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FINITE ELEMENT MODELLING OF INSTALLATION EFFECTS

Finite element modelling of installation effects

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Introduction

The process of excavating a hole in the ground and filling it with wet concrete changes the ground stresses in the vicinity of the hole. These changes are not, generally, taken into account in the design of diaphragm, secant and contiguous bored pile walls. The structural design of these types of wall has normally been based on bending moments and prop or anchor forces derived from limit equilibrium calculations. These calculations can be criticised on the grounds that they do not take into account several factors which might be expected to have some effect on the wall bending moments. In particular the in-situ ground stresses, the stiffness of the ground, the stiffnesses of any props and the effects of the construction procedure do not appear explicitly in the limit equilibrium approach.

A soil structure interaction (SSI) analysis technique, using for example the finite element method, has potential to take into account all the above factors. Finite element techniques have generally been considered too time consuming and complex for routine design but, with continuing falls in the cost of hardware and software, their use is increasing. A well documented case of the use of the finite element method to supplement limit equilibrium calculations was the Bell Common cut and cover tunnel on the M25 (Hubbard et al, 1984). Here the analyses were elastic and the values at the bending moments were calculated to be up to $\approx600\text{kNm/m}$. These values were much lower than those from the limit equilibrium approach (3000kNm/m was obtained using CP2 and eventually a design moment of 1450kNm/m was adopted based on the Burland-Potts revised method (Burland et al, 1981)).

Potts and Fourie (1984) reported on some elasto-plastic finite element analyses of propped cantilever retaining walls of a similar geometry to the Bell Common wall, but where some simplifying assumptions had been made (for example the pore pressures were assumed to be zero everywhere). A later publication (Fourie and Potts, 1989) examined cantilever walls. An important finding from the elasto-plastic analyses of the propped walls was that the bending moments and prop forces were much higher for high K_0 soils (e.g. stiff, overconsolidated clays) than for low K_0 soils (e.g. lightly overconsoli-

dated clays). In particular it was found that the bending moments for high K_o, soils were higher than those obtained from limit equilibrium analyses. These results were regarded as significant since the Bell Common tunnel was constructed partly in London Clay with K_o values in the range of 1.5 to 2.0.

The performance of the Bell Common tunnel was monitored (Tedd et al, 1984; Symons and Tedd, 1989) and it has been found that the higher bending moments suggested by finite element analysis have not been achieved in practice (a maximum bending moment of about 300kNm/m has been deduced from measured earth pressures and wall strains). One explanation of this is that the finite element analyses "wished" the wall in place. In other words they assumed that before excavation in front of the wall took place the wall was installed in the ground with no change in the in-situ stresses. As noted above one can expect significant installation effects which modify the ground stresses.

This Paper reports preliminary results of finite element analyses which include the effect of wall installation. The changes in pore water pressures during wall construction and the resulting time dependent flow of water through the soil were modelled using the coupled consolidation option of the CRISP program (Britto and Gunn, 1987).

Modelling assumptions

The analyses reported here were carried out for a wall with a similar geometry to the Bell Common retaining wall. The basic finite element mesh used in these studies is shown in Fig. 1. We have followed Potts and his co-workers in using an idealised geometry and set of soil properties so that comparisons can be made with limit analyses and previous work. Thus we have assumed that the wall is constructed in a material which yields according to the Mohr-Coulomb condition with a drained angle of friction of 25 degrees and a drained cohesion of zero.

The total height of the wall was 20m and its thickness lm. The excavation for the wall was modelled as a plane strain slot rather than the more realistic three-dimensional sequence of excavation used for real diaphragm or pile walls.The retained height of soil was taken as 8m and two cases were considered - in the first the wall was an unpropped cantilever and in the second there was a single rigid prop at the top of the wall. Three different positions of the ground water table have been assumed: at the ground surface, at the base of the retained soil (i.e. a depth of 8m) and at the base of the wall. The coefficient of earth pressure at rest was taken to be 2 throughout.

The bulk unit weights of the pore water, soil and concrete were taken as $10kN/m^3$, $20kN/m^3$ and $23kN/m^3$ respectively. The Young's modulus of the soil has been taken as being $250p'$ where p' is the mean effective stress at the same depth. This leads to three different distributions of modulus with depth
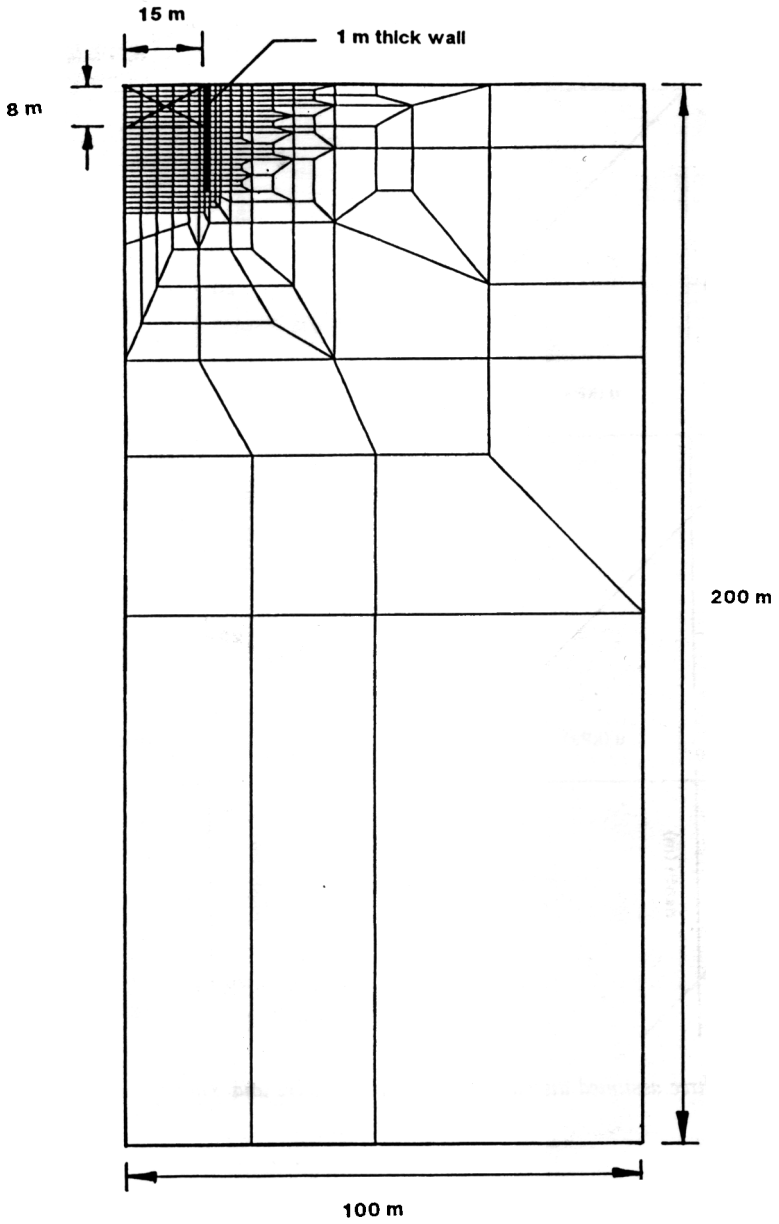
47

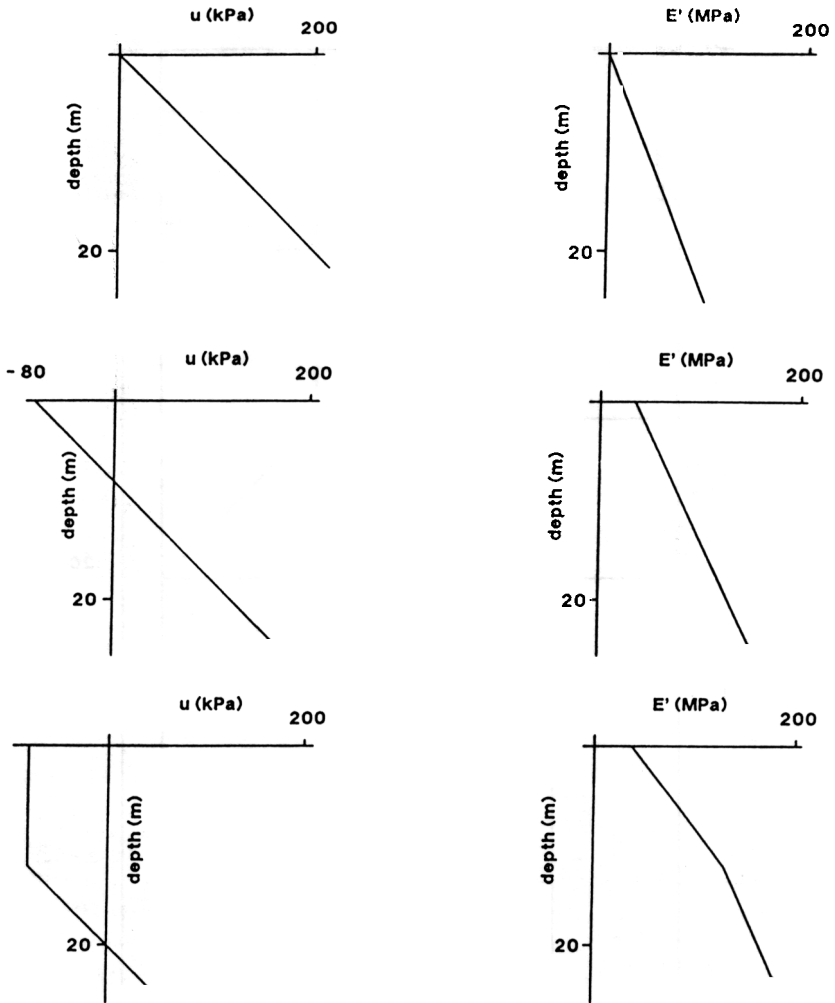Fig. 1. Finite element mesh

Fig. 2. Three assumed distributions of pore pressure and Young's modulus with depth

shown in Fig. 2 for the three different positions of the ground water table. The soil was assumed to have a drained Poisson's ratio of 0.2 and the wall had a Young's modulus of 28 GN/m^2 and a Poisson's ratio of 0.15.

49

stage 1
hole progressively excavated
and supported by bentonite
slurry

stage 2
excavation complete. sides of
hole are impermeable, due
to filter cake

stage 3
wet concrete tremied
in from base upwards

stage 4
hole now full of wet
concrete - water can flow
from concrete to soil

stage 5
concrete sets. 28 days
pass while concrete (now
impermeable) strengthens

stage 6
soil excavated from front
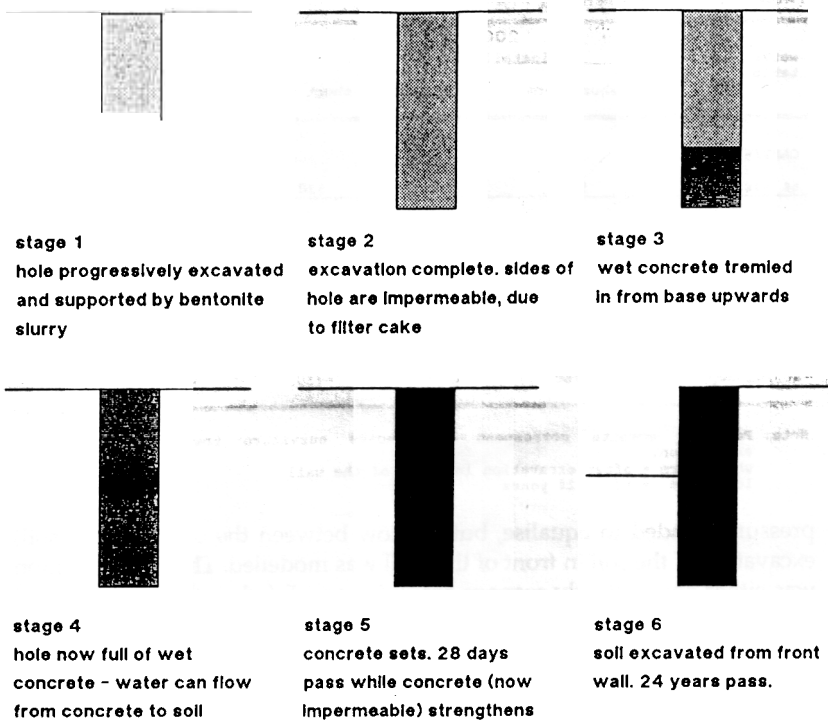wall. 24 years pass.

Fig. 3. Stages in analysis

Two types of analysis were carried out in order to ascertain the effect of wall installation. In the first type ("No Installation" or "NI") the wall is wished into place with no changes in the ground stresses and then the soil is excavated in front of the wall. In the second type of analysis ("Installation"or "I") the whole procedure of installing the wall in the ground is modelled according to the scheme shown in Fig. 3, and then excavation is carried out. Analyses were carried out for both cantilever and propped walls.

The bentonite support was modelled by applying the hydrostatic pressure due to the self weight of the bentonite (12 kN/m^3) to the sides of a hole which were assumed to be impermeable. The wet concrete also initially supports the hole with a fluid pressure, but here water flow was allowed to take place between the concrete and the soil for twelve hours, after which the concrete was assumed to have set. At this point the hydrostatic pressure was removed and finite elements representing the concrete wall were introduced. After 28 days (during which there was flow of water internal to the soil as excess pore

Table 1. Maximum bending moments

water table	no installation		installation	
	short term	long term	short term	long term
CANTILEVER WALLS				
at surface	560	2150	530	2220
at 8m depth	490	-350	-290	730
at 20m depth	640	430	350	760
PROPPED WALLS				
at surface	-1720	-2680	-1210	-2120
at 8m depth	-1810	-2630	-870	-1470
at 20m depth	-1760	-2230	+730	-910

Note: Positive moments correspond to concave curvature towards the excavation.
short term = after excavation in front of the wall
long term = after 24 years

pressures tended to equalise, but no flow between the concrete and soil) excavation of the soil in front of the wall was modelled. The final condition was either one of steady seepage around the wall (when the initial ground water level was at the surface) or otherwise hydrostatic, corresponding to the initial pore water pressure distribution.

Analysis

As may be deduced from the account given above, twelve analyses have been carried out in all: for three initial positions of the ground water table, two types of wall (cantilever or propped at the top) were analyzed according to the NI and I assumptions. We discuss below just one aspect of the results: the bending moment distributions in both the short term (after excavation in front of the wall) and long term (24 years after wall construction). Table 1 summarises the maximum moments in the short and long term for the twelve analyses.

Fig. 4 shows the bending moment distributions calculated for the cantilever walls. The NI and I analyses for the case with the water table at the ground surface are shown in Figs 4(a) and 4(b) respectively. The effect of wall installation is to increase the long term bending moments by a small amount (about 3%). The magnitudes of the moments (around 2200kNm/m) are relatively high, for example Fourie and Potts (1989) report maximum moments of approximately 1500kNm/m and 900kNm/m produced by excavating 8.85m in soil with K_0 values of 2 and 0.5 respectively. There are, of course, several differences in the assumptions made in our analyses and

(a)

bending moment (kNm)

0   2500

wall depth (m)

end of front excavation

24 years after end of
construction

20

no installation (NI)
groundwater table at surface

(b)

bending moment (kNm)

0   2500

wall depth (m)

20

installation modelled (I)
groundwater table at surface

(c)

0   2500

wall depth (m)

20

no installation (NI)
groundwater table at 8 m

(d)

bending moment (kNm)

0   2500

wall depth (m)

20

installation modelled (I)
groundwater table at 8 m

(e)

0   2500

wall depth (m)

20

no installation (NI)
groundwater table at 20 m

(f)

bending moment (kNm)

0   2500

wall depth (m)

20

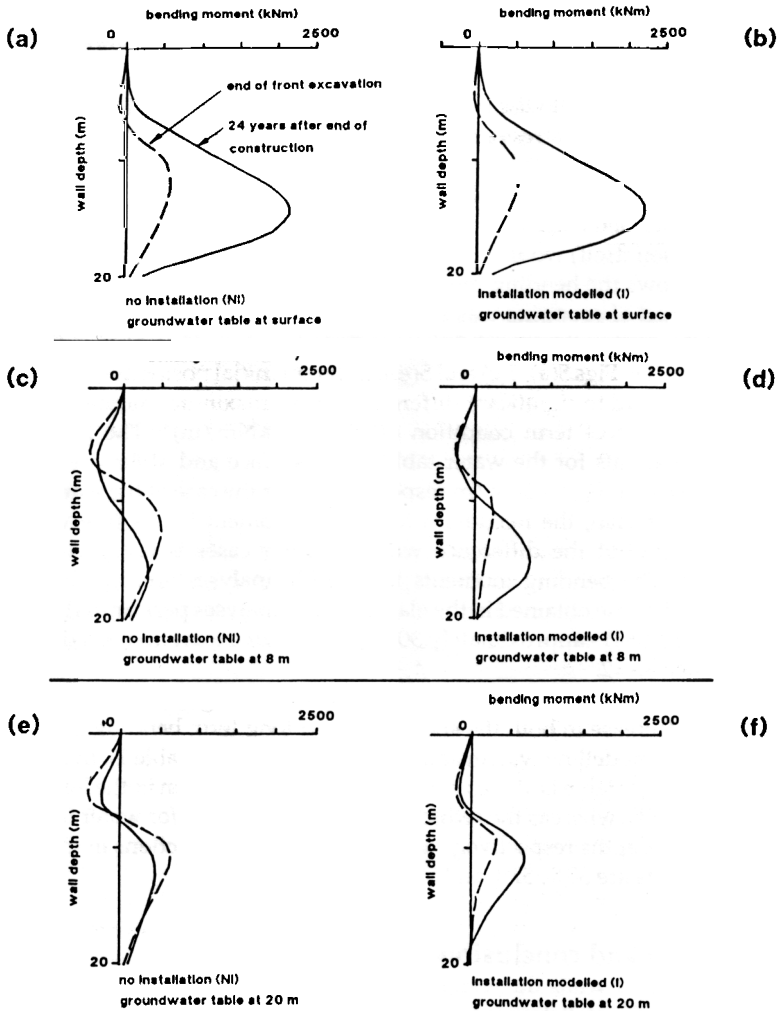installation modelled (I)
groundwater table at 20 m

Fig. 4. Cantilever walls

those of Fourie and Potts. The most significant difference is that Fourie and
Potts assumed that pore pressures in the soil were zero.

Investigation showed that the major contributing factor to our moments
was, indeed, the water pressure behind the wall. This finding prompted the
analyses with water table at 8m and 20m depth, the results of which are

shown in Figs 4(c) to 4(f). As can be seen the lowering of the water pressure greatly diminishes the bending moments (see Table 1). A significant factor in all the long term results is the final pore pressure distribution acting on the wall. Corresponding to the bending moments in Figs 4(c) to 4(f) there is an area of negative total stress acting between the retained soil and the wall, i.e. the soil is "holding up" the wall. Of course this negative total stress,is due to the negative pore pressures that were assumed initially above dredge level and the continued existence of such a suction against a wall (as opposed to within the soil itself) would not be relied on in the long term.

Fig. 5 shows the bending moment distributions for walls propped at the top. The effect of propping was achieved in the analysis by fixing the corner node on the wall so the prop is being modelled as rigid. Considering first the NI analyses (i.e. Figs 5(a), 5(c) and 5(e)), different initial positions of the water table do not lead to significant differences in the maximum moments calculated for the short term condition (about 1800kNm/m). The long term bending moments for the water table at the surface and at 8m are similar (2680kNm/m and 2630kNm/m respectively). For the case of the water table at a depth of 20m, the maximum long term moment is less (the value is 2230kNm/m) but the difference with the other cases is not particularly significant. The bending moments for the NI analyses are broadly comparable with those obtained in the elasto-plastic analyses performed by Potts and Fourie (1984) - approximately 3000kNm/m for excavation to a depth of 9.26m when $K_0=2$.

Turning now to the corresponding I analyses (i.e. Figs 5(b), 5(d) and 5(f)), a marked decrease in both the short term and long term bending moments as a result of modelling wall installation is apparent (see Table 1). In the case where the water table is at the ground surface, the reduction in the long term moment is 21%, whereas the reductions are 44% and 59% for water tables at 8m and 20m depths respectively. The corresponding reductions in the short term moments are 30%, 52% and 59%.

Discussion and conclusions

For cantilever walls the results of our analyses predict that the effect of modelling wall installation is to reduce the short term bending moments and to increase slightly the long term bending moments. The walls are relatively free to move and the stresses acting on them tend to approach the active and passive limits. The most important influence on the final bending moments is the water pressures in the ground. The temporary reduction in ground stresses caused by wall installation has no lasting effect. This is essentially what is assumed when moments are calculated using the limit equilibrium approach - the initial conditions and the construction procedure seems to have little effect.

(a) no installation (NI) groundwater table at surface

(b) installation modelled (I) groundwater table at surface

(c) no installation (NI) groundwater table at 8 m

(d) installation modelled (I) groundwater table at 8 m

(e) no installation (NI) groundwater table at 20 m

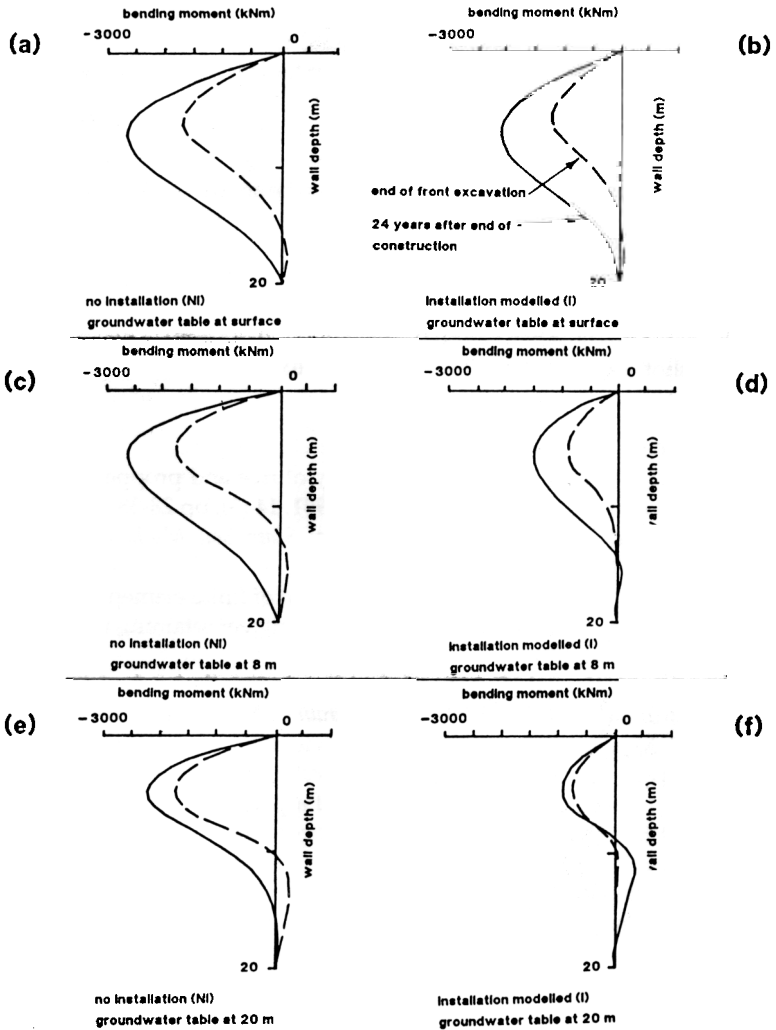(f) installation modelled (I) groundwater table at 20 m

Fig. 5. Propped walls

In contrast, installation effects do seem to be significant when walls are propped. This can be understood as the propping action "locking in" the reduction in lateral stresses associated with wall installation - the soil around the wall does not have the same freedom to strain and approach the classical stress distributions. However, the magnitude of installation effects depends

strongly on the initial position of the ground water table. Installation effects are relatively minor when the groundwater table is high, and become more significant as it falls below excavation level.

When considering the relevance of these results to design it is salutary to recall that in the case of the Bell Common wall that the maximum moments inferred from measurements are estimated to be about 300kNm/m. Not only are these much smaller than those used in design, they are actually of the opposite sign. As Hubbard et al (1984) point out, the designer is unable to predict all the events that might take place during the construction of a wall. This points to the importance of employing all types of predictive analysis to establish the possible bounds on the behaviour of the system being designed. A "best shot" type of analysis is probably unlikely to hit the target.

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