Embankment Settlement Including Delayed Compression

by

P. W. TAYLOR, B.Sc., B.E., Ph.D., F.N.Z.I.E.

Associate Professor of Civil Engineering, University of Auckland, N.Z.

B. G. McLISTER, B.E.

Post Graduate Student, University of Auckland, N.Z.

and

G. R. W. EAST, B.Sc.

Engineering Scientist, Ministry of Works and Development, Auckland, N.Z.

SUMMARY. A method of computation of the amount and rate of one-dimensional consolidation settlement is presented. Equipment and methods of determining the required soil parameters are described. The method is then applied to estimate the settlement of a motorway embankment founded on deep clays, apparently normally-consolidated, in a volcanic explosion crater in Auckland.

1 INTRODUCTION

Settlement calculations are most commonly made using Terzaghi's original theory, on the assumption that primary consolidation is the main source of settlement. Occasionally, "secondary" consolidation is considered, when the assumption is made that this does not commence until 90% of the primary settlement has occurred. This is done to simplify calculations, although it has been realised for some time that the two forms of settlement could occur simultaneously. In 1942, D.W. Taylor published results of fundamental research at MIT on this topic (Ref. 1), but it was not until 1967 that Bjerrum (Ref. 2) suggested a revision of terminology and proposed the terms instant and delayed compression to refer respectively to strain which is dependent on change of effective stress, and to strain occurring as a function of time at constant effective stress. Delayed compression becomes of importance when estimating consolidation settlement of a structure founded on soils which are geologically normally-consolidated. As a result of delayed compression, such soils will behave as though they are preconsolidated, and are termed "apparently" preconsolidated.

Bjerrum did not present a method of calculation in which both instant and delayed compression were taken into account simultaneously. A basic method of accomplishing this using a numerical procedure was presented by Garlanger (Ref. 3).

Here, an extension of Garlanger's model is used to form the basis of a computer program for the calculation of settlement under an embankment. The development of a controlled gradient consolidometer to provide the required soil parameters is described.

Using the new method, estimates are made of the settlement of a motorway embankment constructed on a deep deposit of clay known from geological evidence to be normally loaded. The clay was deposited in the explosion crater of an extinct volcano and its age is known at least approximately. The settlement estimates are compared with field measurements over a period of two years. As significant settlement is expected to continue over several decades, the conclusions drawn at this stage can be only tentative.

2 INSTANT - DELAYED COMPRESSION THEORY AND METHOD OF TEST.

(a) Basic Theory

D.W. Taylor introduced the idea of delayed or secondary compression leading to an apparent preconsolidation pressure, but he did not combine the effects of the preconsolidation pressure and secondary compression in any subsequent consolidation. To replace the terms primary and secondary consolidation which referred to two parts of compression occurring separately, Bjerrum defined the following terms:

(i) 'instant compression', which occurs simultaneously with increase in effective pressure and causes a reduction in void ratio until an equilibrium value is reached at which the structure effectively supports the overburden pressure, and -

(ii) 'delayed compression' representing the reduction in volume at unchanged effective stresses.

Fig. 1 is assumed to represent the consolidation history of any point in an apparently normally-consolidated stratum, as well as its behaviour under

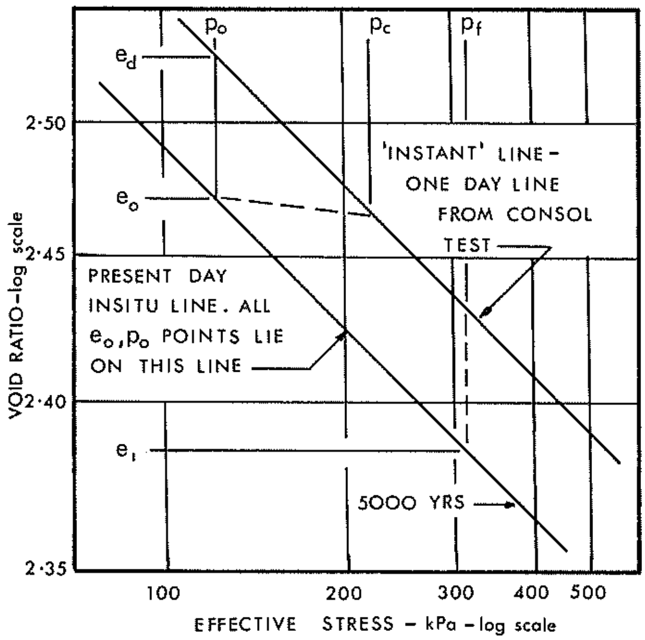

Fig. 1 Consolidation History of a Point in a Normally Consolidated Stratum

a new load. It will be noted that the assumption is made that the log e vs. log p relationship is linear,

where e is the void ratio, and
p is the effective vertical stress.

The consolidation lines are assumed parallel on the log-log plot. Also, the rate of delayed compression (in terms of log e) is assumed to be linear with the logarithm of time.

Since deposition at effective stress p_o the void ratio of the point under p_o has reduced from e_d to e_o due to delayed compression occurring for say, 5000 years. On loading to p_f the void ratio change expected at the point after a further 5000 years would be $e_o - e_1$ although the e,p path taken by the point would not be known. By considering a number of points in the stratum, the settlement after 5000 years could be found. However, settlements for shorter times are of more interest. To calculate these requires a knowledge of the behaviour of void ratio with time, for several points in the stratum.

Assuming Darcy's law is valid, the soil saturated and that permeability may vary with depth, leads to the one-dimensional equation governing compression.

$$\frac{\rho_w g}{1+e_o} \frac{\partial e}{\partial t} = k \frac{\partial^2 u}{\partial z^2} + \frac{\partial k}{\partial z} \frac{\partial u}{\partial z} \qquad (1)$$

where u is excess pore pressure
k is permeability
z is depth
t is time
ρ_w is mass density of water
g is acceleration due to gravity
e_o is initial void ratio

The rate of void change is made up of instant and delayed portions

$$\frac{\partial e}{\partial t} = \left(\frac{\partial e}{\partial t}\right)_i + \left(\frac{\partial e}{\partial t}\right)_d$$

or

$$\left(\frac{\partial e}{\partial t}\right)_i = n \frac{\partial e}{\partial t}$$

where

$$n = \frac{\frac{\partial e}{\partial t} - \left(\frac{\partial e}{\partial t}\right)_d}{\frac{\partial e}{\partial t}}$$

$\left(\frac{\partial e}{\partial t}\right)_i$ is accompanied by a change in pore pressure.

$$\left(\frac{\partial e}{\partial t}\right)_i = -\frac{\partial e}{\partial p} \frac{\partial u}{\partial t} \quad \text{as} \quad \frac{\partial e}{\partial p} = -\frac{\partial e}{\partial u}$$

$$\therefore \quad \frac{\partial e}{\partial t} = -\frac{1}{n} \frac{\partial e}{\partial p} \frac{\partial u}{\partial t}$$

Substituting into equation 1 gives

$$k \frac{\partial^2 u}{\partial z^2} + \frac{\partial k}{\partial z} \frac{\partial u}{\partial z} = \left(-\frac{1}{n} \frac{\partial e}{\partial p}\right) \frac{\partial u}{\partial t} \cdot \frac{\rho_w g}{1+e_o} \qquad (2)$$

Solving this consolidation equation by numerical methods allows evaluation of the coefficients at each step. The coefficients $\partial e/\partial p$ and n can be found by the methods proposed by Garlanger:

$$\frac{\partial e}{\partial p} = \frac{-be}{p}$$

if consolidation occurs on the 'instant' line. b is the gradient of the 'instant' line.

$$\frac{\partial e}{\partial p} = \frac{-ae}{p}$$

if consolidation is not occurring on the instant line, a is gradient of compression at pressures less than the preconsolidation pressure.

From the geometry of the log e vs. log p diagram and the assumed rate of delayed compression,

$$\left(\frac{\partial e}{\partial t}\right)_d = \frac{-ce}{t_i} \left(\frac{e}{e_c}\right)^{1/c} \left(\frac{p}{p_c}\right)^{b/c}$$

where c is the rate of delayed compression,
t_i is the age of the instant line,

and $$e_c = e_o \left(\frac{p_c}{p_o}\right)^{-a}$$

Knowing $\left(\frac{\partial e}{\partial t}\right)_d$ and $\frac{\partial e}{\partial t}$ from equation 1 allows n to be found after each time increment in the numerical analysis procedure described below.

The permeabilities can be interpolated from a permeability-void ratio relationship. Here, it has been assumed that the relationship between log k and e is linear. Equation 2 can be solved for any chosen boundary conditions.

A computer program, RESP, has been written which incorporates the above assumptions for an embankment loading and uses an explicit finite difference solution. This is a step-by-step process in which soil parameters depending on void ratio or effective stress, or both, are re-evaluated after each time increment. The soil profile may consist of several soil strata which may be normally consolidated (with or without delayed compression) or may be truly over-consolidated. For computation purposes, each stratum is subdivided into six layers. The stress changes under the centre-line of the embankment are calculated from the theory of elasticity and loads may be applied over a finite period. The soil parameters required for the analysis are:-

(i) The gradient, a, of the laboratory log e vs. log p curve at pressures less than p_c the preconsolidation pressure.

$$a = \Delta\log e/\Delta\log p$$

(ii) Similarly, b, the gradient above p_c

$$b = \Delta\log e/\Delta\log p$$

(iii) The delayed compression parameter, c

$$c = \Delta\log e/\Delta\log t$$

(iv) The preconsolidation pressure p_c.

(v) Soil permeability, k, and its variation with void ratio.

(vi) Pore pressure parameter A.

All parameters except A can be found from a consolidation test. A is required to determine the pore pressures induced by embankment loading and may be found from a triaxial test.

The accuracy of a settlement prediction using the above method depends largely on the accuracy of the determination of k and p_c, parameters that can be difficult to obtain accurately using conventional testing methods. A continuous consolidation test with small hydraulic gradients should improve accuracy. One such test is the controlled gradient test, originally proposed by Lowe et al (Ref. 4).

(b) The Controlled Gradient Testing Apparatus

In this apparatus, one face of the sample is undrained and pore water pressure is measured at this face. The other face is drained to a constant back-pressure and the rate of increase of stress on the sample is controlled so that a constant pressure difference is maintained across the sample.

The apparatus built uses a modified hydraulically-loaded Rowe cell. The load is applied through a variable pressure source which is controlled by the pore pressure regulator - Fig. 2. The regulator developed is basically a U-tube of mercury in which one column of mercury may be raised higher than the other, the difference in height h_m being a measure of the pore pressure difference across the sample. When the pore pressure at the undrained face falls, the lower mercury column rises

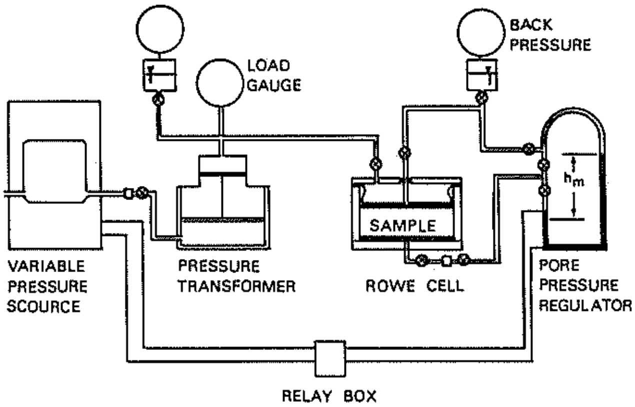

Fig. 2 The Controlled Gradient Apparatus

and makes contact with a platinum wire in a capillary tube. The contact completes a circuit which actuates the variable pressure source and increases the load. This increases the pore water pressure and breaks the mercury - wire contact. The load is then held constant until a further cycle occurs. In this way, an almost constant pressure difference can be maintained across the sample. Pore pressure differences of up to 30 kpa can be applied and the regulator maintains the value to within 1 kPa.

The applied stress, deformation, and pore pressure at the undrained face are continuously recorded. As many points as are required to fully define the e, p curve can be analysed. The permeability of the sample can be found from the test results and thus the void ratio - permeability relationship may be determined. Only a few points are required to define the e vs. log k line.

3 APPLICATION OF THE METHOD TO A SIX-LANE MOTORWAY EMBANKMENT

(a) Introduction

Associated with the Auckland Motorway scheme is the construction of a 6 lane, 700m long bridge across the Manukau harbour at Mangere. The northern approach abutment embankment crosses a 300m diameter. 60m deep, volcanic explosion crater infilled with a normally loaded deposit of marine and fresh water clay. Prior to motorway construction the crater known as Gloucester Park had a 2m veneer of rock fill.

The bridge and embankment heights were determined by the future development of the adjacent coastal shipping port of Onehunga. Initially a height of 15m above M.S.L. was required for a navigation span which entailed embankment heights across the crater of from 4 to 10m. Stability aside, the calculated high magnitude and the slow rate of settlement would have resulted in a very unsuccessful pavement requiring constant regrading. As they would not have affected the rate of delayed compression, sand drains were not considered to provide a satisfactory solution.

After discussion, the harbour authorities allowed the bridge to be lowered 3m. Combined with other modifications, this resulted in only 2 to 4m of embankment height being required across the crater with a maximum of 3m over the deepest infill. After conventional settlement and stability analyses the embankment construction was commenced in 1971. For reasons of stability and the close proximity of link ramps, the crater was completely blanketed by 1m of fill with embankments built up above this level. The bridge and motorway are scheduled for completion in 1976.

Settlements and settlement time gradients were found to be considerably greater than predicted by conventional analysis. The computer program incorporating the 'instant-delayed compression' method of analysis was being developed at that time and the Gloucester Park problem provided an interesting opportunity for its use.

(b) Geological History

The geology of Auckland is dominated by the effects of volanic activity. Eruptions have occurred since the end of the Pleistocene period almost to the present day (Ref. 5). The last volcano to erupt, Rangitoto, was active approximately 800 years ago. More than fifty small volcanoes have erupted in an area of only 200 square miles.

The Auckland volcanoes are either explosion craters or scoria cones with associated basalt lava flows. Many of the explosion craters have been infilled with lacustrine and marine sediments. The basement rock underlying these volcanoes consists of soft sandstones and siltstones of the Waitemata Group deposited during the Miocene period.

Gloucester Park is one of the infilled explosion craters. Geological evidence suggests that the explosion at Gloucester Park occurred less than 20,000 years ago and removed over 50m of material. The basalt of the eruption has its drawback surface 60m below present sea level. It is unweathered, highly vesicular and very permeable. The walls of the crater consist of interbedded sandstone and siltstone of the Waitemata Group with some overlying Pleistocene alluvials. A shallow tuff ring surrounds the crater.

It is thought that the explosion occurred during the last glacial period when the sea level was up to 100m below the present level and allowed a fresh water lake to form in the crater. Vegetation developed round the lake to form a dark brown laminated organic lacustrine clay stratum of present thickness 20m. Interspersed are thin sand bands, and the stratum contains plant remains including relatively undecayed but well compressed leaves. Some solid wood was found in the stratum near the crater edge. Gas is being generated by this layer.

During the post-glacial rise in sea level, the crater was breached and infilled with up to 45m of Recent grey marine clay. This clay has its surface at the present sea level. The top 3-5m of marine clay are overconsolidated from the effects of desiccation. The upper surface of the organic clay indicates that considerable settlements occurred in this material during the deposition of the marine clay.

In recent years, Gloucester Park has been used for a variety of purposes. Up to the 1920's the basin was tidal and small boats were moored in it. Then the sea entrance was blocked and the basin filled with 2-3m of rubble fill and the tuff ring was used as a motor cycle raceway for several years. A locality plan showing the geology of the area is presented in Figure 3 while a cross-section of the site is shown in Figure 4.

(c) Geotechnical Properties and Settlement Parameters

The geotechnical properties presented in Fig. 5 show that the marine clay is homogeneous and highly compressible. The organic clay also has high compressibility but is comprised of layers only a few millimetres in thickness of varied composition.

From a geological point of view both the marine and fresh water clays are normally consolidated. In contrast there exists an apparent preconsolidation pressure that is slightly greater than the present overburden pressure ($p_c/p_o = 1.2$). This phenomenon is even more marked if related to the overburden 40 years ago (prior to rubble fill) ($p_c/p_o = 1.7$). If, as geologically estimated, the clays have been relatively rapidly deposited 3,000 to 5,000 years ago, when sea levels rose, the instant-delayed compression theory is applicable to these soils.

Settlement was estimated by the usual method,

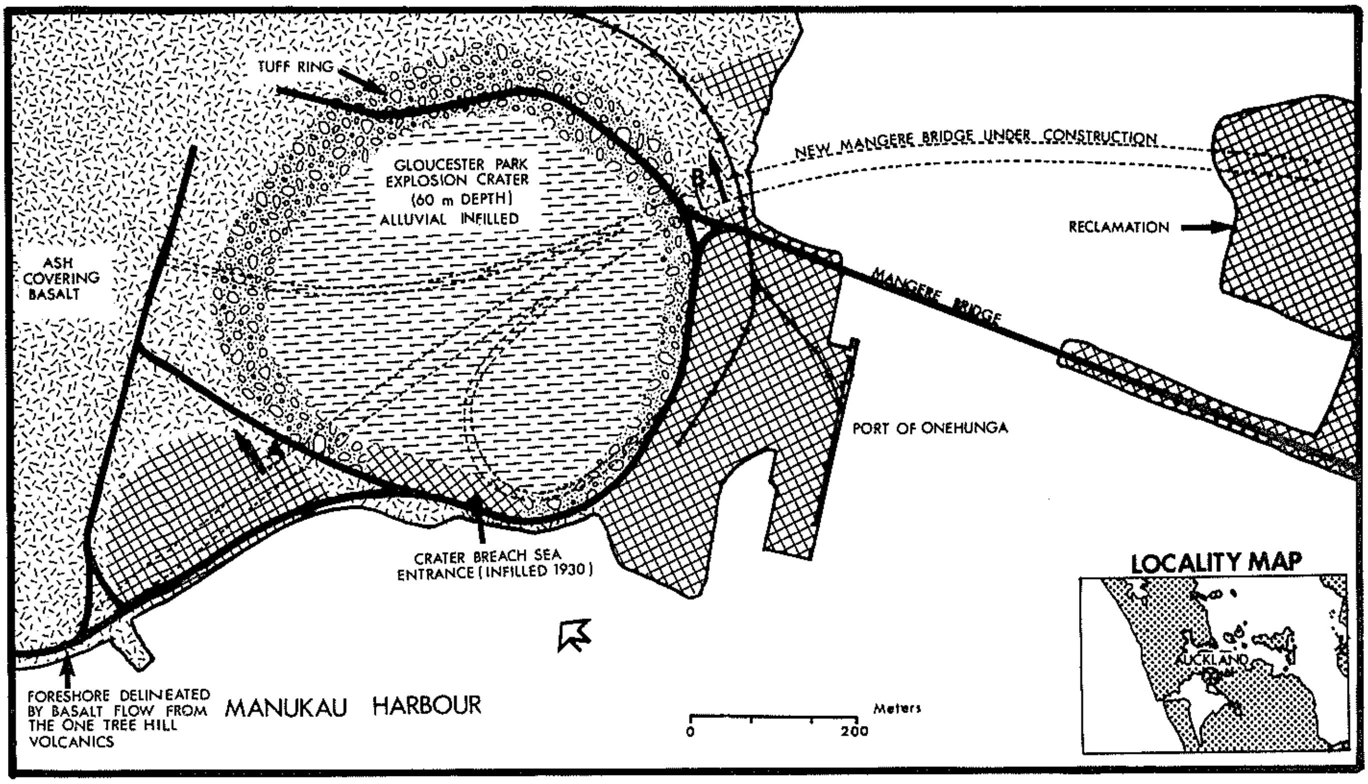

Fig. 3 Locality Plan and Geology of the Motorway Area

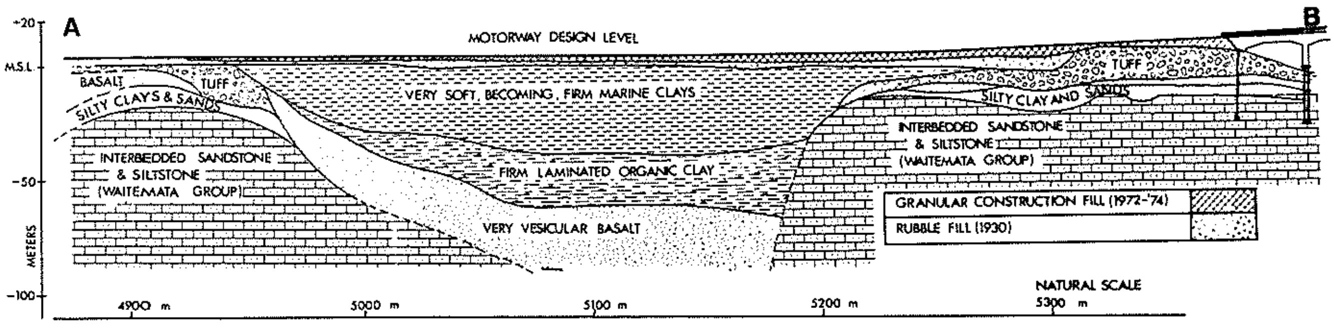

Fig. 4 Cross-section of the Gloucester Park Explosion Crater Along Motorway Centre Line

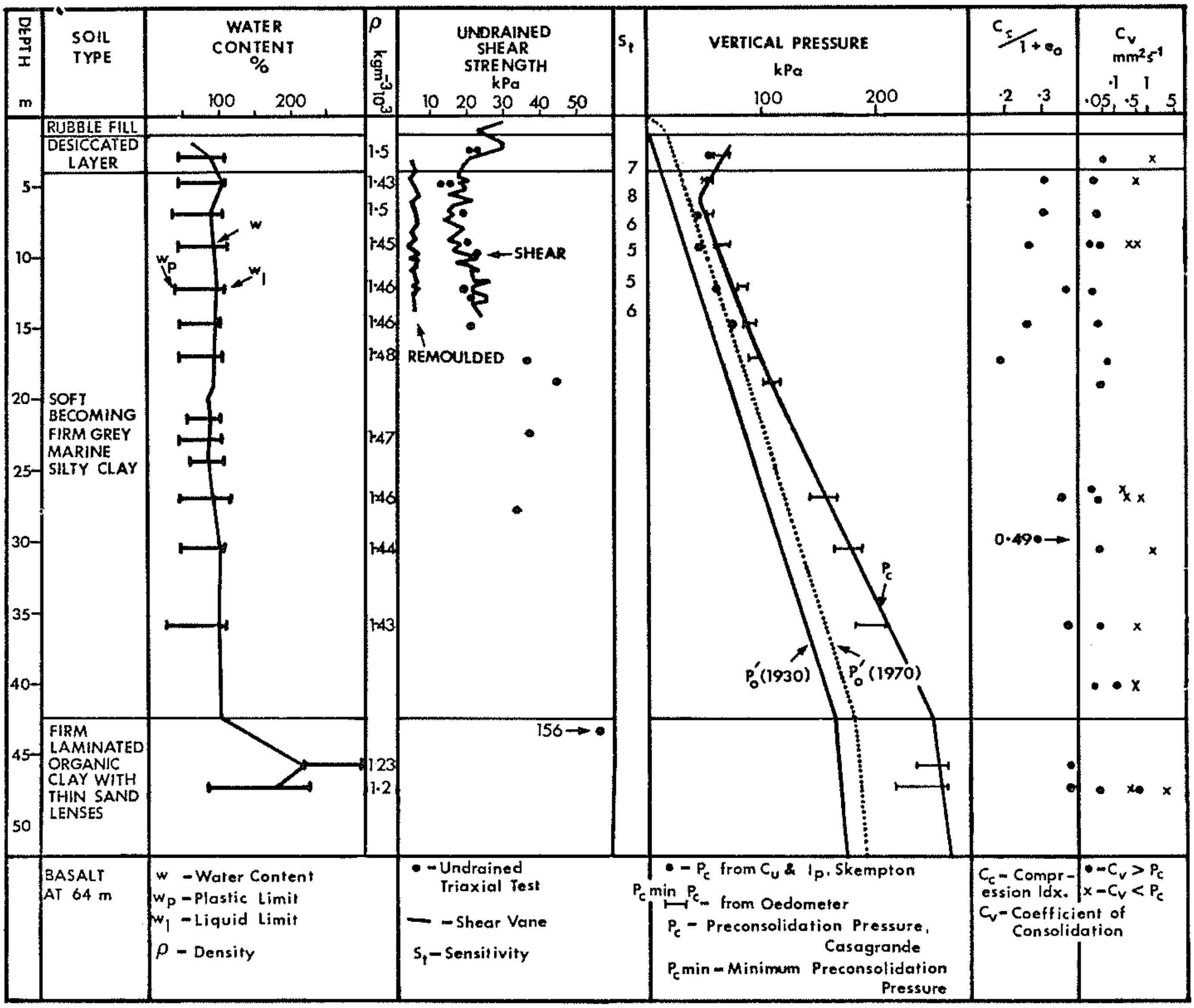

Fig. 5 Typical Geotechnical Properties of The Explosion Crater

with allowance for delayed compression after 90% primary settlement, referred to as "conventional analysis" later in this paper, but the measured settlement and settlement-time gradients are considerably greater than predicted in the first two years. The geometry of the embankment is such that undrained creep (distortion) cannot account for this additional settlement. As the depths of compressible soils are large it was considered that drained creep or seconday consolidation may be an important factor. The instant-delayed compression method of analysis was used with the following parameters derived from tests:

	Marine Clay	Organic Clay
a	0.01	0.04
b	0.30	0.20
c	0.012	0.003
$k_{av}(ms^{-1})$	0.6×10^{-9}	0.3×10^{-8}

(d) Instrumentation

Settlement plates were installed prior to the construction of the embankments, but no piezometers. During the reappraisal of the settlement problem in 1974, electronic vibrating-wire piezometers were placed at various levels in the centre of the crater. These indicated surprisingly high pore pressures even after a considerable settling down period. As these piezometers have low air entry sintered bronze tips, it was considered that gas pressure was being measured, rather than water pressure. This had been expected in the organic clay but not in the marine clay. Conventional hydraulic piezometers with high air entry tips were then installed. (Results from these are shown in Figure 7.)

(e) The Settlement Analyses

The Gloucester Park settlement analysis has been carried out using program RESP. The site details are:

2 layers of compressible material
Layer 1 - 37m of marine clay
Layer 2 - 23m of organic clay

The depth of the water table beneath the surface has been assumed to be 4m and the overburden pressure at the water-table has been taken to be 20 kPa. These values are chosen to take into account the desiccated crust of the marine clay. Drainage has been assumed for both the top and the bottom of the soil profile.

The loads involved in the analysis are -

(1) 1.8m of rubble fill producing a vertical stress increase of 27 kPa. This load will exceed the apparent preconsolidation pressure at shallow depths only. The analysis starts with the placement of this rubble fill (in 1930) and is followed 42 years later by either:

(2i) A 2.1m embankment comprising a 1.6m fill and 1.2 years later a further 0.5m producing a stress increase of 30.8 kPa. The total load applied exceeds the preconsolidation presure for the upper 20m of marine clay,

(2ii) Or for comparison, the originally designed 10m embankment causing a stress increase of 145 kPa. This embankment would cause the preconsolidation pressure to be exceeded at all depths.

The settlement graphs presented in Figures 6 and 8 show the settlements subsequent to embankment construction.

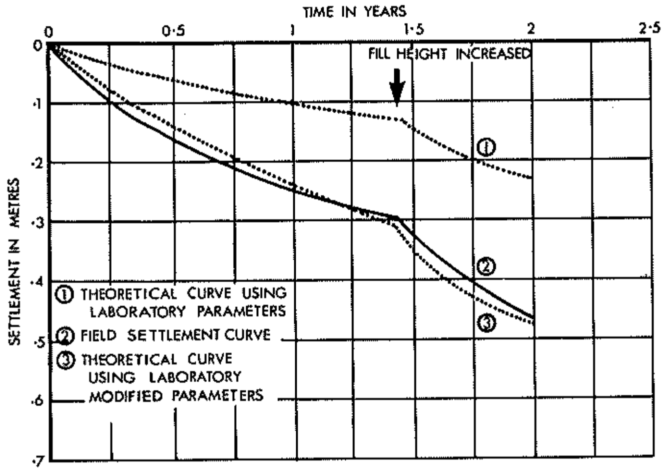

Fig. 6 Actual and Analysed Settlement During the First Two Years

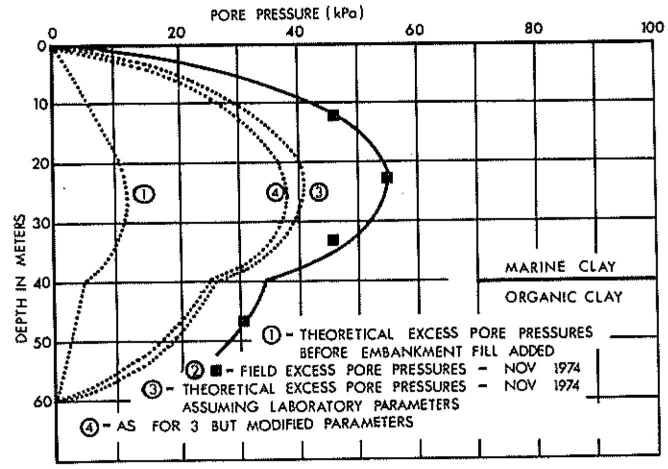

Fig. 7 Actual and Theoretical Excess Pore Pressures

At that time, settlement caused by the rubble fill was calculated to be 1.6m. No records are available to check this value. The pore pressures calculated to be remaining at that time (1972) are presented in Figure 7. In the analyses no account has been taken of immediate settlement or undrained creep under the embankment but these should be negligible for the reasons given previously.

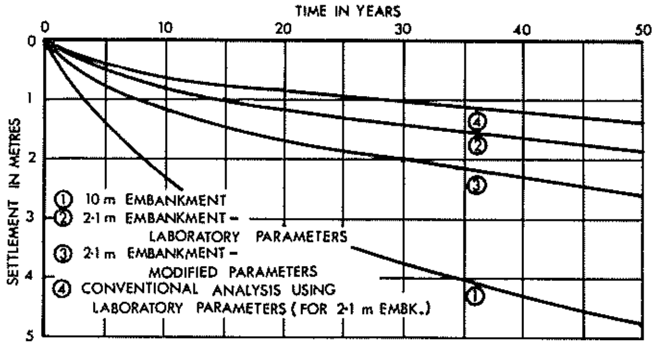

Fig. 8 Predicted Settlement Over a 50 Year Period

The field consolidation behaviour has been compared with the instant-delayed compression analysis results for the two years since construction was commenced. Figure 6 shows that the settlement of the 2.1m embankment is twice the predicted settlement in the first two years. Also, the pore water pressures recorded (Figure 7) show excess pore pressures up to 25% higher than those predicted.

To attempt to match the field results, the soil parameters were realistically modified. The revised parameters $b = 0.430$, $c = 0.017$ and $k_{av} = 0.75 \times 10^{-9}\ ms^{-1}$ were used for the marine clay layer and, as shown in Figure 6, gave a good approximation to the field settlements but the corresponding excess pore pressure distribution (Figure 7), still shows pressures less than the field values.

The change in b (from 0.30 to 0.43) represents a change of compression index from 1.37 to 1.70. The 50 year settlement prediction for the 2.1m embankment is shown in Figure 8. The modified parameters show settlements approximately 1.5 times those found from the original parameters. Settlement of the 10m embankment using the original parameters is shown for comparison.

Also shown in Figure 8 is the result of the conventional settlement analysis carried out for the 2.1m embankment using the original properties. The method took into account the preconsolidation of the clays and secondary compression was added to the primary consolidation when the average degree of consolidation of a stratum was 50%.

It can be seen that this approach grossly underestimates the first stage of the field settlement.

4 CONCLUSIONS

The results indicate the problems of predicting the first stages of settlement after loading. In the embankment example used the laboratory parameters underestimated settlement and overestimated the dissipation of pore pressure for the two year period after construction. Nevertheless the settlement curve is in better agreement with the field case than is the conventional analysis.

It should be noted that the instant-delayed compression analysis, as used here, assumes a mean constant compression gradient b. This has been obtained from the laboratory testing over a somewhat larger stress range than the field case. In detail, the laboratory compression curves indicate that a larger gradient would result from considering a small stress increment immediately

support for the research described.

6 REFERENCES

1. TAYLOR, D.W. Research on Consolidation of Clays, Dept. of Civil and Sanitary Engineering, M.I.T., Serial 82, 1942.

2. BJERRUM, L. Engineering Geology of Norwegian Normally-Consolidated Marine Clays as Related to Settlements of Buildings, Geotechnique, Vol. 17, 1967, pp. 81-118.

3. GARLANGER, J.E. The Consolidation of Soils Exhibiting Creep under Constant Effective Stress. Geotechnique, Vol. 22, 1972, pp. 71-78.

4. LOWE, J., JONAS, E. and OBRICIAN, V. Controlled Gradient Consolidation Test. Jnl. Soil Mech. and F.E.Div., A.S.C.E., Vol. 95, SM1, 1969, pp. 77-97.

5. SEARLE, E.J. A Geology of Auckland Auckland and Hamilton, Paul's Book Arcade, 1964.

inexcess of the preconsolidation pressure. The modified b parameter used to fit the field settlement curve lies in the range of the gradient found over this small stress increment, and in some tests this higher b value was found over larger stress increments. If the laboratory compression curve is to be used, it appears that the analysis could take into account the change in the gradient b with increasing effective stress. It is possible that in the example used the field settlement may approach the original laboratory parameter settlement curve with time.

In general the use of the instant-delayed compression analysis will lead to more meaningful predictions of settlement in compressible normally consolidated soils. The field cases where large unexplainable settlements have occurred with little pore pressure dissipation can be better analysed by this method.

5 ACKNOWLEDGEMENTS

The Authors thank the Ministry of Works and Development for permission to use the embankment data for this paper. Acknowledgement is made to the New Zealand National Roads Board of their