Pressures on a Retaining Wall by an Expansive Clay

Pressions sur un Mur de Soutènement par une Argile Expansive

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SYNOPSIS A reinforced concrete retaining wall, 7.5 m deep, in the basement of the Gouger Street Mail Exchange, Adelaide, South Australia, supports stiff Hindmarsh clay over most of its depth. Earth pressures on the wall and soil suctions in the clay have been monitored for several years. The initial earth pressures were zero. Subsequently a pressure increase occurred at the base of the wall and moved progressively up the wall with time reaching a maximum value of five times the overburden pressure.

A theoretical model using the finite element method of solution with non-linear (hyperbolic) material parameters (previously developed to analyse the load-moisture-displacement response of expansive clays) was used to check the proposed mechanism of behaviour of the wall. The model gave earth pressures on the wall comparable in both magnitude and distribution to those observed. While the interaction problem was more complex than the model used, the results suggested that the interfacial conditions between the wall and clay, the movement of the wall and the magnitude and extent of the swelling of the clay may be significant factors.

INTRODUCTION

With the development of theoretical models which can successfully analyse the immediate and long term behaviour of expansive clay in laboratory test procedures, it is now feasible to extend their use to practical field problems. One problem, which was suitable for analysis and has already been well documented, was the 7.5 m deep basement of the Gouger Street Mail Exchange, Adelaide, South Australia, which required the construction of a reinforced concrete retaining wall (Richards & Kurzeme, 1973). This retaining wall supported stiff Hindmarsh clay over most of its depth. This clay has been well documented previously and shown to be a highly expansive fissured clay, associated with many structural problems in the Adelaide area (Stapeldon, 1970). As free water was observed flowing from the clay during the excavation stage, there was concern that subsequent wetting and heaving of the clay might cause large pressures on the wall.

In order to investigate the behaviour of the wall, earth pressures on the wall and soil suctions in the clay were monitored. These measurements over several years have shown that while the initial earth pressures were negligible, the earth pressures have increased at the bottom of the wall to a value of at least five times the overburden pressure. This pressure increase moves progressively up the wall with time. At the same time, the soil suction measurements, the nearest being 2 m from the wall, have shown no significant change. The most likely cause of this pressure increase was thought to be the accumulation of seepage water in the initial gap between the wall and the clay, followed by the local swelling of the clay. When the clay made contact with the wall, this water would no longer be readily available to the clay at that depth. The earth pressures would then only increase until there was an internal redistribution of soil suction within the fabric of clay.

The theoretical model used in this paper was developed by the author (Richards 1976a) using the basic theory proposed by Livneh et al. (1973). It uses the finite element method of solution for boundary problems with incremental changes in loads, displacements and soil suction. The material parameters are expressed in non-linear (hyperbolic) form as functions of stress and soil suction (Richards, 1976b). During the above investigation, extensive 'intact' sampling and laboratory testing of the clay was carried out. The testing program included a range of triaxial test in which various stress paths with soil suction measurement were followed and volume change measurements were conducted under controlled soil suction. This paper shows how the test data obtained can provide the necessary material parameters for the application of the theoretical model to the problem. While the exact transient soil water conditions were unknown, it was possible to test the hypothesis stated above and demonstrate the relevance of the model to structure - expansive clay interaction problems in general.

DETAILS OF STRUCTURE

The Gouger Street Mail Exchange is a steel-framed building with a reinforced concrete basement structure founded in Hindmarsh clay. Construction of the basement involved excavation to a depth of approximately 7.5 m and the erection of heavy temporary shoring before the permanent basement wall was cast.

For the temporary shoring, vertical taper flange beams (610 x 190 mm) were placed in 0.9 m dia. bored holes, spaced at approx. 2.5 m and surrounded by concrete. The intervening spaces were spanned by reinforced concrete panels 0.2 m thick. The front of the shoring presented a smooth face against which the permanent retaining wall was cast. The temporary shoring remained in place after construction of the 0.3 m thick reinforced concrete basement retaining wall.

The Hindmarsh clay allowed a vertical smooth, free standing face to be excavated to depths of at least 7 metres. The initial pressures on the wall were expected to be low, if not zero as confirmed by the subsequent observations. However, as pointed out above, water was encountered during both sampling and excavation, so both creep and expansion of the clay was considered likely, leading to the long term build up of earth pressures.

SAMPLING AND INSTRUMENTATION

Thin walled and split tube samples of 50 mm dia. were taken behind the retaining wall using a "Proline" drilling rig. Soil suction was monitored by means of psychrometers installed in eight holes behind the wall at depths of 2, 4, 5 and 7.5 m. The psychrometers were developed and constructed by the CSIRO Division of Applied Geomechanics (Richards, 1971).

Twentyfour earth pressure cells operating on the Carlson principle were cast into the retaining wall with the sensitive diaphram preloaded against the exposed clay face. These cells were located at six vertical sections along the length of the wall at depths of approximately 2, 4, 6 and 7 metres.

Full details and location of sampling and instrumentation has been described in Richards and Kurzeme (1973).

SOIL DESCRIPTION

An examination of the borehole logs and the soil samples examined in the laboratory clearly showed four separate and distinct soil types.

The section parallel and immediately behind the wall indicating the relative positions of these soils shows the soil conditions were reasonably uniform over the 25 m length of the test site wall. The surface consisted of 1 to 2 m of rubble, calcrete and clay fill, overlying about 2 m of marl over slightly fissured Hindmarsh clay to a depth of about 10 m. The floor level of the basement was slightly below the mid-height of the Hindmarsh clay layer. The retaining structure was exposed to the Hindmarsh clay over a depth of approximately 5 m.

LABORATORY TESTING OF SAMPLES

1. Stress Controlled Undrained Triaxial Tests

Samples were extruded from the 50 mm dia. sample tubes and trimmed to triaxial specimens, 50 mm dia. by 75 mm long. The specimens were then set up in a conventional triaxial cell in a pneumatically operated stress controlled loading frame and preloaded until vertical displacements were constant over a 5 min. period.

Three types of stress paths were followed.

Firstly, constant stress ratio tests with $\sigma_3/\sigma_1 = 1.0$, 0.75 and 0.5 respectively were attempted. The initial preload used in these tests was σ_1 equal to 69 kPa and the appropriate σ_3. The stress ratio of 1.0 did not give sufficient volume change to be measured accurately. The stress ratio of 0.5 caused almost immediate failure. Consequently the stress ratio of 0.75 gave the only useful results. In all cases the vertical stress was increased in increments of 69 kPa (and the appropriate stress laterally) to the assumed overburden stresses for the depth of the sample, unloaded and then reloaded to failure.

The second type of stress path was used in a series of tests in which the octahedral normal stress was held constant at one of four levels, viz. 69, 138, 207 and 276 kPa. The sample was first loaded with an all-round pressure equal to the octahedral normal stress chosen. The vertical stress was then increased incrementally to failure while the lateral stress was reduced to maintain constant octahedral normal stress.

The third type of stress path was used on a selection of test specimens representative of the material types, viz. constant cell pressure tests with the cell pressure equal to 69, 138, 207, 276 kPa. Some specimens were tested as sampled and others after being equilibrated in a pressure membrane apparatus with air pressures controlled at 35, 105, 350, 1050 and 3500 kPa.

Volume change measurements were obtained for the triaxial tests, enabling both tangential and secant moduli and Poisson's ratio to be determined at each stress level for each specimen. The soil suction of the specimens was determined as the mean of nine psychrometer determinations throughout the specimen at the completion of the test.

2. Moisture Distribution at Time of Sampling

The initial soil suction data were measured psychrometrically on samples taken at intervals of 300 mm from the split tube samples.

3. Volume Change Tests

Five specimens 50 mm dia. by 25 mm high were prepared from the 50 mm dia. thin walled tube sample. They were immediately weighed, their dimensions measured at a number of points and then placed in a pressure membrane with air pressures controlled at five different levels viz. 35, 105, 350, 1050 and 3500 kPa.

At monthly intervals, the specimens were weighed and their dimensions checked. When no change was recorded between two successive monthly measurements, the specimens were oven-dried and re-weighed. The results were plotted as vertical, lateral and volume strains versus the logarithm of the final measured suction, e.g. Fig. 1. The slope of the straight line fitted through these points gave coefficients of strain in two dimensions, CC_1 and CC_2 and the volume change, CC_v respectively.

RESULTS OF THE FIELD INSTRUMENTATION

The earth pressure cell observations were taken on each cell from the time of installation in August 1971 to July 1975. As expected the initial readings indicated very low pressures on the wall over the whole depth. However, at each of the six vertical sections, the lowest earth pressure cell showed an immediate progressive increase in pressure with time as indicated by the examples in Fig. 2. A general progressive increase in pressure of the second lowest row of cells also commenced many months after installation, followed by the third lowest row, 18 to 24 months after. The top row which is at the level of the marl, has shown no increase in pressure as at February, 1976.

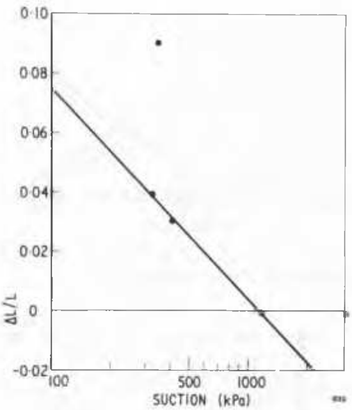

FIG. 1. TYPICAL SUCTION - VERTICAL STRAIN RELATIONSHIP FOR HINDMARSH CLAY

Psychrometer observations taken at the same intervals of time have shown no significant changes of soil suction in the Hindmarsh clay over the same time period. Small fluctuations were observed at locations in the marl, where seepage was detected during installation.

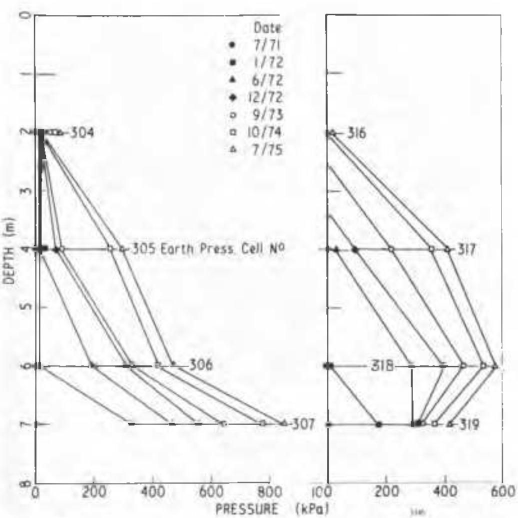

FIG. 2. VERTICAL PRESSURE DISTRIBUTIONS FOR CELLS 304 - 307 and 316 - 319.

THEORETICAL MODEL

The theoretical model employs an incremental iterative finite element method of solution for materials with non-linear elastic parameters subject to both initial volume strains and the Mohr-Coulomb yield criterion. This method considers a number of increments of suction and load from the initial soil suction and stress state to some final state. For each increment, two stages are considered. First, the initial strains arising from the volume change due to the incremental soil suction change is calculated at each node assuming constant stress. Then, the actual elastic response is obtained and any additional applied laods assuming constant soil suction. This two-step process is repeated iteratively until reasonable convergence is obtained before proceeding to the next increment.

SOIL PARAMETERS

The mathematical model requires three sets of parameters as discussed below.

1. Volume Change Parameters, CC_1 and CC_2

These are defined as the slope of the vertical and horizontal strains respectively due to moisture changes versus the logarithm of the soil suction curves from the volume change tests. Over practical changes of soil suction, these curves approximate straight lines (eg. Fig.1). Hence CC_1 and CC_2 can be considered as constants. Results for the three main soil types found behind the retaining wall can be summarized as follows:

Fill	$CC_1 = 0.012$	$CC_2 = 0.010$
Marl	$CC_1 = 0.030$	$CC_2 = 0.026$
Hindmarsh Clay	$CC_1 = 0.072$	$CC_2 = 0.045$

2. Mohr-Coulomb Yield Criterion

Earlier attempts (Kurzeme and Richards, 1973) to define a value of the undrained parameters, C and ϕ for the Hindmarsh clay in terms of total stresses were made difficult by the apparent material variability. Fortunately, soil suction determinations had been made on all test specimens, although it was not previously considered as a possible variable. An examination of the dependence of C and ϕ on soil suction could therefore be made.

Fig. 3 shows the results of the specimens whose soil suction was controlled before the triaxial test was carried out. These test results indicate a good correlation with the measured soil suction. The straight line used as a suggested yield criterion gives C = 0 and the equivalent angle of friction for a change in soil suction, viz. $\phi_h = 8.3^{\circ}$.

The results of all the other triaxial tests on Hindmarsh clay are also plotted in Fig. 3 and the suggested yield criterion is in good agreement. This yield criterion can therefore be written as

$$(\sigma_1 - \sigma_3) = 2.h_t.\tan\phi_h$$

where σ_1 = vertical stress; σ_3 = horizontal stress: h_t = soil suction; $\phi_h = 8.3^{\circ}$.

3. Stress-Strain Relationships

The triaxial test data was back-analysed using a finite element model of the test specimens to determine the hyperbolic non-linear elastic parameters (Richards, 1975, 1976b) for Hindmarsh Clay, viz.

for virgin loadings: $K = 10^6$ kPa
$G = 107\ h_t^{0\cdot61}\ \left(1-\left(\frac{\tau}{\tau y}\right)\right)^{1\cdot0}$ kPa

for unloading or reloading: $K = 10^6$ kPa
$G = 107\ h_t^{0\cdot61}$ kPa

where τ = shear stress $(\sigma_1-\sigma_3)/2$
τy = yield stress = $h_t.\tan\phi$

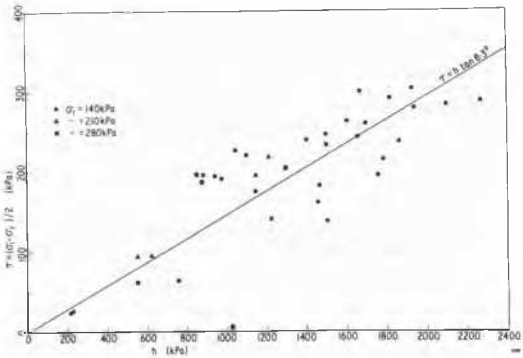

FIG. 3. ULTIMATE SHEAR STRESS - SOIL SUCTION RELATIONSHIP FOR HINDMARCH CLAY - SUCTION CONTROLLED SPECIMENS.

As an example, this relationship gave the predicted stress-strain curves in Fig. 4 for the constant cell pressure (σ_3) triaxial tests compared with typical experimental results with which they were matched.

A check on the parameters was obtained by setting up the theoretical model for a membrane consolidometer test (Richards, 1976a). The results of these analyses are shown in Fig. 5. These results are presented in a form comparable with that commonly reported for expansive clays. This indicates that the volume change parameter for Hindmarsh clay from a consolidation test, viz. CC, for σ_1 = 0 kPa could be of the order of 0.15. This is comparable with values for the most highly expansive clays in the Adelaide area (Aitchison, 1972).

ANALYSES OF RETAINING WALL

The finite element mesh representing the problem is shown in Fig. 6, in which the element dimensions are

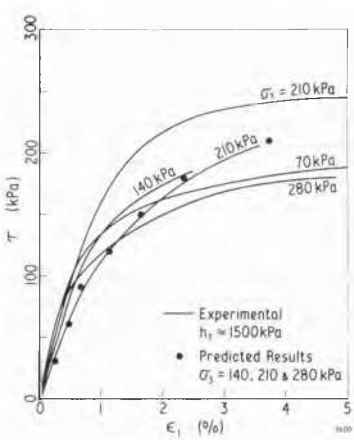

FIG. 4. PREDICTED AND EXPERIMENTAL STRESS-STRAIN CURVES FOR CONSTANT CELL PRESSURE TRIAXIAL TESTS ON HINDMARSH CLAY

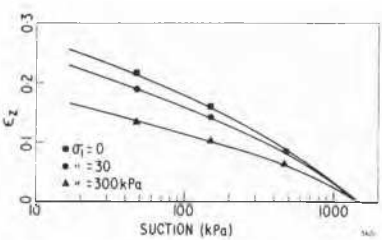

FIG. 5. PREDICTED RESPONSE OF HINDMARCH CLAY IN MEMBRANE CONSOLIDOMETER

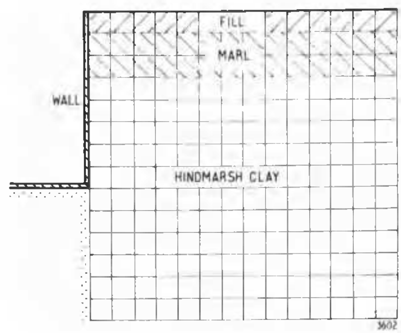

FIG. 6. FINITE ELEMENT MESH REPRESENTING RETAINING WALL

1 m x 1 m. As there was obviously no significant contact between the soil and the wall initailly, the join elements of Ghaboussi et al. (1973) were used to model the interface.

The main unknown was how the soil suctions changed wit time. Any significant changes were restricted to with in a zone 2 metres from the wall according to the psychrometer results. Psychrometer determinations cor ducted on water samples taken from the Marl/Hindmarsh clay interface indicated that due to its electrolyte content, it exhibited an effective suction of 100 to 300 kPa. Consequently, it was assumed that 200 kPa we the limiting soil suction on wetting.

The extent to which wetting took place from the wall would be dependent on the volume of water available tc the clay. However, as the wetting was known to be limited to less than 2 m from the wall, this meant th wetting could only take place as far as the second column of nodes from the wall in the finite element mesh in Fig. 6. The first column represents an effec ive wetting of 0.5 m whereas the second represents 1.

Within these constraints, the following cases were analysed:

1) Progressive wetting up of the single node at the bottom of the wall from 1500 kPa to 200 kPa soil suction (Fig. 7).
2) Progressive wetting up of the single node at the bottom of the wall from 1500 kPa to 200 kPa soil suction before the next node up the wall begins to wet up and so on (Fig. 8).

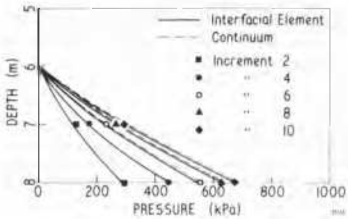

FIG. 7. RESULTS OF ANALYSES FOR CASE 1

3) Progressive wetting up of the single node at the bottom of the wall from 1500 - 1000 - 700 - 450 - 300 - 200 kPa while wetting commences progressively up the wall node by node at each increment.

These three cases were repeated assuming continuity between the soil and the wall and the results obtained are shown as dashed lines in Figs. 8 - 10. These cases were also repeated assuming wetting up of the soil to the second node from the wall. As the pressure patterns were similar except for being slightly higher, only the results for case 3 are shown in Fig. 10.

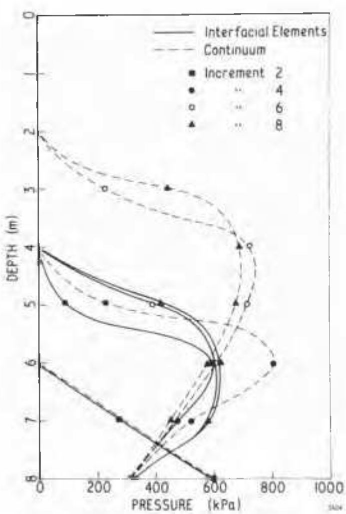

FIG. 8. RESULTS OF ANALYSES FOR CASE 2

DISCUSSION

The results of case 1 in Fig. 7 shows how the earth pressure on the wall increases with reducing soil suction to values comparable to those observed by the earth pressure cells.

The results of case 2 in Fig. 8 show how the earth pressure peak moves progressively up the wall but tends to dissipate. A closer examination of the results suggested that the earth pressures at the lower part of the wall caused the soil adjacent to the upper part

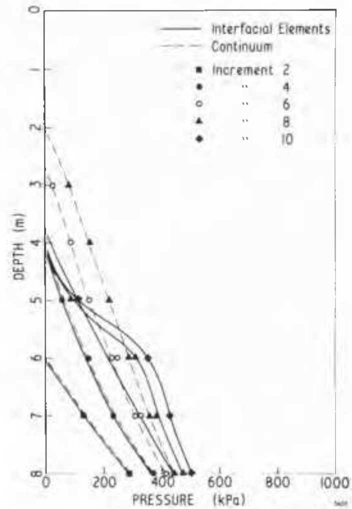

FIG. 9. RESULTS OF ANALYSES FOR CASE 3

of the wall to move away from the wall. Subsequent swelling up the wall was insufficient to close this gap and develop significant earth pressures. This was also verified by the results of the analysis assuming continuity between the soil and the wall also shown in Fig. 8.

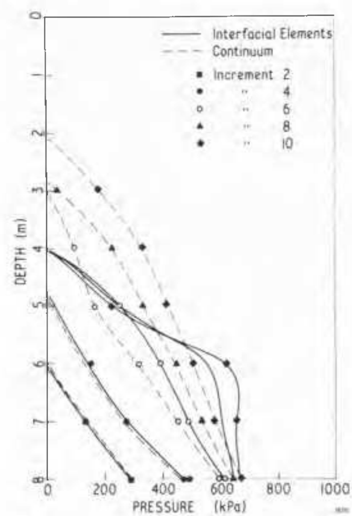

FIG. 10. RESULTS OF ANALYSES FOR CASE 3 WITH WETTING OVER 2 NODES

The results of case 3 in Fig. 9 are in best agreement with the observed earth pressures. As for case 2, the earth pressures dissipate as the wetting front moves up the wall, when no adhesion is assumed between the soil and the wall. The comparable analyses assuming wetting occurs over a distance of 1 m, i.e. to the second node from the wall, is shown in Fig. 10. These show that the earth pressures are similar except that they are higher. Reference to the observed earth pressures in Fig. 2 suggest that the assumptions made in predicting the four results shown in Figs. 9 and 10 could encompass the actual behaviour.

These predicted results therefore help confirm the previously proposed mechanism of behaviour, viz. that the free water encountered at the Marl/Hindmarsh clay interface at a depth of approximately 3 m could penetrate the gap between the temporary shoring and the clay face. This water could collect in this gap at the bottom of the excavation causing swelling of the clay at that point. The swelling front, with the accompanying rise in pressure as indicated by the earth pressure cells, would move upwards as the gap is progressively sealed off to further access of the water.

CONCLUSIONS

The experience with the theoretical model suggests that the soil-wall interaction is complex. However, it is obvious that in general, the earth pressures are dependent on the reduction of soil suction, the volume change parameter and the extent of wetting into the soil. The gap between the soil and the wall appears to have an effect on the earth pressure pattern. Apart from the effect this gap would have on the availability of water, the other factors discussed above could be quantified and considered in the theoretical model.

In practice, the determination of these factors would not be economical in the design of most retaining structures. However, reasonable limits could be placed on them following a simple field investigation, enabling conservative earth pressure patterns to be predicted with the theoretical model. While not considered here, the structural details of the wall itself could have been included in the model, thus permitting a prediction of the wall performance to be made and evaluated for design purposes.

The future trends in the earth pressures will be of considerable interest. If water is no longer available to the clay, the reduction in soil suction will eventually dissipate and the earth pressures will decrease. If water is still available, although at a much reduced rate, the earth pressures will increase to an upper limit of the swelling pressure, which is predicted to be of the order of 1200 kPa.

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