STRESS RATIO EFFECTS ON CONSTRUCTION PORE PRESSURES

LES INFLUENCES DU RAPPORT DES CONTRAINTES AUX PRESSIONS INTERSTITIELLES DANS LA CONSTRUCTION DE PENTES

ВЛИЯНИЕ СООТНОШЕНИЯ НАПРЯЖЕНИЙ НА ВЕЛИЧИНУ ПОРОВОГО ДАВЛЕНИЯ

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SYNOPSIS. Construction pore pressure data from three earth dams have shown trends not indicated by standard laboratory prediction methods. To explain the field data the Skempton equation for the change in pore pressure due to change in stress is expanded in terms of the stress ratio factor K. The resulting equations indicate that the unusual behaviour recorded in the field results from high horizontal stresses caused by heavy compaction. Field measurements of these stresses and simulation tests in a hydraulic oedometer confirm this finding.

SYMBOLS AND DEFINITIONS

u, Δu	Pore pressure and increment thereof.
$\sigma_1, \Delta\sigma_1$	Major principal total stress and increment thereof.
$\sigma_3, \Delta\sigma_3$	Minor principal total stress and increment thereof.
σ_1', σ_3', $\Delta\sigma_1'$, $\Delta\sigma_3'$	Effective principal stresses and increments thereof.
σ_h', σ_v'	Horizontal and vertical effective principal stresses.
γ	Bulk specific weight.
Δh	Increment in height of fill above the point where the pore pressures are considered.
A, B, $\bar{B}$	Skempton's pore pressure parameters.
K	Stress ratio $\Delta\sigma_3'/\Delta\sigma_1'$.
K_o	Stress ratio at no lateral yield.

INTRODUCTION

The stability, during and immediately after construction, of compacted fill structures (earth dams, road embankments) is directly dependent on the construction pore pressures. Design of such structures is usually based on effective stress shear strength parameters and an estimate has to be made of the pore pressure likely to develop during construction.

Prediction of construction pore pressures is usually made by means of "dissipation" tests on compacted samples of the proposed fill materials (Bishop 1957, Bishop and Henkel 1967). In carrying out these tests allowances can be made for the principal stress ratio expected in the field and curves such as shown in Figure 1 are obtained.

Over the past four years pore pressure data have been accumulated from some embankment dams constructed in Southern Africa. These data have shown trends not indicated by test results of the type given in Figure 1. These anomalies can be explained if consideration is given to the stress ratios that exist in the fill as a result of heavy compaction and the changes that take place as the fill is built.

EQUATIONS FOR THE PREDICTION OF CONSTRUCTION PORE PRESSURES

The prediction of the build-up of pore pressures in earth embankments, during construction, is based on the Skempton equation (Skempton 1954) :

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \quad \text{-------------} \quad (1)$$

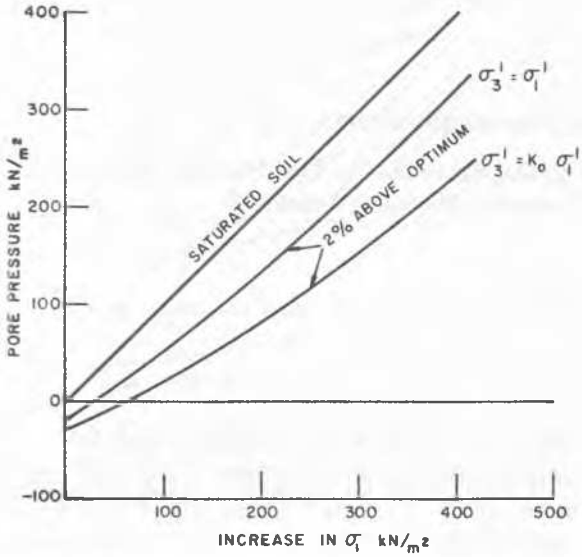

Fig. 1 Undrained pore pressure response to increase in major principal stress

In the field the pore pressure is most conveniently expressed as

$$u = u_o + r_u \cdot \gamma\Delta h^{*} \quad \text{-------------------- (2)}$$

To evaluate the effect of the ratio between the horizontal and vertical effective stresses on the parameter r_u, Skempton's equation is expanded in terms of K (defined as $\Delta\sigma_3'/\Delta\sigma_1'$). Two cases must be considered :-

(a) Horizontal effective stresses less than the vertical

Assuming that σ_v and σ_h are principal stresses and that $\sigma_v = \gamma h$, an increase in overburden pressure will result in the stress changes :

$$\Delta\sigma_1 = \gamma\Delta h \quad \text{------------------------ (3)}$$

$$\Delta\sigma_3 = K[\gamma\Delta h - \Delta u] + \Delta u \quad \text{----------- (4)}$$

Substituting equations 3 and 4 in equation 1 gives :

$$\Delta u = \frac{B[K + A(1 - K)]}{1 - B(1-A)(1-K)}\gamma\Delta h \quad \text{------------ (5)}$$

$$= r_u \cdot \gamma\Delta h$$

(b) Horizontal effective stresses greater than the vertical

An increase in overburden pressure will result in the stress changes :

$$\Delta\sigma_1 = \frac{\gamma\Delta h - \Delta u}{K} + \Delta u \quad \text{------------------ (6)}$$

$$\Delta\sigma_3 = \gamma\Delta h \quad \text{-------------------------- (7)}$$

Substituting in equation 1 gives :

$$\Delta u = \frac{B[K + A(1 - K)]}{K + BA(1 - K)}\gamma\Delta h \quad \text{----------- (8)}$$

$$= r_u \cdot \gamma\Delta h$$

Equations 5 and 8 have been evaluated for various values of K, A and B. The results are shown graphically in Figures 2 and 3. These curves are applicable to any problem of prediction of pore pressure changes due to changes in stress. Consideration will now be given to evaluating the parameters K, A and B for a heavily compacted fill placed at or below optimum moisture content. Immediately after compaction and during the addition of the first few layers of fill the material in the centre of a broad fill will be under conditions of no lateral yield ($K = K_o$).

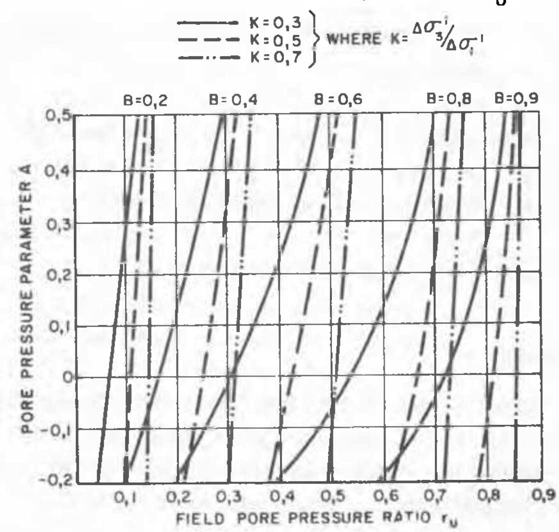

Fig. 2 r_u as a function of A, B and K for vertical effective stress greater than the horizontal effective stress

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* r_u is used here in preference to $\bar{B}$ as $\gamma\Delta h$ need not equal σ_1. The more usual use of r_u includes u_o.

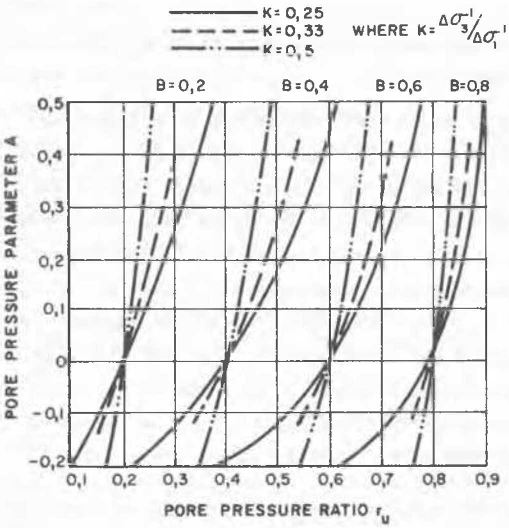

Fig. 3 r_u as a function of A, B and K for horizontal effective stress greater than the vertical effective stress

THE VALUE OF K_o AFTER COMPACTION

It is well known that compacted fill constitutes an overconsolidated material. By considering the stresses beneath, say, a 10-ton tapered-foot roller and the pore pressures set up during compaction, it can be shown that overconsolidation ratios (O.C.R.) of greater than 20 are possible for material placed at or slightly below optimum moisture content.

From the laboratory experiments performed by Brooker and Ireland (1965) (Figure 4) it can be seen that for a typical core material (PI of 10 to 20) and an O.C.R. exceeding 20 the ratio σ_h'/σ_v' could exceed 2,0.

Field confirmation of such high stress ratios in compacted fill has been given by Blight (1970). From in-situ field vane tests and laboratory tests on undisturbed samples Blight determined the stress ratios in the cores of the Bridle Drift Dam (immediately after construction) and the Manjirenji Dam (during construction). At Bridle Drift the ratio σ_h'/σ_v' varied from 7 at a depth of 2 metres below the crest to 3 at a depth of 10 metres below the crest. At Manjirenji the ratio varied between 2 and 3.

As a further check the author placed small earth pressure cells in a 20 cm layer of fill under compaction. After 16 passes of a heavy grid roller and 6 passes of a 10-ton vibratory roller the cells (8 cm below the surface) showed residual horizontal total stresses varying between 65 and 80 kN/m^2.

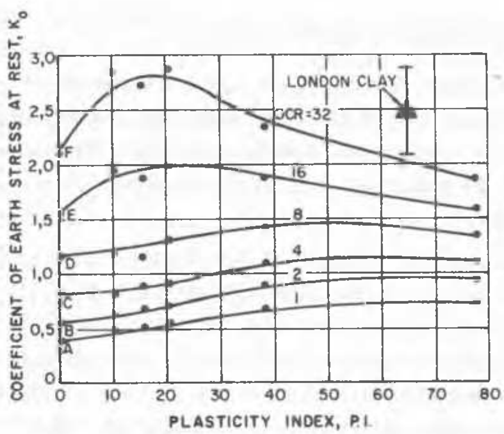

Fig. 4 K_o as a function of the overconsolidation ratio and P.I.

(After Brooker & Ireland, 1965)

THE VARIATION OF K_o WITH LOADING

To investigate the variation in K_o on reloading of a heavily overconsolidated material, experiments were performed using a miniature pressure transducer set in the side of a 50 mm diameter oedometer ring with a 13 mm wall thickness. Figure 5 shows the results from one of these experiments. The results indicate very similar behaviour to the data given in sketch form by Chandler (1967).

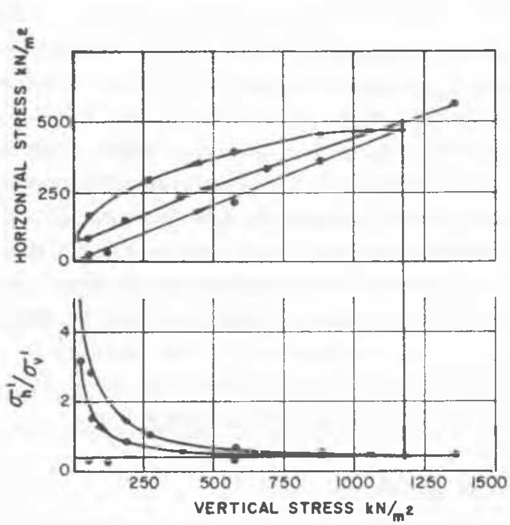

Fig. 5 Effect of load cycling on the value of K_o

THE PORE PRESSURE PARAMETERS A AND B

The pore pressure parameters A and B are always best measured in a laboratory experiment that simulates as closely as possible the field conditions. The parameter B can easily be measured in a standard dissipation test.

In evaluating the parameter A consideration must be given to the deviator stress changes that take place as the vertical load is increased on the freshly compacted layer. Figure 5 shows that, as the overburden pressure first increases, there will be an initial decrease in the deviator stress followed by a slow increase. Initially the major principal stress (σ_1) will be the horizontal stress but will become the vertical stress at some stage as the overburden pressure increases. At some stage also, lateral yield under plane strain conditions will occur due to shear stresses caused by the geometry of the problem.

To simulate these deviator stress changes in the standard triaxial cell in order to measure A is very difficult. An idea of the behaviour of this parameter is gained from the data given by Bishop and Henkel (1953) on cyclic tests on undisturbed samples of overconsolidated Weald clay. These data indicate that as the layer of compacted fill is first loaded A will be positive (0,2 to 0,5) but it will rapidly move into the negative range.

THEORETICAL EVALUATION OF THE TREND IN r_u

With the previous discussion in mind the trend in the parameter r_u can be determined from Figures 2 and 3. If initially B = 0,6, K_o = 0,25 ($\sigma_h' = 4\sigma_v'$) and A = 0,4, then from Figure 3 we have r_u = 0,76. This will apply for the initial increase in overburden pressure. Thereafter K will increase, A will decrease and B will remain fairly constant. The stage is reached when $\sigma_h' = \sigma_v'$ (K = 1,0) and thereafter one must go to Figure 2. As construction proceeds it can be seen that with K and A decreasing, r_u will continue to decrease until the pre-consolidation pressure is reached and there is a marked increase in B.

LABORATORY SIMULATION

The hydraulic oedometer (the Imperial College type has been used here) provides a means of simulating the stress changes that occur in a heavily compacted layer under no lateral yield. This simulation process saves having to measure the changes in K_o, B and A independently.

A series of tests were carried out on a decomposed granite (LL = 47; PI = 22). Specimens were mixed both wet and dry of optimum and tamped lightly into the oedometer ring. The specimens were then loaded and unloaded in steps under undrained conditions. Pore pressures were measured using a high air entry ceramic disc over the whole base of the sample. Figure 6 gives the results from a specimen placed at optimum moisture content. The trend in the value of $\bar{B}$ on the reloading cycle agrees with that expected from the theoretical considerations given above.

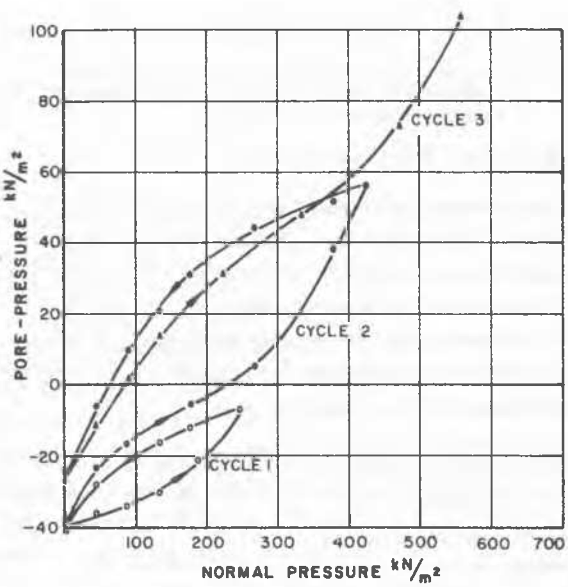

Fig. 6 Undrained pore pressure response in hydraulic oedometer

FIELD DATA

(a) Idas Valley Dam

Figure 7 gives pore pressure data from the 40-metre high embankment of the Idas Valley Dam near Cape Town.

The upstream zone and core of this embankment are virtually homogeneous being constructed of a weathered shale with a laboratory coefficient of consolidation of 1 x 10^{-3} cm^2/sec. A chimney drain was constructed downstream of the crest centreline connected to a drainage blanket beneath the downstream zone. The average field density of the core was 103% Proctor, at a moisture content averaging 1% dry of optimum. Construction took 6,5 months. The shortest drainage path for the piezometers given in Figure 7 was 15 metres. Very little consolidation would have taken place during the construction period. These field data show the same curvature predicted by the theoretical analysis and the hydraulic oedometer test.

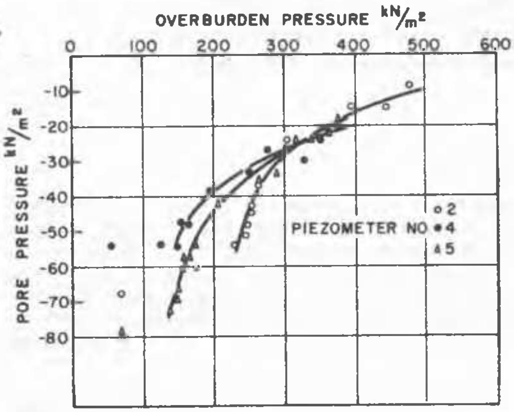

Fig. 7 Piezometer data - Idas Valley Dam

(b) Bridle Drift and Manjirenji Dams

The pore pressure data recorded at the Bridle Drift and Manjirenji sloping core rockfill dams are described in detail by Blight (1970). Figure 8 gives the Bridle Drift piezometer data.

The Manjirenji data are very similar.

The marked decrease in the rate of pore pressure build-up with increasing overburden pressure cannot be explained in terms of partial dissipation of pore pressures. Blight (loc. cit.) has shown that the core material around the bulk of the piezometers at both dams remained sensibly undrained throughout construction. The curvature in the pore pressure build-up can only be explained in terms of the stress ratio changes within the material around the piezometers. Reference has already been made to the high horizontal stresses measured in the cores of both dams. These were obviously the result of heavy compaction of material placed slightly dry of optimum moisture content. (At Bridle Drift the mean field placement density was 102% of Standard Proctor density). Although sufficient laboratory results (i.e. A and B values) are not available to evaluate the data from Bridle Drift and Manjirenji in terms of equations 5 and 8 it is clear that the trend is as given by these equations. It is also clear that a cyclic test in a hydraulic oedometer (Figure 6) would give a reliable indication of the field pore pressure build-up. In a sloping core rockfill dam an even more marked curvature of the pore pressure build-up than that given by the hydraulic oedometer can be expected due to shear stresses resulting from the geometry of the structure. The K_o condition of the laboratory simulation would not remain valid for as long as in the case of a broad central core structure.

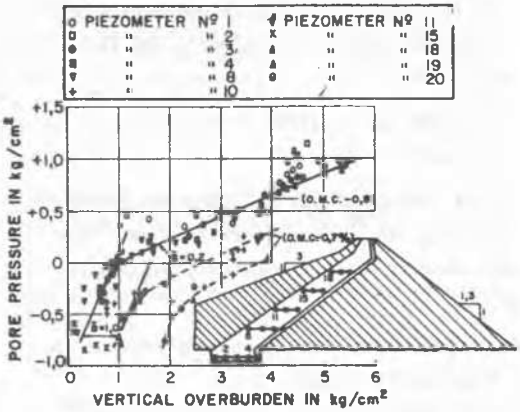

Fig. 8 Piezometer data - Bridle Drift Dam

CONCLUSIONS

In embankments composed of very heavily compacted fill material the initial stresses caused by compaction have been found to have an important effect on the build-up of pore pressures during construction. The initial build-up in pore pressures is greater than expected from standard laboratory "dissipation"-type tests but the rate of build-up decreases as the stress ratio changes under K_o conditions. This decrease is more rapid under conditions where lateral yield takes place.

Cyclic tests in a hydraulic oedometer appear to provide a satisfactory means for predicting these trends.

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