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The use of numerical methods for the design of base propped retaining walls

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ABSTRACT: This paper describes the analysis of a base propped retaining wall constructed in a stiff fissured clay. All the analyses were undertaken before the scheme was constructed in order to assist the design of the permanent and temporary works. The predictions were compared with subsequent measurements of the walls behaviour and good agreement was obtained. This case history provides a valuable insight into the behaviour of these structures.

1 INTRODUCTION

As part of the A406 North Circular Road improvement scheme in London an underpass was to be constructed between Hale End road and Chingford road. The original scheme involved the construction of a series of "T" shaped diaphragm wall panels (see Figure 1) to support the sides of the underpass where depths of excavation exceeded 5m. At deeper sections temporary propping some 6m below ground level was allowed for. The permanent propping comprised a "V" shaped reinforced concrete base slab which dipped towards the centre of the underpass. Pre-cast concrete hinges were used between the slab and the wall; these in conjunction with the "V" shaped slab were designed to accommodate long term heave of the clays beneath the base slab.

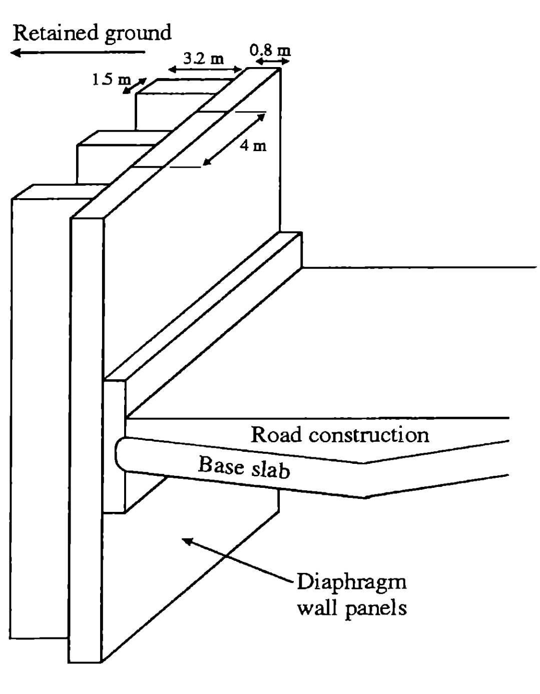

Figure 1. Diagrammatic view of the original scheme

During the tendering process (circa 1988) an alternative scheme was developed which replaced a significant length of the diaphragm wall panels with contiguous bored pile walls. This was over sections of retention less than 12m; where excavation depths exceeded 12m the original design was retained. The form of the base slab remained unchanged as did the slab-wall connection detail (pre-cast hinges with semi-circular bearing surfaces). Temporary props, positioned 1m below the top of the walls, were 1.2m diameter tubular steel sections. Their spacing was dictated by a contractual requirement to achieve a minimum stiffness. As the design evolved alternative methods of supporting the walls were considered and adopted.

The aim of the alternative design was to save construction costs and to develop more efficient and safer methods of working. In the event the revised scheme proved to be approximately 15% cheaper than the original scheme. Cost savings and the benefits of the revisions to the design are discussed by Higgins et al (1998).

This paper discusses the development of the alternative scheme. The design was based on the results of analyses using advanced numerical methods (finite element analyses). Comparatively unsophisticated constitutive soil models were used to develop the scheme and yet the predictions compared well with measurements made during construction and just after it.

2 GROUND INVESTIGATIONS & PROPERTIES

The pre-tender site investigation showed a superficial layer of orange/brown mottled silty clay with coarse gravel about 2m thick, over approximately 5m of weathered London Clay (described as a firm to stiff silty clay). Below this was unweathered London Clay, a stiff to very stiff silty clay with pockets of selenite crystals.

Pressuremeter tests suggested maximum values of K_o (the coefficient of earth pressure at rest) as high as 3.5 (subsequently Carswell et al (1993) reported values ranging from 2 to 4 in the upper 10m of the London Clay. For the analyses an initial stress ("greenfield") condition had to be established. Initial stresses were based on upper and lower bound K_o profiles, (see Figure 2). However, the upper bound profile could not be sustained if typical shear strengths were used for the London Clay. Therefore an enhanced cohesion ($c' = 16$ kN/m^2, $\varphi' = 23°$) was adopted in the top 5m of the London Clay. The lower bound K_o profile was based on the maximum shear stresses that could be sustained when using typical strength parameters ($c' = 8$ kN/m^2, $\varphi' = 23°$).

There were indications from the investigations and published data (CIRIA (1989)) that the site was underdrained primarily due to historic pumping from a Chalk aquifer beneath the clays. The initial condition pore water pressure profile was assumed to be hydrostatic from 1 m below ground level down to a depth of 10m. Below this level pore water pressures

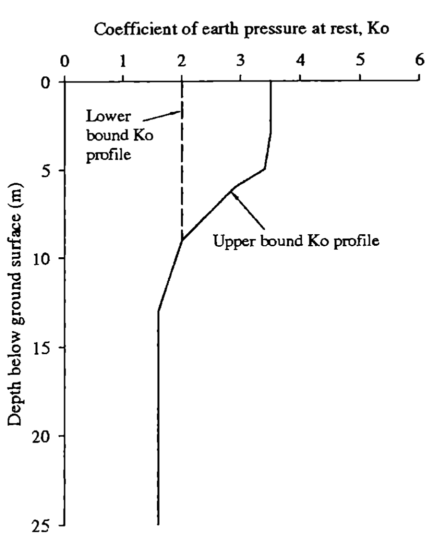

Figure 2. K_o distribution

increased at a rate of 4kN/m^2/m.

Consideration was given to the use of non-linear elastic perfectly plastic constitutive soil models of the form described by Jardine et al (1986). However, given the reliability of data that could be used to assess the non-linear elastic response of the soils, it was considered to prudent to use simpler methods in the first instance. There were severe time restrictions for the design. For the analyses linear elastic perfectly plastic constitutive soil models in which stiffness varied with depth were used (e.g. Burland and Kara (1986)). This was only considered to be an acceptable approach because movements remote from the underpass were not of prime concern. Such an approach would not have been so reasonable if there had not been the same background of construction in the soils encountered at the site (e.g. London Clay) or if predictions of movement remote from the underpass were required.

3 ANALYSES: REQUIREMENTS & FORM

In order to progress the alternative design of the underpass it was necessary to assess the structural integrity and overall stability of the structure and to assess movements of the retaining walls (in this case the index for damage control specified by the Promoter of the scheme and for the design of structures crossing the underpass). Three forms of analyses were undertaken:

- Generalised analyses to examine the sensitivity of the design to various influences
- Analyses of specific sections with special loading conditions from other structures (i.e. bridges)
- Analyses for the design of temporary works and revised methods of construction.

Table 1 summarises the nature of the first set of analyses from which it was possible to draw a number of general conclusions concerning the behaviour of retaining structures and the dominant influences on them. This table also summarises the results of the analyses and the predicted behaviour of the retaining walls. The following influences on the behaviour of the retaining structures were considered:

- Variation in the depth of excavation
- The effect of variations in the K_o profile
- The stiffness (Young's modulus in the short term, $E_{short\ term} = 28$MPa) of the retaining walls. Analyses were run with $E_{long\ term} = ½ E_{short\ term}$ and $E_{long\ term} = E_{short\ term}$.

Table 1. Summary of general analyses

Exc'n depth (m)	Pile dia. (m)	K_0 Profile (Figure 2)		Long term $E_{concrete}$ (MPa)		Rising Groundwater		Wall Ins'tion modelled		Permeable Wall		Summary of Predictions					
		High	Low	28	14	Yes	No	Yes	No	Yes	No	Wall BM	Wall SF	Wall δ_{max}	Prop F	Plinth Force Horz	Plinth Force Vert.
5.5	1.05	■		■		■			■		■	500	350	13	375	700	145
			■	■		■			■		■	500	250	9	250	540	150
7.5	1.20	■		■		■			■		■	1450	750	18	560	1272	281
9.0	1.50	■		■		■			■		■	2300	950	23	700	1740	420
10.0		■		■		■			■		■	3000	1100	27	770	2070	466
11.0	1.80	■		■		■			■		■	4000	1350	29	875	2560	540
			■	■		■			■		■	4100	1250	26	660	2470	540
			■	■		■		■			■	3800	1200	17	494	2368	546
			■	■		■			■	■		3800	1200	17	494	2329	528
		■		■		■			■	■				20		2407	528
12.0	2.1	■		■		■			■		■	5600	1610	32	1015	3105	632
		■			■	■			■		■	5000	1530	34	1019	2999	565
		■		■			■		■		■	3250		42		1859	344
		■			■		■		■		■	2750		51		1783	315
		■			■		■	■			■	2150		37		1490	278
	2.4	■			■		■	■			■	2600	980	32	642	1567	287

(*Wall BM*, maximum bending moment in wall (kNm/m); *Wall SF*, maximum shear force moment in wall (kN/m);*Wall* δ_{max}, maximum wall displacement (mm); *Prop F*, maximum temporary prop force (kN/m); *Plinth Force*, vertical and horizontal components of force generated in wall-slab connection (kN/m))

- The influence of rising groundwater. Approximately 10 years ago it was recognised that groundwater levels in the lower aquifer beneath London were rising due to a reduction in abstraction (to this day schemes are being put forward to control groundwater levels in this aquifer but there is no commitment form government or any other body to do so). Analyses were undertaken in which pore water pressures were allowed to return from the underdrained profile to a fully hydrostatic, steady state, condition. Rising groundwater levels significantly increased the bending moments in the wall (1.2m diameter pile) and appears to reduce the long term wall deflection.
- In the majority of analyses it was assumed that there would be no effect on the ground resulting from installation of the piles forming the walls. In order to assess the effects of any relaxation of stress that might occur when the wall was installed, a number of analyses were run in which the construction of a continuous wall (or slot) was modelled. Although this differs from the mechanism that occurs when a pile is formed (because there is a complex redistribution of stress around it) the "slot effect" was considered to be conservative since it is likely to overestimate the relaxation in stress that is likely to occur.
- The global permeability of the piled wall and the effectiveness of the pore water pressure boundary condition on draining the retained ground. In this case this did not appear to influence behaviour.

Figures 3-6 show, respectively, relationships between the depth of excavation and the bending moments in the pile wall, the force in the connection between the wall and the base slab, the temporary prop force and the maximum lateral displacement of the pile wall. The analyses of sections with applied loadings from other structures were too specific for any conclusions to be drawn concerning the general behaviour of the retaining walls but the analyses used

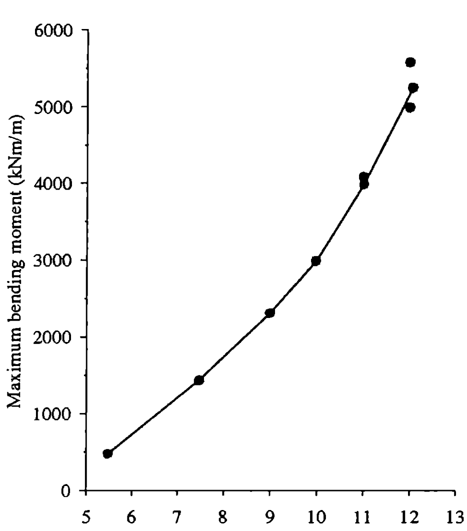

Figure 3. Variation of maximum bending moment with depth of excavation

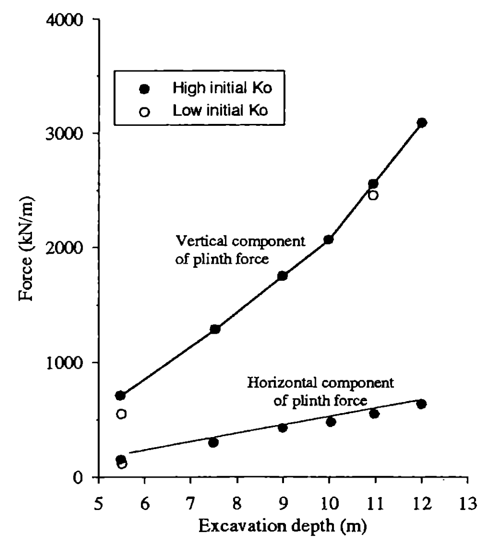

Figure 4: Variation of force in wall/slab connection with depth of excavation

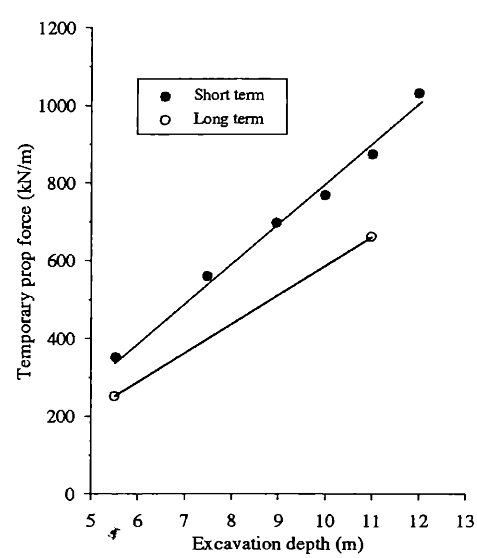

Figure 5. Variation of temporary prop force with depth of excavation

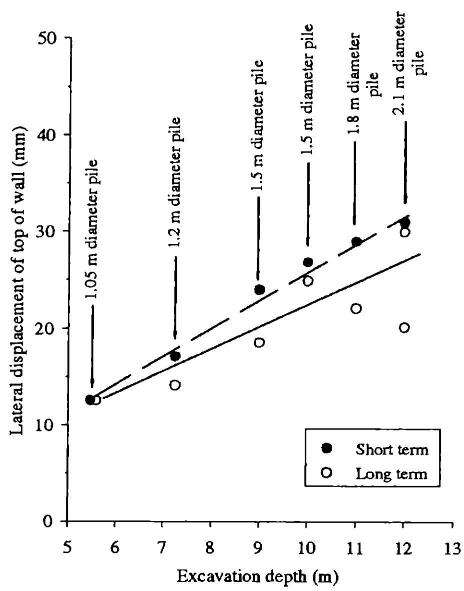

Figure 6. Variation of lateral displacement of top of wall with depth of excavation

for the temporary works assessment did indicate how effective the use of berms might be.

In total, more than 30 finite element analyses were undertaken, unfortunately this was an evolutionary process, and not a parametric study.

Despite this a number of conclusions can be drawn from the analyses as to how base propped retaining structures such as the A406 underpass might perform. Before discussing the behaviour of these structures, in order to validate the analyses some comparisons are made between the predictions (made before construction started; the predictions were therefore Class "A") and measurements made during construction and for a short time after the scheme was complete.

4 PREDICTIONS AND MEASUREMENTS

Carswell et al (1993) reported results of monitoring at the site on a section with an excavation 8m deep. Although there was no directly comparable "Class A" analysis, the results of the analyses outlined in Table 1 can be used to make some simple comparisons as follows:

- A maximum of about 15mm short term wall displacement was measured whereas 20mm was predicted. However, for an 11m (1.8m dia pile) excavation predicted installation effects reduced the maximum wall displacement by up to 7mm. If the same movement occurred during installation for a 1.5m diameter pile wall (as the at the instrumented section) then the predicted displacement would be 13mm.
- The long term prop force was predicted to be 1400 kN/m. Values ranging between 1000 to 1600 kN/m were measured.
- The maximum measured bending moments in the piled wall up to 6 months after construction ranged from 450 to 1450 kNm/m depending on the method of measurement although it is recognised that these are notoriously difficult to measure. In the long term the maximum predicted bending moment was 1750 kNm/m.
- Wall displacements associated with a preliminary trench excavation along the centre of the underpass leaving berms up to 4m deep before the temporary prop was installed (refer to Higgins et al (1998) for further details) were predicted to be between 2 to 3mm. Displacements between 2.5 and 3.0mm were measured.

5 DISCUSSION

This case history gave a valuable insight into the behaviour of base propped retaining walls and this paper has illustrated some of the issues in the design of this form of structure. To validate the results of the analyses the predictions made before construction started were compared with measurements of how the retaining structure performed during construction and in the long term. It is recognised that more sophisticated constitutive models might have been more appropriate, but the analyses, which used comparatively simple models for the soil, appeared to give good agreement with the measurements. However, although the analyses were performed more than 10 years ago, few commercially available finite element programs currently available have more sophisticated soil models. If movements remote from the structure had been required, to assess potential building damage, then use of more sophisticated soil models would have been necessary.

If the clay had been free to swell without restraint, the behaviour of the structure in the long term might have been better predicted (refer to Hight and Higgins (1994)) by using a model such as that described by Jardine et al (1986). Reasons why the simpler model appeared to perform reasonably well may be linked to the restraint afforded by the base slab and walls together with the amount of ground that had yielded in the short term. With such high K_o values, 2.0 to 3.5, the soil would not have needed to strain significantly for yield to occur. Under these

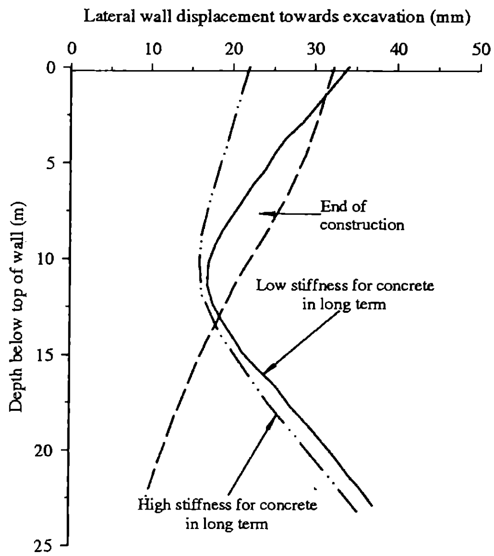

Figure 7. Lateral wall displacements; high and low long term concrete stiffnesses

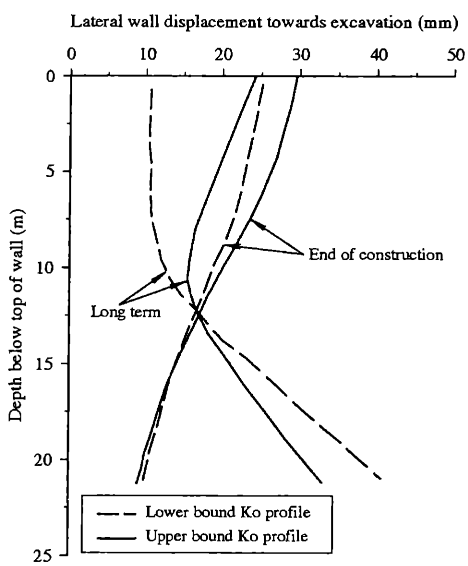

Figure 8. Lateral wall displacements; upper and lower bound K_o profiles

circumstances the behaviour of the ground was probably more associated with post yield behaviour rather than pre-yield characteristics.

Higgins et al (1998) discussed the results of the analyses at greater length and made some observations concerning the way the profile of the piled wall was affected by various influences (long term concrete stiffness (see Figure 7), the assumed K_o profile (see Figure 8), the inclination of the base slab and wall installation). The predictions suggest that in the long term the wall tends to bend about the base prop and, down to prop level, it is pushed back into the retained ground.

6 ACKNOWLEDGEMENTS

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