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# Behaviour of the diaphragm walls of a cut-and-cover tunnel constructed in boulder clay at Finchley

by A H Brookes and D R Carder

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OF TRANSPORT

TRL Report 187

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# TRL REPORT 187

# BEHAVIOUR OF THE DIAPHRAGM WALLS OF A CUT-AND-COVER TUNNEL CONSTRUCTED IN BOULDER CLAY AT FINCHLEY

**by A H Brookes and D R Carder**

**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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1996
ISSN 0968-4107

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# EXECUTIVE SUMMARY

Little information is available for cut-and-cover tunnels where top-down construction is employed with both wall and roof being installed prior to tunnel excavation. Field instrumentation has therefore been installed to investigate the performance of the diaphragm walls of a tunnel during and after construction. The tunnel traverses the ridge of a boulder clay outlier at Finchley and was constructed as part of the A406 North Circular Road Improvement between East of Falloden Way and East of High Road.

Measurements of porewater pressures and ground movements were taken during wall installation and all stages of tunnel construction. Instrumentation was installed in the diaphragm wall to monitor lateral movement and bending moments developed during construction. Axial loads in the roof and carriageway prop slab were also measured.

During installation of the diaphragm wall panels only small movements of the retained ground surface were measured. Lateral movements of 5mm and settlements of no more than 2mm were recorded 1.9m from the wall.

An initial excavation to 3.5m depth was then carried out to provide access for roof construction and during this operation the wall cantilevered towards the excavation. During bulk excavation, the integral roof was effective in acting as a prop and no additional temporary support was used. However some additional lateral movement occurred at depth with a maximum overall movement of 4mm being recorded a few metres above dredge level.

By completion of excavation a mean roof load of about 1000kN/m was measured and this value remained reasonably constant over the next 5 months. Over the same period, only small loads were measured in the permanent structural slab of the tunnel carriageway with no indication of any tendency to increase. Although the tunnel was designed as a doubly-propped structure, the integral roof and the depth of wall penetration appear sufficient to provide short term support.

The measured prop loads and wall bending moments are compared with those determined for overall stability using limit equilibrium methods.

# BEHAVIOUR OF THE DIAPHRAGM WALLS OF A CUT-AND-COVER TUNNEL CONSTRUCTED IN BOULDER CLAY AT FINCHLEY


## ABSTRACT

Field instrumentation has been installed to investigate the performance of the diaphragm walls of a cut-and-cover tunnel during and after construction. The tunnel traverses the ridge of a boulder clay outlier at Finchley and was constructed as part of the A406 North Circular Trunk Road Improvement between East of Falloden Way and East of High Road. The tunnel was constructed top-down with an integral roof slab installed between the planar diaphragm walls prior to bulk excavation and the construction of a structural carriageway slab.

Measurements of porewater pressures and ground movements were taken during wall installation and all stages of tunnel construction. Instrumentation was also installed in the wall panels to monitor lateral movement and bending moments developed during construction. Axial loads in the roof and carriageway prop slab were also measured.


## 1. INTRODUCTION

This study complements a series of studies on the performance of embedded retaining walls, a type that is being used increasingly in the construction of roads below ground level. The method involves wall installation from the existing ground level, and subsequent excavation of soil from the front of the wall down to the required level for road construction. The advantages over conventional methods are reduced land-take and less disturbance of ambient ground.

Earlier TRL field studies have been reviewed by Carder (1995) but largely concentrate on the behaviour of walls which are permanently propped using only a structural slab at carriageway level. Few data are available for cut-and-cover tunnels where top-down construction is employed with both wall and roof being installed prior to tunnel excavation. A previous study was undertaken at Bell Common Tunnel (Tedd et al, 1984) where excavation to 5m depth was unsupported prior to roof construction and a 75mm thick compressible packing used between the roof beams and thrust wall. At Finchley the roof structure is integral with the diaphragm walls and excavation to formation level was carried out with no additional temporary support. At this intermediate stage, the measurements can be compared with design predictions for walls propped near the top and founded in stiff clay as recommended in CIRIA Report 104 (Padfield and Mair, 1984) and in BD42 (DMRB 2.1). Following excavation, the permanent structural slab at tunnel carriageway level was installed to complete the doubly-propped structure. The field results can then be compared with centrifuge and analytical studies of doubly-propped walls reported by Richards and Powrie (1995).

This report describes the field observations made at Finchley during the installation of the 5m wide diaphragm panels, the subsequent construction of the cut-and-cover tunnel, and its performance for a further six months after completion.

## 2. SITE LOCATION

The cut-and-cover tunnel is at the junction of the A406 North Circular Road and Old East End Road, Finchley, London N3, immediately north of the Hungry Horse (also called Manor Cottage) public house. The diaphragm panel section instrumented by TRL forms part of the southern retaining wall and is centred at contract chainage 755.

## 3. SOIL PROPERTIES

The tunnel traverses a boulder clay outlier underlain by London Clay at a maximum depth of 23m, with a substantial gravel layer at the interface. Boulder clay is a glacial till comprising a variety of soil types including stiff clay, chalk fragments and horizons of sand and flint gravel, the latter forming a complex of perched water tables at Finchley. Fig 1 compares a typical borehole log obtained in the instrumented area by TRL with that from borehole 10 (Le Grand Sutcliff and Gell, 1970) sunk within 15m of the instrumented area.

A series of consolidated undrained triaxial tests were undertaken as part of the site investigation (Frank Graham Geotechnical, 1989) to establish peak effective stress parameters. As a consequence the mean peak parameters adopted for design purposes were $\varnothing' = 26°$ and $c' = 6$kPa for the glacial till although a wide variation in values was recorded because of the presence of sand and gravel lenses. Values of $\varnothing' = 25°$ and $c' = 20$kPa were adopted for London Clay, with $\varnothing' = 25°$ and $c' = 0$ for made ground where applicable.

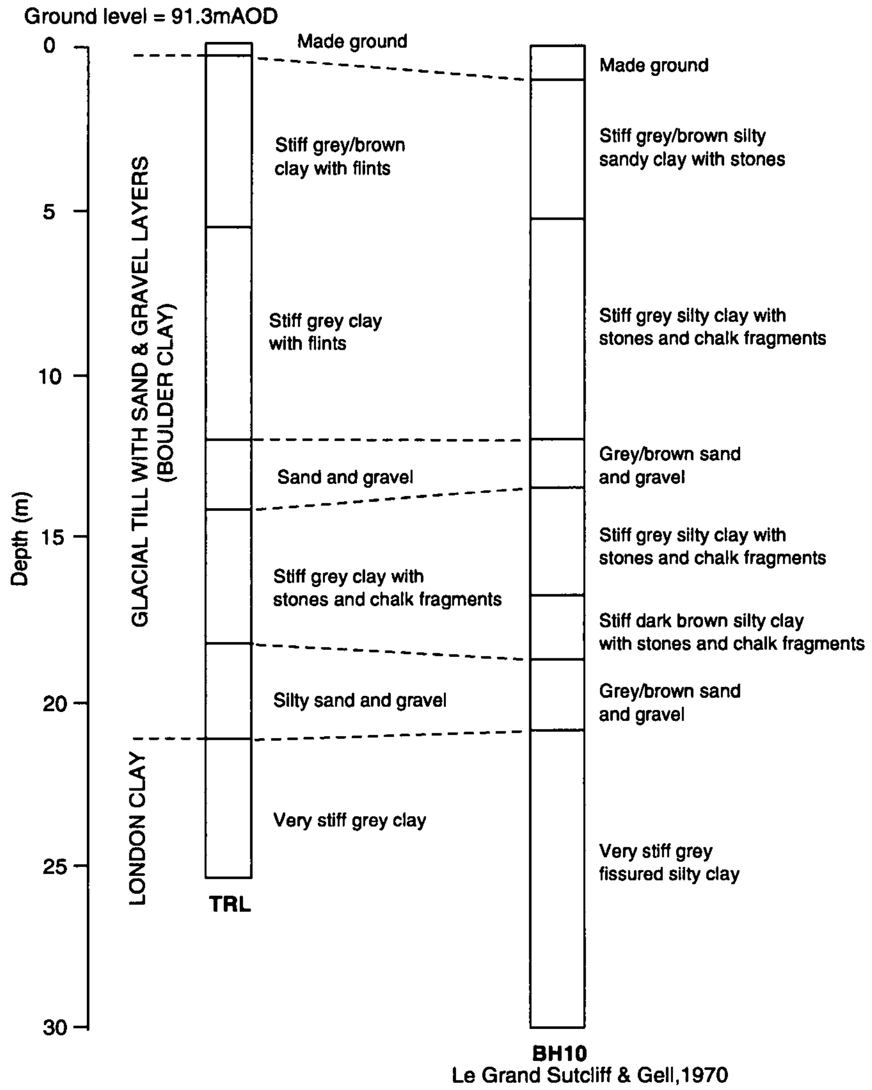



**Fig 1. Soil profile with depth**

The variation of undrained shear strength with depth is shown in Fig 2. There was considerable scatter in the results, although generally strengths of the glacial till were very high with upper bound values of up to 300kPa at 15m depth. Mean plastic and liquid limits, plasticity indices and natural moisture contents are given as percentages below:

| | PL | LL | PI | MC |
|---|---|---|---|---|
| Boulder clay (glacial till) | 17 | 45 | 28 | 16 |
| Weathered London Clay | 27 | 74 | 47 | 30 |

# 4. INSTRUMENTATION

Field instrumentation was installed before the start of construction in January 1994 to measure porewater pressure, surface and subsurface movement of the retained ground. During wall installation in late 1994 instrumentation was incorporated in the diaphragm wall panels to measure lateral movements and bending moments. The last phases of instrumentation occurred in March and May 1995 when strain gauges were installed to monitor the axial loads in the roof slab and carriageway prop slab respectively.

A plan and section showing the layout of instrumentation are given in Figs 3 and 4 respectively.

## 4.1 MEASUREMENT OF POREWATER PRESSURES

Porewater pressures were measured in a 10m deep borehole sunk in the retained ground at 2m from the wall. The borehole accommodated three pneumatic piezometers with high air entry tips located at depths of 4, 7 and 10m. Each tip was encased in a nominal 200mm long cell of pluviated coarse sand with the remainder of the borehole sealed with bentonite pellets.

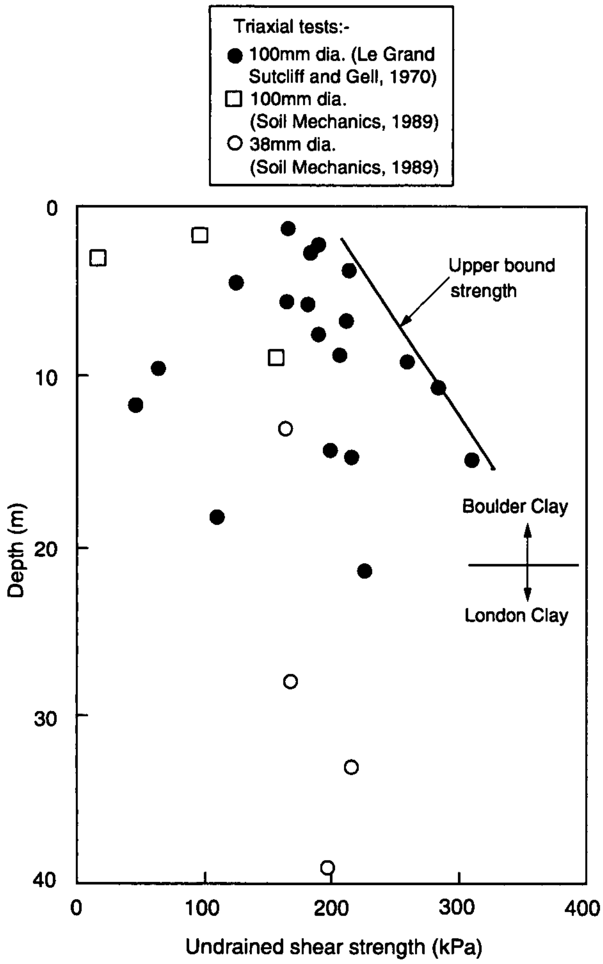



**Fig 2. Variation of undrained strength with depth**

## 4.2 MEASUREMENT OF SURFACE MOVEMENT

Surface movements of the retained ground were measured from anchor stations installed at distances of 1.9m, 9m, and 19m from the wall. Each station comprised a stainless steel shaft with a precision-machined threaded stub at the top and a rebar welded to the bottom. Installation involved excavating a 0.5m deep hole beneath the ground surface, placing the station in the hole and backfilling with firmly-tamped concrete. The stations were then protected by conventional inspection covers.

These stations were designed to receive both the invar staff for settlement measurements using precise levelling and also an extension which allowed a tensioned tape extensometer to be attached to measure changes in lateral movement between adjacent stations. Tape extensometer readings were corrected for temperature effects and absolute lateral movement of each station determined with reference to the station most remote from the construction work.

## 4.3 MEASUREMENT OF SUBSURFACE MOVEMENT

A plastic inclinometer access tube was installed to determine horizontal deflections normal to the wall using a uniaxial inclinometer probe. The tube was located in the retained ground at 1.9m behind the wall and was founded in stiff London Clay at a depth of 25m, i.e. 3m below the wall toe.

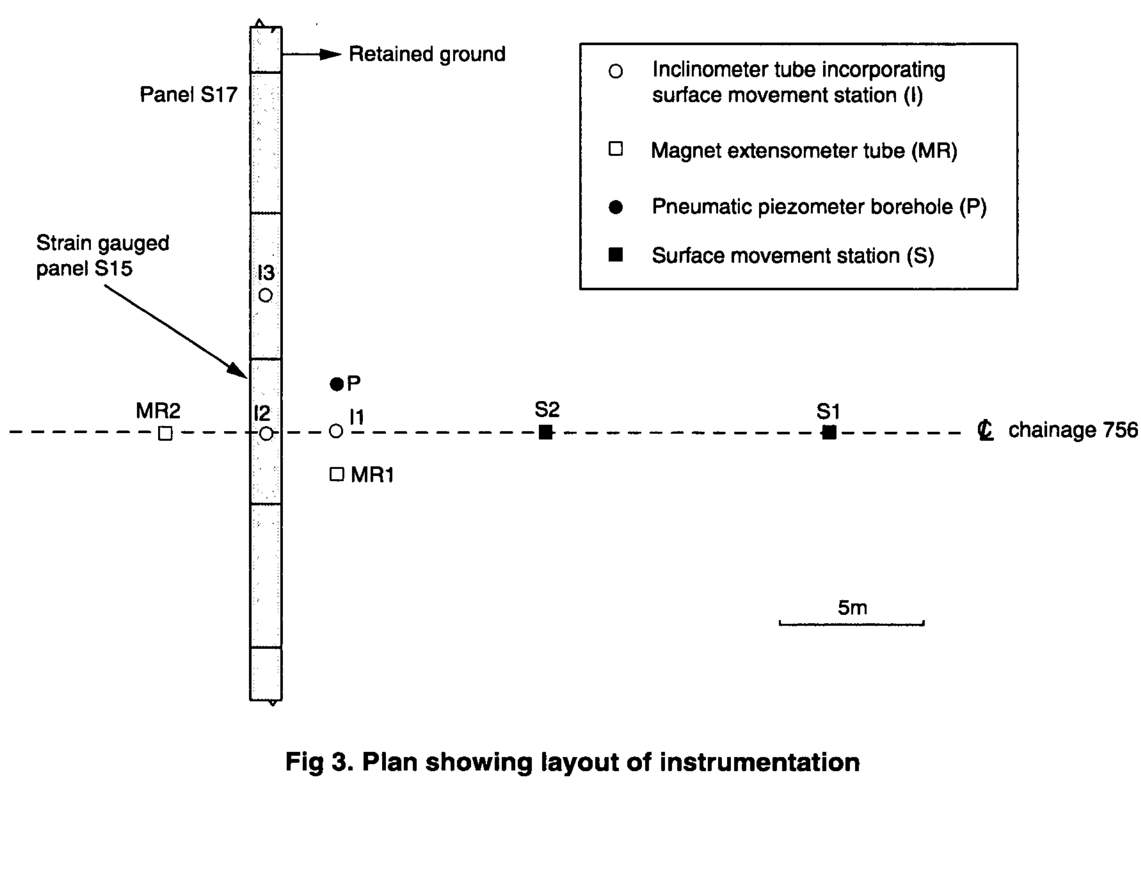



**Fig 3. Plan showing layout of instrumentation**

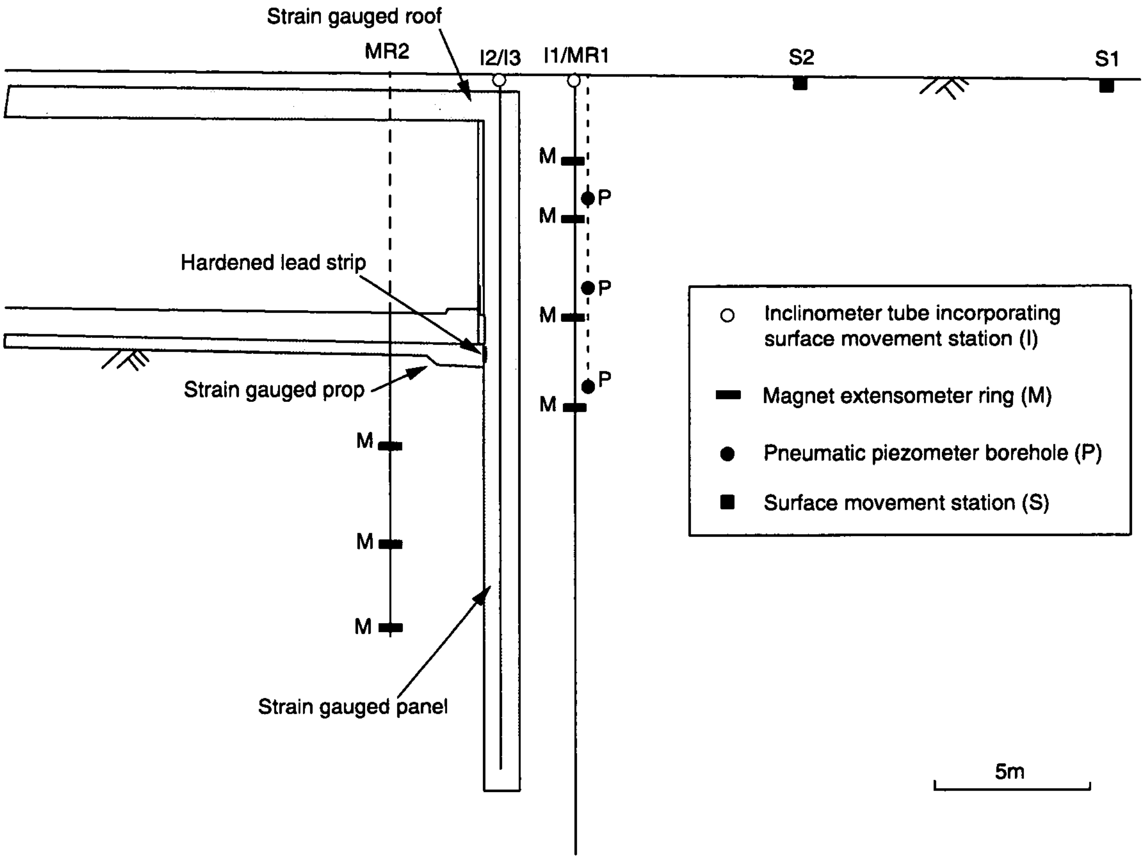



**Fig 4. Composite section showing layout of instrumentation**

The top of the inclinometer tube was terminated in a concrete block into which a ground anchor station was cast, thus allowing the apparent movement of the tube top to be verified by tape extensometer measurement. Settlement of the tube was monitored by incorporating the top rim in the precise levelling schedule.

Vertical subsurface movement was monitored by using magnetic settlement rings in each of two boreholes. Four rings at various depths were accommodated in the first borehole at 1.9m behind the wall, and three rings beneath the tunnel carriageway in a borehole at 3.2m in front of the wall. The precise depths of the rings were determined by lowering a single reed-switch probe attached to a GRP measuring tape down an access tube, inducing a sound signal when passing the rings. Depths were corrected for changes measured by precise levelling on the access tube tops so that absolute vertical movements could be determined.

## 4.4 WALL INSTRUMENTATION

Two steel ducts of nominally 100mm diameter were welded in a vertical position to the reinforcing cages and concreted into adjacent diaphragm wall panels S15 and S16. When wall installation was complete, plastic inclinometer access tubes were grouted into the ducts using a high strength cement slurry. Inclinometer surveys on these tubes were carried out at regular intervals during and after construction. A ground anchor station was also installed immediately adjacent to the inclinometer tube in panel S15 so that absolute movement of the panel could be verified.

Nine pairs of vibrating wire embedment strain gauges, each gauge incorporating a thermistor for temperature measurement, were attached along the reinforcing cage of panel S15. One gauge of each pair was positioned at the back and one at the front of the cage so that both bending and axial strains could be determined. From the bending strains, wall bending moments were calculated using the flexural rigidity (EI) of $6.42 \times 10^6$ kN/m$^2$ per metre run of the wall. This rigidity was calculated assuming that the concrete would remain uncracked at the small strain levels involved.

## 4.5 ROOF AND CARRIAGEWAY PROP INSTRUMENTATION

Axial loads and bending moments developed in the integral roof slab were measured using three pairs of vibrating wire embedment strain gauges installed at the top and bottom of the reinforcing cage at distances of 1.02m, 1.92m and 2.98m from the wall (Fig 4).

Similarly four pairs of embedment strain gauges were installed to measure loads in the carriageway prop slab at 0.9m and 1.6m from the wall, two pairs at each distance.

# 5. CONSTRUCTION SEQUENCE

The cut-and-cover tunnel was designed by Gifford Graham and Partners, who also supervised the construction on behalf of the London Regional Office of the Department of Transport. The main contractor was Edmund Nuttall Ltd, who sub-contracted the diaphragm wall excavation to Taylor Woodrow Ltd. Bulk earthmoving was undertaken by London Haulage Ltd. Table 1 gives the dates of each of the main stages of construction in the instrumented area.

## 5.1 DIAPHRAGM WALL INSTALLATION

After the alignment of the wall was set out, concrete guide walls were cast to aid the accurate excavation by grab of the trench for the diaphragm panels (Fig 5). Throughout the period when the trench was open, a bentonite slurry was used to provide support. When excavation was complete, the reinforcing cage was lowered into the trench and concrete tremied to the bottom. The displaced bentonite slurry was pumped back into storage for reuse. Generally each diaphragm panel was constructed within a two day cycle (Table 1). The TRL instrumented wall panels were 22m deep, 5m wide and 1m thick.

## 5.2 TUNNEL CONSTRUCTION

After completion of the diaphragm wall, temporary sheeting was installed on the retained side of the wall to support a 1.5m deep trench which provided access for panel trimming. Shortly afterwards, excavation to a depth of 3.5m was carried out in front of the wall and a scaffold platform constructed to support the formwork for the 5m wide bays of the reinforced concrete roof. Reinforcement was continuous between the diaphragm wall panels and deck to provide an integral structure. The roof slab in the instrumented area was cast in March 1995 (Table 1).

Excavation to formation level (about 9m depth) took place during May 1995 in the instrumented area. The excavation was completed with the tunnel roof alone providing the support. The permanent prop slabs were constructed between the tunnel walls in 10.5m wide bays as excavation progressed. As shown in Fig 4 the thickness of this reinforced concrete slab was generally 0.5m, although close to the walls it increased to 1m. The slab was cast insitu against 7.5mm hardened lead strips installed on the wall edge beams. This joint between the prop slab and the wall was designed to transmit axial load to the wall whilst accommodating any rotations produced by long term heave of the underlying clay.

## TABLE 1

Construction sequence at the instrumented area

| Date | Day No. | Construction event |
|---|---|---|
| 23 Aug 1994 | 0 | TRL instrument datum |
| 5 Oct 1994 | 43 | Neighbouring panel S14 excavated |
| 6 Oct 1994 | 44 | Panel S14 cast |
| 13 Oct 1994 | 51 | Start of panel S15 excavation |
| 14 Oct 1994 | 52 | Panel S15 excavated |
| 15 Oct 1994 | 53 | Panel S15 cast |
| 1 Dec 1994 | 100 | Neighbouring panel S16 excavated |
| 2 Dec 1994 | 101 | Panel S16 cast |
| 27 Jan 1995 | 157 | Sheeting installed to support a 1.5m deep access trench on retained side for panel trimming |
| 21 Feb 1995 | 182 | Start of excavation to 3.5m |
| 22 Feb 1995 | 183 | Excavation to 3.5m completed |
| 9 Mar 1995 | 198 | Roof slab cast |
| 2 May 1995 | 252 | Excavation to formation level |
| 23 May 1995 | 273 | Prop slab cast |
| 29 Jun 1995 | 310 | Sheet piles removed from access trench after backfilling |

After installation of the carriageway slab, a cladding was attached to the wall and the tunnel road construction completed (Fig 6).

# 6. OBSERVATIONS

## 6.1 DIAPHRAGM WALL INSTALLATION

Fig 7 shows the surface lateral movements measured at 1.9m away from the wall at the location of inclinometer I1. Movements were determined both from tensioned tape

**Fig 5. Excavation of the trench for a diaphragm panel**

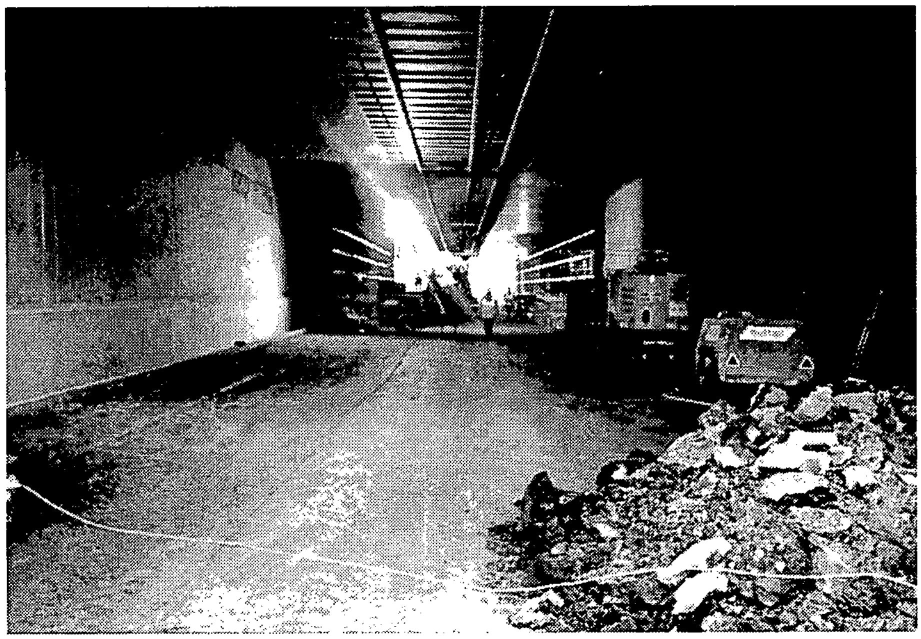


**Fig 6. Road construction within the tunnel**

extensometer measurements and from inclinometer surveys assuming base fixity of the tube. The agreement between results from the two techniques was reasonable. The most significant movement was measured immediately after the excavation for the nearest panel S15 when ground movement of about 5mm towards the excavation occurred (Fig 7). Installation of the adjoining panels S14 and S16 had no measurable effect.

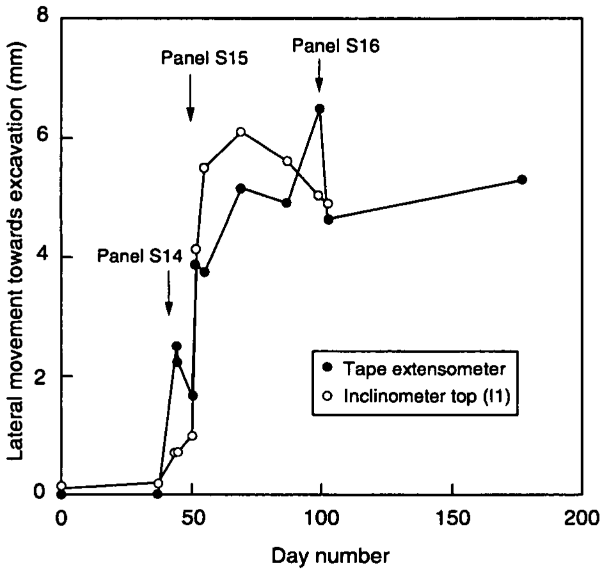



**Fig 7. Surface lateral movement at 1.9m away during wall installation**

Precise levelling of the stations 1.9m behind the wall revealed only small settlements of between 1mm and 2mm at the end of wall installation. Over the same period there were no measurable lateral or vertical movements on station S1 at 9m from the wall.

The profile of lateral movement with depth measured on inclinometer I1 shortly after installation of panel S15 is shown in Fig 8. Movements were generally small and, as would be expected, their magnitude reduced with depth. Subsurface settlements recorded on the magnet extensometer rings, located at 1.9m from the wall at depths between 2.5m and 10.5m, showed settlements of no more than 1mm.

The variation in piezometer measurements during diaphragm wall installation is shown in Fig 9. It should be noted that, although the depths of the piezometer tips varied between 4m and 9.7m, there was only about 1m difference in the initial range of heads with the largest measured on the shallowest piezometer. This is attributable to the perched water tables at this site caused by the presence of numerous sand lenses, identified during the site investigations (Frank Graham Geotechnical, 1989). The piezometer at 7m depth showed a porewater pressure drop of about 13kPa during excavation for panel S15, but the adjacent piezometer measurements at 4m and 9.7m depth remained largely

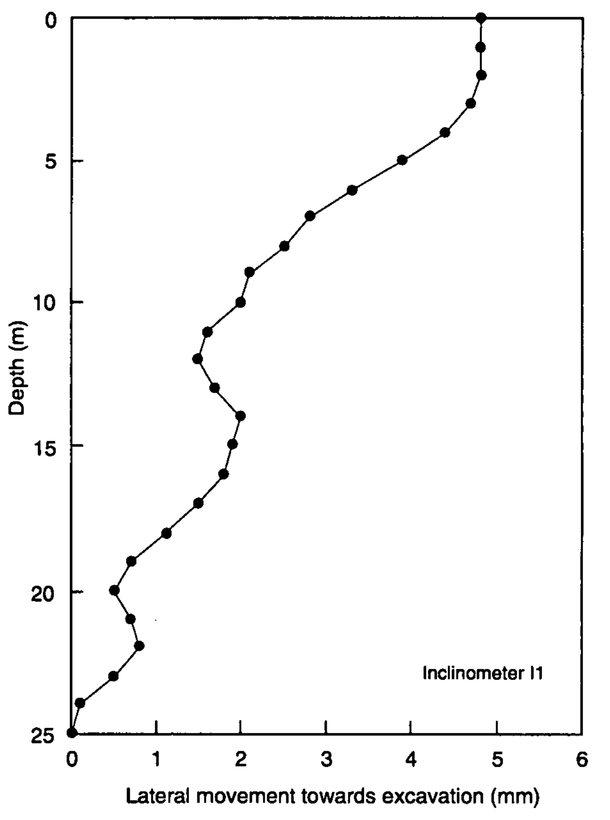



**Fig 8. Subsurface lateral movement at 1.9m away during wall installation**

unchanged. The original value was rapidly restored after the casting of the panel, reflecting a pattern common to sites where diaphragm walls are installed in clay (Symons and Carder, 1992).

## 6.2 TUNNEL CONSTRUCTION

### 6.2.1 Wall and ground movements

Lateral movements of the wall and retained ground measured using the inclinometer system during various stages of construction are shown in Fig 10. Wall movements were monitored on inclinometer tubes I2 and I3 in panels S15 and S16 respectively; ground movements were monitored on tube I1 at a distance of 1.9m behind the wall. The results in Fig 10 are calculated assuming base fixity of each inclinometer tube; the validity of this assumption is discussed later.

An initial excavation to a depth of 3.5m to facilitate construction of the roof slab began at the instrumented section on 21 February 1995 (Day 182) and was completed the following day. During this operation the wall cantilevered towards the excavation and both inclinometers tubes, I2 and I3, indicated a lateral movement of about 3mm at the top. Ground inclinometer tube I1 revealed similar behaviour although only about 1mm surface movement was recorded.

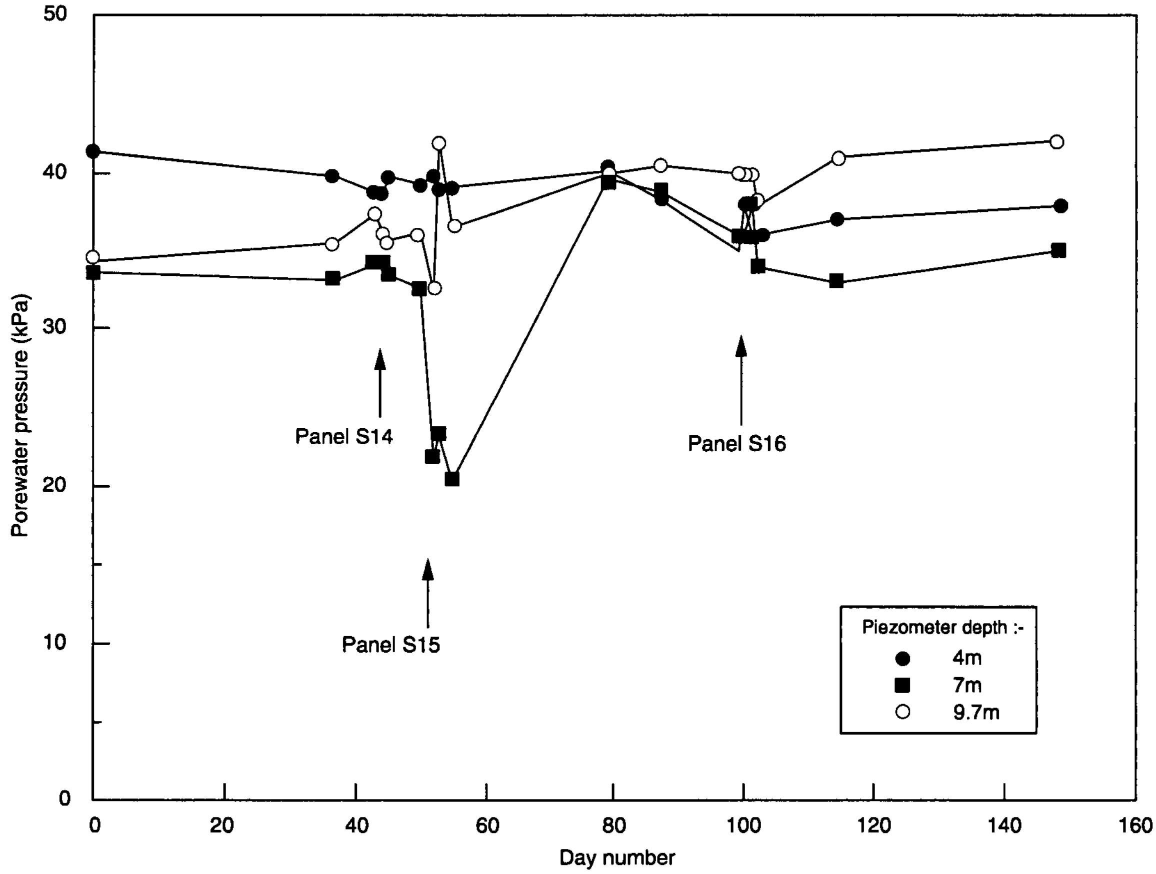



**Fig 9. Porewater pressures measured at 2m away during wall installation**

After installation of the roof slab, excavation to formation level at a depth of 7.5m was carried out on 2 May 1995 (Day 252). During this excavation phase, hardly any further movement was recorded at the top of the wall because of the propping action of the roof slab. However an additional movement of about 2.5mm was recorded at about 7m depth, i.e. just above dredge level. A further lateral movement of about 1mm was measured over the upper 10m of the ground inclinometer tube at this time (Fig 10). Inclinometer readings taken over the following 4 months gave near identical results confirming that no further lateral movement had occurred.

Fig 11 compares the surface lateral movement at 1.9m away from the wall as determined from tape extensometer measurements and from inclinometer surveys assuming base fixity of the tube. Generally results using the two techniques agreed to within about 0.5mm up to completion of excavation to formation level. This confirmed that no significant movement of the base of the inclinometer tube had occurred. Subsequent readings showed slightly larger differences of about 1mm but these may have been caused by ground disturbance during the removal of sheet piling used to shore-up the retained side of the 1.5m deep trench providing access for roof slab construction.

Precise levelling during tunnel construction showed only small settlements of less than 1mm on the surface station at 1.9m away from the wall and no discernible settlements further away. Readings from the magnetic rings in borehole MR1 (Fig 4) at the same distance behind the wall remained virtually unchanged throughout and confirmed that there was little or no subsurface settlement. No readings were available from the magnetic ring borehole (MR2) installed in the carriageway area because of excavation and construction activity until 16 May 1995 (Day 266), when a heave of between 10mm and 12mm was recorded from all three depths (about 4m, 7m and 10m below formation level). The magnitude of the heave was consistent with that recorded in an earlier study and reported by Carswell et al (1993).

### 6.2.2 Porewater pressures

The variation of porewater pressures measured in the retained ground at 2m away during tunnel construction is shown in Fig 12. Little change was recorded on the piezometer at 4m depth which indicated that the perched water table near to the ground surface persisted even after excavation to formation level. However a fall in porewater pressure of about 25kPa was recorded at depths of 7m and 9.7m after bulk excavation.

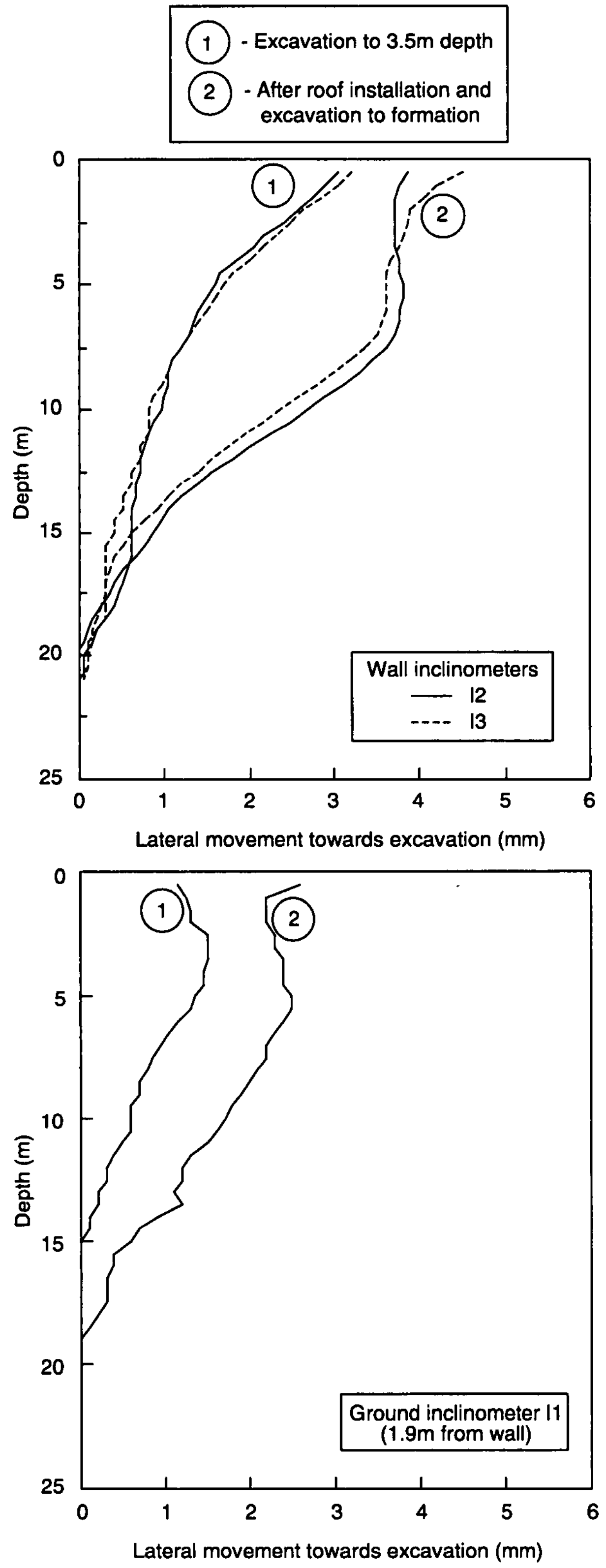



**Fig 10. Wall and ground movements during tunnel construction**

### 6.2.3 Wall bending moments and prop loads

The development of wall bending moment during tunnel construction is shown in Fig 13. Excavation in front of the wall to 3.5m depth prior to construction of the roof, which occurred at about the same time as an excavation to 1.5m depth behind the wall for access purposes, produced very little change in bending moment as shown in Fig 13a. Casting of the roof slab induced an increase in moment of about 150kNm/m. This can be compared with the upper bound moment of about 350kNm/m calculated for the uniformly distributed load of the concrete deck acting over the 13.7m span between the walls and assuming fixed end support.

After excavation to about 9m depth, axial load developed in the roof slab and induced a maximum bending moment of 220kNm/m at 5.5m depth approximately midway between the tunnel roof and dredge level (Fig 13b). An additional moment of 250kNm/m was later induced at the top of the wall by the placement of about 1m of spoil on the tunnel roof. This change was consistent with the 265kNm/m calculated assuming fixed end support of the roof by the walls. These values also agreed well with the bending moment changes shown in Fig 14 which were measured using pairs of strain gauges in the roof slab. Spoil placement resulted in a change of nearly 200kNm/m in the roof moment at a distance of 1m from the wall, with changes at 2m and 3m away reducing progressively.

Fig 15 shows the development of axial load in the tunnel roof and indicates that the magnitudes of load at distances of 1m, 2m and 3m from the wall were very similar. As would be anticipated the load increased as excavation to formation progressed: by completion a mean load of nearly 1000kN/m was measured, a value that remained reasonably stable over the next 6 months.

Only small strains were measured in the permanent structural slab of the tunnel carriageway during the initial 6 months after slab installation. Equivalent compressive axial loads calculated from the strain gauge pairs at the four locations were in the range of +57 to -118kN/m. Over this period there was no indication of any increase in the prop load. Although the tunnel was designed as a doubly-propped structure, the integral roof and the depth of wall penetration appeared sufficient to provide short term support. However it is anticipated that load in the carriageway prop will develop in the longer term as softening and swelling of the stiff clay beneath the carriageway occurs; longer term monitoring will be necessary to confirm this aspect.

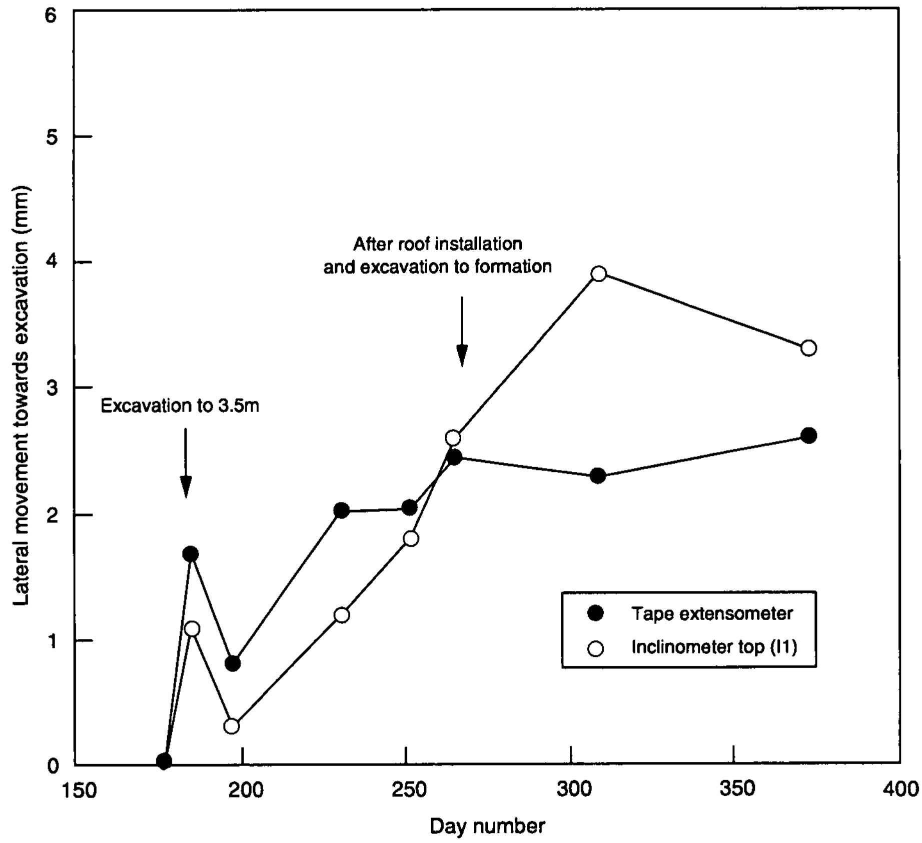



**Fig 11. Surface lateral movement at 1.9m away during tunnel construction**

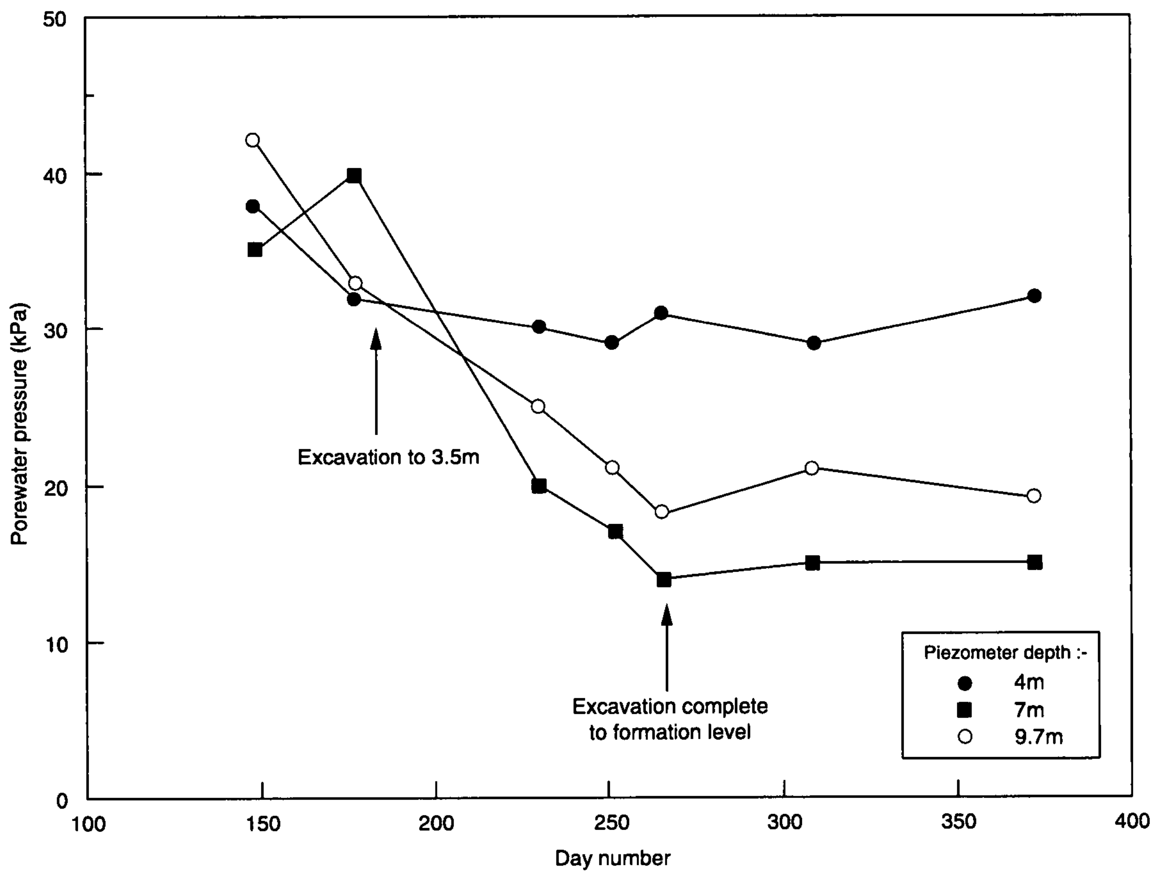
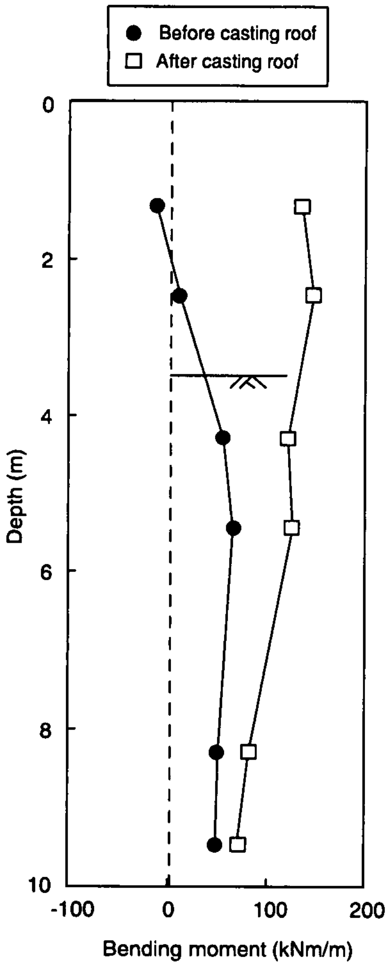
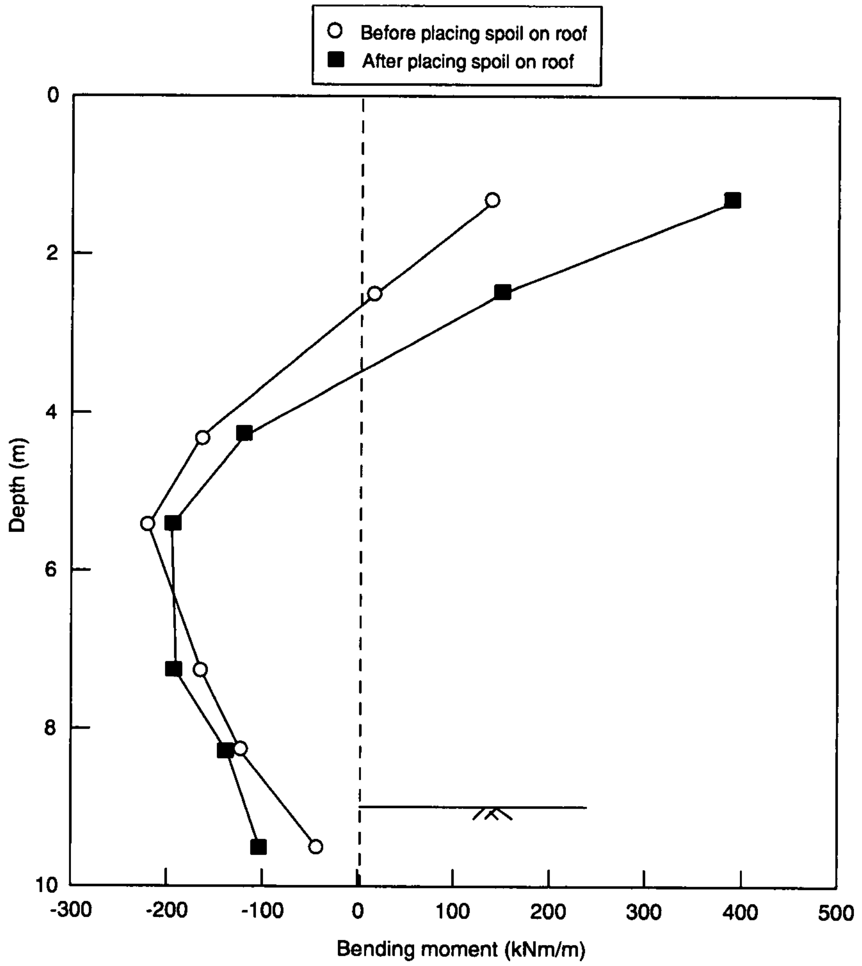



**Fig 12. Porewater pressures at 2m away during tunnel construction**




**(a) After excavation to 3.5m**




**(b) After excavation to 9m**

**Fig 13. Development of wall bending moment**

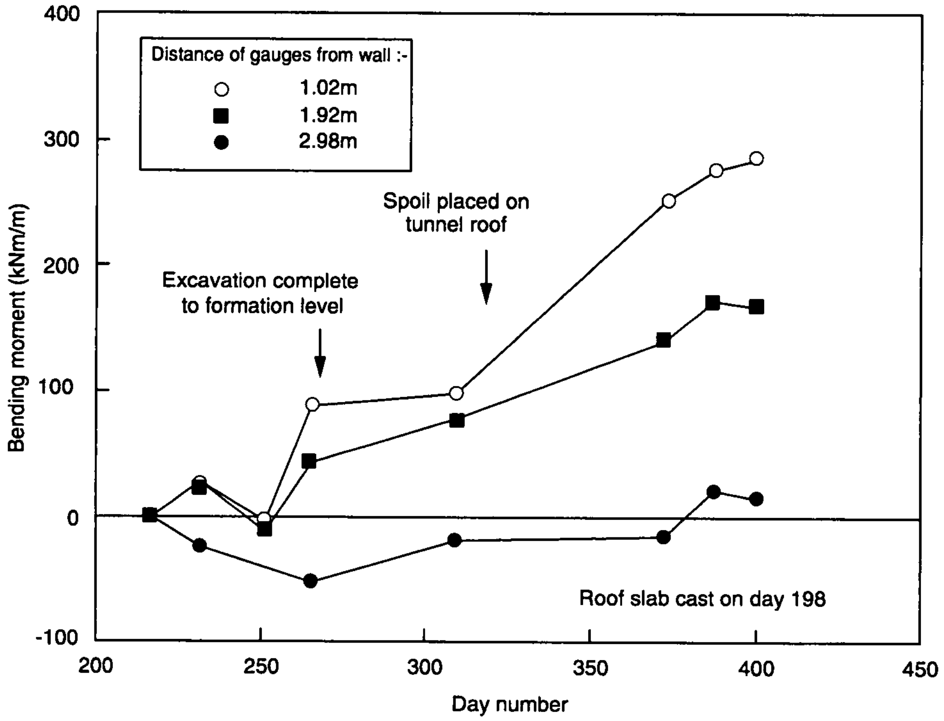



**Fig 14. Development of bending moment in the tunnel roof**

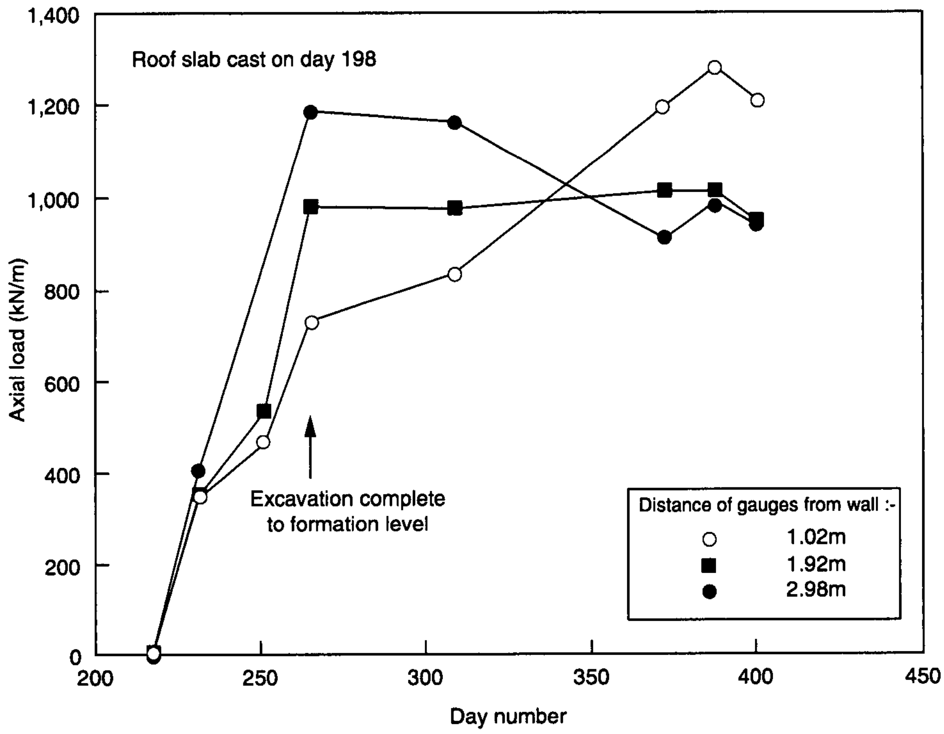



**Fig 15. Development of axial load in the tunnel roof**

# 7. DESIGN IMPLICATIONS

At this cut-and-cover tunnel scheme the roof slab was constructed integrally with the diaphragm walls and measurements indicated that little or no load developed in the short term in the structural slab at carriageway level. For this stage, the analysis of overall stability can therefore be carried out in accordance with the principles of CIRIA Report 104 (Padfield and Mair, 1984) and BD42 (DMRB 2.1) for walls founded in stiff clay and propped near the top. Table 2 shows the factors of safety determined on this basis and compares the design roof loads with those measured. For the purpose of these calculations, hydrostatic water pressure distributions were assumed from 3m depth on the retained side and from beneath the prop slab on the carriageway side. Linear seepage around the wall was considered unlikely in the short term as the diaphragm wall was embedded 2m into the low permeability London Clay. Moderately conservative soil parameters of Ø'=26° and c'=6kPa were used for the glacial till and values of Ø'=25° and c'=20kPa for the underlying London Clay (Section 3). Cohesion values were reduced to zero when carrying out analyses based on worst credible strength parameters.

On this basis, in Table 2, the factors of safety using different methods for the ultimate limit state (ULS) of overall stability were all well in excess of the recommended values when employing both moderately conservative and worst credible soil parameters for permanent work design (CIRIA Report 104). This was not unexpected as wall penetration was deeper than required for stability reasons because of the design requirement for embedment into the London Clay to provide water cut-off. In the original design, factors of safety were also up to 15% lower as reduced values of wall friction were selected to allow for wall installation under bentonite.

Roof prop loads from the ULS calculations ranged between 410 and 577kN/m and were considerably less than the measured value of 1000kN/m. BD42 considers serviceability limit state design of the structural elements to account for the higher earth pressures likely to exist under working conditions and results using this procedure are also given in Table 2. Assuming lateral stresses equivalent to K-values of 1 and 1.5 on the retained side of the wall, roof loads of 737kN/m and 2483kN/m respectively are then calculated. These design values are more in line with the strain gauge measurements. The original site investigation assessed the insitu lateral stress in the glacial till as corresponding to a K of 1.5, but as some stress relief will have occurred during wall installation this is considered to represent an upper bound.

The comparison of predictions and measurements of maximum bending moments over the retained wall height given in Table 2 indicates that measured values are approximately four times below those predicted. This discrepancy reflects the difficulty in predicting wall bending moments using soil strength parameters in limit equilibrium calculations which do not accurately model soil-structure interaction effects and the construction sequence. More realistic prediction of wall bending moments is better undertaken

## TABLE 2

Factors of safety, prop loads and bending moments from limit equilibrium calculations and measurements

| Method | | Factor of safety | Roof prop load (kN) | Max wall bending moment (kNm/m) | Depth to max moment (m) |
|---|---|---|---|---|---|
| CIRIA Report 104 and BD42 | CP2 Method | 2.30 | 410 | 2609 | 9.7 |
| Ultimate Limit State | Burland-Potts Method | 2.58 | 409 | 2596 | 9.7 |
| - Moderately conservative parameters | Factored strength Method | 1.68 | 555 | >3941 | >9.7 |
| CIRIA Report 104 and BD42 | CP2 Method | 1.90 | 473 | 2864 | 9.6 |
| Ultimate Limit State | Burland-Potts Method | 2.11 | 475 | 2875 | 9.6 |
| Worst credible parameters | Factored strength Method | 1.51 | 577 | 3868 | 10.9 |
| BD42 (Clause 3.10) Serviceability Limit State of structural elements | Force balance, K=1 on retained side | - | 737 | 3505 | 8.1 |
| - Worst credible parameters and δ=Ø' on excavated side | Force balance, K=1.5 on retained side | - | 2483 | >17384 | >9.3 |
| Mean measured values | | - | 1000 | 785* | 5.5 |

* For comparative purposes the measured value of 220kNm/m has been increased by 565kNm/m to account for the roof/wall moment connection.

Notes:- (i) Surcharge of 10kPa on retained side assumed for calculations.
(ii) ULS calculations use $\delta={}^{2}/_{3}Ø'$ and $c_w=0$ on retained side, $\delta={}^{1}/_{2}Ø'$ and $c_w=0$ on excavated side.

using finite element analysis (Ng and Lings, 1995; Watson and Carder, 1994).

In the longer term, load is expected to gradually increase on the carriageway prop slab as small wall movements occur owing to softening of the clay in front of the wall. This increase in carriageway prop load is likely to be accompanied by a commensurate decrease in roof load. Bulk excavation at this site was safely undertaken with roof support only. If temporary props had been employed at a lower level their removal would have been expected to have pre-loaded the carriageway prop to some extent (Richards and Powrie, 1994). This in turn would have probably resulted in smaller loads being measured in the roof prop. Further field studies are needed where temporary props are used in the construction sequence to fully validate this mechanism.

Richards and Powrie (1995) also carried out centrifuge modelling of doubly-propped retaining walls and found that, with deeper embedments, the bottom prop load will be reduced although top prop loads and bending moments will be increased. Generally their findings are consistent with the measurements of prop loads at this site, although the apparent magnitude of wall bending moment remains much lower than expected. Richards and Powrie also concluded that neither limit equilibrium methods nor the equivalent pressure diagrams proposed by Terzaghi and Peck (1967) are likely to give reliable estimates of prop load and may seriously underestimate loads in some situations.

# 8. CONCLUSIONS

Field instrumentation and monitoring was carried out to establish the behaviour of the diaphragm walls during construction of a cut-and-cover tunnel at Finchley on the North Circular Road (A406). The following conclusions were reached.

(i) During installation of the diaphragm wall panels only small movements of the retained ground were measured. Lateral movements of 5mm and vertical movements of no more than 2mm were recorded 1.9m from the wall.

(ii) An initial excavation to 3.5m depth was carried out to provide access for roof construction and during this operation the wall cantilevered towards the excavation with about 3mm movement occurring at the top of the wall. During bulk excavation the roof was effective in acting as a prop and hardly any further movement was measured at the top of the wall. However some additional lateral movement occurred at depth with a maximum overall movement of 4mm being recorded a few metres above dredge level. In the tunnel area, a ground heave of up to 12mm was recorded owing to the unloading caused by bulk excavation.

(iii) By completion of excavation a mean roof load of about 1000kN/m was measured and this value remained reasonably stable over the next 5 months. Over the same period, only small loads were measured in the permanent structural slab of the tunnel carriageway with no indication of any potential increase. Although the tunnel was designed as a doubly-propped structure, the integral roof and the depth of wall penetration appear sufficient to provide short term support. If temporary props had been employed at a lower level their removal would probably have pre-loaded the carriageway prop to some extent and also resulted in lower roof loads. Further monitoring is required to establish whether load increases in the carriageway prop in the longer term as softening and swelling of the stiff clay beneath the carriageway occurs.

(iv) Analysis of factors of safety for overall stability using limit equilibrium methods and assuming only a top prop gave values well in excess of the recommended values given in CIRIA Report 104. This was not unexpected as wall penetration was deeper than required for stability reasons because of the water cut-off requirement. Roof loads from these calculations ranged between 410 and 577kN/m and were considerably less than the measured 1000kN/m. Use of BD42 (DMRB 2.1) for serviceability limit state design of the structural elements gave loads more comparable to those measured.

(v) An assessment of wall bending moments indicated that measured values are approximately four times less than those predicted using soil strength parameters in limit equilibrium calculations. Better prediction of bending moments is expected from numerical methods which can model the construction sequence and wall stiffness more realistically.

# 9. ACKNOWLEDGEMENTS

The work described in this report forms part of the research programme of the Civil Engineering Resource Centre of TRL and was funded by Bridges Engineering Division of The Highways Agency (DOT). The authors wish to thank London Regional Office of DOT for permission to carry out this study and also Mr G H Alderman, Mr S N Bennett, Mr P Darley and Mr M D Ryley of TRL for their assistance.

The scheme was designed by Gifford Graham & Partners (Director: Mr E Hollinghurst, Project Manager: Mr M J Ramsey). The assistance on site of Mr J W Dodman, Mr C J Bodley and Miss C Johnston of Gifford Graham & Partners, and Mr J B Hancock, Mr G H Brookes and Miss T M Ashford of Edmund Nuttall Ltd is also gratefully acknowledged.

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