The response of a reinforced soil wall to differential movement

Prepared for Safety, Standards and Research, Highways Agency

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Executive Summary

One of the attributes of reinforced soil systems is that they can tolerate high differential movement and this makes them particularly useful for constructing over soft ground as it eliminates the need for piled foundations. But the tolerance of reinforced soil systems to differential movement has not been investigated systematically and there is little information in the literature to enable the effects of differential movement on the forces sustained by the reinforcements to be assessed. Thus the level of safety of a reinforced soil structure subject to large movements cannot be quantified with much confidence, and this may restrict the use of such structures, for example in areas of mining subsidence.

This report describes the construction and instrumentation of a 4.4 m high reinforced soil wall. It then describes the response of the wall to a differential settlement (sagging) of about 1 in 38 along the face of the wall and 1 in 42 perpendicular to the face of the wall; the settlement profile was intended to simulate the formation of a sinkhole or mine shaft close to the front face of the wall. Finally the behaviour of the wall during its re-levelling is described.

The wall was built from hexagonal shaped reinforced concrete facing units, galvanised mild steel reinforcing strips and a crushed limestone aggregate. The performance of the wall during construction was largely as expected. At depths of cover greater than between about one and two metres, the mean tension developed in the reinforcements at points close to the back of the facing units was about equivalent to at-rest earth pressures. However compaction operations generated higher earth pressures at shallower depths of cover.

The performance of the wall during the settlement stage was good; the imposed settlement did not lead to gross distortion of the face of the wall nor did it generate excessive bending stresses at the connection between the facing units and strips. However, the tensions in the reinforcements increased significantly through the settlement and re-levelling stages. Current methods of design do not explicitly take account of increases in tension due to differential movement.

The response of a reinforced soil structure to differential movement is a function of a number of interacting factors and so the conclusions drawn from this test may not be widely applicable.

According to the BS 8006: 1995, foundation settlements are to be calculated through conventional soil mechanics approaches and it provides a preliminary guide to the maximum differential settlement that can be tolerated along the line of a reinforced soil structure. It states that the ‘normal safe limit … for discrete concrete panel facings’ is 1 in 100, whereas the settlement profile imposed during the test, and which caused no significant damage to the structure, was 1 in 38. Although the guideline values seem overly conservative, it should be appreciated that settlement led to a substantial redistribution of tension forces between the reinforcements in the test wall, and a significant redistribution of forces occurred at the end of the first settlement stage, at a differential movement of only about 1 in 200. Furthermore, the data from this experiment indicate that the common assumption of active earth pressures being generated behind the facing of a deflecting wall may not be borne out in practice. However, it should be appreciated that the imposed settlement profile generated three-dimensional rather than two-dimensional effects. Nonetheless, present methods of design may over-estimate the factor of safety against the rupture of reinforcements.

It would seem difficult to provide a simple calculation method for taking into account the possible effects of differential settlement for all types of reinforced soil structure but, clearly, more guidance is required to enable designers to exploit more fully the advantages of reinforced soil techniques.
1 Introduction

One of the attributes of reinforced soil systems is that they can tolerate high differential movement, see for example Brady (1987) and Worrall (1989). This makes them particularly useful for constructing over soft ground as it eliminates the need for piled foundations. But the tolerance of reinforced soil systems to differential movement has not been investigated systematically and there is little information in the literature to enable the effects of differential movement on the forces sustained by the reinforcements to be assessed. Thus the level of safety of a reinforced soil structure subject to large movements cannot be quantified with much confidence, and this may restrict the use of such structures, for example in areas of mining subsidence.

To investigate the performance of various modular wall systems to differential movement a pilot-scale indoor test facility was developed by TRL and the Bolton Institute of Higher Education (BIHE). The first experiment using this facility was undertaken to determine the tolerance of an anchored earth system to sagging movement: details of this experiment have been provided by Brady et al. (1993).

This report gives details of the response of a 12.6 m long, 4.4 m high reinforced soil test wall to differential settlement (sagging) and its subsequent re-levelling. The settlement profile imposed upon the test wall was intended to simulate the formation of a sinkhole or mine shaft close to the front face of the wall. The structure was built using the same system as used for the construction of a retaining wall at the A74(M)/A75 junction near Gretna: details of that structure have been provided by Brady and Barratt (1994), a view of it is given in Figure 1.

2 Details of test wall

The test facility was housed at Elton Street, Bolton. The central part of the floor, upon which the test wall was constructed, was formed from steel plates supported by a grillage comprising 18 steel beams, set at 0.5 m centres, connected in threes to form six boxed supports; the supported area was 9 m wide and 5 m deep. The arrangement is shown in Figure 2. The front end of each boxed support was supported by two screwjacks (jactuators) each driven by a reversible three-phase electric motor. The side and back walls of the 4.4 m high structure were formed from free-standing reinforced concrete wall units giving an enclosed area 12.6 m wide and 6 m deep.

2.1 Backfill

To provide a reasonable test of the stability of the test wall to differential settlement, a backfill of moderate shear strength was required. A fine grained soil would meet this requirement but would be difficult to use because of problems associated with the control of moisture content and the generation of excess pore water pressures through its compaction. Consequently, although it did not meet the specification for backfill to reinforced soil structures as given in the Specification for Highway Works (MCHW 1), a crushed limestone aggregate having a coefficient of uniformity of about 2 was selected. The grading curve of the aggregate is shown in Figure 3.

The results of a series of direct shear box tests showed that at its dry density within the structure the aggregate had a peak strength equivalent to an angle of friction ($\phi_p$) of 48°.

Figure 1 View of reinforced earth wall at A74(M) / A75 junction near Gretna
Figure 2 General arrangement of pit, steelwork, screwjacks and pivot

Figure 3 Grading curve of backfill to test wall
The specimens were still dilating when the tests were terminated and so constant volume conditions were not attained. However, from the semi-empirical relations proposed by Bolton (1986), the critical angle of friction ($\phi_{cv}$) was estimated to be about 35°; a shear box test undertaken on a loose specimen of the aggregate gave much the same value. These tests were undertaken using a normal stress of between 15 and 65 kN/m$^2$.

2.2 Facing units
Details of the pre-cast reinforced concrete facing units are shown in Figure 4 and their arrangement and the numbering in the wall are shown in Figure 5.

2.3 Reinforcements
The reinforcements were 50 mm wide, 6 mm thick, 5 m long galvanised mild steel strips having a specified minimum coating weight of 1000 g/m$^2$. Their centre-to-centre spacing was 0.8 m horizontally and 0.7 m vertically. Each strip was connected to a facing unit by two grade 8.8 galvanised M12 bolts in the manner shown in Figure 6.

A series of shear box tests showed that the angle of interface friction ($\phi_i$) between the strips and compacted backfill was about 38°.

2.4 Bearing pads and dowels
Bearing pads, comprising 20 mm thick, 40 mm wide, 750 mm lengths of ‘Korkpak’, marketed by Servicised Ltd, were placed between adjacent facing units (see Figure 4). Vertically adjacent units were keyed together with 20 mm diameter polypropylene dowels fitting into 30 mm diameter holes cast into the units.

2.5 Design
Calculations based on the development of at-rest earth pressures for $\phi = 48°$ (and also for active earth pressure conditions for $\phi = 35°$) indicated that the maximum pull-out force on a reinforcing strip would be about 7 kN, and the minimum factor of safety against pull-out failure was about 3.5. The rupture strength of a strip was dictated by the connection arrangement: the results of full-scale tests reported by Brady and Barratt (1994) showed that the strength of the connection was about 95 kN. Increasing the spacing between the strips would have reduced the factors of safety but might also have led to instability during construction. It was also necessary to guarantee the safety of the operatives and equipment throughout the subsequent settlement and re-levelling stages. The effect of differential movement on the behaviour of the test wall could not be forecast with certainty but stability would reduce. The over-provision of stability stems largely from the fact that the facing units and strips were designed for walls up to 10 m high, but the size of the test facility limited the height of the test wall to about 5 m.

3 Instrumentation

3.1 Load cells
The bottom row of half-sized and full-sized facing units was supported on load cells centred on a dowel fixed on the top of a steel channel. As shown in Figure 5, five lengths of channel, each supporting two or three units, were used to support the face of the wall.

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**Figure 4** Details of the facing units
3.2 Pressure cells

Six hydraulic pressure cells were placed on the steel floor along the centreline of the structure at distances from the back of the facing of 0.5, 1.0, 1.5, 2.0, 3.0 and 4.5 m. The cells were 350 mm long and 300 mm wide.

3.3 Strain gauged strips

Electrical resistance strain gauges were attached to 64 of the 132 reinforcements. The gauges were formed into a three-wire half-bridge assembly and attached in pairs on opposite sides of a strip to determine:

i. the mean axial and bending stresses developed close to the connection point of a strip with a facing unit; and

ii. the distribution of these stresses along the strips.

For (i), 64 strips were fitted with a pair of gauges close to their anchorage point, whilst for (ii) 12 strips were fitted with an additional four pairs of gauges located as shown in Figure 7. The position and numbering of the instrumented strips are defined in Figure 5.

Strain was determined from the change in the measured output of the bridge assembly and the sensitivity of the strain gauges quoted by the manufacturer: stress was calculated from the strain and the Young’s modulus of the steel. Checks were undertaken using dead-weight loading to confirm the calibration of a few pairs of gauges. Tension was calculated from the mean stress, which was calculated from the output of a pair of gauges and the cross-sectional area of the strip; tension could be resolved to about 0.1 kN.

The sign convention adopted in this report is for tensile stresses and sagging bending stresses to be defined as positive.
The arrangement of the bridge circuit was designed to eliminate the effects of temperature, but there was some noticeable change in the output of the strain gauges in the three months between the end of construction and the start of the settlement stage. Over this period the temperature reduced by about 5°C and, in all but a few instances, the calculated extreme fibre stress became more tensile: the magnitude of the changes were not randomly distributed. The results of subsequent laboratory checks suggested that the bridge circuit might not be fully compensated for temperature change. However, it is also possible that a change in temperature altered the stresses in the reinforcements. These aspects are discussed in Appendix A.

3.4 Thermometers
The temperature within the backfill was measured using ten platinum resistance thermometers: these instruments were installed about one metre back from the facing units at various horizons within the backfill.

3.5 Data collection system
The output leads from the load cells, strain gauges and resistance thermometers were connected to a computer controlled data logging system. The pressure cells were read manually but the readings were stored on computer.

3.6 Survey technique
At the end of construction, 20 mm diameter black and yellow targets were glued to each facing unit, in the manner shown in Figure 8, and also to points remote from the wall to act as references. In addition, concrete slabs were buried flush with the top of the fill and a pair of bricks mortared end-to-end on the top of each slab to form a target plinth. A short length of steel surveying tape was glued to the front face of the bricks and a ball bearing fixed to the top of them.

The positions of the targets on the face of the wall were determined relative to two reference stations using a Wild T2000 theodolite. The horizontal and vertical angles to each target were measured from both stations. Two sets of measurements were taken prior to the settlement stage and the mean differences between the pairs of readings for the 90 targets were 0.5 mm in the x-direction (parallel to the face of the wall) and about 0.2 mm in both the y (lateral) and z (vertical) directions.

Lateral movements on the upper surface of the backfill were monitored by sighting to the lengths of tape using the Wild T2000 theodolite fitted with a parallel plate micrometer. Settlement was measured by precise levelling of the ball bearings relative to independent benchmarks fixed at the back of the structure.

The survey data were logged and processed by a portable computer: purpose written software allowed the field observations for each target to be computed and compared in real time.

4 Construction stage
The sequence of construction was as follows.

i The steel channels and load cells were set out at the base of the structure; where necessary shims were used to level the cells.

ii The half-sized facing units of the bottom row were installed and the full-sized units then slotted in between them. The units were propped to give an inward batter of 1:20.

iii Fill was placed and compacted in layers up to the level of the reinforcements. To avoid excessive outward movement of the wall the compactor did not encroach within about 250 mm of the back of the facing units.

iv The row of reinforcements was connected to the facing units.

Figure 7 Positions of strain gauges on the steel strips

The gauge pattern on strips 45 to 50, identified in Figure 5

The gauge pattern on strips 51 to 56, identified in Figure 5

All dimensions in mm
Further fill was placed up to the next level of reinforcements or row of facing units.

Having reached the top of a row of facing units the ‘Korkpak’ pads and the polypropylene dowels were installed and the next row of units installed with an inward batter of 1:20. Temporary clamps were fitted to the top of the lower row to provide some restraint to outward movement.

Steps iii to vi were repeated until the wall was complete.

4.1 Compaction

The backfill was placed in 75 mm thick layers and compacted with six passes of a Wacker DPU 3345 vibrating plate compactor. The dimensions of the vibrating plate were 900 × 400 mm and the mass per unit area of the plate was 770 kg/m².

The density and moisture content of the fill were measured during construction. The uniform grading of the fill made it difficult to form a vertically sided hole, as required in a sand replacement test, and also for undertaking a core cutter test in accordance with the methods given in BS 1377: Part 9 (1990). Thus the following method was generally adopted.

i. A steel tube, having a chamfered cutting edge, was driven into the compacted backfill. The 128 mm long tube had an internal diameter of 100 mm and a 1 mm wall thickness.

ii. The aggregate was excavated from inside the tube, weighed, dried, and re-weighed.

iii. The depth of the excavated hole was measured in at least three places and the mean used to calculate the volume of the hole.

In all, six sand replacement tests, six core cutter tests and 55 cylinder tests (to the method described above) were completed. The results of the three sets of tests did not differ substantially and showed that the mean dry density and moisture content of the fill were about 1.68 Mg/m³ and 1.3 per cent respectively. The in situ dry density was about 96 per cent of the maximum dry density of the fill, determined to BS 1377: Part 4 (1990).

4.2 Results

4.2.1 Alignment of the wall face

As shown in Figure 9, at the end of construction the face of the wall had a concave profile. The placement of the backfill did not much disturb the initial 1 in 20 inward batter of the lowest row of units, which were propped, but had an increasing effect on the higher rows such that some of the uppermost row had an outward inclination of about 1 in 20. The absence of any wide steps between the facing units shows that the units rotated about their base and relative translation movements at joints between adjacent units were small.

4.2.2 Foundation loads

The sum of the measured loads increased reasonably uniformly with the height of the constructed wall but there was a wide variation in the load carried by each cell. The pattern in the distribution of load beneath the bottom row of the half-sized units varied somewhat. For two of them the load was reasonably evenly shared between all three supporting load cells, as shown for example in Figure 10(a). For two others, the load was reasonably evenly shared between the two outer load cells – the central cell registering a relatively low load. For the remaining unit, about 75 per cent of the load was carried by the central cell.

Figure 8 Arrangement of survey targets on facing units

Targets not shown to scale
For all the six full-sized basal units between about 75 and 90 per cent of the load was carried by one of the two supporting cells, as shown for example in Figure 10 (b). Thus it would seem that these units were not seated particularly evenly between the adjacent half-sized units.

The load carried per metre run through the full-sized units was consistently higher than carried by the adjacent half-sized units: the mean value for the former was a little over double that for the latter. Thus the tilting of the full-sized units did not seem to transmit any additional vertical force to the adjacent half-sized units.

The data show that downward acting frictional forces were generated on the back of the facing units, the mobilised angle of friction reduced with the depth of cover to about two metres and then remained reasonably constant for greater depths. At the end of construction the summed load measured by the cells was 384 kN whereas the gross weight of the facing units was about 231 kN.

4.2.3 Vertical pressures along centreline of wall
Initially, the pressure measured close to the back of the facing was lower than in the main body of the fill, but at the end of construction the distribution of measured pressure was reasonably uniform. There was little increase in the measured pressures following placement of the first two metres or so of fill. This phenomenon was probably due to arching of the fill over the body of the cell and may have been induced by its mode of operation. This problem is common to all active-type pressure cells.

4.2.4 Stresses generated close to the connection point of reinforcements
Typical data, as presented in Figure 11, show that for more than 80 per cent of the strips, compressive stresses were generated on the top surface and tensile stresses on the bottom surface – most of the other 20 per cent were installed towards the top of the wall. In general the mean extreme fibre stress, i.e. the mean axial or tensile stress, increased with increasing depth of cover but half the
Figure 10 Typical variations in measured vertical load at base of wall during construction
Figure 11 Stresses generated close to the connection detail during construction
difference of the extreme fibre stresses, i.e. the bending stress, did not change much following the placement of the first half metre or so of backfill.

The maximum extreme fibre stress recorded by the strain gauges was about 130 N/mm², comfortably below the nominal yield stress of the steel (350 N/mm²). The maximum mean axial stress of 38 N/mm² corresponds to a tension of about 12 kN, which was much lower than both the rupture strength and the pull-out resistance of the strips. The measured bending stress ranged between 100 N/mm² (sagging) and -42 N/mm² (hogging).

4.2.5 Tensions developed close to connection point of reinforcement

The tensions generated close to the connection points generally increased with increasing depth of cover but, as shown in Figure 12, the relations varied from strip to strip perhaps reflecting their sensitivity to the details of construction such as the positioning and subsequent movement of the facing units. Figure 13 shows the distribution of tension at the end of construction for the strips attached to the column of panels 061 to 064.

4.2.6 Lateral earth pressures

The measured tensions can be used to estimate the value of the ratio (K) of the horizontal and vertical stresses in the backfill adjacent to the facing units. Values were determined by dividing the measured tension (T) by the nominal overburden pressure (γH) acting at the level of the strip, and by the area of the facing unit (A) itself divided by the number of strips (n) attached to the unit (i.e. the assumed face area supported by a strip). Thus:

\[ K = \frac{Tn}{\gamma H A} \]  

(1)

It should be appreciated that the values of K derived for low heights of cover are particularly sensitive to small variations in γH and T.

The value of K for a particular strip generally decreased with increasing γH for pressures up to about 35 and 40 kN/m²: typical data are shown in Figure 14.

To take out some of the variability introduced by, for example, local arching effects in the backfill, the values of K for the end of construction were also determined from the summed tension (ΣT) of all the reinforcements fixed to a particular unit, divided by the overburden (γH) acting at the mid-height of the unit and the area (A) of the unit, i.e:

\[ K = \frac{\sum T}{(\gamma H)_{m} A} \]  

(2)

The distributions of the K values for the individual strips and the facing units, shown in Figures 15(a) and (b) respectively, confirm the general trend of decreasing K value with increasing depth of cover to about two metres. The data also indicate that the side walls did not have a significant influence on the tension generated in the strips close to the centreline of the structure.

The K values in Figures 14 and 15 can be compared to those given in Table 1 for a range of conditions. Note that the K values derived above are based on the nominal overburden stress. (Values of K have also been calculated for particular columns and rows of facing units.)

4.2.7 Distribution of stresses along reinforcements

The distribution of stress along the instrumented strips shows that the maximum tensile stress was generated close to the connection point of the strips and no significant bending stresses were generated further than about half a metre from the back of the facing units. Typical data are shown in Figure 16.

4.2.8 Distribution of tension along reinforcements

Typical distributions of tension, as given in Figure 17, show that in most cases tension was not developed over the full length of the reinforcements. The distributions at the end of construction show that the maximum tension was developed close to the back of the facing unit; this is consistent with data produced from other studies, see for example Brady (1992).

4.2.9 Mobilised angle of interface friction along reinforcements

The slope of the relation between tension and distance along a reinforcement (as shown in Figure 17) is a measure of the mobilised angle of interface friction (φi) between it and the backfill. Assuming, for simplicity and without much loss in accuracy, that friction is developed only on the upper and lower surfaces of a strip:

\[ tan \phi_i = \frac{(T_2 - T_1)}{2\Delta L w \gamma H} \]  

(3)

where T1 and T2 are the tensions mobilised at points a distance ΔL apart along a strip of width w, and γH is the nominal overburden pressure acting on the strip.

The derived values of φi are provided in Table 2.

4.2.10 Temperature

The test wall was constructed through the spring and into the summer of 1993. At the start of construction the mean temperature of the backfill was about 10°C, with the installation of the first row of strips the mean measured temperature was 14°C, but throughout the remaining construction stage the mean temperature was reasonably consistent at about 16°C. In November 1993, some three months following the end of construction, the mean measured temperature within the backfill was 11°C.

4.3 Discussion

4.3.1 Movement of the wall face

The movements of the uppermost rows of facing units were greater than recorded on the wall at Gretna, as reported by Brady and Barratt (1994). The structure at Gretna was up to three metres higher than the test wall but the former was more heavily reinforced and perhaps therefore more able to withstand the effects of compaction operations. Outward movements are also influenced by the
Figure 12 Tensions generated close to connection point of the reinforcements during construction
Figure 13 Distribution of tension generated close to connection point for units 061 to 064
Figure 14 Variation of K determined from the tensions in the strips
Figure 15 Distribution of the values of K at the end of construction in August 1993
Figure 16(a) Stresses generated along strip 45 during construction
Figure 16(b) Stresses generated along strip 46 during construction
Figure 16(c) Stresses generated along strip 53 during construction

### Depth of fill 1.5 metres

- **Mean axial stress (tension positive)**
- **Bending stress (sagging positive)**

### Depth of fill 2.1 metres

### Depth of fill 2.5 metres
Figure 17 Distribution of tension along reinforcements
properties of the backfill, the effectiveness of any props, and the type and weight of the compaction plant. The test arrangement made it difficult to install an effective propping system to the whole face of the wall. (Perhaps of greater importance is the fact that the wall at Gretna was constructed by experienced site operatives whilst that at Bolton was built by technicians and researchers inexperienced in construction at that scale.)

It is important to align an in-service wall within the specified limits and so construction movements are of practical concern. Construction procedures, such as the provision of temporary props and varying the initial inclination of the facing units, can be adopted to provide a better alignment than obtained with the test wall but the detail of such procedures would be determined by site experience. The use of large interlocking panels helps disguise misalignments of the face of a reinforced soil wall and the deflection of the test wall during construction (see Figure 9) did not adversely affect its appearance.

### 4.3.2 Foundation loads

With reinforced soil systems incorporating interlocking facing units, it is usual for the smaller ones (normally half-size units) of the bottom row to be set out at predetermined intervals and then for full-size units to be slotted in between them. This was the procedure followed for the test wall. Despite the care taken to ensure the accurate positioning of the lowest row of facing units in the test wall it is evident that a larger proportion of the weight of the overlying units acted through the full-size units. The distribution of load beneath in-service structures is not usually measured, and so it is not known whether such variations in load would be generated on site. Small movements of a foundation might generate a more even distribution of load but it is clear that the lower row of units, particularly to high walls, should be evenly bedded to avoid unwelcome concentrations of load.

A reinforced soil system based on the use of hexagonal-shaped facing units could be developed such that the bottom row of units were all the same size and so could be installed in a continuous sequence.

### 4.3.3 Lateral earth pressures

The data from the strain gauges installed close to the anchorage point indicated that compaction operations generated horizontal pressures of about 5 kN/m², but exceptionally up to about 8 kN/m², i.e. equivalent to the lateral earth pressure generated by between one and two metres of fill. The impact stress applied by the plate is a function of the dynamic modulus of the soil and the height of drop of the plate, but these cannot be estimated readily for this experiment. A simple doubling of the stress due to the dead weight of the vibrating plate, which was about 7.5 kN/m², may underestimate the maximum vertical pressure imposed during compaction operations.

Comparison with the information produced by O’Reilly (1991) indicates that a locked-in horizontal pressure of up to 8 kN/m² is realistic.

---

### Table 1 Earth pressure coefficients for a range of conditions

<table>
<thead>
<tr>
<th>φ (degs)</th>
<th>K₀ (= 1 – sin φ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>0.43</td>
</tr>
<tr>
<td>48</td>
<td>0.26</td>
</tr>
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</table>

(a) K₀ values

<table>
<thead>
<tr>
<th>φ (degs)</th>
<th>δ* (degs)</th>
<th>K₀</th>
</tr>
</thead>
<tbody>
<tr>
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</tr>
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<tr>
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</tr>
</tbody>
</table>

(b) K₀ values

1. Coulomb’s solution, see for example Terzaghi (1943).

* Positive values are for downward acting frictional forces.

### Table 2 Mobilised angles of friction along reinforcements at end of construction stage

(derived from tension/distance distribution – shown for example in Figure 17)

<table>
<thead>
<tr>
<th>Strip (see Figure 5) (see Figure 5)</th>
<th>Depth of fill above strip (m)</th>
<th>φ_i (peak) ³¹</th>
<th>φ_i (mean) ²²</th>
</tr>
</thead>
<tbody>
<tr>
<td>45</td>
<td>3.85</td>
<td>33</td>
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<tr>
<td>51</td>
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<td>30</td>
<td></td>
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<td></td>
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<td>**</td>
</tr>
<tr>
<td>56</td>
<td>0.35</td>
<td>**</td>
<td>**</td>
</tr>
</tbody>
</table>

1. From steepest slope of distribution, between two gauge points.

2. Best estimate of mean for three or more gauge points.

* Distribution difficult, or impossible, to interpret.

** Near-zero tension developed along strips.
Thus the coefficient of lateral earth pressure (K) had a high value at low depths of cover but it reduced with increasing depths of cover. At the end of construction the mean K value for the lower row of half-sized and full-sized units was 0.24 and equivalent to the at-rest pressure coefficient (K_r) for a \( \phi \) value of about 49°. This value is in excellent agreement with the peak value of K derived from direct shear box tests undertaken on the backfill at the in situ density. However the mean value is also equivalent to the active earth pressure coefficient (K_a) for a \( \phi \) value of 35° (the estimated critical state angle for the backfill) and a downward acting wall friction angle (\( \delta_w \)) of about 16°: see Section 4.3.4. (The mean value is quite different to the K_a value for a \( \phi \) value of 48°.)

Active conditions in granular soils could be expected to be generated at relatively small outward movements of the retaining wall, i.e. where the movement is greater than about 0.1 per cent of the retained height. (The displacement to attain active conditions is a function, inter alia, of the properties of the soil and the mode of movement.) Thus for the test wall, active conditions might be developed at displacements greater than about 5 mm. Because of the test set-up, it was not possible to measure absolute movements during construction. The situation is complicated because the outward movement of the wall increased incrementally through the compaction of each layer of fill and the placement of overlying rows of facing units. Thus the deviation of finished profile from that intended is not necessarily a good measure of the absolute deflection at all wall heights. The fact that higher-than-K_a values were generated at low depths of cover, through compaction operations, supports the view that at-rest pressures would prevail with increasing depth of overburden. Note also that the lowest row of units did not rotate much from their original position.

### 4.3.4 Frictional forces generated on the back of the wall face

The densification of the fill, by the action of the compaction plant and the placing of overlying layers, generated downward acting frictional forces on the back of the facing units. At the end of construction the mean mobilised angle of friction (\( \delta_m \)) between the central columns of units and the backfill was about 16°; this value was derived for a K value of 0.29, which was the mean value over the full height of the wall. This gives a ratio of \( \delta_m \) to \( \phi \) of 0.33 and of \( \delta_m \) to \( \phi_{cv} \) of 0.45. It is probable, however, that the mobilised interface frictional force varied with the depth of cover.

### 4.3.5 Stresses and forces generated close to connection point of the reinforcements

The downward acting frictional force on the back of the facing units would have reduced the vertical stress in a zone of backfill immediately adjacent to them. At the end of construction, the frictional force was equivalent to the weight of a 165 mm thick block of soil. (Note that the compactor did not encroach closer than about 250 mm from the back of the wall.) The density of the backfill adjacent to the units was, at least initially, a little lower than in the main body of the reinforced block. Placing the first few layers of fill over the strips would therefore generate some bending of the strips close to the back of the units. The resulting movements, and the sign and magnitude of the bending stresses, are affected by:

i. the initial relative levels of the anchorage point, reinforcement and backfill;

ii. the initial density of the backfill;

iii. the rigidity of the connection and of the joints between the facing units; and

iv. the bending stiffness of the reinforcement.

Given the inherent variability of construction operations and the sensitivity of the bending stresses to small movements, the magnitude of the bending stresses close to the connection point could not be expected to vary uniformly with either position or depth of cover. Nonetheless, the downward acting frictional force generated on the back of the units and the preponderance of sagging movements are consistent with the settlement of a zone of backfill adjacent to the units.

The movements and the stresses within the backfill adjacent to the wall affect the generation of frictional forces along the reinforcements in this zone and therefore the distribution of tension. Some of the effects could be expected to vary widely from site to site and it is therefore not surprising that the measured distribution of tension derived from studies of in-service structures often presents a rather confused picture. Given that some differential movement between the facing units, adjacent backfill and reinforcements seems unavoidable with in-service structures, it is probable that bending of the reinforcements will occur close to their connection with the facing.

### 4.3.6 Distribution of stress and tension along the reinforcements

The factors governing the generation of bending stresses have been discussed above. Uneven bedding would generate bending stresses in the main body of the reinforced block, but for controlled conditions such stresses should be relatively small – as indeed was the case at gauge points further than 0.5 m from the back of the facing units.

The distribution of tension along the reinforcements showed a peak value close to the back of the facing units, but the shape of the distribution does not allow the magnitude or position of the maximum to be fixed with any certainty. The locus of points of maximum tension could be viewed as the most likely line of rupture through the structure and modelled, for example, by a log spiral. Because the performance of reinforced soil structures can be dependent upon the details of their construction, in general the reported plots of the points of maximum tension derived from numerical analyses, or from experiments on small models, have limited applicability. (The locus will vary with the outward movement, i.e. from the end of construction to a collapse state.)
The horizontal stresses built into the backfill (and the accompanying tension in the reinforcements) by compaction operations would be relieved somewhat by outward movement of the wall face. Thus the reduction in tension in the zone adjacent to the wall and the thickness of the zone, might both be functions of compaction operations (principally the applied stresses), the thickness of the compacted layers, and the outward movement of the face. The line of maximum tension might be largely a function of construction operations but it is an essential feature of reinforced soil structures. The use of thin layers of backfill, as in the test wall, could be expected to produce little loss in the compaction stress built into the structure. In which case the reduction in tension (due to outward movement) and the thickness of the zone might both be small – as seemed to be the case with the test wall.

The variation of tension along the reinforcements indicated that the maximum mobilised angle of interface friction was about 34°: i.e. a little lower than the maximum value obtained from direct shear box tests. The mean values ranged between 13 and 30° and, themselves, had a mean of about 23°.

4.3.7 The effect of temperature change on the output from the strain gauges

**Tension**

As shown by the data given in Figures 12 and 13, in the three months following the end of construction the measured axial tensions increased. Over this period the temperature reduced by about 5°C. A histogram of the measured changes in tension at the gauge points is provided in Figure 18: the mode of the data was 0.8 kN, whilst the maximum was about 2.6 kN. Most of the values less than about 0.3 kN were recorded on the uppermost row of strips, whereas those in excess of 1.7 kN were recorded by gauges installed some way back from the face and towards the bottom of the wall. Indeed five of the seven values in excess of 2 kN were recorded on strips 45 and 51 at gauge points some way from the back of unit 061 (see Figure 5). The data are not entirely consistent, but it would seem that the change in tension varied according to position.

For a number of gauge points, the change in tension between the two stages is high relative to the tension recorded at the end of the construction stage. This is partly because, in this over-reinforced structure, the tension under working conditions was rather low, particularly on the upper rows of reinforcements. Furthermore, the tension could be expected to be low at gauge points towards the distal end of a strip, again particularly on the upper rows.

The changes in tension might have resulted from movements within the backfill, but although there were small changes in the measured foundation loads the surveys did not pick-up any change in the position of any of the targets. The results of the tests reported in Appendix A suggest that some change in tension might arise from the temperature sensitivity of the gauges. The variability of the changes in tension, as shown in Figure 18, makes it difficult to derive a ‘correction’ to account for the change in temperature. For example a ‘correction’ of say –0.8 kN applied to the gauges close to the connection point of the upper rows of strips implies that some of these were in compression at the end of construction.

**Bending**

The bending stresses also changed in the three months following the end of construction.

Leaving aside the uppermost row of strips, at about 70 per cent of the gauge points close to the connection point the bending stress increased only slightly: the mean change

![Figure 18 Histogram of changes in tension between end of construction stage and start of settlement stage](image-url)
in bending stress was about 2 N/mm² (sagging), and 95 per cent of the changes were less than (±)10 N/mm². For the uppermost row of strips, the change in bending stress recorded by 70 per cent of the gauge pairs close to the back of the facing units exceeded (±)10 N/mm² (the modulus of the mean change was about 14 N/mm²): furthermore about two-thirds of these gauge points indicated a reduction in bending stress.

Taken as a subset, it is difficult to see any particular trend in the data obtained from gauge points along the strips. In most cases the change in bending stress was small, i.e. less than about (±)2 N/mm², but changes of up to 12 N/mm² were recorded – with most of the higher values recorded on the upper rows of strips. The changes were equally distributed between hogging and sagging modes. Again, although the data are not entirely consistent, it would seem that the change in bending stress varied with position.

Comment

It would appear that the pattern of stress change was linked to the depth of cover and the distance from the back of the facing unit: taken together these factors might be a measure of the ‘restraining’ effect of the soil on a strip. It is possible that a temperature change would lead to minor distortion, such as twisting, of a strip. The movement would be a function of a number of factors including the ‘restraint’ imposed by the surrounding soil, the flatness of a strip as installed, and any grain introduced into the steel by rolling and cutting operations. The restraint of the soil would vary according to the level of normal stress, but significant variations in the stress acting on the top and bottom sides of a particular strip might only arise with the upper rows of strips. Indeed, as noted above, most of the significant changes in bending stress were recorded on the uppermost row. It is important to note that the highest changes in tension and bending were not recorded at the same gauge points. The absence of a proven mechanism of behaviour should not be used to dismiss the data, but without such a mechanism they cannot be understood and easily accepted. The investigation of the mechanism requires careful, temperature controlled, in-soil tests. The apparent temperature sensitivity of the strain gauges and the strips is discussed in Appendix A.

5 Settlement stage

5.1 Test procedure

Settlement was effected by lowering the jactuators, sequentially, by up to 2.8 mm: the arrangement is shown in Figures 2 and 19. Waves of settlement were applied until the required profile had been established. The movement was controlled manually by switching each jactuator in turn and counting the number of revolutions of the drive shaft.

It was planned to apply ten equal increments of settlement each giving a settlement of 15 mm at the centre of the front of the wall. However teething troubles with the control system led to the first increment being 21 mm and so the second increment was reduced to 9 mm. The gross movements of the jactuators at the end of each increment are shown in Figure 20.

The first increment of settlement was applied in November 1993 and the final increment in January 1994. The application of an increment was completed in half a day and there was a gap of 3 to 4 days between successive increments. The structure was re-levelled in April 1994. (This gap of three months was left to allow any creep movements and accompanying changes in load to occur.)

The initial movements led to the loss of backfill through gaps that opened up between the facing units. At the end of the third increment the total loss of backfill was no more than a few kilograms but to prevent further losses the wider gaps were filled with an expansive foam. This proved effective but further applications of foam were required towards the end of the stage.

Following each increment of settlement, readings were taken from all the instruments and the positions of all the targets were determined. A further round of readings was taken prior to the application of the next increment.

5.2 Results

5.2.1 Movements

The profile of settlement of the facing units at a particular horizon varied with position: as shown in Figure 21(a) to (c) the units arched around the centreline of the wall. At the end of the stage the settlement of the central unit was 117 mm but the movement of the jactuators beneath this unit was 150 mm. The supporting channels to the facing units (the arrangement is shown in Figures 2 and 19) moderated the differential settlement along the face of the wall, the ratio of the movements of the central unit to the underlying jactuators was about 0.8 throughout the stage. With hindsight it would have been better for each facing unit to have had its own supporting channel.

The imposed settlement led to some small rotation of the facing units towards the mid-point of the foundation. When viewed from the front those on the right hand side rotated in an anti-clockwise direction whilst those on the left rotated in a clockwise manner. There was also some small lateral movement of the facing units towards the mid-point of the foundation; the movements at the end of the settlement stage are shown in Figure 21(d).

As shown in Figures 22 and 23, increasing settlement generated increasing outward rotation of the reinforced block. Linear extrapolations of the plots suggest that, at the centreline, movements of the backfill extended to the back of the block. As shown in Figure 23, for the reference points at the centreline at the top of the structure the relations between settlement and outward movement were essentially linear.

The movements generated tension cracks within the reinforced block; a plot of their outcrop at the top of the structure is given in Figure 24. The visibility of the cracks increased through the settlement stage.

Settlement led to the outward translation and rotation of the facing units; the relations for the central column of units (061 to 064) are shown in Figures 25 and 26. The magnitude of the movements decreased with increasing distance from the centreline of the structure – the
maximum outward movement was about 30 mm. There was some relative movement at the horizontal joints between the units; the size of the movement varied with position but the maximum was about 10 mm.

At the end of construction the facing units had a shallow concave profile. The displacements of the units due to the imposed settlement reduced the initial concavity over the mid-height of the wall but exaggerated the outward lean of the upper row of units; nonetheless the movements did not unduly affect the appearance of the wall. There was no change in the position of the units in the three months between the end of the settlement stage and the start of the re-levelling operations.

A simplified block diagram of the movement at the centreline of the wall at the end of the settlement stage is shown in Figure 27. As indicated there, the sum of the area of the settlement trough at the surface of the structure and the outward bulge of the wall face was lower than the area of the settlement trough at the foundation. This might indicate that there was some slight densification of the backfill through the settlement stage, but it should be appreciated that the figure is a plane-strain representation of a three-dimensional situation.

5.2.2 Foundation loads
As shown, by example, in Figure 28 the loads measured by individual load cells were erratic; there was some significant time-dependent change in load following the imposition of an increment of settlement.

As might have been anticipated, the distribution of load along the base of the structure, given in Figure 29, shows that load was shed from the centreline to the sides of the structure; a substantial change was induced by a relatively small settlement.

5.2.3 Vertical pressures within backfill
The variation in the measured pressures is shown in Figure 30. The response of a cell would be affected by small relative movement between it and the backfill but, whilst the absolute measured values may not be reliable, the data clearly show a large reduction in vertical pressure within a 3 m wide zone from the back of the central section of facing units.

Figure 19 Arrangement of jactuators and supporting beams to front face of wall
Figure 20 The vertical movements of jactuators at the end of each increment of settlement
(a) Vertical movement following second increment of settlement

(b) Vertical movement following fifth increment of settlement

(c) Vertical movement following final increment of settlement

(d) Horizontal movement of facing units through settlement stage

Figure 21 Movement of facing units during settlement stage
Figure 22 Vertical movements at the top of the structure during the settlement stage
Figure 23 Relations between vertical and horizontal movements along centreline of the top of the wall
Figure 24 Plots of cracks generated at surface of reinforced block through settlement stage
Figure 25 Outward movements at the centreline of the wall during settlement
Figure 26 Relation between mean settlement and outward movement of facing units
Ignoring second order effects:

Area 1, i.e. of surface depression, = 0.14m²
Area 2, i.e. bulging of wall facing, = 0.08m²
Area 3, i.e. of foundation bowl, = 0.28m²
Apparent change in area = 0.06m²

Figure 27 Simplified block diagram of movements at centreline of wall at end of settlement stage
Figure 28 Typical variations in measured load at base of wall during settlement
Figure 29 Distribution of load on supporting channels during settlement stage
5.2.4 Temperatures
During the construction stage the measured temperature in the backfill increased from about 10°C to 16°C: a temperature of 16°C prevailed over most of the stage. At the start of the settlement stage the measured temperature was about 11°C, the temperature reduced throughout the stage to reach about 6°C at the end. Thus, the difference in temperature from the time of the installation of most of the reinforcements to the end of the settlement stage was about 10°C.

5.2.5 Changes in stress recorded by strain gauges
The changes in strain recorded by the gauges varied spatially and with the magnitude of the imposed settlement, and also sometimes within the time period between the application of successive increments of settlement. Therefore a simple and concise description of the changes recorded by the gauges cannot be provided. By the end of the stage about a dozen gauges did not provide any sensible readings; most of these were installed away from the connection point of the strips and, in most cases, failure could be attributed to the rupturing of the connecting cables.

Interpretation of the data is complicated by the change in temperature from the start of construction to the end of the settlement stage. As discussed earlier, it seems that the output from the strain gauges and/or the stresses developed in the strips were temperature sensitive. It is difficult to ascertain whether these changes (or what part of them) were real or merely a consequence of the measuring system. As stated above, the temperature drop from the end of the construction stage to the start of the settlement stage was about 5°C, and about equal to the temperature drop through the settlement stage. According to the data given in Appendix A, a reduction in temperature of 10°C might generate a change in the tension of about 1 kN (equivalent to a mean axial stress of 3 N/mm²).

Changes in bending stress close to connection point of the strips
About one third of the pairs of gauges installed close to the connection recorded quite small changes in bending stress during the settlement stage. With the application of the first increment, changes of more than 15 N/mm² were recorded only at a few gauge points on the lower three rows and at points on the uppermost row of strips. Furthermore, only a few of the gauge pairs installed on the lower rows recorded a monotonous change in bending stress with increasing settlement, however almost all of those on the upper two rows recorded increasing bending stress (sagging) with increasing settlement. Significant reductions in bending stress (of more than 15 N/mm²) were recorded only by the gauges installed on the lower three rows of strips. Data from a few pairs of gauges are presented in Figure 31.

Changes in mean axial stress close to connection point of the strips
The mean axial stress recorded close to a connection point did not change monotonically with increasing settlement, and the change in stress with settlement varied widely from strip to strip. Some relations between the change in stress and settlement are shown in Figure 32.

The application of the first increment of settlement generated a reduction in the mean axial stress within a triangle roughly bounded by strips 6 to 11 at its base and strips 48 and 54 at its apex; their positions are defined in Figure 5. Little change in stress was recorded on strips around the immediate periphery of this lower central
Changes in bending stress along the strips

The bending stresses derived from most of the gauges installed away from the connection point changed reasonably uniformly with increasing settlement. At the end of the settlement stage:

i. the changes recorded by gauges installed 0.5 m from the front end of the strips ranged from -30 N/mm² (hogging) to +50 N/mm² (sagging), but most changes were between -5 N/mm² and +15 N/mm²;

ii. the changes recorded further than 0.5 m from the front end of the strips were, with few exceptions, less than ±15 N/mm² and in most cases they were less than ±5 N/mm².

The changes in bending stresses for a number of gauge pairs are shown in Figure 33. It is apparent from a comparison of Figures 32 and 33 that the change in bending stress in the period between the application of successive increments of settlement was small compared to the accompanying variation in mean axial stress.

Changes in mean axial stress along the strips

At most gauge points the mean axial stress increased as a result of the imposed settlement: the biggest increases were generated at gauge points some distance from the connection. Examples of the changes are given in Figure 34. Such relations show a reasonably uniform change in mean axial stress over the full length of many of the strips.

5.2.6 Tensions developed close to connection point

The pattern of the change in tension recorded between the end of construction and the start of the settlement stage is shown in Figure 35(a), and the change during the settlement stage is shown in Figure 35(b). Comparison of these shows that in some cases the change in tension between the end of construction and start of the settlement stage was much the same as developed through the settlement stage.

The first increment of settlement led to a reduction in tension for most of the strips attached to the lower central triangle of facing units, but an increase in the tension on the strips outside this area.

The patterns of tension at the end of the settlement stage in January 1994 is shown in Figure 37(a) and (b). The values shown in Figure 37(b) were derived from the tensions reported in Figure 35(e): i.e. some allowance was made for the apparent temperature sensitivity of the gauges.

5.2.7 Coefficient of lateral earth pressure

The change in the values of K varied widely with increasing settlement and position: the pattern at the end of the settlement stage in January 1994 is shown in Figure 37(a) and (b). The values shown in Figure 37(b) were derived from the tensions reported in Figure 35(e): i.e. some allowance was made for the apparent temperature sensitivity of the gauges.

5.2.8 Distribution of tension along the strips

The distribution of tension along the strips changed as settlement proceeded. Interpretation is complicated by the failure of some of the gauges, due to rupture of the connecting lines, and the variability of the data. The distributions of tension along some of the strips at the end of the settlement stage are shown in Figure 38(a): no allowance or correction for temperature change has been applied to these data.

5.3 Discussion

5.3.1 Foundation loads, wall friction and movements within backfill

Foundation loads

A prediction of the distribution of foundation load was derived from the pattern of movement depicted in Figure 20 and the arrangement of the facing units. Overall the predicted and measured distributions, as shown in Figure 39, are in reasonable agreement. (Data are also shown in Figure 29(d), but note that the basal units were not supported on individual channels – see Figure 5.)

Wall friction

Because of the lateral transfer of the weight of the facing units, the distributions in foundation load shown in Figure 29 cannot be used to determine the mean frictional force acting on the wall. The distributions shown in Figure 39 might be used, but the reliability of such predictions would be low. The data indicate that a high frictional force acted downwards on the units at the edges of the structure and a relatively low frictional force acted on the lower central units, but see Section 5.3.4.
Figure 31 Change in bending stress recorded close to connection point of reinforcements
Immediately following increment
Prior to next increment

Figure 32 Change in axial stress through settlement stage
Figure 33 Changes in bending stress through settlement stage
Figure 34 Change in mean axial stress along reinforcements
(a) Changes in tension recorded between end of construction stage and start of settlement stage

(b) Changes in tension through settlement stage to January 1994 (rounded to nearest kN)

(c) Tension at end of settlement stage in January 94 (rounded to nearest kN)

(d) Tension developed close to connection point of reinforcement in April 94 (rounded to nearest kN)

Figure 35 Tension developed close to connection point of reinforcements
Figure 35 (Continued) Tension developed close to connection point of reinforcements
Figure 36 Tension developed along central columns of reinforcing strips during the settlement stage
(a) K values for January 94 (no temperature adjustment, i.e. based on data given in Figure 35(c))

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<td>0.24</td>
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</table>

(b) K values for January 94 (with temperature correction, i.e. based on data given in Figure 35(e))

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<td>0.28</td>
<td>0.20</td>
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Figure 37 Distribution of the values of K, based on nominal overburden stress
Figure 38 Distribution of tension along central columns of reinforcements
Figure 38 (Continued) Distribution of tension along central columns of reinforcements
** Movements within backfill **

The variations in the foundation loads and the output from the strain gauges installed close to the connection point of the strips (in the time between the increments of settlement) indicate that there was some time-dependent adjustment of the facing units, and perhaps also movements within the backfill. The absence of any corresponding variations in the output of the strain gauges installed away from the connection point show that any such movements were restricted to a fairly narrow band of soil adjacent to the units.

The position of an active wedge of soil could not be discerned from the distribution of bending stress or mean axial stress along the strips.

**5.3.2 Vertical pressure within backfill**

The data from the pressure cells show that arching occurred within the reinforced block. Thus the vertical stress at any horizon would have varied with distance from the back of the facing and from the centreline of the structure, and so its distribution within the block cannot be determined with any certainty.

**5.3.3 Stresses along reinforcements**

The changes in bending stress recorded by the gauges installed close to the connection points show that the facing units around the centreline at the base of the wall moved downwards relative to the backfill. On the other hand, towards the sides and the top of the wall the data indicate that the backfill settled more than the facing units. Thus the magnitude and direction of the frictional forces acting on the back of the units varied spatially through the settlement stage.

The maximum extreme fibre stress recorded at the end of the settlement stage was about 160 N/mm², comfortably below the nominal yield strength (350 N/mm²) of the reinforcements. Extrapolations of the data suggest that the extreme fibre stress on some of the uppermost strips would reach the yield strength at a settlement of about 400 mm. And extrapolation of the type of relation shown in Figure 31 suggests that the bending stress would reach the yield strength of some of the lower strips at a foundation settlement of about 700 mm. It should be appreciated that these are rather extended extrapolations. Nonetheless, they do confirm that significant bending stresses can be developed in the reinforcements through large relative movement between them and the backfill. It also confirms the importance of the selection and compaction of the backfill.

Settlement due to (internal) compression of a backfill might be an issue with particularly high walls and where the backfill is prone to crushing. Lee et al. (1994) could not explain completely the collapse of a series of reinforced soil walls in Tennessee and recommended that the effect of movements on the performance of such structures be investigated further. It appeared that the collapse of some of these structures was associated with excessive bending of the steel reinforcements brought about by large relative movement between the facing units and the backfill.

The bending stresses developed close to the back of the facing units, as a result of differential movement, are a function of the relative movement between the facing and reinforcement. Thus for the size of reinforcements typically used, the maximum bending stress might not be too sensitive to variation in the reinforcement spacing. The walls investigated by Lee et al. (1994) were much higher
than the test wall and therefore would have been more susceptible to the effects of differential movement.

5.3.4 Lateral earth pressures

Following first increment

The first increment of settlement led to a reduction in the lateral earth pressures acting on the units at the centreline of the bottom of the wall but the pressures seemed to increase towards the sides and the top of the wall.

At this stage the derived K value for the lower central triangle of units was a minimum. The tension developed at the connection point of 18 strips within this zone corresponded to K values ranging from about 0.14 to 0.36, their mean value was about 0.22. As noted above, the distribution of the vertical and horizontal stresses within the structure is not known but the distribution of the measured foundation load suggests that there was an upward acting frictional force acting on the back of these units at this time, and this fits with the changes in bending stress measured on the strips. But comparison with the type of block diagram used to produce Figure 39, suggests that no substantial friction force was necessarily present at this time. As shown in Table 1, a K value of 0.22 does not fit a value of Kc for a φ value of 35° for an upward acting frictional force. It does fit the Kc value for a φ of 48° but only with an upward acting interface angle of 40°. As can be seen from the table, for a particular value of φ the value of Kc increases as the downward acting frictional force reduces. However, as shown in Table 3, the value of K for this section of the wall seems to have decreased with a reduction in wall friction.

In contrast the K values outside the lower central units increased with increasing downward acting frictional force. The K values for the strips lying outside the lower central triangle varied with the depth of cover. The mean value for the outer units of the lowest row was about 0.4, for the overlying rows it increased successively from about 0.2 to 1.0. (Note that, as shown in Figure 5, the instrumented strips were not uniformly distributed across the full width of the wall.) The K value for the column of units 061 to 064 was about 0.32. The values do not conform to those expected for active pressure conditions and the direction of the frictional forces acting on the back of the wall: see Table 1.

It is evident from the above that the spatial distribution in the lateral earth pressures was not due solely to variations in the frictional force acting on the back of the wall face.

End of settlement stage

It would normally be assumed that active earth pressure conditions are mobilised by small outward movement of a retaining wall. Although some of the measured tensions decreased with the first few increments of settlement, later increments increased the total tension carried by all the strips. According to the data given in Figure 37(a) and Table 3, at the end of the settlement stage the mean K value for the bottom row of half- and full-sized units was 0.35 (the corresponding value from Figure 37(b) is 0.29). The maximum value was about 0.5 (but even with the allowance for temperature change it was about 0.4), and the K value for the central column of units was 0.42. Again these values do not fit the K values given in Table 1. The change in the K values through the settlement stage might have been associated with the lateral redistribution of vertical stress. But a comparison of the data given in Figures 15(b) and 37 shows that at the end of the stage the K values had increased across most of the width of the wall.

It would seem that the force acting on the back of a reinforced soil wall (or some part of the back) subjected to differential settlement can be higher than would be calculated assuming active earth pressure conditions. (Note that the K values given in Table 1 are based on the nominal vertical earth pressure and also that arching of these stresses occurred within the soil block.)

5.3.5 Distribution of tension along the reinforcements

Examples of the change in tension through the settlement stage are shown in Figure 40: as can be seen there was considerable variation in the pattern of change. It would seem that the possible change brought about by a decrease in temperature of about 10°C could, in many cases, equal or exceed that due to the imposed settlement.

Table 3 Values of K at various stages of the test

<table>
<thead>
<tr>
<th>Table 3 Values of K at various stages of the test</th>
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<tbody>
<tr>
<td>End of construction</td>
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<tr>
<td>Lowest row of units (021 to 101) (mean values)**</td>
</tr>
<tr>
<td>Strips attached to lower central triangle of units (mean)*</td>
</tr>
<tr>
<td>(strips 6 to 11, 21 to 24, 46, 52, 47, 53, 48, 54, 32, 33) (range)</td>
</tr>
<tr>
<td>Central columns of units (mean value over full height of wall) (range)</td>
</tr>
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</table>

* Arithmetic mean of individual data points.
** Areas of facing units taken into account in deriving mean value.
The temperature sensitivity of the strain gauges makes it difficult to interpret the data shown in Figures 38 and 40. Furthermore the plots in Figure 38(a) and (b) are rather sparse. Nonetheless the values of tan $\phi_i$ derived from some of the plots given in Figure 38(a), using Equation 3, are substantially higher than the values derived from direct shear box tests. For example, the slope through the gauge points at the end of strips 50 and 56 is equivalent to a tan $\phi_i$ value of about 5.5, i.e. about an order of magnitude higher than derived from the results of the direct shear box tests. Slightly lower values of tan $\phi_i$ can be derived from the plots in Figure 38(b).

A number of explanations can be proposed for this phenomenon, including the generation of high vertical stresses acting on the reinforcements, the development of some adhesive component of pull out resistance and, perhaps the most obvious and simplest, that the data are unreliable.

By analogy with the behaviour, for example, of spill-through abutments, as described by Randolph et al. (1985), the effects of arching might increase the vertical stress acting on the strips by a factor of about 4 to 5. As shown by the measured foundation loads, settlement altered the distribution of vertical effective stress within the backfill. It is probable that settlement of the wall would lead to an increase in vertical stress on some parts of the upper surface of the strips, but it might also reduce it on the underside of a strip. A difference in these vertical stresses should generate some bending, but only low bending stresses were measured during the settlement stage.

An adhesion of 10 kN/m$^2$ developed on both sides of a metre length of a 50 mm wide strip would generate a pull out resistance of about 1 kN. But this mechanism is not entirely convincing here because the accompanying cohesive forces would have presumably reduced substantially the tension required to maintain stability and yet the K values are higher than calculated for active earth pressure conditions.

The angle of interface friction could have been increased by embedment of the soil grains into the relatively soft zinc coating on the strips. At the limit, the value of tan $\phi_i$ would approach the value of tan $\phi$ for the

Figure 40 Change in tension through settlement stage
soil, but the effective dimensions of the strip would also be increased by embedment. Brady et al. (1990) found that the reaction between a zinc coating on a reinforcement and a carbonate-rich backfill can produce a tightly adherent rough surface layer, but the strips recovered from the test wall were not heavily encrusted with particles of backfill.

Pulling a reinforcement out of a granular soil will generate dilatancy within a zone of soil around the reinforcement. Where dilatancy is restrained the effective stress around the reinforcement will increase. The magnitude of the effect is a function of the roughness of the strip, the strength of the soil and the depth of cover (through its effect on the restraint provided by the soil). For rough axially stiff strips the increase in effective stress, and hence pull out resistance, can be substantial – i.e. up to an order of magnitude at low depths of cover (see, for example, Schlosser and Elias, 1978). Any such effect would, however, be expected to be less than this for the relatively smooth strips used in the test wall.

5.3.6 Influence of bending stiffness of reinforcements

John (1983) concluded from a series of tests on model and pilot-scale reinforced structures that their performance was influenced by the bending stiffness of the reinforcements, but the effects could not be quantified easily. The influence of the bending stiffness of soil nails on the performance of nailed slopes has been the subject of much debate, see for example Jewell and Pedley (1991). It is now generally accepted that the bending stiffness of reinforcements has little effect on the ultimate stability of a reinforced soil wall.

Relative movement between the reinforcements and backfill will be influenced by the bending stiffness of the reinforcements, which in turn may affect the arching of vertical stresses onto the reinforcements and hence the distribution of tension along them. Thus effects of large movements on structures incorporating widely spaced steel strips and ones with continuous layers of a polymeric reinforcement may differ in detail. The higher extensibility of polymeric reinforcements may make them less

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**Figure 40** (Continued) Change in tension through settlement stage
susceptible to rupture by concentrated bending forces, but only perhaps at the expense of incurring higher outward movement of the wall face.

6 Re-levelling stage

Some four months after the end of the settlement stage, the steel grillage was brought back more or less to its original level. This was achieved by raising each of the jactuators in turn in increments of about 10 mm: on completion the front of the grillage was between 5 and 11 mm lower than its original position.

Re-levelling was completed in a single day, and measurements were not taken from the various instruments during the operation. Furthermore not all the targets on the front of the wall were surveyed at the end of the stage. The value of closely monitoring the performance during re-levelling was not fully appreciated at the time. But, in mitigation, at that time it appeared as though the data from the strain gauges would not be intelligible.

6.1 Results

6.1.1 Movements

The initial positions of the facing units were more or less restored by the re-levelling operation, but the units were a little lower than at the start of the settlement stage – more or less reflecting the change in the level of the supporting grillage. For example, the units on the upper row were between 2 and 11 mm lower (with a mean of 8 mm) than they were at the end of construction.

The units moved inwards as a result of re-levelling. The units on the upper row moved back about 15 mm to be within 5 to 12 mm (with a mean of 8 mm) than they were at the end of construction.

As might have been expected, re-levelling led to a rotation of the facing units in the opposite direction to that recorded during the settlement stage. But as a result of the settlement and re-levelling operations the units were displaced laterally: the change in position increased, reasonably linearly, from zero at the centreline to about 20 mm at the sides of the structure.

Through re-levelling of the structure, cracks up to 3 mm wide were generated on the surface of the fill. The pattern of cracks at the end of this stage is shown in Figure 41: the cracks ran roughly orthogonal to those formed during the settlement stage: see Figure 24. The width of the cracks generated during the settlement stage narrowed during re-levelling operations but did not close fully.

6.1.2 Foundation loads

The distribution of foundation loads following re-levelling operations is shown in Figure 42.

6.1.3 Temperatures

At the end of the re-levelling stage, the mean measured temperature within the backfill was 6.7°C.

6.1.4 Stresses recorded by strain gauges

A number of the strain gauges installed on the lower rows of strips further than about 1.5 m from the connection point failed during either the settlement or re-jacking stages – the failures were probably due to rupture of the connection lines.

Bending stress

Relatively large changes in bending stress were recorded at the gauge points close to the connection point of the strips. The pattern of the change is shown in Figure 43(a). For more than 80 per cent of the gauge points close to the connection points of the strips to the central three columns of units, the bending stress at the end of the stage exceeded ± 50 N/mm². In about 10 per cent of these cases (all on the central column) the stress was negative (i.e. hogging). Bending stresses in excess of 100 N/mm² were recorded at a number of points, mainly on units adjacent to the central columns. The maximum extreme fibre stress was about 210 N/mm². The pattern of the change in bending stress from the end of the construction stage to the end of the re-levelling stage is shown in Figure 44(a).

At the end of the re-levelling stage, the bending stresses recorded by the gauges other than at the connection point were within about ± 10 N/mm² of the values recorded at the end of the construction stage.

Mean axial stress

In the lower part of the wall, nearly all the gauges installed closer than 3 m from the back of the facing units recorded an increase in mean axial stress as a result of re-levelling. But most of the gauges on the top two rows of strips showed a reduction in stress.

For the gauges close to the connection point, the pattern of the change in mean axial stress generated by re-levelling is shown in Figure 43(b). The change in stress from the end of construction to the end of the re-levelling stage is shown in Figure 44(b).

At the few working gauges further than about 3 m from the back of the facing units, the mean axial stress returned to be within ± 5 N/mm² of that measured (without any temperature correction) at the end of construction.

6.1.5 Tensions developed close to connection point

The pattern of tension developed at the connection points is shown in Figure 45, and the distribution of tension with depth on the back of the three central columns is shown in Figure 46. Note that no ‘compensation’ has been applied to these data to account for the apparent temperature sensitivity of the gauges.

6.1.6 Lateral earth pressures

As previously, K values were calculated using Equation 2. The pattern of the K values across the wall at the end of the re-levelling stage is shown in Figure 47. Taking into account the relative areas of the facing units, the mean K value for the bottom row of half- and full-sized units was 0.37. A mean K value of 0.52 was also derived for the three central columns of units: this was calculated by dividing the sum of the tensions by the total area of the facing units and the appropriate total nominal overburden stress.
Figure 41 Plots of cracks generated within reinforced block by re-levelling
Note weight of facing units, without wall friction, would be about 18kN/m run: total weight of facing units = 231kN

Figure 42(a) Foundation loads following re-levelling of structure

Figure 42(b) Distribution of load on supporting channels following re-levelling
(a) Changes in bending stress, +ve values for sagging

(b) Changes in mean axial stress

**Figure 43** Changes in stress recorded close to the connection point through re-levelling stage (N/mm²)

(a) Changes in bending stress, +ve values for sagging

(b) Changes in mean axial stress

**Figure 44** Changes in stress recorded close to the connection point from end of construction to end of re-levelling stage (N/mm²)
All values in kN, rounded to nearest kN

Figure 45 Distribution of tension following re-levelling

Figure 46 Tension developed along columns of reinforcing strips following re-levelling

Figure 47 Distribution of the values of K for facing units following re-levelling
6.1.7 Distribution of tension along strips
Typical distributions of tension along the reinforcements are shown in Figure 48; again, no temperature compensation has been applied to these data.

6.2 Discussion
6.2.1 Movements within backfill, foundation loads and wall friction
Movements
The level of the foundation at the end of construction was more or less recovered by the re-levelling operation, and the level of the facing units reflected more or less that of the foundation. The outward inclination of the wall reduced through the operation, but there was some slight lateral spreading of the units from the centreline of the wall. The positions of the markers on top of the wall were not much changed from their positions prior to the start of the settlement stage.

At the end of the stage, the appearance of the wall was good, the facing units were not damaged in any way, and the wall was stable.

Foundation loads
As shown by comparison of the distributions given in Figures 39 and 42 the total frictional force acting on the back of the facing units and the distribution of foundation loads changed substantially with re-levelling. The lateral variation in the foundation loads, as shown in Figure 42, is consistent with the transfer of vertical stress from the sides to the centreline of the structure, as could be expected with the imposed movements.

Wall friction
As described earlier, it is not possible to determine the distribution of the wall friction acting on the back of the facing units. But the data shown in Figure 42 show that there was a substantial net downward frictional force acting on the back of the units at the end of the re-levelling operation, and also that it seemed to be concentrated on the central columns of units. Comparison with earlier data, shows that the force was much higher than developed at the end of the construction or settlement stages. It is difficult to predict accurately the distribution in the foundation load resulting from re-levelling. But at the end of the operation, using a block diagram of the type shown in Figure 39, the central three columns of facing units

![Figure 48 Tension along strips following re-levelling operations](image-url)
might support up to about 70 per cent of the total weight of the units. On this basis, and assuming a K value of 0.52 – as given in Section 6.1.6 above, the downward acting friction force would be equivalent to a mobilised angle of about 40°: this value is just about credible.

6.2.2 Lateral earth pressures
Again, because the stress regime in the backfill is not known, it is difficult to compare the measured K values with those calculated for various earth pressure conditions. But there is no reason to expect the K values to fit the values of K_0 given in Table 1.

At the end of the stage, the mean K value for the bottom row of half- and full-sized units was 0.37 and much the same (0.35) as calculated for the end of the settlement stage (see Section 5.3.4). However, at these times, the distributions in foundation load and therefore perhaps also the wall friction, were quite different: this can be seen by comparison of the data given in Figures 39 and 42.

The data for the two outer units of the bottom row indicate a K value of about 0.16; this is much the same as the low values recorded by the central units of the bottom row following the first increment of settlement (again see Section 5.3.4).

6.2.3 Distribution of tension along reinforcements
The consistency of the data, or better their conformity to that expected, suggests that whatever the mechanism responsible for the change in the output from the strain gauges between the end of construction and start of the settlement stage, it was nullified through the re-levelling operations. It could be postulated that the movements in the backfill, brought about by re-levelling, overcame the ‘restraint’ imposed on the strips by the backfill. If this was the case, a plausible explanation has to be found for the very high frictional forces developed along the strips through the settlement stage – see Section 5.3.5.

Because of the failure of the gauges, reliable distributions could only be established for four strips. But the calculated values of ϕ of 26, 27, 29 and 37° are more or less in line with the maximum values measured at the end of construction and are lower than the maximum measured in the direct shear box tests. As shown in Figure 48, a more or less triangular distribution of tension was developed along these four strips. If it is simply assumed that the tension distribution was indeed triangular with a maximum at the connection point and zero at the far end of a strip, the data shown in Figure 48 can be used to derive a lower-bound estimate of the interface coefficient of friction for all strip levels. The estimates of ϕ range between 19 and 25°, and the mean value is about 24°. However, this assumed distribution would underestimate the actual value by a few degrees.

7 Summary and further discussion

7.1 Movements, foundation loads, wall friction, and stresses in the reinforcements
The backfill within about half a metre or so of the facing units was, initially at least, not as dense as within the main reinforced block. Subsequent placement and compaction of overlying backfill led to the settlement of this fill, and generated bending in the reinforcements close to the back of the units and downward acting frictional forces on the units. However, the stress regime in the upper metre or so of soil differed from that in the underlying soil.

The imposed settlement led to differential movement between the reinforcements, backfill and facing units. The direction and magnitude of the movements varied in three dimensions, i.e. with distance from the mid-point of the settlement trough. The reinforced block rotated outwards about the foundation, but the outward rotation of the wall face was not uniform over its height. Differential movement between the facing units and adjacent fill generated bending in the reinforcements and variations in the frictional force acting on the back of the wall. The maximum axial and bending stresses generated in the reinforcements were well below the elastic limit of the reinforcements.

The re-levelling operation restored, more or less, the original position of the structure, but the cycle of movement generated some permanent outward displacement of the facing. The distribution of the foundation load was, as expected, substantially changed by the re-levelling operation. This led to the development of high bending stresses in the reinforcements close to their connection with the facing units. Thus the structure did not behave as an ‘elastic recoverable material’: again this is not surprising. It is possible that cycles of movement might lead to the rupture of the reinforcements, but the scale of the imposed movements in this test is unlikely to be met in practice.

The stability of a particularly high reinforced soil wall could be compromised by substantial differential movement between the facing units and adjacent backfill. Such movements might be generated by internal compression of the reinforcements and also by settlement of the foundation. Also, the behaviour of a structure could be affected substantially by three-dimensional effects. (See Lee et al., 1994).

7.2 Lateral earth pressures
Table 3 provides a summary of the K values determined at various times through the test. These were derived from the measured tensions in the reinforcements, the face areas supported by the reinforcements and the appropriate nominal overburden pressure. The K values can be compared to those provided in Table 1 for various situations.

Compaction operations generated horizontal pressures typically of about 5 kN/m² but exceptionally up to 8 kN/m². Thus at low depths of cover the K values were relatively high, but at greater depths the values conformed to at-rest pressures for a ϕ value in good agreement with the peak value derived from the results of direct shear box tests.

The horizontal pressures acting on the back of the lower
central column of facing units reduced slightly with the first increment of settlement, but they increased on the other units. Further increments of settlement led to an increase in the horizontal pressures acting on the back of the facing units. At the end of the settlement stage, the total pressure was about a third higher than it was at the end of construction.

The horizontal pressures increased further through the re-levelling of the wall. Following re-levelling, the pressure acting on the central columns of units was about 50 per cent higher than at the end of the construction stage. The stress regime within the backfill cannot be determined for the settlement and re-levelling operations, but it was clearly influenced by three-dimensional effects. Perhaps for this reason, the derived K values for the sequence cannot be explained in terms of the generation of at-rest or active earth pressure conditions. An attraction of such a comparison is that it might allow simple rules to be given regarding the design of reinforced soil structures likely to be subject to large differential settlement. For example, that the maximum lateral force is a multiple of the estimated at-rest pressure. This might, however, be unrealistic given that the stresses generated as a result of ground movements are likely to be a function of the dimensions of the structure, the size and interaction of the facing units, the pattern of movement, the properties and spacing of the reinforcements, as well as the properties of the backfill.

7.3 Coefficient of interface friction

During construction, tension was not developed over the full length of the reinforcements. The mean mobilised angle of friction developed on the strips was about 24°, whilst the maximum angle was about 36°, i.e. close to the maximum recorded in the direct shear box tests. In the three months following the end of construction, the ambient air temperature reduced by about 5°C; this seems to have induced an increase in tension in most of the reinforcements. The change varied with the position of the strain gauges. The results of a long series of tests showed that the output of the strain gauges was temperature sensitive, but it is also possible that the tension in a reinforcement was affected by the restraint offered to it by the surrounding backfill. However, neither of these mechanisms can satisfactorily explain the very high apparent angles of interface friction developed along the strips.

Whatever the mechanism, it seemed to be nullified by the re-levelling operation. At the end of the operation, the data indicate that, for all levels of reinforcements, the mobilised angle of interface friction was about 24°, i.e. much the same as developed during construction. However, during construction, tension was not developed along the full length of the strips, but this was the case at the end of the re-levelling stage. It is possible that the K value generated by differential movement is governed by the pull out resistance of the reinforcements. In which case it might be appropriate to design a structure such that the maximum pull out resistance is lower than the rupture strength of the reinforcements. This might be difficult to achieve in practice – not least because pull out resistance can be substantially affected by the effects of constrained dilation, which are difficult to estimate accurately.

7.4 Consequences for further research

7.4.1 Current design requirements

A good deal of information exists in the literature on the effect of differential settlement on the performance of buildings, but with notable exceptions – for example Burland and Wroth (1975) – relatively little is available on its effects on retaining walls.

Current design documents for earth retaining structures require serviceability checks to be undertaken as part of the design process. However, most do not provide explicit guidance on the serviceability limits or on the tolerance of various types of structures to settlement. According to clause 3.1.4 of BS 8002: 1994 – the Code of Practice for earth retaining structures:

‘… for most earth retaining structures the serviceability limit state of displacement will be the governing criterion for a satisfactory equilibrium and not the ultimate limit state of overall stability. However, although it is generally impossible or impractical to calculate displacements directly, serviceability can be sufficiently assured by limiting the proportion of available strength actually mobilized in service …’

This admission leads to serviceability being assured (as best can be) through the use of a ‘sufficiently large’ safety factor in a bearing capacity check. (Settlement does not even appear in the Index of BS 8002: 1994.) This seems to blunt the use of a limit state partial factor approach to design.

According to clause 5.5 of BS 8006: 1995 – the Code of Practice for strengthened/reinforced soils and other fills:

‘The concept of serviceability depends very much on the end use of the structure. Normally serviceability limits for reinforced soil are prescribed in terms of acceptable deformations. Deformations of reinforced soil structures are influenced as much by the construction process as by the design.’

According to the Code, foundation settlements are to be calculated through conventional soil mechanics approaches. Table 4, reproduced from the Code, provides a preliminary guide to the maximum differential settlement that can be tolerated along the line of a reinforced soil structure. It should be noted that the ‘normal safe limit … for discrete concrete panel facings’ is 1 in 100, whereas the settlement profile imposed during the test, and which caused no significant damage to the structure, was 1 in 38 along the line of the wall. Although the guideline values seem overly conservative, it should be appreciated that settlement led to a substantial redistribution of tension forces between the reinforcements in the test wall. A significant redistribution of forces occurred at the end of the first settlement stage, which represents a differential movement of only about 1 in 200. Furthermore, the data from this experiment indicate that the common assumption of active earth pressures being generated behind the facing of a deflecting wall may not be borne out in practice.
The performance of reinforced soil walls has been investigated through numerical analysis, for example using the Finite Element (FE) method as described by Harris (1992) – papers and reports on the subject abound in the literature. Numerical analysis provides a convenient means of investigating the effect on performance of variation in the governing parameters: it is also far less costly than undertaking a series of physical tests. However, there are a number of problems with such analyses – not least that a plane strain formulation is commonly used, for practicality and economy, for what is evidently a three-dimensional structure. Furthermore, it is difficult to confirm the accuracy of the results of such studies and also their applicability to current practice, but this does not seem to perturb many of those undertaking such analyses. Some of the difficulties and limitations of numerical analyses have been discussed by Harris (1992). (A brief summary of the extensive series of analyses undertaken for that study has been produced by Harris et al., 1992.)

Modelling the performance of the test wall through construction, and the settlement and re-levelling stages would be a particularly severe test of the capabilities of any method of numerical analysis. Nonetheless, an analysis that reproduced the essential features of the performance of the test wall could be used with some confidence to investigate the performance of reinforced soil walls to other patterns of ground movements. Such an analysis might then be used to develop semi-empirical rules for the design of reinforced soil walls subject to differential movement.

### 8 Conclusions

A 4.4 m high, 12.6 m wide reinforced soil wall was constructed and subjected to controlled differential sagging movements; it was subsequently re-levelled. The maximum settlement of the facing units was about 117 mm representing a maximum differential movement of about 1 in 38 along the line of the wall and about 1 in 42 perpendicular to its face.

Apart from the apparent temperature sensitivity of the reinforcements, the data from the test are reasonably consistent and provide an understanding of the behaviour of the wall. Further work is required to establish a model to properly explain the apparent temperature sensitivity of the reinforcements.

The response of a reinforced soil structure to differential movement is a function of a number of interacting factors and so the conclusions drawn from this test may not be widely applicable. Nonetheless, the results of the test indicate that current methods of design might underestimate the tensions developed in the reinforcements and thereby over-estimate the stability of a reinforced soil structure subjected to large differential movement.

The overall performance of the test wall was good, the imposed settlement did not lead to gross distortion of the face of the wall nor did it compromise its stability. Provided that design took account of the increase in

---

### Table 4 Guide to effects of settlement (taken from BS 8006: 1995)

<table>
<thead>
<tr>
<th>Maximum differential settlement</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 in 1000</td>
<td>Not normally significant.</td>
</tr>
<tr>
<td>1 in 200</td>
<td>Full height panels may be affected by joints closing or opening.</td>
</tr>
<tr>
<td>1 in 100</td>
<td>Normal safe limit, without special measures, for discrete concrete panel facings.</td>
</tr>
<tr>
<td>1 in 50</td>
<td>Normal safe limit for semi-elliptical steel face elements. Discrete concrete panels may suffer closed joints if special measures not included.</td>
</tr>
<tr>
<td>1 in &lt;50</td>
<td>Soft facings may suffer distortion affecting their retaining ability.</td>
</tr>
</tbody>
</table>

*There is no intended firm limit between categories. This is a preliminary guide only.*
tension resulting from differential movement, the reinforced soil system might be suitable for use in areas of mining subsidence, but it would be necessary to install some wide seals to prevent the loss of backfill between the opening joints of the facing units.

9 Acknowledgements

A number of researchers have been involved with the experiment described herein. The TRL team included Mr D Barratt and Mr J Hanley; the authors would like to thank them for their contribution and also Prof. C Melbourne (formerly of BIHE, but now Salford University) and Prof. R W Sarsby (formerly of BIHE, but now Wolverhampton University).

10 References


British Standards Institution: London.


Volume 1: Specification for Highway Works (MCHW 1).


Appendix A: Temperature sensitivity of strain gauges

A1 Introduction
Details of the wiring circuit for the gauges installed on the strips are shown in Figure A1. Strain gauges were bonded to either side of a strip to enable both axial tensile and bending stresses to be calculated.

For the same change in applied load a half-bridge circuit will, due to Poisson’s ratio effect, display a higher change in resistance than a single gauge. Furthermore, often a passive gauge is introduced to balance the effect of temperature change on the active gauge.

Following the re-levelling of the structure, a parapet, with supporting slab, was constructed on top of the test wall. Tests were undertaken on the arrangement to assess the appropriateness of current design methods for such slabs and also the effect of a horizontal load on the stability of the test wall. (The results of the test on the base slab have been reported by Brady et al., 1996). Strain gauged strips were recovered from the structure at the completion of the test sequence at the end of 1995. Despite careful handling, some of the strips were distorted during extraction and the connection wires to some gauges were damaged; however several were recovered intact.

A total of eight strain-gauged sections were cut from the recovered strips. These were subjected to a number of cycles of tension in a calibrated loading rig to determine the sensitivity of the gauges. The magnitude of the forces was within the range of tensions measured in the experiment. The ambient temperature was not controlled during these tests: it varied from about 12 to 18°C. The sensitivities of the gauges derived in this series of tests did not seem to vary much from the values assumed at the outset of the test. In some cases the output of the gauges under zero load seemed to have changed, but this ‘zero shift’ could have been associated with bending of the strips through their recovery.

The output from the gauges in the above experiment was measured on a digital voltmeter and not the data logging system used at the Bolton Institute of Higher Education (BIHE). A series of tests was undertaken to confirm that the data provided by the logging system were insensitive to temperature change.

A2 In air sensitivity
A four metre length of a recovered strip was hung from the rafters of the ‘Q’ Wing test bay at TRL. Readings were taken from the gauges to determine their zero output at various temperatures. A concrete block (giving a load of about 5kN) was then attached to the bottom end of the strip and readings taken at various temperatures. This was repeated with two concrete blocks attached to the strip (giving a load of about 10kN). The results from this test indicated that the output of the gauges was affected by temperature, but the effect could not be quantified reliably because (a) the temperature range was small and (b) forced ventilation/heating was used in the test and so the temperature of the various gauge points might have varied at any one time along the strip.

Following this, additional gauges and thermocouples were attached to the strip. Details of the gauges and their configuration are given on Table A1. The strain-gauged strip was placed, horizontally in a flat bed test rig in the ‘Q’ Wing test bay. One end was fixed whilst the other was connected to a movable beam. The beam was connected to two jactuators whose position was controlled through a stepper motor. The force on the beam was measured by a load cell placed in line with each jactuator. The sensitivities of the gauges to changes in load and to temperature were established by applying a load of up to about 30 kN at different temperatures. Because forced cooling/heating was not used, these tests took several months to complete.

The strip was then re-hung from the rafters of the test bay, and readings taken at various temperatures whilst the strap supported one or two concrete blocks. To cover a wide range of temperatures, without resorting to forced heating or cooling, readings were taken over a period of a year or so: the test was not completed until 1998. Readings were only taken when consistently equal temperatures were measured along and adjacent to the gauge points. This generally only occurred when the test bay was not in use, for example at weekends when still air conditions were achieved.

The results of the above experiments are provided in Table A2. Note that because the strip was not perfectly straight it is necessary to consider the response of pairs of gauges installed on opposite sides of the strip. The data

![Figure A1](image_url) Arrangement and wiring of strain gauges on the steel strips
It could be expected, therefore, that the sensitivity introduced by mis-matched gauges would generate a random, rather than a systematic, variation in measured load with change in temperature, i.e. the changes in the output of the gauges might reinforce or partly cancel. As can be seen from Table A2, the half-bridge circuit (1/2) showed an increase in tension with a decrease in temperature: as seems to be the case with the strain gauges in the test wall. Both gauges (1) and (2) showed a decrease in tension with increasing temperature. However, the single gauges (9) and (10) showed a reverse sensitivity; tension decreasing with decreasing temperature.

Whilst the response of the set of gauges in the test wall cannot be bounded particularly well, the higher changes in mean axial stress should coincide with the lower changes in bending stress, and vice versa. This was the case for the test wall. As discussed in Section 4.3.7 of the main text, the changes in bending stress recorded, over the period where the temperature varied by about 5°C, by the gauges in the main body of the fill were equally distributed between hogging and sagging.

For the range 0-10 kN, the temperature sensitivities of gauges (1) and (2) were about 0.18 and 0.08 kN/°C respectively: giving a mean value of 0.13 kN/°C. It is not known how representative the half-bridge circuit (1/2) is of the full set of gauges installed in the test wall. But if the gauges were reasonably representative of the array in the test wall, apparent changes in tension of 0.6 kN and bending stress of up to about 2 N/mm² might be anticipated for a change of 5°C. As shown by Figure 18, a value of 0.6 kN is close to the modal value of 0.8 kN determined for the whole array of gauges: this agreement is fortuitous. A change in bending stress of up to 2N/mm² was recorded by about 50 per cent of the gauge points following the change in temperature of about 5°C.

It might be postulated that the gauges in the test wall had a much wider temperature sensitivity than gauges (1) and (2), and that this could account for the wider variation in tension and bending stress. However, as shown in Figure 18, the change in temperature seems to have induced a systematic rather than a random change in tension. Following the foregoing it would seem that, on top of the random variation introduced by the temperature sensitivity of the strain gauges, there was a systematic increase in tension of about 2 kN.

### Table A2 Temperature sensitivity of various types and arrangements of strain gauges

<table>
<thead>
<tr>
<th>Gauges (see Table A1)</th>
<th>Temperature sensitivity* kN tension/°C</th>
<th>Load sensitivity mV/kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 - 10 kN range (from dead load test)</td>
<td>0 - 30 kN range (from test rig)</td>
</tr>
<tr>
<td></td>
<td>0 - 10 kN range (mean value)</td>
<td>0 - 10 kN range (mean value)</td>
</tr>
<tr>
<td>1/2</td>
<td>0.134 0.116 0.127</td>
<td>0.113</td>
</tr>
<tr>
<td>3/4</td>
<td>0.144 0.131 0.136</td>
<td>0.086</td>
</tr>
<tr>
<td>5/6**</td>
<td>0.675 0.369 0.566</td>
<td>0.067</td>
</tr>
<tr>
<td>7/8</td>
<td>-0.004 -0.004 -0.004</td>
<td>0.221</td>
</tr>
<tr>
<td>9/10</td>
<td>-0.156 -0.140 -0.146</td>
<td>0.089</td>
</tr>
<tr>
<td>11/12</td>
<td>0.051 0.048 0.049</td>
<td>0.116</td>
</tr>
<tr>
<td>13/14</td>
<td>0.018 0.005 0.012</td>
<td>0.108</td>
</tr>
<tr>
<td>15/16</td>
<td>-0.004 -0.004 0.004</td>
<td>0.117</td>
</tr>
</tbody>
</table>

* +ve value = increase in tension with decreasing temperature.
** Erratic data.

from the dead load test for gauges 1 and 2 are provided in Figures A2(a) and (b) respectively. These data indicate that the output of the gauges attached to the buried strips was temperature sensitive. The data for gauge pair (1/2) show that a 5°C decrease in temperature could be interpreted as an increase in load of about 0.6 kN. This value is significantly lower than displayed by most of the gauges installed on the strips within the test wall, but is higher than the values determined for some of the gauges installed on the uppermost layer.

If the gauges making up a half-bridge circuit were perfectly matched, the arrangement should not be sensitive to changes in temperature. It could be expected, therefore,
The maximum frictional resistance that can be generated on a strip is a function of its dimensions, the vertical stress ($\gamma H$) acting on the surface of the strip, the angle of interface friction, and the boundary conditions. The frictional resistance that could be developed along a strip ($F_L$) fixed at one end is (ignoring the thickness of the strip),

$$F_L = 2 \gamma L H \tan \phi$$

The frictional resistance ($F_L$) generated transverse to the strips (assuming fixity along one edge of a strip and ignoring end effects) is given by the same equation. (Because temperature is reducing, end-bearing resistance is irrelevant.)

The values of $F_L$ and $F_T$ for the lowest row of strips are about 25 kN. For these strips, the calculated longitudinal ‘restraint force’ (3.6 kN) due to a 5°C reduction in temperature is much less than $F_L$ but the calculated transverse frictional restraining force (360 kN) is much higher than $F_T$. Thus these strips might be prevented from shrinking longitudinally but not laterally. (Note that different boundary conditions have been assumed for the above mechanisms.)

The values of $F_L$ and $F_T$ for the uppermost row of strips are about 2.3 kN. The calculated restraint forces are higher than these values: thus these strips might be less restrained. The generation of constrained dilation of the soil around

**Figure A2(a)** Results of sensitivity checks on strain gauges (gauge 1)

**Figure A2(b)** Results of sensitivity checks on strain gauges (gauge 2)
the strip might support the assumption of a higher 
mobilised value of \( \tan \phi_i \). However, this does not 
undermine the proposition that frictional forces developed 
on the surface of a strip might act as a restraint against 
movement induced by temperature change: this restraint 
might then affect the stresses within a strip.

Simplistically, and in the extreme, a 5°C reduction in 
temperature might generate zero longitudinal strain (\( e_L \)) 
and a transverse compressive strain (\( e_T \)) of \( 60 \times 10^{-6} \). The 
latter will generate a change in the resistance of the bridge 
circuit which is numerically equivalent to a change in \( e_L \) of 
\( 60 \times 10^{-6} \). \( \nu \) where \( \nu \) is Poisson’s ratio. In effect a 
compressive strain generated in the transverse direction 
due to a reduction in temperature) will generate an 
increase in tension in the longitudinal direction. The data 
from the strain gauges indicate a value of 0.23 for \( \nu \), thus a 
5°C reduction would give an apparent increase in tension 
of about 2.8 kN. This is only a little above the largest 
increase (2.6 kN) measured between the end of the 
construction stage and start of the settlement stage, when 
the temperature reduced by about 5°C.

The longitudinal frictional force that could be developed 
at a point along a strip varies with the depth of cover and 
distance along the strip, and, therefore, so would the 
restraint available to counter a change in temperature. It was 
noted that the increases in tension recorded over the period 
between the construction and settlement stages increased, 
admittedly not uniformly, with increasing depth of cover. 
The frictional resistance available would also vary along a 
particular strip: it would be a function of the distribution 
developed in response to the earth pressures acting on the 
back of the facing unit, and the maximum frictional restraint 
available. Where the frictional resistance has not been 
exceeded along a strip, the pattern of the change in tension 
due to a reduction in temperature should be triangular. The 
pattern of change recorded on some of the central columns 
of strips is shown in Figure A3. Although the data are 
 somewhat scattered, they provide some support to the 
mechanism postulated above. Nonetheless, the above can 
only be regarded as an hypothesis.

A4 Interpretation of measured change in tension 
The measured changes might be a combination of (a) the 
temperature sensitivity of the gauges (b) the differing 
frictional restraint offered across and along a strip and also 
(c) the effect of temperature on the properties of the 
backfill, concrete facing units etc. As no changes in the 
position of the facing units were detected between the 
experimental stages, the latter effect would not seem to be 
important. (Other hitherto unexplored mechanisms might

![Graph](image-url)
also be partly responsible for the measured changes.

There are insufficient data to separate out, with confidence, the effects of (a) and (b) for this experiment. It should be appreciated that (a) is merely a characteristic of the instrumentation for which a 'correction' could be applied, but (b) is a function of the interaction of the reinforcement and backfill. A 'correction' for (b) might however be applied to disentangle the effects of temperature change and, in this case, differential settlement.

Whilst the results of laboratory tests showed that the output from a few pairs of gauges attached to the strips was found to vary with temperature, the variability in the temperature sensitivity of the gauges used in the test wall is unknown. The value quoted in Table A1 might not be representative of the stock of gauges used in the experiment. Furthermore, although the temperature should not have varied much within the experimental structure, there were insufficient thermometers to confirm this. The application of a standard correction for the temperature sensitivity of the gauges is problematic. For example, a correction of say -0.6 kN (see Table A1) to account for a reduction in temperature of 5°C (between the construction and settlement stages) would suggest that some of the uppermost row of straps were in compression at some time: this seems unlikely.

The data given in Figure A3 are scattered somewhat but some of the plots suggest a near-triangular distribution of the change in tension. The position of the maximum change might reflect the longitudinal restraint along a strip – as discussed above this is likely to have a greater effect on the lower row of strips.

A5 Implications
The temperature sensitivity of the gauges and/or the steel reinforcements is of little importance in engineering terms. However, as shown by this report, both can have a significant effect on the interpretation of the data obtained from an experiment undertaken on a reinforced soil structure. This would be particularly noticeable on an over-reinforced structure, such as the test wall, where the possible change in tension due to temperature variation was a significant fraction of the maximum measured tension. Depending on the properties of the reinforcements and the configuration and characteristics of the strain gauges, the effect of temperature variations might be important for small-scale model tests.

A6 Reference
Abstract

This report describes the construction and instrumentation of a 4.4 m high reinforced soil wall. It then describes the response of the wall to a differential settlement (sagging) of about 1 in 38 along the face of the wall and 1 in 42 perpendicular to the face of the wall, and goes on to describe the behaviour of the wall during its re-levelling.

The wall was built from hexagonal shaped reinforced concrete facing units, galvanised mild steel reinforcing strips and a crushed limestone aggregate. The performance of the wall during construction was largely as expected. At depths of cover greater than between about one and two metres, the mean tension developed in the reinforcements at points close to the back of the facing units was about equivalent to at-rest earth pressures. However compaction operations generated higher earth pressures at shallower depths of cover.

The performance of the wall during the settlement stage was good; the imposed settlement did not lead to gross distortion of the face of the wall nor did it generate excessive bending stresses at the connection between the facing units and strips. However, the tensions in the reinforcements increased significantly through the settlement and re-levelling stages. Current methods of design do not explicitly take account of increases in tension due to differential movement.

Related publications

<table>
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</tr>
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