TRANSPORT RESEARCH LABORATORY

TRL REPORT 244

BEHAVIOUR OF A CANTILEVER CONTIGUOUS BORED PILE WALL 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 E457A/BG, Improved Design of Retaining Walls.

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EXECUTIVE SUMMARY

For environmental reasons highways within built-up areas are increasingly being constructed below ground in retained cutting using bored pile or diaphragm wall techniques. Walls of this type can be constructed with high or low level props or anchors as methods of long term support, however the most basic form of construction is that of a cantilever wall which relies on base fixity in firm ground for its stability. Case histories of the performance of embedded cantilever walls where wall movements and earth pressures have been monitored are available. However there is a paucity of data where the wall bending moments developed by this form of construction have also been monitored.

This report describes the field instrumentation and monitoring carried out to establish the behaviour of a cantilever contiguous bored pile wall constructed as part of the A406 North Circular Road Improvement Scheme between East of Falloden Way and East of High Road, Finchley. The geology of the site was predominantly boulder clay, a typically heterogenous glacial till comprising stiff clays interspersed with sand and gravel lenses. Measurements of wall movement and bending moment together with movements in the adjoining ground were monitored during the various construction stages and for a period of about 10 months after completion of construction.

During bulk excavation the wall cantilevered towards the excavation with a lateral movement at the top of the wall of about 5mm. Following excavation, the rate of increase of movement slowed with movements of 9mm and 11mm being measured after 4 months and 10 months respectively. Throughout the monitoring period there was no evidence of any movement of the wall toe. Ground heave monitored at 3.2m in front of the wall and at 4m depth below the carriageway reached 12mm after 5 months had elapsed following excavation.

Immediately after excavation to formation level, the soil responded in an undrained manner with bending moments of up to 225kNm/m being measured over the instrumented upper 11m of the wall. Measured moments at 10 months after excavation were significantly higher and increased with depth to a value of about 400kNm/m just below final carriageway level.

The aim of this project is to provide advice that will be of value in updating BD 42 Design of embedded retaining walls and bridge abutments (DMRB 2.1).

BEHAVIOUR OF A CANTILEVER CONTIGUOUS BORED PILE WALL IN BOULDER CLAY AT FINCHLEY

ABSTRACT

For environmental reasons highways within built-up areas are increasingly being constructed below ground in retained cutting using bored pile or diaphragm wall techniques. Walls of this type can be constructed with high or low level props or anchors as methods of long term support: however the most basic form of construction is that of a cantilever wall which relies on base fixity in firm ground for its stability.

This report describes the field instrumentation and monitoring carried out to establish the behaviour of a cantilever contiguous bored pile wall constructed as part of the A406 North Circular Road Improvement Scheme between East of Falloden Way and East of High Road, Finchley. Measurements of wall movement and bending moment together with movements in the adjoining ground were monitored during the various construction stages and for a period of about 10 months after completion of construction.

1. INTRODUCTION

Over the past two decades increasing use has been made of embedded retaining walls of the diaphragm or bored pile type for the construction of roads in retained cutting or cut-and-cover tunnels. Although walls of this type can be constructed with high or low level props or anchors as methods of long term support, the most basic form of construction is that of a cantilever wall which relies on base fixity in firm ground for its stability.

Case histories of the performance of embedded cantilever walls where wall movements and earth pressures have been monitored are available (Garrett and Barnes, 1984; Clarke and Wroth, 1984; St John, 1975; Carder and Symons, 1984). However there is a paucity of data where the wall bending moments developed by this form of construction have also been monitored. Moran and Laimbeer (1994) investigated the moments developed in a cantilever diaphragm wall primarily retaining terrace gravels and founded in London Clay. This report gives details of a study undertaken at Finchley on the performance of a cantilever bored pile wall founded predominantly in boulder clay.

The study of the cantilever wall was undertaken concurrently with an investigation into the performance of the cut-and-cover tunnel at the same site: the latter results have already been reported (Brookes and Carder, 1996). The cantilever walls were used to flank the approaches to the tunnel, and comprised contiguous bored piling with 1.5m diameter, 24m long piles installed at 1.98m centres. The retained height of wall in the instrumented area was 4.75m.

The results from measurements of wall movement and bending moment are reported together with the movements developed in the adjoining ground. A back-analysis of the design was carried out and bending moment predictions compared with measured values.

2. LOCATION

The site is part of the A406 North Circular Road Improvement Scheme at the junction with East End Road, Finchley, London N3. The section instrumented by TRL is on the eastern approach to the cut-and-cover tunnel where the southern approach wall truncates the southern end of Briarfield Avenue. The location is shown in Fig 1 and is centred on contract chainage 820.

3. SOIL PROPERTIES

Typical borehole logs derived from site investigations in the instrumented area by TRL and at 17m away by Le Grand Sutcliff and Gell (1970) are compared in Fig 2. The tunnel and approaches traverse a boulder clay outlier underlain by London Clay at a maximum depth of 23m. The composition of the boulder clay (glacial till) was extremely varied, and included numerous sand and gravel lenses creating a complex of perched water tables.

Considerable variation of undrained shear strength with depth was obtained as shown in Fig 3, although upper bound strengths of the boulder clay were generally high approaching 300kPa at 15m depth. Mean plastic limits (PL), liquid limits (LL), plasticity indices (PI) and natural moisture contents are summarised in Table 1.

The heterogenous nature of the soil was reflected in the wide range of effective stress parameters determined from triaxial tests undertaken as part of the site investigation for the scheme (Frank Graham Geotechnical, 1989). The mean peak soil strength parameters adopted for the design are given in Table 1.

4. INSTRUMENTATION

Instrumentation of the retained ground was completed in the summer of 1994. During wall construction the follow-

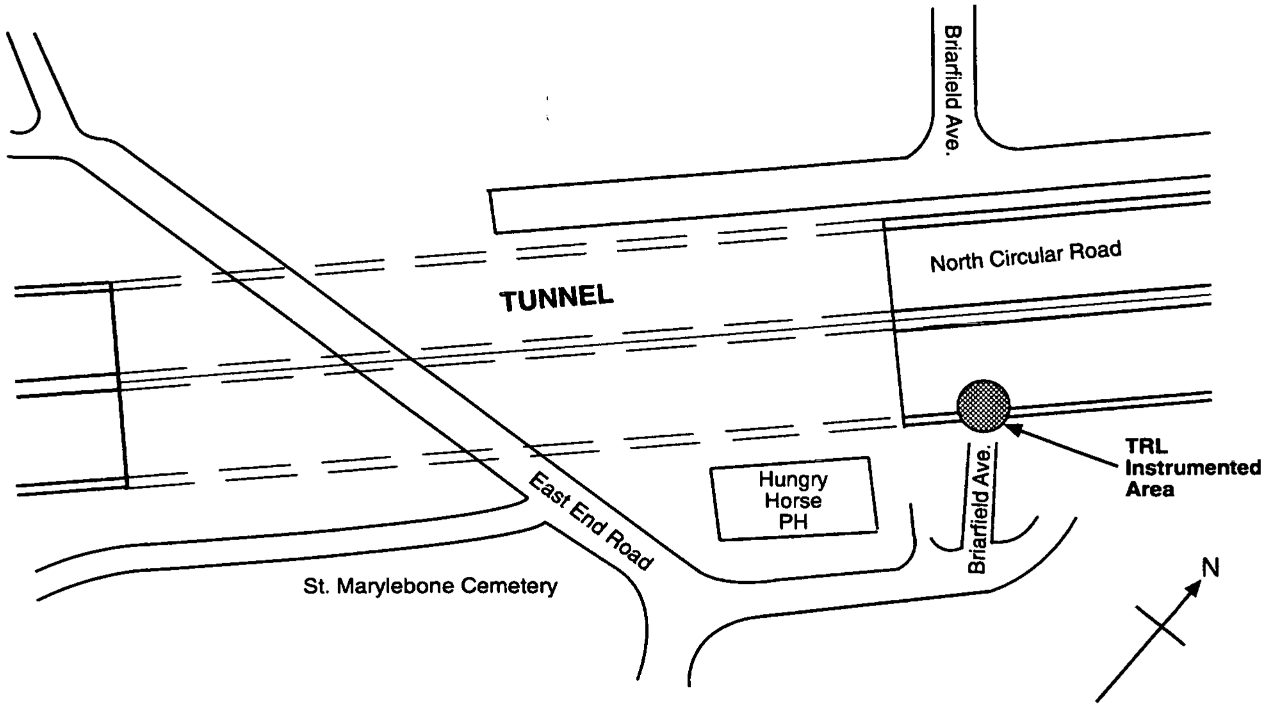

Fig 1. Site location plan (not to scale)

ing spring, three consecutive bored piles were instrumented to monitor lateral movement and bending moments. Schematic diagrams of the instrumentation layout are given in Figs 4 and 5.

4.1 MEASUREMENT OF GROUND SURFACE MOVEMENT

Stainless steel ground anchor stations were installed at distances of 1m, 6m and 11m from the wall to monitor surface movement of the retained ground. Each station included a precision-machined threaded stud at the top onto which an attachment could be fixed for tensioned tape extensometer measurements of lateral movement. All extensometer readings were adjusted for temperature effects and referenced to the station furthermost from the wall which was assumed to be stationary. Surface settlement caused by the construction was monitored by precise levelling using an invar staff on the same anchor stations.

4.2 MEASUREMENT OF GROUND SUBSURFACE MOVEMENT

Lateral movements normal to the wall in the retained ground were monitored from a single inclinometer tube (I1 in Fig 4) using a uniaxial 0.5m inclinometer probe. The tube was installed in a borehole sunk 1m behind the wall to a depth of 27m, approximately 3m beyond the toe of the wall. Verification of the apparent movement of the top of the tube calculated from inclinometer surveys, which assume base fixity, was obtained from tape extensometer measurements made using an anchor station located in a concrete block cast around the top of the tube.

Magnetic settlement rings (M in Fig 5) located in boreholes on either side of the wall were used to determine vertical subsurface movements. The borehole in the retained ground 1m behind the wall accommodated four rings at various depths between 3m and 10m. The second borehole 3.2m in front of the wall accommodated one ring at about 4m depth below the finished carriageway. The precise depth of each ring was determined by lowering a probe incorporating a reed switch down a central access tube. As the reed switch senses the position of the magnetic ring, its depth below the top of the tube is recorded from the measuring tape on which the probe is attached. Absolute vertical movement of each ring was established by precise levels taken on the top of the access tube.

4.3 WALL INSTRUMENTATION

Two steel ducts of nominal 100mm diameter were attached to the reinforcing cages of two of the piles prior to their installation. After pile installation, plastic inclinometer access tubes were grouted into these ducts using a non-shrink cementitious grout. Two inclinometer tubes, I2 and I3, were installed at the locations indicated in Fig 4 to determine lateral movements. In each case, anchor stations were installed at the tops of the tubes so that absolute movements could be confirmed by tape extensometer measurements.

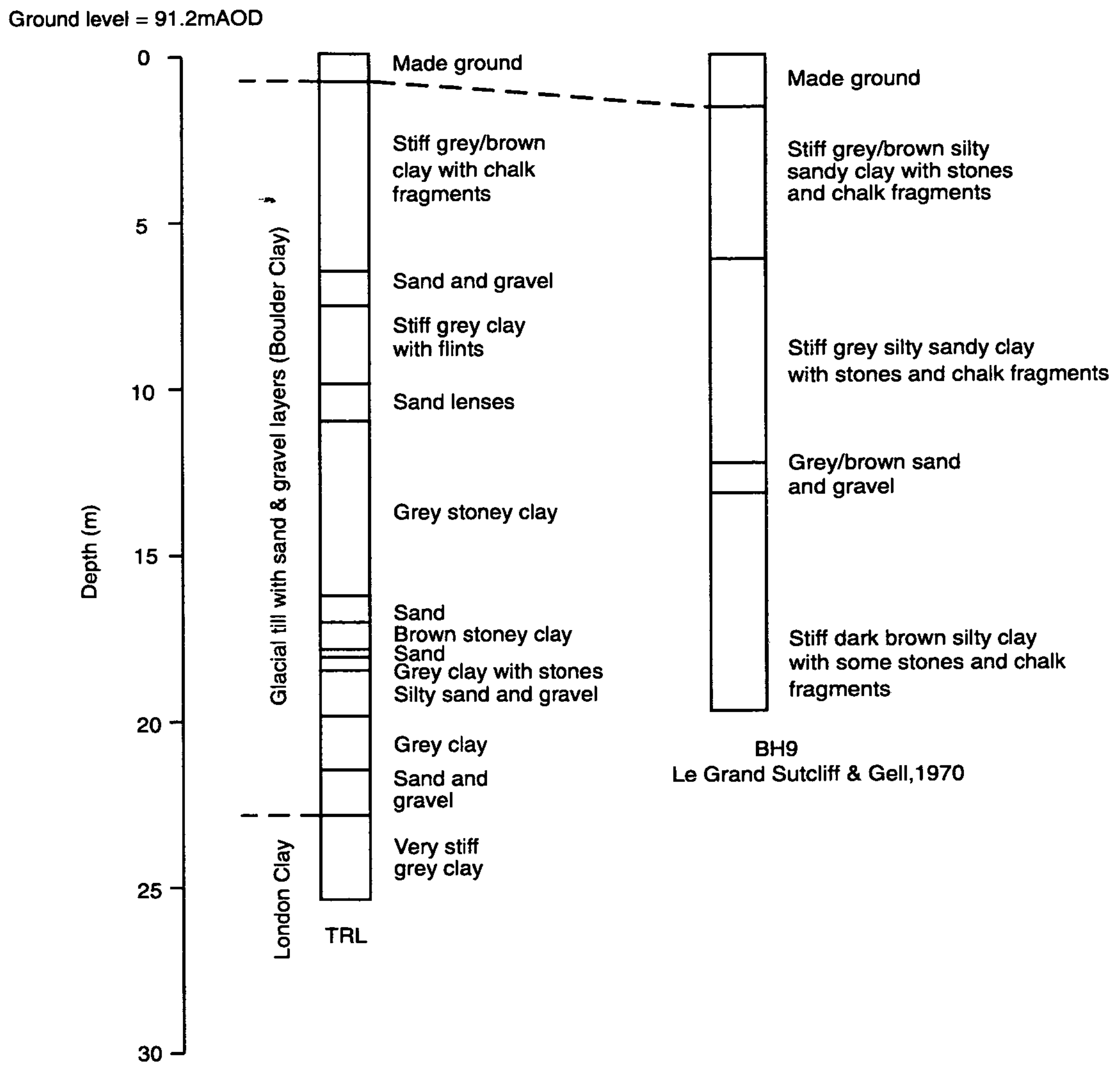

Fig 2. Soil profile with depth

The strains developed in one of the piles during construction of the tunnel approach were monitored using vibrating wire embedment strain gauges. A total of 18 gauges were fitted as diametrically opposed pairs to the front and rear of the reinforcing cage at intervals of depth reducing from 1.5m near the top to 0.9m at 11m. This procedure enabled both bending and axial strains to be determined. From the bending strains, wall bending moments were calculated per metre run of wall assuming the concrete would remain uncracked at the small strain levels involved.

5. CONSTRUCTION SEQUENCE

The tunnel and its approaches were designed by Gifford Graham and Partners, who also supervised the construction on behalf of the London Regional Office, Department of Transport. The main contractor was Edmund Nuttall Ltd, who sub-contracted the bored piling to Taylor Woodrow Ltd. Bulk earthmoving was undertaken by London Haulage Ltd. Table 2 gives a record of the main construction phases in the vicinity of the instrumented piles.

5.1 BORED PILE INSTALLATION

Contiguous bored piles of 1.5m diameter were installed at 1.98m centres to a depth of 24m in the instrumented area. The pile borehole was augered through a 5m stub casing and using a bentonite slurry to provide additional temporary support. After the reinforcing cage had been lowered into position, concrete was tremied to the bottom of the hole and the displaced slurry returned to storage.

On completion of a sequence of piles, the immediate area to the front and rear was excavated to a depth of 2m to facilitate breakout of the pile tops and construction of the capping beam (Fig 6).

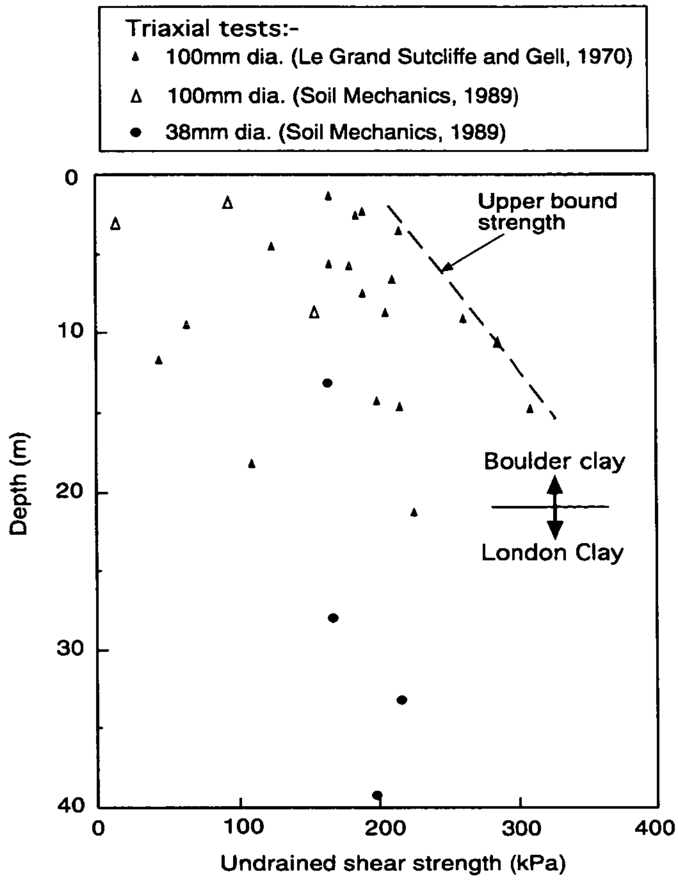

Fig 3. Variation of undrained strength with depth

TABLE 1

Soil parameters

Soil	Mean values PL (%)	Mean values LL (%)	Mean values PI (%)	Mean values Moisture content (%)	Design values ϕ_p' (°)	Design values c_p' (kPa)
Made ground	-	-	-	-	25°	0
Boulder clay	17	45	28	16	26°	6
London Clay	27	74	47	39	25°	20

TABLE 2

Construction sequence at the instrumented area

Date	Day No.	Construction event
30 Nov 1994	0	TRL instrumentation datum
22 Mar 1995	112	Bored pile 2/3 installed
29 Mar 1995	119	Bored pile 2/4 installed
4 Apr 1995	125	Bored pile 2/5 installed
10 Apr 1995	131	2m deep access trench dug on both sides of piles
15 May 1995	166	Excavation of central haul road
7 Jun 1995	189	Excavation to formation level
27 Nov 1995	362	Carriageway opened to traffic

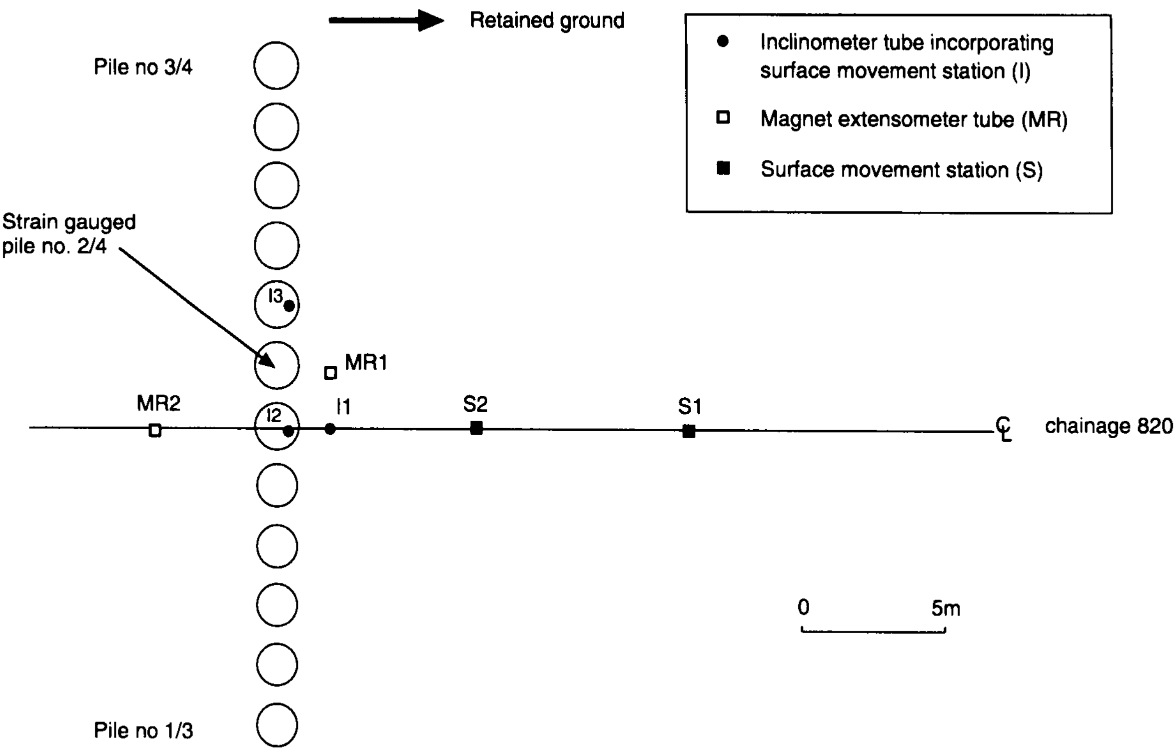

Fig 4. Plan showing layout of instrumentation

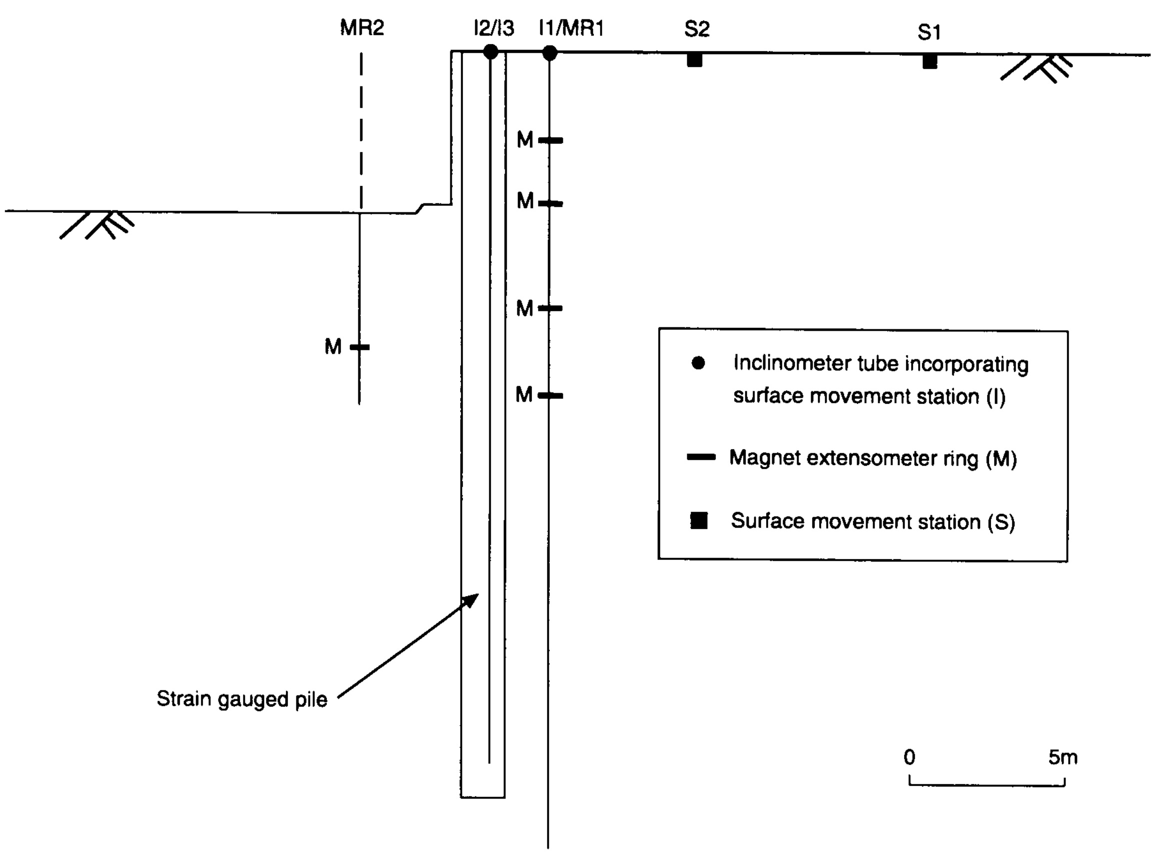

Fig 5. Composite section showing layout of instrumentation

Fig 6. Pile tops being trimmed to cut-off level

5.2 TUNNEL APPROACH EXCAVATION

The approach to the tunnel was excavated in two stages. Initially a haul road was excavated along the approach, leaving a 5m wide berm against the wall. Bulk excavation to maximum depth (average 5.2m in the instrumented area) occurred several months later following the completion of the capping beam along the entire approach.

After bulk excavation, the final road surface was constructed (Fig 7) and the road opened to traffic in November 1995.

6. OBSERVATIONS

6.1 BORED PILE INSTALLATION

Lateral movements of the ground caused by wall construction were measured using inclinometer tube I1 located 1m behind the wall, and are shown in Fig 8. Movement towards the wall was minimal; about 1mm was measured at the surface after installation of the nearest pile and a total of 1.7mm on completion of pile installation in the instrumented area. These values agreed closely with the change of 1.6mm measured by tape extensometer which is shown in Fig 9. The assumption of base fixity of tube I1 for the inclinometer surveys was therefore considered justified during wall installation.

Ground surface settlement was also small during this period, with precise levelling indicating settlements of no more than 1.5mm at 1m away from the piles. Measurements in the magnetic ring borehole MR1 showed subsurface settlements of less than 1mm, which were considered negligible. No readings were possible from MR2, located in front of the wall, as the access tube was buried beneath spoil at this stage.

6.2 TUNNEL APPROACH EXCAVATION

6.2.1 Wall and ground movement

The lateral movement profiles with depth recorded on inclinometer tubes I1 and I2 located in the retained ground and the wall respectively are shown in Fig 10. Measurements from pile inclinometer tube I3 are within 0.5mm of the results shown for inclinometer tube I2 at every stage of construction. The movements recorded on tube I1 are from 1 May 1995, the same datum date adopted for tubes I2 and I3 after wall installation.

In considering the overall ground movement recorded on tube I1, the movements caused by pile installation given in Fig 8 need to be added to those in Fig 10. A further 4mm movement was also measured after completion of the wall and before bulk excavation as indicated in Fig 9. However, this was a localised effect, caused by the excavation of a 2m deep trench to provide access for pile trimming. The excavation resulted in the top of tube I1 being left temporarily at the edge of the trench and the effects of this have consequently been omitted from the inclinometer results in Fig 10.

Fig 7. Carriageway construction in front of the wall

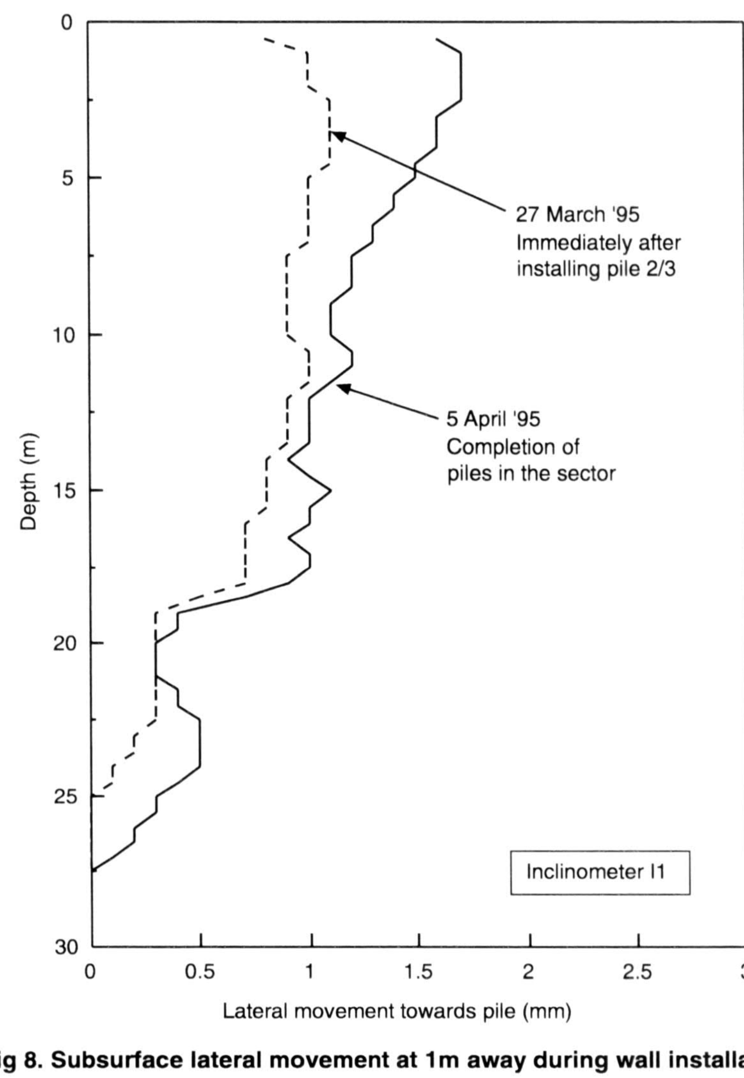

Fig 8. Subsurface lateral movement at 1m away during wall installation

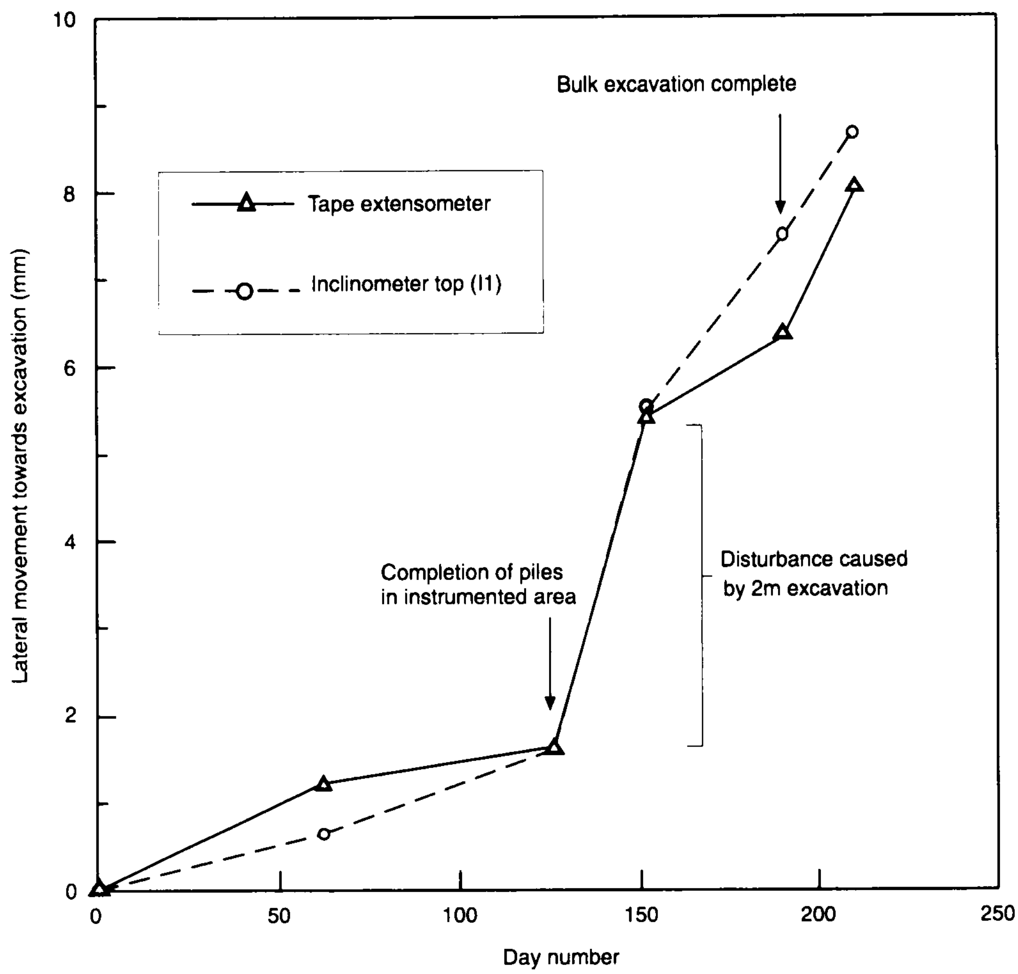

Fig 9. Surface lateral movement at 1m away during wall installation and bulk excavation

During bulk excavation in early June 1995 the wall cantilevered towards the excavation with a lateral movement at the top of the wall of about 5mm. This was accompanied by a ground surface movement of about 2mm measured at inclinometer tube I1. In both cases a further movement of about 1mm was recorded over the next three weeks. Movements recorded using the tape extensometer showed close agreement with the inclinometer survey results confirming that the assumption of base fixity for all inclinometer tubes was valid. This was also evident from the movement profiles in Fig 10 where no change in deflected shape was measured over the bottom 5m of each tube.

Further readings on ground inclinometer tube I1 were not possible after July 1995 when it sustained irreparable damage from heavy plant traffic. However later readings on inclinometer tube I2 showed overall movements at the top of the wall of 9mm and 11mm at 4 months and 10 months respectively after bulk excavation (Fig 10).

Precise levelling and measurements from the magnetic ring borehole MR1 showed that the excavation of the approach caused no significant (< 1mm) surface or subsurface settlements at 1m behind the wall (Fig 4). Measurement from MR2, the magnetic ring borehole 3.2m in front of the wall, indicated an overall heave of 11mm at 4m depth below the carriageway from before construction until 3 months after excavation. After a further 6 weeks, the ground heave at this location had increased to 12mm.

6.2.2 Wall bending moments

The development of wall bending moments at various times after the excavation of the tunnel approach is shown in Fig 11. Moments were determined from bending strains measured using the pair of gauges at each depth, based on pile flexural rigidities (EI) ranging from 7.3 to 8.3 x 10^6 kNm^2. The variation of EI values was commensurate with the changes in cross-sectional area of reinforcing steel with depth and was calculated assuming the concrete would not be cracked at the low strains recorded.

The initial stage of excavation to 2m on both sides of the wall had negligible effect, however excavation to final depth immediately induced bending moments of up to 225kNm/m over the instrumented upper 11m of the wall. In Fig 11, bending moments below 5m depth taken after July '95 were calculated assuming no change in axial strain because subsequent construction work had damaged the cables to the front gauge in those pairs.

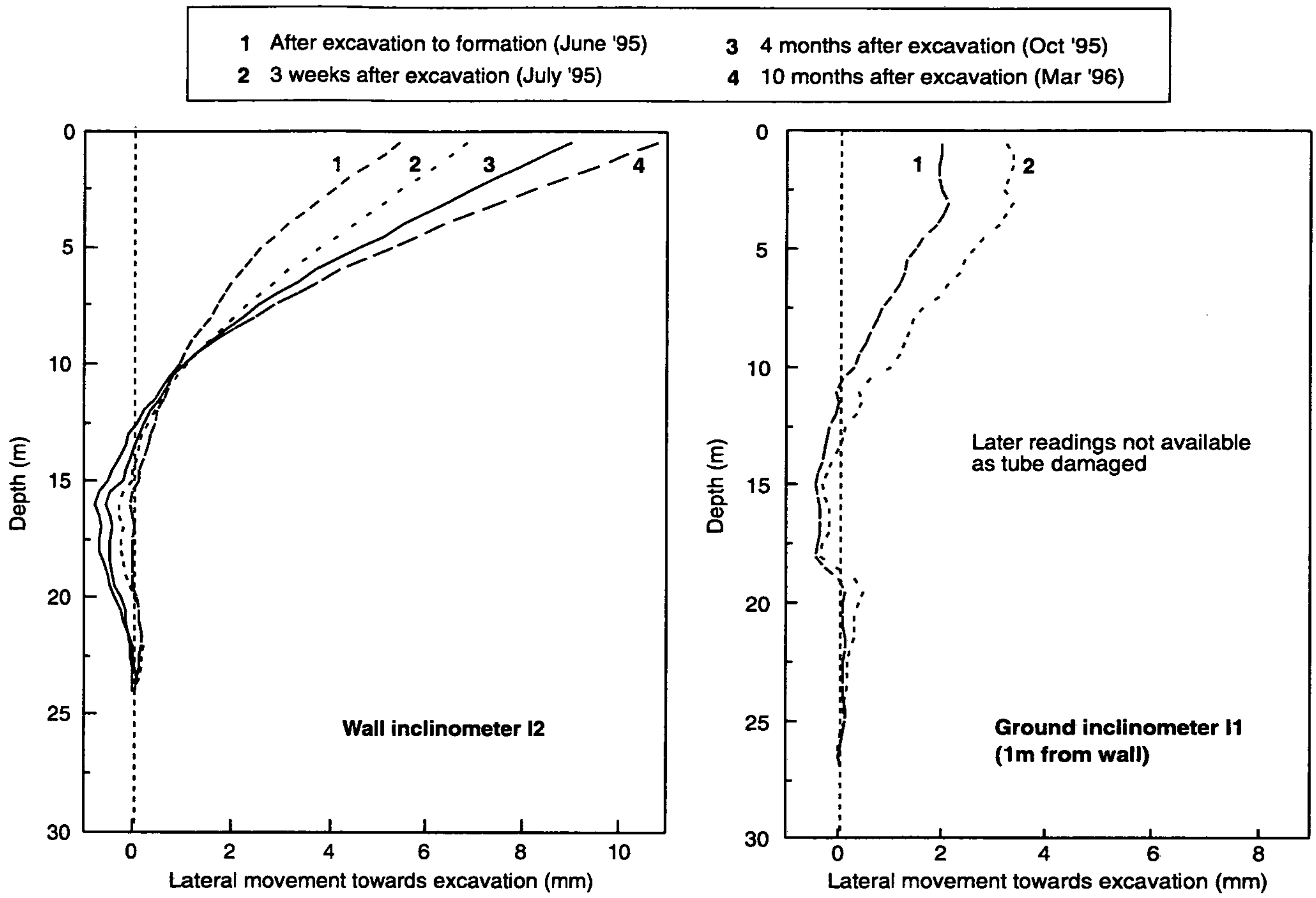

Fig 10. Wall and ground movements following bulk excavation

The results in Fig 11 show a general increase in bending moment over the 10 month period following excavation, the major part of this change occurring in the first 4 months. This suggested an associated increase in the lateral load on the retained height of wall possibly as drained soil behaviour started to occur following the initial undrained response to excavation. In Fig 11 the bending moments at 10 months after excavation increased with depth to a value of about 400kNm/m just below final carriageway level. Values then remained fairly constant with depth, but this may in part be a consequence of the assumption about axial strain described above. Although measurements were not available below 11m depth, bending moments would then be expected to decrease rapidly with depth. A comparison of measured and predicted bending moment profiles with depth is discussed in Section 7.

7. DISCUSSION

The stability of the wall was back-analysed and the resulting factors of safety are compared with the values currently recommended in CIRIA Report 104 (Padfield and Mair, 1984) for the design of embedded walls in stiff clay. The results of the analyses are given in Table 3 and have been calculated by reducing the actual depth of penetration of the wall below carriageway level by 20% to offset the increase applied in design to compensate for assuming base fixity. Hydrostatic water tables from 3m below the retained ground and 0.5m below final carriageway level were assumed throughout the analyses. The effect of water table depth on the stability of cantilever walls has been reported in detail by Symons and Kotera (1987).

A requirement of the original design included an allowance of 10kPa surcharge on the retained ground and a 1.8m deep unplanned excavation in front of the wall. On this basis the factors of safety indicated in Table 3 conformed closely to the recommended values given by CIRIA Report 104 and adopted by BD 42 (DMRB 2.1) when using limit equilibrium pressures based on both moderately conservative and worst credible soil strength parameters. Table 3 also gives results from the same analyses based on the final geometry and excluding the requirements of surcharge and unplanned excavation described above. As would be expected factors of safety were then well above the recommended safe values.

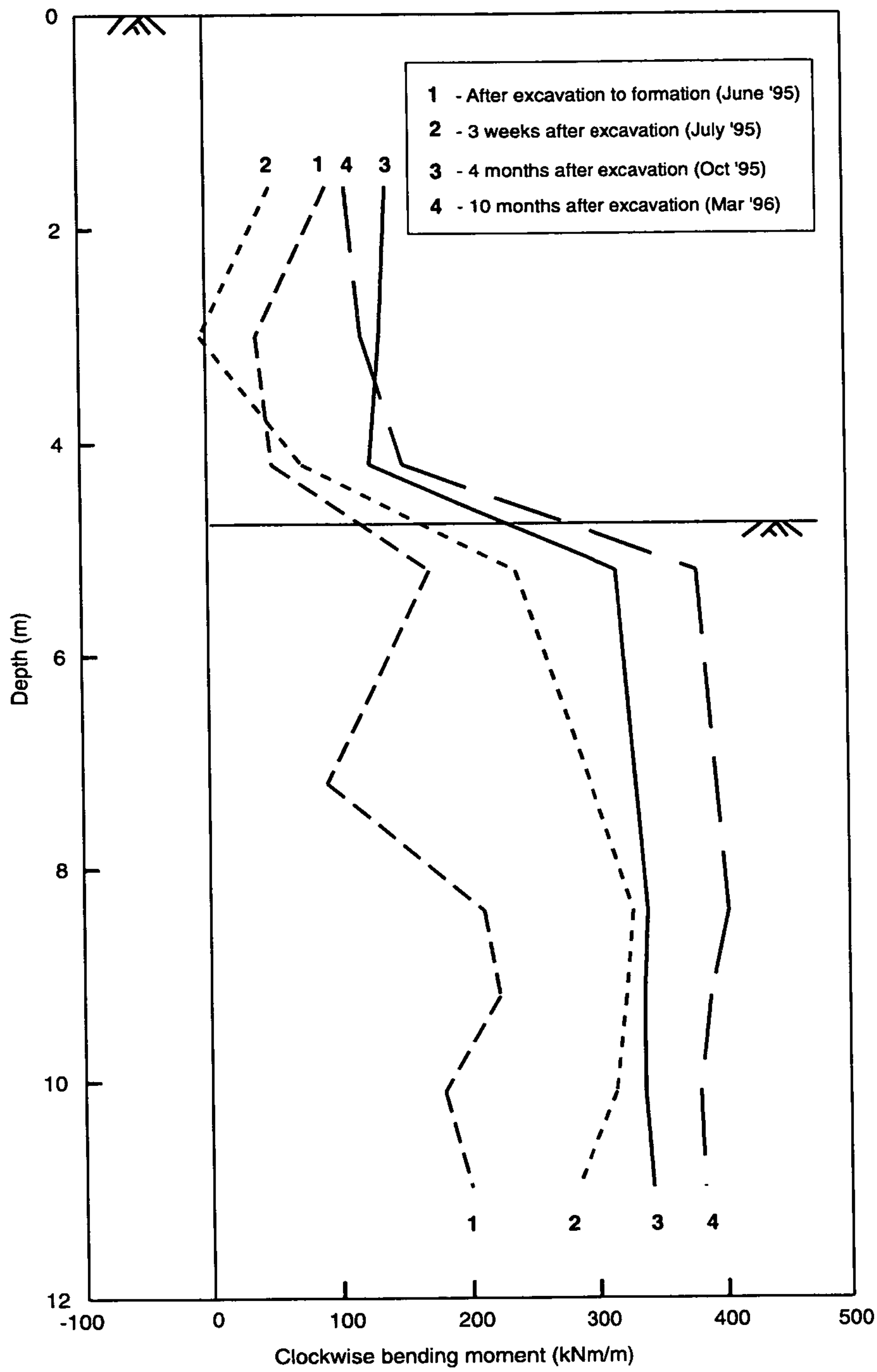

Fig 11. Development of wall bending moments

Also shown in Table 3 are the factors of safety calculated assuming lateral stresses higher than active and equivalent to a K value (ratio of effective horizontal to vertical stress) of 1 in the retained ground. This value was inferred from the insitu K value of 1.5 established during the site investigation (Frank Graham Geotechnical, 1989) with an arbitrary reduction to allow for stress relief during wall installation and tunnel approach excavation. On the basis of these calculations, factors of safety in excess of 1.2 were obtained using moderately conservative soil strength parameters. Factors calculated using worst credible parameters were above 1.1.

Fig 12 compares the measured bending moments (reproduced from Fig 11) with those from the stress distributions employed in the factor of safety calculations given for the final geometry of the structure in Table 3. Immediately after excavation, the soil exhibited undrained behaviour and measured moments were considerably less than those recorded after about 4 months as the soil started to behave

TABLE 3

Factors of safety using current design methods

Design method	Soil parameters	CP2 Method	Burland -Potts Method	Factored Strength Method	Notes
CIRIA Report 104 and BD 42 - Design includes allowance for items in note (iii) - Active stresses on retained side of wall	Moderately conservative	1.88 (1.8)	2.04 (2.0)	1.41 (1.5)	Active: $\delta = {}^{2}/_{3}\phi'$, c_w=0.
	Worst credible	1.48 (1.38)	1.55 (1.5)	1.24 (1.2)	Passive: $\delta = {}^{1}/_{2}\phi'$, c_w=0.
CIRIA REPORT 104 and BD 42 - Final geometry, excludes items in note (iii) - Active stresses on retained side of wall	Moderately conservative	3.56 (1.8)	4.52 (2.0)	2.20 (1.5)	Active: $\delta = {}^{2}/_{3}\phi'$, c_w=0.
	Worst credible	2.78 (1.38)	3.32 (1.5)	1.95 (1.2)	Passive: $\delta = {}^{1}/_{2}\phi'$, c_w=0.
K of 1 on retained side of wall - Final geometry, excludes items in note (iii) - Passive stresses on excavated side of wall	Moderately conservative on excavated side	1.29	1.41	1.22	Passive: $\delta = {}^{1}/_{2}\phi'$, c_w=0.
		1.54	1.76	1.38	Passive: $\delta = \phi'$, c_w=1.
	Worst credible on excavated side	1.16	1.23	1.12	Passive: $\delta = {}^{1}/_{2}\phi'$, c_w=0.

Notes:

(i) Retained height of 4.75m; effective overall wall height of 20.95m (depth of embedment has been reduced by 20% - see text).

(ii) CIRIA Report 104 recommended values are in brackets.

(iii) Allowance for 10kPa surcharge of retained ground and 1.8m deep unplanned excavation in front of the wall used where indicated.

(iv) Moderately conservative parameters of $\phi'=26°$ and c'=6kPa, worst credible parameters of $\phi'=26°$ and c'=0, assumed for boulder clay.

(v) δ is the effective angle of wall friction and c_w is the wall adhesion in kPa.

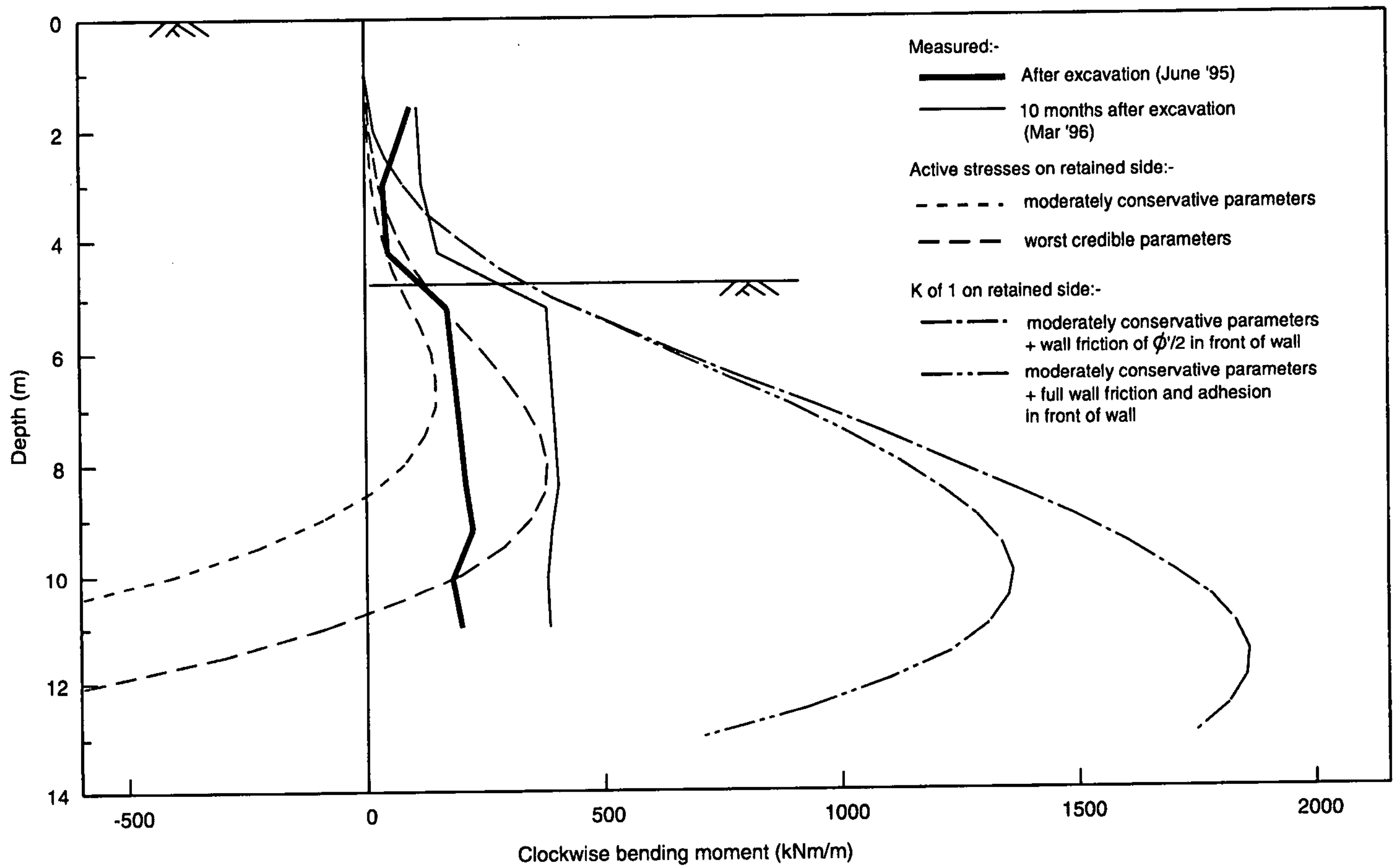

Fig 12. Comparison of measured and design bending moments

in a drained manner. Moments above excavation level measured 10 months after excavation were significantly higher than those determined from active stresses in the retained ground and closer to those calculated using a K value of 1 (Fig 11). Below excavation level, moments were less than those calculated assuming passive pressures on the excavated side and a K of 1 on the retained side, even if full wall friction and adhesion together with moderately conservative soil strengths are used. Pressure cell measurements of lateral stress in front of a cantilever diaphragm wall reported by Carder and Symons (1989) have established that the mobilisation of full passive pressure is likely in stiff clays.

8. CONCLUSIONS

Field instrumentation was installed to study the performance of a cantilever contiguous bored pile wall during and shortly after construction. The wall formed part of the eastern approach to a cut-and-cover tunnel at the junction between the A406 North Circular Road and East End Road, Finchley. The following conclusions were drawn.

(i) The geology of the site was predominantly boulder clay, a typically heterogenous glacial till comprising stiff clays interspersed with sand and gravel lenses.

(ii) Installation of the bored piles was carried out under bentonite and only small movements of the adjoining ground were observed. Lateral movements and settlements measured 1m away at the ground surface were less than 2mm and 1mm respectively. Maximum movements were recorded at the ground surface, subsurface movements at the same distance away were hardly measurable.

(iii) During bulk excavation the wall cantilevered towards the excavation with a lateral movement at the top of the wall of about 5mm. Following excavation, the rate of increase of movement slowed with movements of 9mm and 11mm being measured after 4 months and 10 months respectively. Throughout the monitoring period there was no evidence of any movement of the wall toe. Ground heave monitored at a point 3.2m in front of the wall and 4m below the carriageway reached 12mm after 5 months had elapsed following excavation.

(iv) Immediately after excavation to formation level, the soil responded in an undrained manner with bending moments of up to 225kNm/m being measured over the instrumented upper 11m of the wall. Measured moments at 10 months after excavation were significantly higher and increased with depth to a value of about 400kNm/m just below final carriageway level. Moments above carriageway level were more consistent with lateral stresses in the retained ground corresponding to a K of 1 than with active earth stresses.

(v) Further monitoring is required to establish longer term performance of the cantilever wall.

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- Please debit my Deposit Account
no ....
- Please debit my Credit Card by £...............
- Credit card no ....
Expiry date ....
Signature ....