NRC Publications Archive Archives des publications du CNRC

A method of estimating excess pore pressures beneath embankments on sensitive clays

Law, K. T.; Bozozuk, M.

This publication could be one of several versions: author's original, accepted manuscript or the publisher's version. /
La version de cette publication peut être l'une des suivantes : la version prépublication de l'auteur, la version
acceptée du manuscrit ou la version de l'éditeur.

Publisher's version / Version de l'éditeur:

Canadian Geotechnical Journal, 16, 4, pp. 691-702, 1979-11

NRC Publications Record / Notice d'Archives des publications de CNRC:

<https://nrc-publications.canada.ca/eng/view/object/?id=1b71cc9e-bd3b-4336-ac06-f52d9183330d>
<https://publications-cnrc.canada.ca/fra/voir/objet/?id=1b71cc9e-bd3b-4336-ac06-f52d9183330d>

Access and use of this website and the material on it are subject to the Terms and Conditions set forth at

<https://nrc-publications.canada.ca/eng/copyright>

READ THESE TERMS AND CONDITIONS CAREFULLY BEFORE USING THIS WEBSITE.

L'accès à ce site Web et l'utilisation de son contenu sont assujettis aux conditions présentées dans le site

<https://publications-cnrc.canada.ca/fra/droits>

LISEZ CES CONDITIONS ATTENTIVEMENT AVANT D'UTILISER CE SITE WEB.

Questions? Contact the NRC Publications Archive team at

PublicationsArchive-ArchivesPublications@nrc-cnrc.gc.ca. If you wish to email the authors directly, please see the first page of the publication for their contact information.

Vous avez des questions? Nous pouvons vous aider. Pour communiquer directement avec un auteur, consultez la première page de la revue dans laquelle son article a été publié afin de trouver ses coordonnées. Si vous n'arrivez pas à les repérer, communiquez avec nous à PublicationsArchive-ArchivesPublications@nrc-cnrc.gc.ca.

Ser
TH1
N21d

National Research
Council Canada

Conseil national
de recherches Canada

no.866
cop.2
BLOG.
RES.

7587

A METHOD OF ESTIMATING EXCESS PORE PRESSURES BENEATH EMBANKMENTS ON SENSITIVE CLAYS

~~ANALYZED~~

by K. T. Law and M. Bozozuk

Reprinted from
Canadian Geotechnical Journal
Vol. 16, No. 4, November 1979
p. 691-702

DBR Paper No. 866
Division of Building Research

Price \$1.00

OTTAWA

NRCC 17858

This publication is being distributed by the Division of Building Research of the National Research Council of Canada. It should not be reproduced in whole or in part without permission of the original publisher. The Division would be glad to be of assistance in obtaining such permission.

Publications of the Division may be obtained by mailing the appropriate remittance (a Bank, Express, or Post Office Money Order, or a cheque, made payable to the Receiver General of Canada, credit NRC) to the National Research Council of Canada, Ottawa. K1A 0R6. Stamps are not acceptable.

A list of all publications of the Division is available and may be obtained from the Publications Section, Division of Building Research, National Research Council of Canada, Ottawa. K1A 0R6.

A method of estimating excess pore pressures beneath embankments on sensitive clays¹

K. T. LAW AND M. BOZOUK

Geotechnical Section, Division of Building Research, National Research Council of Canada, Ottawa, Ont., Canada K1A 0R6

Received October 17, 1978

Accepted July 4, 1979

A laboratory testing program for predicting undrained excess pore pressures beneath embankments constructed on sensitive clays is reported. A study of the pore pressure response, based on Henkel's equation applied to triaxial and plane strain tests performed on marine clay from South Gloucester, Ontario, provides a basis for comparing behaviour under conditions of axisymmetry and plane strain. Comparison shows that (1) pore pressure is smaller in plane strain than in axisymmetric conditions; (2) a below-elastic response prevails at the prepeak stage of the stress-strain curve; and (3) there is an elastic response at the peak.

Finite element and modified elastic methods are developed for estimating undrained excess pore-water pressures from laboratory test results. The estimated values are used to evaluate the effect of consolidation under the center of an embankment during construction. The procedure is illustrated with case records.

On présente les résultats d'un programme d'essais de laboratoire pour prédire les surpressions interstitielles en état non drainé sous des remblais construits sur des argiles sensibles. Une étude du comportement des pressions interstitielles, fondée sur l'équation de Henkel, appliquée à des essais triaxiaux et de déformation plane réalisés sur l'argile marine de South Gloucester, Ontario, fournit une base de comparaison des comportements en état de symétrie axiale et de déformation plane. La comparaison montre que (1) la pression interstitielle est plus petite en déformation plane qu'en condition axisymétrique, (2) une réaction "sous-élastique" se développe avant le pic de la courbe contrainte déformation et (3) le comportement est élastique au pic.

Des méthodes élastiques modifiées et d'éléments finis sont développées pour estimer les surpressions interstitielles en état non drainé à partir des résultats d'essais de laboratoire. Les valeurs estimées sont utilisées pour évaluer l'effet de la consolidation sous le centre d'un remblai durant sa construction. La procédure est illustrée au moyen de cas types.

[Traduit par la revue]

Can. Geotech. J., 16, 691-702 (1979)

Introduction

Excess pore pressure plays an important role in determining the stability of embankments on soft ground. Basically, two methods are available for analysing stability: total stress analysis and effective stress analysis. Although effective stress analysis is fundamentally more correct, total stress analysis is simpler and has been used more often, with the result that much more experience has been accumulated. Problems such as undrained strength anisotropy, strain rate effect, and progressive failure can be treated empirically, based on observed failures. Although the total stress approach has proved successful for some clays under certain loading conditions (Bjerrum 1972; Dascal and Tournier 1975), its general validity is still in doubt. A recent symposium at the Massachusetts Institute of Technology (1975), for example, has indicated that such analysis may give unrealistic results.

Although effective stress analysis is preferable, it is less popular because of the many engineering parameters involved, which are generally time-consuming and sometimes difficult to determine. For example, estimation of excess pore-water pressures, a vital step in effective stress stability analysis, requires a careful study before design and construction of a proposed embankment on soft clay is carried out.

The development and dissipation of excess pore pressures beneath embankments on soft clays has long been of interest to the Geotechnical Section, Division of Building Research, National Research Council of Canada. Measurements of excess pore pressures have been made in a number of embankments in the Ottawa region. Laboratory tests have also been carried out, using triaxial and plane strain cells to study the pore pressure response of the subsoils under various stress conditions. This has led to improved methods of estimating excess pore pressures under embankments.

Laboratory Study

Soil samples were taken from the soft sensitive clay

¹Presented at the 31st Canadian Geotechnical Conference, Winnipeg, Man., October, 1978.

TABLE 1. Summary of laboratory test results

Test series	Test type	Sample type	K	Number of tests	Average α	
					α_t	α_f
1	Plane strain (PSU)	Osterberg	0.5	10	-0.10	-0.022
2	Triaxial (CKU)	Osterberg	0.5	15	-0.054	0.016
3	Triaxial (CKU)	NGI tube	0.62	11	-0.0027	0.036
4	Triaxial (CIU)	Osterberg and block	1.0	7	0.025	0.13

NOTES: K = ratio of horizontal to vertical consolidation pressures; all vertical consolidation pressures are equal to the *in situ* effective overburden pressures.

deposit at South Gloucester near Ottawa using the 54 mm diameter NGI² piston sampler (Bjerrum 1954), 127 mm diameter Osterberg sampler (Osterberg 1952), and the block sampling technique (Law 1974). Consolidated, undrained plane strain and triaxial compression tests were conducted in four series, as listed in Table 1.

Henkel's (1960) pore pressure equation, which includes the intermediate principal stress, was used to provide a basis for comparing the measured pore pressure response obtained from the plane strain and triaxial tests on the saturated soils.

$$[1] \quad \Delta u = \frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) + \alpha[(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2]^{1/2}$$

where α = pore pressure parameter; Δu = excess pore pressure; and $\Delta\sigma_1, \Delta\sigma_2, \Delta\sigma_3$ = change in major, intermediate, and minor principal stresses, respectively.

The pore pressure parameter, α , is a variable depending on the test conditions and stress levels. When it is zero, the soil displays elastic behaviour since the excess pore pressure response is equal to the average change in total principal stresses. When the response is less than the elastic case (α is negative), it is defined as below elastic; if it is greater (α is positive), it is above elastic. The elastic condition was used as a reference case. Two particular values of α were considered in analysing the test results: α_f at the failure condition, and α_t at the working stress level corresponding to that induced by the test embankment at this site (Bozozuk and Leonards 1972).

A summary of the laboratory test results is given in Table 1. The pore pressure parameters α_t and α_f both increase under the following conditions: (1) from plane strain to axisymmetric strain; (2) from large size (127 mm diameter) to smaller size (54 mm diameter); and (3) from a lower to a higher lateral consolidation pressure. Conditions (2) and (3) are predictable because obtaining and trimming small samples and

TABLE 2. Results of statistical analysis

Test series	Type of test	α_t	α_f
1	PSU, K = 0.5	-1	0
2	CKU, K = 0.5	-1	0
3	CKU, K = 0.62	0	1
4	CIU, K = 1.0	0	1

NOTES: -1 denotes statistically below-elastic response; 0 denotes statistically not different from elastic response; 1 denotes statistically above-elastic response.

applying high confining pressures cause more disturbance in this soft sensitive clay. Condition (1) is seldom reported, probably because relatively few studies of this behaviour have been conducted.

Using statistical analysis a hypothesis testing procedure was applied to the results of each test series to determine whether the pore pressure response is elastic. A null hypothesis of $\alpha = 0$ was first chosen, and Student's *t* test was employed (Hald 1952). This test indicates whether the hypothesis should be accepted at the assigned confidence level (5% in this case). If acceptable, the pore pressure response would be elastic. If rejected, a one-sided test would be carried out to examine whether the pore pressure response is statistically below or above elastic behaviour.

The statistical analysis summarized in Table 2 for plane strain and triaxial tests performed on Osterberg samples (series 1 and 2) showed that at failure α_f was elastic, and at the working stress level α_t was below elastic. Applying the analysis to tests performed with NGI tube samples or to those consolidated at higher lateral pressures (series 3 or 4), it was found that α_t was elastic and α_f above elastic.

An additional statistical test was carried out to compare the relative values of α_t obtained from the plane strain with the triaxial tests. It showed that α_t was significantly lower at plane strain than at axisymmetric strain. The gradual change of α with respect to shear stress level *R*, defined as the ratio of shear stress to shear strength, was plotted for several samples in Fig. 1. The average slope was used to

²Norwegian Geotechnical Institute.

approximate the relation between α and R and is represented by

$$[2] \quad \alpha = 0.24 (R - 1)$$

Substituting [2] in [1] gives

$$[3] \quad \Delta u = \frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) + 0.24 (R - 1) \times [(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2]^{1/2}$$

Theoretical Development for Estimating Undrained Excess Pore Pressure

Finite Element Method

The stresses in the subsoil under the applied load are determined by the finite element method (FEM) incorporating progressive failure and embankment rigidity (Law and Lo 1976).

To estimate the excess pore pressure from the computed stresses prepeak and postpeak stages are considered. Equation [3] can be used directly for the prepeak stage. For the postpeak stage, consider an elastoplastic clay stressed to failure, i.e., $R = 1.0$. The excess pore pressure Δu^f , as shown earlier, will be elastic and given by

$$[4] \quad \Delta u^f = \frac{1}{3}(\Delta\sigma_1^f + \Delta\sigma_2^f + \Delta\sigma_3^f)$$

where the superscript f refers to the failure condition. Upon further straining into the plastic range, the soil will maintain a constant shear resistance such that the major and minor principal stress increments $\Delta\sigma_1^p$ and $\Delta\sigma_3^p$, respectively, will be equal. They will also be equal to the intermediate principal stress increments $\Delta\sigma_2^p$ under plane strain conditions. The net result is a hydrostatic increase in total stress, which will increase the excess pore pressure by

$$[5] \quad \Delta u^p = \frac{1}{3}(\Delta\sigma_1^p + \Delta\sigma_2^p + \Delta\sigma_3^p)$$

or $\Delta u^p = \Delta\sigma_1^p$ for this plane strain condition.

Combining [4] and [5] gives the final excess pore pressure

$$\begin{aligned} \Delta u &= \Delta u^f + \Delta u^p \\ &= \frac{1}{3}(\Delta\sigma_1^f + \Delta\sigma_2^f + \Delta\sigma_3^f) \\ &\quad + \frac{1}{3}(\Delta\sigma_1^p + \Delta\sigma_2^p + \Delta\sigma_3^p) \end{aligned}$$

Noting that both terms on the right side are stress invariants, the equation simplifies to

$$[6] \quad \Delta u = \frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)$$

where all the stress increments refer to the final state. These increments are automatically calculated in the FEM, with local failure and the resulting stress transfer being accounted for. Due to this transfer of stress,

FIG. 1. Pore pressure parameter vs. shear stress level from plane strain tests.

FIG. 2. Loading condition for use with Poulos' charts (1967).

[6] cannot be obtained by setting $R = 1$ in [3] because it would give the false impression that the final stress increments were obtained from linear elasticity.

Modified Elastic Method

The FEM, though more rigorous, requires a large-capacity computer for the calculations. For preliminary estimates a simple method was developed that would give the induced stress by hand calculation. This method is based on Poulos' (1967) charts modified to include the phenomenon of local failure that may exist when the safety factor of the embankment is low.

Stresses in the subsoil are computed using Poulos' (1967) charts, with the trapezoidal embankment load simulated by a series of rectangular loads (Fig. 2). The vertical and horizontal stress increments $\Delta\sigma_v$ and $\Delta\sigma_h$ as well as the shear stress along the vertical or horizontal plane are determined from the charts. Consequently, the major and minor principal stress increments and the resultant maximum shear stress τ_{max} can be computed. τ_{max} is then used to calculate the shear stress level R . For the prepeak stage and $R < 1.0$, the excess pore pressure can be estimated from [3]. Introducing an undrained Poisson's ratio, $\nu = 0.5$, the equation reduces to

$$[7] \quad \Delta u = 0.5(\Delta\sigma_1 + \Delta\sigma_3) + 0.29(R - 1)(\Delta\sigma_1 - \Delta\sigma_3)$$

For the postpeak stage and $R > 1.0$ (possible with

FIG. 3. Assumptions used in the consolidation formulation. (a) Loading rate. (b) Initial undrained excess pore pressure profile.

purely elastic analysis of Poulos 1967) stress transfer will take place and the following equation should be used

$$[8] \quad \Delta u = \Delta \sigma_v^e \left(1 - \frac{1}{2R}\right) + \frac{1 - K_0}{2} \sigma_{v0}' \left(1 - \frac{1}{R}\right) + \frac{1}{2R} \Delta \sigma_h^e$$

where K_0 = coefficient of earth pressure at rest; σ_{v0}' = initial effective vertical stress; and the superscript e refers to elastic state. This equation reduces to the following simple form along the centreline:

$$[9] \quad \Delta u = \Delta \sigma_v^e + [(1 - K_0)/2] \sigma_{v0}' - S_u$$

where S_u = the undrained shear strength of the clay.

Derivations of [8] and [9] are shown in the Appendix.

Consolidation at the End of Construction

Some dissipation of excess pore pressure is possible in the field during construction (D'Appolonia *et al.* 1971; Foss 1972; Moh *et al.* 1972; Leroueil *et al.* 1978). From the analysis of settlement profiles and improved measurements of consolidation coefficients, Law (1974) concluded that partial dissipation significantly influences the distributions of settlement

and excess pore pressure with depth in the foundation soils at the end of construction. Accordingly, a better understanding of field observations can be obtained by incorporating the consolidation effect.

An approximate method applicable under the centreline of the embankment was developed. It is based on Terzaghi's consolidation theory, modified to satisfy two additional conditions. During construction the load is assumed to increase linearly with time, and the initial undrained excess pore pressure from the finite element method is approximated by a straight line or a combination of straight lines (Fig. 3). Because of the second condition, Schiffman's (1958) method, which applies to constant excess pore pressure with depth, is not used here.

A solution to this problem is shown as follows. Let t_c be the time elapsed after the beginning of construction at any time in the construction stage. The accumulated load at this point is used to estimate the initial undrained excess pore pressure. To account for consolidation, it can be shown (Law 1974) that, under linear loading rate with time, the accumulated load can be treated as an instant load being applied for a period of $t_c/3$. The excess pore pressure $\Delta u(z)$ at a distance z from the top drainage boundary is then shown to be

$$[10] \quad \Delta u(z) = \sum_{n=1}^{\infty} \left[\frac{2}{n\pi} (u_t - u_b \cos n\pi) \times \left(\sin \frac{n\pi z}{2H} \right) \exp \left(-\frac{1}{4} n^2 \pi^2 T_v \right) \right]$$

where $2H$ = thickness of compressible layer; u_b , u_t = initial undrained excess pore pressure at the bottom and top drainage boundaries, respectively; T_v = time factor = $c_v t_c / 3H^2$; and c_v = coefficient of consolidation corresponding to the appropriate stress range.

This method is illustrated with three case records. In these cases the end of construction excess pore pressure is considered and t_c is taken at this point.

Case Records

Three well-instrumented embankments founded on soft, sensitive marine clay near Ottawa, Ontario are considered. The subsoil conditions are summarized in Figs. 4–6 and the construction schedules are shown in Fig. 7.

Boundary Road Embankment (Law *et al.* 1977)

This embankment is situated about 20 km south-east of Ottawa on Boundary Road at Highway 417. It was first constructed as a two-level test embankment with heights of 2.75 and 4.0 m. The geometry of each section and the location of the Geonor piezom-

FIG. 4. Soil profile at Boundary Road.

eters are shown in Figs. 8 and 9. Both sections are founded on the same soil profile, (Fig. 4), a top 2.4 m layer of silty sand followed by 18.9 m of sensitive marine clay underlain by sand. The significant difference based on a computation between the two sections is that local failure occurs only under the higher section at the end of construction.

In determining the local failure zone (Fig. 10), an elastoplastic behaviour was assumed with an undrained Poisson's ratio equal to 0.49 and prepeak modulus E given by $E = 1000S_u$ where S_u is the NGI field vane shear strength. This strength, which probably represents the average mobilized strength of this soil (Bozozuk 1977), was also used to obtain the shear stress level R . The coefficient of earth pressure at rest K_0 was measured using hydraulic fracture tests (Bozozuk 1974) and found to average about 0.9. Laboratory plane strain undrained tests were also conducted and gave essentially the same results as those of the clays from South Gloucester (discussed in the next section).

Because of the appreciable difference in the shear stress level developed under each section, estimates

of undrained excess pore pressures, using both FEM and the modified elastic method, were compared throughout the prepeak and postpeak stages of straining. The results are shown in Figs. 8 and 9, together with field measurements.

Agreement of results computed by FEM and the elastic method is generally good. Under the lower section (Fig. 8), where elastic behaviour prevails, the elastic method produced results largely within 10% of those from the more rigorous FEM. The differences are due to four sources of error: (1) simulation of the trapezoidal embankment load by three stages of rectangular loads; (2) limited precision in the charts; (3) neglect of embankment rigidity; and (4) variation of modulus in the subsoil. Despite these sources of error, the elastic method is considered acceptable for a preliminary analysis.

When local failure took place under the higher section, the modified elastic method incorporating local failure also gave satisfactory results (Fig. 9). The local failure zone computed from the elastic method was slightly larger than that from the FEM shown in Fig. 10 so that the top two piezometer loca-

FIG. 5. Soil profile at South Gloucester.

FIG. 6. Soil profile at Kars bridge.

FIG. 7. Construction schedules for the three embankments.

FIG. 8. Comparison of measured with estimated excess pore pressures beneath the 2.75 m high Boundary Road embankment at the end of construction.

tions below the shoulder were within this zone. All the equations developed for this modified elastic method were therefore utilized. The results seem to be even better than those for the lower embankment.

A comparison of estimated with measured excess pore pressures shows good agreement, except near the drainage boundary and beyond the shoulder. Near the drainage boundary the measured values are,

FIG. 9. Comparison of measured with estimated excess pore pressures beneath the 4.0 m high Boundary Road embankment at end of construction.

FIG. 10. Zone of local failure under the 4.0 m high Boundary Road embankment.

as expected, smaller than the estimated values, which is due to the dissipation of excess pore pressures during construction. To account for this dissipation, the method described in the previous section was applied. The coefficient of consolidation was obtained from independent laboratory measurements

FIG. 11. Excess pore pressures below centreline of the 2.75 m high Boundary Road embankment.

FIG. 12. Excess pore pressures below centreline of the 4.0 m high Boundary Road embankment.

of the coefficient of permeability k and the coefficient of volume change m_v . The k values were determined from soil specimens consolidated in a triaxial cell to stresses they carry in the field. The m_v values were measured on Osterberg samples tested in a conventional oedometer (5.0 cm diameter by 2.0 cm thick). This oedometer size, however, would overestimate m_v by a factor of about 2 (Lo *et al.* 1976). The m_v used in calculating c_v was therefore reduced by one half that of the oedometer tests. From 12 pairs of tests, c_v was determined to be $8.85 \times 10^{-3} \text{ cm}^2/\text{s}$. Based on this value, the estimated excess pore pressures taking into account the dissipation at the end of construction are shown in Figs. 11 and 12. Very good agreement exists under both sections.

The measured excess pore pressures beyond the

FIG. 13. Comparison of measured with estimated excess pore pressures beneath the Gloucester test fill at end of construction.

shoulder are, generally, higher than the estimated values. This may be due to the Mandel-Cryer effect in this region (Schiffman *et al.* 1969), in which both the total and shear stresses increase appreciably some time after the start of loading. This would increase the excess pore pressure over that for the conditions of constant shear stress assumed in the FEM and the elastic method. In addition, the rotation of principal stresses in this region may also increase pore pressure response.

Gloucester Test Fill (Bozozuk and Leonards 1972)

This test fill, 3.8 m high, is founded on similar subsoils (Fig. 5) about 6.5 km west of the Boundary Road embankment. Fill geometry and piezometer locations are shown in Fig. 13. The fill was constructed in a 1.2 m deep excavation and this caused an initial reduction in total stress and pore pressure. To eliminate ambiguity in time and excess pore pressure, all readings at the beginning of refilling were taken for reference. Excess pore pressure therefore refers to change of pore pressure during construction of the entire fill.

Analysis was carried out using FEM; the modulus for each subsoil layer was determined from laboratory tests. Only a very small zone of local failure was detected. The estimated excess pore pressures based on the two methods are compared with measured values in Fig. 13. Again, there is acceptable agreement between the results computed by FEM and

FIG. 14. Excess pore pressures below centreline of Gloucester test fill at end of construction.

those of the elastic method. Furthermore, the estimates are generally quite close to the measured values except near the drainage boundary and beyond the shoulder of the fill.

At the end of construction the predicted excess pore pressures below the centreline were compared with observed values in Fig. 14. Very good agreement may be noted.

Kars Bridge Embankment (Eden and Poorooshasb 1968)

A granular approach fill for a bridge near Kars, 40 km south of Ottawa, was built in two stages. The first reached a height of 6.1 m, and the final stage raised it to 8.0 m 20 months later. The geometry of the fill and the piezometer locations are shown in Fig. 15. The reported excess pore pressures refer to the first stage.

The subsoil (Fig. 6) comprises a 6 m layer of fissured crust lying over 10.7 m of soft marine clay similar to that at both South Gloucester and Boundary Road. FEM analysis showed no zone of local failure under the first stage of construction. The shear stress level R is generally high, averaging about 0.9 below the centreline. In the modified elastic method the presence of the thick, stiff crust was not accounted for and the estimated excess pore pressure was lower than that estimated from FEM (Fig. 15). In comparing estimated with observed excess pore pressures it was found that unlike the previous cases the measured excess pore pressures were smaller at most piezometer locations. The analysis incorporating dissipation of pore pressures during construction revealed the reason for these high estimates, as follows.

The coefficient of consolidation of this subsoil is very high ($0.12 \text{ cm}^2/\text{s}$) because of the thick fissured crust. Using this value, the estimated excess pore pressures at the end of construction agree much

FIG. 15. Comparison of measured with estimated excess pore pressures beneath the Kars bridge embankment at end of construction.

FIG. 16. Excess pore pressures below centreline of Kars bridge embankment at end of construction.

better with measured values (Fig. 16). Both the measured and estimated pore pressures at the middle of the clay layer, however, are significantly smaller than the maximum possible for undrained conditions. At the end of construction, therefore, the excess pore pressures through the entire subsoil have dissipated to some extent under the influence of the embankment load; hence the lower measured values.

Conclusions

Analysis of laboratory and field experiments on the generation of excess pore pressure in soft marine clay leads to the following conclusions.

1. The laboratory test data showed that excess pore pressures were higher under the axisymmetric state than under the plane strain state.
2. The excess pore pressure response was elastic ($\alpha = 0$) at failure and below elastic ($\alpha < 0$) before failure. It can be described by

$$\Delta u = \frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) + 0.24(R - 1) \times [(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2]^{1/2}$$

Symbols are defined in the Nomenclature.

3. The modified elastic method, using Poulos' (1967) charts, gave undrained pore pressure estimates comparable to those from the finite element method incorporating local failure and embankment rigidity.

4. Dissipation of excess pore pressure under the embankment was significant at the end of construction.

5. The estimated excess pore pressure accounting

for consolidation during construction compares well with the observed values along the centreline at the end of construction.

6. The estimated excess pore pressures were generally higher than measured values beyond the shoulder except near the drainage boundary.

Acknowledgements

The idea of significant excess pore pressure dissipation during construction was conceived at The University of Western Ontario while the first author was a research student under the supervision of Professor K. Y. Lo.

This paper is a contribution from the Division of Building Research, National Research Council of Canada, and is published with the approval of the Director of the Division.

- BJERRUM, L. 1954. Geotechnical properties of Norwegian marine clays. *Géotechnique*, 4(2), pp. 49-69.
- . 1972. Embankment on soft ground. State-of-the-Art Report. Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-supported Structures, Lafayette, IN, Vol. II, pp. 1-54.
- BOZOUK, M. 1974. Minor principal stress measurements in marine clay with hydraulic fracture tests. Proceedings, Engineering Foundations Conference on Subsurface Exploration for Underground Excavation and Heavy Construction, Henniker, NH, pp. 333-349.
- . 1977. Evaluating strength tests from foundation failures. 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, Vol. 1, pp. 55-59.
- BOZOUK, M., and LEONARDS, G. A. 1972. The Gloucester test fill. Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-supported Structures, Lafayette, IN, Vol. I(1), pp. 299-317.
- BURLAND, J. B. 1971. A method of estimating the pore pressures and displacements beneath embankments on soft, natural clay deposits. Proceedings, Roscoe Memorial Symposium, Cambridge, England, pp. 505-536.
- D'APPOLONIA, D. J., LAMBE, T. W., and POULOS, H. G. 1971. Evaluation of pore pressure beneath an embankment. Proceedings, ASCE Journal of the Soil Mechanics and Foundations Division, 97(SM6), pp. 881-897.
- DASCAL, O., and TOURNIER, J. P. 1975. Embankments on soft and sensitive clay foundation. Proceedings, ASCE Journal of the Geotechnical Engineering Division, 101(GT3), pp. 297-314.
- EDEN, W. J., and POOROOSHASB, H. B. 1968. Settlement observations at Kars bridge. *Canadian Geotechnical Journal*, 5, pp. 28-45.
- FOSS, I. 1972. Measurements on two test fills in Drammen. Norwegian Geotechnical Institute, Oslo, Norway, Technical Report No. 12.
- HALD, A. 1952. Statistical theory, with engineering applications. John Wiley & Sons, Inc., New York, NY.
- HENKEL, D. J. 1960. The shear strength of saturated remoulded clays. Proceedings, ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, CO, pp. 533-554.
- HOEG, K. H., CHRISTIAN, J. T., and WHITMAN, R. V. 1968. Settlement of strip load on elastic-plastic soil. Proceedings,

- ASCE Journal of the Soil Mechanics and Foundations Division, **14**(SM2), pp. 431-445.
- LAW, K. T. 1974. Analysis of embankments on sensitive clays. Ph.D. thesis, The University of Western Ontario, London, Ont.
- LAW, K. T., and LO, K. Y. 1976. Analysis of shear-induced anisotropy in Leda clay. Proceedings, 2nd International Conference on Numerical Methods in Geomechanics, Blacksburg, VA, Vol. 1, pp. 329-344.
- LAW, K. Y., BOZOUK, M., and EDEN, W. J. 1977. Measured strengths under fills on sensitive clay. Proceedings, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, Vol. 1, pp. 187-192.
- LEROUËL, S., TAVENAS, F., TRAK, B., LA ROCHELLE, P., and ROY, M. 1978. Construction pore pressures in clay foundations under embankments. Part I: the Saint-Alban test fills. Canadian Geotechnical Journal, **15**, pp. 54-65.
- LO, K. Y., BOZOUK, M., and LAW, K. T. 1976. Settlement analysis of the Gloucester test fill. Canadian Geotechnical Journal, **13**, pp. 339-354.
- MASSACHUSETTS INSTITUTE OF TECHNOLOGY. 1975. Report No. FHWA-RD-75-516. Proceedings, Foundation Deformation Prediction Symposium, Cambridge, MA, Vols. 1 and 2.
- MOH, Z. C., BRAND, E. W., and NELSON, J. D. 1972. Pore pressures under a bund on soft fissured clay. Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-supported Structures, Lafayette, IN, Vol. 1(1), pp. 243-272.
- MORGENSTERN, N. R., and PHUKAN, T. 1968. Stresses and displacements in a homogeneous non-linear foundation. Proceedings, International Symposium on Rock Mechanics, Madrid, Spain, pp. 313-320.
- OSTERBERG, J. O. 1952. New piston tube sampler. Engineering News-Record, **148**, pp. 77-78.
- POULOS, H. G. 1967. Stresses and displacements in an elastic layer underlain by a rough rigid base. Géotechnique, **17**, pp. 378-410.
- SCHIFFMAN, R. L. 1958. Consolidation of soil under time-dependent loading and varying permeability. Proceedings, Highway Research Board, **37**, pp. 584-617.
- SCHIFFMAN, R. L., CHEN, A. T. F., and JORDAN, J. C. 1969. An analysis of consolidation theories. Proceedings, ASCE Journal of the Soil Mechanics and Foundations Division, **95**(SM1), pp. 285-312.

Appendix A

Derivation of Excess Pore Pressure Expressions after Local Failure Using Computed Elastic Stresses

From [6]

$$\Delta u = \frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)$$

and with $v = 0.5$

$$[A1] \quad \Delta u = 0.5(\Delta\sigma_1 + \Delta\sigma_3)$$

As the right side is a stress invariant, the equation can be rewritten as

$$[A2] \quad \Delta u = 0.5(\Delta\sigma_v + \Delta\sigma_h)$$

In the elastic method (no consideration for local failure) the stress increments ($\Delta\sigma_v^e$ and $\Delta\sigma_h^e$) have to

be modified before they can be substituted in [A2]. Two conclusions from numerical analyses are employed. The first is that change of vertical stress is relatively insensitive to the assumed stress-strain characteristics even if local failure occurs (Hoeg *et al.* 1968; Morgenstern and Phukan 1968). This leads to

$$[A3] \quad \Delta\sigma_v^e = \Delta\sigma_v$$

or

$$[A4] \quad \sigma_v^e = \sigma_v$$

The second conclusion (Law 1974) shows that there is no significant difference in the orientation of the final principal stresses from the elastic and the elastoplastic analyses below the loaded area. From this, it can be established that

$$[A5] \quad \frac{\sigma_v - \frac{\sigma_1 + \sigma_3}{2}}{\frac{\sigma_1 - \sigma_3}{2}} = \frac{\sigma_v^e - \frac{\sigma_1^e + \sigma_3^e}{2}}{\frac{\sigma_1^e - \sigma_3^e}{2}}$$

The left side refers to the elastoplastic case, and the right side to the elastic case. By definition

$$[A6] \quad R = (\sigma_1^e - \sigma_3^e)/(\sigma_1 - \sigma_3)$$

Substituting [A6] in [A5] gives

$$[A7] \quad \sigma_v - (\sigma_1 + \sigma_3)/2 = (1/R)[\sigma_v^e - (\sigma_1^e + \sigma_3^e)/2]$$

From the property of stress invariant,

$$[A8] \quad (\sigma_v - \sigma_h)/2 = (1/2R)[\sigma_v^e - \sigma_h^e]$$

Substituting [A1], [A3], and [A4] in [A8] and rearranging gives

$$[A9] \quad \Delta u = \Delta\sigma_v^e \left(1 - \frac{1}{2R}\right) + \frac{1 - K_0}{2} \sigma_{v0}' \left(1 - \frac{1}{R}\right) + \frac{\Delta\sigma_h^e}{2R}$$

which is [8].

Along the centreline,

$$[A10] \quad R = [\Delta\sigma_v^e - \Delta\sigma_h^e + (1 - K_0)\sigma_{v0}']/2S_u$$

Substituting [A10] in [A9] gives [9], which is

$$[A11] \quad \Delta u = \Delta\sigma_v^e + [(1 - K_0)/2]\sigma_{v0}' - S_u$$

A similar method was used by Burland (1971), but no explicit excess pore pressure expressions were derived.

Nomenclature

c_v	= coefficient of consolidation
E	= undrained Young's modulus
$2H$	= thickness of compressible layer
k	= coefficient of permeability
K_0	= coefficient of earth pressure at rest
m_v	= coefficient of volume change
R	= shear stress level
S_u	= undrained shear strength
t_c	= time elapsed after start of construction
T_v	= time factor
u_b, u_t	= initial excess pore pressures at the bottom and top drainage boundaries, respectively

Δu	= excess pore pressure
α	= pore pressure parameter
α_f, α_t	= pore pressure parameters at failure and at working stress, respectively
ν	= undrained Poisson's ratio
σ_{v0}'	= initial vertical effective pressure
$\Delta\sigma_h, \Delta\sigma_v$	= stress increments in the horizontal and vertical directions, respectively
$\Delta\sigma_1, \Delta\sigma_2, \Delta\sigma_3$	= major, intermediate, and minor principal stress increments, respectively

Superscripts

e	= elastic range
f	= failure condition
p	= plastic range