A STUDY OF PORE PRESSURES BENEATH AN OIL STORAGE TANK

A Thesis
Presented to
the Faculty of Graduate Studies and Research
University of Manitoba

In Partial Fulfillment
of the Requirements for the Degree
Master of Science

by
Rupert Kit-Yee Hon
September 1975

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A dissertation submitted to the Faculty of Graduate Studies of the University of Manitoba in partial fulfillment of the requirements of the degree of

MASTER OF SCIENCE

ACKNOWLEGEMENTS

The study was carried out under the supervision of Dr. L. Domaschuk. His patient guidance and valuable advice and criticism during the preparation of this thesis are highly appreciated. He is the one who deserves special thanks.

The writter wishes to express his gratefully appreciation to Professor A. Baracos for his helpful suggestions and encouragement.

Sincere thanks are given to Dr. B. Muir for reviewing the thesis and making useful comments.

The permission to use the oil-storage tank to obtain pore pressure data from Winnipeg Imperial Oil Company is acknowledged.

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NOTATIONS

A = Skempton's pore pressure parameter.

A_i = Skempton's pore pressure parameter considering incremental change in pore pressure.

B = Skempton's pore pressure parameter.

B_i = Skempton's pore pressure parameter considering incremental change in pore pressure.

a = Henkel's pore pressure parameter.

a_i = Henkel's pore pressure parameter considering incremental change in pore pressure.

b = Henkel's pore pressure parameter.

b_i = Henkel's pore pressure parameter considering incremental change in pore pressure.

α = Domaschuk's pore pressure parameter.

α_i = Domaschuk's pore pressure parameter considering incremental change in pore pressure.

β = Domaschuk's pore pressure parameter.

β_i = Domaschuk's pore pressure parameter considering incremental change in pore pressure.

p = Applied pressure.

P_o = Overburden pressure.

ΔP = Change in applied pressure.

E_t = Initial tangent modulus of elasticity.

E_s = Secant modulus of elasticity.

Δs = Vertical normal deflection.

Δu = Change in pore pressure.

Δu_a = Change in pore pressure caused by ambient stresses.

Δu_s = Change in pore pressure due to deflection of the soil.

e_1 = Major principal strain.

e_3 = Minor principal strain.

$\Delta \sigma_1$ = Change in major principal stress.

$\Delta \sigma_2$ = Change in intermediate principal stress.

$\Delta \sigma_3$ = Change in minor principal stress.

$\Delta \sigma_z$ = Change in vertical normal stress.

$\Delta \sigma_r$ = Change in radial stress.

$\Delta \sigma_t$ = Change in tangential stress.

$\Delta \sigma_m$ = Change in mean normal stress.

$s_d = \sqrt{(\Delta\sigma_1 - \Delta\sigma_m)^2 + (\Delta\sigma_2 - \Delta\sigma_m)^2 + (\Delta\sigma_3 - \Delta\sigma_m)^2}$

$s_{hd} = \sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_1 - \Delta\sigma_3)^2}$

$\Delta \sigma_x, \Delta \sigma_y, \Delta \sigma_z,$ = Change in normal stress on x, y, z, planes with reference to the x, y, z, co-ordinates.

$\Delta \tau_{xy}, \Delta \tau_{yz}, \Delta \tau_{xz},$ = Change in shear stress on x, y, z, planes with reference to the x, y, z, cor-ordinates.

SUMMARY

A study of pore pressures generated in a clay soil by boundary loads, was carried out by installing pizometers beneath an oil-storage tank. Pore pressure data were collected from March of 1971 to January of 1972 which was divided into five periods as follows;

1) From the time of the piezometer installation to the start of tank construction.

2) Tank construction, which was a period of 7 days.

3) Post tank construction, a period in which the tank was left empty for 14 days.

4) Tank testing period, during which the tank was filled with water in 7 days and was left full for 5 days, then was drained in 23 days and was left empty for another 12 days.

5) In service period, during which petroleum was added and removed daily, but with the amount removed less than the amount added, so that the tank was filled in 4 months.

The pore pressure data were analysed using Skempton's, Henkel's, Domaschuk's, and Lo's methods. The first three methods related the change in pore pressure to the change in ambient stresses, whereas in Lo's method, the change in pore pressure was related to the stresses and to the deflections of the soil. The analysis was carried out following two approaches. One approach was to assume that no consolidation occured and all pore pressure changes were computed relative

to the initial values. In the other approach, pore pressure changes were taken over time intervals in which there was a significant change in load, and it was thus assumed that no consolidation took place over the short interval, but that consolidation may have taken place prior to the load change. The stresses and deflections at the piezometer locations induced by the loads from the tank, were computed using the Boussinesq theory and the finite element method.

Average pore pressure parameters for Skempton's, Henkel's and Domaschuk's methods were obtained and are summarized in the table below;

Methods \ Piezometers		Assuming no consolidation		Assuming consolidation occurred	
		Parameters A, a, & α		Parameters A_i, a_i & α_i	
		P1	P2	P1	P2
Skempton (A)	B=1.0	0.66	0.65	0.70	0.51
	B=0.5	1.87	1.71	-	-
Henkel (a)	b=1.0	0.44	0.32	0.33	0.23
	b=0.5	0.53	0.50	-	-
Domaschuk (α)	β=1.0	0.64	0.61	0.28	0.25
	β=0.5	0.85	0.79	-	-

CHAPTER I

INTRODUCTION

In foundation engineering, the problems of primary concern are the bearing capacity of the soil and the settlement of the foundations. In cohesive soils, these two important factors are influenced by the pore-water pressure in the soil.

The importance of the existence of pore pressure in soil was realized by engineers as early as at the beginning of the 19th century. In 1809, Telford (27) was the first one to apply the method of preloading to consolidate a 55-foot layer of soft clay to increase the soil strength. Most engineers did not fully understand the true mechanisms of the effective stress, until the effective stress principle was put forward by Karl Terzaghi in 1923 (39). At the same time, the consolidation theory was developed which made it possible to compute the transfer of load from water to the soil skeleton as drainage of the pore-water occurred.

Pore-water pressure in soils varies with the change of the stress system which accompanies the application of loads. In order to prevent failure of foundations, the magnitude of pore pressure in field problems has been monitored by engineers during and after construction and necessary remedial measures were made, when the pore pressures were high.

In this thesis, several conventional methods used to predict the change in pore-water pressure associated with changes in stress are presented and the pore-water pressure developed under an oil storage tank in Winnipeg clay are analysed by these methods. The objective of the study is to establish the relationships between the change in pore pressure and the changes in stress and deflection, such that the pore pressure parameters for those conventional methods were obtained and they can readily be applied in pore pressure computations in Winnipeg clay.

CHAPTER II

METHODS AND ASSOCIATED EQUATIONS FOR COMPUTED PORE PRESSURE IN SOIL

SKEMPTON'S METHOD

The earliest equation relating the changes in pore pressure to the changes in principal stresses was suggested by Skempton in 1948 (38) for saturated soils. The relationship is given by equation (1).

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

in which A = pore pressure parameter

Δu = change in pore pressure

$\Delta\sigma_1$ = change in major principal stress

$\Delta\sigma_3$ = change in minor principal stress.

In 1954 (39), the equation was further developed for the case of partially saturated soils by introducing another pore pressure parameter B and the equation became

$$\Delta u = B\ (\ \Delta\sigma_3 + A\ (\ \Delta\sigma_1 - \Delta\sigma_3\ ) \quad \text{............. (2)}$$

$$= B\ \Delta\sigma_3 + \bar{A}\ (\ \Delta\sigma_1 - \Delta\sigma_3\ ) \quad \text{............... (3)}$$

in which B = pore pressure parameter

$\bar{A}$ = AB

The above proposed equation was based on several assumptions. These assumptions were: the elastic theory is valid for soils, the law of superposition is applicable, the change of intermediate principal stress has no effect on the pore-water pressure, and the solid particles in the soil are incompressible.

In applying Skempton's equation to a field problem, the pore pressure parameters A and B must first be determined in the laboratory.

Two methods can be adopted in the evaluation of the parameter B. They are as follows:

Method 1. By consolidation test.

When a soil element is subjected to loading, the change in volume of the soil skeleton must be equal to the change in volume of pore fluid and from this we get

$$B = \frac{1}{1 + n \frac{C_w}{C_c}} \qquad (4)$$

where n = porosity of soil

C_w = the compressibility of water

C_c = the compressibility of soil skeleton

In order to obtain the value B of soil, it is necessary to determine n, C_w, and C_c.

The compressibility of the soil skeleton can be determined from an isotropic, consolidation test. The soil is assumed to be isotropic, so that the compressibility is the same in all directions and may be determined from the change in volume, ΔV, which accompanies a change in effective stress, $\Delta\bar{\sigma}$ according to the equation

$$C_c = \frac{\Delta V}{V_o} \frac{1}{\Delta\bar{\sigma}} \qquad (5)$$

where V_o = original volume.

For anisotropic soils, the compressibility in a specific direction can be found by applying uniaxial loading in that direction in an odeometer drained test. The pore-water pressure in the specimen should always be kept zero. The change in volume is measured by the volume of water squeezed out when load is applied. With this data, the curve of unit volume versus effective stress is plotted. The compressibility is the slope of this curve. The same procedure can be applied to the other two directions.

As for the compressibility of water C_w, it is determined by an undrained isotropic compression test. The sample is assumed to be isotropic so that the strain is the same in any direction. The cell pressure is increased in steps, for every increment of the cell pressure, the vertical shortening of the sample is measured and the strain is obtained. With this strain, the decrease in the diameter of the sample is found. Then, the change in volume of the sample is computed. The change in water pressure is equal to the change in stresses. A curve of $\Delta V/V_o$ versus Δu, is plotted and the slope of this curve is C_w.

The porosity of soil is obtained by direct measurement of the weight and volume of the soil and determining the specific gravity of the soil solids.

Having determined C_w, C_c, and n, the B value can be calculated by applying equation (4). From experience in practice, most soils encountered in field are highly saturated. When the soil is 100% saturated, the value of B is unity.

<u>Method 2.</u> By triaxial compression test.

This method is simpler than the previous one and therefore it is currently used. An isotropic compression test is carried out. The cell pressure is raised in increments. For each increment of the cell pressure, the pore-water pressure is recorded. A curve of pore-water pressure versus the cell pressure is plotted. The slope of this curve is the B value.

Two methods can be used to determine the pore pressure parameter A.

<u>Method 1.</u> By triaxial undrained compression test with pore pressure measurement.

The usual way to determine the parameter A is to carry out a consolidated undrained triaxial test with pore pressure measurements. The selection of the consolidation pressure is important. It must be made similar to the actual field stress conditions. In a standard triaxial test, the pore-water pressure at start is usually zero and the cell pressure is kept constant, therefore, for a saturated soil equation (2) is written

$$A = \frac{\Delta u}{\Delta \sigma_1 - \Delta \sigma_3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

which permits the direct determination of the A parameter.

<u>Method 2.</u> By triaxial drained test.

If the change of volume in soil skeleton is equal to the change of volume of the pores in the soil element under consideration and the pore-fluid is incompressible, then we get,

$$A = \frac{1}{1 + \frac{C_{s2}}{C_{c1}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

in which C_{s2} = swellability in horizontal direction

C_{c1} = compressibility in vertical direction

The swellability, C_{s2}, is obtained from the triaxial drained test, in which σ_1 is held constant and the cell pressure ($\sigma_2 = \sigma_3$) is reduced in steps. A period of time is allowed to elapse to make sure that the pore-water pressure is completely dissipated. The volume change is measured. This same