AN ANCHORED EARTH RETAINING WALL ON THE OTLEY BYPASS: CONSTRUCTION AND EARLY PERFORMANCE

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AN ANCHORED EARTH RETAINING WALL ON THE OTLEY BYPASS: CONSTRUCTION AND EARLY PERFORMANCE

ABSTRACT

The first retaining wall using the principle of anchored earth was constructed in 1984 on the A660 Otley bypass in West Yorkshire. The retaining wall is 86 m long with a maximum height of 6 m: in addition to supporting a small road embankment, the wall also acts as an abutment for a pedestrian footbridge.

This Report describes the general principles of anchored earth together with the design method and construction procedures used at Otley. The instrumentation installed to monitor the behaviour of the wall is also described and the results obtained are analysed and discussed in the context of the design method.

1 INTRODUCTION

The concept of anchored earth originated at the Transport and Road Research Laboratory (TRRL) as a logical progression from reinforced earth in the development of tied-back earth retaining structures (Murray and Irwin 1981). In 1982, following an approach from the West Yorkshire Metropolitan County Council (WYMCC) it was decided to construct the Silver Mill Hill retaining wall using anchored earth (Jones et al 1985). This wall, the first of its kind, forms part of the A660 Otley bypass and the scheme was designed by WYMCC acting on behalf of the Department of Transport (DTp): the main contractor was A. F. Budge and Co. The anchored earth design was adopted instead of the more conventional reinforced earth method, with a view to producing a structure with both technical and economic advantages.

In the area near the retaining wall, the bypass follows the course of an abandoned railway line. The wall is located on the south side of the new carriageway on a hillside of weathered, friable mudstone. The wall is 86 m long and up to 6 m high; it was designed to support an access road and footbridge bankseat shown in Plate 1 using anchor elements and a granular frictional fill. Technical approval for the wall, which was designated a Category III structure (DTp Departmental Standard BD2/79), was obtained after an independent design check had been conducted and pull-out tests on anchor elements had been undertaken at the TRRL.

2 SOIL REINFORCEMENT

The conventional method of soil reinforcement for structures, known as reinforced earth, is described in the Department of Transport Technical Memorandum BE 3/78. This method utilises the frictional forces generated at the interface between the reinforcing element and the fill material. To ensure overall stability in this system, a significant portion of the overall length of the reinforcing element needs to extend beyond the potential failure zone so that adequate tension can be developed in the elements.

In anchored earth, the majority of the pull-out resistance is developed by the anchor head, with only a relatively small contribution being made by friction along the anchor shaft. Thus the anchor needs to be just long enough to ensure that the head is outside the potential failure zone.

A review of both anchored earth and reinforced earth is given by Jones (1984).

3 DESIGN AND CONSTRUCTION

3.1 THE SITE

Silver Mill Hill retaining wall is situated in an existing railway cutting constructed in 1860:
the cutting was excavated in ground which
descended relatively steeply from the Chevin to
the River Wharfe. There is a history of
landsiding in the area which takes the form of
rotational slips in the upper part of the hillside
accompanied by shallower solifluction
movements in the lower slopes and extending
into the railway cutting. During the design of
the bypass particular consideration was given to
a number of houses located about 20 m behind
the wall.

A typical cross section through the construction
is shown in Figure 1: an outline of the ground
conditions, based on borehole logs and trial pit
data, is also given. It can be seen that the
depth of solifluction material was about 3.5 m
with the underlying mudstone varying from a
completely weathered to a weak condition.

Undrained and consolidated undrained triaxial
tests with pore pressure measurements were
carried out on the weathered mudstone.
Undrained shear strengths were extremely
variable, ranging between 21 and 210 kPa, with
an average value of 70 kPa.

Effective stress parameters were found to lie
within the following ranges:

- Effective angle of shearing resistance
  \( (\phi') = 22^\circ \) to \( 34^\circ \)
- Effective cohesion \( (c') = 10 \) kPa to \( 30 \) kPa.

### 3.2 DESIGN THEORY

Jones *et al* (1985) have suggested that there
are two possible mechanisms of pull-out failure
for a triangular anchor element. In the first
(Figure 2a), soil flows around the two leading
members of the anchor and around the back bar
as the anchor is pulled through the soil. In this
case, the pull-out resistance of the anchor head
\( (P_A) \) is given by

\[
P_A = 2P_B \quad \ldots \quad (1)
\]

where \( P_B \) is the pull-out resistance of either
the inclined leading members or the back bar of the anchor.

In the second mechanism, a triangular wedge of
soil is retained within the anchor and shear
occurs on the upper and lower faces of the
retained wedge together with a flow around
mechanism at the leading members. In this
case, the pull-out resistance of the anchor head
\( (P_A) \) is given by

\[
P_A = P_B + 2P'_F \quad \ldots \quad (2)
\]

where \( P'_F \) is the shear resistance on either
the upper or lower face of the retained
wedge of soil.

The value of the pull-out resistance \( P_B \) may be
obtained by analogy with the bearing capacity
of a deep strip footing. A number of approaches
to this problem were considered and the
following expression was chosen on the basis
of simplicity:

\[
P_B = 4 k_p \sigma_v BD \quad \ldots \quad (3)
\]

where \( k_p \) is the coefficient of passive earth
pressure

\( \sigma_v \) is the vertical effective stress

\( B \) is the length of the back bar of the
anchor which is used to define
anchor size

\( D \) is the diameter of the anchor bar.
For calculating $P'_F$, a lower bound case was adopted such that

$$P'_F = A \alpha' \tan \phi'$$

(4)

where $A$ is the plan area of the triangular anchor

and $\phi'$ is the peak angle of shearing resistance measured in a shear box.

For design, the lower value of $P_A$ was adopted by using the smaller of expressions (1) and (2). In addition to the anchor head resistance, an allowance was also made for shaft friction by using

$$P_{ult} = P_A + P_S$$

(5)

where $P_{ult}$ is the ultimate pull-out capacity of the anchor

and $P_S$ is the frictional resistance developed along the shaft.

For the Silver Mill Hill retaining wall, the capacity of the anchors used was confirmed by a series of pull-out tests using the proposed backfill material. Results obtained from these tests generally confirmed the use of (1) to determine the anchor head pull-out resistance for the majority of anchor sizes used at Otley. For the larger and more flexible sizes, the tests indicated that the higher values of $P_A$ given by (2) were more applicable: this was thought to be due to the fact that at larger sizes, the anchor could deform sufficiently to induce friction along the boundary of the soil wedge inside the triangle. The results also indicated that the displacements required to develop full
working load in the anchors were less than 10mm for all the anchors tested. In addition, strain gauges fixed to the heads of a number of the anchors tested indicated that the strains in the anchors did not exceed half the yield strain at working load†.

The results of the pull-out tests in the backfill for this structure, together with results from other backfill materials will be the subject of a subsequent Report. However, a summary of the design values adopted and the actual values obtained from pull-out tests performed on the wall at Otley is given in Table 1.†.

The design of the Silver Mill Hill retaining wall was then conducted in accordance with DTp Technical Memorandum BE 3/78, with minor modifications to cover the use of anchor elements rather than reinforcing strips; details of these modifications are given by Jones (1984). The design was then checked by Geotechnical Consulting Group, who acted as Category III checkers for this structure. The checks performed included the use of finite element analyses: details of these analyses are given in Jones et al (1985).

An amendment to DTp Technical Memorandum BE 3/78 is currently being developed that will include a design method for anchored earth. The method adopted will be similar to that described above but will include additional information and advice on various aspects of the detailed design of anchor elements.

3.3 MATERIALS AND CONSTRUCTION

The toe of the existing hillside was cut back about 5.5m behind the line of the wall, to a previously installed 2.5m deep porous drain (as shown in Figure 1). A concrete footing was then cast in situ to form a foundation for the wall panels. Temporary timber props and walings were placed to support the exposed porous drain.

Seventy-two prestressed concrete panels were used to form the wall. These panels were 1194mm wide by 150mm thick and varied in height between 2.3 and 6.7m. Prestress forces, applied during casting, were 650kN for the panels under 5m long and 1240kN for

† Note: design pull-out force = 2 x working load

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Fig. 3 Anchor specification

Material:
20mm dia. cold worked steel reinforcing bar. Galvanised coating weight not less than 1000g/m²
those over 5 m. Clearance holes for the anchors and Abbey slots, to permit the tying in of the masonry facing, were also cast into the panels: typical facing panels are shown in Plate 2. A masonry facing was required to harmonise with the local landscape.

A total of 588 triangular anchors were used and comprised three basic sizes: 495 had a nominal back bar length of 650 mm, 81 were 900 mm wide and 12 were 1200 mm. Overall lengths varied between 5.2 and 6.7 m depending upon their location in the structure. The anchors were spaced at intervals of 1200 mm horizontally and 500 mm vertically: two additional layers were placed under the footbridge bankseat to provide extra reinforcement. Cold worked steel reinforcing bar, 20 mm in diameter, was used with an M18 thread cut for a nut and washer plate at the panel connection. A galvanised coating weight of not less than 1000 g/m² was specified as a protection against long-term corrosion. Full details of the anchor specification are given in Figure 3.

In February 1984, construction of the wall started with the erection of facing panels 72 to 47 inclusive. These were individually propped from the base course of the carriageway which had already been placed: the panels were bedded on a 15 mm thick layer of dry sand and cement mortar mix on a 150 mm wide strip of roofing felt. The props were adjusted to give the panels a 1 in 40 slope towards the hillside: two coats of pitch epoxy waterproofing were then applied to the backs of the panels. A 150 mm diameter porous pipe drain was laid at the base of the panels and 225 by 450 mm porous block drainage pillars were installed at the vertical panel joints.

A well graded crushed limestone frictional fill with a maximum particle size of 40 mm was obtained from Kilnsey Quarry, Grassington. This was placed in 150 mm thick layers and compacted with six passes of a Bomag BW 605 twin vibrating roller: a Robin EY20D vibrating plate was used close to the wall and around the drainage blocks.

Sand replacement densities to BS 1377: 1975 were performed on the compacted backfill at regular intervals throughout construction. These gave an average dry density of 2195 kg/m³.

Length of coupons = 1 m
Coupons buried at same level as top row of anchors (0.5 m below surface)

| G | Galvanised re-bar | 18 off |
| P | Ungalvanised re-bar | 18 off |
| W | Welded re-bar | 9 off |

Fig. 4 Layout of corrosion coupons
with a coefficient of variation of 5.9 per cent at an average moisture content of 3.7 per cent. Laboratory recompaction tests performed to BS 1377: 1975. Test 14 (vibrating hammer method) showed that the maximum dry density was 2215 kg/m³ at an optimum moisture content of 3.5 per cent. Therefore, the average dry density obtained on site was 99 per cent of the maximum dry density obtained from laboratory recompaction. Shear box tests performed in accordance with BE 3/78 gave a peak angle of shearing resistance of 57° with a
residual value of 40° at a dry density of 1960 kg/m³. This is considerably higher than the maximum angle of 35° used in the design.

The welded joints at the anchor heads were wrapped with a layer of Densopol 60 prior to installation to provide additional protection against crevice corrosion.

The anchors were then placed on top of the compacted fill at 500mm vertical intervals and the ends passed through the facing panels as shown in Plate 3. Pieces of compressible foam rubber were placed underneath the anchor shafts where they passed through the slots in the facing panel, to allow for any differential movements when subsequent layers of fill were placed and compacted. Galvanised steel washer plates and nuts were then attached to the anchors and pulled tight to remove any free play between the washer plates and the facing panel. The timber props and walings, used to support the porous drain at the back of the excavated area, proved to be an obstacle to placing and compacting the fill for the first 2 m or so. They were withdrawn when the level of fill had reached to within about 0.5 m of the top of the drain.

The same method of construction was repeated for panels 46 to 1 inclusive, with all the panels being positioned and propped before any of the fill was placed. A general view of the site during construction is shown in Plate 4.

At the top level of anchors, a selection of 45 ungalvanised, galvanised and welded sections of anchor reinforcing bar, each 1 m long, was buried between the anchors in panels 32 to 49. These samples, shown in Figure 4, will provide corrosion coupons to be recovered for analysis over a period of several years.

When all the fill had been placed, the props positioning the facing panels were removed. The bankseat for the footbridge was then cast in situ above panels 60 to 62. A rubber bearing pad was then glued to the bankseat with epoxy adhesive and the 29 tonne pre-cast concrete footbridge deck was subsequently installed.

Compressible caps were placed over the anchor nuts and washer plates prior to building the masonry block face on the wall. The masonry was tied to the facing panels with dovetails in the Abbey slots and the gap between the masonry and the concrete facing was filled with well-rammed cement mortar. The masonry blockwork was continued above the tops of the panels to provide a 300mm thick, 1 m high parapet wall.

4 INSTRUMENTATION

A total of 49, 150mm diameter, vibrating wire soil pressure cells manufactured by Gage Technique and 38 anchor load cells, from Industrial Transducers, of 30kN capacity were used to monitor the performance of the wall. Panels 50, 60 and 61 each had 10 pressure cells epoxy mortared into pre-cast recesses to monitor the horizontal earth pressures on the wall. At panel 50, vertical ground pressures were monitored by seven pressure cells placed under the footing and in the backfill at the footing level; the layout of these cells is shown in Figure 5. A further 8 cells were placed behind panel 61 in the fill under the bankseat, as shown in Figure 6, with the remaining four cells being cast into the back face of the bankseat to monitor the horizontal pressure.

Load cells were installed between the anchor nut and washer plate to measure the tensile loads in the anchors. Spherical washers were used to ensure that the load cells were axially aligned on the anchors. Each of the 10 anchors on panel 50 were fitted with load cells as were each of the 14 anchors on panels 60 and 61; see Plate 5.

The cables from the load and pressure cells were run to a small manhole adjacent to the footbridge. This contained switch boxes into which portable instrumentation could be connected to energise and read the cells.

A contract was placed with City University to monitor movements of the facing panels during
and after construction. A Zeiss Elta 2 electronic tacheometer was used to observe targets screwed into sockets cast into panels 50, 51, 60, 61 and 62, from four stable survey stations located along the verge on the opposite side of the carriageway. The layout of the survey stations is shown in Figure 7. Extension pieces were installed into the target sockets as the masonry facing was built in order to allow continued monitoring of wall movements. Displacements were measured in three orthogonal directions (vertical, horizontal and lateral) to an accuracy of approximately ± 1 mm.

The instrumentation installed to monitor the performance of the wall has performed well, with only three vibrating wire earth pressure cells failing during the first year. The reason for these failures is thought to be water infiltration.

5 DISCUSSION OF RESULTS

Preliminary results have been obtained from the measurements of earth pressures, anchor loads and panel movements for a period of 263 days from the start of construction. Typical data covering the period of construction of 75 days and the first six months after completion of the structure are presented in the following sections.

5.1 VERTICAL EARTH PRESSURES

Seven cells were installed at the footing level behind panel 50 to measure vertical earth pressures: the layout of these cells is shown in Figure 5. In addition, eight cells were installed behind panel 61 underneath the footbridge bankseat to measure vertical pressures: the layout of these cells is shown in Figure 6.

Although all the cells used were calibrated prior to installation using hydrostatic pressure, previous studies have shown that the *in situ* calibration is affected both by the physical properties of the soil and cell and also by the nature of the compaction (Carder and Krawczyk 1975). An *in situ* calibration was therefore obtained by comparing the indicated vertical pressure with the overburden pressure for some of the cells behind panels 50 and 61. The data from cells 1, 3 and 5 behind panel 50 were not used; cell 1 was below the footing of the
Figure 6 Location of cells to measure vertical pressure at panel 61

Figure 9 shows the changes in vertical pressure, corrected by the calibration given in Figure 8, for the cells located at the footing level behind panel 50 as the fill was placed during the construction period. Cell 1, located directly beneath the wall, shows only a gradual increase in vertical pressure: this is possibly due to a bridging effect of the concrete footing beam. Unfortunately cell 2, which was located adjacent to cell 1, failed shortly after installation. Of the remaining cells, 3 and 5 register low vertical pressures, whilst cells 4, 6 and 7 all recorded pressures close to the overburden pressure. To some extent this is due to the method of calibration.

Figure 10 shows the distribution of vertical earth pressure behind panel 50 on day 263.
The trapezoidal pressure distribution derived from BE 3/78 is also shown, as is the profile of full overburden pressure. Figure 10 shows that cells 1, 4, 6 and 7 all indicate pressures of similar magnitude to the trapezoidal and overburden profiles, whilst cells 3 and 5 show pressures considerably lower than anticipated. The low values obtained from cell 3 may be due to wall friction effects on the back of the panel, but the behaviour of cell 5 would appear to be anomalous.

The pressures indicated by the eight cells installed behind panel 61 to measure vertical earth pressures underneath the footbridge bankseat are shown in Figure 11. Cells 40, 41 and 42, located immediately below the bankseat, indicate pressures of the order of 100–130 kPa. The remaining cells, located 0.5 m below the bottom of the bankseat indicate pressures ranging from 15 kPa just behind the wall up to 60 kPa at a distance of 5 m from the wall.
The estimated weight of the footbridge bankseat is 150 kN, the base area being 3.6 m long by 1.75 m deep. The footbridge beam weighs 284 kN and, as it is simply supported at both ends, it has been assumed that 142 kN acts on the bankseat. This load is carried by a 900 mm long bearing, the centre of which is located 615 mm behind the back of the wall and inclined at 11.5° to the direction of the wall.

The stress distribution caused by the bankseat and footbridge is rather complex. Several attempts have been made to calculate the stresses by making various simplifying assumptions. These calculations suggest that the stresses will be greatest near the back of the wall and reduce considerably towards the rear of the bankseat. However, no account has been taken of wall friction and it may be that the low stress recorded by cell 40 is due to wall friction. For cells 43 to 47, located 0.5 m below the bottom of the bankseat, the picture is even more complicated. Cells 46 and 47, which should be little affected by the bankseat, are both recording pressures considerably lower than predicted, with the cell closest to the wall showing the lowest pressure. One possible explanation for this effect might be that the stresses generated by the bankseat loading are being transferred more rapidly than anticipated from the backfill into the facing panels by wall friction.

5.2 HORIZONTAL EARTH PRESSURES

Horizontal earth pressures were measured behind panels 50, 60 and 61 by 10 cells cast into each of the facing units. Although the cells had been calibrated prior to installation, as described in Section 5.1, an in situ calibration was performed by comparing the pressures indicated with the anchor loads recorded using the load cells. This was relatively simple because each pressure cell was located at the same level as an anchor load cell. The anchor load was therefore divided by the effective area per anchor (the product of the vertical and horizontal anchor spacing) to give a calculated horizontal pressure. This calibration was performed only for the pressure cells in panel 50, because it was considered that the proximity of the bankseat to panels 60 and 61 would make it more difficult to use these as a calibration. The calibration obtained is shown in Figure 12: this shows that the data are rather scattered, probably because the facing panels are sufficiently stiff to allow some transfer of load between adjacent anchors. However, the best line through the data indicates that at low pressure the cells overread by about 25 per cent. This is reasonably consistent with the behaviour of the cells recording vertical pressures and with the observations of Carder and Krawczyk (1975).

The corrected horizontal earth pressures recorded behind panels 50, 60 and 61 are shown in Figure 13. For small depths of fill the pressures are higher than the calculated values of the horizontal earth pressure at rest, k_o. In the case of panel 50, these high pressures may be the result of locked-in compaction stresses. For panels 60 and 61, the effects of the bankseat loading will also tend to increase the recorded horizontal stress. However, the differences in behaviour between panel 50 and panels 60 and 61 are not particularly marked: this suggests that the effects of the bankseat loading are not large.

At higher depths of fill, the pressures become less than those indicated by the k_o line and generally tend towards the magnitudes of the calculated values of the active horizontal earth pressure, k_a. At depths of fill greater than 3 m, there appears to be no significant difference between the behaviour of panel 50 and panels 60 and 61.

The general trends in the horizontal earth pressures recorded at Otley are similar to those observed by other workers (see for example Murray and Boden, 1979). As the fill is compacted, the imposed vertical stresses induce horizontal stresses: when the compaction equipment is removed, the vertical stresses reduce dramatically but high horizontal stresses can remain locked-in. The addition of
subsequent layers of fill will steadily increase the vertical stress but the horizontal stress will remain unchanged until the vertical stress is sufficiently large to induce a horizontal stress greater than the locked-in compaction stress. As more fill is added and compacted, the horizontal earth pressure will then follow either the $k_o$ or the $k_a$ line or fall between them depending upon the properties of the backfill and the wall. At very small depths of cover, the compaction stresses frequently produce horizontal pressures above the passive horizontal earth pressure, $k_p$. However, only very small outward movements of the wall would be required to significantly reduce pressures from the passive towards the at-rest condition.

5.3 TENSILE ANCHOR LOADS

The anchor loads recorded at Panel 50 and the average values for panels 60 and 61 are shown in Figure 14. The highest recorded loads were in the region of 8–10 kN and occurred at both the top and the bottom of the wall. For panel 50, the high load recorded in the uppermost anchor may be a result of locked-in compaction stresses, but for panels 60 and 61 it is likely that the high load is a result of both locked-in compaction stress and the bankseat loading. For depths of cover between about 1 m and 4 m, the anchor loads for panels 50, 60 and 61 are reasonably constant at about 5 kN. The loads then increase to about 10 kN for the lower layers in each panel. With the exception of the uppermost anchor in panel 50, all the anchor loads are less than the working loads derived by applying a factor of safety of 2 to the ultimate design pull-out forces derived by the method given in Table 1. The average factors of safety on design pull-out force are 3.6 for panel 50 and 5.7 for panels 60 and 61.

A comparison between the results obtained from one of the anchor load cells and an adjacent earth pressure cell is shown in Figure 15. The data have been plotted against time for each of these cells, which were located in panel 50 at anchor level 2 under approximately 4.5 m of fill. After the backfill was placed, the anchor load cell recorded a reasonably constant load: the horizontal earth pressure cell showed a
similar trend but the data were rather more scattered. This is probably because pressure cells respond to changes of pressure in a relatively small local volume of fill whereas anchor load cells respond to changes over a significantly larger volume and are therefore more likely to provide representative results. Indeed, it is generally accepted that the measurement of loads is considerably more straightforward and accurate than the measurement of earth pressures; see, for example Green (1973).

5.4 PANEL DISPLACEMENTS
Displacement measurements were made during and after construction on panels 50, 51, 60, 61 and 62. Mean settlements and horizontal displacements at the top of the panels are plotted against time in Figure 16. For simplicity the data for panels 50 and 51, which were some distance from the footbridge bankseat, have been combined together as have the data for panels 60 to 62 which were directly under the bankseat. It can be seen that the majority of the settlement and horizontal movement occurred during construction and that the removal of the props after all the fill had been placed had very little effect. The data also show that the panels under the footbridge behaved in a similar manner to those some distance away from it. In both cases, the maximum settlements were about 10 mm and the horizontal displacements at the top of the panels were about 20 mm. Measurements of
horizontal displacement made near the base of the wall were consistent with an outward rotation of the wall about its base of 0.2°.

5.5 ANCHOR PULL-OUT TESTS

A series of pull-out tests was conducted on a selection of anchors located in panels 34 to 36, well away from the other instrumented sections of the wall. The anchors included four of 650 mm, two of 900 mm and one of 1200 mm with depths of fill ranging between 1.75 and 4.25 m. Incremental loads were applied with a hydraulic hand pump and hollow jack. The loads were measured with a 200 kN transducer and digital voltmeter: a dial gauge with a resolution of 0.01 mm was used to monitor anchor displacement. The system used is shown in Plate 6. The anchors were pulled in 0.5 mm steps until the design load had been reached: after this the rig was removed and the anchor returned to its original tension. The results obtained are summarised in Table 1: these show that the largest anchor displacement required to reach the design load was less than 3 mm.

For anchors of the type used at Otley, the pull-out resistance comprises two components, one due to shaft friction and the other due to the resistance of the anchor head. When an unsleeved pull-out test is performed, the measured resistance includes the shaft friction generated along the entire length of the shaft. In addition, this shaft friction includes an enhanced contribution from the restraint of dilation within the fill immediately behind the wall. However, in design, only the shaft friction for the portion of the shaft beyond any potential failure plane can be used. To overcome this problem, pull-out tests in the laboratory are normally conducted with the shaft of the anchor enclosed in a sleeve so that the pull-out resistance of the anchor head alone...
can be determined. The shaft friction component can be determined by performing a similar sleeved test on a short section of shaft with no anchor head or by calculation, as described in Section 3.2. In the present tests, the anchors were not sleeved because of the difficulty of installing such a system on site: the results of these tests were therefore nonconservative and should not be compared with design pull-out resistances, which were based on sleeved pull-out test results.

6 CONCLUSIONS

Construction of the anchored earth wall at Otley proved to be straightforward: after an initial learning period, the panel erection, alignment and propping rate reached a maximum of 7 units per day. The anchors were easy to handle and after positioning did not hinder the placing and compaction of the backfill. The temporary timber propping for the deep porous drain at the back of the fill area did, however, cause a problem. These props proved to be a considerable obstacle to placing and compacting the fill for the first 2 m depth. Once the timber props had been removed, the rate of placing of the fill improved considerably and peaked at a rate of 100 m$^3$ per day.

The preliminary results obtained from this study indicate that the following conclusions can be made:

(1) The vertical settlement of the wall was about 10 mm and the wall rotated outwards by about 0.2° about the base. The majority of these displacements occurred during construction.

(2) The tensile loads in the anchors are less than the working loads derived from the design, which is based on sleeved pull-out test results: the average factor of safety is 3.6 for panel 50 and 5.7 for panels 60 and 61.

(3) The pull-out resistances of the unsleeved anchors measured on site all exceeded the design values for anchor displacements of less than 3 mm.

(4) Although the data are rather scattered, the anchor loads and horizontal earth pressures are reasonably consistent for panels 60 and 61 below the footbridge bankseat and for panel 50 where there is no surcharge loading.

(5) The vertical earth pressures measured at the base of the structure are reasonably consistent with both the profile of overburden stress and the trapezoidal stress distribution assumed in DTp Technical Memorandum BE 3/78.

(6) The vertical earth pressures measured under the footbridge bankseat indicate that the stresses induced by the bankseat loading are rapidly dispersed in the first metre or so of fill.

(7) During the first six months since the end of construction, no significant movements or changes in earth pressures and anchor loads have occurred.

Although the measurements taken to date indicate that the structure is performing well, long-term monitoring of loads, pressures and displacements will continue so that a history of the performance of the structure can be built up. In addition, the long-term monitoring of the corrosion specimens in the structure will yield useful information on the durability of both reinforced and anchored earth structures.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

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