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POREWATER PRESSURES INDUCED IN SOFT CLAYS UNDER AN EMBANKMENT

DES PRESSIONS INTERSTITIELLES QUI SE PRODUISENT DANS LES ARGILES MOLLES AU-DESSOUS D'UNE BANQUETTE

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SYNOPSIS : In 1985, construction was begun on an 8.9 km relief road for the town of Athlone in Co. Westmeath. The route bridges the River Shannon and crosses its flood plain on approach embankments of up to 8.5 metres high. The flood plain is underlain by peat, and very soft deposits of late-glacial and post-glacial lacustrine clays. These include deep deposits of sensitive brown laminated clay. To measure the pressures and deformations occurring in the subsoil, the construction of the embankment was thoroughly instrumented at various control sections. This paper describes the development of a computer model to predict the response of porewater pressures to loading in the brown laminated clay and the ensuing rate of dissipation of the excess porewater pressures. The results from the model are compared with the measured porewater pressures.

INTRODUCTION

With the development of communication networks, the design and construction of embankments on soft clays have become problems of major importance to geotechnical engineers. In recent years, with changes in design requirements and construction practices these problems have become more important. Increased traffic loadings, environmental considerations and economic constraints have resulted in route selections over difficult soils like soft or organic soils.

The construction of an embankment on such soils may cause instability during construction, followed by substantial compression of the subsoil resulting in unacceptable magnitudes of settlement of the embankment.

The prediction of porewater response to loading is critical to the assessment of stability of the embankment during construction and the subsequent rate and magnitude of deformation of the subsoil foundation.

A model was developed to predict porewater pressure response to loading in a subsoil foundation under a 'bermed' trapezoidal, embankment loading. The model was tested using data collected during the construction of a road embankment on a soft laminated clay.

THE CASE STUDY: THE ATHLONE RELIEF ROAD

As part of a national programme of road improvements, an east-west relief road was constructed for the town of Athlone in Co. Westmeath. The route selected is 8.9 km long, passes to the north of Athlone and bridges the River Shannon. It crosses the river flood plain for nearly 1.4 km. Approach embankments to the bridge of up to 8.5 m high were constructed on the peat and soft lacustrine clays of the flood plain.

The site investigation revealed that the flood plain is underlain by deposits of very soft late-glacial and post-glacial lacustrine clays attaining maximum thickness of about 14 m. Underlying these clays there are up to 8 m of sandy glacial lake deposits. The lacustrine deposits are underlain by glacial till which overlies limestone bedrock. A more recent deposit of peat, up to 2 m in thickness, has developed over parts of the flood plain.

A staged construction was used in order to raise the embankments to the required level. In addition, vertical drains were constructed to accelerate the rate of consolidation of the soft clay deposits. The drains were constructed on a rectangular grid and resulted primarily in the occurrence of radial drainage.

For the design of the embankments, the route was divided into six sections with a typical soil profile for each section. In order to monitor the performance of the embankment and its foundation, the embankments were instrumented at a control chainage within each section. This study is based on the layer of brown laminated clay occurring at the control chainage in Section F.

SECTION F OF THE APPROACH EMBANKMENTS

On section F, instrumentation was installed under the centreline and at various locations under the berms and toe of the embankment (see Figure 1). The instrumentation consisted of settlement gauge rods, piezometers and magnetic extensometers. Using the piezometers, porewater pressures were monitored at the centre of each soil layer and at the lower boundary of the brown laminated clay. For the purposes of this study, the computer model was applied to the prediction of porewater pressure behaviour under the centreline of the embankment.

The subsoil profile and instrumentation under the centreline at the control chainage of section F is shown in Figure 2. It can be seen that a 1.1 metre layer of peat overlies 1.4 metres of grey organic clay and 4.3 metres of brown laminated clay. These are underlain by deep deposits of glacial till.

The brown laminated clay is described as a very soft greyish brown, thinly laminated inorganic silty clay (Ove Arup, 1981b). The laminae consist of alternating layers of silt and clay of 1 to 2 mm in thickness.

A comprehensive programme of tests was carried out on the soft lacustrine deposits. These comprised the standard tests for Atterberg Limits, particle size distribution, chemical tests, organic content tests and consolidation tests.

Prior to raising the embankments for the Athlone Relief Road, two trial embankments were constructed and fully instrumented. The main trial embankment was designed to investigate the compressibility and consolidation characteristics of the soft lacustrine deposits. The subsidiary trial embankment was built to investigate the shear strength of the soft deposits.

The design parameters used for the construction of the trial embankments were based on the results of the original site investigation. The information collected from the construction of the trial embankments was used to carry out a back-analysis and determine the design parameters for the relief road embankments.

Table 1 shows the soil parameters for the construction of the trial embankments, the back-calculated parameters used for the construction of the relief road embankments and the deformation parameters used in the computer model.

A computer model was developed to predict the rate of deformation of the soil strata under the centreline of the embankment. The model was written in the programming language Fortran 77, in the form of a sequence of three programs called "CONSOL".

The consolidation equations used to predict the rate of deformation of the soil stratum are solved using a finite difference method. This involves the discretization of the soil stratum into an orthogonal grid. It is assumed that the properties of the soil layer are not variable laterally, but that its properties may vary with depth. The grid is described horizontally by a fixed number of columns between the vertical

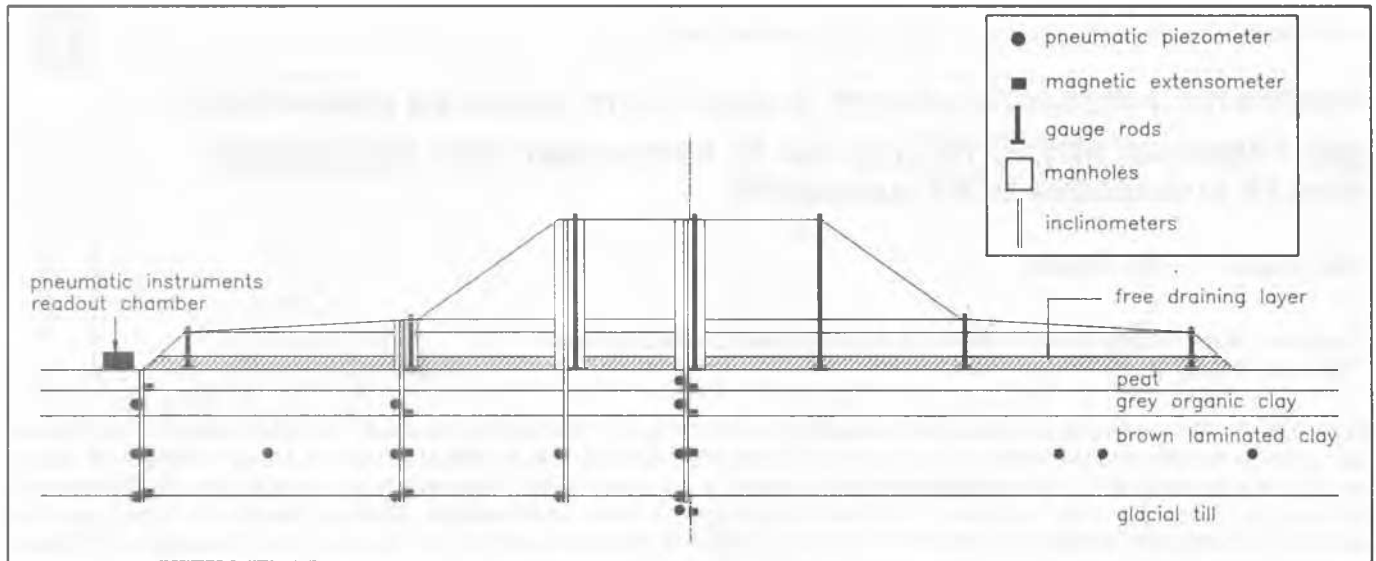


Fig. 1 The layout of ground instrumentation at the control section of Section F.

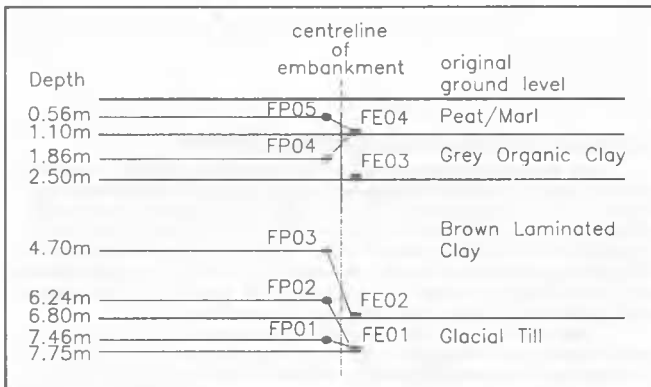


Fig. 2 The layout of piezometers and extensometers under the centreline of embankment at Section F.

drain centres; these columns are of a constant width. To accommodate a variation in soil properties throughout the depth of the soil layer, the orthogonal grid is described vertically by a number of sublayers of constant or variable depths. Each sublayer can be assigned a different value for each soil property. The accuracy of description of the soil profile depends on the number of sublayers in the grid.

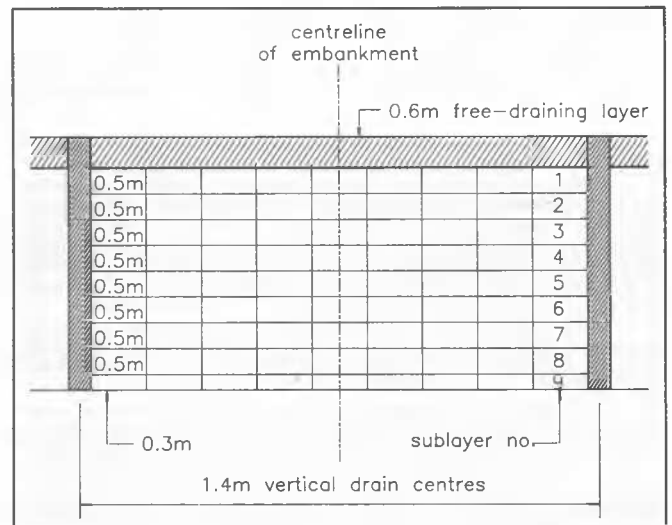


Fig. 3 The orthogonal grid used for the finite difference solution.

For the application of this model to the case study, the 4.3 metre brown laminated clay stratum was divided into 9 sublayers and 9 columns between the vertical drains (Figure 3).

Table 1 Soil parameters for embankment design at Section F.

Soil Parameter	Symbol	Units	For Relief Road Embankment and consolidation model	For Trial Embankment
Effective preconsolidation pressure	σ'_c	kN/m ²	$\sigma'_{v0} + 17$	$\sigma'_{v0} + 14$
Unit Weight	γ	kN/m ³	19	19
Coefficient of horizontal drainage	c_h	m ² /year	3	3
Porewater Parameter, A	A			0.48
Porewater Parameter, B	B		1.0	1.0
Undrained shear strength	c_u	kN/m ²	min. 0.05	0.064

The Extent of the Deformation Study

Construction of the embankment on Section F was in progress for 572 days, from day 340 to day 912 of the Relief Road construction programme (see Figure 4). For the purposes of this study, a small portion of the loading sequence was extracted for study of the porewater response during loading, the porewater decline during consolidation and the rate of deformation during consolidation. The study period chosen begins after a long period of no load application. It is assumed that full dissipation of excess porewater pressure has taken place from the last application of load, 128 days previously.

The study period begins on day 669 with a height of fill of 2.28 metres. From day 674, a series of load increments are applied. This is followed by a period of no load application for 22 days in which the porewater pressures are allowed to dissipate. A second period of load increments are applied from day 711 to day 744. The study period ends at day 744.

THE COMPUTER MODEL

The three program units within the sequence are called STRESS.FOR, ISO.FOR and SETT.FOR.

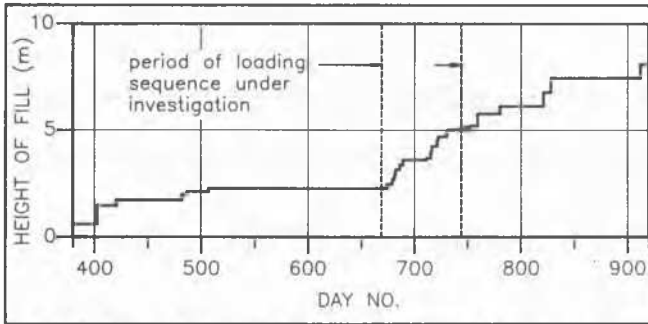


Fig. 4 The loading sequence for the construction of the embankment at Section F.

STRESS.FOR

The main function of the program STRESS.FOR is the calculation of total and effective stresses throughout the soil stratum. Preconstruction *insitu* stresses are initially calculated followed by the calculation of the stresses induced by the embankment. The normal stresses, σ_x , σ_z , and τ_{xz} , induced at the centre of each sublayer are calculated using elastic stress theory in the form of equations developed by Carrothers (Jurgenson, 1934). Equation 1 presented here, was derived by the superimposition of two systems of elastic stress equations; for rectangular strip loading and triangular strip loading.

$$\Delta\sigma_x = \frac{\Delta p}{\pi} \left[(\alpha_1 - \sin \alpha_1) + 2 \left(\frac{b}{a} \alpha_2 - \frac{z}{a} \ln \frac{R_1}{R_2} + 1/2 \sin \alpha_1 \right) \right] \quad (1a)$$

$$\Delta\sigma_z = \frac{\Delta p}{\pi} \left[(\alpha_1 + \sin \alpha_1) + 2 \left(\frac{b}{a} \alpha_2 - 1/2 \sin \alpha_1 \right) \right] \quad (1b)$$

$$\Delta\tau_{xz} = \frac{\Delta p}{\pi} \left[(1 + \cos \alpha_1 - \frac{2z}{a} \alpha_2) \right] \quad (1c)$$

The model takes each increment in embankment height, calculates the dimensions of the resulting trapezoidal shape (see Figure 5) resulting from that increment and Equation 1 is applied to determine the increments in normal stresses due to each step in the loading sequence. Following this, from elastic theory, the major and minor principal stresses are calculated.

For the calculation of excess porewater pressures induced by the application of incremental loads the computer model offers a choice of three methods;

- (a) Isotropic elastic theory : $\Delta u = \Delta\sigma_1$
- (b) Skempton's Theory : $\Delta u = (\Delta\sigma_1 + A(\Delta\sigma - \Delta\sigma_3))$ (Skempton, 1954)
- (c) Henkel's Theory : $\Delta u = (\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)/3 + \tau_{oct}$ (Henkel, 1960)

The excess porewater pressures are calculated at the centre of each sublayer at each time step in the loading sequence.

ISO.FOR

The next program in the "CONSOL" sequence generates the isochrones arising from the dissipation of excess porewater pressure. The consolidation equation used by this model is an adaptation of Rendulic's consolidation equation (Rendulic, 1936).

The construction of vertical drains usually results in an anisotropy of coefficient of consolidation values, with c_v in the vertical direction and c_h in the horizontal or radial directions. An adaptation of Rendulic's equation expressed in polar co-ordinates is,

$$\frac{\delta u}{\delta t} = c_h \left[\frac{\delta^2 u}{\delta R^2} + \frac{1}{R} \frac{\delta u}{\delta R} \right] + c_v \frac{\delta^2 u}{\delta z^2} \quad (2a)$$

For the application of this case study to the model, it was assumed that primarily radial drainage occurs *insitu* and equation (2a) reduces to,

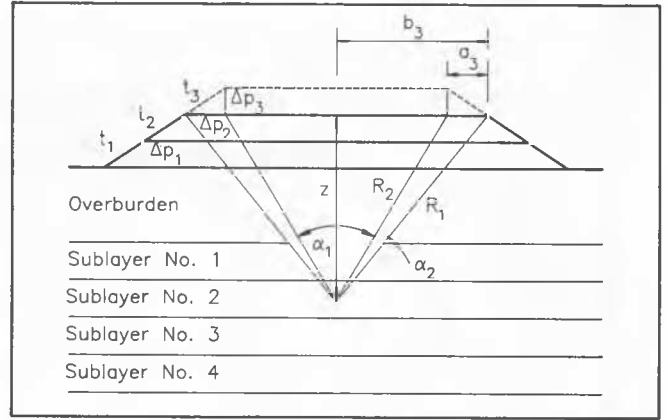


Fig. 5 The application of incremental trapezoidal loads.

$$\frac{\delta u}{\delta t} = c_h \left[\frac{\delta^2 u}{\delta R^2} + \frac{1}{R} \frac{\delta u}{\delta R} \right] \quad (2b)$$

Equation (2b) was formulated for an explicit finite difference solution and solved at each discrete time step in the loading sequence generating an isochrone. When each isochrone is generated, an average degree of consolidation is calculated for the particular sublayer at that moment in time. An array of U against time is generated for each sublayer in the soil stratum. This array is used by the final program in the sequence SETT.FOR to calculate the rate of compression of each sublayer.

RESULTS FROM THE MODEL — A COMPARISON OF MEASURED AND CALCULATED EXCESS POREWATER PRESSURES

The calculated excess porewater pressures under the centreline of the embankment were plotted. The piezometer FP03 (see Figure 2) roughly coincides with the centre of sublayer no. 4. The measured excess porewater pressures from FP03 were plotted against the calculated excess porewater pressures at the centreline of the isochrones produced for sublayer no. 4. These were plotted using output from the three methods of prediction for porewater response to loading which are presented by the computer model to the user.

One-dimensional $\Delta u = \Delta\sigma_1$

At the centre of sublayer no. 4, the overconsolidation ratio is about 1.4, with a $\Delta\sigma_{v0}'$ of about 38 kN/m² and a σ_c' of about 55 kN/m². The preconsolidation pressure is attained on day 682. Figure 6 shows the measured and calculated excess porewater pressures at the centre of the brown clay layer, where u_{excess} has been estimated using the one-dimensional method.

Up to σ_c' , the one-dimensional method predicts the porewater response closely. This method of prediction is an elastic method and its accuracy of prediction in the overconsolidated range may be attributable to the near-elastic behaviour of overconsolidated soils.

Beyond σ_c' , this method overpredicts the response by approximately 15%. In comparison to other studies this is unusual, and the pore pressure response would be expected to be of greater magnitudes than in the overconsolidated range. However, D'Appolonia et al. (1971) reported a similar overprediction using this method for an embankment built on Boston blue clay.

From Figure 6 it can be seen that the predicted rate of consolidation agrees with that measured i.e. the rate of decay, du/dt of the two graphs is the same. This would imply that the value for c_h chosen is good and that the radial consolidation equation is a reasonable representation of *insitu* behaviour

Semi-empirical : Henkel's Method

Figure 7 shows the measured values of porewater pressures at FP03 against the calculated values in sublayer no. 4, where Henkel's semi-empirical method was used to predict the porewater response.

As before, there is good agreement between measured and calculated values in the overconsolidated range up to day 682. Beyond this, the model overpredicts the porewater pressures by about 18%.

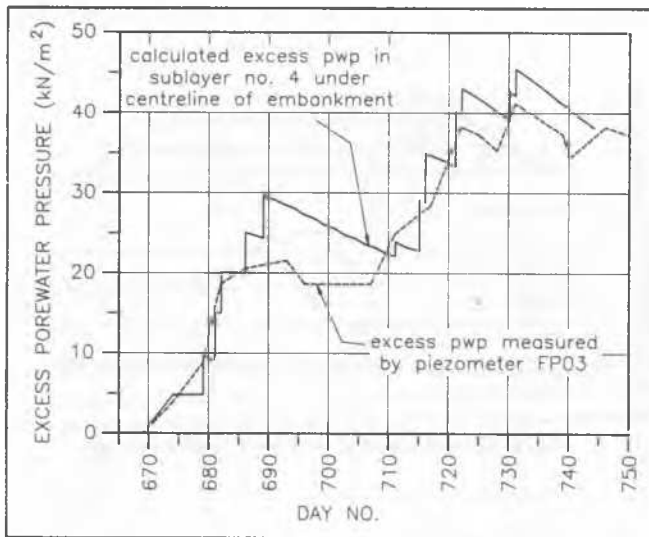


Fig. 6 Graph of *excess porewater pressure vs. time* for Brown Laminated Clay at centre of soil layer.
 Note : Excess porewater pressure calculated using $\Delta u = \Delta \sigma_1$.

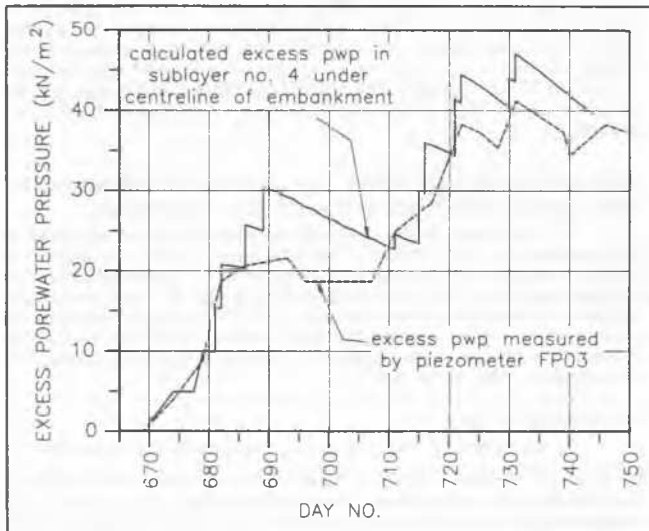


Fig. 7 Graph of *excess porewater pressure vs. time* for Brown Laminated Clay at centre of soil layer.
 Note : Excess porewater pressure calculated using Henkel's Equation (1960).

From Figure 7, it can be seen that the rate of consolidation and the chosen value of c_h are in good agreement with the measured rate. This is particularly evident after day 722.

Two-dimensional semi-empirical : Skempton's Method

The third option presented by the model showed very good agreement between calculated and measured values of excess porewater pressures at the centre of the brown clay (Figure 8). For the range before σ_c' , there is a slightly lower estimation of calculated values than given by the other two methods. In the normally consolidated range, both the porewater response to loading, and the subsequent rate of dissipation are very close to *insitu* measurements.

SUMMARY

For this particular case study, relative to the prediction of porewater pressures, the model produced output which closely matched the *insitu* behaviour of porewater pressures. Of the three methods of prediction for porewater response to loading, Skempton's (1954) formula

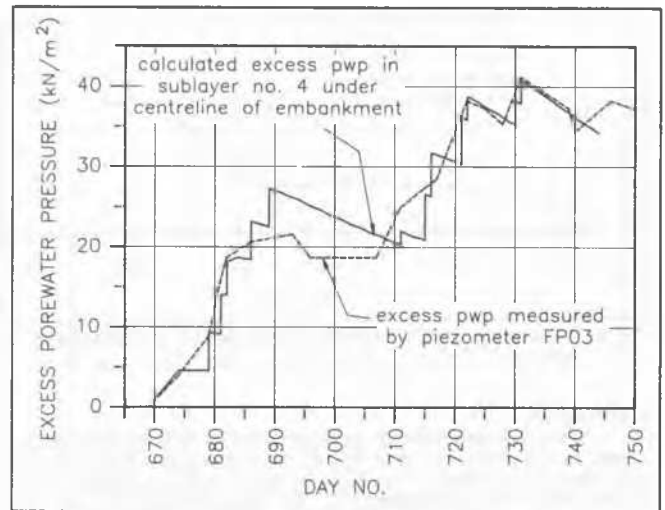


Fig. 8 Graph of *excess porewater pressure vs. time* for Brown Laminated Clay at centre of soil layer.
 Note : Excess porewater pressure calculated using Skempton's Equation (1954).

produced the best results. The one-dimensional method and Henkel's (1960) method, although slightly overestimating the response to loading, produced graphs which followed the general trend of measured porewater behaviour.

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