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Finite element analysis of construction sequences for propped retaining walls

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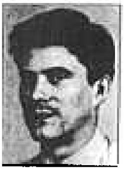
■ A series of finite element analyses has been carried out, in which the effects of the construction sequence and pre-excavation lateral earth pressures on the behaviour of an in-situ retaining wall propped at both crest and formation level in a clay soil were investigated. Three sequences of construction were considered: excavation in open cut with a temporary prop at approximately mid-depth; top down construction with a temporary prop at mid-depth; and top down construction with no temporary prop. The early installation of the permanent prop at the crest was found to be very effective in limiting wall movement at this level. Minimum wall movements occurred with top down construction using temporary props. Minimum bending moments occurred with construction in open cut, but wall movements were largest in this case. In addition to limiting wall movements during excavation, temporary props are advantageous in that their removal will effectively pre-load the permanent prop at formation level. Bending moments, and the component of wall deflection due to flexure, are largest in the case of the top down construction sequence without temporary props, because the permanent prop takes comparatively little compressive load. However, the reduced permanent prop load may be due to soil/prop interaction effects. These considerations, together with the implications for cost and safety, will influence the programming of excavation and construction sequences for propped retaining walls in practice.

Introduction

Underpasses and deep basements in urban areas are increasingly constructed from the top down, between retaining walls installed using in-situ techniques. These methods are particularly useful where the land available for temporary works activities is restricted. The incorporation into the permanent structure of props at formation level has been shown to be advantageous, in that the depth of embedment required for stability is comparatively small (Powrie and Li, 1991).[1,2] In a structure such as a road underpass or a tunnel, it may also be possible to utilize the overbridge or the tunnel roof as a permanent prop at crest level. This will result in a structural system in which collapse is unlikely to be an issue, but whose statical indeterminacy may lead to problems in estimating prop loads and wall bending moments.

2. A further consideration is that substantial savings in cost may be made by the judicious specification of the sequence of excavation and prop installation (perhaps including temporary props) between the side-walls of the underpass or the tunnel. The design might be governed by the need to limit deformations during construction, which could call for the crest level prop to be installed at the earliest opportunity. On the other hand, the problems associated with working in a confined space might militate against this, so that excavation to formation level in an open cut, with the provision of temporary props at one or more levels, might be preferred. However, the potential dangers of placing and removing large, heavy temporary props would require their use to be kept to a minimum.

3. The influence of the sequence of excavation and propping during the construction of an underpass with a retained height of approximately 9·5 m has been investigated by means of a series of finite element analyses carried out using the program CRISP (Britto and Gunn, 1987),[3] which features fully coupled consolidation. The soil parameters used represented a stiff, overconsolidated boulder clay, and were chosen to facilitate comparison with previous work (Powrie and Li, 1991).[1]



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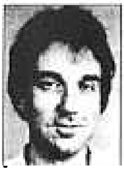

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Geometry and construction sequences investigated

4. Figure 1 shows an idealized cross-section through the underpass structure modelled in the finite element analyses. The finished road surface is 9·55 m below the original ground level (OGL). The side walls of the completed tunnel are propped at both crest and carriageway level as indicated. The connections between the props and the wall were assumed to be incapable of transmitting bending moments. For a formation level reinforced concrete prop slab which is cast up against the retaining wall, this will lead to the overprediction of wall movements and bending moments (Powrie and Li, 1991).[1] However, the neglect of the self-weight of the higher level props will have the opposite effect.

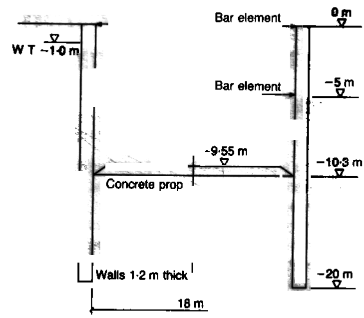

Fig. 1. Idealized cross-section through underpass (not to scale)

5. Three different construction sequences were investigated:

(*a*) *Construction in open cut, with one level of temporary props.* Excavation to 5·5 m below OGL; installation of temporary props at 5 m below OGL; excavation to 10·3 m below OGL; installation of a 0·75 m thick reinforced concrete permanent prop slab; installation of the permanent props at crest level; removal of the temporary props.

(*b*) *Top down construction, with one level of temporary props.* Excavation to 2·2 m below OGL; installation of the permanent props at crest level; excavation to 5·5 m below OGL; installation of temporary props at 5 m below OGL; excavation to 10·3 m below OGL; installation of a 0·75 m thick permanent reinforced concrete prop slab; removal of the temporary props.

(*c*) *Top down construction without temporary props.* Excavation to 2·2 m below OGL; installation of the permanent props at crest level; excavation to 10·3 m below OGL; installation of a 0·75 m thick permanent reinforced concrete prop slab.

6. The overall depth (retained height plus embedment) of the wall was 20 m in all cases. In reality, the self-weight of a reinforced concrete prop slab at crest level would be significant, and the side walls of the underpass would probably be required to carry this load. To facilitate comparison with previous analyses, the vertical loads transmitted to the walls by the crest level props have been ignored. It must be appreciated, however, that these could have a significant effect on the behaviour of the structure, especially if the roof slab is subjected to traffic or other live loads in addition to its self-weight. Where the retaining wall is well propped, the vertical loading might be a governing factor in the determination of the required depth of embedment.

Input parameters

Materials properties

7. The soil was modelled using a finite element formulation of a behavioural regime proposed by Schofield (1980),[4] shown in Fig. 2. This model incorporates the Cam clay yield surface on the wet side of the critical state, and the Hvorslev surface and a no-tension cut-off on the dry side. The values of the soil parameters used are given in Table 1: these are representative of a stiff, overconsolidated boulder clay (Powrie and Li, 1991).[1] The slope of the line joining critical states in $q:p'$ space was taken as $M = 1{\cdot}03$. This was based on a critical state angle of shearing resistance $\phi'_{crit} = 26°$ measured in drained triaxial compression and might lead to an unrealistically high value of ϕ' in plane strain. However, the present analyses are, in the main, substantially insensitive to the value chosen for M because the soil is generally remote from the critical state. The geological stress history of the clay was assumed to com-

Table 1. Soil parameters used in the analysis

Parameter	Symbol and value
Slope of one-dimensional compression line in v–$ln\ p'$ space	
Slope of unload/reload line in v–$ln\ p'$ space	
Specific volume (and void ratio) on critical state line at $p' = 1$ kPa in $v = ln\ p'$ space	
Slope of critical state line in $q:p'$ space	
Poisson's ratio	
Unit weight of water	
Bulk unit weight of soil	
Permeability in vertical direction	
Permeability in horizontal direction	
Slope of Hvorslev surface in $q:p'$ space	
Slope of no-tension cut-off in $q:p'$ space	
Permeability in vertical direction for tensile fracture region	
Permeability in horizontal direction for tensile fracture region	

prise one-dimensional compression, followed by the removal of an effective overburden of 2500 kPa to the current effective stress state. Lateral effective stresses were calculated assuming $K_0 = \sigma'_h/\sigma'_v = (1 - \sin \phi')\mathrm{OCR}^{\sin \phi'}$ (Mayne and Kulhawy, 1982).[5] The water-table was set at 1 m below original ground level.

8. The wall and carriageway slab were modelled using impermeable elastic elements having a Young's modulus $E = 17 \times 10^3$ MPa, Poisson's ratio $\nu = 0{\cdot}15$ and unit weight 22 kN/m^3. This value of Young's modulus is on the low side, but takes account of possible long-term cracking. The temporary struts were modelled using bar elements with a stiffness in axial compression $P/\delta = EA/L = 2{\cdot}56 \times 10^5$ kN/m per m, where δ is the end displacement in response to an axial load P, and E is the Young's modulus, A the cross-sectional area and L the length of the bar element. This is equivalent to 600 mm dia. × 12·5 mm thick circular hollow section steel props placed at 2 m centres, spanning the entire width of the excavation. The interface between the wall and soil was modelled using slip elements having an elastic shear modulus $G = 7{\cdot}5$ MPa ($E' =$ 20 MPa and $\nu' = 0{\cdot}2$) until a shear stress of σ' tan 26° was reached, when the shear modulus was reduced by a factor of 100.

Finite element mesh and boundary conditions

9. The finite element mesh and the displacement boundary conditions are shown in Fig. 3. The idealized geometry (see Fig. 1) is symmetrical about the centre line, so the mesh represents one-half of the cross-section through the tunnel. The lower horizontal boundary to the mesh was set at the interface between the boulder clay and the underlying bedrock. The far vertical boundary is sufficiently remote from the wall for changes in stress and strain to be negligible in practice.

10. The soil, wall and carriageway slab were modelled using eight noded quadrilateral elements. The temporary prop and the permanent roof level prop were modelled using 3-noded bar elements.

Pre-excavation stress state

11. The initial in-situ stresses were calculated from the estimated stress history of the deposit, as outlined above. Two analyses were carried out for each construction sequence. The effects of wall installation were represented by a reduction in the value of the earth pressure coefficient K from its in-situ value K_0 to a value of 1·0 over the depth of the wall for the first analysis, and to 2·0 for the second. This representation of wall installation effects is not ideal, because it extends horizontally across the entire mesh. Nonetheless, it should provide upper and lower estimates of the lateral stresses after installation of the wall but prior to excavation. Fig. 4 shows the initial in-situ lateral earth pressure coefficient profile, together with the $K_i = 1{\cdot}0$ and $K_i = 2{\cdot}0$ profiles used as the starting states in the analyses. For the second analysis, with the higher pre-excavation lateral earth pressure coefficients behind the wall, the lateral stresses were assumed to remain unaffected by the installation process where the initial in-situ value of K_i was less than 2·0.

12. In analyses of this type, a surcharge is sometimes applied to the soil behind the retaining wall, to account for the effects of nearby buildings and construction traffic etc. Such a surcharge was not applied in the analyses described in this Paper.

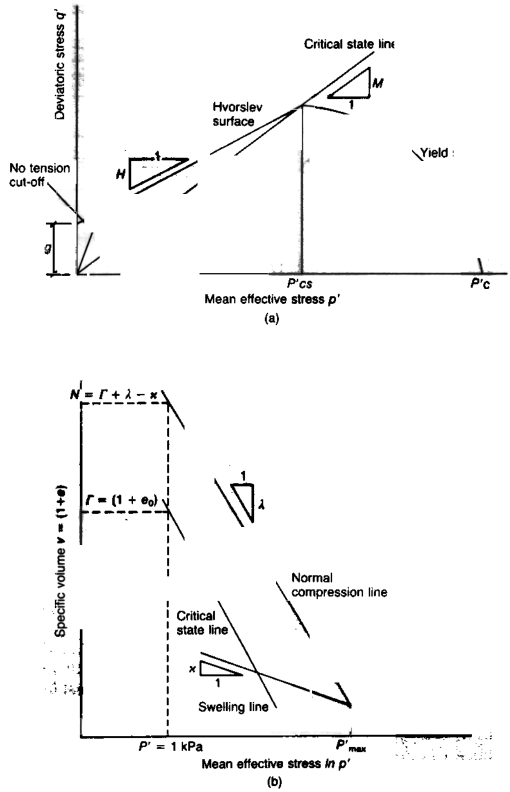

Fig. 2. Soil model: (a) Cam clay yield surface, Hvorslev surface and no-tension surface (schematic); (b) Cam clay model in v–ln p' space

Sequence and results of analysis

13. The sequence of each analysis, starting with the wall already in place, was as follows.

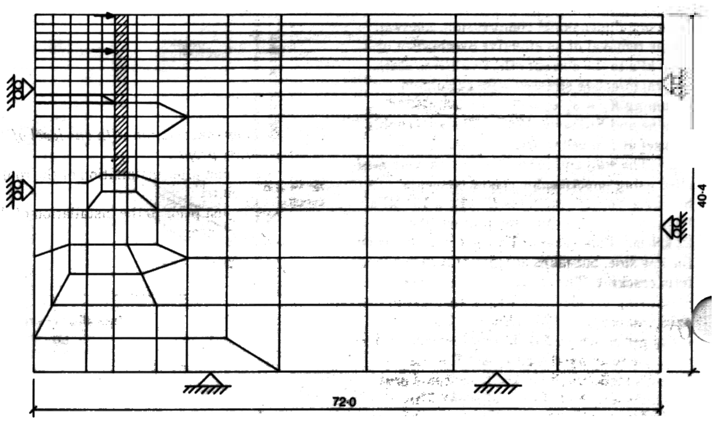

Fig. 3. Finite element mesh (dimensions in m)

Sequence A—construction in open cut with onc level of temporary props

(*a*) Removal of elements over a period of 22 days, simulating excavation to 5·5 m below OGL (Stage 1).
(*b*) Addition of a bar element to limit the lateral movement of the wall, simulating the installation of the temporary prop at a depth of 5 m below OGL.
(*c*) Removal of elements over a period of 28 days, simulating excavation to 10·3 m below OGL (i.e. 0·75 m below final formation level).
(*d*) Seven days' excess pore-water pressure dissipation.
(*e*) Addition of concrete elements, simulating the placement of a permanent reinforced concrete slab at formation level (Stage 2).
(*f*) Addition of a bar element, at the top of the wall, simulating the installation of the permanent prop at crest level.
(*g*) Removal of the bar element at 5 m below ground level, simulating the removal of the temporary prop (Stage 3).
(*h*) 120 years' excess pore-water pressure dissipation, modelling the long-term behaviour of the wall (Stage 4).

Sequence B—top down construction with one level of temporary props

(*a*) Removal of elements over a period of nine days, simulating excavation to 2·2 m below OGL.
(*b*) Addition of a bar element at the top of the wall, simulating the installation of the permanent crest level prop (Stage 0).
(*c*) Removal of elements over a period of 18 days, simulating excavation to 5·5 m below OGL (Stage 1).
(*d*) Addition of a bar element to limit the lateral movement of the wall, simulating the installation of the temporary prop at a depth of 5 m below OGL.
(*e*) Removal of elements over a period of 28 days, simulating further excavation to a depth of 10·3 m below OGL (i.e. 0·75 m below final formation level).
(*f*) Seven days' excess pore-water pressure dissipation.
(*g*) Addition of concrete elements, simulating the placement of a permanent reinforced concrete slab at formation level (Stage 2).
(*h*) Removal of the bar element at 5 m below OGL, simulating the removal of the temporary prop (Stage 3).
(*i*) 120 years' excess pore-water pressure dissipation, modelling the long-term behaviour of the wall (Stage 4).

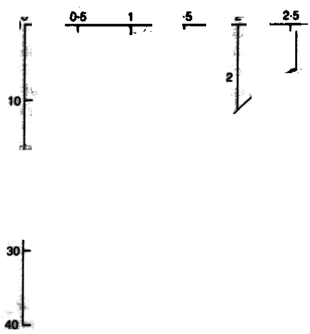

Fig. 4. Initial in-situ lateral earth pressure coefficient profile

Sequence C—top down construction with no temporary props

(*a*) Removal of elements over a period of nine days, simulating excavation to 2·2 m below OGL.
(*b*) Addition of a bar element at the top of the wall, simulating the installation of the permanent crest level prop (Stage 0).
(*c*) Removal of elements over a period of 45 days, simulating excavation to 10·3 m below OGL.
(*d*) Seven days' excess pore-water pressure dissipation.
(*e*) Addition of concrete elements, simulating the placement of a permanent reinforced concrete slab at formation level (Stage 2).
(*f*) 120 years' excess pore-water pressure dissipation, modelling the long-term behaviour of the wall (Stage 4).

14. Note that Stage 0—the early placement of the top prop—applies only to construction sequences B and C. Similarly, Stage 1 (which is associated with the placement of temporary props at 5 m below OGL), and Stage 3 (which is associated with the removal of the temporary props) are omitted from sequence C.

15. The results of the analyses are presented in Fig. 5 (deflections, $K_i = 1$), Fig. 6 (bending moments, $K_i = 1$), Fig. 7 (deflections, $K_i = 2$) and Fig. 8 (bending moments, $K_i = 2$).

16. For each analysis, profiles of wall deflection and wall bending moments are given at the stages indicated below.

(*a*) *Construction in open cut with one level of temporary props*
Stage 1: just before installation of the temporary prop.
Stage 2: just after the placement of the carriageway slab.
Stage 3: after the addition of the roof level prop and immediately after the removal of the temporary prop.
Stage 4: 120 years after construction.

(*b*) *Top down construction with one level of temporary props*
Stage 0: just after installation of the roof level prop.
Stage 1: just prior to the installation of the temporary prop.
Stage 2: just after placement of the carriage slab.
Stage 3: immediately after the removal of the temporary prop.
Stage 4: 120 years after construction.

(*c*) *Top down construction with no temporary props*
Stage 0: just after installation of the roof level prop.
Stage 2: just after placement of the carriageway slab.
Stage 4: 120 years after construction.

Table 2 provides a summary of the key results at certain of these stages, including maximum prop loads and wall deflections and bending moments.

Discussion: $K_i = 1$ analyses

Construction in open cut with one level of temporary props

17. The deflected profiles shown in Fig. 5(a) for construction in open cut with one level of temporary props are consistent with rigid body rotation of the wall about the prop positions at

Table 2. Key results from all analyses

Result		Stages							
		Sequence A			Sequence B			Sequence	
						3	4	2	4
Outward deflection at crest of wall: mm	$K_i = 1$	28·04		28·29	14·33	14·61	14·67	14·67	14·68
	$K_i = 2$	28·61		29·01	11·79	12·18	12·21	12·26	12·28
Outward deflection at toe of wall: mm	$K_i = 1$	15·17		13·14	17·19	15·31	13·05	15·03	14·39
	$K_i = 2$	11·28		10·65	13·11	11·56	10·79	11·15	12·07
Outward deflection at temp. prop position: mm	$K_i = 1$	28·66		33·27	19·92	24·81	25·44	30·06	30·44
	$K_i = 2$	26·68		33·51	17·82	24·61	25·09	30·36	30·63
Outward deflection at formation level: mm	$K_i = 1$	27·77		28·81	24·49	24·93	25·36	32·06	32·37
	$K_i = 2$	23·66		24·91	21·18	21·97	22·19	29·68	29·88
Approximate max. wall bending moment: kNm per m	$K_i = 1$	694		1082	782	1176	1272	1589	1638
	$K_i = 2$	785		1482	1061	1721	1795	2157	2192
Load in temporary prop: kN/m (where applicable)	$K_i = 1$	560			858				
	$K_i = 2$	928			1011				
Load in permanent prop (carriageway level): kN/m	$K_i = 1$		323	212		396	224	0	−17·6
	$K_i = 2$		526	571		572	549	0	183
Load in permanent prop (crest level): kN/m	$K_i = 1$		220	247		284	300	353	364
	$K_i = 2$		352	395		470	492	548	571

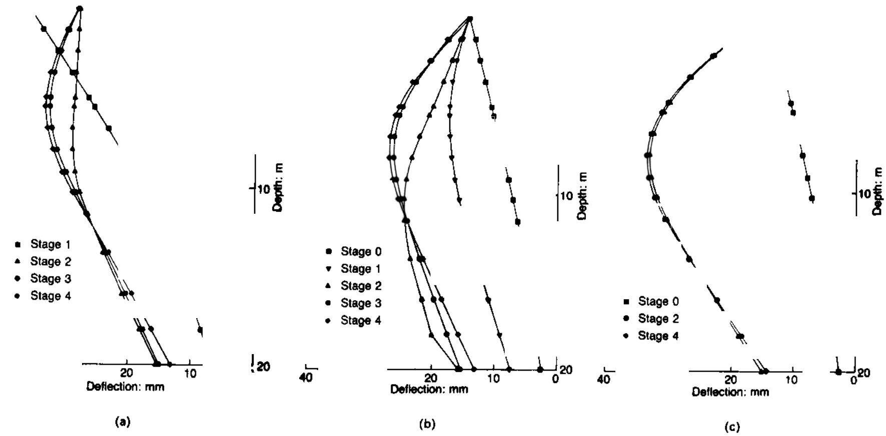

Fig. 5. Deflections, $K_i = 1$

each stage, with bending deformation superimposed. It can be seen that the deflections due to compression of the temporary and formation level props are generally not insignificant. During excavation following the installation of the temporary prop, the wall rotated about the prop producing increased toe deflections and a reduced crest deflection (Stage 2). Placement of the carriageway slab and permanent prop at crest level, together with the removal of the temporary prop, produced an increase in bending deflection above formation level, with the lower part of the wall moving away from the excavation (Stage 3). In the long term, as the excess pore-water pressures dissipated (Stage 4), this trend continued but further movement was small (0·7 mm at 5 m below OGL, and 1·8 mm at the toe).

18. Placement of the carriageway slab and permanent prop at crest level, together with the removal of the temporary prop, produced a 40% increase in the maximum bending moment (from 694 to 980 kNm/m: Fig. 6(a)). The position of the maximum bending moment moved

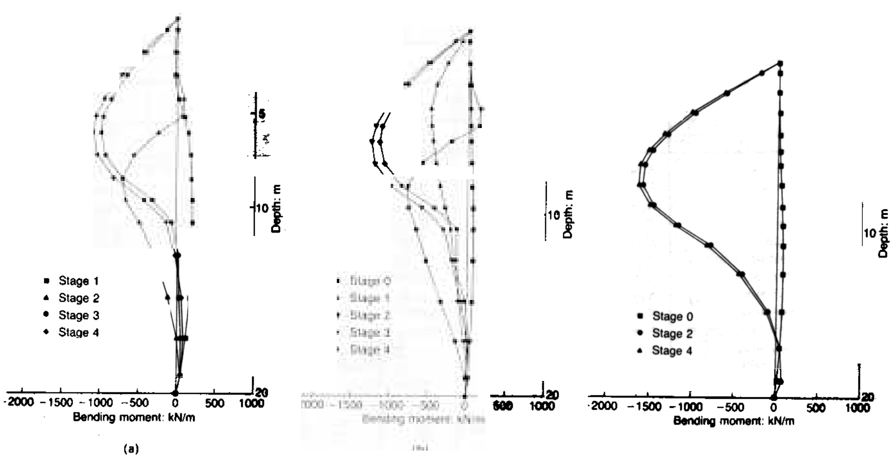

Fig. 6. Bending moments, $K_i = 1$

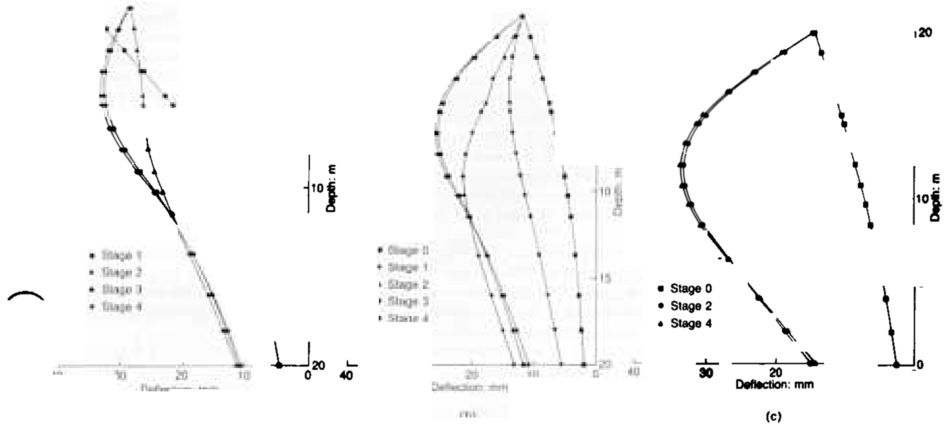

Fig. 7. Deflections, $K_i = 2$

up to the position of maximum deflection, at about 5 m below OGL.

19. The maximum load in the temporary prop, 560 kN/m, occurred just prior to its removal. The removal of the temporary prop resulted in an increase in the wall bending moments and deflections. In the long term, the load in the permanent prop slab at carriageway level decreased. This is, at first sight, surprising but may be due to the hogging of the prop slab (which would tend to pull the ends together) as a result of the restrained swelling of the soil below it. The effect might not be observed with a propping system which did not restrain swelling, for example discreet props at intervals, or a slab with a void beneath it.

Top down construction with temporary props

20. Figure 5(b) shows the deflected profiles for the top down construction method with one level of temporary props. These are again consistent with rigid body rotation about the prop

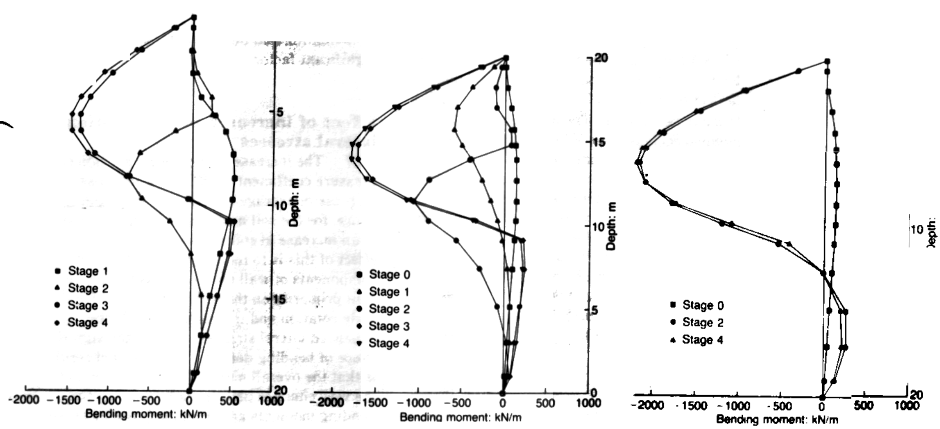

Fig. 8. Bending moments, $K_i = 2$

positions, with the effects of bending and prop compression superimposed. After excavation to a depth of 2·2 m, and just prior to the installation of the permanent prop at crest level (Stage 0), the wall movements at the crest and the toe were 14·3 mm and 2·5 mm respectively. Installation of the top prop and further excavation to a depth of 5·5 m (Stage 1) caused the wall to deflect further to a maximum of 17·4 mm, at the current level of the excavated soil surface. The toe deflection increased to 7·4 mm at this stage.

21. Excavation to 0·75 m below final formation level (i.e. a depth of 10·3 m below OGL) resulted in a maximum wall deflection of 24·5 mm prior to placement of the carriageway slab. Generally, the maximum wall deflection at each stage of construction occurred close to the current level of the excavated surface. Although early installation of the permanent prop is clearly important in limiting the magnitude of the crest deflections, at the stage just prior to the installation of the carriageway prop slab the wall is supported substantially by the temporary prop. The temporary prop load at Stage 2 was 858 kN/m, while the load in the crest level permanent prop was only 4·5 kN/m. This is consistent with the sense of incremental wall rotation between placement of the temporary prop and excavation to 10·3 m below OGL.

22. The maximum bending moment at Stage 2 (Fig. 6(b)) was 782 kNm/m, which occurred at 8·5 m below original ground level. The bending moment profile is consistent with the system of loading and support present at this stage.

23. The removal of the temporary prop, seven days after the placement of the carriageway slab (Stage 3), caused further bending between the permanent supports, and the lower part of the wall rotated back into the soil on the retained side. The maximum deflection increased by 1·8 mm, and the toe deflection decreased by 1·9 mm to 15·3 mm. The point of maximum deflection moved up from 10·3 m below OGL to 8·5 m below OGL. The maximum bending moment increased by 50% compared with Stage 2. The load in the carriageway level prop was 396 kN/m, and the load in the permanent prop at crest level increased to 284 kN/m. As long term conditions were approached, bending moments and wall deflections both increased slightly in the long term to maxima of 1272 kNm/m and 27·0 mm respectively. The prop force in the carriageway level slab decreased to 224 kN/m, while the prop force in the permanent prop at crest level increased slightly to 300 kN/m.

Top down construction with no temporary props

24. Comparison of Fig. 5(c) with Fig. 5(b) shows clearly the effect of the temporary prop in limiting the deflection of the wall during excavation. The final maximum deflection of 33·7 mm for top down construction without temporary props is actually slightly greater than the maximum deflection resulting from construction in open cut with one level of temporary props. Perhaps more importantly, the bending moments resulting from top down construction without temporary props (Fig. 6(c)) are 50% higher than those associated with construction in open cut with temporary props at one level (Fig. 6(a)).

25. For the top down construction sequence without temporary props, the long term deflections and bending moments are very close to those at the end of the construction phase (Figs 5(c) and 6(c)).

26. The formation level prop will only begin to work in compression as the movement of the wall into the excavation is restrained. Compressive loads might be reduced or negated by the tension induced as a result of the tendency of the prop slab to hog as it restrains the swelling of the clay below it. The removal of a temporary prop, some of whose load is then redistributed into the permanent props, is an efficient method of pre-loading the permanent prop at carriageway level, which might otherwise carry little load. This effect is clearly seen in the top down construction sequence without temporary props. The long term load in the permanent prop at crest level is 364 kN/m: the largest of the three construction sequences examined. For the carriageway level prop slab, a small tensile stress is indicated. This, together with the slight inward movement of the wall at formation level after placement of the formation level props, is consistent with the dominant effect of hogging due to the restraint of swelling of the underlying clay. Current research suggests that the slope of the Hvorslev surface, which controls dilation and peak strengths, may also be a significant factor.

Effect of increased pre-excavation lateral stresses

27. The increase in the pre-excavation earth pressure coefficient (to $K_i = 2$) leads to an increase in average effective stress p' and hence, for the soil model used in the analyses, to an increase in soil stiffness. Generally, the effect of this is to reduce the magnitude of the components of wall movement which are dependent primarily on the soil stiffness (i.e. rigid body rotation and translation), while the enhanced lateral stresses increase the significance of bending deformations. The net result is that the overall maximum displacement of the wall changes little in each case, but the bending moments and prop loads are increased quite significantly. Two points are of particular interest

(*a*) With the increased pre-excavation earth pressure coefficients, the magnitudes of the reverse bending moments which occur at later stages towards the bottom of the wall are increased in all three cases (Figs 6 and 8).

(*b*) Prop loads are increased significantly in every case. For the permanent props at carriageway level, the decrease in prop load between the end of construction and the long term which occurred with $K_i = 1$ is reversed or markedly reduced. This implies that the effect of slab hogging due to the restraint of swelling of the underlying clay is much less significant relative to the tendency of the walls to move in to the excavation after construction. For the top down construction sequence with no temporary props, the analysis indicates a compressive load in the permanent prop slab at formation level of 183 kN/m in the long term, in contrast to the apparent tensile stress in the $K_i = 1$ analysis.

Conclusions

28. The early installation of the permanent prop at crest level is very effective in limiting the movement of the crest of the wall. This is consistent with the observations made by Peck (1969).[6] In general, the earlier in the construction sequence a prop is installed, the more load it is able to carry. This is because a prop will only be compressed by resisting movement of the wall into the excavation. A prop which is installed after the wall has already moved will be of limited use. The formation level prop slab must inevitably be installed late in the construction process. However, it can be pre-loaded to some extent by the removal of temporary props, because at least part of the load taken by the temporary props is redistributed into the permanent prop slab.

29. In comparison with construction in open cut, the use of top down construction with temporary props at one level reduces the maximum wall displacement, but the component of wall deflection due to bending is increased. This is reflected in an increase in bending moments. The omission of the temporary prop from the top down construction sequence seems to make little difference to the ultimate deflection of the wall at the crest and the toe. However, bending moments and bending deflections are increased, with the result that the maximum wall deflection is larger than for either of the other construction sequences. In top down construction without temporary props, there is no preloading of the formation level prop slab, and the compressive load in the crest level prop is substantially higher than in either of the alternative construction sequences. These factors are related to the increase in bending moments. Minimum bending moments occur with excavation in open cut, with temporary props at one level. However, this is at the expense of increased rigid body movement of the wall because the crest is allowed to move forward during the early stages of excavation.

30. A second effect of a permanent prop slab at formation level is that it restrains the swelling of the clay beneath it. This will result in a hogging deformation of the prop slab, which will tend to pull the ends together, perhaps resulting in tensile stresses. This may be the dominant effect in the prop slab, if it is not pre-loaded by the removal of temporary props, and if there is little tendency for the wall to move into the excavation in the long term. Apparently tensile stresses in the permanent prop slab were calculated for the top down construction sequence without temporary props, with a pre-excavation earth pressure coefficient of unity. If the propping system does not restrain swelling, this effect will probably be much less significant.

31. With the soil model used in the analyses described in this Paper, the effect of an increased pre-excavation lateral earth pressure coefficient was generally to increase bending moments and compressive prop loads. The apparent dependence of the prop loads and bending moments on the pre-excavation stress state K_i is of potential practical significance. Wall movements were less significantly affected, due to the increase in soil stiffness which accompanied the increase in average effective stress p' at the start of the analysis.

32. The analyses have shown that the sequence of excavation and propping in front of an in-situ retaining wall in clay may be chosen so as to minimize overall wall deflections (top down construction with temporary props) or bending moments (construction in open cut with one level of temporary props). It might be considered that the bending moments, prop loads and wall movements associated with any of the three construction methods are acceptable. In this case, safety and cost would be the deciding factors in choosing which method to adopt, and top down construction without temporary props might well be an attractive proposition in practice.

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