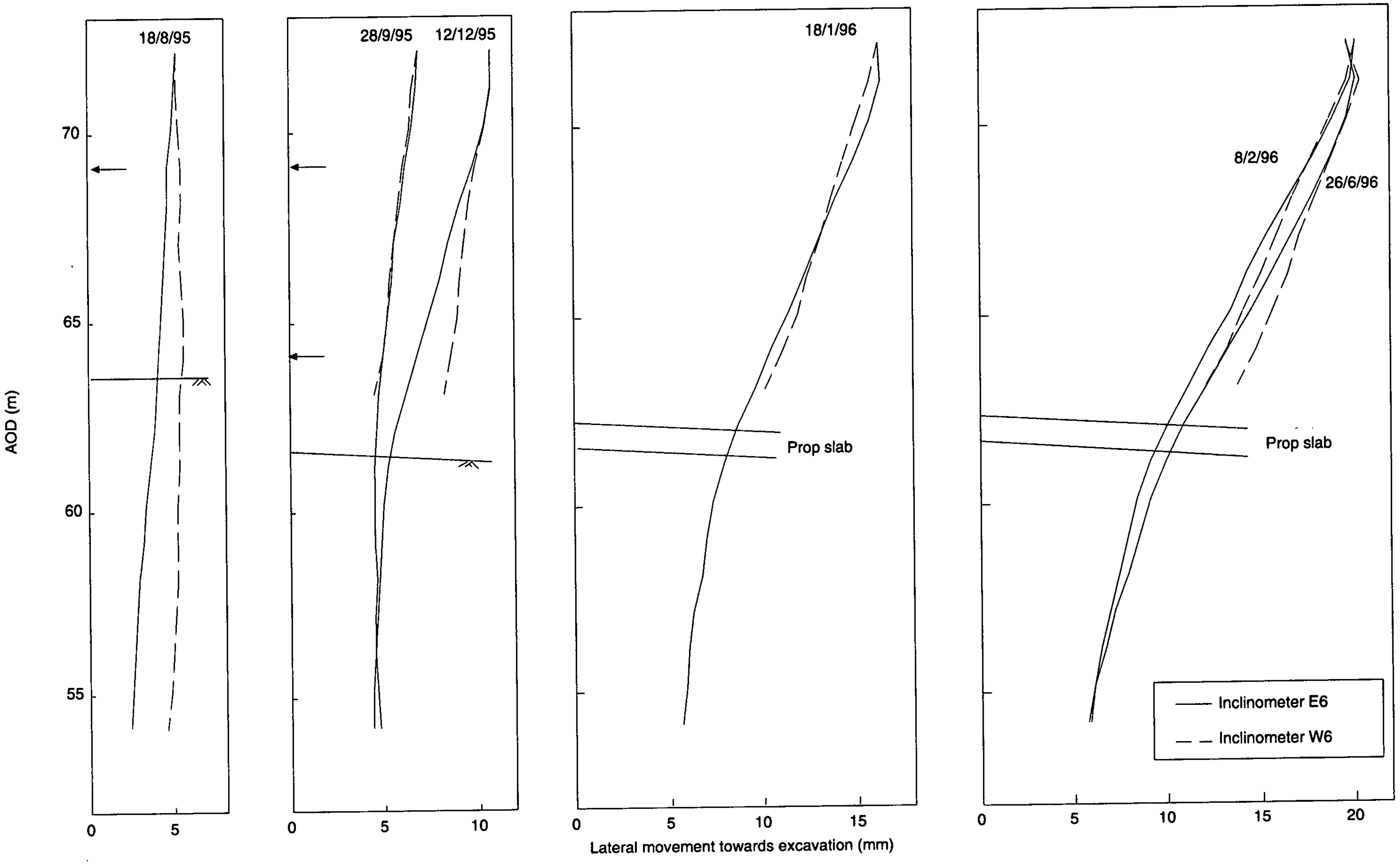
6.2.3 Porewater pressures

Figs 15a to 15e show the variation of porewater pressure measured by the pneumatic piezometers below formation level during the construction sequence. The locations of the piezometers are shown in Fig 7. Readings were not possible during the period of diaphragm wall installation because the piezometer cables were temporarily buried within manholes to prevent damage from construction traffic. However the results in Fig 15 show that the original ground water levels re-established themselves fairly rapidly after wall installation and that, over a depth of up to 20m, the porewater pressures approximated to a hydrostatic distribution from ground surface.

An immediate reduction in porewater pressure was measured upon excavation to formation level. Following completion of excavation, a small recovery in values was then measured during the construction of the permanent prop slab and carriageway. Latest piezometer results are compared in Figs 15a to 15e with the values calculated assuming a hydrostatic distribution from the drainage blanket beneath the prop slab. Generally measured values were either equal to or greater than those calculated indicating that some seepage around the wall was occurring.

6.2.4 Temporary prop loads

Readings from the strain gauges and thermistors were logged at hourly intervals after temporary prop installation. Fig 16 shows the variation of measured prop loads and temperatures during the construction period. Fig 16a shows the results from the upper prop U51 and Fig 16b shows those from the lower prop L34 in the research area.



(a) Before installing lower props

(b) After bulk excavation

(c) Shortly after temporary prop release

(d) Longer term

Fig.11 **Development of lateral wall movement**

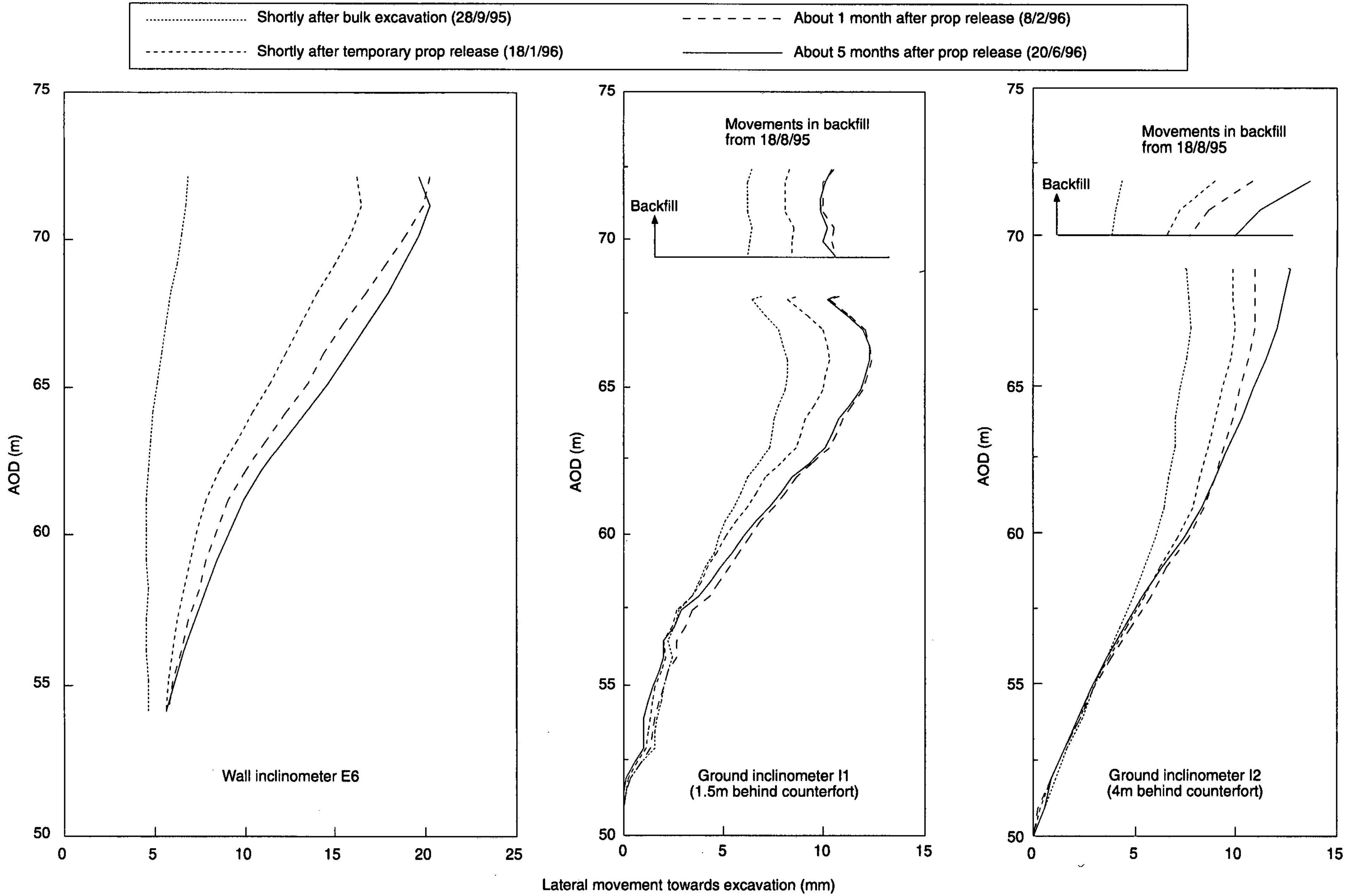


Fig.12 Development of lateral wall and ground movement

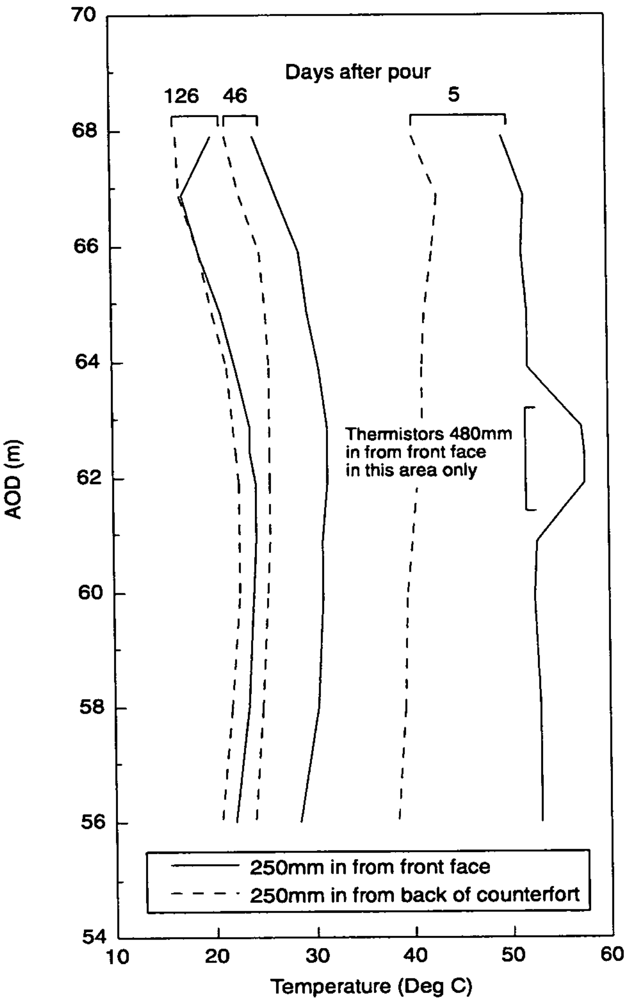


Fig.13 Variation of temperature with level in panel W35

Considerable variation in measured loads was obtained with temperature. The upper prop was installed at the end of July and the lower prop at the end of August 1995. As excavation progressed under the props, the loads on the upper prop steadily built up and reached a peak of 2300kN in early October 1995, shortly after excavation to formation level was completed. Loads on the lower prop were much smaller with a maximum of only 300kN being recorded.

From plots of average frequency against time, an estimate was established of the base temperature at which the prop was locked in position and started to resist load. Whenever the prop was at this temperature, the total load was assumed to be solely due to the soil loading component with no load due to thermal expansion/contraction of the steel. On a daily basis, a best fit line was drawn through the data of measured load against temperature. Determination of the load at the base temperature, using extrapolation where necessary, enabled the measured load to be separated into its soil load and thermal load components. This method worked well when there was a large temperature variation during the day but, at other times, the extrapolated value was more difficult to obtain accurately.

Fig 17 shows the results when the measured loads (averaged over a daily cycle) on the upper prop are assessed using their soil load and thermal load components. Also indicated in Fig 17 are values of measured total load when the prop temperature was equal to the base temperature, i.e. no thermal load existed. Where this value was available it agreed well with the calculated soil load and tended to validate the method of separating the load components. On this basis the major part of the soil load developed during bulk excavation although there was a further small increase during the 3 month period when the permanent prop was being constructed. It must be noted in Fig 17 that the base temperatures for both props were in excess of 30°C and this meant that the additional load from thermal effects allowed for in the Contractor's temporary works design did not occur; in fact the thermal load was generally tensile.

6.2.5 Permanent prop loads

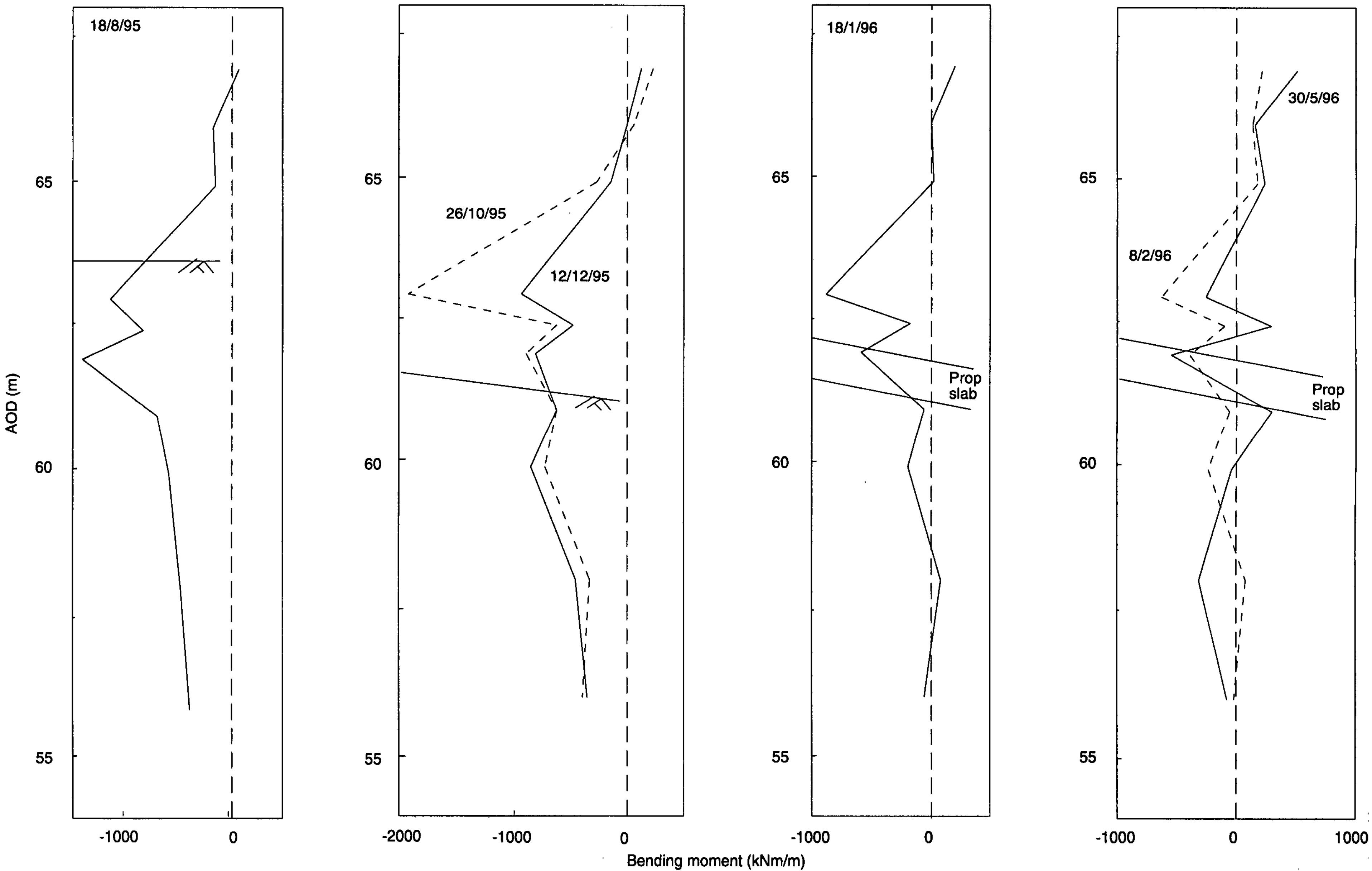
The variations in permanent prop load and temperature with time measured using vibrating wire strain gauges and thermistors respectively are shown in Fig 18. Each load value in Fig 18a and 18b is the axial load calculated from averaging the measurements from the four strain gauges located at that particular distance from the wall (Fig 8). The calculations of axial load were based on an estimated value of 30GN/m^2 for the elastic modulus of the reinforced concrete prop slab.

The plots show that there was an immediate increase in load on removing the temporary props which was followed by a steady increase in load during the following 5 months. This latter increase may be due to the thermal expansion of the concrete prop slab as its temperature rose during the summer months (Fig 18c). The loads within the western side increased to between 157 and 343kN per metre run whilst loads in the eastern side increased to between 228 and 471kN per metre run. In general axial loads measured near to the wall were about 150kN per metre run less than those measured elsewhere.

6.2.6 Heave below the carriageway

Subsurface heave measurements below formation level on the magnetic ring extensometer systems MR9, MR10 and MR11 (Fig 8) are shown in Fig 19. The datum date for readings in Figs 19a and 19b was prior to any excavation; results in Fig 19c were calculated from a later date as this particular system was a replacement for one accidentally damaged during the excavation. Results after excavation to formation level indicate that a maximum heave of 33mm was recorded on the shallower magnetic rings M7 and M13 at 6m from the west and east edge beams respectively. Generally heave was observed at a significant depth below formation, for example a heave of about 9mm was

+ve wall moments correspond to compression in the front face



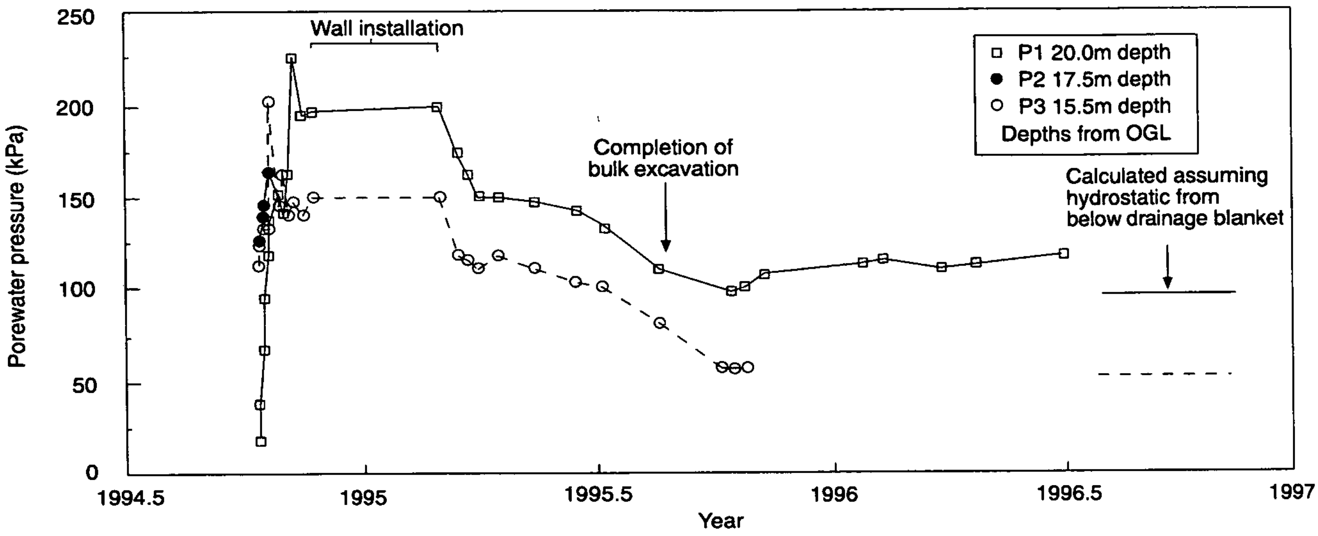
(a) Before installing lower props

(b) After bulk excavation

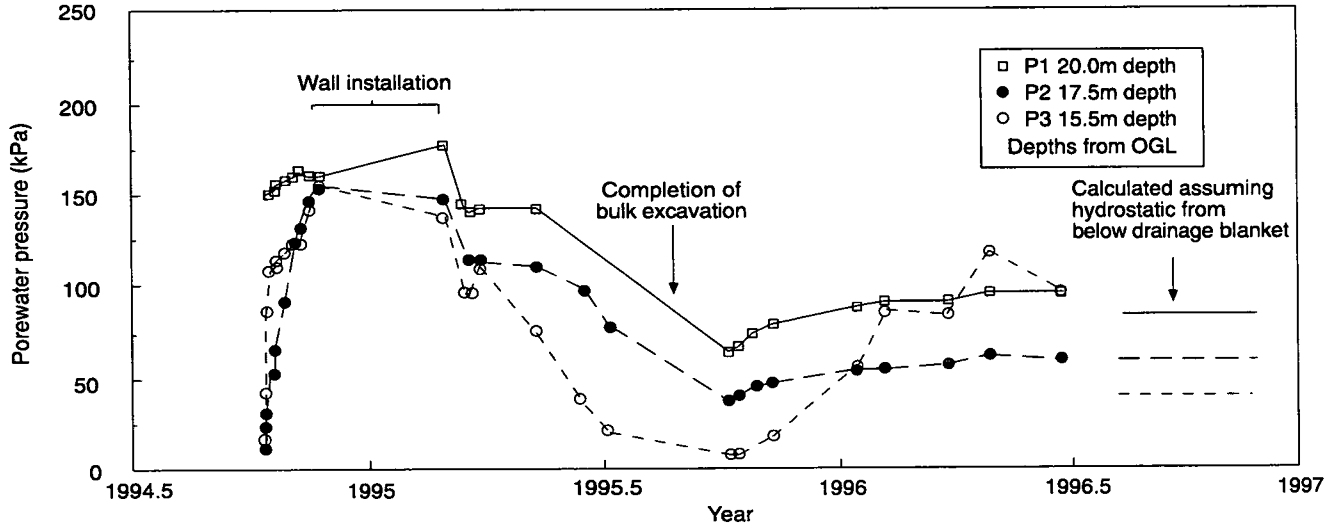
(c) Shortly after temporary prop release

(d) Longer term

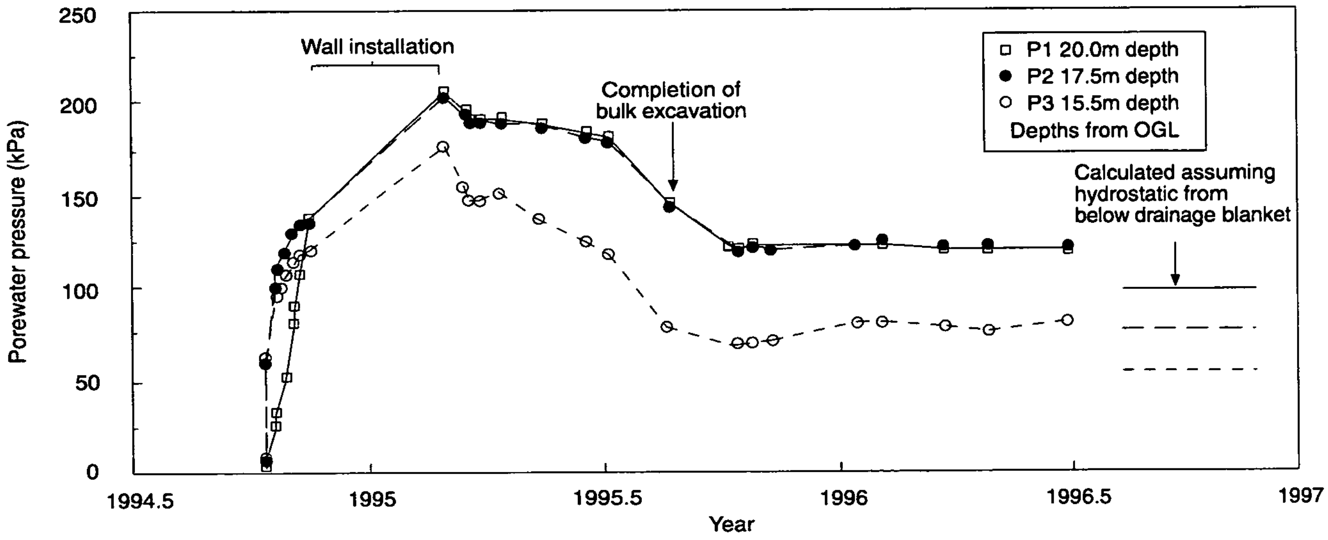
Fig.14 Development of wall bending moments



(a) At 1.5m from west wall edge beam

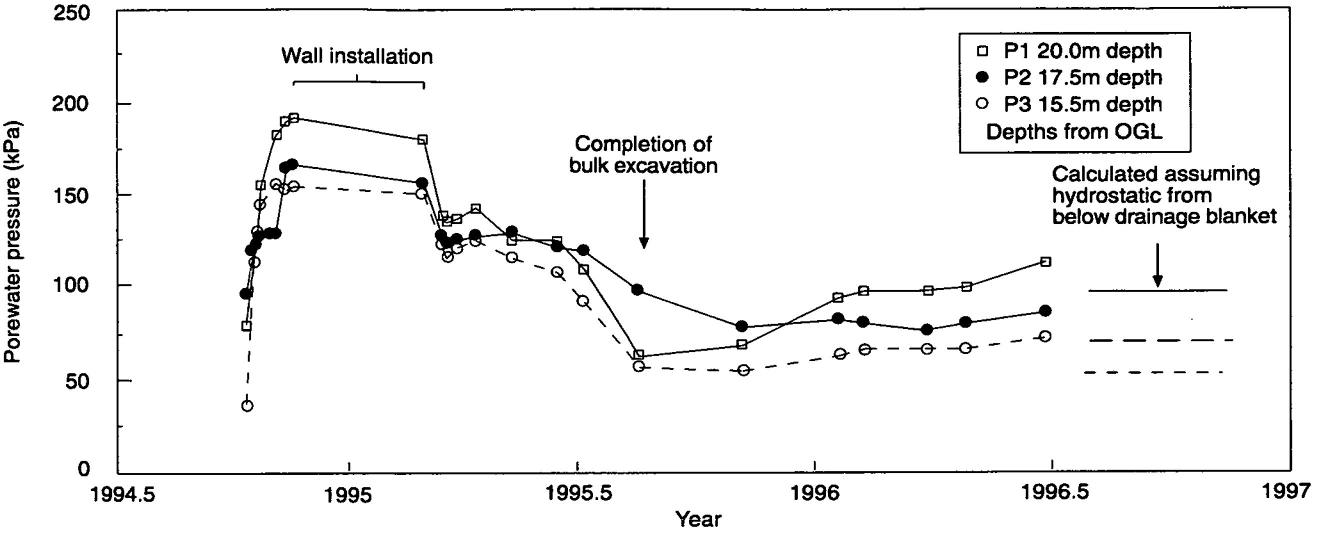


(b) At 14.5m from west wall edge beam

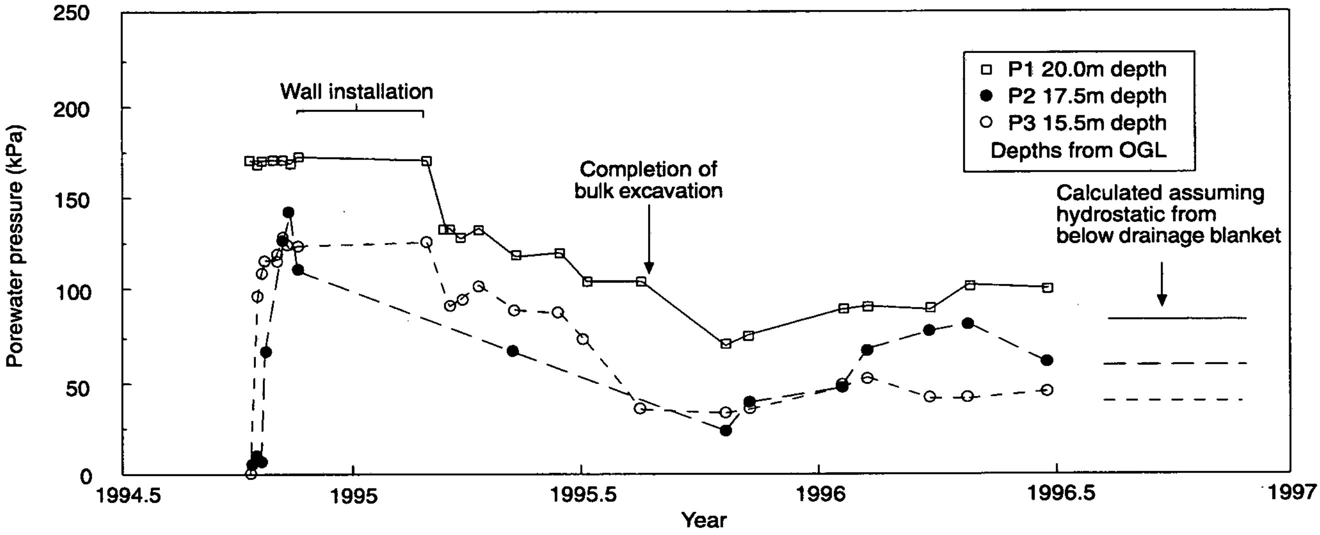


(c) At 1.5m from east wall edge beam

Fig 15 (a-c). Porewater pressures recorded by pneumatic piezometers



(d) At 6.3m from east wall edge beam



(e) At 6.3m from west wall edge beam

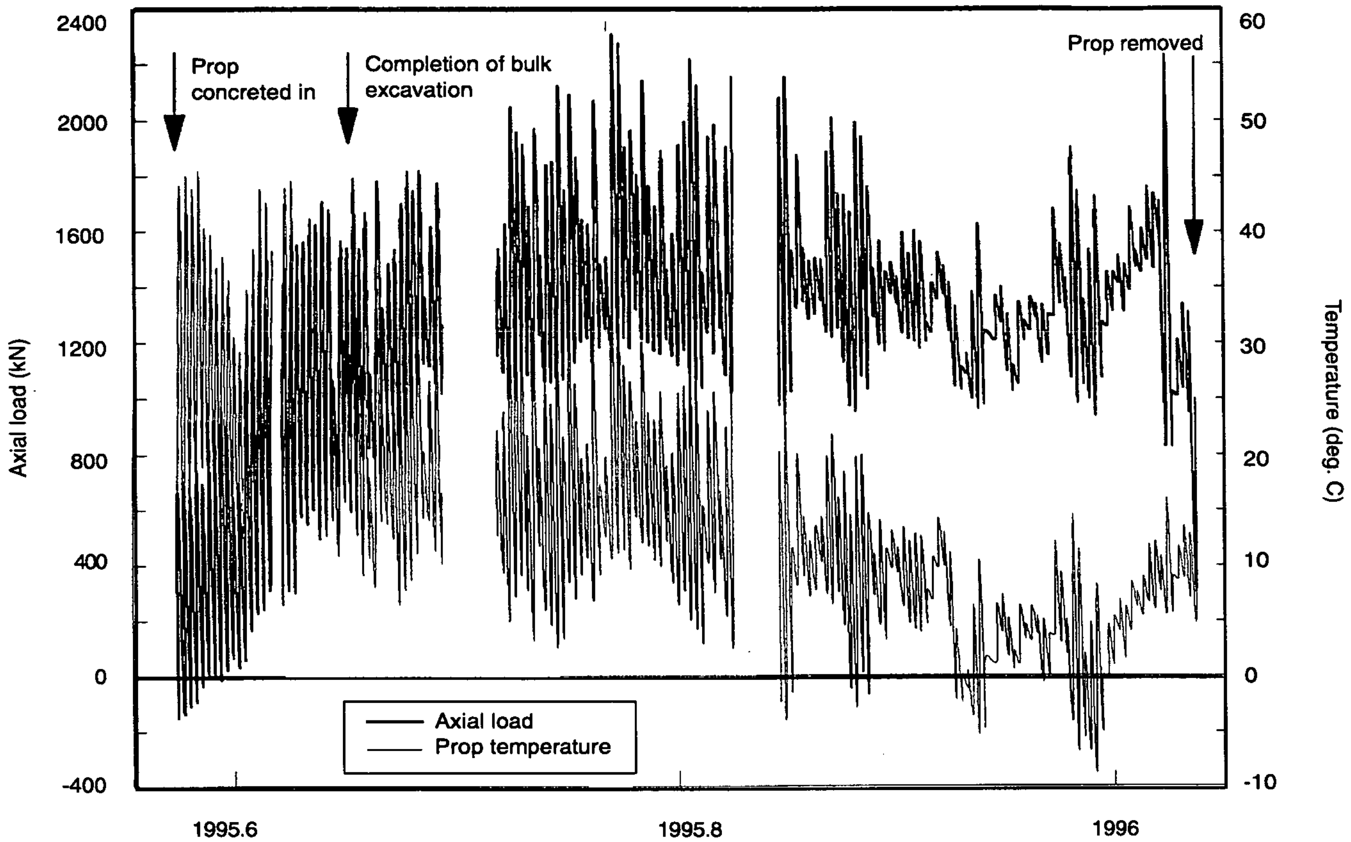
Fig 15(d-e). Porewater pressures recorded by pneumatic piezometers

measured on magnetic ring M9 at 44.5mAOD (i.e. 9.5m below the toe of the wall).

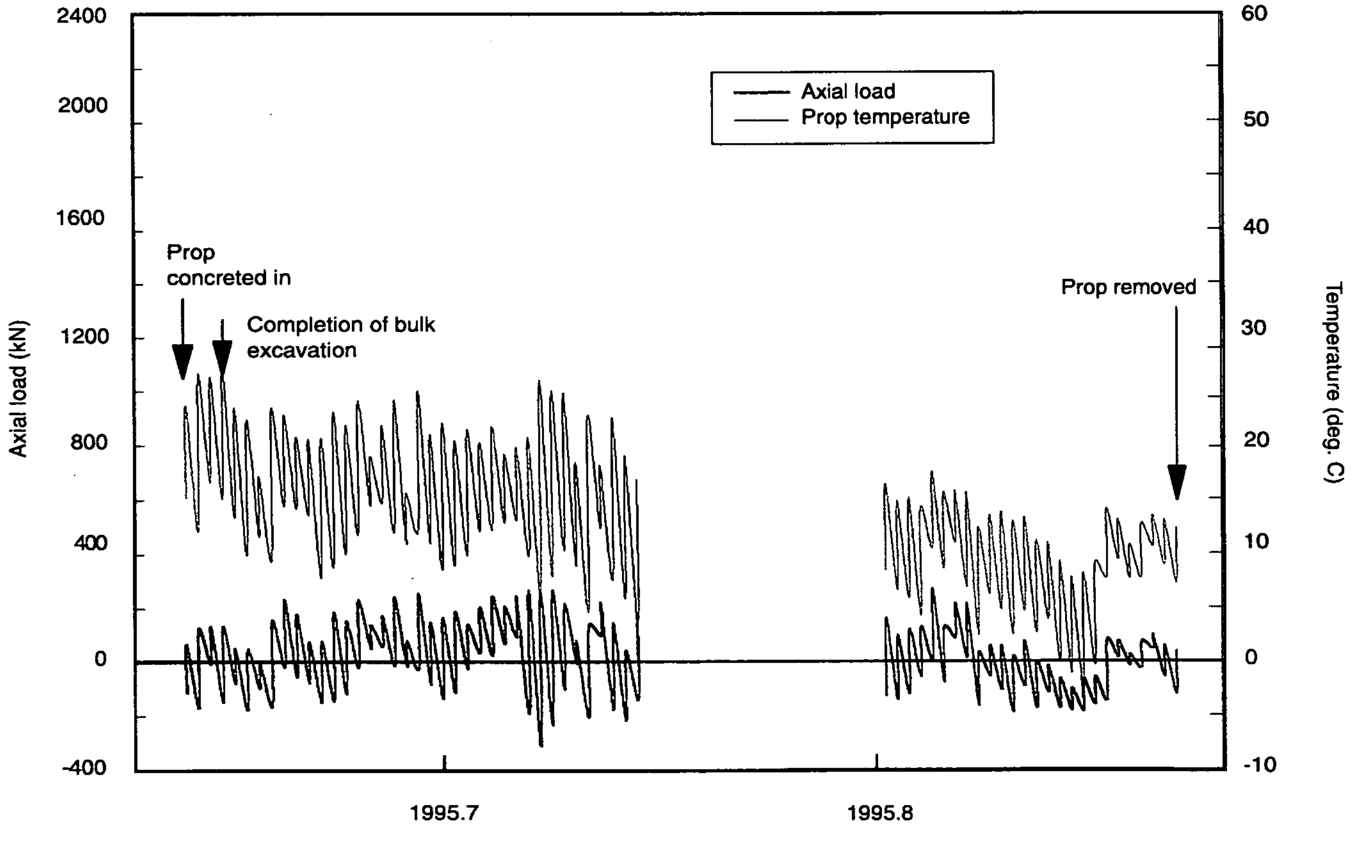
Following excavation, small decreases in the magnitude of the heave were then observed in response to the additional surcharge caused by casting of the permanent prop slab; extensometer system MR11 in the centre of the underpass showed the largest change as the central hinge was constructed. A steady increase in the magnitude of the heave measured on all magnetic rings then occurred over the following 5 months. By the time the road was opened to traffic, overall heaves of about 45mm had been recorded on the shallower rings at 6m from both wall edge beams (Fig 19a and 19b). An associated heave of approaching 25mm had been measured on the deepest ring (M10). If allowance is made for the different datum date, the overall heaves on the magnetic extensometer system (MR 11) were similar to the changes calculated on the systems at 6m from the wall edge beam.

6.2.7 Vertical stress beneath the permanent prop slab

Five pairs of pressure cells were installed in the sand filter layer at different distances from the wall to measure total vertical stresses below the permanent prop slab. The average results for each pair are shown in Fig 20. Generally the measured stresses beneath both the western and eastern prop slabs tended to be lower closer to the wall than they were further away. This was not unexpected as carriageway construction involved placing considerably more fill towards the centre of the underpass than nearer to the wall (Fig 7). For example, if the amount of overburden is calculated at the cell positions, the vertical stress would



(a) Upper prop U51



(b) Lower prop L34

Fig.16 Temporary prop loads and temperatures

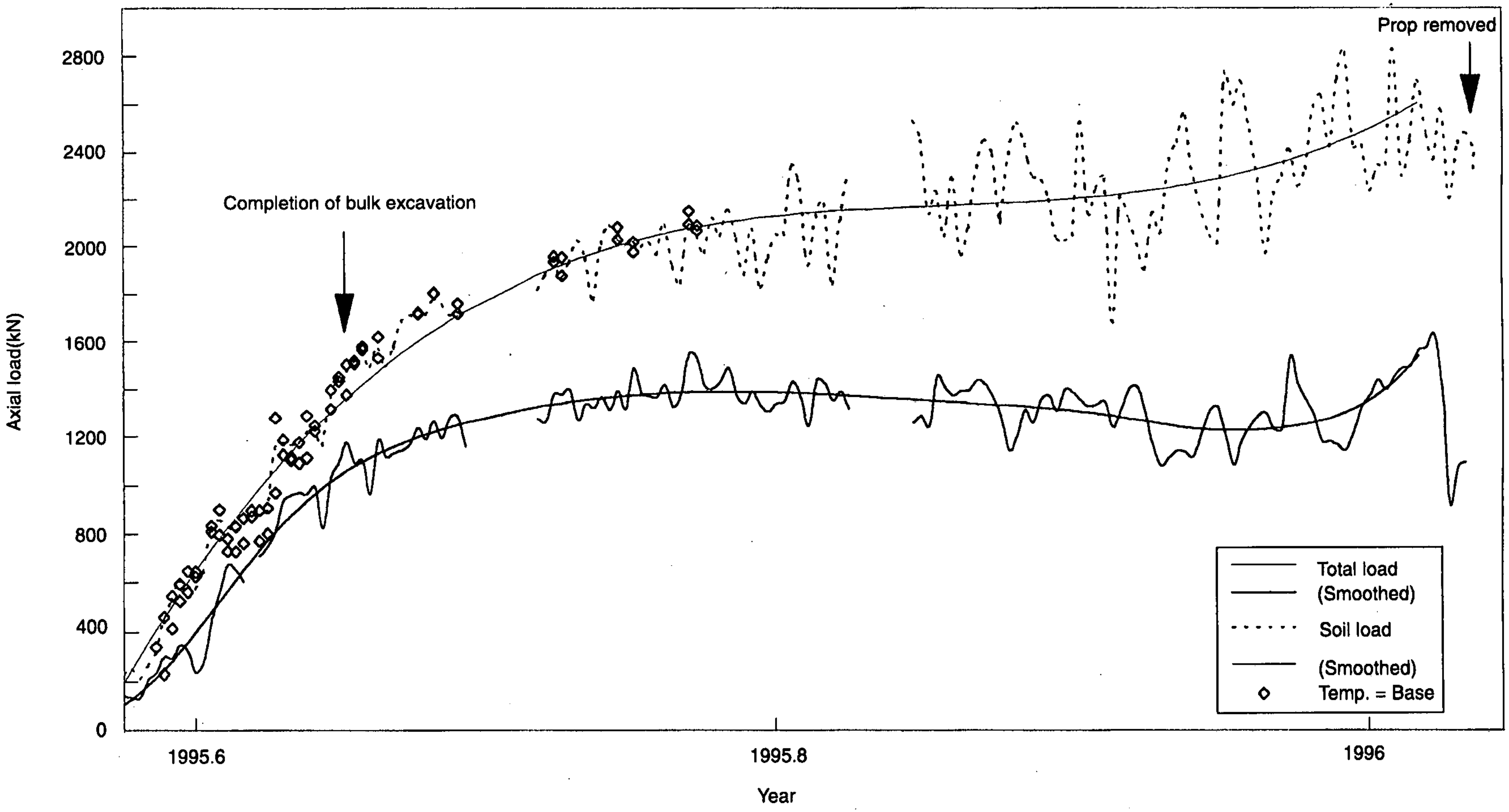
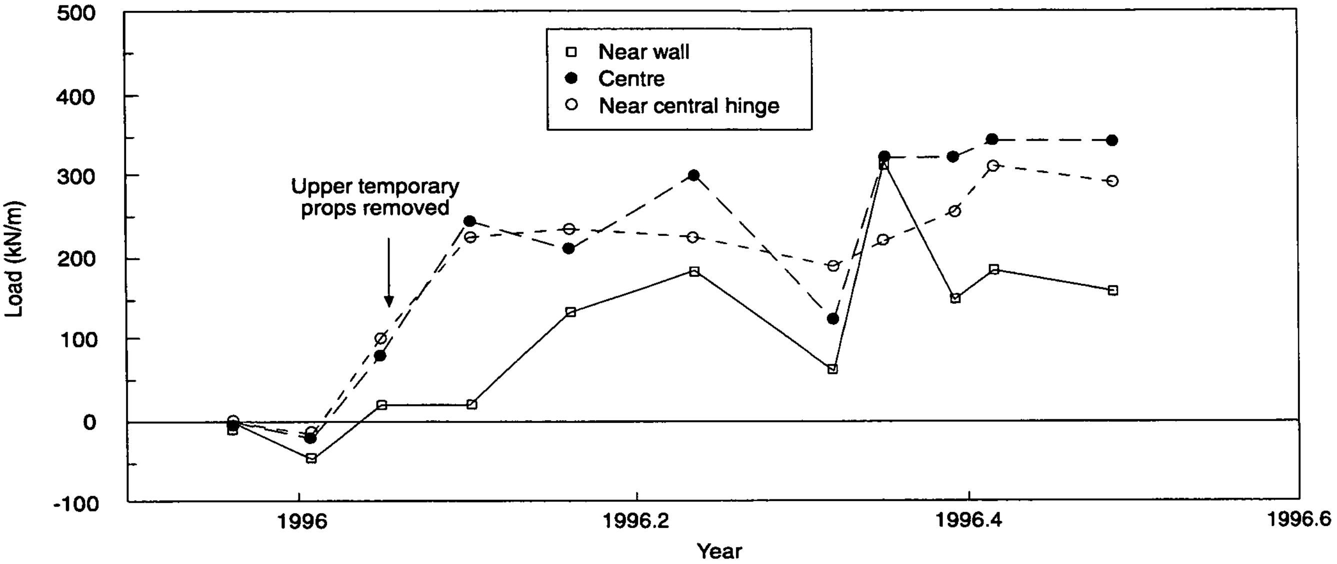
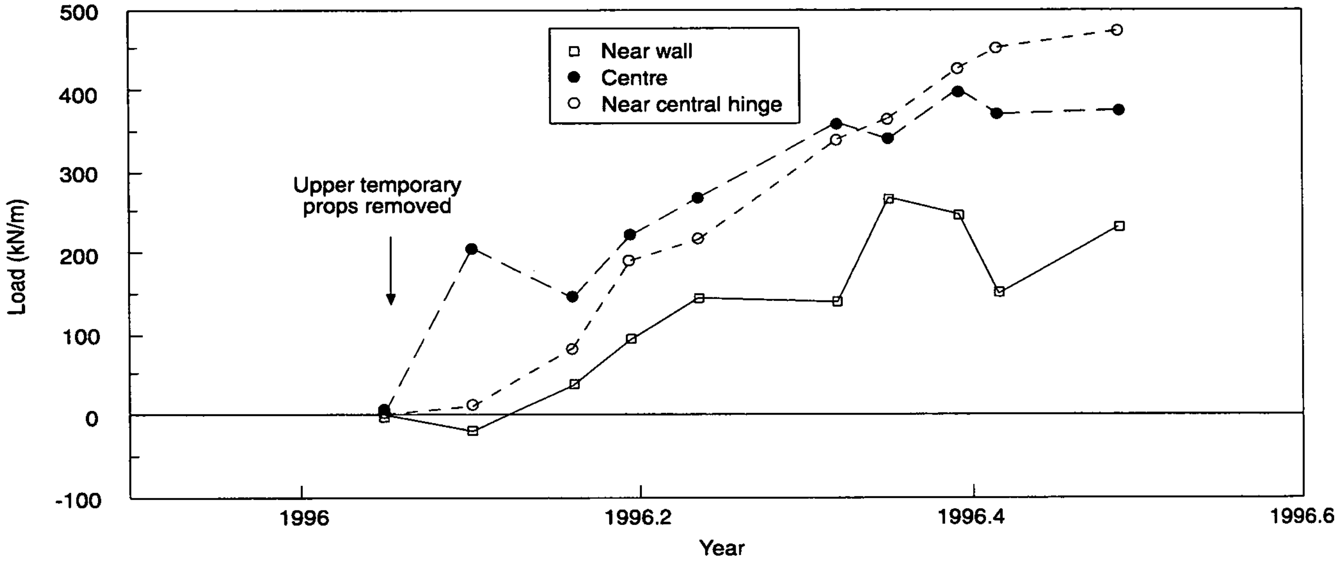


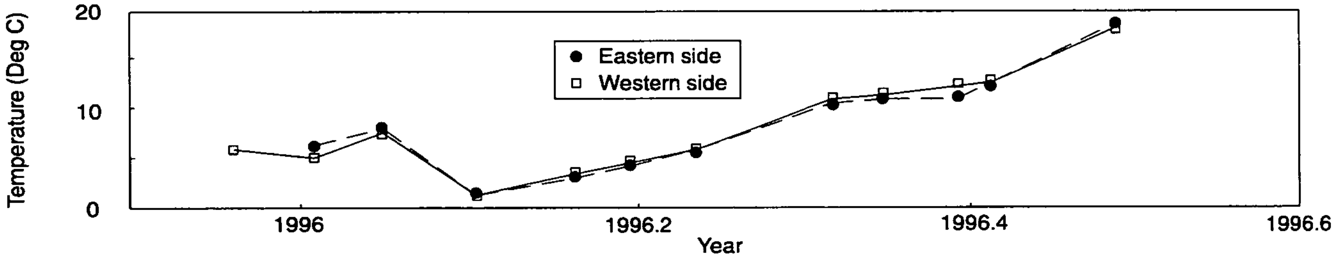
Fig.17 Daily average axial loads in upper temporary prop U51



(a) Western side of area 6 prop slab

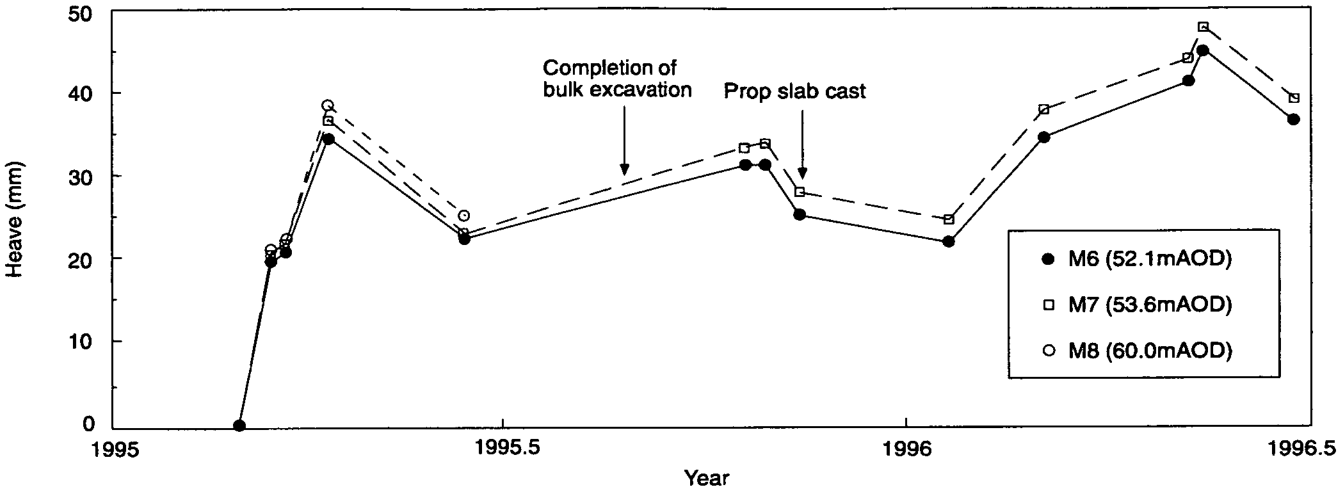


(b) Eastern side of area 6 prop slab

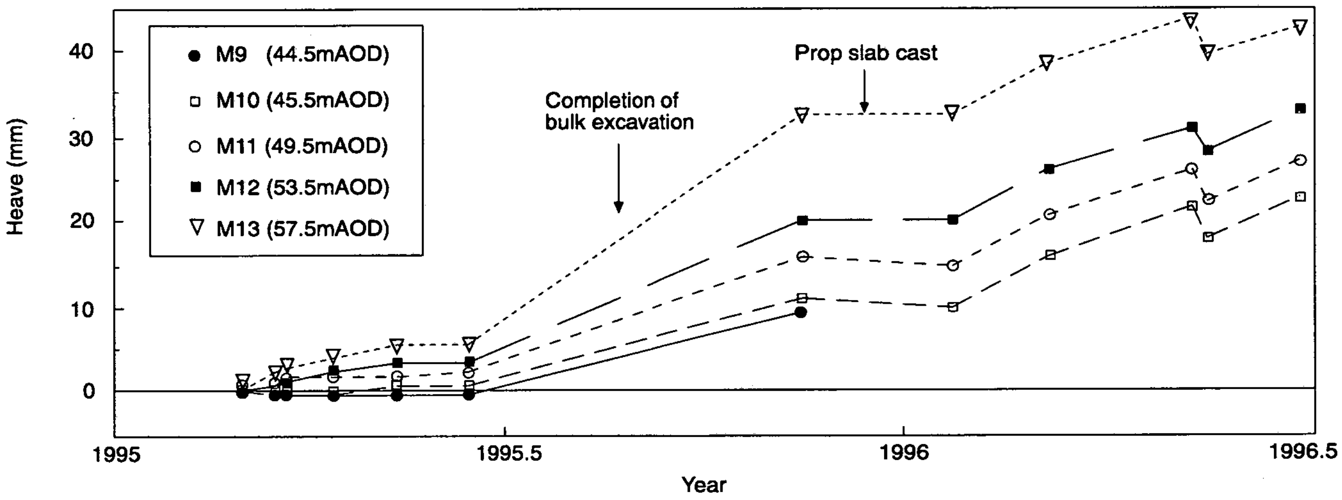


(c) Temperatures of prop slabs

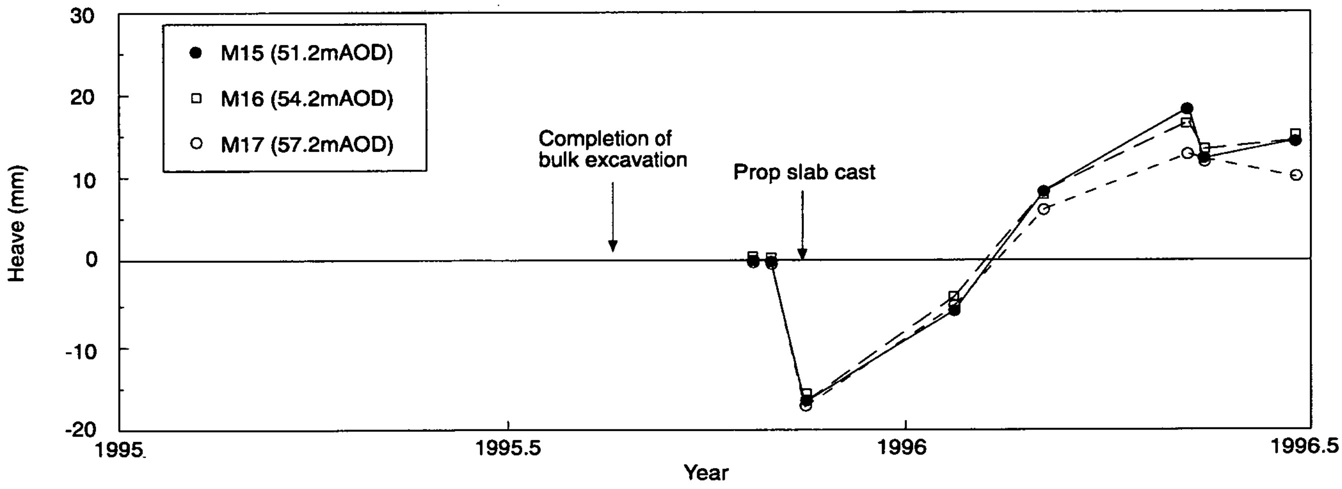
Fig.18 Loads and temperatures in the permanent prop slab



(a) 6m from west wall edge beam (MR9)

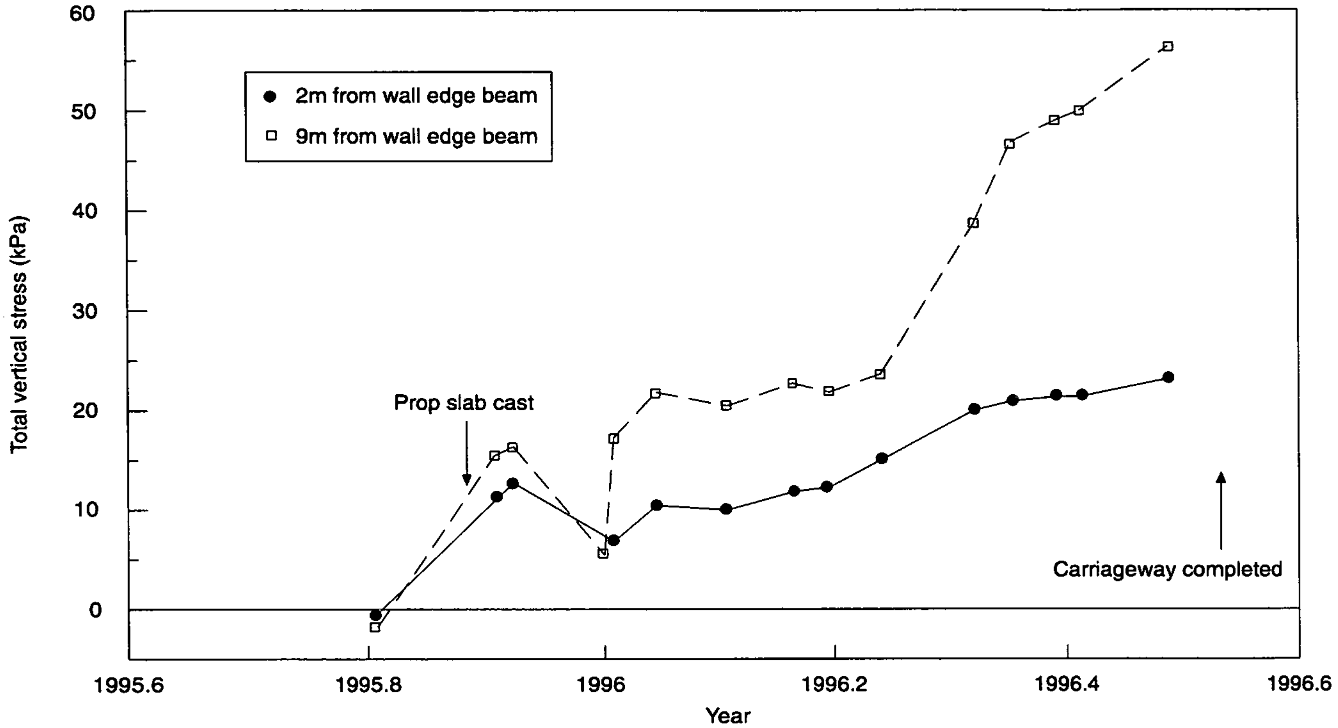


(b) 6m from east wall edge beam (MR10)

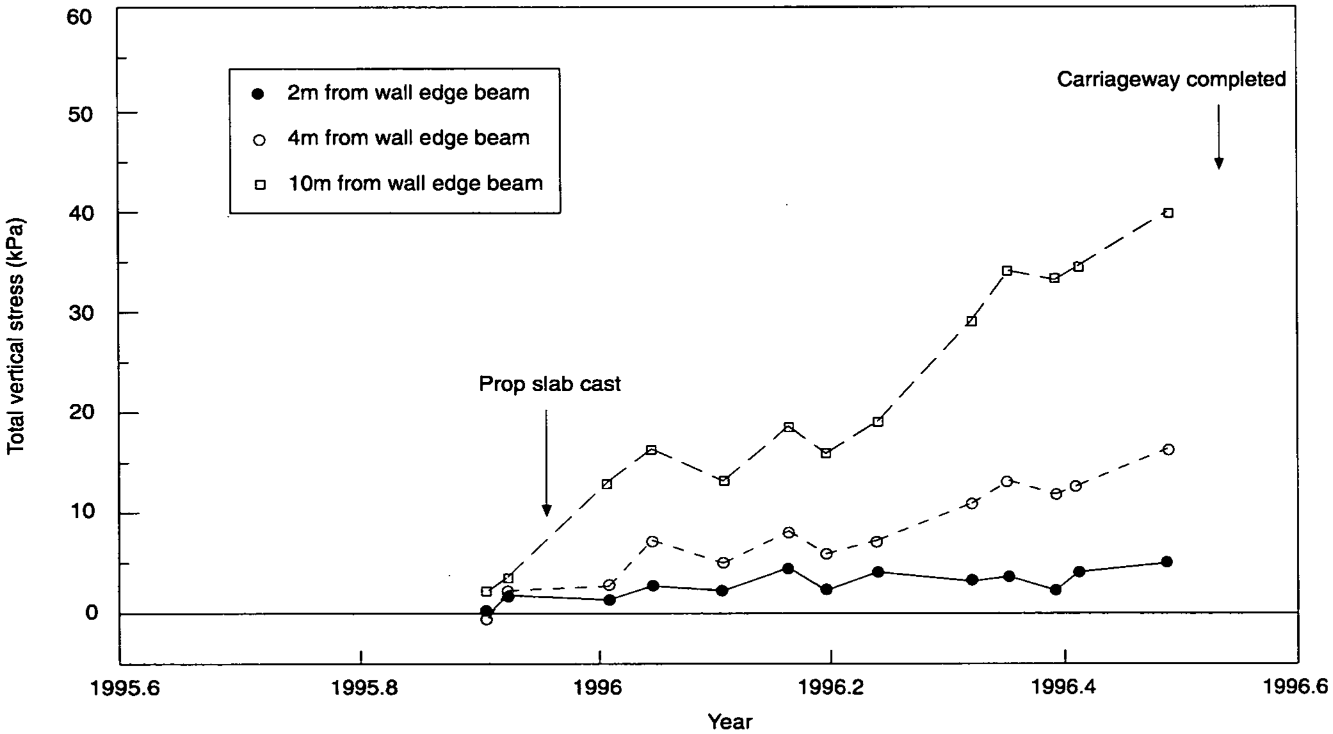


(c) Centre of underpass (MR11)

Fig.19 Development of subsurface heave beneath the prop slab



(a) Below western side of area 6 prop slab



(b) Below eastern side of area 6 prop slab

Fig.20 Development of vertical stress beneath the prop slab range from a minimum of about 35kPa at 2m from the edge beam of the east wall to a maximum of about 65kPa at 9m from the west wall beam. Measured values towards the centre of the underpass shown in Fig 20 are therefore approaching their calculated overburden values, although the pressure cells nearer to both walls still read less than expected on this basis possibly because some stress is being transferred to the wall edge beam. The results indicate that either the hinged prop slab is moving to accommodate any heave pressures due to swelling of the underlying clay or that no significant heave pressures have developed in the initial 5 months. Measurements to assess the development of heave stresses in the longer term are continuing.

7. DISCUSSION

The magnitudes of the surface lateral movement during wall installation are compared with those recorded at other sites of T-shaped diaphragm wall construction and produced in non-dimensional form by Carder (1995). The non-dimensional plot is reproduced in Fig 21 and includes the data for Aldershot Road underpass. Generally the data for Aldershot Road underpass were very similar to those from the other sites and surface movements were well inside the upper bound previously suggested for installation of T-shaped diaphragm walls. Upper bound movements reported by Carder (1995) after installation of counterforted walls were 0.11% of the trench depth with a zone of movement extending to a distance of about 1.0 times the trench depth.

Following the same approach, the surface lateral movements developed after excavation in front of the wall were compared non-dimensionally with data from other sites in Fig 22. In this case the movements and distances from the wall were calculated non-dimensionally by dividing by the maximum depth of excavation in front of the wall rather than the wall penetration. The distance from the excavation has been taken as that from the front face of the wall so that movements at zero distance represent wall movement rather than ground movement. The results in Fig 22 for Aldershot Road underpass agree closely with those from other sites where the walls were propped at carriageway level and founded in London Clay. Movements were within the upper bound of 0.2% of the excavation depth proposed by Carder (1995). It must be noted that movement measurements have been separately quoted for the wall installation and the main construction stages, i.e. overall surface ground movements have to be obtained by summation of the two effects.

A major consideration in the design of walls propped at carriageway level and founded in overconsolidated clay is the magnitude of the heave which develops in both the short and long term beneath the structural slab. The type of connection between the wall and prop can have a significant effect on behaviour (Powrie and Li, 1991). At Aldershot Road underpass, the design incorporated hinged connections both at the walls and in the centre of the slab which was constructed as a shallow V-shape. Because of the proximity of the River Blackwater and the high groundwater levels, this design was required to restrict the uplift from groundwater and also long term swelling pressures in the clay. On completion of construction, the measurements showed that overall heaves of about 45mm had been recorded at depths of a few metres below dredge level; the heave was fairly deep seated with values of approaching 25mm being measured at 15m below the slab (Fig 19). This heave was due to the immediate undrained response of the ground to the removal of overburden and to some swelling as excess porewater pressures dissipated over the following 5 months as indicated by the piezometer results in Fig 15. The magnitude of the heave can be compared with the value of 35mm measured at 0.8m below formation level during bulk excavation at Bell Common tunnel (Tedd et al, 1984) and the heave of 19mm recorded at 3m depth at Hackney to M11 Link (Bennett et al, 1996). Measurements of long term heave at Bell Common tunnel (Symons and Tedd, 1989) showed that a further 40mm of heave occurred at 1.5m depth over the first 4 years in service and further monitoring is therefore continuing at Aldershot Road underpass. Measurements from pressure cells in the drainage blanket below the prop slab currently indicate that total vertical stresses are no more than expected from overburden calculations (Fig 20) and that no significant pressures have developed in the initial 5 months.

Measurement of axial loads in the permanent prop slab on completion of construction gave values ranging between 157kN/m and 471kN/m with the smallest loads generally being measured nearer to the wall (Fig 18). These values can be compared with the load of 600kN/m predicted at this construction stage by numerical analysis undertaken during the design. The measured loads were slightly less than the 500kN/m recorded on completion of an underpass at Walthamstow with a similar hinged prop slab, although loads at the latter scheme then subsequently built up and showed a seasonal thermal variation between about 1000kN/m and 1500kN/m (Carswell et al, 1993).

8. CONCLUSIONS

The behaviour of a T-shaped diaphragm wall founded in London Clay and permanently propped at carriageway level has been monitored during the construction of an underpass at Aldershot Road. The construction procedure involved excavation beneath two levels of temporary props spanning the underpass. The following conclusions were reached.

(i) During diaphragm wall installation, surface lateral movements towards the excavation of 6.2mm and 5.5mm were

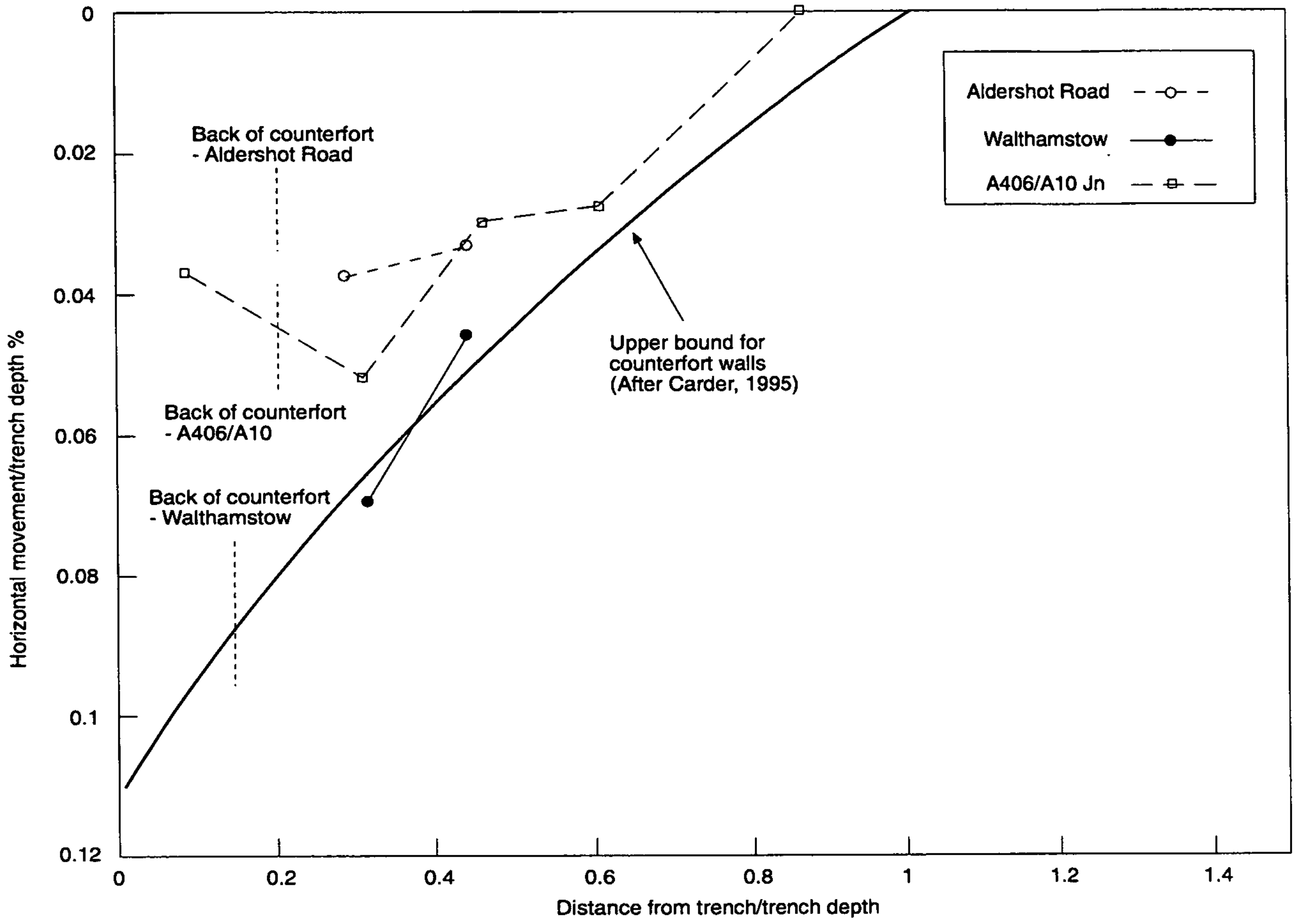


Fig.21 Horizontal surface movement during diaphragm wall installation in stiff clay

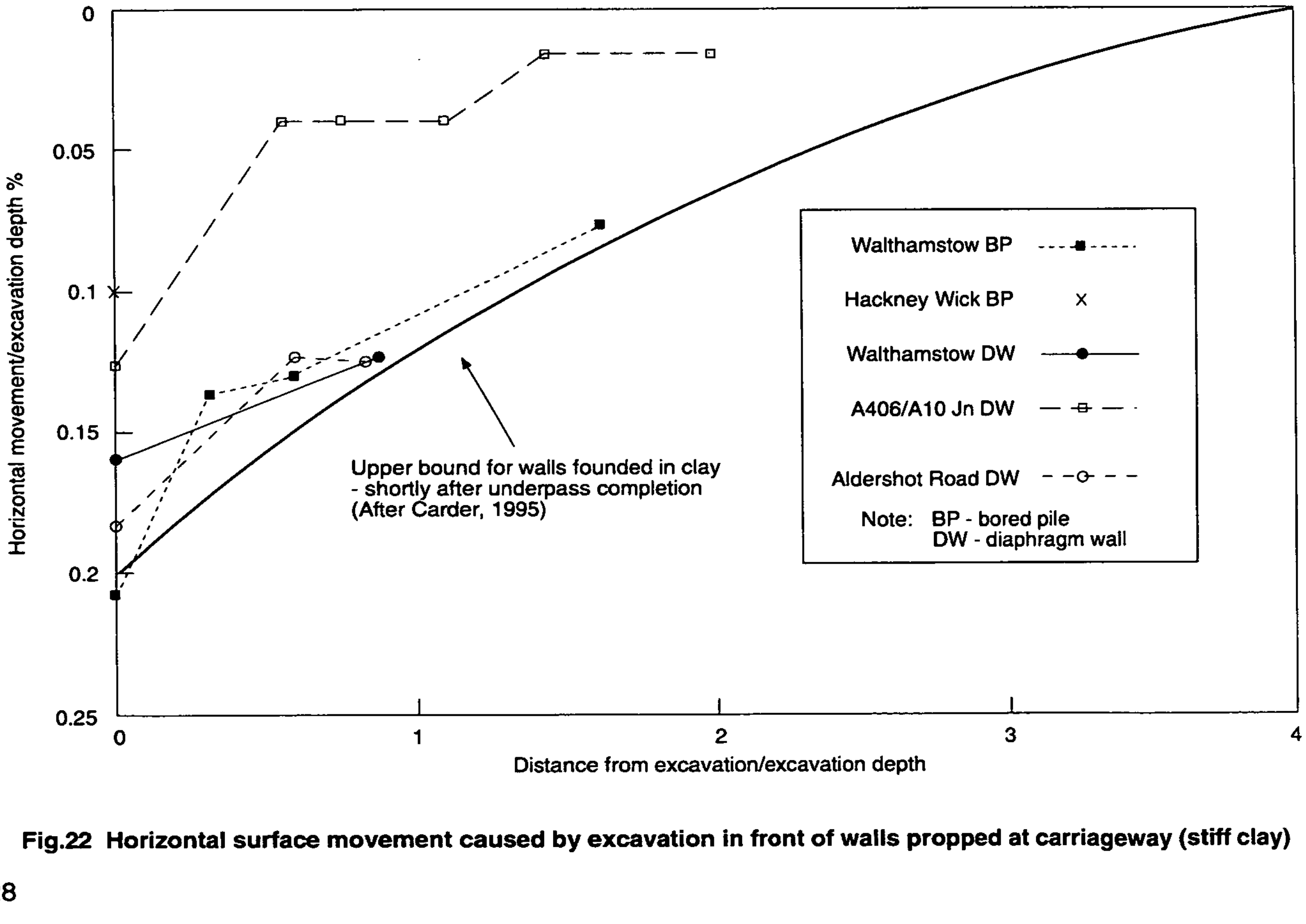


Fig.22 Horizontal surface movement caused by excavation in front of walls propped at carriageway (stiff clay)

recorded at 1.5m and 4m behind the counterfort of panel W35. These results were similar to those recorded at two other sites and were inside the upper bound movements of 0.11% of the trench depth with a zone of movement extending to a distance equal to the trench depth reported by Carder (1995).

(ii) Generally, up until the start of excavation to 6.9m depth below the upper level of temporary props, lateral movements at the top of the wall were no more than 5mm with toe movements of about 2mm. Movement of the toe increased to 5mm during excavation to 6.9m depth, although installation of the lower temporary props was then effective in minimising further toe movements during excavation to formation and permanent prop construction. On release of both levels of temporary props, movement at the top of the wall increased immediately to 16mm and reached 20mm after a further month. Overall wall and ground movements were within the upper bound of 0.2% of the excavation depth proposed by Carder (1995) on the basis of measurements at four other sites where underpass walls were propped at carriageway level.

(iii) Wall bending moments measured 5 months after release of the temporary props were generally less than 500kNm/m. Further changes are expected in the longer term as consolidation of the clay slowly occurs under the new stress regime. Calculations based on K values of 1 and 1.5 in the clay predict wall bending moments of the order of 1700kNm/m and 2000kNm/m respectively at permanent prop slab level.

(iv) At this site the porewater pressure distribution with depth approximated to a hydrostatic distribution from ground surface. Upon excavation to formation level an immediate reduction in porewater pressure was measured on piezometers below dredge level. Generally values measured 5 months after release of the temporary props were either equal to or greater than those calculated assuming a hydrostatic distribution from the drainage blanket beneath the prop slab indicating that some seepage around the wall was occurring.

(v) The measured loads in the steel temporary props varied considerably with temperature. As excavation progressed under the props, the loads on the upper prop steadily built up and reached a peak of 2300kN shortly after excavation to formation level was completed. Loads on the lower prop were much smaller with a maximum of only 300kN being recorded. As both props were installed at base temperatures in excess of 30°C, the thermal loads were generally tensile and acted to reduce the overall load in the props.

(vi) Measurement of axial loads in the permanent prop slab on completion of construction gave values ranging between 147kN/m and 440kN/m with the smallest loads generally being measured nearer to the wall. Although evidence from other schemes suggests that these loads will increase with time, they are well below the load of 600kN/m predicted for this stage.

(vii) On completion of construction, overall heaves of about 45mm had been recorded at depths of a few metres below final dredge level; the heave was fairly deep seated with values of approaching 25mm being measured at 15m below the slab. Measurements from pressure cells in the drainage blanket below the prop slab currently indicate that total vertical stresses are no more than those expected from overburden calculations. These results suggest that either the hinged prop slab is moving to accommodate any heave pressures in the underlying clay or that no significant pressures have developed in the initial 5 months. Further monitoring is continuing to evaluate longer term effects.

9. ACKNOWLEDGEMENTS

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