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Observed heave and swelling beneath a deep excavation in Gault clay

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ABSTRACT: A deep excavation constructed by top-down methods in Gault clay has been carefully monitored. The lowest floor slab was cast over a void-former to allow for long-term swelling of the underlying clay, and unusually, observations were made of both movements and pore pressure changes during excavation and since construction finished in 1990. The total movement now amounts to around 110 mm of which three quarters has developed since the end of construction. Pore pressure equilibration has been comparatively quick, and the clay did not remain fully undrained during excavation. The data have been interpreted assuming swelling has been one-dimensional and show that strain is approximately linearly related to log vertical effective stress, but that the derived swell index is two to five times smaller than laboratory values.

1 INTRODUCTION

The design of a deep excavation in over-consolidated clay often includes consideration of the long-term swelling of the clay beneath the basement slab as a result of reduction of effective stress. For a piled building, this may result in the development of swelling pressures against a ground-bearing basement slab which are resisted by the sub-structure; alternatively a void may be left beneath a suspended slab. Despite the routine monitoring of numerous deep excavations in London, relatively few data on long-term swelling have been published. Notable exceptions are the observations of underground tunnels beneath the Shell Centre (Burford, 1988), and at a site at Horseferry Road (May, 1974).

The authors have recently had the opportunity to monitor the behaviour of a 10m deep excavation in Gault clay in the centre of Cambridge. A cross section is shown in Fig. 1. Whilst the site provided an unusual opportunity to measure strut loads and earth pressures at a multi-level excavation in a stiff fissured clay constructed top-down, interesting observations were also made of soil movements and pore pressure changes beneath the centre of the excavation. Excavations such as this result in immediate inward and upward movements of the clay. If excavation is rapid, the unloading is substantially undrained, and in this paper such upward movements will be termed *heave*. Subsequent movement of porewater results in reduction of effective stress, and associated upward movements will be termed *swelling*.

A preliminary appraisal of the data was given by Lings et al. (1991). Monitoring has continued since construction finished in 1990, and the results are presented here together with an examination of the development of heave and swelling.

2 GROUND CONDITIONS

2.1 *Geology*

The solid strata in the Cambridge area were deposited during the Cretaceous (Worssam and Taylor, 1969). The Lower Greensand sands were deposited in shallow water at the start of a marine transgression. Up to 50m Gault clay was then deposited in deeper water, and is generally a stiff grey highly calcareous clay or marl, often containing phosphatic nodules. Some erosion of its surface occurred before deposition of the Chalk, which elsewhere in East Anglia attains a maximum of about 400m. Uplift and denudation during the Tertiary eventually left the Gault exposed in a band some 5km wide to the north-west of Cambridge, dipping gently to the south-east. During the Pleistocene the frozen surface of the Gault was probably denuded by ice, and during interglacial periods its surface was further

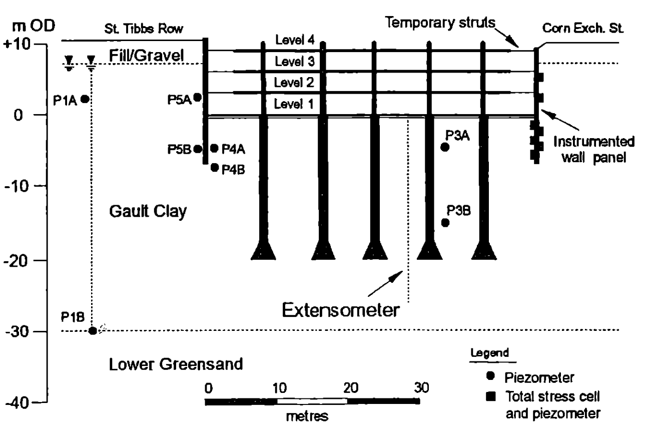

Figure 1. Cross-section through site.

eroded by the River Cam, and several gravel Terraces were formed. The site is located at one edge of the first River Terrace.

2.2 *Soil and groundwater conditions*

Ground level at the site is at approximately +10m OD. Boreholes showed that beneath some 3m of made ground and gravel, there is nearly 40m thickness of Gault clay which overlies the Lower Greensand. The Gault here is a stiff to hard grey silty clay of high plasticity. Within the depth of the excavation, the clay is closely fissured and jointed (discontinuity spacing frequently less than 25mm), and in places it is weathered and cryoturbated.

Initially the pore pressures were hydrostatic below a level of about +7m OD, influenced both by water present in the made ground and gravel just over the surface of the clay, and by water in the Greensand aquifer at depth. Rather surprisingly there were several shallow brick-lined wells extending down into the Gault clay on the site.

2.3 *Geotechnical properties*

The geotechnical parameters measured during the ground investigations are shown in Fig. 2. The site is adjacent to a previous development at Lion Yard for which extensive ground investigations had been carried out. For this development a number of pushed 100mm diameter thin-walled samples were taken from boreholes for laboratory testing, and additional in-situ testing was carried out. These tests have been compared with other similar testing of the Gault clay at Madingley (3km to the west) by Butcher and Lord (1993).

The Gault clay here is of very high plasticity over its full thickness, with average liquid and plastic limits of 77 and 32% respectively. Within the depth of the excavation the natural moisture content is at around the plastic limit, and below 10m it reduces slightly with depth; the clay fraction is in the range 40-60% (Butcher and Lord, 1993) and the carbonate content was found to be around 28%.

The stress history of the Gault clay at this site is complex. During the Cretaceous the clay was normally consolidated beneath the Chalk, under vertical effective stresses as high as 8000 kPa (as found by Samuels (1975) for specimens from the Ely-Ouse tunnel). Subsequently the stresses reduced as the Chalk was eroded, but the lateral stresses could also have been affected by tectonic movements. During the Pleistocene the top of the clay was probably frozen, and after some further erosion, deposition of the river terrace increased the vertical effective stresses by about 60 kPa.

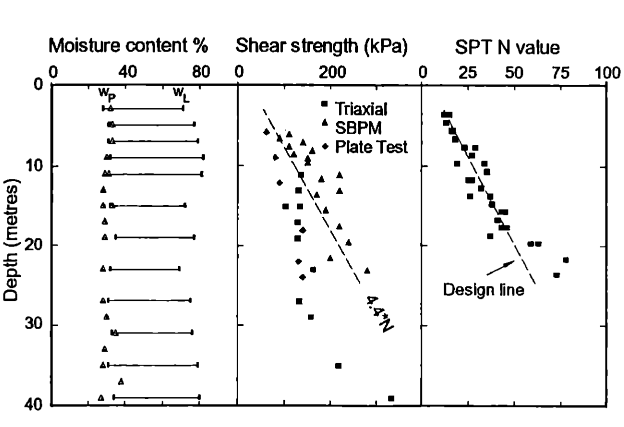

Figure 2. Geotechnical profile

3 THE STRUCTURE

The sub-structure consists of three levels of underground car park beneath a five storey hotel (see Fig. 1). The basement was constructed top-down and is approximately 65m x 45m in plan, with each floor sloping at about 1 in 30 from south to north; excavation depth varied from 9.5 to 10.5m. The adjacent ground is retained by a 17m deep perimeter diaphragm wall, and the structure is supported partly on large diameter under-reamed bored piles founded 30m below ground level, and partly on the diaphragm wall. Columns are positioned on a variable grid of approximately 8m. The car park floors which prop the diaphragm wall consist of in-situ R.C. waffle slabs supported on steel columns connected to the tops of the bored piles. The lowest floor slab is solid and was cast on a 150mm thick proprietary void-former to allow for clay swelling in the long term. The average gross loading exerted by the structure is around 140 kPa which compares with a reduction of total vertical stress due to excavation of around 190 kPa.

Soon after the diaphragm wall and bored piles had been completed there was a reduced level dig and the ground floor slab (level 4) was cast. Soil was then excavated from beneath it by mechanical plant down to the next level (level 3), and removed through an opening left in the slab. Similar operations were repeated for subsequent stages of construction until the bottom level (level 1) was reached. On two opposite sides of the site 19m long rectangular openings were left in the slabs adjacent to the diaphragm wall at the positions of the sloping vehicle ramps which allow access to the underground car park levels.

The main stages of construction are summarised in Table 1.

Table 1. Main stages of construction.

Stage	Construction operation	Period
I	Construction of diaphragm wall	18/4 - 3/6/89
II	Construction of bored piles	6/6 - 12/7/89
III	Reduced level dig to level 4	11/7 - 20/7/89
IV	Casting of level 4 slab	25/7 - 14/8/89
V	Installation of level 4 props	18/8 - 12/9/89
VI	Excavation to level 3	21/8 - 6/9/89
VII	Casting of level 3 slab	12/9 - 3/10/89
VIII	Installation of level 3 props	26/9 - 29/9/89
IX	Excavation to level 2	28/9 -13/10/89
X	Casting of level 2 slab	18/10 -3/11/89
XI	Installation of level 2 props	23/10 -2/11/89
XII	Excavation to level 1	9/11-28/11/89
XIII	Casting of level 1 slab	21/12 -30/1/90
XIV	Removal of all props	26/2 - 27/2/90

4 INSTRUMENTATION

The instrumentation installed at the site was concentrated in two main areas (see Fig. 1). A panel of the diaphragm wall adjacent to one of the ramp openings was selected for study, as the presence of temporary steel props across the openings allowed the propping loads to be measured (Lings et al. 1991, Ng 1992). Near the centre of the site, pneumatic piezometers and an extensometer were installed 8.5m apart and about 5m from adjacent piles, by Geotechnical Instruments (UK) Limited at the end of July 1989 just after the reduced level dig for level 4. In addition several tensiometers were installed following later stages of excavation to measure suctions just beneath the excavated clay surface.

The datum magnet of the extensometer was installed at -25.1m OD, a few metres above the base of the Gault clay, and arrow magnets enabled measurements to be made of movement at various elevations. To prevent soil blocking the tubing during each stage of main excavation, a bung was temporarily inserted down the tubing to below the next formation level. The upper magnets and lengths of tubing were then removed when exposed.

5 OBSERVATIONS OF MOVEMENT

Figure 3 shows the change of elevation of the magnets with time (time since 1/4/89 plotted on a log scale) and indicates that each stage of excavation was accompanied by immediate upward movement. In the weeks following each excavation stage there were further upward movements, and these are still continuing at a decreasing rate. Figure 4 shows the distribution of movement with depth at various dates. The uppermost extant magnet is about 1.3m below the excavated clay surface, so the lines have been extrapolated upwards to indicate the movement there. There would have been small unrecorded movements before the extensometer was installed due to the reduced level dig to level 4, and in the later stages of excavation small unrecorded movements occurred at the base of the Gault. There could also have been unrecorded movement of the underlying Greensand, but this is likely to have been very small.

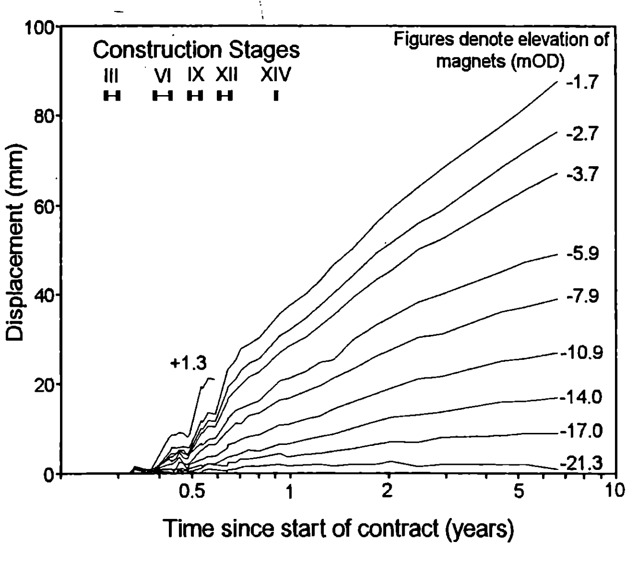

Figure 3. Movements vs time.

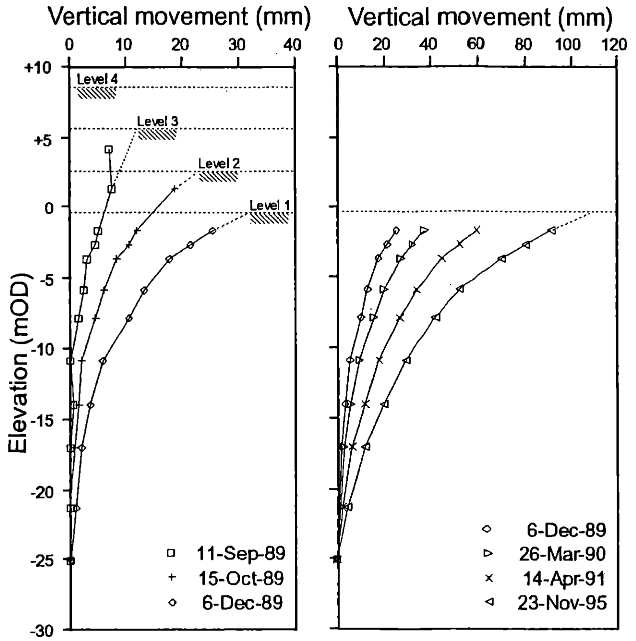

Figure 4. Movements at various dates - a) after each stage of excavation, and b) since end of excavation.

Figure 4a indicates that during the first stages of excavation, there were significant upward movements of the clay. By the end of excavation the clay formation had moved by about 32mm. These immediate movements are interpreted as heave. It is clear from Fig. 4a that these movements did not immediately extend through the full thickness of clay, and did not reach the level of the datum magnet until the last stage of excavation. This distribution implies a marked variation of undrained clay stiffness with depth, perhaps due to small strain effects, but it should also be noted that loading on the piles was increasing as construction of the hotel proceeded.

Subsequent movements are interpreted as swelling. The total movement of the clay beneath the level 1 slab now amounts to about 110 mm, nearly three-quarters of which has developed since the end of excavation. The data from the extensometer have been analysed to assess the distribution of vertical strain (slope of lines in Fig. 4 calculated from the relative movement of adjacent magnets). Strain is shown plotted against depth in Fig. 5a for various dates. Bearing in mind that the piezometric levels through much of the clay appear to have been rising fairly uniformly (see next section), the distribution of strain again implies a marked variation of stiffness in swelling with depth.

Figure 3 shows that the continuing movements plot linearly against log time which suggests the development of secondary swelling.

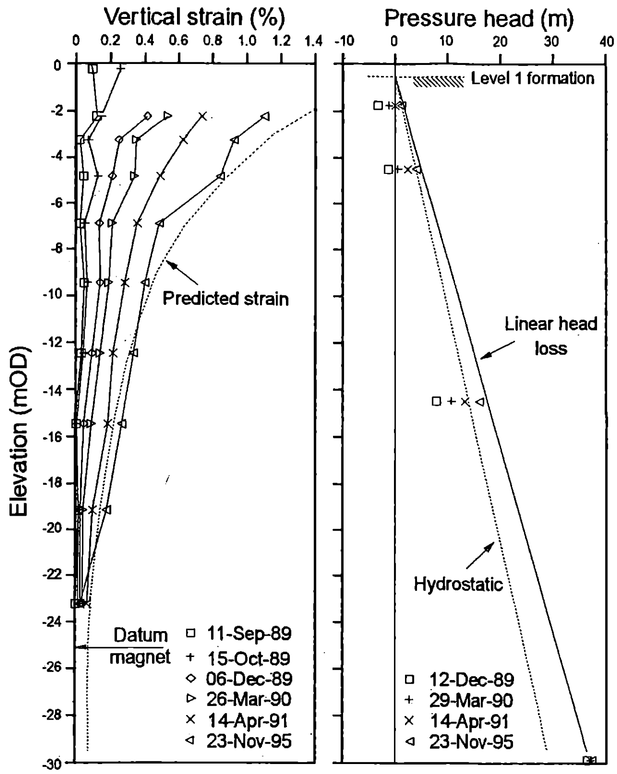

Figure 5. a) Vertical strains, and b) Pressure heads at various dates.

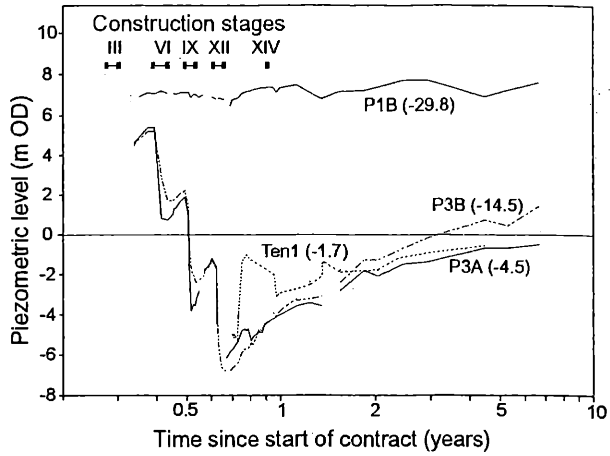

Figure 6. Piezometric levels vs time.

6 PIEZOMETRIC RESPONSE

The long-term conditions will result from seepage through the Gault clay into the void beneath the basement slab, both upwards from the Greensand and inwards from the fill and gravel outside the site. Prior to construction, these were predicted on the assumption that the clay had uniform permeability, and that the piezometric level in the Greensand would remain unaltered. The flow net indicated that seepage beneath the centre of the excavation would be essentially one-dimensional, and would not be affected significantly by flow from the fill and gravel.

The piezometers mounted on the diaphragm wall showed that before bulk excavation, the piezometric level inside and outside the box was between +6m and +7m OD (Lings et al. 1991). The subsequent variations of piezometric level beneath the centre of the site with time are shown in Fig. 6. During construction there were small variations in the piezometric level in the Lower Greensand (see P1B curve) which were probably unrelated to the construction activity, but overall it remained approximately constant at about +7m OD.

6.1 Response during excavation

Pore pressure changes in the clay resulted both from the stress changes due to construction and from the dissipation of negative excess pore pressures during the transition to steady-state seepage. The central piezometers were installed soon after 1 to 2m of soil had been excavated for construction of the level 4 slab. Figure 6 shows that the piezometric level there (P3A, P3B) had reduced by 2 to 3m, but was rising quickly. During each subsequent stage of excavation substantial head reductions occurred beneath the centre of the site as 3m soil was removed, but piezometric levels started to recover immediately afterwards.

One tensiometer was installed beside the extensometer with its tip 1.3m below the clay surface, and Fig. 6 shows the observations plotted as piezometric level against time. Apart from a period early on when the instrument needed de-airing, its response of gradually increasing pressure is consistent with that of P3A several metres below it. Immediately after the final stage of excavation, suctions developed in the clay adjacent to P3A; suctions are difficult to measure with pneumatic piezometers, and although a special procedure was developed, these observations are probably not particularly accurate.

Each stage of excavation appears to have been sufficiently rapid that beneath the excavation the clay was substantially undrained during unloading. Values of $\overline{B}=\Delta u/\Delta\sigma_v$ have been calculated for piezometers P3A and P3B for each excavation stage assuming a constant reduction of vertical total stress with depth, and are given in Table 2.

Table 2 - Values of $\overline{B}$ for each excavation stage

Excavation stage	$\Delta\sigma_v$ (kPa)	$\overline{B}$ for P3A (-4.5mOD)	$\overline{B}$ for P3B (-14.5mOD)
Excavation to Level 3	63	0.73	0.55
Excavation to Level 2	63	0.79	0.71
Excavation to Level 1	63	0.73	0.86
Notes : $\Delta\sigma_v$ includes 5 kPa allowance for overlying concrete slab. Pore pressure went negative in P3A during excavation to level 1.			

Values of $\overline{B}$ increased for successive excavation stages as the piezometers were buried less deeply, until the pore pressures went negative in the upper layers of clay at the last stage of excavation. The tensiometers confirmed this, generally registering suctions of less than 50 kPa just below the excavated surface, which implies $\overline{B}$ of only about 0.5 there. At this stage it is possible that there was cavitation within the fissures which limited the suction.

The rapid increase of pore pressures in the clay beneath the centre of the site after each stage of excavation was surprising. A small proportion of the rise of pore pressures (say 5kPa) is attributable to the undrained response to casting the concrete slabs over the excavated formation. The sloping excavation surface was mostly dry, and although some water may have been available both from rainfall (through the temporary access hole in the slabs), from site activities, and locally from an ungrouted borehole (Nash et al. 1996), the surface may be considered to have been initially impermeable.

At an impermeable boundary of a swelling clay layer the gradient of excess pore pressure is zero. Within a few weeks the rising piezometric level was indeed approximately constant with depth, so the surface clay layers were clearly sufficiently permeable to allow a rapid equalisation of recorded piezometric level with depth. Classical swelling theory (assuming constant coefficient of consolidation c_v) predicts no immediate change of excess pore pressure, which is at variance with the observations. However an analysis which takes account of stiffness which reduces with effective stress during swelling predicts substantial pore pressure changes throughout the clay layer proportionately sooner after unloading. A better match with the field data might also be obtained if the permeability were reduced with depth.

6.2 *Post construction response*

Since the end of basement construction the piezometric levels within the site have risen gradually in a manner which is consistent with the transition to steady-state seepage through the clay. The pressures observed on various dates are plotted against depth in Fig. 5b, which shows that at the time the lowest slab was cast, the distribution in the upper half of the clay was approximately parallel to the hydrostatic line. Subsequently the piezometers indicate a slight excess head at depth, consistent with upward seepage. The long-term distribution predicted assuming linear variation of head with depth shown in Fig. 5b fits the most recent piezometric data well, and conditions appear to have nearly stabilised.

7 DISCUSSION

7.1 *Correlation of piezometer and extensometer data*

Since the piezometers, one tensiometer and the extensometer were all in close proximity, it is possible to examine the relation between observed vertical strain and effective stress change. Figure 7 shows strains deduced from pairs of magnets at similar elevations to the piezometers and tensiometer,

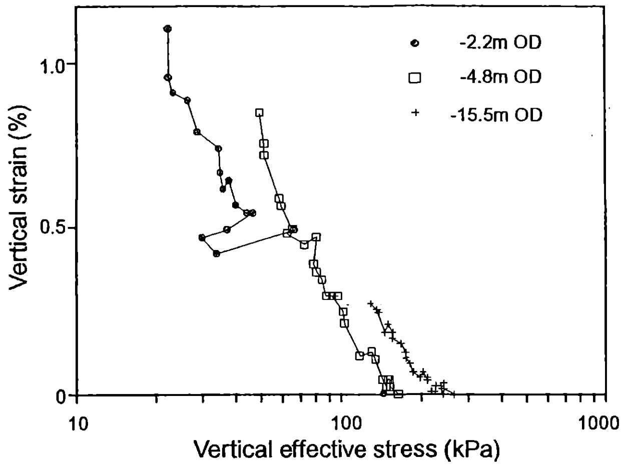

Figure 7. Vertical strain vs vertical effective stress.

plotted against vertical effective stress for the whole period. In calculating effective stress it was assumed that the total vertical stress was due to the weight of the overlying soil and was not affected by the piles.

Figure 7 shows that the strain varies approximately linearly with $\log\sigma_v'$, with average slopes of 0.010 to 0.015 strain/$\log\sigma_v'$. If the strain is assumed to be entirely one-dimensional swelling, the slope is equivalent to $C_s/(1+e_0)$. Assuming an average value of e_0 of 0.8 (consistent with moisture content of about 30%), this implies a swell index of C_s of 0.018 to 0.027. The implied coefficient of volumetric expansion m_s is inversely proportional to stress level. The slope is least for the deepest level, which probably reflects the overestimate of effective stress change arising from ignoring the effect of the piles. The slope is greatest for recent data from the upper levels, behaviour which is consistent with secondary swelling.

7.2 *Comparison with laboratory swelling behaviour*

It is of interest to compare the field behaviour presented above with laboratory oedometer tests carried out on the clay. Prior to construction, four tests were carried out on specimens taken from the thin-walled samples. During excavation a number of block samples were taken from which additional specimens were prepared (Ng and Nash, 1995). In these tests the specimens were loaded up to about 1600 kPa and then unloaded in steps to low stresses. The swell index C_s was determined from the rebound data, and for both sets of tests was found to lie in the range 0.05-0.09 with $C_s/(1+e_0)$ of 0.03-0.05, that is two to five times greater than that observed in the field. The swell index for reconstituted Gault clay was found to be significantly greater than that for the natural soil at a similar OCR (Ng and Nash, 1995).

Burland (1990) presented tests on Gault clay by Samuels (1975) and showed that the swell index for tests in which swelling was initially prevented and which were then unloaded, was smaller that that measured on swelling after loading up to high effective stresses. Tests on reconstituted Gault clay also showed significantly greater swell index. He suggested that the presence of fabric and bonding in the natural soil inhibit swelling, and that these are at least partially destroyed by loading to high stresses as was done for the Lion Yard specimens.

In an oedometer test even with perfect fit, it is not possible to restore the in-situ stresses on the specimen prior to loading or swelling, so the stress path in a laboratory test cannot match that in the field. It may well be that the strains involved in sampling, followed by an incorrect stress path, mean that the results of such tests cannot be representative of field conditions.

7.3 Prediction of long-term conditions

One method of predicting the long-term distribution of strain would be to assume the behaviour could be approximated to one-dimensional swelling. Such a calculation has been carried out using a single average value of $C_s/(1+e_0) = 0.013$ over the full depth of clay, and the result is shown in Fig. 5a. The reduction of total vertical stress of 190 kPa due to the excavation was assumed to extend to 5m below the clay surface (-5m OD). To take some account of reloading from the building of 140 kPa which is transferred into the ground by the piles, the stress reduction of 190 kPa was reduced linearly below -5m OD to reach 50 kPa at -25m OD. The calculated distribution of strain was based on the linear variation of pore pressure with depth shown in Fig. 5b, and it may be seen that the calculated strains are broadly consistent with the present values, bearing in mind that equilibrium has not yet been reached. The total movement of the excavated surface implied by this calculation is predicted to be 135mm. The development of secondary swelling could well result in larger movements.

7.4 *Comparison with other sites*

The rapid equilibration of pore pressures at this site contrasts with the behaviour beneath excavations in London clay for example at the underground car park at the House of Commons, and at the Shell Centre, and also beneath many cuttings in London clay. Chinsman (1972) reported rapid equilibration of pore pressures at an excavation in Gault clay in Kent, and Chandler (1984) drew attention to the different rates of equilibration at sites in Gault and London clays.

8 CONCLUSIONS

Data on both movements and the rise of pore pressures after excavation have been presented. Both are still continuing at decreasing rates, and there are now signs of secondary swelling. The distribution of pore pressures appears to be consistent with the transition to long-term upward seepage from the Greensand aquifer. Large strains were observed at shallow depths which are consistent with a logarithmic stress strain relationship, and it has been possible to relate strains to changes of effective stress and to derive a field value of the swell index C_s. The observed strains show reasonable agreement with the predicted long-term distribution using this value. The field swell index is two to five times smaller than the value obtained from laboratory one-dimensional swell tests. The rapid rise of pore pressures after excavation implies that during construction the clay did not remain fully undrained. This has implications for the design of embedded walls.

9 ACKNOWLEDGEMENTS

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