THE UNIVERSITY OF MANITOBA

AN INVESTIGATION AND REVIEW OF
OEDOMETER AND TRIAXIAL TESTS ON WINNIPEG CLAYS

by

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ABSTRACT

An extensive laboratory program of small diameter CIŪ triaxial tests and oedometer tests on undisturbed Winnipeg clay has been executed. Blocks of clay from depths of 5.5 m, 9.0 m and 11.5 m depth were chosen for the study to substantiate and augment previous research. Significant new insights have been gained into the behaviour of the clay, and explanations have been offered for previous apparent anomalies. The traditional semi-logarithmic plot of the compression versus the logarithm of the effective stress in an oedometer test is inappropriate for Winnipeg clay. Arithmetic scale plots of compression versus effective stress are much more useful. The latter show that in the recompression range the relationship is linear, but the compressibility is unusually high. After recompression a yield occurs at the preconsolidation pressure, p'_c, but the higher p'_c, the lesser the amount of yield. The value of p'_c decreases with depth so that at 5.5 m the yield is barely noticable, whereas at 11.5 m the yield is very sharp. To increase the definition of the yield and also to complement the arithmetic scale plotting method, many of the oedometer samples were loaded with small equal increments. The more usual method is to load samples using a constant increment ratio, which suits logarithmic plotting.

In triaxial compression the Winnipeg clay behaves as a lightly to moderately overconsolidated deposit, with A_f values ranging from 0.0 to 1.1. The effective-stress strength envelopes are, however, unusual in shape. At low effective pressures, the envelopes are curved and appear to have zero cohesion intercept. It is demonstrated that this curvature at low pressures is likely caused by a general softening of

the microstructure, rather than by failure along fissures. At higher pressures there is apparently no abrupt transition between normally consolidated and overconsolidated envelopes, even though normally consolidated and overconsolidated failure modes were distinctly different. A possible explanation for both the curvature and the lack of a transition is a high content of the swelling clay mineral calcium montmorillonite or of a similar mineral. Both normally consolidated and overconsolidated strength envelopes for this mineral are curved, and the behaviour of the natural Winnipeg clay may be similar. Although the author's envelopes are unusual in shape, the test results are highly compatible with the results of previous research. The unusual shape is in fact an explanation for the widely conflicting c' and ϕ' parameters previously quoted for normally consolidated and overconsolidated envelopes.

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LIST OF SYMBOLS AND ABBREVIATIONS

a'	-	intercept of q_f versus p'_f
a_v	-	coefficient of compressibility in oedometer test
a_{vo}	-	average value of a_v in the recompression range
$a_{v(iso)}$	-	average value of compressibility = $\Delta e/\Delta\sigma'_c$ in isotropic consolidation
A, B	-	porewater pressure parameters
A_f	-	value of A at failure
c'	-	effective cohesion intercept
C_α	-	coefficient of secondary compression
C_c	-	compression index
C_v	-	coefficient of consolidation
C.I.Ū	-	isotropically consolidated undrained triaxial compression test
e	-	voids ratio
$e_{n/c}$	-	voids ratio in a normally consolidated state at given stress
e_o	-	initial voids ratio
E_{50}	-	elastic modulus to 50% of failure stress
H	-	height, thickness
k	-	coefficient of permeability
K_o	-	coefficient of earth pressure at rest
L.I.R.	-	load increment ratio; = final stress/initial stress
n/c	-	normally consolidated
o/c	-	overconsolidated
p'	-	average normal stress; = $(\sigma'_1 + \sigma'_3)/2$
p'_f	-	p' at failure

p'_c - effective vertical preconsolidation pressure in oedometer test

$p'_{c(iso)}$ - effect preconsolidation pressure in an isotropic (triaxial) consolidation test

p'_i - initial vertical equilibrium pressure under no-swell conditions

p'_o - in-situ vertical effective overburden pressure

p'_s - low vertical pressure causing sample to swell ($p'_s < p'_i$)

q - shear stress; $(\sigma_1-\sigma_3)/2$

q_f - q at failure

t - time

t_f - time to failure

u - porewater pressure

$\bar{u}$ - excess porewater pressure

v - volume

z - depth, vertical axis direction

α' - slope of q_f versus p'_f

γ - unit weight

γ_w - unit weight of water

Δ - change, e.g., Δe

Δh - layer thickness

ΔT - loading period

ε_1 - major principal strain (i.e axial strain in a triaxial compression test)

ρ - settlement

σ - normal stress

σ_3 - minor total principal stress = cell pressure

σ'_1, σ'_3 - major and minor effective principal stresses, i.e vertical and horizontal stresses in triaxial compression

σ'_c - applied consolidation stress; vertical stress in an oedometer test, spherical stress in triaxial tests

σ'_{ff} - effective normal stress on the theoretical failure plane at failure

σ'_{oct} - effective octahedral normal stress

σ'_r - correction to $(\sigma_1-\sigma_3)$ due to the strength of membranes and filter drains

τ - shear stress

τ_{ff} - shear stress on the theoretical failure plane at failure

ϕ' - effective angle of shearing resistance

$\phi'_{n/c}$ - effective angle of shearing resistance for soil in the normally consolidated state

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CHAPTER 1

INTRODUCTION

1.1 GENERAL INTRODUCTION

In any construction involving soil, an estimate is required of the deformations at the boundaries of a bulk of soil due to the internal stress changes caused by construction loads. These loads may be imposed through structural foundations, from the addition or removal of a large volume of soil, or from a change in groundwater conditions.

The deformations may be due to either a widespread compression, expansion,or shearing, of the soil particles in the loaded area,or due to shear strains concentrated in a highly localized set of rupture zones. If the shear stresses in the soil are low, the deformation rates will generally decrease with time and a settlement or heave analysis can be based on the compressibility of the soil. If the shear stresses approach the shear strength of the soil particles, an increase in deformations followed by a sudden collapse may follow a period of apparent stability. In the latter case a stability analysis is usually based on an assumption of rigid-plastic behaviour, using an estimation of the shear strength of the soil.

The general aim of this thesis was to improve the state of knowledge of the basic material properties of undisturbed Winnipeg clay, properties which may be used in simple settlement and stability analyses. Specifically,the laboratory work involved two testing programs:

(a) A series of oedometer tests, primarily to measure the drained compressibility, a_v, in the in-situ vertical direction.

(b) A series of consolidated-undrained triaxial compression tests with pore-pressure measurement, ($CI\bar{U}$), primarily to measure the variation of peak shear strength with effective normal stress.

More complex analyses, require an evaluation of the anisotropic nature of the soil, and the degree to which this affects both compressibility and shear strength. However, an attempt to account for anisotropy inevitably requires a number of more complicated tests. Because the structure of a natural deposit can easily be disturbed, some of the required tests are difficult to perform, and the measured parameters may be inaccurate. Further, no great progress can be made in the evaluation of anisotropy unless simple parameters can be measured with accuracy and repeatability. It is to this end that this thesis is addressed. A review of previous research on undisturbed Winnipeg clay revealed some conflicting results and uncertain conclusions about the behaviour in the oedometer and $CI\bar{U}$ triaxial tests. A general review of these results and an outline of the current research is contained in Section 1.4.

1.2 DISCUSSION OF SETTLEMENT AND STABILITY ANALYSES

1.2.1 Analysis of Settlements

The design of a vertically loaded structural foundation on a horizontal stratum is almost always governed by settlement criteria. For a typical design safety factor of 2.5, the maximum design load is only governed by stability considerations in the case of small footings on soils with a limited range of intermediate plasticity or on soils which are hard and brittle (Burland, 1977). In the classical settlement analysis, a one-dimensional approximation is obtained by using linear elastic theory to predict average vertical stress increases in n horizontal layers. The one-dimensional compressibility, a_v, is then used to compute the final compression in each layer. The final settlement is then given by:

$$\rho_{final} = \sum_{1}^{n} \frac{a_v}{(1+e_o)} \cdot \Delta\sigma \cdot \Delta h \qquad (1.1)$$

The inherent assumptions that the axes of principal stress and strain remain coincident and vertical beneath the loaded area, and the use of a single coefficient of compressibility for a material as complex and non-linear as a natural clay,have often been questioned by engineers. However, studies which have compared the above classical analysis to more exact analytical or numerical solutions which take account of non-linearity, anisotropy, non-homogeneity, and rotation of the axes of principal stress, have somewhat allayed these concerns. These studies, e.g Gibson (1974), Burland (1977) show that:-

(a) The vertical stress distribution is not greatly affected by the likely range of non-linearity or non-homogeneity for a typical soil, except where a stiff layer overlies a soft layer.

(b) Rotations from the vertical, of principal stress increments are minor in regions where major strains occur.

The results of simple one-dimensional analyses compare very favourably with more sophisticated methods, and errors are usually small compared to those that can be attributed to sampling and testing of the soil. Settlement is dependent upon the mean values of the soil compressibility in the profile. An accurate determination of a_v from a series of oedometer tests at different depths is more likely to improve the settlement estimate than a sophisticated analysis.

1.2.2 Rates of Settlement

Prediction of settlement rates is much less reliable than prediction of the settlement magnitude. The bulk in-situ permeability, the controlling factor, is dependent on extreme values of permeability occuring locally in fissures, horizontal beds, or varves, which linked together form a secondary permeability system. Estimates of field rates of settlement are likely to be gross underestimates if based on laboratory tests on small samples, in which the effective permeability is close to that of the intact clay. Another cause of error is that the compressibility of a clay has been shown to be strain-rate dependent. As a result laboratory values of the coefficient of consolidation, c_v, are a function of load increment ratio, and sample height, in deviation from Terzaghi's Consolidation theory. Since field rates of strain may be many thousands of times slower than rates in laboratory oedometer tests (Crawford, 1964(b)), the "final" strain in the laboratory differs from that in the field. This is discussed in Chapter 3.

1.2.3 Analysis of Stability

Failure of a natural slope or a foundation on a natural soil is dependent upon weak "links" in the profile, rather than average soil conditions. Examples of weak links are; previous shear planes, fissures, natural beds of weak materials, and tension cracks. The probability of failure, is almost impossible to estimate, being dependent upon the statistics of extreme values (De Mello, 1977). Design is then based on the correlation of an acceptable factor of safety with local or general experience. The factors of safety have to be recognised as being purely nominal. Two ideal slopes, each composed of a different homogeneous material, may be shown to have equal factors of safety, but widely different probabilities of failure (De Mello, 1977).

Since the strength of a soil is dependent upon the effective stress conditions,and the pore-pressure, u, has a dominant influence on most failures, it would seem prudent to use only effective stress analysis. However an accurate prediction of the most unfavorable distribution of pore-water pressures is extremely difficult. In contrast, accumulation of local experience may lead to an good estimation of the range of undrained strengths likely to exist at failure, in which case total stress analysis can be justified.

In an effective stress analysis of an overconsolidated clay deposit, the results of laboratory peak strengths over some middle range of normal stresses may be approximated by the linear relationship; $\tau_{ff} = c' + \sigma'_{ff} \tan\phi'$. The c' and ϕ' values from curve fitting do not represent true angles of friction or true cohesion between particles. In particular, use of the c' and ϕ' relationship for a range of field stresses outside the range tested in the laboratory can lead to a gross overestimate of the in-situ strength.

1.3 STABILITY PROBLEMS - LOCAL EXPERIENCE

Since Winnipeg is topographically flat, the most troublesome soil instability problems have occured on the banks of the Red and Assiniboine Rivers, although there have also been failures of man-made fills, excavations, and foundations. Although most of the brown clay and blue clay deposits are of a medium-stiff constituency, riverbank slides have been common even at slopes flatter than 9:1. The difficulties in reliable analysis, and reasons for such poor and erratic stability have been discussed by Baracos and Graham (1980), and Freeman and Sutherland (1974), and are summarized below:

(a) Residual strengths are very low, ($\phi'_r \simeq 7\text{-}9^{\circ}$, $c'_r \simeq 0$), and are considerably less than peak strength values due to the high smectite content of the clay. The existence of old failure surfaces will considerably reduce stability.

(b) The brown clay has been observed to contain numerous slickenslides and fissures, and a nuggety structure can be seen when the soil is allowed to swell. Triaxial tests on the brown clay have shown a wide scatter in peak effective strengths from $c' = 42$ kPa, $\phi' = 19^{\circ}$ to $c' = 3.5$ kPa, $\phi' = 14.5^{\circ}$.

(c) Actual slides have often involved large components of horizontal movement. Since peak effective strength has been shown to be reduced for shear along clay laminations, analyses should include the possibility of non-circular slip surfaces with long flat horizontal portions.

(d) A considerable reduction in strength appears to occur when the clay is allowed to swell under low confining pressures. Thus strength at the toe of a slope may be very low.

1.4 OBJECTIVES OF THE LABORATORY TESTING PROGRAM

1.4.1 General

This work was undertaken as a part of an ongoing research program into the mechanical properties of undisturbed Winnipeg clay at the University of Manitoba. Completed studies by Pietrzak (1979), Noonan (1980), and Lew (1981), have all made use of clay samples taken from an undeveloped university site, situated opposite the present physical education building (Figure 1.1). All studies made use of an excellent block sampler, (Domaschuk, 1977), to recover undisturbed blocks of clay. For the present study blocks were taken from depths of 5.5 m, 9.0 m and 11.5 m.

1.4.2 Oedometer Testing Program

A review of previous oedometer testing of Winnipeg blue clay, Crawford (1964), Noonan (1980), and Martin (1961), revealed the following characteristics of the void ratio versus log. effective-stress relationship:

(a) A highly curved section at stresses below the preconsolidation pressure.

(b) A very steep initial "virgin compression" section with C_c values ranging from 0.6 to 1.0.

(c) Distinct curvature of the "virgin compression" section

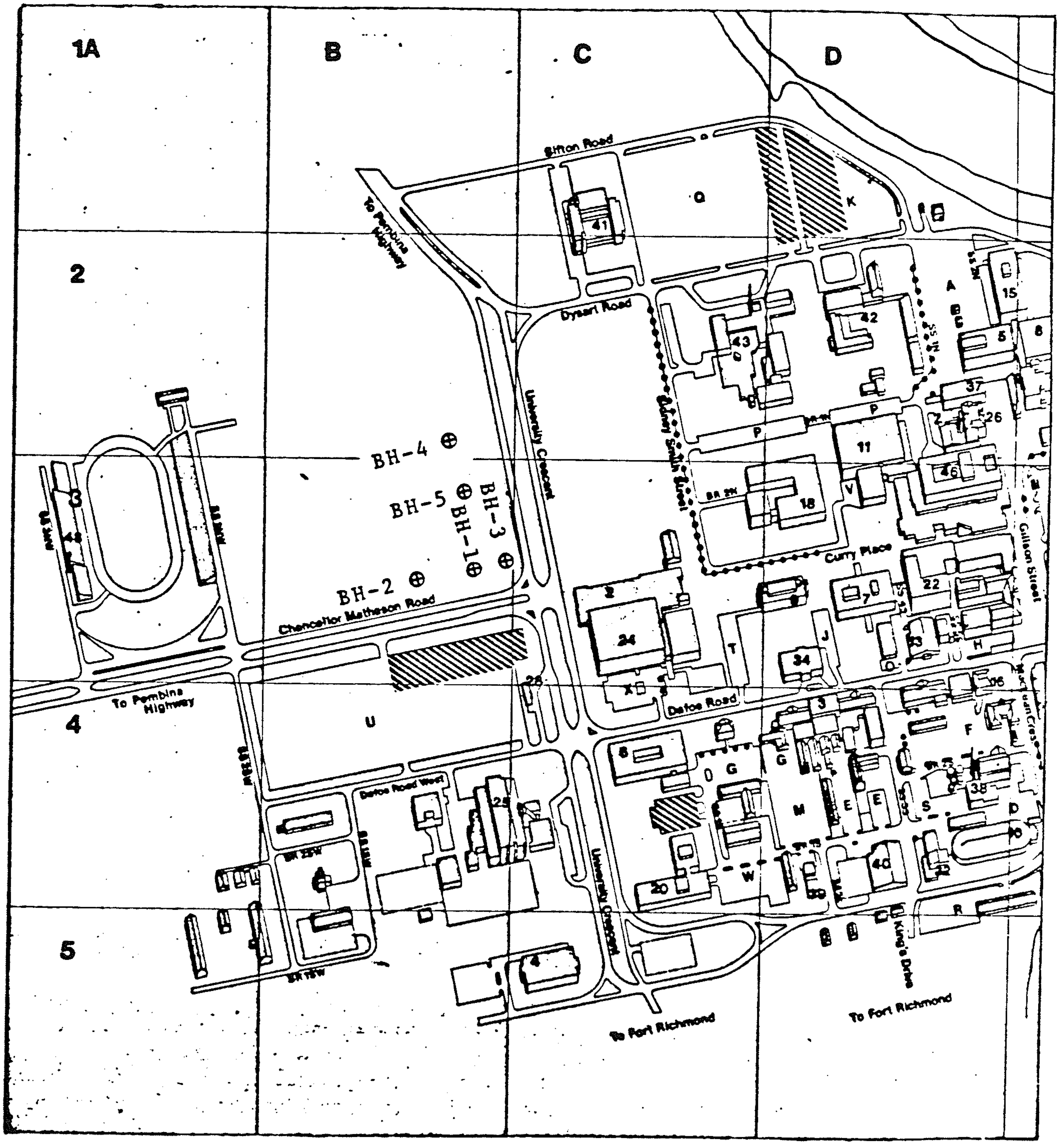

Figure 1.1 Site Plan and Location of Boreholes

at high stresses, (concave upward).

The pronounced curvature at low stresses causes difficulty in estimation of p_c' by the Casagrande Method, and has been attributed to disturbance caused by allowing samples to swell, (Baracos et. al., 1980), or to other disturbance during sampling or preparation.

The initial aim of the author's research was to see if the determination of the value of p_c' could be improved by careful trimming procedures and by eliminating the initial swelling. The first samples were loaded using a load increment ratio (L.I.R.) of 1.6, and showed an apparent yield point in the void ratio versus (arithmetic) stress relationship. It was concluded that the arithmetic scale plot gave a better understanding of behaviour than the traditional semi-logarithmic plot. Because of the method of loading,however, the data points become spaced further and further apart. To produce equally spaced data points in an arithmetic plot, the loading sequence was altered to make all the loads small and equal.

In the above "constant load increment" test,however, the load increment ratio is reduced at each increment and deviations from Terzaghi Theory occur at low load increment ratios (Leonards, 1961). As a result after a few constant increments of load, the standard S-shaped consolidation curves were no longer produced. Although "primary" and "secondary" consolidation can no longer be distinguished without direct pore-pressure measurement, the test still gives a valid estimation of compressibility. The interesting results obtained from the first constant increment test led to the decision to undertake further tests at depths of 5.5 m, 9.0 m and 11.5 m. The expanded objectives of the testing program are summarized below:

(a) To achieve a more accurate determination of p_c' by altering the loading sequence and the plotting techniques.

(b) To estimate average values of p_c' and coefficient of compressibility, a_v, for each of the studied depths, 5.5 m (brown clay) and 9.0 m and 11.5 m (blue clay).

(c) To study the effect of initial swelling on the void ratio versus effective stress curve, in particular the effect on the initial compressibility and the preconsolidation pressure.

(d) To study the effect of load increment and interval of loading on the void ratio versus effective stress curve.

(e) To study the effect of load increment on the shape of the consolidation-time curves, and evaluate the coefficient of consolidation, C_v, and the coefficient of secondary compression, C_α.

1.4.3 Consolidated Undrained Triaxial Testing Program

Previous determinations of the peak effective stress failure envelopes for Winnipeg clay have been reported by Mishtak (1964), Crawford (1964(a)), Freeman and Sutherland (1974), and Pietrzak (1979). An important conclusion from Pietrzak's study was that the effective strength envelope for Winnipeg blue clay was approximately independent of depth, and could be approximated by three linear sections, (Baracos et. al., 1980), as follows:

(a) A low stress section in which strength is believed to be controlled by fissures or the nuggety structure of the clay

$p' < 60$ kPa $c' = 6$ kPa, $\phi' = 32^{o}$

(b) An overconsolidated section:

60 kPa $< p' < 200$ kPa $c' = 33$ kPa, $\phi' = 13^{o}$

(c) A normally consolidated section:

200 kPa $< p'$ $c' = 3$ kPa, $\phi' = 22.5^{o}$

The three stage hypothesis helps explain discrepancies in the c' and ϕ' parameters cited by the previous authors, in an attempt to fit a single straight line to their data. The problem is partially recognised in these publications. Crawford, noted a substantial reduction in strength in two drained tests on samples which had been allowed to swell, and urged further research into this effect. Freeman, suggested that his lower c' values were due to testing at a lower range of effective stresses; a visual inspection of the grouping of his test data for the blue clay gives support to the hypothesis of a low strength envelope.

A review of Pietrzak's data,however,showed that only four samples out of seventeen had actually been tested in the low-stress range. In the light of this,further research was needed and the aims of the present program were primarily:

(a) To evaluate the peak strength failure envelope for Winnipeg clay at depths of 5.5 m (brown clay), and 9.0 m and 11.5 m (blue clay), using CI$\bar{U}$ triaxial tests.

(b) To perform sufficient tests at each depth to accurately define the above envelopes over a wide range of stresses. Of particular interest was the envelope for the brown clay (5.5 m) which had not been tested by Pietrzak.

Also, the moisture content at 11.5 m depth was higher than that at 9.0 m depth, (Chapter 2), and it was suspected that the envelopes for the two strata of blue clay might differ.

(c) To examine the modes of failure occuring in each sample and identify any changes with consolidation stress or with depth.

(d) To measure the compressibility in isotropic compression, and compare results with those from oedometer tests.