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THE EFFECTIVENESS OF BERMS AND RAKED PROPS AS TEMPORARY SUPPORT TO RETAINING WALLS

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This report describes work commissioned by the Bridges Engineering Division of the Highways Agency under E554C/BG. Use of Soil Berms and Raked Props for Temporay Support of Embedded Retaining Walls.

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EXECUTIVE SUMMARY

Many deep excavations for road improvement schemes in urban areas are currently being constructed using bored pile and diaphragm retaining walls which are permanently propped at carriageway level. This technique is advantageous in reducing the depth of wall penetration required for overall stability and thus results in significant cost savings. However the reduced penetration means that during bulk excavation in front of the wall and prior to installation of the carriageway prop, some form of temporary support has to be employed to prevent excessive wall movement and provide safe working.

Traditionally this support has been supplied by steel temporary props installed horizontally and spanning between the opposing walls of an underpass. However there are many construction situations where this technique is either excessively costly or is impractical, e.g. if there is no opposing wall to react against or if the carriageway width is excessive.

Finite element modelling (3D and 2D) has been undertaken to investigate the effectiveness of both soil berms and raked props as temporary support during construction of embedded retaining walls permanently propped at carriageway level. In the former case construction involving a hinged permanent prop spanning an underpass has been assumed and in the latter the use of a stabilising base integral with the wall. The results have been compared with those calculated if temporary horizontal props were used or if the wall were allowed to cantilever. The analyses have been carried out assuming the wall was founded in stiff overconsolidated clay and may not be applicable if water bearing strata or weak layers are present.

The results indicate that short and long term lateral movements of the top of the wall when using progressive construction with 6m high berms were similar to those expected if using temporary horizontal props. A 6m high berm would therefore be equally effective in controlling wall movements. Short term movements with a 2m high berm were closer to the cantilever situation although longer term movements were similar to those with horizontal propping. The advisability of totally relying on the smaller berm without employing monitoring using the Observational Method must be questionable.

Temporary raked props were not as effective as horizontal props in controlling the short term movements of the top of a stabilising base retaining wall, although overall movements in the longer term were the same.

The report also discusses heave below formation level and the wall bending moments calculated immediately after construction and after 120 years in service.

THE EFFECTIVENESS OF BERMS AND RAKED PROPS AS TEMPORARY SUPPORT TO RETAINING WALLS

ABSTRACT

Finite element modelling has been undertaken to investigate the effectiveness of both soil berms and raked props as temporary support during construction of embedded retaining walls permanently propped at carriageway level. In the former case construction involving a hinged permanent prop spanning an underpass has been assumed and in the latter the use of a stabilising base integral with the wall. The results have been compared with those calculated if temporary horizontal props were used or if the wall were allowed to cantilever. The analyses have been carried out assuming the wall was founded in stiff over-consolidated clay. The effect of using different soil models was investigated.

1. INTRODUCTION

Many deep excavations for road improvement schemes in urban areas are currently being constructed using bored pile and diaphragm retaining walls which are permanently propped at carriageway level. This technique is advantageous in reducing the depth of wall penetration required for overall stability and thus results in significant cost savings. However the reduced penetration means that during bulk excavation in front of the wall and prior to installation of the carriageway prop, some form of temporary support has to be employed to prevent excessive wall movement and provide safe working.

Traditionally this support has been supplied by temporary steel props installed horizontally and spanning between the opposing walls of an underpass (Symons, 1992). However there are many construction situations where this technique is either excessively costly or is impractical, e.g. if there is no opposing wall to react against or if the carriageway is very wide. In these situations the use of soil berms or raked props as methods of temporary support may therefore be appropriate. As only limited documented evidence exists on the field performance of retaining structures supported in this manner (Burland et al, 1979; Powrie et al, 1992; Carder and Brookes, 1992), finite element modelling has been undertaken to investigate the effectiveness of these temporary support systems.

The modelling was carried out using wall designs typical of those used in current practice on road schemes. The use of soil berms was analysed assuming a hinged permanent prop slab at carriageway level, whilst the raked prop situation included a stabilising base (integral with the wall) at carriageway level. The results of the analyses in both cases were compared with those obtained from control analyses where either horizontal temporary props were used or the wall was unsupported and cantilevered towards the excavation. The effect of different soil models was investigated and throughout the study it was assumed that the embedded walls were founded in stiff over-consolidated clay.

2. BASIS OF ANALYSES

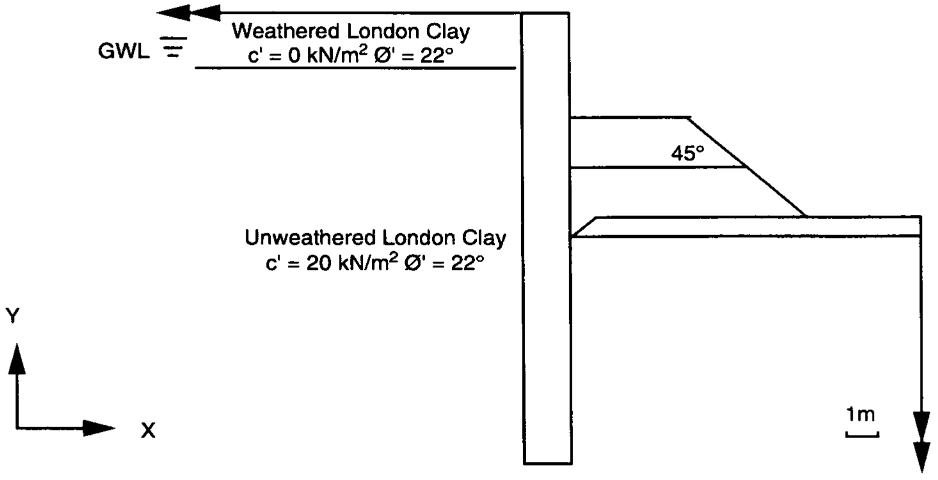
The finite element analyses were performed using the CRISP90 package (Britto & Gunn, 1987). The soil was generally modelled as an elastic perfectly plastic material with the yield surface being defined by the Mohr-Coulomb yield criterion, although a limited sensitivity study was also undertaken using other soil models. Longer term performance was simulated using a coupled consolidation analysis.

All analyses were conducted with the wall being 'wished in place' that is assuming the effects of bored pile or diaphragm wall construction are negligible on the insitu stresses within the soil. This assumption is appropriate in a design situation as it will be conservative in taking no account of the stress reduction which normally occurs during wall installation (Symons and Carder, 1992). In all cases, permanent support for the wall comprised a reinforced concrete prop at carriageway level.

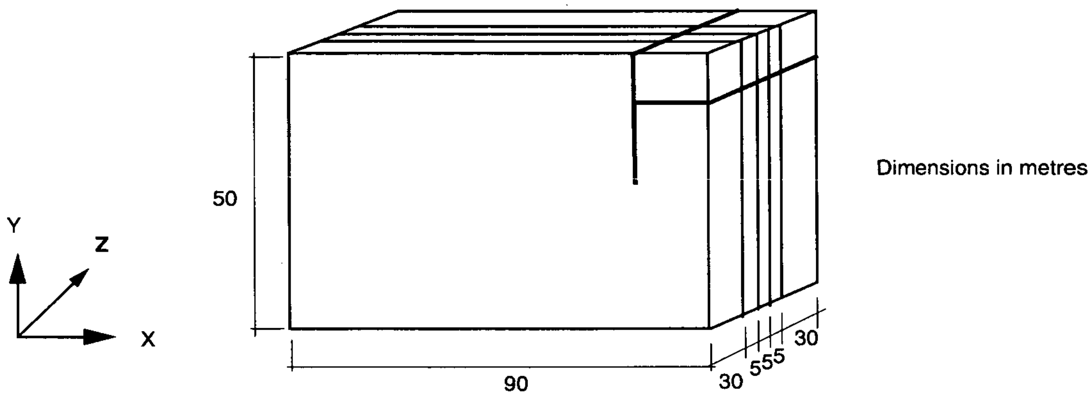
2.1 SOIL BERMS

A database of previous embedded retaining wall schemes was used to give typical values for the wall dimensions used in the finite element mesh and these are as shown in Fig 1a. Simulations were constructed for berm heights of 6m and 2m which were $^3/_4$ and $^1/_4$ of the retained height respectively. In all cases the base of the berm extended out a distance of 8m from the wall and the berm slope was 45°; this modelled underpass construction where a central haul road was excavated leaving berms to support the opposing walls. The analyses were carried out using an overall wall penetration of 18m which was typical of that used in practice. The permanent prop at carriageway level was assumed to be hinged at the wall and to be continuous and horizontal across the width of the underpass.

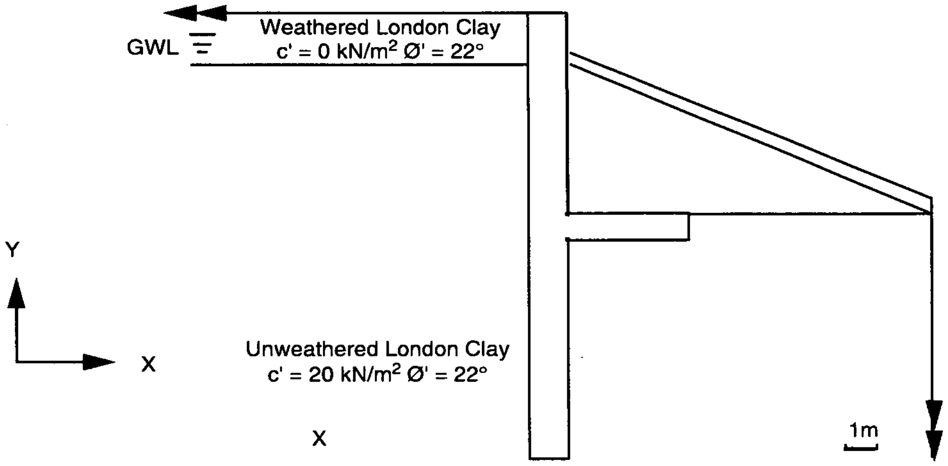
Based on this geometry, a simple three dimensional mesh (Fig 1b) was designed so that the construction process using berms for support could be modelled without an excessive computational run time. A two dimensional mesh was not considered adequate as the support, provided by either an adjacent berm or finished permanent prop slab, when excavating a bay could not be represented. The width of bays used in the construction process was assumed to be 5m.



(a) Geometry for soil berm analyses



(b) 3D representation used for soil berm analyses



(c) Geometry for temporary raked prop analyses

Fig.1 Problem geometry

2.2 TEMPORARY RAKED PROPS

Temporary raked props are frequently used in construction situations where either there is no opposing retaining wall for a horizontal prop to react against or the carriageway width is excessive. The latter makes temporary horizontal props costly and presents practical difficulties. The analysis of the raked prop case was carried out using an embedded wall with a stabilising base at carriageway and the typical geometry is shown in Fig 1c. Unlike the berm situation, the analysis could conveniently be carried out using a two dimensional plane strain analysis.

Where possible dimensions and properties conformed to those used in the analysis of soil berm behaviour so that comparisons could be drawn. An overall wall penetration of 15m was used for the analysis.

2.3 CONTROL ANALYSES

In order to assess the effectiveness of berms and raked props as a method of support, the results were compared with a 'best case' and a 'worst case' scenario. The best case scenario was with a similar construction sequence but support provided by temporary props horizontally spanning between the underpass walls. The worst case scenario allowed the wall to cantilever outwards without support during the bulk excavation. Both of these 'control' cases were modelled using a two dimensional mesh.

3. MATERIAL PROPERTIES

3.1 SOIL PARAMETERS

The soil modulus used in the main analyses was typical of that used in earlier analyses for over-consolidated London Clay (Cole and Burland, 1972; St. John, 1975; Carswell, Carder and Symons, 1991). The drained elastic modulus (E' in MN/m^2) was varied with depth (z in metres) according to the equation:

$$E' = 32 + 8.4z$$

Generally the shear modulus calculated from this E' and the assumed Poisson's ratio of 0.2 agreed well with the upper bound values from reload loops measured using the self-boring expansion pressuremeter (Watson and Carder, 1994). The effect of using a lower soil modulus and a different soil model were also investigated and are discussed in Section 6.

The finite element mesh was constructed using two different soil layers. The top 2m layer was assumed to be weathered London Clay with typical strength parameters of $c' = 0$ kN/m^2 and $\phi' = 22°$. The underlying clay layer had strength parameters of $c' = 20$ kN/m^2 and $\phi' = 22°$ which were typical of those expected from consolidated undrained triaxial tests on unweathered London Clay. For the consolidation analysis, horizontal and vertical permeabilities of the clay were assumed to be 5×10^{-10} m/sec and 1×10^{-10} m/sec respectively; the retaining wall was assumed to be impermeable throughout.

The bulk density of the soil was taken as 19.9 kN/m^3 and the water table was assumed to be 1m below ground level on the retained side of the wall and at the ground surface or underside of the permanent prop slab on the excavated side.

The initial value of the earth pressure coefficient K (the ratio of horizontal to vertical effective stress) is influenced by the depth below ground, the stress history in particular the maximum preconsolidation pressure, the Poisson's ratio and the angle of internal friction of the soil (ϕ'). For unweathered London Clay a K value of 2 was assumed to be reasonable for the analyses (Symons, 1992).

3.2 STRUCTURAL COMPONENTS

In the analyses, linear elastic elements were used to represent the three main structural components, i.e. the wall, the permanent prop slab at carriageway level and temporary props (where appropriate).

The wall comprised 1.5m diameter bored piles at 1.7m centres with a pile flexural stiffness of 7.6×10^6 kN/m^2. This was modelled using a 1.5m thick rectangular wall section with an equivalent stiffness E of 1.6×10^7 kN/m^2 per metre run of wall. These values were calculated using a concrete stiffness of 2.6×10^7 kN/m^2 and assuming that the concrete remained uncracked at the small strain levels involved.

In the analysis of berm behaviour, the permanent prop slab was 0.65m thick reinforced concrete and modelled as being installed horizontally. The connection between wall and slab was represented as a pin joint connection (Powrie and Li, 1991) to allow the wall to rotate about the prop and accommodate heave beneath the carriageway. A typical stiffness E of 3×10^7 kN/m^2 was drawn from previous analyses (Watson and Carder, 1994).

In the assessment of raked prop performance, the stabilising base was 1.0m thick reinforced concrete and extended out a distance of 4m from the wall. The wall and prop were considered as integral, i.e. a full moment connection. The temporary raked props were installed at approaching 30° to the horizontal as shown in Fig 1c and were modelled as rectangular elements (0.5m wide) having an equivalent stiffness of 2.1×10^6 kN/m^2 per metre run. This is equivalent to using steel props of nominal 500mm diameter and 20mm wall thickness at a spacing of 6m between prop centres: this is typical of current designs.

The temporary horizontal props used in the control analysis were 1m diameter and installed at 1m depth below the top of the wall. These were modelled using rectangular elements (1m wide) with an equivalent stiffness of 4.8×10^6 kN/m^2.

4. CONSTRUCTION SEQUENCE

4.1 SOIL BERMS

The following stages of construction were modelled in the analyses of the effectiveness of berms as a method of support.

a) Excavation of haul road along centre of underpass (30 days).

b) Excavation of berm in central bay 2 (30 days).

c) Installation of permanent prop in central bay 2 (1 day).

d) Excavation of adjacent berm in bay 1 (30 days).

e) Installation of permanent prop slab in bay 1 (1 day).

f) Excavation of berm in bay 3 (30 days).

g) Installation of permanent prop slab in bay 3 (1 day).

h) Excavation of remaining berms throughout underpass and installation of permanent prop slabs (30 days).

i) Consolidation over a period of 120 years.

4.2 TEMPORARY RAKED PROPS

A similar sequence of construction was adopted to model the construction sequence using temporary raked props.

a) Excavation to underside of raked prop (10 days).

b) Installation of raked prop (2 days).

c) Excavation to full depth (30 days).

d) Installation of permanent prop slab (30 days).

e) Removal of temporary prop (1 day).

f) Consolidation over a period of 120 years.

4.3 CONTROL ANALYSES

The analysis constructed to model the cantilever case was a simple excavation to full depth followed by the consolidation phase of 120 years. No support was given to the wall and no permanent prop slab was installed.

For the temporary horizontal prop case the process of construction was as commonly used on site: excavation to 2.0m depth, installation of temporary props, excavation to full depth, installation of permanent prop slab, temporary prop release and finally consolidation.

In both control cases the duration of the various construction stages matched those used in the main analyses as closely as possible.

5. DISCUSSION OF RESULTS

5.1 SOIL BERMS

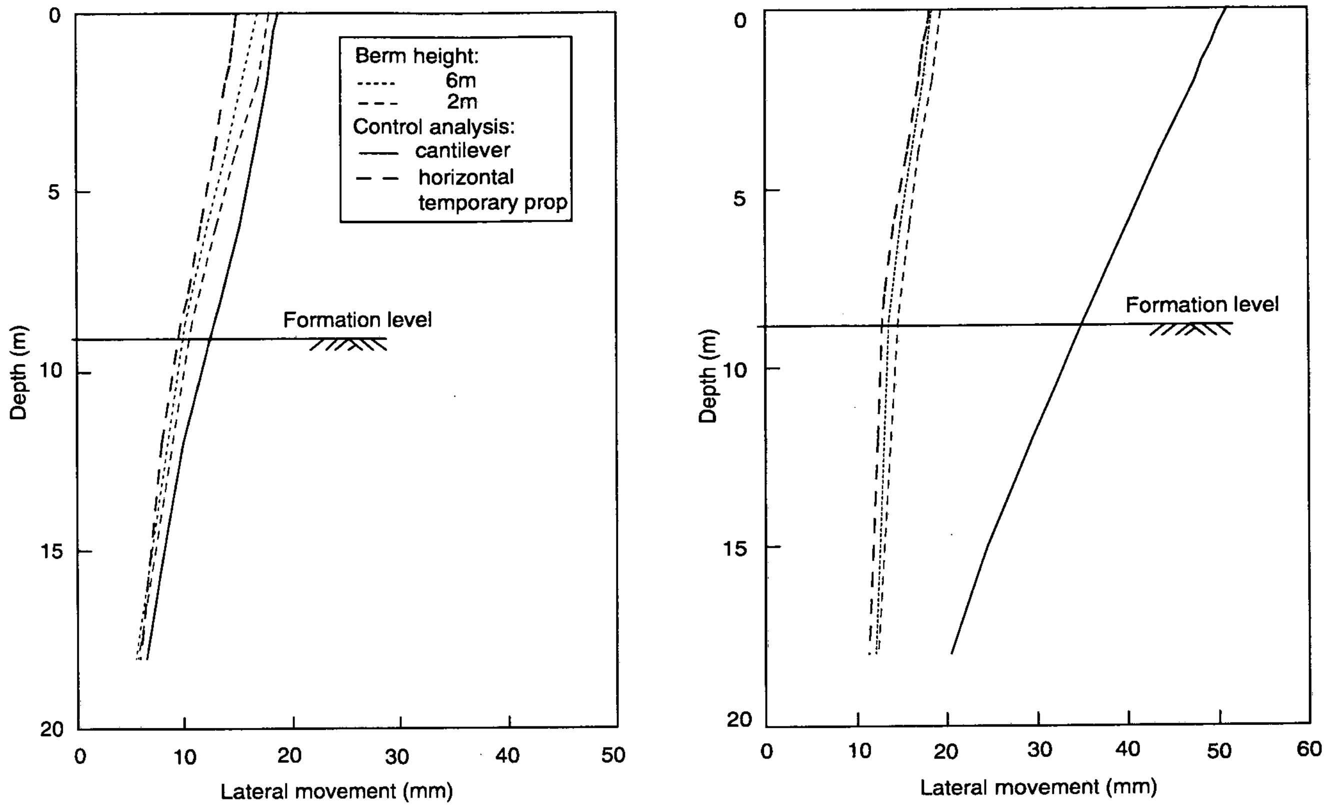
The lateral wall movements determined for the two berm heights of 2m and 6m are shown in Fig 2. The results are compared with the movements calculated from the control analyses, i.e. the temporary horizontal prop and the cantilever cases.

Fig 2a shows the movements calculated immediately after completion of construction, i.e. after installation of permanent prop slab and removal of temporary support (stage (h) for the berms), and after excavation to formation in the cantilever case. Lateral movements determined at the top of the wall were 16.8mm and 17.9mm when using 6m and 2m high berms respectively. These values fell within the lower and upper limits of 15mm and 19mm calculated for the temporary horizontal prop and the cantilever cases. At this stage only small differences were found between the various cases analysed as wall movement was largely a function of the undrained response of the ground. Larger differences are found if other soil models are used (see Section 6) and would also be anticipated if water bearing strata or a weak soil layer were present.

As would be expected, more movement was recorded with the smaller berm height. Potts et al (1992) reported that the efficiency of berms reduced rapidly when their height reduces below 2.5m. Although there seems little doubt that a 6m high berm would be effective in controlling wall movements, the advisability of relying totally on a 2m high berm without employing monitoring using the Observational Method (Peck, 1969) must be questionable.

The longer term development of lateral wall movement is shown in Fig 2b. In the control case using horizontal props the final movements at the top and toe of the wall were 18.2mm and 11.5mm respectively. The significant increase in toe movement was associated with the long term heave which occurred below formation level. Final movements when using a 6m high berm and constructing progressively in bays were very similar to the propped control case. Lateral movements using the smaller 2m high berm were up to 2mm higher. These results can be compared with the movements of the top of the wall of 51mm which were calculated for the unsupported cantilever situation.

The development of surface heave at formation level is shown in Fig 3. Generally immediate heave during construction (Fig 3a) was of similar magnitude whichever construction technique was employed with maximum val-



(a) Immediately after completion of construction

(b) After 120 years

Fig 2. Development of lateral wall movement when using soil berms

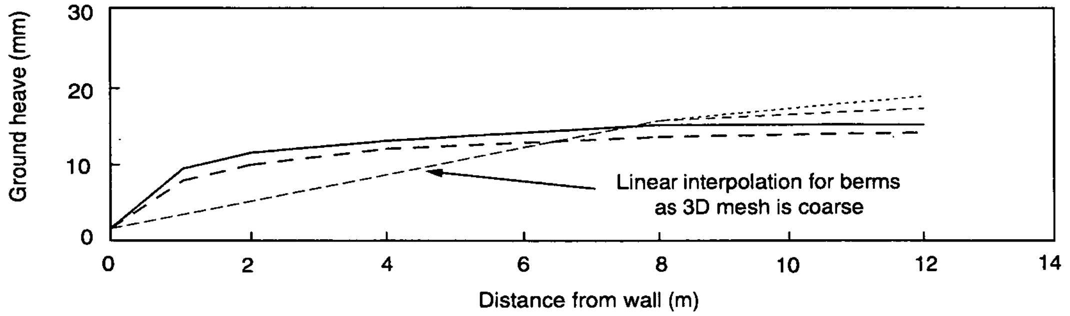
ues near the centre of the excavation ranging between 14mm and 19mm. In the longer term (Fig 3b), a maximum heave of 77mm was calculated for the horizontal prop case with larger heaves of about 94mm being recorded when using berms as support. A much larger heave of about 135mm was predicted for the cantilever control case where there was no permanent prop at carriageway level to restrain movements.

Wall bending moments developed on completion of construction and after a further 120 years are shown in Fig 4. On completion of construction the shape and magnitude of the bending moment profiles with depth are somewhat similar for the different construction procedures as short term behaviour is largely governed by the undrained soil response to bulk excavation. In the longer term the negative excess porewater pressures dissipate and the bending moment profiles show more variation according to the construction procedure (Fig 4b). In all cases clockwise overturning moments increase with depth over the 8m retained height of the wall. A peak moment of 1000kNm/m was calculated at permanent prop level when using temporary horizontal props. Lower values of 676kNm/m and 607kNm/m were found at the same level when using berms of 6m and 2m height respectively. A peak moment of 300kNm/m was calculated for the cantilever case at a lower depth of 11.6m when sufficient passive soil resistance had been mobilised for bending moment reversal to occur.

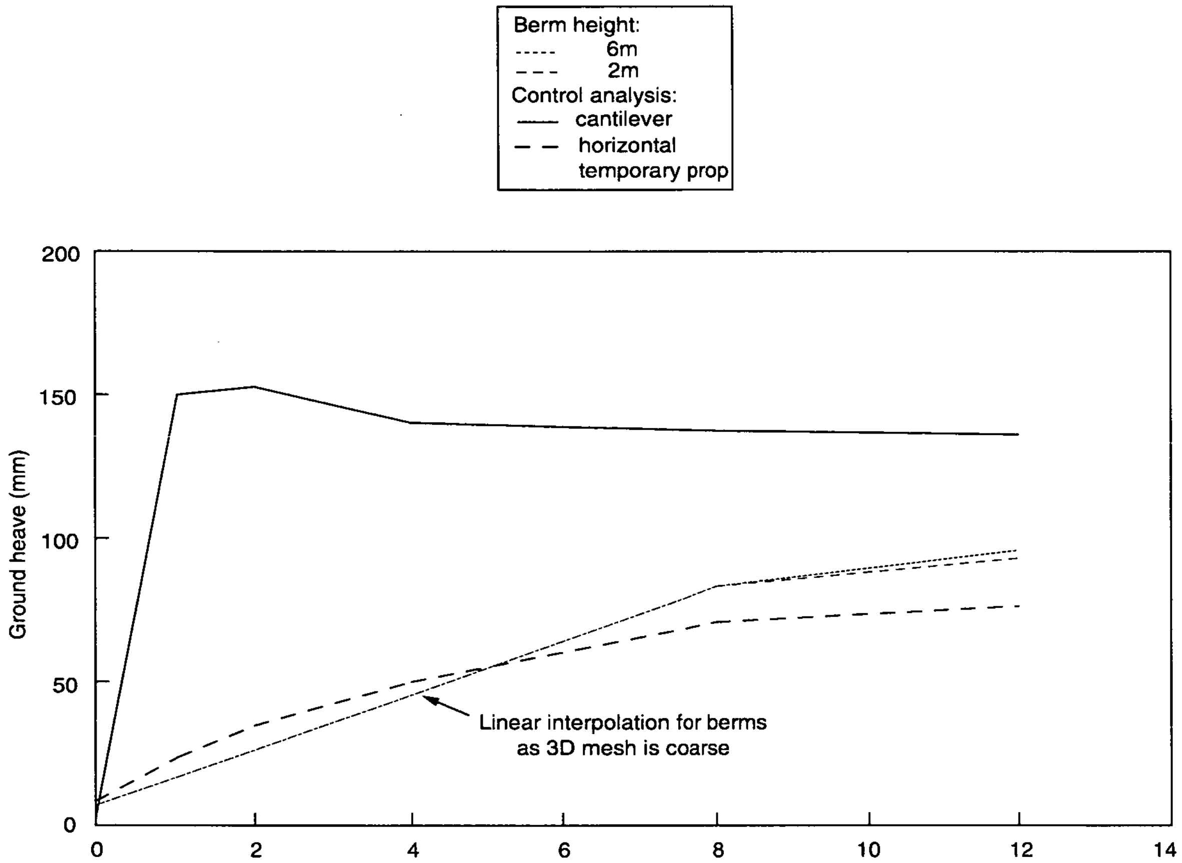
5.2 TEMPORARY RAKED PROPS

The lateral wall movements calculated when using temporary raked props as a method of support are compared with the control analyses in Fig 5. Top of wall movements determined immediately after completion of construction using temporary horizontal propping and for a cantilever were 15.4mm and 18.2mm respectively. The result for the raked props was 17.8mm and close to that for the cantilever wall. This indicated that the temporary raked props were not as effective in reducing short term movements as the horizontal props.

Fig 5b shows the wall movements developed after 120 years and, in the longer term, the raked props proved as effective as the horizontal props. In both cases movements were about 17mm and 19mm at the top and toe of the wall respectively. The significant outward movement of the toe of the wall was in response to long term heave caused by the unloading effect due to bulk excavation. Toe movement with the cantilever analysis was of similar magnitude although much larger movements at the top of the wall of about 40mm were determined. It must be noted that the control analysis on the cantilever wall was only carried out for purposes of comparison and the depth of wall penetration was insufficient for overall stability.



(a) Immediately after completion of construction

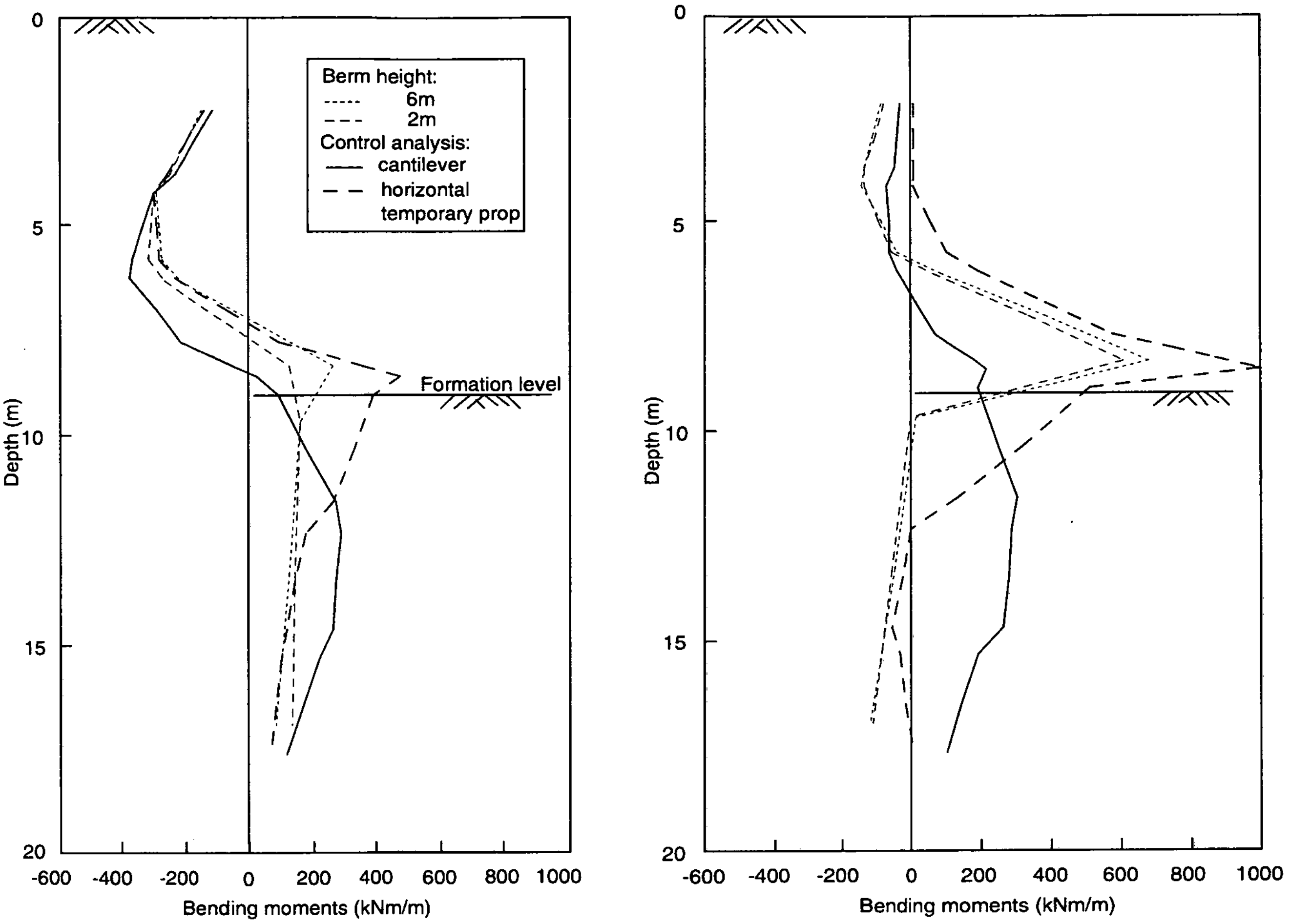


(b) After 120 years

Fig 3. Development of ground heave when using soil berms

Ground surface heaves at formation level both immediately after completion of construction and after a further 120 years are shown in Fig 6. With the exception of the cantilever analysis, heave close to the wall was less than elsewhere because the 4m wide stabilising base was constructed integrally with the wall, i.e. using a full moment connection. In all cases the maximum heave near the centre of the excavation was between 15mm and 17mm at the completion of construction (Fig 6a). After a further 120 years (Fig 6b), identical heaves of up to 94mm were predicted for stabilising base walls constructed with temporary support from either horizontal or raked props. The control analysis using a cantilever showed much larger heaves of about 120mm.

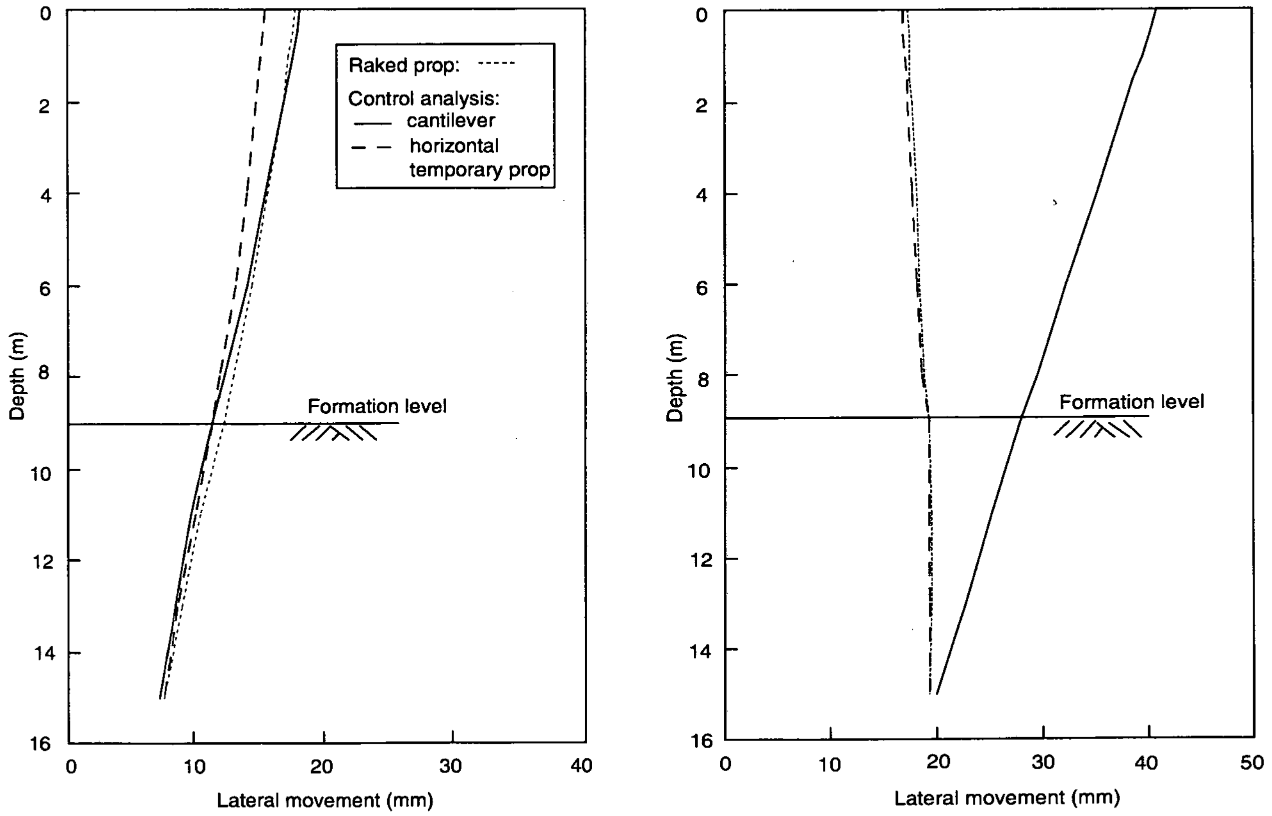
The general pattern of ground movements when using raked props as temporary support is summarised in the vector diagrams in Fig 7. During the first phase of excavation and completion of the stabilising base, the vectors were consistent with undrained soil behaviour and indicated mainly lateral movement towards the excavation with some ground heave below formation level starting to occur. The movement vectors for the long term analysis (Fig 7b) confirmed that heave below formation level was the main



(a) Immediately after completion of construction

(b) After 120 years

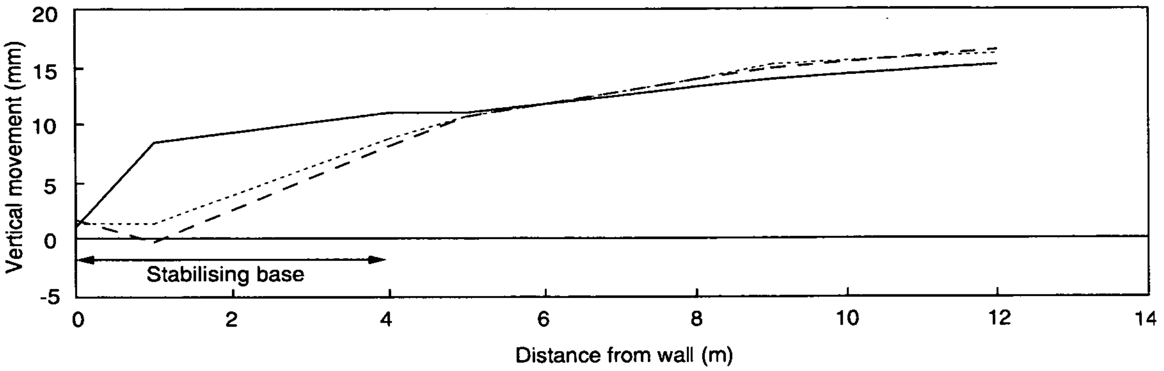
Fig 4. Development of bending moments when using soil berms



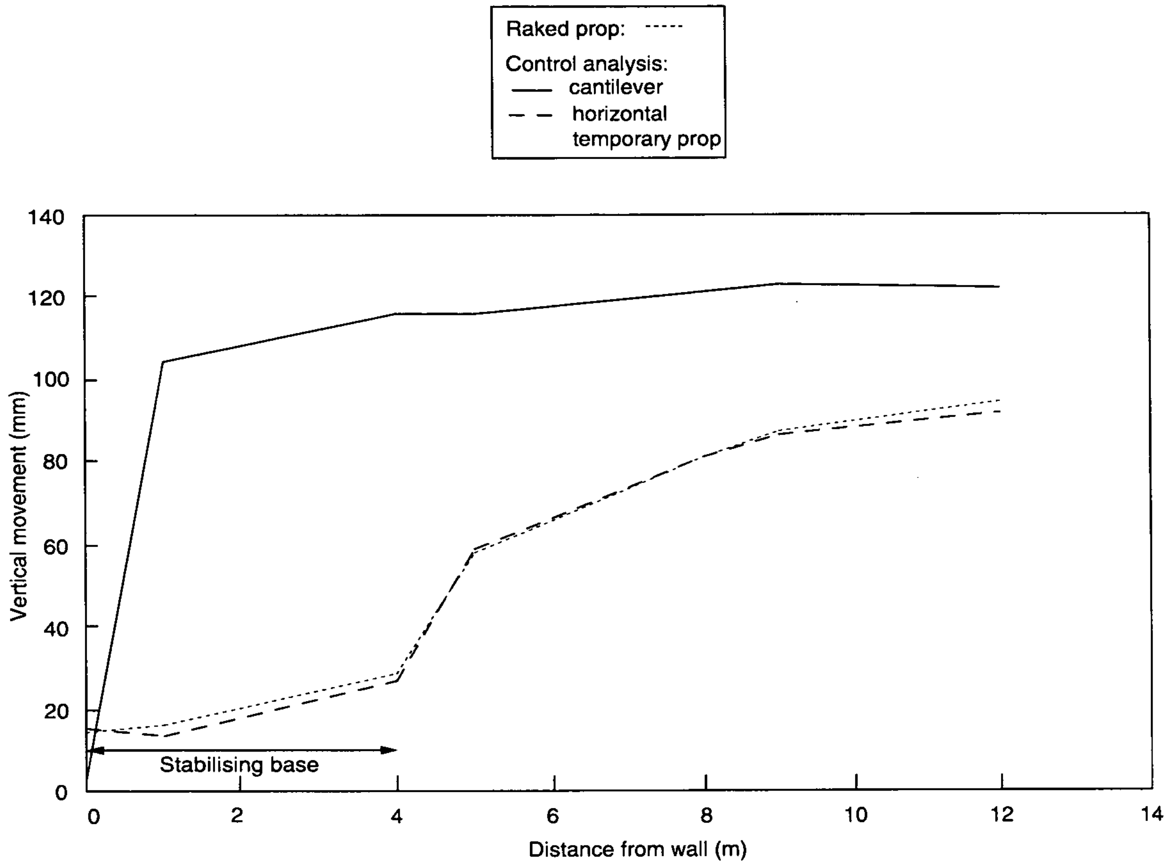
(a) Immediately after completion of construction

(b) After 120 years

Fig. 5 Development of lateral wall movement when using temporary raked props



(a) Immediately after completion of construction

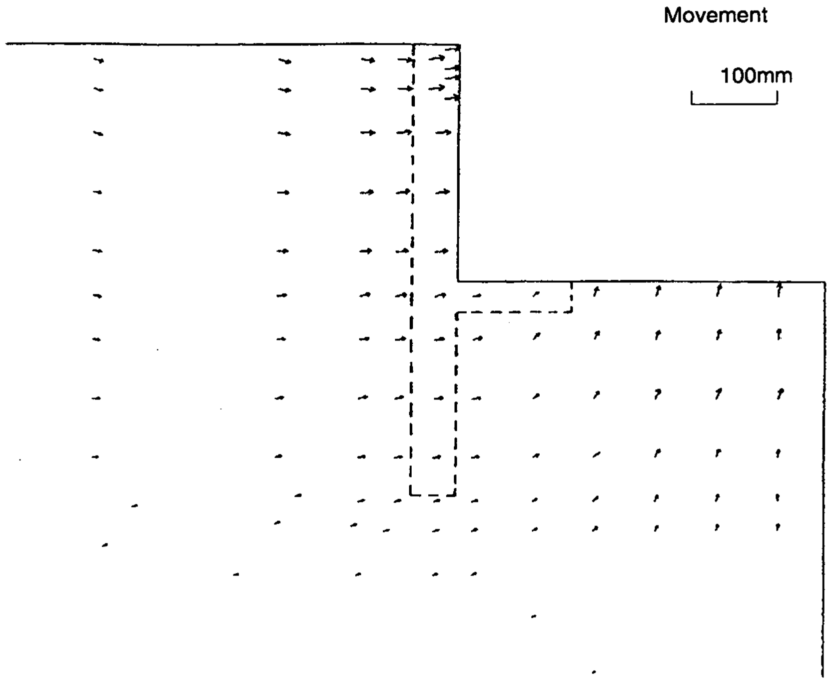


(b) After 120 years

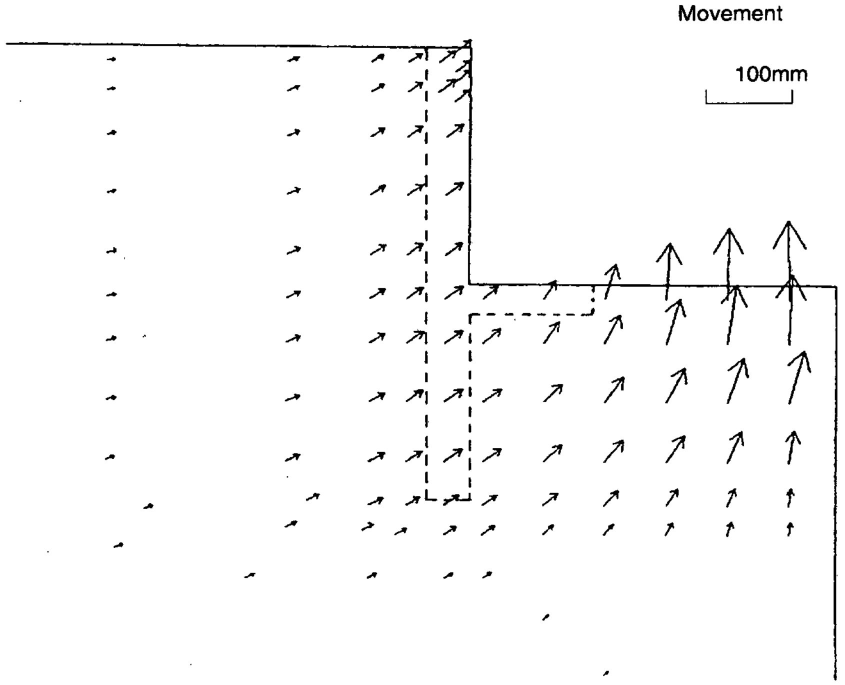
Fig. 6 Development of ground heave when using temporary raked props

mechanism as dissipation of excess porewater pressures slowly occurred in the impermeable clay.

Bending moment distributions with depth for the two control cases and the raked propped analysis are shown in Fig 8. Immediately after construction had ended and any temporary propping had been removed, bending moments over the uppermost 8m of the wall were similar in all three cases. However the influence of the stabilising base was already apparent in so far as a sharp bending moment reversal occurred around this level for both the raked and horizontal prop analyses. After 120 years the wall bending moments calculated when using both propping sequences were nearly identical with peak clockwise moments of about 460kNm/m being predicted at the top of stabilising base level (Fig 8b). Peak anticlockwise wall moments of about 700kNm/m were also determined near the underside of the stabilising base as a result of the moment connection to the wall: these results can be compared with those in Fig 4b where the permanent prop slab was hinged.

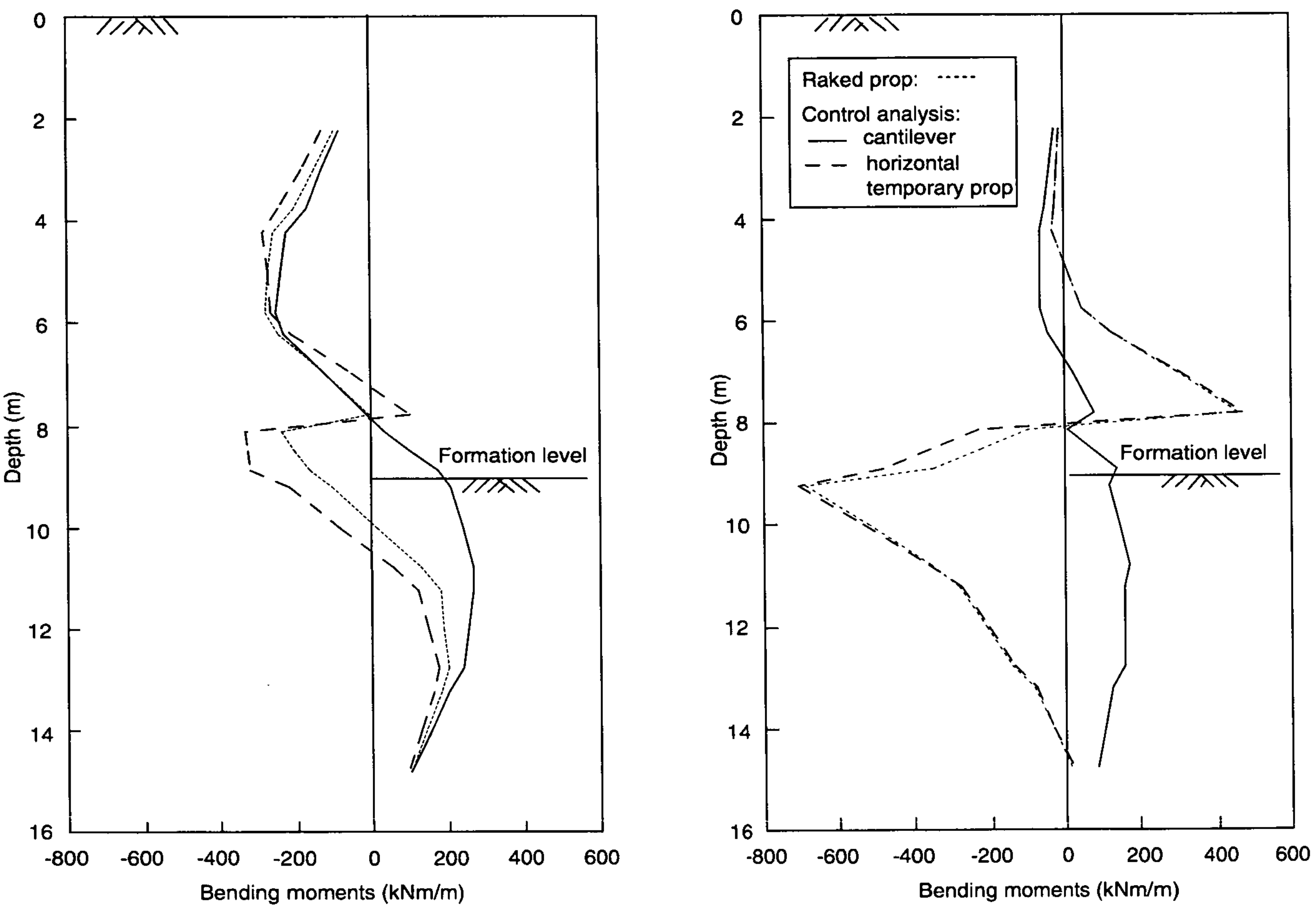


(a) Immediately after completion of construction



(b) After 120 years

Fig. 7 Development of movement when using temporary raked props



(a) Immediately after completion of construction

(b) After 120 years

Fig. 8 Development of bending moments when using temporary raked props

6. EFFECT OF USING OTHER SOIL MODELS

In addition to the above analyses (Case 1) which were carried out using a Mohr-Coulomb soil model and a linear variation of soil modulus with depth ($E' = 32 + 8.4z$, see Section 3.1), the sensitivity of the results to the following changes in soil model was investigated:-

Case 2: Using a reduced linear soil modulus of $E' = 12 + 6.24z$. This equation for horizontal modulus was suggested for London Clay by Burland and Kalra (1986).

Case 3: Using a model proposed by Schofield (1980) which incorporates the Cam-clay yield surface on the wet side of critical state and the Hvorslev surface and a no tension cut-off on the dry side. Soil parameters used in this model were those reported for London Clay by Chandler (1995).

6.1 SOIL BERMS

A comparison of the wall movements from the berm analyses predicted using the different soil models is given in Table 1. When using both the lower modulus equation and the Schofield model, the wall movements calculated for the berm and control analyses formed a similar pattern to those described in Section 5.1. The magnitudes of the movements were however greater as in both Case 2 and Case 3 the soil modulus is effectively lower than in Case 1. For example if the equivalent modulus is calculated arbitrarily at 6m depth, the ratios of $1/E'$ are approximately 1 : 1.7 : 2.7 for Cases 1, 2 and 3 respectively. This can be crudely compared with the short term movements at the top of the wall given in Table 1 which were in the ratio of 1 : 1.5 : 2.7 for walls supported during excavation. Corresponding ratios for lateral movement at the toe of the wall were slightly smaller (1 : 1.4 : 2.9). In the longer term, behaviour for the walls which are supported during excavation is characterised by a large outward movement of the toe and hardly any movement at the wall top. In the cantilever case, with no permanent prop at carriageway level, an increase in top of wall movement of between 32mm and 84mm was determined after 120 years.

TABLE 1

Wall movements (mm) from berm analyses using different soil models

Analysis	Wall movement	After completion of construction			After 120 years		
		Case 1	Case 2	Case 3	Case 1	Case 2	Case 3
Cantilever	Top	18.7	31.0	62.2	51.0	77.0	146.3
	Toe	6.4	9.2	20.3	20.5	28.8	77.4
2m high berm	Top	17.9	27.2	50.6	19.6	28.6	41.6
	Toe	5.5	7.8	16.4	12.3	16.5	39.9
6m high berm	Top	16.8	25.2	46.6	18.5	26.6	38.2
	Toe	5.4	7.6	16.1	12.1	16.2	39.7
Horizontal props	Top	14.9	22.2	38.8	18.2	25.1	26.2
	Toe	5.8	8.3	16.7	11.5	15.4	43.4

Note: Case 1 is Mohr-Coulomb, $E' = 32 + 8.4z$
Case 2 is Mohr-Coulomb, $E' = 12 + 6.24z$
Case 3 is Schofield model with no tension cut-off

The movements predicted from the finite element study using the three soil models can be compared with the database of field measurements established by Carder (1995). For walls propped at carriageway level and founded in stiff clay, an upper bound value of lateral wall movement of 0.2% of the excavation depth was found. On this basis a maximum movement at the top of the wall of no more than 17mm would be anticipated which correlates well with the analytical results for case 1. Further validation of the suitability of this model was obtained from Watson and Carder (1995) where wall construction was carried out using temporary horizontal props. Movements measured immediately after construction were very similar to those calculated using a Mohr-Coulomb back-analysis employing the same soil modulus as for case 1.

6.2 TEMPORARY RAKED PROPS

A summary of the movements obtained from the sensitivity analysis of the efficiency of raked props as temporary support and the related control cases is given in Table 2. For all soil models the general pattern of behaviour of the stabilising base wall constructed using temporary raked props compared with the two control cases is similar to that described in Section 5.2. However the magnitudes of the lateral wall movements are very different. At completion of construction movements of the top and toe of the wall were in the approximate ratios 1 : 1.5 : 3.1 and 1 : 1.5 : 2.8 respectively for Cases 1,2 and 3. This increase broadly reflected the $1/E'$ ratios for the various soil models as given in Section 6.1.

The results after 120 years given in Table 2 confirm that for the walls constructed with a stabilising base the dominant movement mechanism is one of heave below formation level which results in considerable outward movement of the toe of the wall. In the cantilever situation, with no prop at carriageway level, this toe movement was accompanied by large (>20mm) movements of the top of the wall.

There is a paucity of field data on wall movements when using temporary raked props. One of the few examples is given by Carder and Brookes (1992) for the A40 Long Lane Improvement Scheme where temporary raked props were used during construction of a bored pile wall with a stabilising base. At this site, lateral movements of the top of the wall of about 10mm were measured on completion of construction although it must be noted that some earlier movement may have occurred because readings commenced after excavation of the central haul road. These measurements tended to indicate that the analysis using Case 1 parameters provided the better fit.

7. CONCLUSIONS

Finite element modelling has been undertaken to investigate the effectiveness of both soil berms and raked props as temporary support during construction of embedded retaining walls permanently propped at carriageway level. In the former case construction involving a hinged permanent prop spanning an underpass has been assumed and in the latter the use of a stabilising base integral with the wall. The results have been compared with those calculated if temporary horizontal props were used or if the wall were allowed to cantilever. The analyses have been carried out assuming the wall was founded in stiff overconsolidated clay and may not be applicable if water bearing strata or weak layers are present. The following main conclusions were reached

TABLE 2

Wall movements (mm) from raked prop analyses using different soil models

Analysis	Wall movement	After completion of construction			After 120 years		
		Case 1	Case 2	Case 3	Case 1	Case 2	Case 3
Cantilever	Top	18.2	28.7	57.4	40.8	62.3	143.8
	Toe	7.2	10.4	21.4	20.1	29.0	69.7
Raked props	Top	17.8	27.7	54.9	17.4	27.8	38.9
	Toe	7.6	11.1	20.8	19.4	28.3	68.8
Horizontal props	Top	15.4	23.8	47.1	16.9	26.9	47.0
	Toe	7.5	10.9	22.2	19.3	28.2	87.3

Note: Case 1 is Mohr-Coulomb, E' = 32 + 8.4z
Case 2 is Mohr-Coulomb, E' = 12 + 6.24z
Case 3 is Schofield model with no tension cut-off

from the modelling, but need to be validated by field monitoring studies.

(i) A berm base of 8m and slope of 45° were assumed throughout as being appropriate for a road scheme. On this basis, short and long term lateral movements of the top of the wall when using progressive construction with 6m high berms were similar to those expected if using temporary horizontal props. A 6m high berm would therefore be equally effective in controlling wall movements. Short term movements with a 2m high berm were closer to the cantilever situation although longer term movements were similar to those with horizontal propping. The advisability of relying totally on the smaller berm without employing monitoring using the Observational Method must be questionable.

(ii) When using soil berms, heaves at formation level after 120 years were 20% higher than those calculated with horizontal props as temporary support. However peak bending moments in the wall near formation level were only about two thirds of those predicted for horizontal propping.

(iii) Temporary raked props were not as effective as horizontal props in controlling the short term movements of the top of a stabilising base retaining wall, although overall movements in the longer term were the same. Heaves at formation level and wall bending moments in both the short and long term were near identical whether using horizontal or raked propping.

(iv) Generally wall movements in the short term were primarily a function of the undrained response of the soil to bulk excavation. In the longer term, dissipation of excess porewater pressures occurred which was accompanied by heaves at and below formation level. Investigation using different soil models demonstrated that soil stiffness was of paramount importance in predicting wall movements.

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