

The long term performance of embedded retaining walls

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Executive Summary

Much of the urban development in the UK is founded on heavily overconsolidated sedimentary clays, which are particularly susceptible to swelling and softening following the reduction in stress caused by retaining wall construction. Moreover, the low permeability of these clays means the swelling and softening is likely to extend over many years or decades following completion of construction. For these reasons, design of embedded walls for long term stability may be critical. This report describes the results from continued monitoring over many years of various embedded retaining structures instrumented by TRL.

The report covers walls instrumented during construction where monitoring has been continued in the longer term. It also covers walls constructed between 1972 and 1975 and instrumented whilst in service to evaluate their long term behaviour.

For the walls instrumented during construction, comprehensive data on wall movements and bending moments, ground stresses and porewater pressures, together with prop loads, are generally available. Information on the walls in service is more limited and tends to focus on ground stresses and porewater pressures. The report covers cantilever walls, walls propped at carriageway level, anchored walls and cut-and-cover tunnels.

One of the main design uncertainties is the magnitude of heave which will occur below the new carriageway in the longer term due to the unloading caused by excavation. Heave measurements at Bell Common tunnel are reported over an 11 year period of monitoring.

Measurements of anchor head load on under-reamed anchors founded in clay at Neasden are reported over a 24 year period since construction and show the anchors are performing satisfactorily.

1 Introduction

Over the last two decades, a major programme of research has been carried out at TRL for the Highways Agency into the behaviour of earth retaining structures for road schemes, with the aim of improving their design and construction. The current focus of attention on the need for highways within built-up areas, and the road construction below ground level that these usually entail, means that this database of research information is now being used extensively in the design of retained cutting, cut-and-cover tunnels, and bridge abutments in the UK. Guidance on the design of embedded walls based on TRL research is given in BD 42 (DMRB 2.1).

Particular problems in embedded wall design arise when they are founded in heavily overconsolidated clays because of the presence of high *in situ* lateral stresses. These stresses produce high structural loading on the wall and any associated propping system, and also render the clays susceptible to swelling and softening following excavation. Furthermore, as the clays are of low permeability and water percolation is therefore very slow, it may take many decades to reach equilibrium under the new stress regime caused by construction.

This report discusses the results from TRL site monitoring of embedded retaining walls founded in stiff clay carried out to develop a better understanding of long term behaviour. The studies fall into two categories:

- Embedded walls instrumented at the time of construction where measurements have been continued for a number of years. Six walls which are permanently propped at carriageway level, one cantilever wall and two cut-and-cover tunnels are included in this category.
- Walls constructed between 1972 and 1975 and instrumented whilst in service to evaluate long term behaviour. Three different types of wall have been investigated, ie. cantilever, permanently propped at carriageway level, and anchored.

For the walls instrumented during construction, comprehensive data on wall movements and bending moments, ground stresses and porewater pressures, together with prop loads are generally available. Information on the walls in service is more limited and tends to focus on ground stresses and porewater pressures.

2 Embedded walls instrumented during construction

Detailed results on the performance of the instrumented walls during construction have already been reported and recourse should therefore be made to the appropriate source reference. A summary of the measurements at the various sites of wall and ground movements on completion of construction has been produced by Carder (1995). This report now deals with the longer term changes which have occurred since construction ended.

2.1 Cut-and-cover tunnels

2.1.1 Bell Common tunnel

In 1982-83 a 475m long cut-and-cover tunnel was constructed at Bell Common to carry the M25 through the northern edge of Epping Forest. The tunnel was formed from two secant pile walls founded in London Clay and propped apart by a roofing slab which was simply supported on the walls and on a central line of piles. A section of the tunnel was extensively instrumented to measure the stresses on, and movement of, the wall and the adjacent ground movements (Tedd et al, 1984). The field measurements drew attention to the substantial ground movements which can occur during the installation of an embedded wall: these were accompanied by significant reductions in the total lateral stress in the ground close to the wall. The magnitude and extent of these movements and stress changes when installing an embedded wall will be highly dependent on the ground conditions and the method of construction; the process of wall installation is an essential component of the overall construction sequence which generally needs to be simulated carefully in a numerical analysis to obtain a reliable prediction of performance.

At Bell Common, a finite element prediction of behaviour (Potts and Burland, 1983) was carried out prior to construction based on published stiffness properties of the clay derived from previous back-analyses. Discrepancies between the predicted and measured wall bending moments were accounted for by the combined stiffness of the roofing slab and compressible packing being less than assumed, the process of wall installation not being modelled, and the excavation sequence varying from that used in the analysis. Further finite element analyses (Higgins et al, 1989) were therefore carried out which took these factors into account and utilised soil stiffnesses derived from laboratory tests where the stress paths of the soil in the vicinity of the wall were accurately followed. The revised prediction gave better agreement with the measurements when comparing the bending moments in the upper part of the wall, but over-predicted the magnitude of moments at greater depth.

Measurements taken 4 years after the end of construction have been reported by Symons and Tedd (1989) who concluded that there was little indication of any significant change in lateral earth pressure in the London Clay behind the wall while, in front of the wall, earth pressures reduced by only a small amount. The changes in wall deflected shape suggested that there has been a reduction in the propping action of the roof slab possibly due to time dependent stress-strain properties of the compressible packing installed between the roof and the wall. The changes in lateral movement of the wall over a 9 year period following the end of construction are shown in Figure 1. Generally only small changes were measured after the reading taken in May 1988 and, after a further 4 years, movement at the top of the wall record in July 1992 indicated only a 2mm difference.

The wall movements appeared to have occurred primarily by rotation and there was little evidence of any translation of the wall toe in the longer term. This



Figure 1 Long term wall movements at Bell Common tunnel

conclusion was verified by the tape extensometer measurements in Figure 2 of convergence inside the tunnel between the secant pile wall and the central dividing line of piles. Also shown in Figure 2 is the movement at 6.7m depth measured using the inclinometer tube in the secant pile wall. The reasonable correlation between the results would not exist if any significant movement of the toe of the wall were occurring. The results in Figure 2, particularly the earlier results where readings were taken more frequently, also show some small seasonal fluctuations which are probably a result of thermal expansion and contraction of the roof slab.

For walls founded in stiff clay, one of the major design uncertainties is the magnitude of heave which will occur below the new carriageway in the longer term due to the unloading caused by bulk excavation. Measurements at Bell Common tunnel are unique in so far as comprehensive monitoring of clay heave was undertaken. Figure 3 shows the heave measured at 1.5m depth across the carriageway using seven settlement cells. These cells were installed on completion of bulk excavation and, after 11 years in service, a maximum heave of nearly 50mm was measured at 10.5m away from the wall, ie. at approximately midway between the tunnel wall and the central supporting line of piles. Examination of the results in Figure 3 indicates that little or no further heave has occurred in the last few years and this is confirmed by the time plots of subsurface heave given in Figure 4.

The subsurface heave measurements were carried out using a magnetic extensometer system installed in a borehole at 9.3m distance from the wall (Tedd et al, 1984). As with the settlement cells, the instruments were installed shortly after bulk excavation was completed. The measurements in Figure 4 indicate an approximately linear reduction in heave with depth at all stages and also confirmed that the major part of the heave occurred within the first 3 or 4 years after bulk excavation. Since then the rate of increase of heave with time has slowed and an essentially stable situation has been reached after about 10 years.

2.1.2 Finchley tunnel

In 1995 field instrumentation was installed to investigate the performance of the diaphragm walls of a cut-and-cover tunnel during its construction at the junction of the A406 North Circular Road and Old East End Road, Finchley. 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. The tunnel was



Figure 2 Convergence across eastbound carriageway at Bell Common tunnel



Figure 3 Heave at 1.5m below the carriageway at Bell Common tunnel



Figure 4 Subsurface heave below carriageway at 9.3m from wall at Bell Common tunnel

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. In addition to monitoring lateral movements and bending moments developed in the wall during construction, axial loads in the roof and carriageway prop slab were also measured. The field data covering six months after the completion of tunnel construction were reported by Brookes and Carder (1996a).

Figure 5 shows the lateral movements of the wall and retained ground measured using an inclinometer system during construction and the 18 months following bulk excavation. Only a small additional lateral movement of about 1mm was measured close to the top of the wall during this period. No significant changes were measured using the ground inclinometer tube at 1.9m from the wall.

A feature of the construction reported by Brookes and Carder (1996a) was the high initial axial load in the roof of about 1000kN/m, whilst no significant loads were recorded in the permanent structural slab forming the tunnel carriageway. If temporary props had been employed at a lower level during construction, their removal would probably have pre-loaded the carriageway prop to some extent and resulted in lower roof loads. Figure 6 shows the longer term changes in axial loads in the tunnel roof measured by the strain gauges. There is some evidence of the high roof loads reducing with time and this reduction has been accompanied by a small increase in the load in the carriageway prop to a mean value of no more than 160kN/m.

Figure 7 shows that there is little change in the bending moments in the tunnel roof from the values reported by Brookes and Carder (1996a) at the end of construction. The results do however demonstrate a reduction in bending moments when the spoil on top of the roof was removed and a subsequent increase when fill was placed after waterproofing the deck.

2.2 Walls propped at carriageway level

2.2.1 Walthamstow bored pile wall

A contiguous bored pile retaining wall founded in stiff London Clay was constructed in 1991 as part of the A406 North Circular Road improvement scheme between Chingford Road and Hale End Road in Walthamstow. Field monitoring was carried out to measure movements, total lateral stresses and porewater pressures in the ground behind and in front of the wall. The performance of the structural members was also monitored by measuring wall movements and bending moments together with loads in the permanent hinged prop at carriageway level. Carswell et al (1993) presented the results for the wall installation



Figure 5 Wall and ground movements during tunnel construction at Finchley



Figure 6 Development of axial load in the tunnel roof at Finchley



Figure 7 Development of bending moment in the tunnel roof at Finchley

stage, underpass construction and the six months following opening of the road to traffic.

Generally no significant changes have been recorded in the total lateral stresses and porewater pressures close to the wall over the following 5 years and results are therefore similar to those reported by Carswell et al (1993). However some small additional movements of the wall have occurred and these are shown in Figure 8. In this figure, inclinometer readings have been corrected by electronic distance measurements (Geomensor) to the top of the wall. Measurements on both inclinometer tubes I5 and I6 indicate a further outward rotational movement about the permanent prop slab of about 2mm at the top of the wall between August 1992 and September 1994. The results confirm that there has been no translation of the toe of the wall in the 2 year period up to September 1994. Further measurements in the 4 year period up to February 1998 show the same trend continuing for both inclinometer tubes I5 and I6, ie. no translation of the toe and a further small outward movement of the top of the wall of about 1.5mm. Over the 6 years in service, there is no indication of any significant outward translation of the toe which might be expected because of longer term consolidation effects (Watson and Carder, 1994).

Longer term variations in permanent prop load measured using vibrating wire strain gauges in three adjacent hinges are shown in Figure 9. Generally a seasonal variation in prop load consistent with thermal expansion and contraction of the slab was recorded. Particularly noticeable are the peaks in the compressive loads monitored towards the end of the hot summers of 1995 and 1997. The underlying trend of the results suggests that the gradual increase in prop load measured over the first 3 years in service has slowed and that the main changes are now thermal effects.

Watson and Carder (1994) carried out finite element modelling which indicated that measured and predicted bending moments in the wall were broadly similar at the various stages of construction. The results in Figure 10 indicate that wall bending moments have increased during the 6 years since the road was opened. However the maximum moment of 570kNm/m measured at permanent prop level is still well below the 1750kNm/m calculated on the basis of the total lateral stresses measured at 1.1m behind the wall (ie. a K value of about 1). This discrepancy indicates that some further relief of lateral stress in the retained ground has occurred very close to the wall.

2.2.2 Rayleigh Weir bored pile wall

At this site in Essex, the contiguous bored pile wall founded in London Clay was allowed to cantilever outwards during bulk excavation to 3m above final dredge level: temporary props were then installed for the remainder of the excavation and permanent prop construction. This procedure relieved lateral stress on the retained side of the wall and permitted the use of a mass concrete rather than a reinforced concrete prop slab at carriageway level. Monitoring of ground and water pressures, wall movements and bending moments, temporary and permanent prop loads, was carried out during construction and in the following 10 months until July 1992 and was reported by Darley et al (1994).

Continued monitoring of spade pressure cells and



Figure 8 Development of movement of the piled wall at Walthamstow



Figure 9 Variation of permanent prop load at Walthamstow bored pile wall



Figure 10 Development of bending moment in the bored piled wall at Walthamstow

piezometers over a 3 year period from the last reported data indicated that no significant changes had occurred in the total lateral stresses and porewater pressures in the ground close to both sides of the wall.

Lateral movements of the bored pile wall were determined from inclinometer surveys on a tube in the wall and Geomensor electronic distance measurements to the top of the wall: the results are shown in Figure 11. Movements evaluated in April 1995, about 3½ years after construction ended, were within the cluster of results obtained during the initial 10 months in service. It was therefore evident that, within the accuracies of measurement and allowing for some seasonal variations, little or no movement had occurred over the 3½ year period.

Vibrating wire strain gauges were installed both within the thrust blocks for the permanent prop slab and within the bored pile wall. Readings from these instruments indicated that changes had occurred over the monitoring period. Variation of axial loads in the thrust blocks are shown in Figure 12 (page 12). Generally the results indicated a seasonal thermal effect with load increasing in the summer months and decreasing in the winter; this behaviour was consistent with thermal expansion and contraction of the concrete of the prop slab. Peak loads of about 900kN were measured on thrust blocks 1 and 3 at the end of the hot summer of 1995. These loads corresponded to a value of 176kN per metre run of the wall.

The changes in wall bending moment calculated from the strain gauges are shown in Figure 13 (page 13). The last results reported by Darley et al (1994) were for June 1992 and since then an increase in bending moment has occurred in the upper part of the wall. This increase mainly developed over the winter of 1992 and was evident in the readings taken in the following March. Since then values have remained fairly constant with a peak of about 500kNm/m at 5.5m depth. This measured



Figure 11 Development of movement of the piled wall at Rayleigh Weir

bending moment still remains well below that of 830kNm/m calculated for this depth assuming earth pressures corresponding to a K of 1.

2.2.3 A406/A10 Junction diaphragm wall

Field instrumentation was installed to monitor the behaviour of a counterforted (T-shaped) diaphragm retaining wall founded in overconsolidated clay during its construction as part of the A406 North Circular Road, Great Cambridge Road Improvement Scheme. The wall was constructed with a permanent reinforced concrete prop slab at carriageway level which was hinged at the joints between the prop and the wall. Results during construction and the first year of service after the road was opened in February 1990 were reported by Carder et al (1991).

Monitoring carried out from 1990 until 1996 showed no significant changes in the total lateral stresses and the porewater pressures measured close to the wall on both the retained and excavated sides. Distributions of stress and porewater pressure with depth are therefore as reported by Carder et al (1991).

Figure 14 (page 14) shows the lateral movement of the top of the wall measured over the same period. This shows a broad correlation between the results from electronic distance measurements (Geomensor) and those from two separate inclinometer tubes I1 and I2. As the analysis of movements from the inclinometer tubes assumes base fixity, this finding confirms that little or no movement of the toe of the wall has occurred over the 6 year period. Generally the results in Figure 14 also suggest that there is no underlying trend in the magnitude of wall movements with time. However there is a variation of about ± 1.5 mm in the movement of the top of the wall and close examination indicates that a small movement towards the underpass is recorded in winter and the converse in summer. This behaviour is consistent with the thermal contraction and expansion of the permanent prop slab at carriageway level which is likely to occur seasonally.

The lateral wall movement profile with depth measured using an inclinometer system at 3 yearly intervals is shown in Figure 15 (page 15). The range of measurements showed variations of a few millimetres as would be expected from the results given in Figure 14.

2.2.4 Walthamstow diaphragm wall

This counterforted diaphragm wall was constructed for an underpass on the A406 North Circular Road improvement scheme in 1991. The underpass was part of the same contract as the bored pile wall described in Section 2.2.1: however the two locations were about 700m apart. The wall was founded in London Clay which became stiffer with depth. The clay was encountered to the surface although the upper 8.5m was weathered in nature. The structure was propped at carriageway level with precast concrete hinges at the joints between the slab and the wall to accommodate ground heave during the life of the structure. Instrumentation was similar to that at the bored pile wall, although measurements of permanent prop load were not made at this site. The instrumentation and monitoring results during construction and the first 3 months of service are described by Carder et al (1994).



Figure 12 Variation of load in thrust blocks at Rayleigh Weir



Figure 13 Development of bending moment in the piled wall at Rayleigh Weir



Figure 14 Movement of top of wall at A406/A10



Figure 15 Range of movement of the diaphragm wall at A406/A10

Figure 16 shows the movement of the top of the wall determined from electronic distance measurements (Geomensor) and from inclinometer surveys assuming base fixity of the tube in the wall. The difference between the results indicates the magnitude of wall toe movement which has occurred. A toe movement towards the underpass of about 5mm developed primarily during bulk excavation and, over the 5 year period since the underpass was opened, very little further movement has developed. The profiles of wall movement with depth measured between 1992 and 1998 are compared in Figure 17. The results indicate only a small increase in movement which may reflect seasonal changes due to thermal expansion and contraction of the permanent prop slab rather than a trend of increasing lateral movement.

Measurements of total lateral stress acting on both sides of the wall taken 3 months and nearly 5 years after the opening of the underpass are compared in Figure 18. Results are very similar with only minor redistributions of stress having occurred. The four uppermost spade cells located in the retained ground at 1.5m behind the counterfort of the T-panels recorded stresses corresponding to a K value of 1. The deepest cell initially gave higher values in 1992 but has since malfunctioned so that no further readings are available. The spade cell measurements in front of the wall are compared with passive values calculated using the best fit soil parameters from triaxial compression tests of c'=13kPa and ϕ '=23°. The values of passive earth pressure coefficient have been determined assuming a wall friction angle of $\phi'/2$ and zero wall cohesion in accordance with the recommendations of Padfield and Mair (1984). The measured pressures on the upper two cells were slightly above the K_p value calculated in this way, although stresses on the deeper two cells were below this K_n.

2.2.5 Hackney to M11 Link bored pile wall

The secant pile wall being investigated forms part of the south wall on the George Green tunnel approach which is located on the new alignment of the A12 to M11 Link Road. The secant bored piles were of 1.2m diameter and installed at 1m centres; each pile was reinforced by a universal I-beam (914×305mm, 224kg/m). The wall at the instrumented section has a nominal retained height of 7.5m and an overall penetration of 18m. The wall was founded in London Clay although made ground and sandy gravel (Boyn Hill Gravel) existed to a depth of about 6m from the ground surface. Bulk excavation was carried out beneath temporary steel props which were released after construction of a permanent reinforced concrete prop slab at carriageway level. The slab was nominally 1m thick and was constructed using a shear connection between the slab and the wall which allowed some rotation to occur thus accommodating long term heave of the underlying clay. The field instrumentation and monitoring carried out during construction and the following five months have been reported by Bennett et al (1996).

Figure 19 shows the lateral movement of the top of the wall obtained using three techniques. The inclinometer results assume base fixity, which takes no account of any movement of the toe of the wall. This was assessed using the Geomensor electronic distance measuring system in two ways. Firstly by assuming the reference pillar at 14m from the wall remained stationary and secondly by measuring the changes in span between the opposing walls and assuming they moved identically. Measurements in Figure 19 using the latter two techniques were not available throughout the period as the sight lines became obscured after parapet construction. Readings using all three techniques over the 2½ year period since construction ended (around day 600) indicated very little change. The final profile of wall movement with depth is therefore much as reported by Bennett et al (1996) with lateral movements at the top and toe of the wall of no more than 25mm and 18mm respectively.

The changes in wall bending moment over the same period are shown in Figure 20. These bending moments were calculated from pairs of strain gauges at various depths on the I-beam reinforcement of one of the piles. Generally there were only minor changes in the profile with depth over the $2\frac{1}{2}$ year period of monitoring. If the bending moments are calculated on the basis of active pressures on the retained side of the wall assuming zero cohesion and ϕ ' of 36° for the overlying gravel, a peak bending moment of 400kNm/m is determined at carriageway level. This agrees reasonably with the measured values shown in Figure 20.

Measurements of lateral stress using spade cells at 1.25m behind and in front of the wall are given in Figure 21. Very little difference existed between the measurements taken in December 1995 (5 months after construction) and much later in March 1998. Lateral stresses in the retained clay were generally just below a K value of 2. Stresses on the excavated side of the wall were below the line calculated using best fit soil strength parameters (Bennett et al, 1996) assuming full wall friction and in closer agreement with that using wall friction of $\frac{1}{2}\phi'$.

Axial loads in the permanent prop slab were calculated from strain gauge measurements and the variation of mean prop load with time is shown in Figure 22. In common with the results from other sites, prop load was found to vary seasonally with the higher loads being recorded due to prop slab expansion during the summer. Over the period of measurement, the mean prop load varied between 200kN/m and 700kN/m at this site.

The subsurface heave of the clay below the permanent prop slab was monitored using a combination of magnet extensometers and precise levelling. Five months after completion of construction, mean heaves of 19mm, 12mm and 7mm were recorded at 3m, 7m and 10m depths below the prop slab. After 2½ years these heaves had increased to 26mm, 17mm and 10mm respectively.

2.2.6 Aldershot Road underpass diaphragm wall

Field instrumentation was installed to monitor the behaviour of a T-shaped diaphragm retaining wall founded in overconsolidated London Clay during its construction as part of the Aldershot Road underpass in 1995 and 1996. The construction sequence involved installation of the wall under bentonite followed by



Figure 16 Movement of top of diaphragm wall at Walthamstow



Figure 17 Lateral movement of the diaphragm wall at Walthamstow



Figure 18 Lateral stresses on the diaphragm wall at Walthamstow



Figure 19 Lateral movement of the top of the wall at Hackney to M11 Link



Figure 20 Changes in wall bending moment at Hackney to M11 Link



Figure 21 Lateral stress distribution at Hackney to M11 Link



Figure 22 Development of permanent prop load at Hackney to M11 Link

excavation below two levels of temporary steel props and casting of a permanent reinforced concrete prop slab below the final carriageway. The potential for uplift from ground water and long term swelling pressures in the clay was such that the slab was designed as a shallow V-shape with hinges incorporated both at the wall connections and in the centre of the slab. In addition a 200mm thick sand filter layer covered with a geocomposite drainage sheet was laid below the slab. In the instrumented area the diaphragm panels had an overall penetration of 14.6m and were capped with a 4m high reinforced concrete capping beam. The final retained height in this area was about 10m. The field monitoring data during and immediately after construction are reported by Carder et al (1997).

The results in Figure 23 show the lateral movements of the top and toe of the wall measured since the road was opened to traffic in July 1996. Generally no significant movement of the wall toe has occurred from this time up to March 1998, although additional outward movements of a few millimetres have occurred at the top of the wall. In both cases there is a small seasonal fluctuation in movement which is consistent with thermal expansion and contraction of the reinforced concrete prop slab.



Figure 23 Development of lateral movement of the wall at Aldershot

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 plotted against time in Figure 24. Generally the measured stresses beneath the western and eastern slabs were lower closer to the wall than they were further away because more fill was placed towards the centre of the underpass. A sudden reduction in vertical stress was measured on all cells, especially those nearer the centre of the underpass, when the road was opened to traffic. It is not clear whether this redistribution of stress due to trafficking was related to the presence of the sand and geocomposite layer on which the slab was seated, or the hinged design of the slab.

Figure 25 shows the subsurface heaves below formation level measured using the magnetic ring extensometer systems in conjunction with precise levelling. The detailed locations of the magnetic rings are described by Carder et al (1997) with the uppermost ring at each location being between 1m and 3m below formation. 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. A smaller heave of about 20mm was recorded on the shallow magnetic ring in the centre of the underpass (Figure 25c) and this was because the initial datum reading was taken after bulk excavation was completed. Whereas there was little change over the first year in service in heave values recorded at 6m from the wall edge beams (Figure 25a and b), measurements on the central magnetic ring system (Figure 25c) suggested that a settlement of about 10mm had occurred below the central hinge. This may be related to the fall in vertical stress due to trafficking discussed earlier.

The variations in permanent prop load and temperature with time measured using vibrating wire strain gauges and thermistors respectively are shown in Figure 26. Measurements of prop load at this site were unusual in so far as the compressive loads of between 157kN/m and 471kN/m recorded before opening of the road to traffic have reduced over the first year in service, possibly because of the 'seating' effects just described. There are now some early signs that loads are beginning to stabilise and in the longer term any further heave of the clay may result in increases of prop load.

2.3 Cantilever wall at Finchley

This 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 been discussed in Section 2.1.2. The cantilever walls were used to flank the approaches to the tunnel, and comprised 24m deep contiguous bored piles of 1.5m diameter installed at 1.98m centres. The retained height of the wall in the instrumented area was 4.75m. 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 were monitored during construction and for a period of about 10 months after completion of construction (Brookes and Carder, 1996b).

Continued monitoring took place over a further 21 month period and indicated very little change. Figure 27 shows the profiles of lateral movement against depth.

Following bulk excavation in June 1995 the wall cantilevered towards the excavation with a lateral movement at the top of the wall of about 5mm. Respective movements at the top of the wall of 9mm and 11mm were observed at 4 and 10 months after excavation. Readings taken at 31 months after excavation showed no measurable change from those recorded at 10 months. Movements at ground surface using a tape extensometer system showed close agreement throughout with the inclinometer surveys, confirming that no movement occurred at the base of the pile inclinometer tubes.

Figure 28 shows the development of wall bending moment with time. As would be anticipated from the movement results, no significant change in bending moment was measured between 10 and 31 months after bulk excavation. In Figure 28 the bending moments at 31 months after excavation increased with depth to a value of about 400kNm/m just below carriageway level. This value was more consistent with lateral stresses in the retained ground corresponding to a K value of 1 than with active earth stresses. This point is discussed in more detail by Brookes and Carder (1996b).

3 Embedded walls instrumented whilst in service

For embedded retaining walls in stiff clay the critical design condition often occurs in the long term when full porewater pressure equilibrium has been reached under the new stress regime. Field studies have been undertaken to measure earth and porewater pressures acting on various structural types of embedded wall which were constructed between 1972 and 1975 and therefore have been in service for about two decades. Data on the performance of a cantilever diaphragm wall, a contiguous bored pile wall propped at carriageway level and an anchored diaphragm wall are now considered.

3.1 Cantilever diaphragm wall at Reading

The diaphragm wall was constructed in 1972 and instrumentation was installed to establish its long term performance in 1984. At this site, a layer of terrace gravel overlies the London Clay which extends from a depth of 3m to 20m. Below about 14.5m the clay is very stiff, with thin sand bands occurring in places. Instrumentation was installed to monitor the total lateral stress and porewater pressures both behind and in front of the wall. Results up to 1987 were reported by Carder and Symons (1989a), although monitoring has continued in the longer term.

Figure 29 shows the variation of total lateral stress with time as recorded by push-in spade cells at different depths on the retained side of the wall. Some small readjustments in stress were recorded on shallow cells SC1 and SC2: initially SC1 gave the lower values, but by 1995 the converse was true. Generally however, the only significant change was the gradual reduction in measured stress with time on the deepest cell SC4. Carder and Symons (1989a) in interpreting the results recognised that cell SC4 was giving a higher value than anticipated possibly because the cell was installed in lower plasticity clay.



(a) Below western side of area 6 prop slab



(b) Below eastern side of area 6 prop slab

Figure 24 Vertical stress beneath the prop slab at Aldershot



Figure 25 Subsurface heave beneath the prop slab at Aldershot



Figure 26 Loads in the permanent prop slab at Aldershot



Figure 27 Wall movements following bulk excavation at Finchley



Figure 28 Development of wall bending moment at Finchley



Figure 29 Variation of lateral stress on retained side of wall at Reading

The profiles of total lateral stress measured in 1987 are compared with those determined in 1995 in Figure 30. Generally stress levels have not changed significantly over the period apart from the small reduction in stress on the deepest cell in the retained ground which has also occurred on the equivalent cell in front of the wall. The results confirm that for this cantilever wall founded in London Clay with an *in situ* K value (ratio of horizontal to effective stress) of between 1.5 and 2, lateral stresses on the retained side of the wall were stable at a K of 1. It must be noted however that final K values near walls in stiff clay are likely to be higher when they are supported during excavation as less stress relief will occur when movement is restrained.

No significant changes in ground water level on either side of the wall were recorded since the data were first reported by Carder and Symons (1989a).

3.2 Propped bored pile wall at New Malden

A field study was carried out to investigate the stresses acting on a contiguous bored pile retaining wall propped beneath the carriageway which was constructed in 1975. The ground conditions at the site comprised up to 2m of made ground overlying London Clay. The weathered brown clay extended to a depth of about 6.6m: below this the clay was grey and became very stiff with depth. The wall was instrumented in 1984 to study its long term behaviour and its in service performance was reported after 5 years of monitoring by Carder and Symons (1989b).

A feature at this site was the large seasonal fluctuations in ground stresses and porewater pressures near the retaining wall. These changes were broadly compatible with the thermal expansion and contraction of the prop slab located immediately beneath the carriageway. However it must be noted that the magnitude of these changes was larger than measured at similar sites probably because no hinges were incorporated in the prop slab. Further monitoring since 1989 has shown that the same behaviour has continued in the longer term.

Figure 31 shows the distributions of total lateral stress acting on both sides of the wall in September 1995. Generally conditions were unchanged from those reported by Carder and Symons (1989b) although some minor redistributions of stress had taken place. On the retained side of the wall, lateral stresses remained between a K value of 1 and 2. In front of the wall, the pressures measured on the upper two spade cells were close to those predicted from full passive pressures calculated from average measured strength parameters of c'=18kPa and $\phi'=24^{\circ}$ assuming full wall friction and adhesion.

3.3 Anchored diaphragm wall at Neasden

The retaining wall, which was founded in stiff London Clay, was installed in early 1972 as a diaphragm wall constructed under bentonite. Each reinforced concrete wall panel (4.57m long \times 0.6m thick) was supported by four levels of under-reamed inclined anchors founded in the clay. The ground behind and in front of the wall was instrumented with spade pressure cells and piezometers by TRL in 1988 to establish long term behaviour. Performance of the ground anchors was assessed from anchor head load cells which were installed at the time of construction (Sills et al, 1977) and found to be still functional. The results of the study up to 1990 were published by Carswell et al (1991).



Figure 30 Lateral stress distribution acting on the Reading wall



Figure 31 Lateral stress distribution acting on the Malden wall

Continued monitoring of lateral stresses and porewater pressures up to September 1995 has shown little change in values with the possible exception of the stresses below the carriageway in front of the wall. A summary of these stress changes is given in Table 1. Although the results indicate some stress reduction on the three cells over the 5 year period, latest values still correspond to a K value in excess of 4. If the passive value is calculated from the soil strength parameter of $\phi'=24^{\circ}$ assuming full wall friction, a value of 3.64 is obtained.

 Table 1 Variation in lateral stress in front of the wall at

 Neasden

Depth (m)	Cell no.	Stresses (kPa) - Aug '90	Stresses (kPa) - Sept '95	Equivalent K - Sept '95
2	SC6	189	166	4.6
3.5	SC7	289	262	4.8
5.1	SC8	339	314	4.2

Long term measurements of anchor head loads taken between 1988 and 1996 are shown in Figure 32. During construction the anchors were installed with an initial prestress of 460kN. The average measured loads in the anchors on completion of construction in 1972 was close to the prestress value (Sills et al, 1977). Between then and 1990 there were only small increases in load of up to 55kN on the upper three levels of anchors. Continued monitoring over a further 8 years has shown that only minor redistributions of load have occurred and the anchors, which are founded in the clay, appear to be performing satisfactorily. In general, the maximum load on any anchor is currently no more than 75kN above the initial prestress value of 460kN.

4 Conclusions

Long term monitoring has been carried out to ascertain the in service behaviour of a number of embedded retaining walls founded in overconsolidated clay. These walls were constructed using bored piling and diaphragm wall techniques for either underpass or cut-and-cover tunnel schemes. The following main conclusions have been reached.

i One of the main design uncertainties is the magnitude of heave which will occur below the new carriageway in the longer term due to the unloading caused by excavation. Measurements at Bell Common tunnel have demonstrated that the major part of the long term heave occurred within the first 4 years after bulk excavation. Heave then continued at a more gradual rate until a near stable situation was reached after about 10 years had elapsed from the time of construction. At this time, measurements at 1.5m depth below the carriageway indicated a maximum heave of 50mm at a location approximately midway between the tunnel wall and the central supporting line of piles. It must be noted that heave measurements at Bell Common were commenced after bulk excavation was complete. Measurements at other schemes, where the wall was founded in London

Clay, indicated that additional heaves of between 10 and 50mm occurred immediately on excavation.

- ii Measurements of total lateral stress were taken in the retained clay at between 1.1m and 1.5m from the wall during the first year in service at various construction schemes. The stress results indicated an equivalent K value of between 1 and 1.5 which can be compared with the *in situ* values of 2 to 2.5 which existed prior to any construction. Generally longer term measurements over periods ranging from 5 to 24 years after construction indicated only minor readjustments in the stresses acting over the depth of the wall.
- iii Total lateral stresses measured soon after construction between 1.5m and 2m in front of the walls were typically of a similar magnitude or higher than those predicted from passive design values calculated using best fit soil strength parameters determined from triaxial compression tests. Some small decreases were observed with time but stresses remained similar in magnitude to passive design values.
- iv Where a permanent prop slab was used to support the wall at carriageway level, seasonal thermal expansion and contraction of the slab gave rise to cyclic changes in load in the permanent prop slab. In general, monitoring of the permanent props at a number of sites suggested some increase in load during the first few years in service although loads then appeared to stabilise. The seasonal changes in prop load produced only small changes in the measured soil stresses acting on the wall. The exception to this was the bored pile wall at New Malden (A3) where seasonal ground stress and porewater pressure changes were more apparent possibly because the permanent prop slab was cast directly against the wall without any hinges.
- v A feature of the construction of the Finchley tunnel was the high initial roof load of about 1000kN/m, whilst no significant loads were recorded in the permanent structural slab forming the tunnel carriageway. If temporary props had been employed at a lower level during construction, their removal would probably have pre-loaded the carriageway prop to some extent and resulted in lower roof loads. Close attention needs to be given to the construction sequence when designing multi-propped structures.
- vi Monitoring over the first 5 years in service of walls propped at carriageway level showed additional lateral movements of a few millimetres at the top of the walls, although generally there was little or no toe movement.
- vii In most cases, wall bending moments immediately after construction were lower than those calculated from the soil stresses measured between 1m and 1.5m from the wall. A small increase in moments was noticeable over the first 5 years in service, although values were generally still less than those determined from stress measurements. This discrepancy indicates that some further relief of stress in the retained ground has occurred very close to the wall. Generally agreement between measured bending moments and predictions from finite element analyses (which were able to model the construction sequence) was better than with predictions based on limit equilibrium methods.



Figure 32 Variation of ground anchor loads at Neasden

viii The variation of anchor head loads on four levels of under-reamed anchors founded in stiff clay and supporting a diaphragm wall at Neasden was measured over a 24 year period since construction. Only minor redistributions of loads have occurred over the last 8 years. The maximum load on any anchor is currently no more than 75kN above the initial prestress value of 460kN: the major part of this increase occurred in the first decade after construction.

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Abstract

Much of the urban development in the UK is founded on heavily overconsolidated sedimentary clays, which are particularly susceptible to swelling and softening following the reduction in stress caused by retaining wall construction. Moreover, the low permeability of these clays means the swelling and softening is likely to extend over many years or decades following completion of construction. For these reasons, design of embedded walls for long term stability may be critical.

This report describes the results from continued monitoring over many years of various embedded retaining structures instrumented by TRL. The types of structure fall into two categories. Firstly, the report covers walls instrumented during construction where monitoring has been continued in the longer term. Secondly, it covers walls constructed between 1972 and 1975 and instrumented whilst in service to evaluate their long term behaviour.

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TRL244	Behaviour of a cantilever contiguous bored pile wall in boulder clay at Finchley by A H Brookes and D R Carder. 1997 (price £25, code E)		
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