THE UNIVERSITY OF MANITOBA

RATE EFFECTS AND LOW STRESS STRENGTH OF WINNIPEG CLAY

by

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ABSTRACT

The two principal purposes in this thesis are:

1. to investigate the influence of test duration and strain rate on the stress-strain behaviour of Lake Agassiz clay from Winnipeg.
2. to examine the strength of Winnipeg clay at low stress levels.

The study also investigated the Undrained Strength of the clay at Large Strains (USALS) and its relationship with the normally consolidated Coulomb-Mohr envelope.

Six drained stress-controlled triaxial tests on undisturbed samples were used to study the time-dependent aspects of the YLIGHT model of soil behaviour. Six non-standard and four strain-rate controlled oedometer tests were performed to examine the effects of time and strain rate on the preconsolidation pressure, p'_c. The samples were taken from 11.6 m depth, and the sample diameter was 76 mm.

The preconsolidation pressure p'_c decreased from 249 to 225 kPa as the duration of the load application increased from 0.1 to 100 days. This supports previous findings by Bjerrum (1967), Tavenas and Leroueil (1977). Strain controlled oedometer tests also show that the preconsolidation pressure is strain rate dependent.

Five 76 mm diameter undisturbed triaxial samples taken from 11.6 m, six 'fully-softened' and five 'freeze-thaw' triaxial samples taken from 8.7 m were tested to study the low stress strengths of Winnipeg clay. Data was obtained on both drained and undrained triaxial behaviour. The low stress strengths were the highest for undisturbed samples, followed by the 'fully-softened' and 'freeze-thaw' samples. These low stress strength envelopes were considerably curved and parallel to each other.

The Undrained Strength at Large Strains (USALS) obtained for all the undrained tests lay close to the normally consolidated Coulomb-Mohr envelope ($\phi' = 17.5^{\circ}$; $c' = 4$ kPa).

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LIST OF SYMBOLS

A, B	- porewater pressure parameter (after Skempton, 1954)
A_f	- value of A at failure
c'	- effective cohesion intercept
C_c	- compression index
c_v	- coefficient of consolidation
C_α	- coefficient of secondary compression
CAD	- stress-controlled, consolidated anisotropically drained test
CAD(U)	- strain-controlled, undrained compression test with porewater pressure measurements preceded by CAD test
CAU	- strain-controlled, consolidated anisotropically undrained compression test
CRS	- constant rate of strain oedometer test
e	- voids ratio
e_o	- initial voids ratio
e_f	- final voids ratio
E_{50}	- elastic modulus to 50 per cent of failure stress
G	- shear modulus
G_s	- specific gravity
G.W.L	- groundwater table or phreatic surface

I_p - plasticity index

K_o - coefficient of earth pressure at rest

LSSV - Length of Stress Vector

OCR - overconsolidation ratio

p' - mean principal stress; = $(\sigma_1' + \sigma_2' + \sigma_3')/3$

p_c' - effective preconsolidation pressure

p_o' - effective vertical overburden stress

q - principal stress difference; = $(\sigma_1 - \sigma_3)$

s_u - undrained strength; = $(\sigma_1 - \sigma_3)/2_{max}$

u - porewater pressure

V - specific volume; = $(1 + e)$

w - natural moisture content

w_i - initial moisture content

w_f - final moisture content

w_L - liquid limit

w_p - plastic limit

W_T - strain energy absorbed per unit volume

γ_{sat} - saturated unit weight

ε_1, ε_3 - major and minor principal strains (i.e. axial and radial strains in triaxial compression test)

ε - shear strain; = $2(\sigma_1 - \sigma_3)/3$

ε_{1c}, ε_{3c} - ε_1 and ε_3 at the end of triaxial consolidation to σ_{1c}', σ_{3c}'

ε_v	- volumetric strain in triaxial compression test
ε_{vc}	- ε_v at the end of triaxial consolidation to σ'_{1c}, σ'_{3c}
ε_{VR}	- vertical strain for oedometer test
	- average axial strain during relaxation test in undrained compression test
$\dot{\varepsilon}_1$	- axial strain rate
$\rho_{0.1}$	- strain rate effect parameter for undrained strength
$\eta_{0.1}$	- strain rate effect parameter for preconsolidation pressure p'_c
σ'_1, σ'_3	- major and minor effective principal stresses
σ'_{1c}, σ'_{3c}	- σ'_1 and σ'_3 at the end of triaxial consolidation
σ_{oct}	- total octahedral normal stress
σ'_{oct}	- effective octahedral normal stress
σ'_v	- effective vertical stress
ϕ'	- effective angle of shearing resistance

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CHAPTER 1

INTRODUCTION

1.1 INFLUENCE OF TIME EFFECTS ON THE STRESS-STRAIN BEHAVIOUR OF SOIL

Before 1960 it was considered that time effects such as straining rate or test duration influenced stress-strain behaviour of soil in a relatively minor way, and could be included with other effects whose magnitude, could not be determined, but which produce compensating errors (Bishop and Henkel, 1957). Since Bjerrum (1967) introduced the "delayed compression" concept, more attention had been paid to time dependent and strain rate dependent properties of carefully sampled natural clay (Crooks and Graham, 1976). The "delayed compression" concept suggested that normally consolidated clay subjected to a constant overburden stress after a long period could be referred to as "aged normally consolidated clay" having a value of preconsolidation pressure p_c' greater than p_o' due to delayed compression. Delayed compression acts to reduce the void ratio and develop a more stable arrangement of soil particles. This leads to greater strength and reduced compressibility. A result of this delayed compression is the development of a reserve resistance against further consolidation. Since more load can be carried in addition to the overburden stress without significant volume change, the preconsolidation pressure p_c' appears to increase with time. By monitoring the settlements of five buildings in the Drammen area, Bjerrum (1967) further observed that the effect of the reserve resistance of the plastic clay on the settlement was most pronounced during the initial period after completion of the buildings. The effect of reserve resistance disappeared with long time increments.

Crooks and Graham (1976), working with the post-glacial organic silty clays of the Belfast area, showed that rate effects were also significant in the stress-strain behaviour of less sensitive, plastic clays. They reported that the undrained strength of samples increased by between 7 and 17 per cent for tenfold increases in strain rate.

Based on tests on sensitive clays from Eastern Canada, Crawford (1965), Conlon (1967) and Jarrett (1967) demonstrated that the time dependency of both undrained shear strength (s_u) and preconsolidation pressure (p_c') was significant. Recently tests on the compressibility and strength of Canadian natural clays, especially their creep behaviour under constant effective stress (Campanella and Vaid, 1974; Vaid et al., 1979; Tavenas et al., 1978) also indicated the pronounced influence of time and rate effects on the compressibility and strength of natural clays.

To provide more rational framework for understanding the stress-strain behaviour of natural lightly overconsolidated clay, Mitchell (1970), Crooks and Graham (1976), and Tavenas and Leroueil (1977) developed qualitative behavioural models based on consideration of yielding of these materials. A generally accepted definition for the yield envelope of a natural clay is a locus joining a set of yield points in the (p', q)* stress space, inside which strains, strain rates and porewater pressure generation were much higher. The locus depends on the stress history of the clay as expressed by its preconsolidation pressure p'_c, or voids ratio e. The practical significance of the limit-state concept in understanding the behaviour of clay and in the design of structures on clay foundations had been shown by Tavenas and Leroueil (1977); Tavenas et al. (1978 and 1979); and Tavenas (1979). Although yield envelopes for various clays had been found (Mitchell, 1970; Crooks and Graham, 1976), a general understanding of the nature of the yield envelope for a clay and the factors affecting it was not clear until the development of the YLIGHT model by Tavenas and Leroueil (1977). A description of the YLIGHT model has been given by Noonan (1980), and summarized by Lew (1981).

A particular feature of the YLIGHT model was that the magnitude of the preconsolidation pressure governed the position of the yield envelope in the (p', q) stress space (Tavenas and Leroueil, 1977). This was also shown by Graham (1974). Crawford (1964) and Bjerrum (1967) both demonstrated that the apparent preconsolidation pressure of a clay determined by oedometer tests was reduced if the rate of loading was reduced, or if the duration of loading was increased. Tavenas and Leroueil (1977), using oedometer tests and triaxial tests, confirmed the effect of rate, or duration of

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*Symbols are defined in LIST OF SYMBOLS on page vii loading, on the preconsolidation pressure and yield envelope. As special cases of more general behaviour, they showed that the preconsolidation pressure of a clay was reduced if the duration of loading was increased. Similarly, undrained triaxial tests at different strain rates indicated a reduction in strength as the strain rate decreased. More importantly, the displacement of the yield envelope indicated a homothetic movement inwards with time. On this basis Tavenas and Leroueil (1977) concluded that the known effects of aging and strain rate on p_c' applied to the entire yield envelope.

The applicability of the limit-state or yield concept has been part of a larger investigation by the geotechnical group at the University of Manitoba into the geotechnical properties of the glacial Lake Agassiz clay which underlies the Winnipeg area. This work was initiated by Dr. J. Graham in 1976 at the University of Manitoba. The testing program consisted of 76 mm diameter samples, trimmed using equipment specially designed to minimize disturbance, and tested in large diameter, rotating-bush triaxial cells. Samples were taken from 6 m to 12 m depth at the University of Manitoba campus using the block sampler devised by Domaschuk (1977). Preliminary information was presented by Baracos et al. (1980), Noonan (1980) and Lew (1981). Yield envelopes were found from intact overconsolidated clay samples taken from various depths. A summary of the existing information is presented in Figure 1.1.

It should be noted that the yield envelope defined by Lew (1981) was based on two sets of samples taken from different boreholes at different times. Samples T303 to T313 were taken from borehole 4 (Figure 1.1) in July, 1980; samples T315 to T319 were taken from borehole 5 (Figure 1.1) in January, 1981. Lew (1981) suggested that the p'_c values for the two sets of sample were the same and an average value of p'_c = 218 kPa was used for normalizing test results. However, in studies on the elastic and limit-state properties, Dr. J. Graham (1982) has shown that the shear and bulk moduli of the two sets of samples were different (Graham and Houlsby, 1982; Graham et al., 1982b). Samples T315 to T319 appeared to be stiffer than samples T302 to T314. He therefore modified the yield envelope proposed by Lew (1981) by using different values of p'_c for the two sets of samples. Values of p'_c equal to 191 kPa and 241 kPa were used for samples T302 to T314 and T315 to T319 respectively. This modified normalized yield envelope was transformed back to the (p', q) stress space in Figure 1.2 by using p'_c equal 241 kPa. The modified yield envelope in this figure therefore corresponds to p'_c equal 241 kPa. The modified yield envelope and the modified normalized yield envelope are shown in Figure 1.2 and Figure 1.3 respectively.

Lew (1981) began the study of time effects on the yield envelope for clays taken from 11.5 m. He concluded that the yield envelope was displaced towards reduced preconsolidation pressures and shear strengths as the load duration increased. However, this conclusion was based on a limited number of tests and the time and rate effects on the preconsolidation pressure p'_c had not been examined in previous work. One major purpose of the present study is to continue the study of time effects on the yield envelope initiated by Lew (1981), and especially the effects of time and straining rate on the preconsolidation pressure p'_c.

1.2 LOW STRESS STRENGTHS

The properties of the lacustrine clays underlying Winnipeg continue to cause problems for geotechnical engineers. Natural riverbank slopes are often marginally stable at slopes as flat as 8:1; compacted clay fills will occasionally fail in shallow planar slides at moderate inclinations; and excavation stability is lower than implied by measured unconfined compression strengths. In an investigation of the yielding and rupture of Winnipeg clay, Baracos et al. (1980) proposed a 3-section strength envelope for the full depth of the blue clay (Figure 1.4):

Section 1 (low pressure σ'_{1c} < 60 kPa)

c' = 6 kPa; ϕ' = 31.7°

Section 2 (intermediate pressure 60 kPa ≤ σ'_{1c} ≤ 200 kPa)

c' = 33 kPa; ϕ' = 13.0°

Section 3 (high pressure σ'_{1c} > 200 kPa)

c' = 3 kPa; ϕ' = 22.5°

They postulated that the strength of the soil at low effective stresses was largely controlled by a highly fissured and nuggety clay structure which was easily observed in shallow excavations. Mitchell (1970) identified similar behaviour in Leda (Champlain Sea) clay, and concluded that failure would be accompanied by strong dilation of a nodular or prismatic granular structure behaving as an essentially cohesionless material. Crawford (1964), using samples from the Greater Winnipeg Floodway Test Pit, reported a substantial reduction in strength when the soil was allowed to swell. After the investigation of landslide problems in Winnipeg, Baracos and Graham (1980) stated:

> "At low effective stresses, Winnipeg clays behave as cohesionless, softened materials, a fact that must be adequately considered for low effective stress zone, such as the submerged toe of a riverbank, or to shallow depth beneath the faces of all slopes, (excavated or embankments) subject to snow-melt, rainfall, etc."

They further suggested that low-stress strengths were applicable for first-time shallow slides such as those induced by erosion at the toe of riverbanks, or shallow planar slides paralleling the face of a slope. It was necessary to use the concept of "fully-softened strength", with conservatively assumed zero cohesion. Further attention will be paid in a later section to the strength of Winnipeg clay at low stresses.

The climate in Winnipeg is "continental", with temperatures varying over wide extremes through the year. The average daily temperature curve is at its lowest (-20°C) during the period January 17 to 27 and its highest (22°C) from July 19 to 27 (Environment Canada, 1980). Therefore Winnipeg clay at low effective stress zones near the ground surface are susceptible to alternating cycles of freeze and thaw. Nuggets and fissures are frequently formed due to the effects of freezing and thawing. The accompanying destruction of the intact clay structure may lead to a reduction in strength. This effect will also be examined in a later section.

By studying the behaviour of a test embankment founded on a well-instrumented foundation of soft Champlain clay at Saint-Alban, LaRochelle et al. (1974) showed that the strength mobilized at failure under the test fill was approximately equal to the "undrained residual strength", a term hitherto used to designate the undrained strength at large strains (USALS), from undrained (CIU) or unconsolidated, undrained (UU) tests at strains of about 15 per cent. Lefebvre (1981) successfully demonstrated that the use of 'post peak' or large strain strengths allowed a reasonable estimate of the stability of natural or man-made slopes in Champlain Sea clays.

By studying case histories of failure of water-retaining structures on highly plastic clay, Rivard and Lu (1978) concluded that the intact strength of the clay did not reliably predict the stability condition. A study of the foundation conditions revealed the presence of structural discontinuities such as nugget and blocky structures, joints, fissures and slickensides. These structural discontinuities were probably caused by weathering. They further suggested that embankments on soft highly plastic clay soils with structural discontinuities should be designed using the normally consolidated strength, as suggested by Skempton and Hutchinson (1969) for stiff fissured clays.

With these points in mind therefore, the second major purpose of the present study was to investigate the low stress strengths of Winnipeg clay under several sets of controlled conditions. The changes in strength from the natural "undisturbed" strengths studied by Baracos et al., 1980; Noonan, 1980 and Lew, 1981 were investigated when the soil was a) allowed to swell freely, and b) was subjected to a series of 'freeze-thaw' cycles. Undrained strengths at large strains (USALS) (LaRochelle, 1974) and their relationship with the normally consolidated strength (Rivard and Lu, 1978) were also examined.

1.3 OUTLINE OF THESIS

The previous section (1.1) showed that although preliminary work on the time-dependent aspects of the YLIGHT model on yielding was studied by Lew (1981), only a limited number of tests were performed. Time and rate effects on the preconsolidation pressure p'_c measured by oedometer, were not examined. The low stress strength envelope shown by Baracos et al. (1980) was based on limited data and further examination of this envelope was required.

As mentioned previously, the two major topics for investigation in the present study were:

1. Examination of test duration and strain rate effects on the yield envelope for Lake Agassiz (Winnipeg) clay.
2. Investigation of low stress strengths including 'fully-softened' strengths and 'freeze-thaw' strengths for Winnipeg clay.

More specifically the aims of this thesis were:

1. To examine the time-dependent aspects of the YLIGHT model for yielding of clays as they applied to Winnipeg clay.
2. To examine the effect of time and strain rate on the preconsolidation pressure, p_c'.
3. To investigate the low stress strengths of Winnipeg clay, and to study the effects of swelling and freeze-thaw degradation on them.
4. To examine the undrained strength at large strains in Winnipeg clay using the USALS method described by LaRochelle et al. (1974), and by implication by Rivard and Lu (1978).
5. To study the effects of changes of strain rate on undrained shear strength.

Large diameter (76 mm) samples were used for all the triaxial tests and oedometer tests performed in the present study. Samples used for the study of time effects and undisturbed low stress strengths were taken from 11.6 m.

'Fully-softened' and 'freeze-thaw' samples were taken from 8.7 m, preparation of these samples will be described in Chapter 2. Six drained stress-controlled tests with two samples running on the same stress path but with different load durations (1 day and 5 days) for each load increment were used to examine the time effects on the yield envelope. Six non-standard oedometer tests similar to those performed by Tavenas et al. (1977) were used to examine the time effects on the preconsolidation pressure p'_c. Four strain-controlled oedometer tests (Sallfors, 1975; Bell, 1977) were employed to investigate rate effects on p'_c. The undisturbed low stress strengths were examined using three undrained strain-controlled and two drained stress-controlled triaxial tests. Three drained stress-controlled and three undrained strain-controlled triaxial tests were performed on the 'fully-softened' samples to examine the effect of swelling on the low stress strengths. Finally, three undrained strain-controlled and two drained stress-controlled triaxial tests were applied to the 'freeze-thaw' samples to study the effect of freeze-thaw degradation on the low stress strengths. The laboratory testing program will be described in detail in Chapters 3 and 4.

Samples which were not stressed to rupture during the drained portion of the triaxial test were tested to failure in undrained shear. The undrained part of the test allowed examination of the following characteristics of clay behaviour: the influence of consolidation history on porewater pressure generation and elastic moduli; the effects of changes of strain rate on the undrained shear strength; the normally consolidated strength for samples consolidated to stresses well past p_c'; the low stress strength envelope and the undrained strength at large strains (USALS).

The thesis begins with a review of the general properties of the lacustrine clays and test procedures (Chapter 2). It continues with the results for time and rate effects (Chapter 3) and low stress strengths (Chapter 4). Discussion of results are presented in Chapter 5. Finally, conclusions and suggestions for further research are presented in Chapter 6.