EARTH PRESSURES AGAINST AN EXPERIMENTAL RETAINING WALL BACKFILLED WITH HEAVY CLAY

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ABSTRACT

This report describes a pilot scale study of the compaction and swelling pressures developed by a clay backfill against two experimental retaining walls. The plasticity of the clay was well outside the upper limits for cohesive fill to structures given in the current specification (Department of Transport 1986). The investigation forms a sequel to studies using backfills of sand and silty clay (Carder, Pocock and Murray 1977, Carder, Murray and Krawczyk 1980) and was carried out in three main stages.

During Stage 1 the clay was placed in an ‘as dug’ condition and compacted in layers to a depth of 3 metres. On completion of filling the measured total lateral pressures were considerably in excess of the calculated ‘active’ and ‘at rest’ values. Stage 2 was a four week rest period during which significant reductions in lateral pressure were measured on both experimental walls. Sand drains were then installed and the water level maintained constant at the surface of the fill during the 20 month swelling stage (Stage 3). The average total lateral pressures on both walls rose rapidly to reach maximum values about six months after the start of swelling and then decreased over the following seven months. On completion of Stage 3 the measured lateral pressure over the upper 1–1.5 m on both walls were substantially in excess of the calculated limiting passive values.

1 INTRODUCTION

New road construction and improvement schemes to the existing road network form a major component of renewal and development programmes in urban areas of the United Kingdom. Earth retaining structures above and below existing ground level are required to minimise land take and provide grade separation at intersections. The cost of these structures frequently constitutes a significant proportion of the total cost of such schemes.

The material used for backfilling behind conventional retaining walls above ground level should ideally be free draining, easy to compact, and have high strength and stiffness properties in service. These requirements are of a particular concern for walls forming part of the abutments for road bridges, where differential movements between the backfill and the structure must be minimised to prevent a deterioration in riding quality. For these reasons engineers have traditionally used high quality granular materials to backfill the zone between the approach fill and retaining structure.

In some areas of the United Kingdom, such as the south east of England, granular materials are becoming increasingly scarce, and in urban areas particularly, their importation from further afield is likely to be expensive. Lower quality materials many of them cohesive are often available on site and if suitable, their acceptance could lead to significant savings in the cost of backfill material. This situation is reflected in the latest edition of the Specification for Highway Works (Department of Transport 1986) which permits the use as backfill, of clay soils of up to intermediate plasticity (Liquid Limit \( \leq 45 \) per cent, Plasticity Index \( \leq 25 \) per cent).

The use of cohesive materials as backfill introduces additional uncertainties into the retaining wall design since there is no established method of estimating the pressures produced by compaction, and little information available on the magnitude or effect of volume changes occurring after construction. In assessing the economic advantages of using such materials as backfill, full account must be taken therefore of any additional structural costs arising from such uncertainties.

This report describes the results of a study of the compaction and swelling pressures developed by a heavy clay backfill placed between two experimental retaining walls in a pilot scale facility at TRRL. The plasticity properties of the clay were well outside the limits currently specified. The investigation was carried out in conjunction with the Civil Engineering Department of the University of Surrey. The work forms a sequel to earlier studies of the lateral pressures developed during construction in backfills of sand and silty clay (Carder, Pocock and Murray 1977, Carder, Murray and Krawczyk 1980).

2 THE EXPERIMENTAL RETAINING WALL

The study made use of a pilot scale retaining wall facility housed within a large warehouse type building at the Laboratory. A cross section through the experimental structure is shown in Figure 1 and consists of a 2 m high movable metal wall and a 3 m high rigid concrete wall, both 6 m in length and separated by a 5 m wide central bay for placement of the fill. These walls are located at the centre of a 20 m long soil bay and precast concrete wall units provide support for approach ramps of fill placed on each side of the structure (Plate 1).

The movable wall is made of three independently supported steel panels, designed for minimum
Plate 1: TRRL Pilot Scale Retaining Wall Facility

structural deformation. This wall can be moved laterally by up to 400 mm on low friction bearings or rotated by up to ±17 degrees out of vertical, using a jacking system capable of applying a total thrust of up to about 2 MN (200 tonnes). The central panel is mounted on 4 horizontal and 2 vertical load cells and fitted with three vertical profiles each containing six cells to measure the total normal stress. Each cell is 140 mm in diameter and is mounted flush with the face of the wall. The two outer panels protect the instrumented central panel from side effects during movement of the wall. The rigid reinforced concrete wall is 1 metre thick and forms a continuous unit with the base of the trough (Figure 1). A two metre square steel panel inset into the upper part of this wall contains an array of total pressure cells similar to that on the central panel of the metal wall.

Displacement of the movable wall can be determined by reference to tensioned wires positioned at two levels behind the wall. Measurement of movement can be made at six positions along each wire, and involves the use of a micrometer to reposition an ‘air sensor’ onto the wire. The absolute positions of these wires are established using an optical plummet sighted onto two invar tapes fixed at the centre of the compaction bay. Fuller details of this measurement system are given by Carder et al. 1977.

3 MAIN STAGES OF THE INVESTIGATION

The study was carried out in three main stages. The first stage (Stage 1) involved the construction of the fill between the experimental retaining walls and extended over a 13 day period. The second stage (Stage 2) comprised a 27 day period (Day 14 to Day 40) during which the fill was sheeted to prevent
dessication and the pressures induced by compaction were allowed to reach equilibrium. Vertical sand drains were then installed through the fill and additional instrumentation incorporated in preparation for the final stage of the investigation (Stage 3) during which swelling of the fill took place. This stage extended over a 20 month period (Day 69 to Day 692).

4 THE FILL MATERIAL

The fill material used was London Clay obtained from Summerlease clay pit located near Colnbrook, Buckinghamshire. The clay was excavated by backacter from depths of between 2 m and 5 m below natural ground level and transported to the Laboratory by lorry.

To minimise any changes in moisture content, the fill was sheeted in transit and placed as soon as possible after delivery. The maximum delay between excavation and compaction was 48 hours. The particle size distribution and results of plasticity and specific gravity measurements on the fill are given in Figure 2.

5 INSTRUMENTATION

Three types of boundary total pressure cell, employing different operating principles, were used to measure the distribution of lateral pressure acting on the walls. Cells of the hydraulic and pneumatic types described by Carder and Krawczyk (1975) were used together with vibrating wire cells of the type described by Moore (1985).

The movable wall was fitted with one vertical profile of hydraulic cells and two vertical profiles of pneumatic cells while the rigid wall contained one vertical profile of each of the three types of cell. To extend the depth of measurement on the concrete wall, two additional cells in each profile were installed below the inset metal panel. The additional vibrating wire cells were mounted flush with the wall face while the hydraulic and pneumatic cells stood proud by 5 mm and 11 mm respectively. The layout of the pressure cells is shown in Figure 3.

Pneumatic cells were also used to measure the total vertical pressures on the concrete base slab and within the fill. The location of these cells are shown in Figure 4. At the lowest level the cells were bonded directly onto the base slab while at the higher levels each cell was installed on the surface of a compacted layer and placement and compaction of the fill above the cell continued in the normal manner.

Prior to the commencement of the swelling stage (Stage 3), four hydraulic piezometers using high air entry ceramic tips were installed in boreholes sunk to depths of 1, 1.5, 2 and 2.5 metres below the surface of the fill at positions midway between sand drains. These enabled the changes in porewater pressure occurring during swelling of the fill to be monitored.
Seven surface levelling stations, each consisting of an 150 mm square steel plate attached to a 600 mm long spike were installed to measure heave of the fill surface. Levels were also taken at two points on the top of the movable wall and benchmarks established on the top face of the concrete wall. The layout of the sand drains used for Stage 3 of the experiment, and of the additional instrumentation is shown in Figure 5.

**6 CONSTRUCTION OF THE FILL (STAGE 1)**

Prior to the construction of the fill a compaction trial was carried out in the soil bay to the west of the experimental structure. The purposes of this trial were:

(i) to compare the results from standard methods of determining density and moisture content of the fill with those obtained using a nuclear density gauge (TROXLER 3411-B), operated in direct transmission mode.

(ii) to establish the layer thicknesses and compactive effort necessary to achieve an air void content in the fill of less than 10 per cent.

For the trial the clay was rotavated and its moisture content adjusted to give values of 25 and 30 per cent. At each moisture content the fill was compacted using two levels of compactive effort; namely 2 and 8 passes of a DYNAPAC LC70 vibrating tamper. The results from the study demonstrated the feasibility of the nuclear method for density measurements during the main construction stage. The compaction trial is fully described by Hiedra-Cobo (1986).
Fig. 4 Location of total pressure cells within the fill
(After Hiedra – Cobo (1986))
Fig. 5 Location of sand drains, piezometers and levelling stations
Construction of the fill commenced on 3 October 1985 (Day 0). To simulate normal construction practice, the fill was placed in its 'as dug' condition without rotation, and this resulted in lumps of intact clay of up to 200 mm in size being present in the completed fill. Because of the restricted access to the area between the 0.5 m high side walls to the experimental structure, heavy plant could not be used and the first six layers of fill were therefore compacted using two DYNAPAC LC70 vibrating tampers. Each layer was given eight passes of the vibrating tamper and the average compacted layer thickness achieved was 125 mm. Above this level the fill was spread in layers and the surface levelled using two passes of a 7 tonne self propelled vibrating roller operating without vibration. The roller used was a DYNAPAC CA15R fitted with a steel roll (Plate 2). A 700 mm wide strip adjacent to each wall was then compacted using twelve passes of the vibrating tampers. The main body of the fill was compacted with a minimum of eight or ten passes of the vibrating roller operating at maximum amplitude, using the rolling pattern shown in Figure 6. The final height of the fill was 3.15 metres and the average thickness of a compacted layer was 121 mm.

During construction of the fill up to five sets of measurements, comprising readings of the pneumatic and vibrating wire pressure cells and of the position of the metal wall, were taken each day. Readings of the load cells and hydraulic pressure cells were recorded on a data logging system. A complete set of density measurements using both the sand replacement (BS 1377—Test 15A) and nuclear methods was also carried out on the fill twice each day at the locations shown in Figure 7. About half of the layers placed were tested in this way and density measurements were made on the remainder at four positions using only the nuclear density gauge. The average bulk density of the fill was found by the nuclear gauge, and sand replacement methods to be 1.88 Mg/m$^3$ and 1.83 Mg/m$^3$ respectively. The moisture content of the fill varied between 27 and 30 per cent with a mean value of 28.5 per cent, while the average air void content determined by the nuclear gauge and sand replacement methods were 8 per cent and 6 per cent respectively.

Following completion of the construction stage, continuous 100 mm diameter undisturbed samples of the fill were obtained by jacking in thin walled tubes at the locations shown in Figure 5. Triaxial compression tests carried out on 38 mm diameter specimens gave effective strength parameters of $c' = 15\text{ kN/m}^2$ and $\phi' = 19^\circ$, and a mean undrained shear strength of 125 kN/m$^2$.

7 THE PERIOD AFTER CONSTRUCTION OF THE FILL (STAGE 2)

During Stage 2 (October to November 1985) the fill was sheeted to prevent moisture loss and a gradual reduction in total pressures and thrusts was observed. Measurements were made twice daily over this period and by Day 40, twenty seven days after completion of filling, the readings from the instruments had become sufficiently stable to allow preparation for the swelling stage to commence.

It was appreciated that the London Clay would take many years to swell if wetted only from its upper surface. To accelerate the swelling process, forty-two sand drains, 75 mm in diameter, were installed to depths of 2.0 m or 2.5 m at the positions shown in Figure 5. The sand in each drain was compacted using a hand tamper and a 150 mm diameter plastic pipe installed at the top of each drain to act as a water reservoir.

8 SWELLING OF THE FILL (STAGE 3)

On Day 69 the reservoirs were filled with water and the swelling stage commenced. After about two months a reduction in the rate of intake of water by the fill was observed, and the sand drains were modified as it was thought that they were becoming ineffective. The sand was removed and a 20 mm diameter perforated plastic pipe installed in each hole. The annulus around the pipe was then backfilled with sand and a porous cover fitted to the top of the pipe to prevent ingress of solid matter. The water reservoirs were then kept full until the study was terminated (September 1987).
Fig. 6 Pattern of compaction used for layers 8–26 (after Hiedra-Cobo (1986))
Fig. 7 Location of density tests (after Hiedra-Cobo (1986))
About three months after the start of swelling it became necessary to reduce the lateral thrusts on the metal wall to prevent overload damage to the upper load cells. This was achieved on Day 172 by making small adjustments to the jacking system.

Within a few days of the completion of the study, three 75 mm diameter boreholes were sunk through the fill to determine the moisture content profile and to obtain 38 mm diameter samples for quick undrained triaxial compression tests. These boreholes were located close to each of the experimental walls and near to the centre of the fill (Figure 5). Moisture content determinations and hand shear vane tests (Geonor) were also conducted on an 0.5 m square grid pattern established on two near-vertical faces A and B, cut into the fill during excavation of the clay at the locations shown in Figure 5.

9 RESULTS OF THE STUDY

9.1 OBSERVATIONS DURING AND AFTER CONSTRUCTION OF THE FILL (Stages 1 and 2)

The distribution of total lateral pressure for each profile of pressure cells on the metal and concrete walls on completion of construction (Day 13) are shown in Figure 8a and 8b respectively. The pressures on both walls show a considerable variation both within and between profiles and this can probably be attributed to a non uniform stress distribution on the walls caused by the lumpy nature of the fill. The smaller pressures measured on the metal wall are a result of the small movements which occurred during construction and of the vibration of this wall during compaction with the 7 tonne
vibratory roller. These movements will have caused mobilisation of shearing stresses within, and at the boundaries of the fill. At each level on the metal wall the reduction in pressure will be governed by the movements which took place subsequent to compaction of the fill at that level and not by the total movement of the wall which occurred throughout the construction period. Figure 8c therefore shows the movements of the metal wall which occurred between compaction of the fill at each level and completion of construction.

The distribution of total lateral pressure on the walls at the end of Stage 2 (Day 40) are given in Figure 9. Comparing these results with those on completion of filling (Figure 8), shows that the average reductions in total pressure on the three profiles of cells on the concrete wall ranged between about 3 per cent and 20 per cent with a mean value of 12 per cent. On the metal wall the profile of hydraulic cells and one of the profiles of pneumatic cells showed corresponding reductions of 12 and 6 per cent respectively, while the second profile of pneumatic cells showed an

![Figure 9 Horizontal total stresses at end of Stage 2 (Day 40)](image-url)
average increase of 18 per cent over the same period. This increase however was largely due to a substantial rise in pressure on one of the cells which had recorded a low pressure at the end of construction.

The total lateral thrust on the central panel of the metal wall determined from the horizontal load cell readings are shown in Figure 10. The total thrust rises to a peak value of 107 kN (10.9 tonnes) by the end of construction, although the reductions which are apparent during pauses in the work suggest that this maximum value is dependent on the rate of filling. Over the 27 day period of Stage 2, following completion of filling, the lateral thrust shows a steady fall to about 85 per cent of the value on completion of construction. The movements of the central panel of the metal wall measured at two elevations during Stage 1 and Stage 2 are shown in Figure 11. This illustrates that most of the wall movement away from the fill occurred during the later stages of construction and was followed by small movements in the opposite direction, that is towards the fill, as the lateral thrust decreased.

Figure 12 gives the corresponding total vertical shear force recorded by the load cells on the central panel of the metal wall. During placement of the first 0.75 m of fill in contact with the wall, the force acted in an upward direction. As filling progressed this upward force reduced until on Day 11 the direction reversed. The initial upthrust was possibly a result of upward displacement of the fill as the 7 tonne roller passed close to the wall. As the height of fill increased the direction of the imposed shear stress is likely to have reversed over the lower part of the wall leading to a nett downward force on completion of construction. During Stage 2 the downthrust showed a gradual increase to reach a maximum at about Day 35. The total vertical shear force acting on the metal wall was then 24.5 kN or about 25 per cent of the total lateral thrust recorded by the horizontal load cells (Figure 10).

The average vertical total stresses in the fill measured by the four pneumatic pressure cells installed at each of three levels are shown in Figure 13. The cells at the middle level showed close agreement in measured pressures throughout Stages 1 and 2 whereas the cells at the other two levels showed a considerable range of pressure readings. For example, on completion of construction the four cells installed on the concrete base gave a range of pressures of between 36 kN/m² and 87 kN/m² with a mean value of 64 kN/m², while at the shallowest depth the readings ranged between 19 kN/m² and 64 kN/m² with a mean value of 38 kN/m². These large variations in pressure can also be attributed to the lumpy nature of the fill. The average vertical pressures measured by these pneumatic cells

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**Fig. 10 Variation of lateral total thrust (measured by load cells) on 4m² central panel of metal wall during Stages 1 and 2**
Fig. 11 Movement of centre panel of metal wall during Stages 1 and 2

Fig. 12 Variation of total shear force on central panel of metal wall during Stages 1 and 2
Fig. 13 Average measured vertical total stress for pneumatic pressure cells in fill during Stages 1 and 2
significantly exceeded the calculated overburden pressures based on the bulk density and heights of fill above the cells at all stages of the construction. As would be expected the discrepancies between the calculated and measured values were much greater at the upper and middle levels where the cells were surrounded by fill; the boundary cells located on the concrete base giving average pressures much closer to the calculated overburden stress during placement of the fill. During the first week following completion of construction the cells at all three levels showed further increases in pressure but remained fairly stable thereafter. The performance of these pneumatic cells is described more fully in Appendix A.

9.2 OBSERVATIONS DURING SWELLING OF THE FILL (STAGE 3)

The total lateral thrust on the central panel of the metal wall during Stage 3 is shown in Figure 14. The thrust increased rapidly following inundation and had more than doubled about seven weeks later when reinstatement of the sand drains commenced. Little change occurred during modification of the drains but the thrust again increased rapidly following their refilling with water. On Day 172, adjustments were made to the jacking system to reduce the thrusts acting on the wall. The resulting decrease proved to be temporary, and by about Day 210 the measured horizontal thrust had reached a value which exceeded the previous peak. A maximum total lateral thrust of 265 kN (27 tonnes) was recorded at about Day 250 and the thrust then gradually reduced to reach a minimum value at about Day 500. Thereafter small increases in thrust occurred until cessation of the study on Day 692. The sudden fall in total lateral thrust which occurred at about Day 470 was mainly a result of a decrease in load of 15 kN recorded on both of the upper horizontal load cells. These reductions took place at the end of a 5 day period when the ambient temperature was below zero.

The corresponding total vertical shear force on the 4 m$^2$ central panel of the metal wall is given in Figure 15. Swelling of the fill caused a force to act in the

Fig. 14 Variation of total lateral thrust on central panel of metal wall (measured by load cells) during Stage 3
Fig. 15 Total vertical thrust acting on metal panel during Stage 3
upward direction on the wall so that after about 10 days the nett downward force induced during placement of the fill had reduced to zero. Thereafter the results show a similar pattern to those of the horizontal thrust with a peak vertical shear force of 61.5 kN (6.3 tonnes) acting in the upward direction occurring on about Day 250 and a subsequent reduction to a steady value by about Day 525. The maximum vertical force was about 23 per cent of the corresponding horizontal thrust and this decreased to about 16 per cent by completion of Stage 3. At the end of the period of sub zero temperature (Day 470) a reduction in shear force was recorded by the vertical load cells which was of similar magnitude to the decrease in total lateral thrust. These changes in load are possibly a result of a freeze-thaw cycle in the fill located close to the wall.

The distributions of total lateral pressure measured by the cells on both walls on Day 210 and Day 682 are shown in Figures 16 and 17. Comparison of these profiles shows a marked reduction in pressures over
the upper half of both walls by the end of the swelling stage (Day 692) together with much greater consistency in the pressure readings, from the cells in different profiles. This can probably be attributed to a more uniform stress regime caused by the softening of the clay. By Day 692 the average lateral total pressures had decreased by about 29 per cent on the metal wall and about 34 per cent on the concrete wall from the maxima recorded at about Day 210.

Damage to and malfunction of a number of cells occurred during the course of the investigation and the values have therefore been omitted from the plots. While the loss of these cells had only a second order effect on the average lateral pressures measured on the experimental walls, it resulted in significant changes in the average vertical pressures measured within the fill. This was a consequence of both the smaller number of cells and the large range of pressures measured by those cells installed on the

![Graph of Movable metal wall](image1)

![Graph of Rigid concrete wall](image2)

Fig. 17 Horizontal total stresses at the end of Stage 3 – Day 692
base and at 1 metre below the surface of the completed fill.

The vertical heave measured on the levelling stations positioned on the surface of the fill are given in Figure 18 and show movements of up to 128 mm. Similar measurements on the top of the metal wall showed a vertical movement of up to about 2 mm by about Day 210 with little change thereafter. This can be largely attributed to movement within the wall bearings during reversal of the direction of the shearing stress applied by the fill. The results in Figure 18 indicate that slightly smaller movements occurred on the stations closest to the experimental walls. The heave of the surface did not occur at a uniform rate; a marked increase taking place on most of the stations following the reduction in lateral thrust on the metal wall on Day 172. The rate of heave also increased in July 1986 (Day 275 to 300) during a period of hot dry weather. At about this time large reductions in lateral total pressure also occurred on some of the cells on both the metal and concrete walls. The results for three of the hydraulic cells on the metal wall over this period are shown in

![Figure 18 Heave of clay surface](image-url)
Figure 19. These reductions extended to a depth of about 1 metre, starting with the shallowest cells and progressing deeper with time. It was considered that these effects were possibly due to desiccation and onset of cracking within the fill and the surface was therefore flooded and resheeted.

The results of porewater measurements at four levels within the fill during the swelling stage are shown in Figure 20. In spite of the use of fine-pored ceramic piezometer tips with an air entry value of 1 atmosphere, regular deairing was necessary and resulted in considerable fluctuations in the measurements during the early stage of swelling. There is little evidence of significant changes in the initial suctions, except possibly at the shallowest depth of measurement, before about Day 350. By the end of Stage 3 the results indicate that porewater pressure equilibrium had been achieved over the upper half of the fill, although significant negative excess porewater pressures remained in the lower half relative to the hydrostatic distribution in the sand drains. Profiles of moisture content and undrained shear strength of the fill shortly after completion of Stage 3 are shown in Figure 21. These were determined from tests on samples obtained from boreholes sunk through the fill and from the grid established on two near vertical faces cut during excavation of the clay. The results show some reduction in undrained shear strength and increase in moisture content occurred over the full depth of the fill, with the greatest changes apparent towards the surface. Also shown on the profiles in Figure 21, are the ranges of values of moisture content and vane shear strength measured at each level on the grid of points on the two faces of the clay. Although these indicate some variations in the condition of the fill this was probably a result of differences in the positions of the sampling points relative to the sand drains. The results indicate no significant trend with position relative to the experimental walls.

![Figure 19: Seasonal effect on three typical pressure cells (hydraulic)](image1)

![Figure 20: Porewater pressures relative to surface level](image2)
10 DISCUSSION

In this section the lateral pressures measured on both the experimental walls during the three main stages of the study are considered in relation to conventional retaining wall design. Figure 8 gives the measured total lateral pressures on the metal and concrete walls on completion of construction. Also shown are the distributions corresponding to the at rest pressure (Ko line) and active pressure (Ka line) which represent reasonable design assumptions for a rigid and yielding wall respectively. These distributions have been calculated using the measured bulk density and effective angle of shearing resistance  $\phi'$ of the clay (Section 6), assuming zero porewater pressure within the fill. The at rest earth pressure coefficient is based on Jaky's (1944) expression, $K_0 = 1 - \sin \phi'$, and the active coefficient $K_a$ determined from Rankine's formula.

For both walls the measured pressures substantially exceed these distributions.

The average lateral pressure measured by the cells on each wall at various key stages in the study are summarised in Table 1 together with the average lateral pressure and shear stress on the central panel of the metal wall determined from the load cell readings.

On completion of construction the average lateral pressure on the metal wall was about 64 per cent of the average measured on the rigid concrete wall, and between 46 and 57 per cent based on a comparison of the results from earth pressure cells of the same type. The reduced pressures may be attributed to the small movements of the metal wall which occurred in the period between completion of compaction at any level and completion of filling (Figure 8c).
TABLE 1
Average Pressures Measured on the Concrete and Metal Walls

<table>
<thead>
<tr>
<th>Day</th>
<th>Average Lateral Total Pressure kN/m²</th>
<th>Average Shear Stress on Metal Wall kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>On Concrete Wall</td>
<td>On Metal Wall</td>
</tr>
<tr>
<td>13 End of Stage 1</td>
<td>Pressure Cells</td>
<td>47</td>
</tr>
<tr>
<td>40 End of Stage 2</td>
<td>Pressure Cells</td>
<td>41</td>
</tr>
<tr>
<td>210 During Stage 3</td>
<td>Pressure Cells</td>
<td>98</td>
</tr>
<tr>
<td>692 End of Stage 3</td>
<td>Pressure Cells</td>
<td>70</td>
</tr>
</tbody>
</table>

* Denotes shear stress acting upwards on the metal wall
+ Denotes shear stress acting downwards on the metal wall

Over the upper part of a rigid retaining wall backfilled with clay, the pressures 'locked in' by compaction are likely to be a function of the undrained shear strength of the fill. On this basis the average lateral pressures on the concrete and metal walls on completion of construction correspond respectively to about 38 per cent and 24 per cent of the measured undrained shear strength of 125 kN/m² (Section 6). These values are subject to the inaccuracies inherent in the measurement of earth pressure which are discussed more fully in Appendix A. In this context the average lateral pressure on the metal wall determined from load cell readings (Table 1) represents about 22 per cent of the undrained shear strength of the fill. The significant reductions in total lateral thrust on the metal wall measured during pauses in filling (Figure 10), indicate that these values are likely also to depend on the duration of the construction period.

The time dependent nature of the lateral pressures is further demonstrated by the changes which occurred during Stage 2, illustrated in Table 1. Comparison of the measured pressure distributions on Day 40 (Figure 9) with those on Day 13 (Figure 8) show that the largest reductions in pressure on both walls were measured on the cells located nearest to the surface of the fill. A similar relaxation in shear stresses acting upwards over the upper part of the wall (Figure 12) could account for the observed increase in nett downward shear stress during Stage 2 (Table 1).

During Stage 3, the thrusts on the metal wall (Figures 14 and 15) and the lateral pressures on both walls, increased rapidly to reach a maximum value about 6 months after the start of swelling (Day 250). Over this period the surface of the fill heaved by up to 50 mm (Figure 18). Comparison of the lateral pressure distributions on Day 210 (Figure 16) with those at the end of Stage 2 (Figure 9) indicate increases in pressure at all depths with no significant change in the shape of the pressure distributions on either wall. As indicated in Table 1, the average increase in lateral total pressure on the metal wall of 41 kN/m² over this period was less than the 57 kN/m² rise on the concrete wall; and this is probably a result of the adjustments to the jacking system carried out on Day 172 to reduce the lateral thrust. During the early stages of swelling the direction of the average shear stress acting on the wall reversed, to give a maximum upward shear stress of about 15 kN/m² on Day 250.

Over the remaining 15 month period of Stage 3 the surface of the fill continued to heave (Figure 18) and the piezometers showed a gradual rise in porewater pressures within the fill (Figure 20). These suggest that further swelling would have occurred if the duration of the study had been extended. However the total thrusts on the metal wall (Figures 14 and 15) started to reduce at about Day 275 and the reduction continued until a sharp fall occurred following a period of sub zero temperature (Day 470). Corresponding decreases in total lateral pressure acting over the upper part of both walls commenced at about the same time (Day 270) and these corresponded to the onset of a period of hot dry weather during the summer of 1986 (Figure 19). From about Day 500 the lateral pressures and thrusts showed only a small increase until completion of the study on Day 692.

Comparison of the pressure distributions on Day 210 (Figure 16) and Day 692 (Figure 17) show that during this period the largest reductions in lateral pressure occurred over the upper 1.2 metres of the concrete wall and over the upper 0.6 metres on the metal wall. The average decrease in lateral total pressure on the metal wall was only about half of that measured on the concrete wall over this period. The
average lateral pressures measured on both walls on completion of Stage 3 are shown Figure 22 together with the distribution of porewater within the fill. Over the upper half of the fill, the porewater pressures were approximately in equilibrium with the hydrostatic distribution in the sand drains, although significant negative excess pressures remained in the lower half relative to this distribution. The reduction in porewater pressure with depth may have resulted in part from underdrainage occurring at the interfaces of the fill with the timber packing and concrete base and also as a result of a reduction in permeability with increase in stress level. The departure from a hydrostatic distribution may not necessarily indicate the extent of the zone in which swelling was incomplete.

The average lateral total pressure and shear stress measured on the central panel of the metal wall (Table 1) correspond to a mobilised angle of wall friction of about $11^\circ$ assuming the porewater pressures in the fill were transmitted onto the wall. Also shown in Figure 22a are the limiting passive pressures on the metal wall calculated assuming $c' = 0$ and using the measured porewater pressure distribution and angle of shearing resistance of the fill ($\phi' = 19^\circ$) together with the mobilised angle of wall friction. The coefficient of passive pressure ($K_p$) was obtained from Caquot and Kerisel (1948). The measured pressures are considerably in excess of these calculated values over the upper metre of the metal wall, suggesting that a much higher angle of shearing resistance was being mobilised over the
upper metre of the fill. Also shown are the limiting passive pressures obtained for an angle of shearing resistance of twice the measured value (\(\phi' = 38^\circ\)) and these are in much closer agreement with the measurements over the upper half of the metal wall. Similar distributions of total passive pressure are shown for the concrete wall (Figure 22b). However since the shear stresses acting on this wall are not known a limiting angle of wall friction equal to the angle of shearing resistance has been assumed in the calculations. The apparent high angle of shearing resistance \(\phi'\) of 38° on the metal wall and 32° on the concrete wall may reflect the low stress levels and stress path to failure in the upper levels of the fill. In this context similar high angles of shearing resistance have been reported for the London Clay from tests under conditions of passive stress relief (Burland and Fourie 1985).

### 11 CONCLUSIONS

The earth pressures developed against two experimental retaining walls during the construction and swelling of a high plasticity clay backfill have been investigated. Based on the findings from this study the following conclusions were reached:

1. On completion of filling the lateral total pressures acting on both walls were considerably in excess of the calculated ‘active’ and ‘at rest’ pressures assuming zero porewater pressure in the fill. On the movable metal wall smaller pressures were measured than on the rigid concrete wall and this can be attributed to small wall movements during placement of the fill and to the effects of vibration from the compaction plant. The average pressure on the metal and concrete walls corresponded to about 24 per cent and 38 per cent of the undrained shear strength of the fill.

2. During short pauses in filling and over the four week period following completion of construction, significant reductions in lateral pressure were measured on both walls. These decreases were more pronounced near the surface of the fill and are probably associated with relaxation in the stresses induced by compaction.

3. Swelling of the fill was monitored over a 20 month period and resulted in surface heave of between 104 mm and 128 mm. By completion of the study the porewater pressures measured over the upper half of the fill were approximately in equilibrium with the hydrostatic distribution in the sand drains, although significant negative excess pressures remained in the lower half relative to this distribution.

4. The lateral pressures on both walls rose rapidly to reach maximum values about 6 months after the start of swelling. Over this period fairly uniform increases in pressure occurred at all depths with little change in the shape of the distributions. On the concrete wall the pressure cell readings indicated an average total lateral stress of 98 kN/m² compared with an average of 70 kN/m² on the metal wall. During the following seven months the average pressures decreased to reach steady values which then remained fairly constant until completion of the study. The largest reductions in pressure occurred towards the surface of the fill to give an approximately linear increase in lateral total pressures with depth over the upper 1–1.5 m on both walls. In this region the lateral pressures were then substantially in excess of the limiting passive values calculated using the measured angle of shearing resistance (with \(c' = 0\)) and porewater pressure distribution in the fill. This suggests that the mobilised angle of shearing resistance in the upper levels of the fill was much greater than the value determined from conventional triaxial compression tests.

5. Comparison of the moisture content and undrained strength profiles on completion of the study with the values measured on completion of construction showed some softening and increase in moisture content at all depths with the greatest changes apparent towards the surface of the fill.

### 12 ACKNOWLEDGEMENTS

The work described in this report was carried out jointly with the University of Surrey and forms part of the research programme of the Ground Engineering Division (Division Head: Dr M P O'Reilly) of the Structures Group, TRRL. The authors would like to thank the late Mr M J Irwin, Mr J V Krawczyk, Mr M D Ryley, and Mr G Alderman of TRRL, and Dr J C Hiedra-Cobo, Mrs M C Wicks and Mr P F Cheesman of the University of Surrey for their assistance in carrying out the study.

### 13 REFERENCES


APPENDIX A

MEASUREMENT OF EARTH PRESSURES

The difficulties of reliably determining the pressures within a soil mass or at a soil-structure boundary are well recognised and have been the subject of extensive investigation (US Waterways Experimental Station 1944, Trollope and Lee 1961, Tory and Sparrow 1967). Nevertheless the accurate measurement of earth pressures remains one of the most difficult tasks in geotechnical monitoring. The inclusion of a pressure cell within soil inevitably causes complex alterations in the stress field in its vicinity. Moreover the operation of the cell may cause additional strains in the soil and result in further modifications to the local stress regime. Superimposed on these considerations are the real variation in the stresses within the soil mass. Unless the cell is large in relation to the particle size of the soil, large variations in contact area between the cell and the soil will occur leading to significant errors in the measurement of the average pressure. This is particularly pertinent in the present study as the compacted fill comprised an assemblage of clay lumps within a matrix of remoulded clay; the size of many of the lumps equalling or exceeding that of the pressure cells. In general differences between the measured and actual pressures are likely to depend on the type of soil, the imposed stress level, the operating principle of the cell, and the method of installation (Symons 1973, Carder and Krawczyk 1975).

Pressure cells were used in the present investigation to measure the lateral total pressures on the experimental walls, and the vertical total pressures within the fill and on the concrete base. Differences in the performance of the cells can be expected in each of these applications and these are discussed in the following sections.

A.1 MEASUREMENTS OF LATERAL TOTAL PRESSURE ON THE EXPERIMENTAL WALLS

The action factor of a cell can be defined as the ratio of the measured to actual total pressure. For the pneumatic and hydraulic cells installed in vertical profiles on the metal wall average values of the cell factors can be determined by considering horizontal force and moment balance using the horizontal thrusts determined from the load cell readings. This procedure involves some extrapolation of the measured pressure distribution to the surface of the fill and to the base of the metal wall.

Throughout most of the investigation no clear trends with depth were apparent in the measured pressure distributions and it was therefore assumed that the lateral total pressures acting at the fill surface and...
the base of the wall were the same as the values measured by the nearest cells in each profile. Towards the end of the swelling stage the lateral pressures measured on both experimental walls showed approximately linear increase in pressure with depth over the upper 1 to 1.5 metres of the fill and zero pressure was therefore assumed at the fill surface for Day 692. The cell factors calculated by both methods at key stages in the investigation are shown in Table 2.

The results indicate that the hydraulic cells performed much more consistently than the pneumatic cells. During the first two stages of the investigation the pneumatic cells showed much greater overread than the hydraulic cells. Part of the increase in cell factors for the pneumatic cells between Day 13 and Day 40 can be attributed to a large increase in reading on one cell from an initially low value on completion of construction (Day 13). If an allowance is made for this it would increase the calculated cell factors for the pneumatic cells on Day 13 by about 0.1. During the early stages of the investigation the cell factors show reasonable agreement with the values of 1.04 for hydraulic cells and 1.42 for pneumatic cells obtained in an earlier study using silty clay as the backfill (Carder, Murray and Krawczyk 1980). Both types of cell gave measurements very close to the average pressure on Day 692 when appreciable softening of the clay had taken place. The effect of this softening is likely to reduce both the real variation in stresses within the fill and the overread caused by the mode of operation of the pneumatic cell.

Taking into account the cell factors given in Table 2 an assessment of the performance of the vibrating wire cells can be made by comparing the measured pressure distributions given by all three types of cell installed on the concrete wall. The measured pressure distributions shown in Figures 8 and 9 indicate that the vibrating wire cells were appreciably underestimating the lateral pressures during the first two stages of the investigation. However during the later stages (Figures 16 and 17) the readings from all three types of cell showed better agreement, particularly in the upper part of the fill where the greatest softening had taken place.

![Figure A1 Variation of measured total vertical pressure with increasing overburden](image_url)
TABLE 2
Average Cell Action Factors—Metal Wall

<table>
<thead>
<tr>
<th>CELL TYPE</th>
<th>METHOD</th>
<th>DAY 13</th>
<th>DAY 40</th>
<th>DAY 210</th>
<th>DAY 692*</th>
</tr>
</thead>
<tbody>
<tr>
<td>HYDRAULIC</td>
<td>FORCE BALANCE</td>
<td>1.09</td>
<td>1.08</td>
<td>1.05</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>MOMENT BALANCE</td>
<td>1.07</td>
<td>1.11</td>
<td>1.11</td>
<td>1.02</td>
</tr>
<tr>
<td>PNEUMATIC†</td>
<td>FORCE BALANCE</td>
<td>1.14</td>
<td>1.40</td>
<td>0.96</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>MOMENT BALANCE</td>
<td>1.05</td>
<td>1.39</td>
<td>0.95</td>
<td>1.06</td>
</tr>
</tbody>
</table>

† Mean of two vertical profiles
* Assumes zero lateral pressure at fill surface

A.2 MEASUREMENT OF TOTAL VERTICAL PRESSURE

The average total vertical pressure measured immediately after compaction of the fill by the four pneumatic cells installed at each level in the fill is shown in Figure A1. In addition the cell action factors based on the calculated overburden pressures and range of pressures measured on completion of filling are given. The cell action factors generally tend to decrease with increase in height of the fill above the cells. At the lower levels this may be due in part to reductions in vertical stress in the fill caused by the effect of shearing stresses acting at the interfaces with the experimental walls (Figure 12).

During Stage 2 the average vertical pressure measured by the cells at the base of the fill increased to about 1.3 times the overburden stress by Day 40, while the average vertical pressures within the fill showed little change. The cells installed on the concrete base therefore gave average cell factors on completion of Stages 1 and 2 which were very similar to those shown in Table 2 for the pneumatic cells on the metal wall. These compare with average apparent overreads of about 60 and 80 per cent for the cells installed within the fill at the lower and upper levels respectively.

Figure A2 shows the average total pressures measured on cells located 2 m below the fill surface during installation of the sand drains and swelling of the fill. Only three of the four cells installed at this level continued to function throughout this period. The results show a significant rise in vertical pressure during the early phase of swelling and similar increases were also measured by the cells which survived at the other levels within the fill. Some of this increase may have been due to the reversal in direction of the shear force acting at the interfaces with the experimental walls which occurred over this period (Figure 15). Towards the end of the swelling stage the average vertical pressure measured by the three cells at this level corresponded to a cell factor of about 1.5 based on the calculated overburden stress.

![Graph showing average total vertical stress in fill at 2m level during Stage 3](image-url)