Effects on diaphragm walls of groundwater pressure rising in clays

Levée de la pression interstitielle dans l'argile: Effets sur les écrans d'étanchéité

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SYNOPSIS: Data are presented of the behaviour of model walls in overconsolidated clay, tested in a centrifuge at 125 g. The removal in flight of a heavy fluid in front of the model wall was used to simulate excavation in front of a prototype in-situ construction. Measurements of bending moment, prop force, wall deflection and ground movement were made as pore pressure transducers responded to imposed variations in groundwater conditions. In particular two walls, one unpropped and one propped, are investigated. The consequential deformations of the unpropped wall, and the bending moments in the propped wall, are described and discussed in relation to the anticipated behaviour of a full-scale construction.

INTRODUCTION

Concern is now being expressed over the possible consequences of groundwater pressures rising in clays, especially in urban areas where the reduction of industrial activity has led to a decrease in abstraction from deep aquifers. A particular cause for concern relates to walls retaining deep basements or cuttings for roads and railways. Where such walls have been constructed in-situ as diaphragm or secant pile walls, they will usually have been designed either using conventional active and passive earth pressure coefficients or by the use of some effective "at rest" earth pressure coefficient together with an assumed water level. In either case the raising of water levels may lead to lateral pressures in the retained clay which exceed design values, partly due to the extra pore pressure, and partly due to the increase in the effective earth pressure coefficient following swelling. If the structural system is very stiff these increased stresses will create increased bending moments and propping forces. These extra stresses can be relaxed only if the wall can be permitted to deflect or yield. There is therefore a danger of violating either strength or displacement criteria.

CENTRIFUGE TESTS

In order to address the problem of rising groundwater pressures in a sufficiently short time-scale it is necessary to investigate the behaviour of a 1/n scale model under n "gravities" of centrifugal acceleration. Times for swelling and softening would then be reduced by n^2, if other soil parameters (permeabiltiy, coefficient of volume expansion, angle of dilation, angle of shearing resistance, unit weight) varied correctly as the effective stresses achieved full scale values. This is precisely what the centrifuge test offers.

This paper describes two of the 22 model tests on in-situ walls carried out on the Cambridge Geotechnical Centrifuge. Both walls were embedded in overconsolidated kaolin clay and tested in plane strain as shown in figure 1. The preparation of the clay consisted of one-dimensional consolidation from a slurry to a vertical effective stress of 1250 kN/m², followed by relaxation to 400 kN/m². The clay block was then trimmed to receive the wall, and to create the required excavation. A rubber bag placed in the excavation was filled with zinc chloride solution of the same density as the kaolin. The evacuation of the heavy fluid would later simulate excavation in flight.

Each model was instrumented with internal pore pressure transducers, and displacement transducers measuring the settlement of the retained soil and the movement of the retaining wall. An array of markers on the cross-section could also be measured photographically (Powrie 1986).

All the model walls retained a clay face initially 80 mm high and 150 mm wide, with a permeable sheet placed over a rigid base 257 mm below the retained surface. Each was tested at 125 g, representing a prototype wall retaining 10 m of a 32 m kaolin stratum, employing walls of various penetration and propping conditions. All lengths and movements will be reported in prototype terms using a scale factor of 125. One hour of testing corresponds to 1.8 years at full scale. Equilibration under a small increment of base water pressure was typically 90% complete beneath the excavation in a time of 3 hours, indicating a coefficient of transient swelling $c_{vs} \approx 2.5$ mm²/s. A similar degree of equilibration in kaolin at full scale would take over 5 years, and if clays in general were considered to have in-situ permeabilities ten times smaller than kaolin (10^{-9} m/s) the delay would be proportionately longer.

UNPROPPED WALL: EXCAVATION PHASE

In test DWC22 the model of a 30 m wall of bending stiffness 2.9×10^6 kNm²/m was first brought into approximate equilibrium with the zinc chloride in place. The groundwater regime was established by feeding water into the base acquifer at the piezometric level of the excavation. A flownet suggested upward flow, satisfying suctions at the ground surface of 125 kN/m².

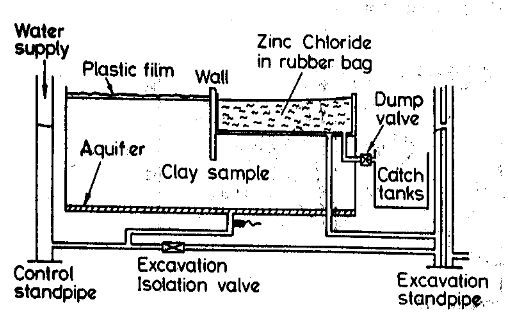

Figure 1 Arrangement of centrifuge models

The excavation of 10 m of soil was simulated in a scaled-up time of 1 month. The immediate soil settlements measured by the displacement transducers increased to a maximum value of about 0.16 mm near the wall, which showed a similar outward movement at its crest. At full scale this corresponds to only 20 mm of movement. Unfortunately, such small movements were at the limit of resolution for film measurement so the photographic evidence of internal deformations can not be shown. Experience showed that the mode of deformation was mainly a function of model geometry. The much larger movements shown in figure 2 refer to a similar model, DWC 08, in which the water table was at the ground surface prior to excavation. This pattern can be compared favourably with the simplified displacement field of figure 3 in which the wall is supposed to rotate by an amount $\delta\vartheta$ about its toe, causing simple shear of 45° triangles of soil, with a shear strain increment $\delta\gamma = 2\delta\vartheta$.

Shear at 45° is consistent with vertical and horizontal principal directions. Figure 4 is a simplified equilibrium stress field constructed on this basis, for a frictionless wall which is taken to be pinned at its base. In reality, such a wall would rotate about a point just above its base so that the concentrated force Q would be replaced by an intense zone of passive pressure acting from the retained side. Lateral stresses prior to excavation correspond to K = 1. In the field, this would be due to the pressure of wet concrete during construction of the wall. In the centrifuge it is due to the fluid pressure of zinc chloride and the assumption that similar conditions will occur beneath. On the undrained removal of the zinc chloride the lateral stress beneath the excavation would fall by γh in the absence of wall movement. If an undrained wall rotation is now assumed, it is consistent to invoke a mobilized undrained strength c_{mob}, taken to be constant in figure 4, which permits the lateral stresses to change by $2c_{mob}$ towards "active" and "passive" conditions respectively on the retained and excavated sides. The soil is taken to crack in tension.

Moment equilibrium about the toe of the wall in figure 4 leads to the relationship in figure 5 between the mobilized strength index $c_{mob}/\gamma h$ and the penetration ratio d/h. It will be seen that for d/h = z corresponding to the model, $c_{mob}/\gamma h = 0.28$ so that $c_{mob} = 48$ kN/m².

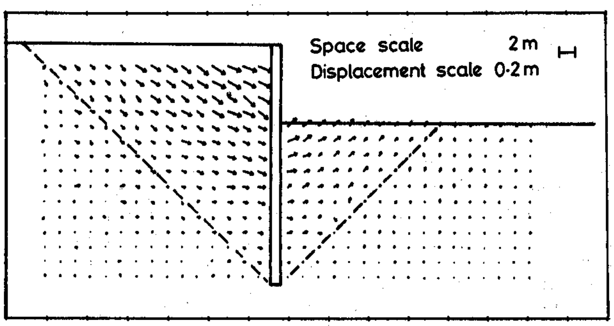

Figure 2 Displacement pattern for prototype of DWC 08 after excavation

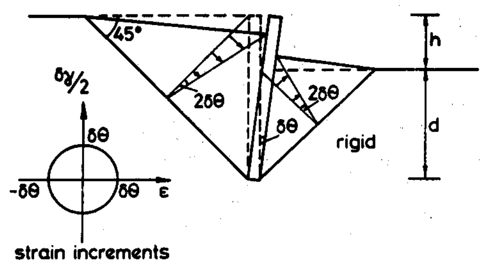

Figure 3 Idealised strain field for undrained shear

For the wall of test DWC 22, the undrained shear strength on excavation was estimated, from the known stress history of the sample in relation to the results of various triaxial tests, to vary approximately linearly from 65 to 140 kN/m² on the retained side, and beneath the excavation from 100 to 140 kN/m². For the purpose of this simple analysis the soil was taken to have a uniform strength of 113 kN/m², taking the average for soil in contact with the wall. This leads to an estimate of $c_{mob}/c_u = 48/113 = 0.42$.

Figure 6 shows curves of c_{mob}/c_u versus log γ obtained from a typical plane compression test on kaolin (Powrie 1986). It will be seen that $c_{mob}/c_u = 0.42$ requires $\delta\gamma$ of about 0.15%. This would lead to a wall rotation of 0.075% according to figure 3, which would imply a crest movement of 22.5 mm at full scale in comparison with a scaled observation of 20 mm. The analysis is apparently justifiable.

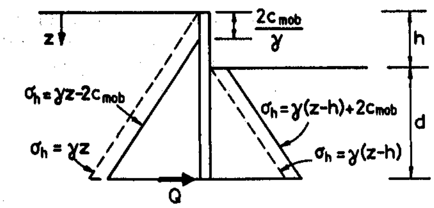

Figure 4 Idealised stress field after excavation

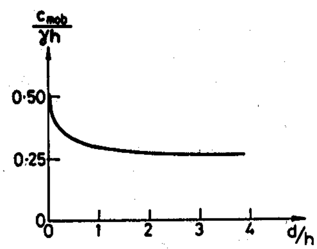

Figure 5 Variation of mobilised strength with penetration ratio

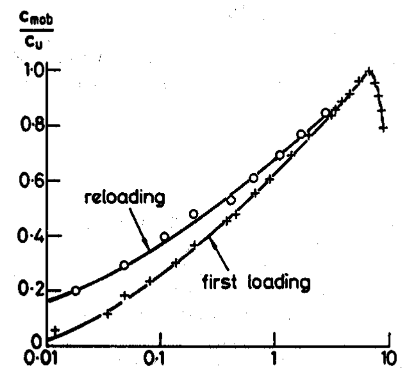

Figure 6 Proportional mobilized strength versus log shear strain from undrained plane compression test

UNPROPPED WALL: PHASE OF GROUNDWATER RAISING

With the dissipation of the excess pore water suctions induced on excavation, the deflection at the crest of the wall gradually increased to about 126 mm after 12.4 years at prototype scale, while the piezometric level in the underlying acquifer remained at the level of the excavation. On raising the water pressure at the bottom boundary, pore water pressure throughout the model began to rise and the rate of movement of the wall increased significantly as shown in figure 7. Pressures were raised in this way twice more during the test, leading both to swelling and to shear deformation as shown in figure 8. This shows the increments of soil displacement from excavation up to equilibrium with the piezometric head of the aquifer raised by 10 m to the retained surface.

The ultimate consequences of 10 m of groundwater recovery in kaolin are seen to be 800 mm of heave in the excavation and a wall rotation of 0.017 radians leading to a crest deflection of 500 mm albeit with negligible soil settlement. Generalisation of this result must presently be hypothetical. Kaolin's plasticity index is 40%, and the slope κ of its rebound curve on an e - lnp' diagram is about 0.05. Movements might crudely be assumed to be proportional to either of these indices, and to be proportional to the average reduction in the logarithm of effective stress brought about by recharge.

Analytical prediction of such events would entail the prior acquisition of stress-strain data from tests with a variety of stress paths, including shear tests with increasing back pressure which follow both active and passive limit lines; (Burland and Fourie 1985). Approximate methods following the style of figure 3 but in terms of effective stresses and the mobilization of angles of shearing might then be possible; (Powrie 1986).

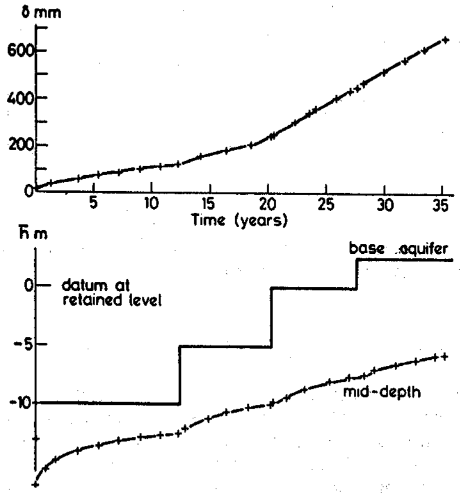

Figure 7 Long term crest deflection δ and piezometric head $\bar{h}$ for prototype of DWC 22

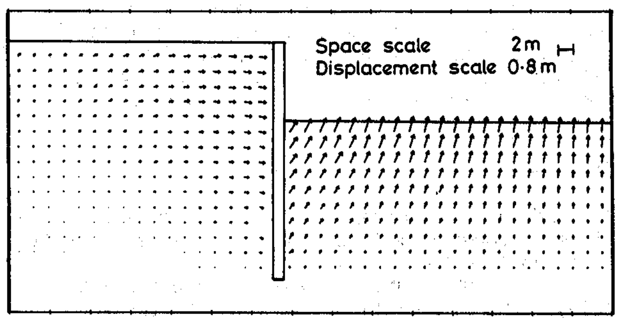

Figure 8 Displacement pattern for prototype of DWC 22 after 10 m of groundwater raising

PROPPED WALL: EXCAVATION PHASE

In test DWC 21 the model of a 15 m deep in-situ wall of bending stiffness 1.2×10^6 kNm²/m was first brought approximately into equilibrium with the piezometric head of the 32 m deep aquifer set at 8 m below ground level. The surface suctions were larger in this case - probably of the order of 220 kN/m². The extra surface suction, together with the reduced wall depth, led to an estimated initial undrained strength profile which was nearly constant at 113 ± 2 kN/m² for the soil in contact with the wall.

In this experiment, a pair of tubular props had been inserted beneath the rubber bag, at the level of the excavation. Prop forces and wall bending moments were monitored as the model was taken through a similar sequence to that of DWC 22.

On "excavation" a small forward movement of the wall crest was observed, of the order of 0.14 mm in the model or 8 mm at prototype scale. This was interpreted as due to the closure of a small gap between the wall and the props since the propping force remained constant while the bending moments were changing. During "excavation", therefore, the wall was analysed as unpropped, and figure 5 should apply. At d/h = 0.5 we read that $c_{mob}/\gamma h = 0.32$ is required for equilibrium. This gives c_{mob} = 56 kN/m², so that $c_{mob}/c_u = 56/113 = 0.5$. Figure 6 would then indicate a mobilised shear strain of about 0.3%. According to figure 3, the wall rotation should have been 0.15% radians which would have generated 22.5 mm crest deflection full scale, or 0.18 mm at model scale which compares favourably with the observation of 0.14 mm.

The mobilization of c_{mob} = 56 kN/m² would leave a tension crack open to a depth of 6.4 m: it follows that bending moments immediately after excavation are rather small. Figure 9 compares the measured values with a prediction based on the method of figure 4, but for a wall of 15 m overall depth, and mobilizing 56 kN/m² of shear strength. It will be seen that the bending moments and propping forces at the completion of excavation were too small for accurate measurement.

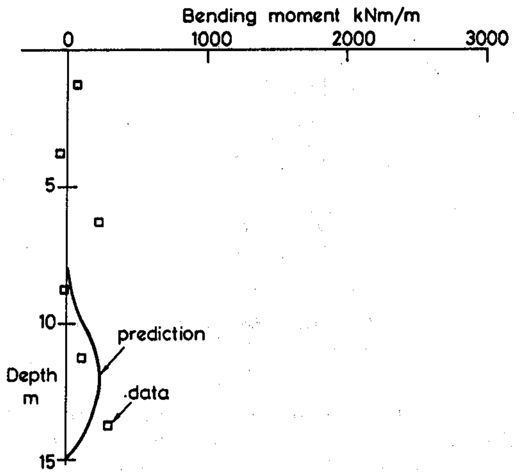

Figure 9 Bending moments for prototype of DWC 21 after excavation

PROPPED WALL: PHASE OF GROUNDWATER RAISING

The props began to take up load soon after excavation; see figure 10. Just after the first base pressure increment it proved necessary to stop the centrifuge for a short time, but this was not felt to have caused any major disturbance in the long-term (Stewart 1986). The pore pressure transducer at mid-depth was brought approximately into equilibrium after each subsequent increment.

Back analysis of the states of equilibrium of the wall showed that retaining pressures soon surpassed the active minimum. The possible situation after 12 m of groundwater rise is sketched in figure 11. Active and passive total stresses are constructed using $\varphi' = 21.6°$ for the kaolin and taking account of extrapolated surface suctions; they are compared with stress distributions based on an earth pressure coefficient of unity. Figure 12 compares the

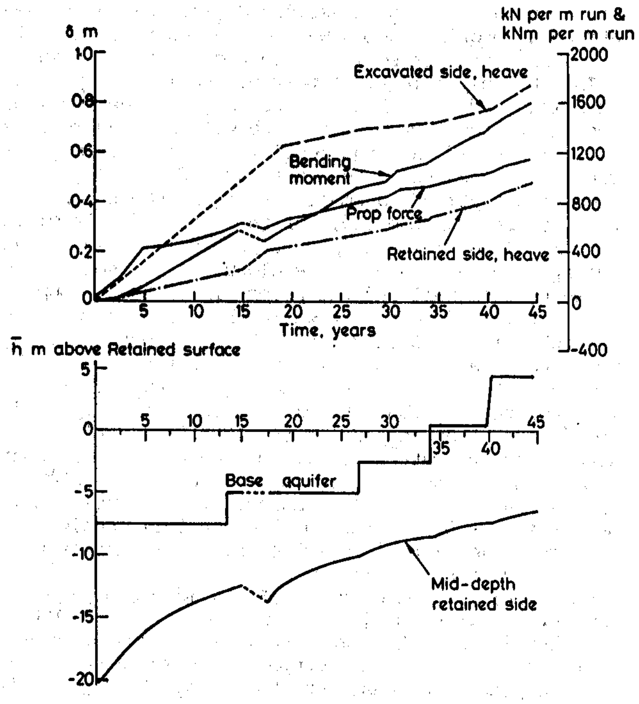

Figure 10 Long term heave and piezometric head for prototype of DWC 21

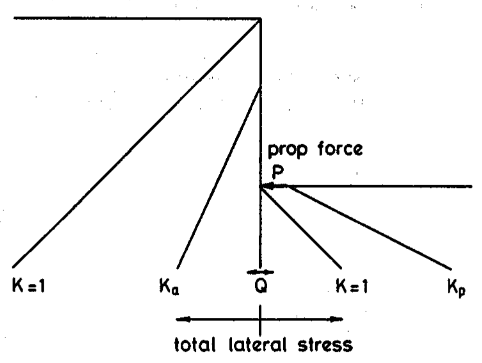

Figure 11 Trial stress distributions for equilibrium after 10 m of groundwater raising

alternative bending moment distributions with the measured performance. Evidently, the earth pressure coefficients after 12 m of base pressure increase were close to unity.

The inability of DWC 21 to mobilise its drained shear strength in the long term is consistent with the soil displacement field recorded for 12 m of drained base pressure rise - see figure 13. It will be observed that the heave in the excavation was about 800 mm full-scale, as with the unpropped wall in DWC 22. In this case, however, there is heave on the retained side of 325 mm and a forward wall movement of only 125 mm full-scale. In comparison with the unpropped wall, the situation is much more akin to swelling in an oedometer, and earth pressure coefficients must be expected to conform to this case.

CONCLUSIONS

When the water table rises in clay, swelling occurs. If an embedded retaining wall is stiff and well propped, the earth pressure coefficients will mimic those for oedometer unloading, (Schmidt 1966). In the model a 12 m rise of groundwater head brought about earth pressure coefficients of close to unity. Larger coefficients could have been generated

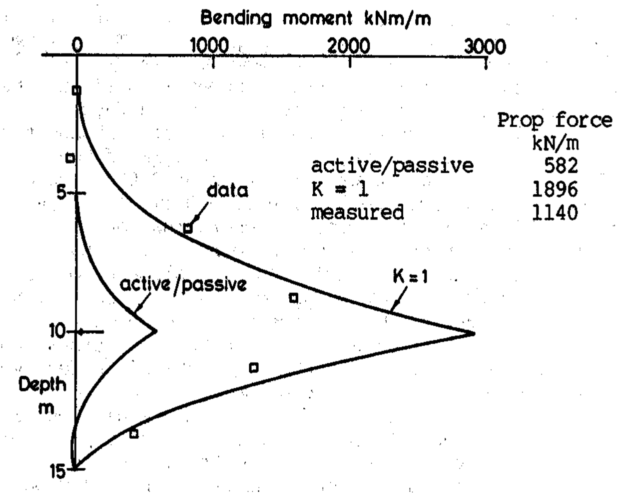

Figure 12 Bending moments for prototype of DWC 21 after 10 m of groundwater raising

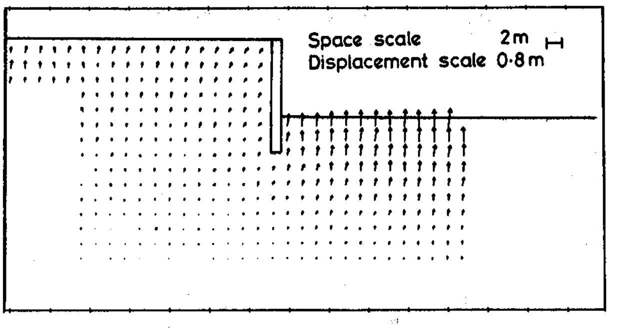

Figure 13 Displacement pattern for prototype of DWC 21 after 10 m of groundwater raising

by larger proportional reductions in vertical effective stress.

The greatest soil movements are measured where the greatest proportional effective stress reductions are felt. Heave in excavations will provide the best indication of water pressure rise in deep aquifers.

If, on the other hand, the wall is a simple unpropped cantilever, it will rotate with increasing severity as the pore water pressure rises. The potential for soil swelling on the retained side is converted into an ability to shear and dilate. Sudden collapse is unlikely if water is kept out of tension cracks. The problem may be experienced as a progressive outward wall rotation with sub-horizontal movement of the retained soil, accompanying the inevitable heave of soil in the excavation.

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