THE ESTIMATION OF THE EFFECTIVE SHEAR

STRENGTH PARAMETERS OF LEDA CLAY

by

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in memory of my father

ABSTRACT

A study of the relationship between undrained shear strength and the effective shear strength parameters of c' and ϕ' of Leda clay is presented.

High quality block samples were obtained from five sites located within a 70 km radius of the City of Ottawa. The samples obtained represent three of the major facies of Leda clay known to exist in the Ottawa River Valley Region, that are of interest to geotechnical engineers. The relationship between total and effective stress shear strength behaviour of the clay at each site was studied by means of field and laboratory vane shear tests, unconfined compression tests and a series of CIU triaxial tests.

The following aspects of the behaviour of Leda clay are discussed with reference to the test results: the concept of yield during isotropic consolidation and triaxial shear, the development of porewater pressures during shear, the mobilization of c' and ϕ' parameters with strain, the geotechnical properties and behaviour of each facies of Leda clay and finally the relationship between vane shear strength and c', ϕ' parameters determined from the peak and residual effective stress failure envelopes.

The following topics are discussed with reference to the current literature: the occurrence and behaviour of intact and microfissured Leda clays, the effect of test procedure on measured shear strengths, the effect of sample size, sample disturbance, anisotropy, time and progressive

failure on the mobilization of measured shear strengths along in-situ failure surfaces and finally the use of peak and residual shear strengths in slope stability analyses.

The most significant contribution of the thesis is a proposed relationship between vane shear strength and the effective shear strength parameters of c' and ϕ'. This relationship will allow the effective shear strength parameters of c' and ϕ' to be estimated on the basis of simple field vane tests for the purpose of a preliminary assessment of the stability of a slope.

RESUME

Cette thèse présente une étude de la relation entre la résistance au cisaillement non-drainée et les paramètres effectifs c' et ϕ' de résistance au cisaillement de l'argile Leda.

Des blocs échantillons de très bonnes qualité furent prélevés à cinq emplacements situés dans un rayon de 70 km de la ville d'Ottawa. Les échantillons prélevés représentent trois des facies principaux de l'argile Leda dont on connaît l'existence dans la région de la vallée de la rivière des Outaouais et qui intéressent l'ingénieur en Géotechnique. La résistance au cisaillement de l'argile en termes de contraintes totales et effectives fut étudiée pour chaque emplacement par des essais au scissomètre de chantier et de laboratoire, des essais de compression simple et une série d'essais triaxiaux CIU.

On discute les aspects suivants de comportement de l'argile Leda en s'appuyant sur les résultats d'essais: le concept de rupture lors de la consolidation isotropique et le cisaillement triaxial, le développement des pressions interstitielles pendant le cisaillement, la mobilisation des paramètres c' et ϕ' au fur et à mesure que la déformation s'accroît, les propriétés géotechniques et la comportement de chacun des facies de l'argile Leda et finalement la relation entre la résistance au cisaillement du scissomètre et les paramètres c' et ϕ' déterminés à partir des enveloppes de résistance maximale et résiduelle en contraintes effectives.

On discute des sujets suivants en se rapportant aux publications récentes: l'existence et le comportement des argiles Leda intact et microfissurées, l'influence de la procédure d'essai sur les résistances au

cisaillement mesurées, l'influence de la grandeur de l'échantillon, du remaniement de l'échantillon, de l'anisotropie, du temps et de la rupture progressive sur la mobilisation de la résistance au cisaillement mesurée le long des surfaces de rupture in-situ et enfin l'utilisation des résistances au cisaillement maximales et résiduelles pour l'analyse de stabilité des pentes.

La contribution la plus importante apportée par cette thèse est la relation qu'on propose entre la résistance au cisaillement du scissomètre et les paramètres c' et ϕ' de résistance au cisaillement en contraintes effectives. Cette relation permettra d'évaluer les paramètres c' et ϕ' à partir des essais simples du scissomètre de chantier pour l'étude préliminaire de la stabilité d'une pente.

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LIST OF SYMBOLS

A_f	Pore pressure parameter 'A' at failure
A_r	Pore pressure parameter 'A' at residual strength
B	Pore pressure parameter 'B'
c'	Effective cohesion intercept
γ	Unit weight of soil
γ'	Unit weight of submerged soil
γ_w	Unit weight of water
e	Void ratio
ε	Unit axial strain
F.S.	Factor of Safety
G_s	Specific gravity of solids
I_L	Liquidity index
I_P	Plasticity index
kPa	kilopascal
K_o	Coefficient of earth pressure at rest
σ_n	Normal stress
σ_v	Vertical stress
σ_h	Horizontal stress
$\sigma_1, \sigma_2, \sigma_3$	Major, intermediate and minor principal stresses, respectively
ϕ'	Effective angle of internal friction
p'_o	Effective overburden pressure
p'_c	Effective preconsolidation pressure
p'_m	Mean effective stress = $\frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3}$
p'	Average effective stress $\frac{\sigma'_1 + \sigma'_2}{2}$
q	Deviator stress = $\frac{\sigma_1 - \sigma_3}{2}$

S	Sensitivity as measured by field vane test
S_u	Undrained shear strength
S_r	Degree of saturation
τ	Shear stress
u	Porewater pressure
w%	Water content
w_L%	Liquid Limit
w_P%	Plastic Limit

CHAPTER 1

INTRODUCTION

1.1 General

The Ottawa River Valley and the St. Lawrence River Valley have extensive deposits of post-glacial, sensitive marine clay. These sensitive soils exhibit unusual geotechnical properties and are very susceptible to landslides that may result in costly damage or loss of life. Much work has been carried out in an effort to understand the mechanisms of slope failure and to predict or prevent slope instability.

A simple approach to the prediction of unstable slopes may involve a survey of actual failed and unfailed slopes. When the height and angle of the slope are plotted as illustrated in Figure 1.1a, a line separating failed and unfailed slopes can usually be drawn. This figure is the result of a survey of 11 stable slopes and 50 landslides along the Madawaska River by Wong (1975). The good correlation obtained indicates a similarity of clay properties, shear strength and groundwater conditions over the particular area surveyed.

In an effort to establish guidelines for land-use planning and development in the Regional Municipality of Ottawa-Carleton, Klugman and Chung (1976) published a series of maps indicating areas of stable and potentially unstable slopes in the regional municipality. Stability was assessed according to a computer calculated factor of safety. Computations were based upon average effective stress shear strength

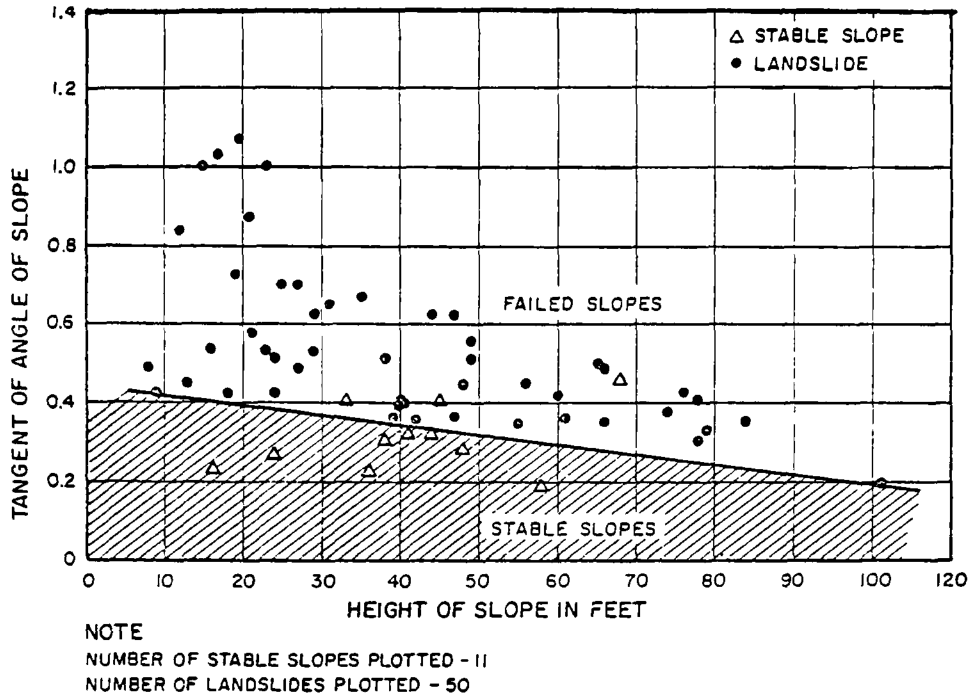


Figure 1.1a Survey of stable and failed slopes along the Madawaska River (after Wong,1975)

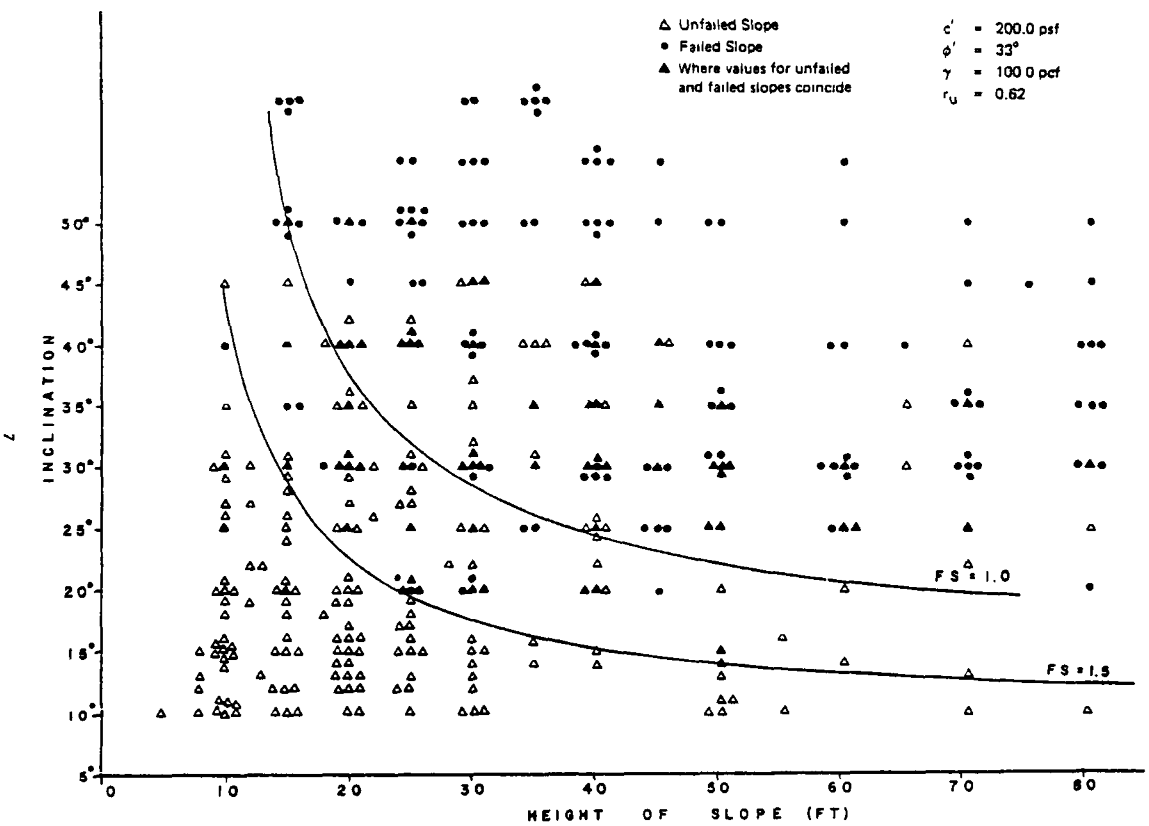


Figure 1.1b Computed Factors of Safety and actual failed slopes for 419 slopes in the Ottawa - Carleton Regional Municipality. (after Klugman and Chung, 1976)

parameters of c' and ϕ' reported in the literature, and assumed hydrostatic saturation (groundwater level at the surface and horizontal flow) of the slope. The use of the average shear strength parameters ignores regional variations in the characteristics and strength of the clay deposits.

Klugman and Chung's work is no doubt valuable as it does fulfill its objectives in providing preliminary guidelines to land use planners and developers, but the accuracy of their calculated factors of safety are however, questionable. The error associated with the use of average shear strength parameters and assumed groundwater conditions over such a large area, as in the regional municipality, is evident in Figure 1.1b. Many unfailed slopes exist below the F.S. = 1.0 line and several failed slopes exist above the F.S. = 1.0 line. Also much uncertainty exists due to the large number of data points where values for failed and unfailed slopes coincide.

A more accurate means of slope stability prediction could be made if a method was developed by which regional variations in clay strength could be simply accounted for. Such a method, it is believed, would greatly reduce the scatter in Figure 1.1b. It would still be necessary to assume groundwater conditions, but it is felt that if the regional geologic conditions were considered, a better estimate of groundwater conditions could also be made.

The need for actual field investigations at specific sites for engineering projects would continue, however, because it would be necessary to ensure precise shear strength data at the specific site and

and also to determine whether unusual groundwater conditions exist that may significantly affect the stability analysis.

1.2 Statement of the Problem

Of prime importance in the analysis of the stability of a slope is to determine the governing resistance to shear deformation (i.e., shear strength). It is generally acknowledged that the long term stability of a slope is governed by the drained shear strength of the clay. To determine the drained shear strength parameters c' and ϕ' requires expensive and time consuming laboratory procedures. It would be of significant benefit to geotechnical engineers and engineering geologists concerned with slope stability, if these drained strength parameters could be more easily but just as accurately determined.

A relationship between the undrained strength, Su, commonly determined by the field vane test, and the apparent preconsolidation pressure, p'_c, determined from a drained standard oedometer test, is known to exist for marine clays and is dependent upon plasticity index (Lambe and Whitman, 1964). This relationship may be expressed as the ratio of Su/p'_c and for Ottawa area clays may vary from 0.24 to 0.40 with a mean of 0.30 (Crawford and Eden, 1965). Furthermore, it is generally the case that the stronger the undrained shear strength, the greater the value of the drained shear strength parameters. It appears then that it might be possible to estimate drained shear strength parameters by the use of simple, fast, inexpensive, undrained field and laboratory tests.

Thus the specific problem to be researched is: "Can the effective stress shear strength parameters of c' and ϕ', used in slope stability analyses, be easily yet reliably estimated on the basis of simple, undrained, field or laboratory shear strength tests?"

1.3 Objectives of the Research

The principal objective of the research is to investigate the relationship between the undrained and drained shear strength behaviour of Leda or Champlain Sea clays and to provide a means by which the effective stress shear strength parameters of c' and ϕ' may be estimated by the use of simple, undrained shear strength tests.

Along with the principal objective, there are the following additional objectives:

a) to review the current literature concerning slope stability and the measurement of shear strength of Champlain Sea deposits, and attempt to explain the many conflicting results and viewpoints that have been presented,

b) to review and provide a clearer picture of the geological history of the Ottawa River Valley region based on recently published articles in the literature, and

c) to discuss the geotechnical behaviour of the different geological facies of clay known to exist in this region.

1.4 Outline of Thesis

The following is a chapter-by-chapter summary of the thesis contents:

CHAPTER 2 - A commentary on the methods by which shear strength is commonly measured, both in the field and in the laboratory, is presented. This is followed by a discussion of several factors such as sampling disturbance, anisotropy, time effects and progressive failure, which may affect the mobilization of the measured shear strengths along an actual in-situ failure surface.

CHAPTER 3 - This chapter provides a review of the current literature concerning the shear strength of Leda clays and the results of effective stress stability analyses. The basic soil properties, structural integrity, stress-strain behaviour and measured shear strengths of Leda clay are discussed. In addition, the effects of various factors such as sampling disturbance, time, anisotropy and stress path followed to failure on the effective stress failure envelope are also discussed. An explanation for the conflicting results reported in the literature is provided by considering the effect of different test procedures and the structural integrity of the clay tested. The relative success with the use of peak and residual effective stress shear strength parameters in slope stability analyses is discussed.

CHAPTER 4 - A review is made of the current literature concerning the recent (Pleistocene) geological history of the Ottawa River Valley region. The heterogeneous nature of the sedimentary suite of deposits is described and the nature and geotechnical properties of the various facies of the deposited clay are discussed.

CHAPTER 5 - Presented are the site descriptions, recent geology and soil stratigraphy of the five sites located throughout the Ottawa River Valley region from which block samples were obtained for laboratory testing.

CHAPTER 6 - This chapter discusses the procedures followed in the field regarding block sampling and in-situ vane shear testing. In addition, the rationale and the procedures followed for all the laboratory tests are discussed.

CHAPTER 7 - This chapter presents the results of all the laboratory tests performed for each site. These test results include: basic identification tests, grain size analyses, oedometer tests, unconfined compression tests, laboratory vane tests and a series of consolidated isotropically undrained triaxial tests with pore pressure measurements. The concept of a yield point under a certain value of average effective stress is discussed with respect to isotropic consolidation and triaxial shearing. Strain contours are plotted for the CIU tests and plots indicating the mobilization of c' and ϕ' with strain are presented. Pore pressures mobilized during shear are discussed in terms of the pore pressure parameter A with reference to sample strength. Modes of failure observed during the test are described. Finally, both p-q and τ-σ_n failure envelopes are presented for each site studied and the results are discussed.

CHAPTER 8 - This chapter interprets the effective stress shear strength envelopes of the thesis test results in an attempt to relate the undrained vane shear strength and the effective stress shear strength parameters of c' and ϕ'. Such a relationship is presented for both peak and residual effective shear strength parameters. In addition, results reported in the literature for both intact and microfissured clays are compared with the thesis results. Finally, a proposed relationship is illustrated that will enable an estimate of c' and ϕ' to be made for either microfissured or intact clays based on field vane strengths alone.

Conditions for which the proposed relationship will not apply are then discussed. Lastly, procedures that should be followed for the preliminary estimate of the stability of a slope are recommended.

CHAPTER 9 - A summary of accomplishments, list of important conclusions and a list of additional noteworthy conclusions are presented. Recommendations and suggestions for further work are provided.

APPENDIX A - Presented in this Appendix are additional laboratory test results for laboratory vane tests and unconfined compression tests. Also presented are summary tables for the results of the CIU triaxial tests and the unconfined compression tests.

APPENDIX B - Presented in this Appendix are several effective stress failure envelopes for microfissured and intact Leda clays from the literature and from other sources available to the author. These failure envelopes were used to develop the proposed relationship between vane strength and effective shear strength parameters presented in Chapter 8.

APPENDIX C - Presented in this Appendix is the computer program written to compute and plot the results of the triaxial tests. Also presented is a sample of the computer output for one particular test.

APPENDIX D - Presented in this Appendix are the calibrations performed on two oedometer devices, two porewater pressure transducers, and two load cells.

CHAPTER 2

METHODS OF MEASURING SHEAR STRENGTH AND FACTORS AFFECTING THE MOBILIZATION OF MEASURED SHEAR STRENGTHS ALONG IN-SITU FAILURE SURFACES

2.0 Introduction

This chapter is divided into two parts:

Part A - Methods of Measuring Shear Strength

Prior to any detailed discussion of the shear strength of clay, it is necessary to appreciate the merits and deficiencies of the test methods by which shear strength is measured. This section presents a brief commentary on the methods by which the shear strength of clay is commonly measured. Such a commentary is pertinent since several of these test methods were used in the thesis testing program, while others are discussed frequently in the literature.

Part B - Factors Affecting the Mobilization of Measured Shear Strengths Along In-situ Failure Surfaces

Occasionally, it has been found that the shear strength determined in the laboratory does not correspond to the shear strength that must have existed along an in-situ failure surface at the time of failure. This section discusses several important factors that may have a significant effect on whether the laboratory measured shear strengths will be mobilized along in-situ failure surfaces. These factors include: sample disturbance, sample size, anisotropy, time effects and progressive failure.

Part A - Methods of Measuring Shear Strength

This section presents a brief commentary on the methods by which shear strength is most commonly measured. Many of these methods have been used in the testing program for this thesis and the others are frequently mentioned in the literature. For a discussion of methods not mentioned in this chapter, the reader is referred to Ladd et al (1977) and Schmertmann (1975).

2.1 Undrained or Total Stress Shear Strength

Undrained shear strength may be measured in-situ or in the laboratory using undisturbed samples. In-situ shear strength is most commonly measured by the field vane shear test and is therefore discussed below.

2.1.1 Field Vane Shear Test

Due to its relative simplicity and low cost, the field vane shear test has become popular throughout the world. It has however, received severe criticism for the following reasons by Schmertmann (1975); Roy (1975); Arman, Poplin and Ahmed (1975); LaRochelle, Roy and Tavenas (1973) and Flaate (1966).

1) Disturbance during insertion of the vane causes excessive strain and may break "cementation bonds".

2) The actual failure surface at peak strength is not cylindrical as assumed.

3) Progressive failure may occur at the edges of the blades.

4) The rate of strain during execution of the test is excessive.

5) A delay between vane insertion and testing may cause a substantial increase in shear strength.

Its usefulness has been defended however, by Bjerrum (1973), Ladd (1975), Ladd et al (1977) and Gregerson (1975). Bjerrum (1973) presents correction factors for the field vane based on documented embankment failures to account for anisotropy and time effects. However, it is not definitely known whether these correction factors are applicable to Leda clays. Bozozuk (1977) presents further evidence in support of the use of the field vane test. He reports the use of undrained shear strengths from field vane tests best predicts the shear strength mobilized along a failure surface in-situ, based upon back calculations of three silo tower foundation failures. Ladd et al (1977) in their State-of-the-Art paper conclude that at present the field vane test offers the most reliable estimate of undrained shear strength for stability analyses when used in conjunction with Bjerrum's correction factors.

2.1.2 Laboratory Vane Shear Test

The laboratory vane shear test performed on an undisturbed sample provides a good check on the field vane test results. Most comments listed earlier for the field vane test apply equally well for the lab vane.

Attempts to use the lab and field vane test to study strength anisotropy by using different size vanes and various orientations, such as described by Aas (1965) and Donald et al (1977), are of questionable value according to Ladd et al (1977).

2.1.3 Unconfined Compression Tests

Unconfined compression tests have long been used by geotechnical engineers as a simple, inexpensive, rapid measure of the undrained shear strength of clay. However, they generally provide an uncertain measure of shear strength for the following reasons:

1) Unconfined compression tests are often performed on small diameter tube samples, which are somewhat disturbed. Studies show that sampling disturbance can reduce the undrained shear strength by 30% to 60% compared to tests performed on block samples [Bozozuk (1970), LaRochelle and Lefebvre (1971), Raymond et al (1971)].

2) The high strain rate, typically 1% per minute - 60% per hour, overestimates the undrained strength for most clays.

Therefore caution must be exercised when applying the results of unconfined compression tests to practical problems.

2.1.4 Triaxial Tests

The triaxial test is the most common and versatile test used to determine the stress-strain properties of a soil. It has the following advantages over most other tests:

a) The principal stresses are known and controllable.

b) Porewater pressures or volume changes may be determined.

c) The failure plane may be directly observed.

Difficulties in interpretation of the strength measurements arise if the sample deforms into a distorted shape. Distortion may arise as an effect of restraint imposed by the rigid end caps. In general, the distorted shape is often ignored by the use of simplified assumptions concerning the change in area with axial strain of the sample. Another difficulty with the test is that porewater pressures are measured at the base of the sample, but shear failure generally takes place in the middle of the sample. If the test is run at standard rates of strain that vary from 0.5 to 2.0% per hour [Bjerrum (1973), Bozozuk (1977)], it is generally assumed that the porewater pressures have had time to equilibriate. Thus, the porewater pressure measured at the base presumably represents the porewater pressure on the failure plane.

Unconsolidated undrained (UU) tests and consolidated undrained (CU) tests are popular since they are simple and may be performed rapidly. If tube samples are used, the effects of sample disturbance and stress relief can be reduced by consolidation. Consolidation is generally isotropic but the influence of an anisotropic stress system ($\sigma_h = K_o \sigma_v$) maybe studied by anisotropic consolidation of laboratory samples. If porewater pressures are measured, both the effective and total stress shear strengths may be determined.

2.2 Drained or Effective Stress Shear Strength

2.2.1 Consolidated Drained (CD) Triaxial Tests

CD tests are also commonly used in engineering practice. However, in order to ensure full drainage they must be run at very slow strain rates. Typical strain rates are from 1 to 2% per day. Test duration depends upon the permeability of the material and the final strain desired but typically may last from 10 hours to several days or even weeks. By carrying out a series of CD tests at different confining pressures, the effective stress parameters of c' and ϕ' can be determined directly.

It is generally agreed that both CU and CD tests describe basically the same effective stress envelope. Evidence to support this conclusion may be found in Lefebvre and LaRochelle (1973), Mitchell (1970a) and Bjerrum and Simons (1960).

2.2.2 Drained, Constant p'_m Triaxial Tests

Eden and Mitchell (1970), Mitchell (1970,1975) and Mitchell and Wong (1973) favour the use of a drained, constant p'_m test, ($p'_m = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3}$) to describe the failure envelope of Leda clay for the following reasons:

1) the constant p'_m test eliminates volume changes due to changes in mean normal stress, so that volume changes due to shear (increase in deviatoric stress) can be measured,

2) the principal stresses are low and representative of the stress levels occurring in a real slope,

3) the mode of failure in the test is more characteristic of what has been observed for failed slopes in the Ottawa area.

The constant p'_m test may be stress controlled (incremental dead loading) or strain controlled. The physical boundary conditions on the sample are the same for each method. A constant p'_m test can only be performed drained however, since it is not possible to maintain p'_m constant in an undrained test by reducing the cell pressure. As an explanation of this, during the test, σ'_1 increases as the applied load increases. To maintain $p'_m = \frac{\sigma'_1 + 2\sigma'_3}{3}$ constant, a reduction in σ'_3 equal to $-\frac{1}{2}\Delta\sigma_1'$ is required. However, a reduction in cell pressure (σ_3) results in an equivalent reduction in porewater pressure (u) hence σ_3' remains the same. Thus, it is not possible to maintain p_m' constant in an undrained test.

2.2.3 Lateral Reduction Triaxial Tests

Lateral reduction triaxial tests (i.e., σ_1 constant, σ_3 reducing) are thought to best simulate the field condition of erosion or excavation at the toe of a slope.

For both the lateral reduction and constant p_m' tests, a sufficiently high value of σ_3' must be used at the start of the test, depending on the strength of the clay being tested, or else the case may occur that σ_3 will reach zero and the sample may not have failed. The sample would then have to be loaded essentially as an unconfined compression test until failure occurs.

2.2.4 Direct Shear Tests

Direct shear tests have been frequently criticized in the past for the following reasons:

1) The failure plane is predetermined along a horizontal surface and may not represent the weakest plane.

2) The magnitude and direction of the principal stresses at failure are not accurately known.

3) Progressive failure may occur due to these stress concentrations.

For these reasons, direct shear tests cannot be realistically employed to measure the peak shear strength behaviour of sensitive, brittle, cemented Leda clays. They have however, been used with success to measure residual shear strengths at large deformations.

2.3 Stress Paths Observed in Triaxial Tests

Figure 2.3a illustrates the various effective stress paths for each of the previously mentioned drained triaxial tests. It should be noted that the effective stress path of an undrained test is very difficult to predict. It depends upon the porewater pressures induced during shear. The porewater pressures may be described by the pore pressure parameter 'A' which is known to be dependent upon the degree of overconsolidation (Bishop and Henkel, 1962). Typical effective stress paths for various values of 'A' are illustrated in Figure 2.3b.

2.4 Tensile Tests

Triaxial compression tests are limited to defining the Mohr-Coulomb failure envelope to the right of the unconfined compression line indicated on Figure 2.3a. The region to the left of this line is a tensile zone.

For some overconsolidated clays, it may be desired to investigate the apparently high values of the cohesion intercept that results from the extrapolation of a triaxial compression Mohr-Coulomb failure envelope back to the shear stress axis. To determine failure points or Mohr's circles to the left of the unconfined compression line requires some form of tension testing. Simple extension type tension tests have been reported in the literature by Conlon (1966), Jarrett (1972) and

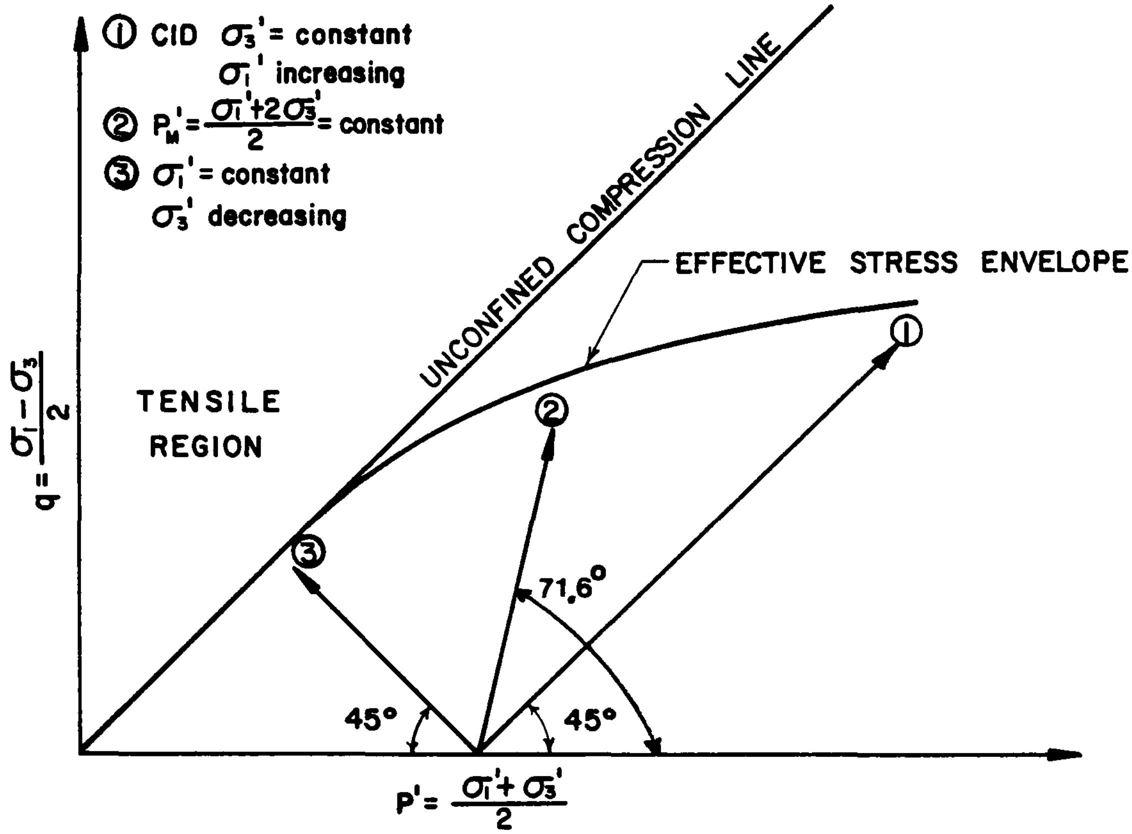


Figure 2.3 a Stress paths for drained triaxial tests.

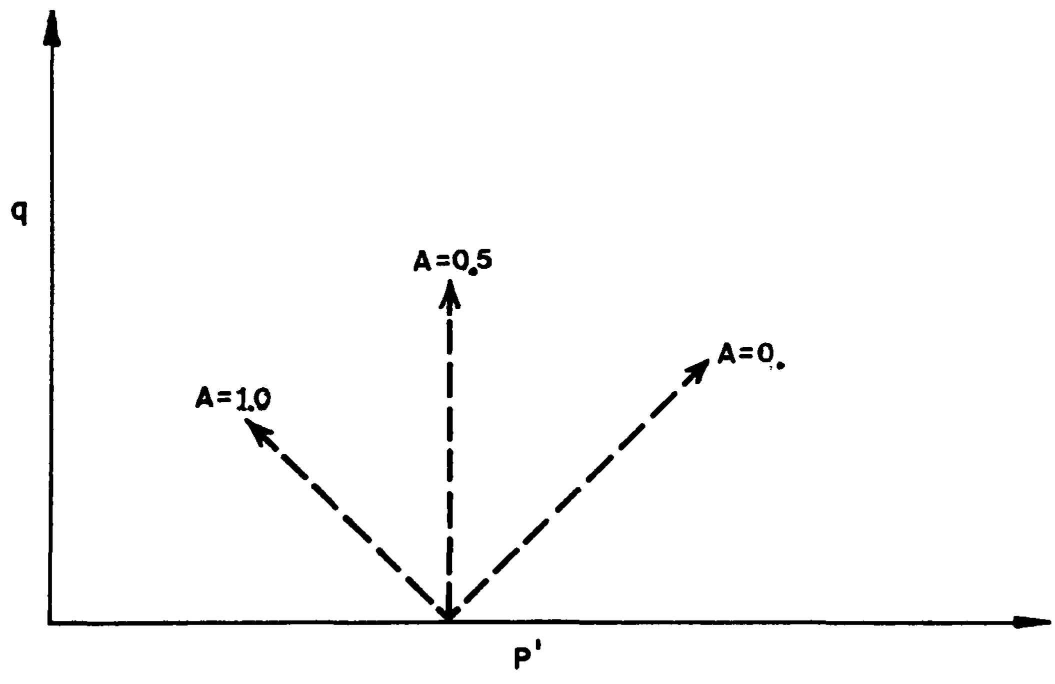


Figure 2.3 b Possible stress paths for undrained triaxial tests.

Mitchell and Wong (1973). Although these tests are crude and may not provide an exact value of the tensile strength of the soil, they do provide an estimate which is most probably a lower bound.

Brazilian type splitting tests have been carried out on Leda clays by Lawrence (1969), Lo and Morin (1972), Yong (1976) and Sangrey (1972). Brazilian tests provide a simple, rapid estimation of the tensile strength of cohesive soils of low plasticity that exhibit brittle characteristics. Erroneous results can be obtained if the assumption that brittle, tensile failure occurs in a diametric plane in the direction of loading is not satisfied. The authors of the references cited above remark that failure did occur in this mode and therefore the results should be considered reliable. Krishnayya and Eisenstein (1974) discuss the Brazilian test and propose a numerical solution of the biaxial stress state of tension and compression existing at the center of the specimen.

Part B - Factors Affecting the Mobilization of Measured Shear Strengths Along In-Situ Failure Surfaces

2.5 Sampling Disturbance

Eden (1971a), LaRochelle and Lefebvre (1971), Bozozuk (1971), Lo, Adams and Seychuck (1969) all indicate that serious disturbance may results with the use of thin-walled tube samples in clay. This disturbance becomes more severe as clays progress from the normally consolidated state to overconsolidated cemented clays (Eden, 1971b). This disturbance is demonstrated by a reduction in unconfined compressive strength and modulus of elasticity by as much as 50 to 60% of the values measured by block samples.

Bozozuk (1971) indicates that less disturbance is obtained with larger diameter sample tubes. LaRochelle and Lefebvre (1971) indicate that it is not the size of the tube sampler that is important but rather the "area ratio" that controls the degree of disturbance. This "area ratio" is defined as the area of the annular ring between the inside and outside diameters of the tube sampler divided by the area of the actual clay sample. Sampling disturbance is the result of the "strain occurring from the change of volume consecutive to the intrusion of the sampling tube into the soil mass," (LaRochelle and Lefebvre, 1971). The authors indicate from the results of triaxial tests on block samples that the lateral strains necessary to destroy the "cementation bonds" are smaller than the probable strains resulting from the use of thin-walled samples. They suggest that in order to obtain undisturbed samples of cemented clays, "a special type of sampler will have to be developed with an area ratio much smaller than 10 percent and possibly with a diameter larger than 54 mm.....".

Eden (1971a) suggests that large, undisturbed block samples yield the most reliable results for sensitive clays. LaRochelle and Lefebvre (1971) state that block samples do not appear to be significantly affected by stress relief.

The effect of sampling disturbance on the peak effective stress envelope of Leda clays will be discussed in more detail in Chapter 3.

2.6 Sample Size

Testing small diameter samples in the laboratory, even if completely undisturbed, may not yield strength results that are representative of the strength that may be mobilized along a failure surface in the field. This is the result of the existence of structural imperfections such as fissures, silt and sand seams, etc., that may not be present in these small diameter 'intact' laboratory samples. The laboratory test results may thus indicate significantly higher strength results than that which may actually be mobilized in the field, and consequently predict unrealistically high factors of safety that would result in an unsafe design.

The extent of the inaccuracy of using small diameter laboratory samples depends upon the frequency of structural imperfections in-situ. Lo (1970) has proposed a 'strength-size relationship' to predict the operational undrained strength in the field based upon a series of laboratory strength measurements on various sizes of samples. He indicates that strength of fissured clays decrease with increasing sample size and that significant error would result if the intact clay strength was applied to a slope of fissured clay.

A good example of this problem is presented by Lafleur (1978) who carefully determined effective shear strength parameters of $c' = 12.5$ kPa and $\phi' = 57.5°$ for peak strength and $c' = 2.8$ kPa and $\phi' = 40°$ for residual strengths from conventional CIU and CID tests. The samples tested were 3.55 cm in diameter by 7.1 cm in length and were cut from block samples of a stiff, brown, fissured Leda clay crust taken from a slope on Normandie Street in Hull, Quebec. However, back calculations of a slope failure occurring in this material, where the groundwater conditions were well known, indicated a $c' = 3$ kPa and a $\phi' = 27°$ to have been mobilized at the time of failure. Therefore, significant error can occur when attempting to extrapolate laboratory results to field problems if great care is not taken to consider the presence of in-situ structural imperfections.

2.7 Anisotropy

During sedimentation, clays are deposited under anisotropic stress conditions. The major principal stress is in the vertical direction(σ_v) and the minor principal stress is in the horizontal direction ($\sigma_h = K_o\ \sigma_v$ where K_o = coefficient of lateral earth pressure at rest). The continued deposition of more sediment results in one-dimensional consolidation of the clay under this anisotropic stress system. It is generally accepted that clay platelets tend to become oriented perpendicular to the direction of the major principal stress during one-dimensional consolidation. The extent of this orientation depends upon the magnitude of the major principal stress and the amount of induced strain. This preferential orientation of clay particles could conceivably cause both the strength and compressibility of the clay to vary with direction (i.e., could cause the clay to be anisotropic with respect to strength). This preferential orientation of clay particles is called "inherent' or 'fabric' anisotropy.

Failure surfaces in the field are generally approximated by circular arcs such as illustrated in Figure 2.7a. The inclination of

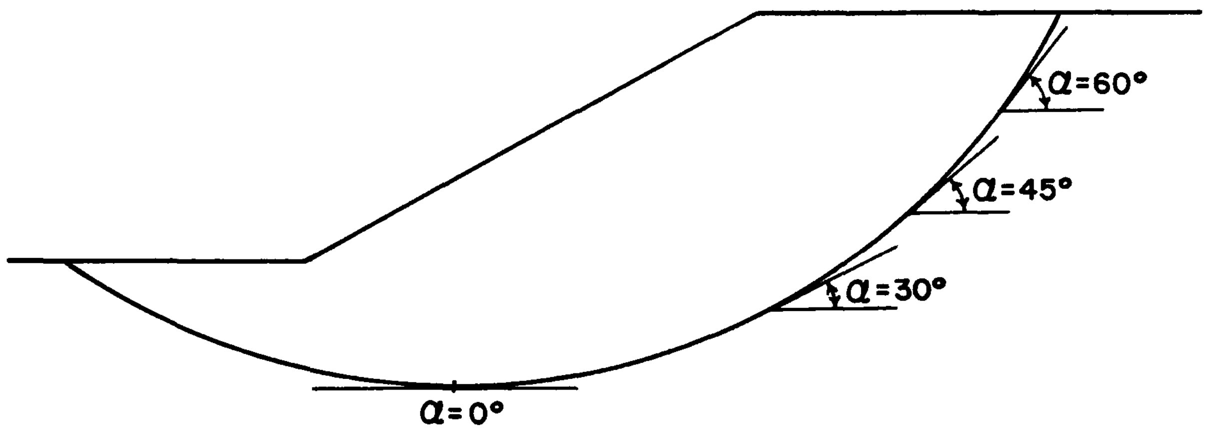


Figure 2.7 a Variation of angle of inclination along a failure arc.

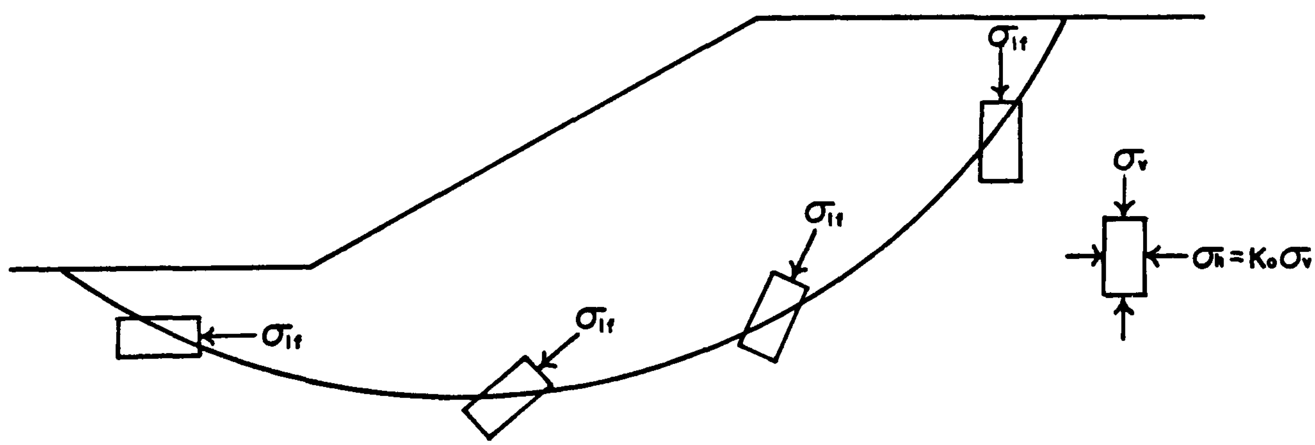


Figure 2.7 b Rotation of principal stress along a failure arc in an anisotropic stress system.

the failure surface with respect to the horizontal plane varies along this failure surface as illustrated. It would be expected then, that if the soil is anisotropic with respect to strength, that different values of shear strength would exist along this circular arc at failure. Failure to account for such anisotropy could result in an unsafe design.

In addition to inherent anisotropy there also exists a "stress system induced" anisotropy. This results from the fact that most in-situ soils are subjected to an anisotropic stress system where $\sigma_1 = \sigma_v$ and $\sigma_3 = K_o\ \sigma_h$. In an anisotropic stress system, different amounts of shear stress are required to produce failure as the major principal stress at failure (σ_{1f}) varies between the vertical and the horizontal direction, "even if the clay were isotropic with respect to all of its physical properties" (Duncan and Seed, 1966a). Figure 2.7b shows an anisotropic stress system and the variation of the direction of principal stress at failure along a circular arc.

Since it is likely that nearly all clays are anisotropic to some degree, and because stresses in-situ are also anisotropic, the effects of inherent anisotropy and the reorientation of principal stress are usually combined in field problems and may be termed "overall anisotropy".

Testing samples trimmed from different orientations in compression, such as discussed by Lo and Morin (1972), Eden and Mitchell (1970) measures inherent or fabric anisotropy only. In order to measure overall anisotropy, the clay fabric should be oriented as it would occur in the field, an anisotropic stress system should be applied and then it must be possible to reorient the principal stresses as they would occur along a failure surface. Such a test procedure is difficult to obtain with present day equipment.

The best procedure proposed to date to determine overall anisotropy is that discussed by Bjerrum (1973). He recommends that active (A) and passive (P) triaxial tests, together with direct simple shear (D) tests, be carried out as a standard for obtaining 3 sets of shear strength results. This testing technique abbreviated 'ADP', illustrated in Figure 2.7c, better represents the conditions of shear (i.e., the reorientation of principal stresses) along a failure surface. By combining the results of these 3 shear strength measurements, a better estimate of the mobilized shear strength along a failure surface is believed to be obtained. Bjerrum (1973) reports that good results have been obtained using this method to determine anisotropic undrained shear strengths.

Anisotropy has great practical significance, especially with lean (low clay content, low plasticity) sensitive clays, and may result in a ratio of undrained strength in the horizontal and vertical directions (Su(H)/Su(V)) in the order of 0.5 ± 0.2 (Ladd et al, 1977). Anisotropy may also exist with regard to elastic moduli, porewater pressures and Mohr-Coulomb failure envelopes. Anisotropy is related to plasticity index (Bjerrum, 1973; Ladd et al, 1977). It is greatest in clays of low plasticity and decreases with increasing plasticity. A possible explanation for this would be that in lean clays friction plays a predominant role in the shear strength of the sample, and friction depends upon particle contact which has a preferred orientation.

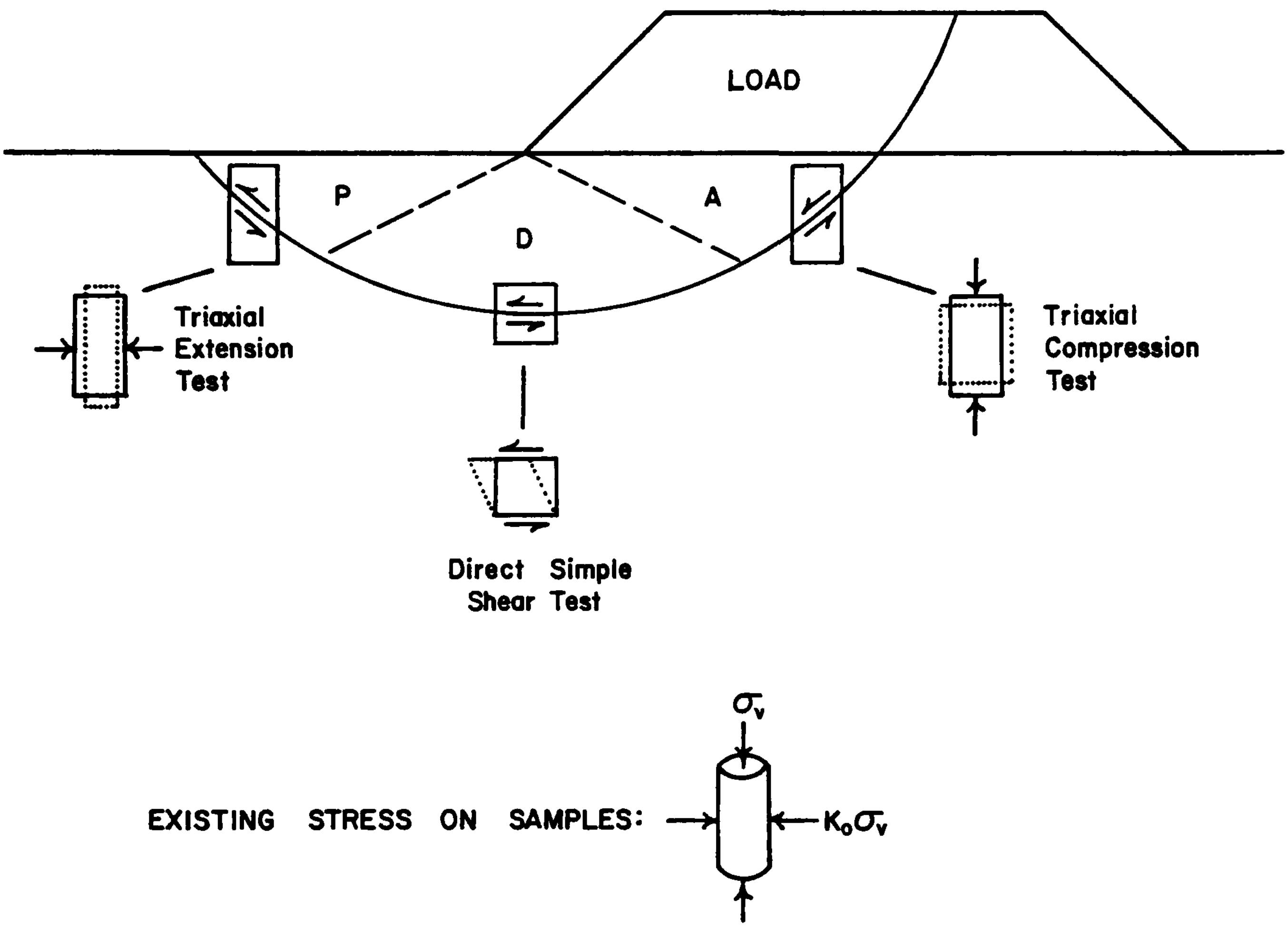


Figure 2.7 c Shear and stress conditions along a failure surface and recommended tests (after Bjerrum, 1973).

Duncan and Seed (1966a) found undrained strengths and the pore-water pressure parameter A_f to be highly anisotropic for overconsolidated kaolinite. However, the effective stress parameters of c' and ϕ' were found to be isotropic. Most natural clays are believed to have anisotropic undrained shear strengths but it is not known whether the effective stress parameters are also anisotropic for all clays. Measured values of anisotropy for Leda clay will be discussed in Chapter 3.

2.8 Time Effects

Time has an important effect on the shear strength mobilized in the field. This effect was recognized as a consequence of a review of a number of failures of embankments constructed on soft clay by Bjerrum (1972) which indicated that the shear strength at failure was considerably less than the value observed in conventional shear tests. In addition, it was known that some clays would creep under a sustained load and ultimately fail under a sustained load that was appreciably less than the strength indicated by normal laboratory tests.

Bjerrum (1973) explains that the reason time has an effect on the properties of clay is that the cohesive component of shear strength is of a viscous nature due to the resistance of the adsorbed water layer that surrounds a clay particle. In contrast, the frictional component of shear strength is to a greater extent independent of time.

An understanding of the effect of time on shear strength lies with the knowledge that failure is controlled more directly by strain than by stress. As an explanation of this, it is generally accepted that macroscopic failure will take place when a slip surface or a system of slip surfaces are formed which represent planes of weakness and thus establish a possibility of accelerated shear movements (Bjerrum, 1973). The formation of a slip surface is likely to occur when the shear strain reaches a point where cohesive bonds begin breaking and a local re-orientation of particles starts to take place allowing for continued shearing along a thin plane of weakness, which is the observed failure plane. The effect of time on the shear strength of clay is thus a question of the time required to reach the critical shear strain to allow the re-orientation of particles along a developed plane of weakness. Rizkallah (1977) presents electron micrographs of this thin zone of particle re-orientation along the failure plane.

Failure will occur in some clays under a sustained load less than ultimate when the critical strain is reached by creep over a certain period of time. In other words, the cohesive bonds have a certain 'life-time'. If the shear stress is small, the lifetime will be long, perhaps hundreds of years. If the shear stress is high, the lifetime will be short, perhaps only minutes or days.

Generally, the more rapidly a shear stress is applied,the higher the shear stress at failure. This is simply due to the fact that less time is available for straining (i.e., more time was allowed to increase the load before the critical strain was reached). Failure therefore, appears to be strain and time controlled as well as stress controlled.

Bjerrum (1973) estimates that triaxial compression undrained strength values decrease 10 ± 5% per log cycle of strain rate for sensitive and plastic clays. Lo (1972) mentions a 10% reduction in strength per log cycle of time.

2.9 Progressive Failure

Most present day methods of slope stability analyses are based upon the concept of limiting equilibrium. Limit equilibrium assumes that at the moment of failure every part along the failure surface simultaneously attains the maximum strength, drained or undrained as the drainage conditions dictate. This is equivalent to assuming that the material exhibits elastic-perfectly plastic stress-strain properties which satisfies the Mohr-Coulomb criterion.

This method of analysis has proved acceptable for soils which exhibit elastic-plastic behaviour but yields misleading results for brittle or strain softening soils (i.e., soils that have a pronounced decrease in shearing resistance after peak failure at low strains). The assumption that peak shear strength is reached simultaneously along a failure surface in a brittle soil would require that an equivalent value of strain must exist along the complete length of the failure surface. This can occur only if the failing mass of soil moves as a single solid body, which is unlikely.

In a slope, the most stressed area occurs near the toe as illustrated by finite element programs by Lo and Lee (1973). As the shear stress increases relative to the shear resistance, possibly due to erosion at the toe of a slope or due to loading at the top of the slope, strains progress non-uniformly along the critical shear surface. Because of this progressive development of strains that must take place, it is impossible to mobilize the peak shearing resistance along the complete failure surface.

At the moment of incipient failure in a slope (i.e., when the factor of safety equals 1.0), the shearing resistance mobilized along a failure surface must vary from post-peak in the most overstressed zone, to peak conditions over part of the failure surface and most probably pre-peak conditions in the least stressed zone. As the slope begins to move, the peak strength is surpassed along the unfailed portions of the slip surface and only the residual strength is operative. Therefore, one must consider that at the moment of incipient failure, when the factor of safety of the slope reaches a value equal to 1.0, that some average value of shear strength, less than the peak, but greater than residual, acts along the failure surface as a whole.

The incompatability of strains along a failure surface are further illustrated by the fact that the strain to failure is anisotropic for many clays (Lo and Morin, 1972; Duncan and Seed, 1966b). That is to say, that different values of strain are required to cause failure as the inclination of the failure surface changes along its length. Generally samples fail at a strain less than that determined from a vertical test, as the inclination varies. Thus, certain parts of the failure surface may reach peak strength even before that anticipated according to vertical compression tests.

The process of individual soil elements in a soil mass reaching peak strength and then successively failing is called progressive failure. The conditions necessary for progressive failure to occur are according to Lo (1972):

1) the material should possess a strain softening post-peak stress-strain relationship,

2) non-uniformity of stress and strain should exist so that at some points in the soil mass the peak strength is exceeded, leading to redistribution of stresses.

To use peak strengths and not account for the possibility of progressive failure could lead to an unsafe design. A thorough treatment of progressive failure however, may be quite demanding of time and effort. It requires a "precise knowledge of the initial and long-term conditions in the field" (Law and Lumb, 1978), which may be too difficult to obtain. Lo and Lee (1973) have developed a finite element program to obtain a solution to this problem, but such a technique appears to be quite complex.

A much more simple approach using stability charts has been presented by Law and Lumb (1978). This method permits the detection of local failure in a slope and then assumes post-peak strength to be operative over that locality while peak strength is operative over the portion of the slope that has not suffered local failure. The authors report the method to be successful based upon the re-examination of three, well documented case records. The stability charts presented are,however, limited to soils that have the same post-peak behaviour of that assumed, i.e., residual $c'_r = 0$ and residual $\phi'_r = \phi'_p$(peak).

Lo (1972) also discusses a simplified approach to be used as a first attempt to assess the state of stability of the slope. He explains that when the effects of anisotropy, time and progressive failure are thoroughly accounted for, the resultant shear strength envelope may be quite similar to the residual strength envelope. He suggests that the use of the residual strength envelope in stability analyses will result in:

a) a slightly conservative factor of safety in design,

b) a factor of safety somewhat lower than unity in the analysis of landslides.

Lo and Lee (1973) and Lefebvre and LaRochelle (1973) report good results using the residual strength envelope in stability analyses.

Another interesting approach to the problem of progressive failure in an undrained analysis can be concluded from the work of Bozozuk (1977). He states that the results of shear strength measurements determined from vane strengths best predicts the "average" shear strength mobilized along a failure surface caused by the foundation failures of silo towers. He explains that this may be due to the fact that both the foundation failures and vane shear test mobilize a value of shear strength that is intermediate between peak and residual values due to the effect of progressive failure. From this result, it may be implied that progressive failure can be incorporated into an undrained analysis by the use of field vane shear strength.

2.10 Summary

Part A of this chapter provided a commentary on the various types of tests presently used to measure both the drained and undrained shear strength of clays. It was mentioned that the field vane test appears to provide the best estimate of the total stress or undrained shear strength. Conventional CIU and CID triaxial tests are the most widely used to estimate the effective stress or drained shear strength of clays. The CIU test is the easier and more rapid of the two tests to perform, and is able to provide information on both the undrained and drained shear strengths, if porewater pressures are measured. The drained, constant p'_m test is, however, gaining wider acceptance especially for fissured clays.

Part B of this chapter discussed the important factors of: sample disturbance, sample size, anisotropy, time effects and progressive failure and their significant influence on the mobilization of the laboratory measured shear strengths along in-situ failure surfaces. It was mentioned that if peak strengths were used, failure to account for these important aspects could result in an unsafe design.

CHAPTER 3

REVIEW OF MEASURED SHEAR STRENGTHS AND SLOPE STABILITY ANALYSES OF LEDA CLAY

3.0 Introduction

Although much research has been carried out and reported concerning Leda clay, much controversy still exists with respect to most aspects of its behaviour. The purpose of this chapter is to provide a review of the current literature concerning the aspects of measured shear strengths and slope stability analyses and to attempt to clarify and explain the reasons behind the conflicting results reported in the literature. Furthermore, the explanation and discussion of these results provides the justification for the testing programme of this thesis and the interpretation of the test results.

3.1 General

Current research studies on the numerous slope instabilities in Leda clay deposits of Eastern Canada have resulted in two directly opposite schools of thought on virtually every aspect of the shear strength of these clays and the correct approach to the stability analysis of slopes. These two schools of thought may be separated geographically and designated as either those in the vicinity of the Ottawa River Valley region or those near the Quebec City area in the St. Lawrence River Valley region.

Most members of the Ottawa Valley region school of thought were at one time or are presently associated with the National Research Council or nearby Universities and include such respected authors as C.B. Crawford, W.J. Eden, R.J. Mitchell, P.M. Jarrett, J.D. Scott and D.A. Sangrey. Most members of the Quebec City school of thought were at one time or are presently associated with Laval University in Quebec City and the University of Sherbrooke, Sherbrooke, P.Q., and include such respected authors such as P. LaRochelle, K.Y. Lo, G. Lefebvre, C.F. Lee and F. Tavenas. The above people are the authors of frequent publications concerning aspects of slope stability and shear strength of Leda clay and are listed here so that the reader may recognize names mentioned in the forthcoming discussion and associate them with the appropriate school of thought. This list is by no means complete and many other authors have made significant contributions to the available knowledge.

3.2 Nomenclature

To begin this discussion, it is perhaps best to start off with a simple topic such as the correct nomenclature for the clay deposit. LaRochelle et al (1970) suggest the name 'Champlain Sea' clay and rightly so since these sediments were deposited during the period of inundation by the Champlain Sea. The Ottawa people generally maintain the theoretically incorrect yet more recognizable name 'Leda' clay. The justification for the retention of this name and a detailed description of the recent geology of Eastern Canada is presented in Chapter 4. No special distinction should be made between these names, however, and both are used interchangeably to describe the same clay deposit.

3.3 Basic Soil Properties and Structural Integrity of Leda Clay

3.3.1 Basic Soil Properties

Crawford (1968) has thoroughly described Ottawa area clays in terms of composition, sensitivity, compressibility and overconsolidation. Crawford and Eden (1965) present a table listing the basic soil properties of the clay at thirteen sites in the Ottawa area. These soil properties are summarized in Table 3.3.1.

The people in the Quebec City region apparently have concentrated much of their reported research at two sites: St. Louis de Bonsecours and St. Vallier de Bellechasse. The soil properties at these two sites are presented by Lefebvre and LaRochelle (1973) and are also summarized in Table 3.3.1.

As can be seen from the results listed in Table 3.3.1, the soil properties of these clays are quite similar. However, the next section will illustrate that their structural integrity is quite different and this partly accounts for the conflicting results of measured shear strengths and approach to slope stability analyses.

3.3.2 Structural Integrity of Leda Clays

Scott et al (1976) suggest that two distinct Leda clay types from an engineering or structural viewpoint exist. One clay generally is a stratified, fissured material and the other clay type is a fairly homogeneous, intact material. Both materials are said to be cemented.

		Water Content (%)	Plasticity Index (%)	Liquidity Index (%)	Clay Size (%)	γ (kN/m^3)	$\frac{Su}{p'_c}$	Sensitivity	Salt Content gms/litre
Ottawa Area Clays[1]	Range	49-79	14-40	0.9-2.5	50-80	15.1-17.6	0.24-0.40	5-500+	0.4-13.7
	Mean	63.2	26.9	1.88	64.5	16.1	0.30	-	1.48
	Standard Deviation	9.3	9.1	0.48	9.8	0.8	0.05	-	2.94
St. Vallier[2]		59	37	.98	65	16.1[3]	.21	7	4
St. Louis[2]		69	23	1.82	79	17.4[3]	.29	12	0.4

[1] Data from 13 sites presented by Crawford and Eden, 1965

[2] Data from LaRochelle and Lefebvre, 1971

[3] Calculated from water contents.

TABLE 3.3.1 Basic Soil Properties of Leda Clays

The cause of the interparticle cementation is not fully understood but is believed to be the result of the deposition of cementing agents such as iron oxides or carbonates around the points of contact between particles (Penner and Burn, 1978), or these cementing agents may be smeared over the entire surface of the particle (Bjerrum, 1973).

A description of both fissured and intact clays follows.

a) Fissured clays

The cause, nature and extent of fissuring is not fully understood. Eden and Mitchell (1970) indicate that fissuring may occur on either a 'micro' or a 'macro' scale but fail to define the limits to either scale.

'Macrofissures' are visible in the oxidized crust of a clay and may be attributed to stress relief and shrinkage of the clay due to loss of moisture (Eden and Mitchell, 1970; Eden, 1975). Such noticeable fissures may also occur in clay exposed to erosion or excavation. Macrofissures do not, however, appear to be confined to the desiccated crust, which would suggest that other reasons besides loss of moisture result in their formation.

Sangrey and Paul (1971) report macrofissures at a spacing of 15 to 20 cm in a marine clay along Green Creek. Macrofissures were also observed by the author at a spacing of about 15 to 30 cm while attempting to obtain block samples from a 4.5 m deep excavation near the northern end of Green Creek. The surfaces of these fissures were stained due to the effect of water passing through them. Macrofissures oriented parallel to the slope face were also observed by the author at several other locations in the Ottawa Valley area. These macrofissures are probably the result of stress relief.

'Microfissures' are described by Eden and Mitchell (1970) as "closely spaced hairline fractures or closed fissures". According to the same authors, microfissures may occur as a result of stress relief due to overburden removal and slope cutting. The formation of microfissures by this method depends upon whether the magnitude of the swelling pressures is greater than the strength of the 'cementation bonds' in the clay. A second possible cause of microfissures proposed by Eden and Mitchell (1970) may be the fatiguing effect of seasonal changes in temperature and/or groundwater pressures on the cemented clay. However, none of these reasons can be considered satisfactory to explain why microfissuring occurs at depths as great as 20 m (Eden and Mitchell, 1970). Hence, other as yet unexplained reasons must exist to account for the formation of microfissures.

Since microfissures are apparently closed and thus not readily visible, it is difficult to identify a clay as being microfissured or intact. In fact, the actual existence of microfissures in an undesiccated, undisturbed clay has been postulated on the basis of indirect evidence only. Eden and Mitchell (1970) present a well known photograph showing a piece of clay fractured into small blocks or nodules by a torsional shearing action. They describe the nodules as generally prismatic in shape with well defined planar sides. The size of these nodules as observed in samples sheared in torsion, in landslide debris, and in samples sheared in the triaxial test, range from 1 to 10 mm on a side. The application of finger pressure to these small nodules causes remoulding into a quasi-liquid state. These dimensions of 1 to 10 mm perhaps define the spacing between what are termed 'microfissures' by Eden and Mitchell. One could briefly say then, that a microfissured clay has a system of closed fractures or planes of weakness at a spacing of 1 to 10 mm and a macrofissured clay has a system of open joints or fissures at a spacing greater than 10 mm, but more commonly in the range of 10 to 30 cm.

No studies are known to this author that provide direct evidence to prove that these microfissures exist naturally in an undesiccated clay in-situ. Such direct evidence might be provided in the form of X-ray photographs of undisturbed clay samples. Until such direct evidence is available, it will not be known whether microfissures exist in the field or if they are merely the result of sampling disturbance, stress relief, sample storage, or induced shear stresses during laboratory testing or slope failure in the field.

b) Intact clays

Apparently intact clays are known to exist in many areas in the Ottawa and St. Lawrence River Valley region (Gadd, 1975; Sangrey and Paul, 1972; Lo and Morin, 1972). These clays appear to be free of noticeable fissures. When subjected to a torsional shearing action, fracturing into small blocks or nodules is not observed. This suggests that the failure mechanism is different than that of microfissured clays. Landslide scars are known to exist in both intact and fissured clays (Gadd, 1975,1976; Sangrey and Paul, 1971; Scott et al, 1976).

3.4 Stress-Strain Behaviour and Test Procedure

This section discusses the influence of test procedure on the observed stress-strain behaviour of Leda clays.

3.4.1 Drained Constant p'_m Triaxial Tests

Eden and Mitchell (1970) and Mitchell (1970a,b; 1975) recommend the use of drained constant p'_m triaxial tests for the reasons discussed in Section 2.2.2. Constant p'_m tests may be stress controlled or strain controlled and each of these tests is now discussed.

a) Stress Controlled, Constant p'_m Triaxial Tests

During this test, a dead load is placed on the sample and maintained until the rate of strain reaches a typical value less than 2% per day (Toombs, 1974). Approximately 8 to 10 increments are applied prior to failure, with smaller increments being applied as the anticipated failure stress is approached. As the load approaches the ultimate value, increased axial straining takes place until plastic yielding under a sustained load occurs. When the critical failure strain is reached, catastrophic failure will occur under that load. Thus, the test procedure would induce an apparent elastic-plastic stress-strain behaviour similar to that illustrated in Figure 3.4.1a.

Residual strengths cannot be measured because of the catastrophic type of failure. Residual strengths may however, be measured by strain controlled, constant p'_m tests.

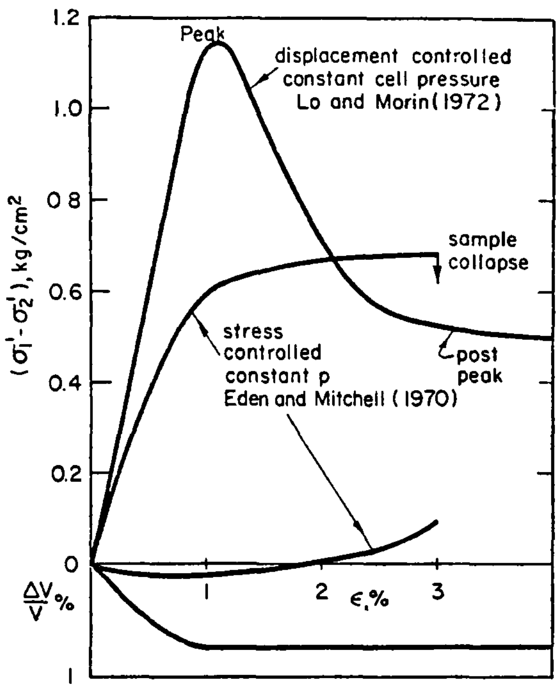


Figure 3.4.1 a Typical stress-strain curves.
(after Mitchell, 1975)

b) Strain Controlled, Constant p'_m Triaxial Tests

Strain controlled, constant p'_m triaxial tests must often be run at faster rates of strain than stress controlled tests due to the limiting displacement rates available on commercial testing equipment. Thus, as illustrated in Figure 3.4.1b, the initial tangents to the stress-strain curves of strain controlled tests are generally steeper and the samples fail at higher deviatoric stresses than the incrementally loaded stress controlled tests. This is a result of the effects of time, discussed in Section 2.8. More precisely, it is a result of the fact that as the ultimate load is approached, creep deformation under a sustained load occurs in the stress controlled tests. This causes increased straining with the result that the critical strain at failure, approximately 1% in this case, is reached at lower values of deviatoric stress.

The stress-strain behaviour resulting from a strain controlled constant p'_m triaxial test generally exhibits a peak value followed by a small reduction in deviatoric stress to a residual value. However, the validity of the residual strength measurement may be open to question. After the sample fails during a test, the principal stress σ'_1 reduces. To maintain p'_m constant, the confining pressure σ'_3, must be increased. This increase in confining pressure after failure does not represent the field condition and furthermore, effectively increases the apparent load supporting capacity of the failed sample. This results in an apparently small reduction in strength after the peak and indicates an artificially high value of residual strength.

The volume changes in both the stress controlled and strain controlled constant p'_m tests are dilative as illustrated in Figures 3.4.1a and b.

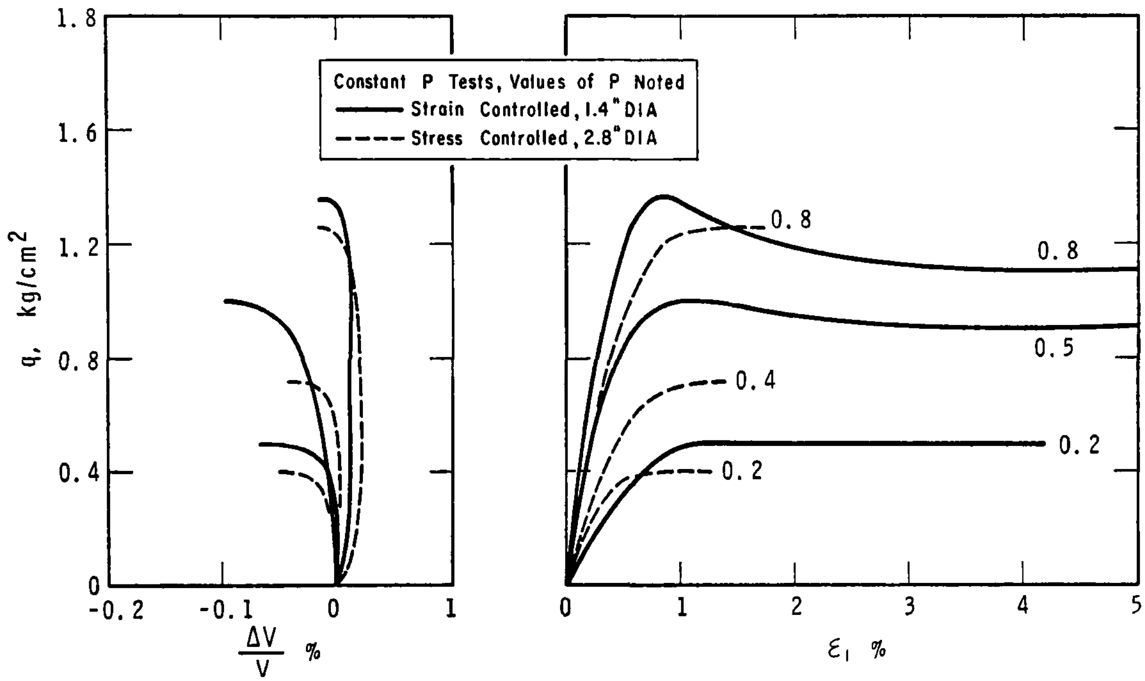


Figure 3.4.1 b Stress and strain controlled, constant p_m' tests on fissured Rockcliffe clay. (after Eden and Mitchell, 1970)

3.4.2 Conventional CIU and CID Triaxial Tests

LaRochelle and Lefebvre (1971), Lo and Morin (1972) favour the use of the more conventional strain controlled CIU and CID triaxial tests which were discussed in Chapter 2.

A CID test (σ_3' = constant) results in a constantly increasing value of $p' = \frac{\sigma_1' + \sigma_3'}{2}$ until failure occurs. A CIU test (σ_3 = constant) results in a constantly increasing value of $p = \frac{\sigma_1 + \sigma_3}{2}$ until failure occurs. Both test procedures prevent or reduce sample dilation and generally induce volume compression.

Lo and Morin (1972), Bozozuk (1977), and many others indicate that the stress-strain behaviour of Leda clay, according to the results of CIU and CID tests, is generally that of a brittle strain-softening material, although elastic-plastic behaviour is occasionally observed at very low confining pressures. The clay usually displays a peak strength at about 1% strain and then exhibits a rapid decrease in strength to a residual value. This stress-strain behaviour is also illustrated in Figure 3.4.1a. In both CIU and CID tests, the cell pressure is held constant throughout the test, not like the constant p_m' test, hence the residual strength may be more meaningful.

3.4.3 Discussion

Thus, it would appear that test procedure has a significant effect on the stress-strain behaviour of Leda clay. The apparent elastic-plastic behaviour from the results of stress controlled, constant p_m' triaxial tests appears to be merely the result of test procedure.

Although elastic-plastic behaviour occasionally occurs at very low confining pressures in conventional CIU and CID triaxial tests, brittle strain softening, stress-strain behaviour may be more characteristic of Leda clay.

The next section presents and discusses typical effective stress failure envelopes for Leda clay determined from the results of both conventional CIU and CID triaxial tests on intact clays, and stress controlled constant p'_m triaxial tests on microfissured clays.

3.5 Effective Stress Failure Envelopes

3.5.1 Intact Clays

Figure 3.5.1 illustrates a typical peak failure envelope reported for an intact clay from St. Louis, Quebec, as determined from the results of conventional CIU and CID triaxial tests. The failure envelope exhibits a high point that occurs approximately at a value of effective confining pressure $p' = 0.5\ p'_c$ (where p'_c = the preconsolidation pressure measured in the oedometer test), and then decreases to a low point. When the value of $p' = p'_c$, the failure envelope merges with the straight line corresponding to the unstructured strength of the material.

The value of $p' = 0.5\ p'_c$ indicates the average effective stress at which structural breakdown of an intact clay begins to occur. This point is of interest and will be discussed further in other sections of the thesis.

At values of $p' < 0.5\ p'_c$, the stress-strain behaviour of the material is brittle with a decrease in strength to a residual value after

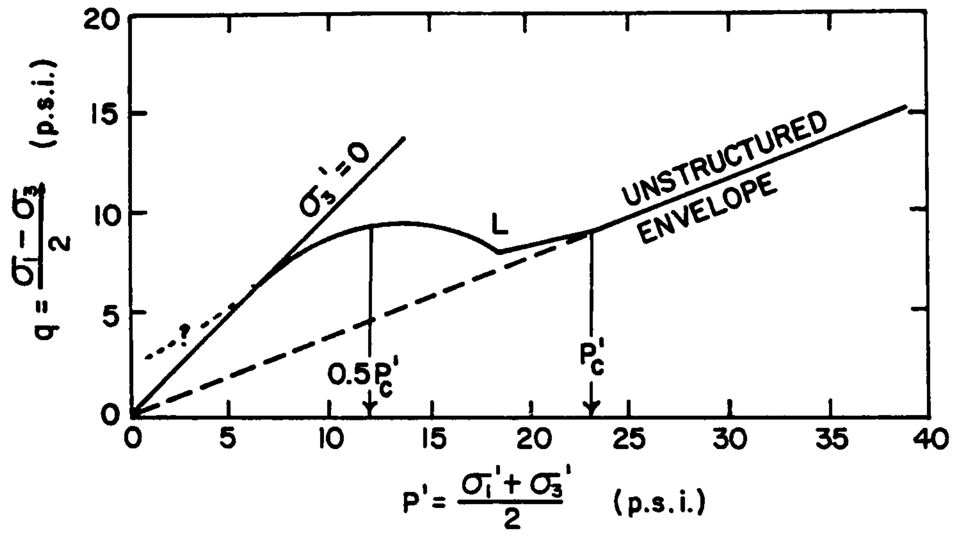


Figure 3.5.1 Peak failure envelope, intact St. Louis clay. Determined from conventional CID and CIU triaxial tests. (after Lo and Morin, 1972)

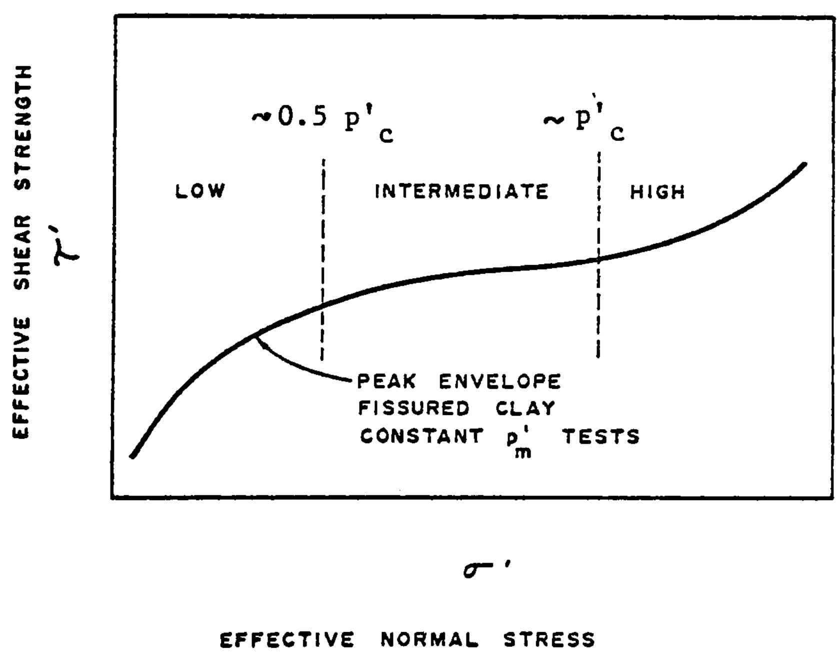


Figure 3.5.2 Typical failure envelope for microfissured clay. (after Scott et al, 1976)

the peak stress is exceeded (Lo and Morin, 1972). Failure strains are generally below 1%. The pore pressure measured in undrained tests decrease after the maximum stress difference $(\sigma_1 - \sigma_3)$ is reached. The mechanism of failure is generally by a combination of vertical splitting and a shear plane.

At values of $p' > 0.5\ p'_c$ but less than where the low point occurs in the failure envelope (point L), the failure strains increase with consolidation pressure. Pore pressures remain constant or increase slowly after $(\sigma_1 - \sigma_3)_{max}$ is reached.

At values of p' greater than point L, the stress-strain behaviour becomes elastic-plastic and the failure strains increase. Pore pressures also increase with strain. Samples generally fail by bulging.

3.5.2 Fissured Clays

Figure 3.5.2 illustrates a typical failure envelope reported for a fissured clay. More precisely the term microfissured is generally used to designate clays with this type of behaviour. Macrofissured clays can often exhibit an apparently intact failure envelope since the spacing of the fissures is often greater than the dimensions of triaxial samples.

Three effective normal stress ranges are apparent for microfissured clays. These are the low, intermediate and high stress range.

In the low effective stress range, below about $0.5\ p'_c$, the shear strength is strongly dependent upon normal stress. This shear behaviour appears to be a result of the presence of closely spaced hairline fractures or microfissures. The clay mass behaves as a "densely packed granular material" in this stress range (Eden, 1975). The failure surface of samples in this low stress range is often "stepped or nodular" (Toombs, 1974) due to dilation. Jarrett (1972) reports that for samples of microfissured Rockcliffe clay, the failure surface is reduced to a "disintegrated, wet mass of clay granules ... similar to the condition of soil debris left after landslides in this type of soil." Low cohesion intercepts and high friction angles are common in this stress range.

At stress levels between about 0.5 p'_c and p'_c, the intermediate stress range, the behaviour of the clay is controlled by the strength of the cemented clay structure and is less dependent on effective normal pressure. The stress-strain behaviour of the clay in this region is reported as being characteristic of a brittle material (Eden, 1975). This range exhibits high cohesion intercepts and low ϕ' angles.

At stress levels above the preconsolidation pressure (p'_c), the high stress range, the clay behaves in a normally consolidated, unstructured manner that results in large strains to failure and large volume reduction during consolidation and shear. The stress-strain behaviour is either that of an elastic plastic or strain hardening material. Failure of triaxial samples occurs as bulging with no distinct failure surface. This range exhibits a smaller or zero cohesion intercept and a friction angle about 2/3 the value determined in the low stress range.

3.5.3 Discussion

The low effective stress range is considered applicable for slope stability considerations according to Eden and Mitchell (1970), Mitchell (1970b), Jarrett (1972), Lo and Lee (1973) and Lefebvre and LaRochelle (1973). The low effective stress range can generally be considered to be below values of $p' = 0.5\ p'_c$. This will be further demonstrated in Chapter 6.

It would appear then that two distinctive failure envelopes exist in the low stress range; one for an intact clay and the other for a microfissured clay. Figure 3.5.3 illustrates typical effective stress failure envelopes for an intact and microfissured clay of the same undrained vane strength and with the same preconsolidation pressure.

The intact failure envelope intersects the unconfined compression line at a stress level corresponding quite well with the shear strength measured by the field vane. (This is further demonstrated in Chapter 7). The exact point at which the intact envelope intersects the $q = \frac{\sigma_1 - \sigma_3}{2}$ axis is not precisely known since tensile tests are required to determine failure points to the left of the unconfined compression line, as discussed in Chapter 2. The intercept with the q axis, which is simply related to the effective cohesion parameter c', is generally determined by the extrapolation of the failure envelope from the low effective stress region on the right of the unconfined compression line.

The apparent effect of microfissures is to weaken the strength of the intact clay. The peak microfissured envelope lies below the peak intact strength envelope in the low effective stress range. It intersects the q axis at a quite small value.

Both peak intact and peak microfissured envelopes, for clays of similar vane strength and preconsolidation, would appear to intersect at a

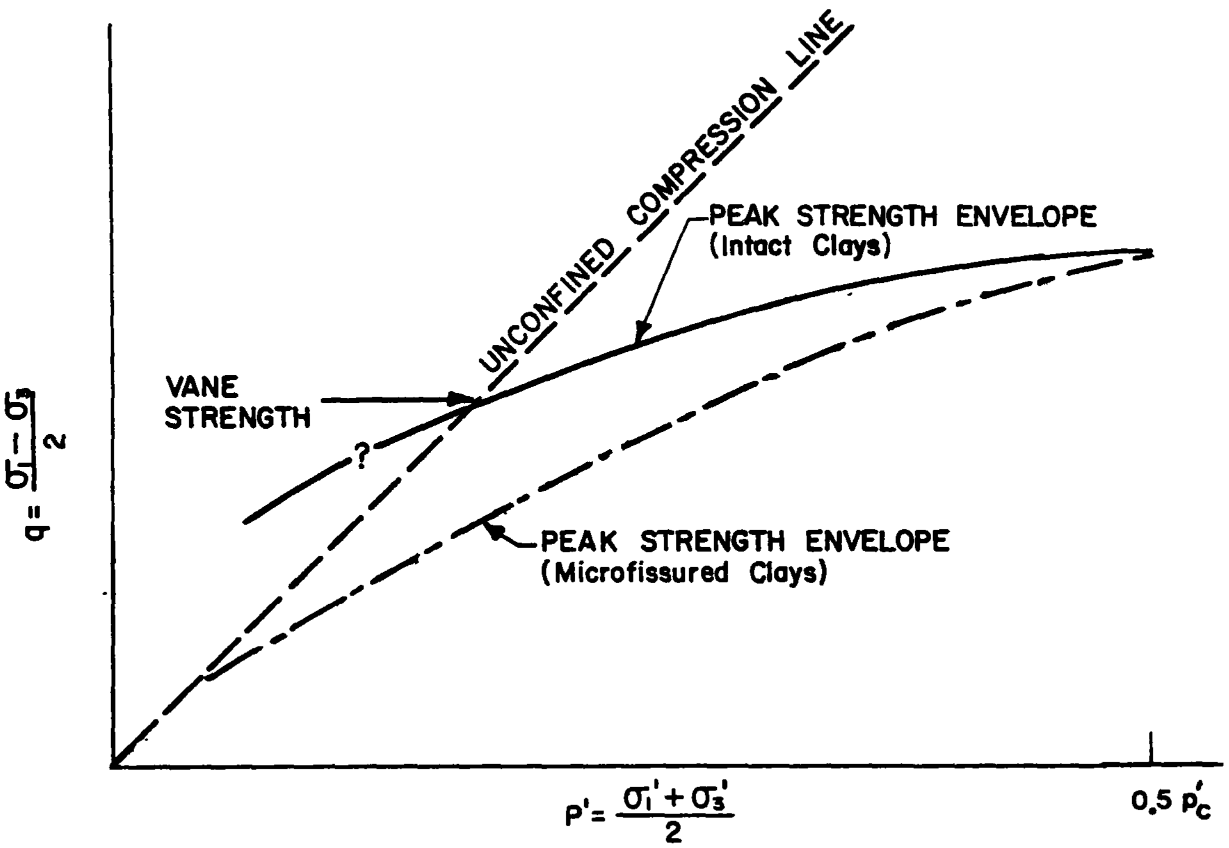


Figure 3.5.3 Typical effective stress shear strength failure envelopes of Leda clay.

value of $p' = 0.5\ p'_c$ which defines the beginning of the intermediate stress range. This is a result of the fact that the higher confining pressure causes microfissured clay to behave essentially the same as an intact clay mass. Thus, the measured shear strength of both materials is quite similar in the intermediate stress range.

The effect of various factors such as sample disturbance, sample size, anisotropy, time and stress path followed to failure on the effective shear strength envelope in the low effective stress region is discussed in the next section.

3.6 The Effect of Certain Factors on the Effective Stress Shear Strength Envelope

This section presents and discusses the effects of certain factors such as sample disturbance, sample size, aniostropy, time and stress path on the effective stress failure envelope of both intact and microfissured clays. In addition, the effect of progressive failure occurring in the field is also discussed. As the reader will realize, there is a real conflict of results and opinions reported in the literature.

3.6.1 Sampling Disturbance

In Section 2.5 references were cited that indicated sampling disturbance, as a result of using small diameter thin walled tubes, could cause a reduction of unconfined compressive strength and modulus of elasticity by as much as 50 to 60% of the values measured on samples trimmed from blocks. LaRochelle and Lefebvre (1971) indicate that the peak effective stress failure envelope is also affected by sampling disturbance. Figure 3.6.1a illustrates the results of CIU and CID tests on tube and block samples from St. Louis. The results show that "peak" failure conditions on the tube samples correspond to the "post-peak" strength at large deformation on the block samples. This indicates that the "cementation" bonds are broken during the process of sampling. Note that the peak failure envelope of the block samples begins at a point on the unconfined compression line defined by the shear strength determined from the field vane test. It would thus appear that any results lying below the shear strength level defined by the field vane would probably be from disturbed samples as indicated on Figure 3.6.1a.

Contrarily, Mitchell and Lawrence (1973), Toombs (1974) and Mitchell (1975) have found that for the microfissured Leda clays of Eastern Ontario good tube samples give the same shear strength as block samples in the low stress range of testing since the dilative frictional strength attributed to the imperfections in the material is not sensitive to disturbance. The shear strength determined by the field vane for microfissured Leda clays does not mark the beginning of the failure envelope as it did for the intact Leda clays, as illustrated in Figure 3.6.1b. It does, however, appear to indicate the shear stress level of the intermediate stress range, where the cementation bond strength dominates. The field vane then appears to measure this cementation bond strength. All points in the low effective stress range fall below the field vane strength indicating that structural imperfections such as fissures control the shear strength in this stress range, as suggested by Eden and Mitchell (1970).

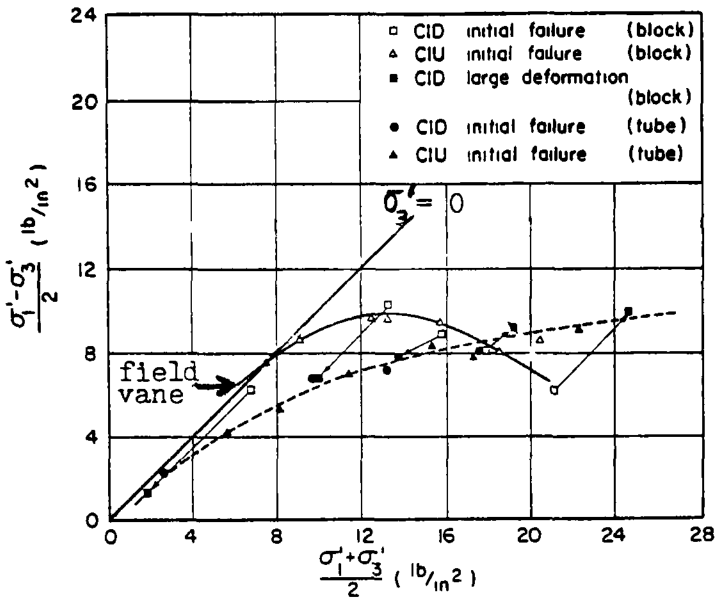


Figure 3.6.1 a Effect of disturbance on failure envelope of Champlain Sea clay from St. Louis. (after LaRochelle and Lefebvre, 1971)

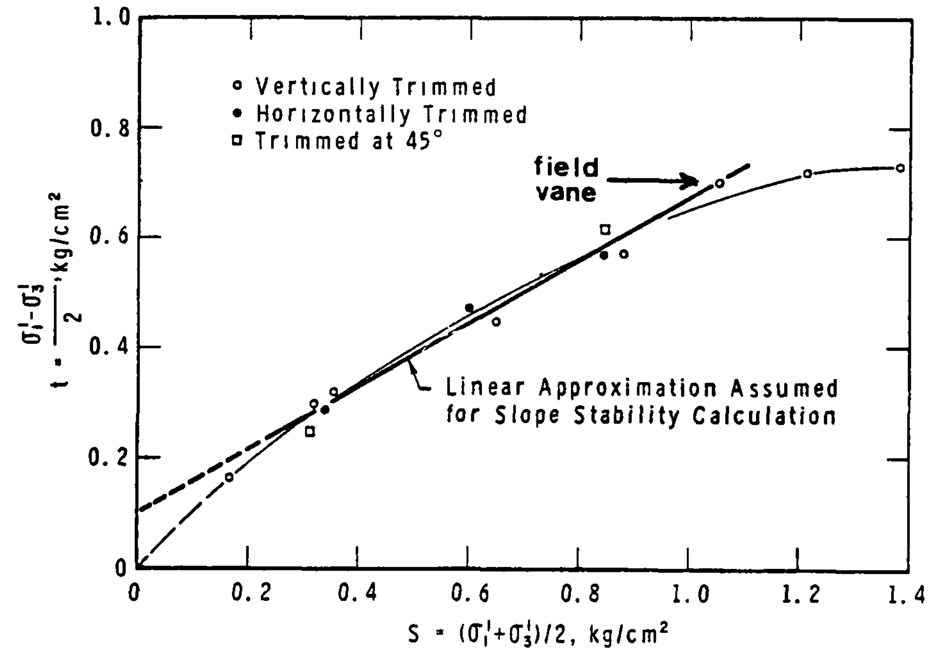


Figure 3.6.1 b Dilative failure envelope for micro-fissured Rockcliffe clay in the low stress range. (after Mitchell, 1970)

Lo (1970) has explained that the field vane test actually measures the intact shear strength of a clay even if it is fissured. This is a result of the fact that in a vane test the clay is forced to fail along a predetermined cylindrical surface, whereas the fissure pattern is predominantly planar in shape. Thus, the probability of failure occurring on any significant area of fissure surface is small, and the vane test would then in fact measure the intact strength. That is why the vane strength indicates the cementation bond strength in Figure 3.6.1b.

3.6.2 Sample Size

Intact, homogeneous clays are obviously not affected by sample size. However, the extrapolation of shear strength results, determined on apparently intact triaxial samples to a clay mass that may be macro-fissured in the field, may result in an unsafe estimate of the factor of safety. A good example of this, reported by Lafleur (1978), was discussed in Section 2.6.

Mitchell (1970b) found little affect due to sample size (3.8, 6.4, 10.2 cm diameter specimens) on the measured shear strengths of micro-fissured clays from Eastern Ontario. This is a result of the extensive occurrence of microfissures throughout the samples. Eden and Mitchell (1970,1973) suggest that 5 cm diameter samples are sufficiently large to eliminate any effects of sample size.

3.6.3 Time Effects

Mitchell (1970b) reports that for tests on microfissured Leda clay in the low stress region "failure is not affected by rates of testing ranging from 3 hours to one (The general dilatancy allows fluid to enter the specimen on demand at failure.)"

Lo and Morin (1972) report that a forty-fold reduction of strain rate for a series of CID tests resulted in a 20% reduction in the peak drained shear strengths of intact clay samples. However, they also report that the residual or post-peak envelope is independent of type of test or strain rate.

3.6.4 Anisotropy

Mitchell (1970b), testing microfissured Rockcliffe clay at inclinations of 0°, 45°, 90° from the horizontal found shear strength to be essentially independent of anisotropy in the low stress region, but dependent on anisotropy in the intermediate stress region. Jarrett (1972) also tested Rockcliffe samples at inclinations of 0° and 90°, but in the very low stress range in tension according to the method of Bishop and Garga (1969). He noted that shear strength was independent of inclination, but that mode of failure was dependent on inclination.

Lo (1972) reports that peak strengths of intact Leda clays are highly anisotropic in the low stress range, but that post-peak or residual shear strengths are independent of anisotropy in the low stress region.

It should be noted that published studies of anisotropy in Leda clay on samples cut at various orientations have measured only inherent or fabric anisotropy and not the stress-system induced anisotropy that was mentioned in Chapter 2. In order to study stress-system induced anisotropy, tests similar to those proposed by Bjerrum (1973) would have to be performed.

Tensile strengths also appear to be anisotropic although limited data exists on this aspect. Lo and Morin (1972), using Brazilian splitting tests, found a tensile strength along the stratification of 2 kPa, and across the stratification of 7 kPa. Further tensile tests have been reported but without measurements of anisotropy. For example, Conlon (1966) reports a tensile strength of 16.5 kPa for a stiff, Tolunostoc clay, determined from extension tests. Yong (1976) carried out Brazilian tests on clay from Orleans, Ontario and found tensile strengths to be 1/8 to 1/10 of the unconfined compressive strength.

3.6.5 Stress Path Followed to Failure

a) Microfissured clay

Mitchell (1975) reports the results of a series of conventional, strain controlled CID triaxial tests, as well as stress controlled, drained constant p'_m triaxial tests on samples cut from blocks from a site on Heron Road, near Bronson Avenue in Ottawa. The results of these triaxial tests, conducted in the low effective stress range, are illustrated in Figure 3.6.5a. The results indicate that conventional CID triaxial tests, even at very slow rates of strain, yield strength values significantly higher than those determined from stress controlled, constant p'_m tests. Mitchell concludes that this is a result of the dilative mode of failure that occurs at low effective stresses in constant p'_m tests, which he attributes to the presence of microfissures. He further proposes that because of the constantly increasing values of $p' = \frac{\sigma'_1 + \sigma'_3}{2}$ in a conventional CID test, dilation is prevented which thereby results in a stronger sample.

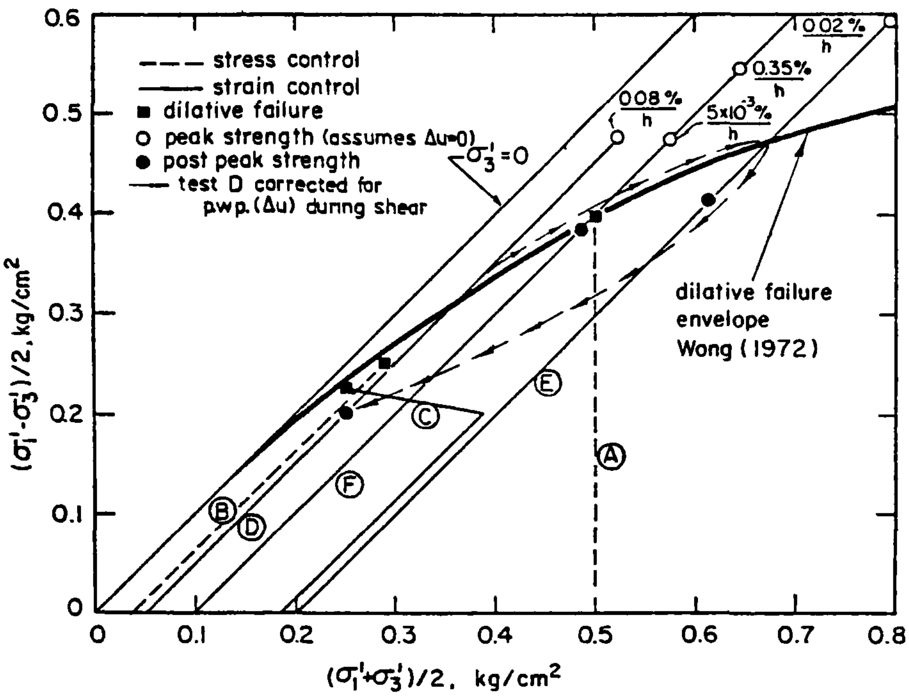


Figure 3.6.5 a Triaxial tests on clay from Heron Road. (after Mitchell, 1975)
Field vane strength = 60 kPa, $p'_c = 200$ kPa.

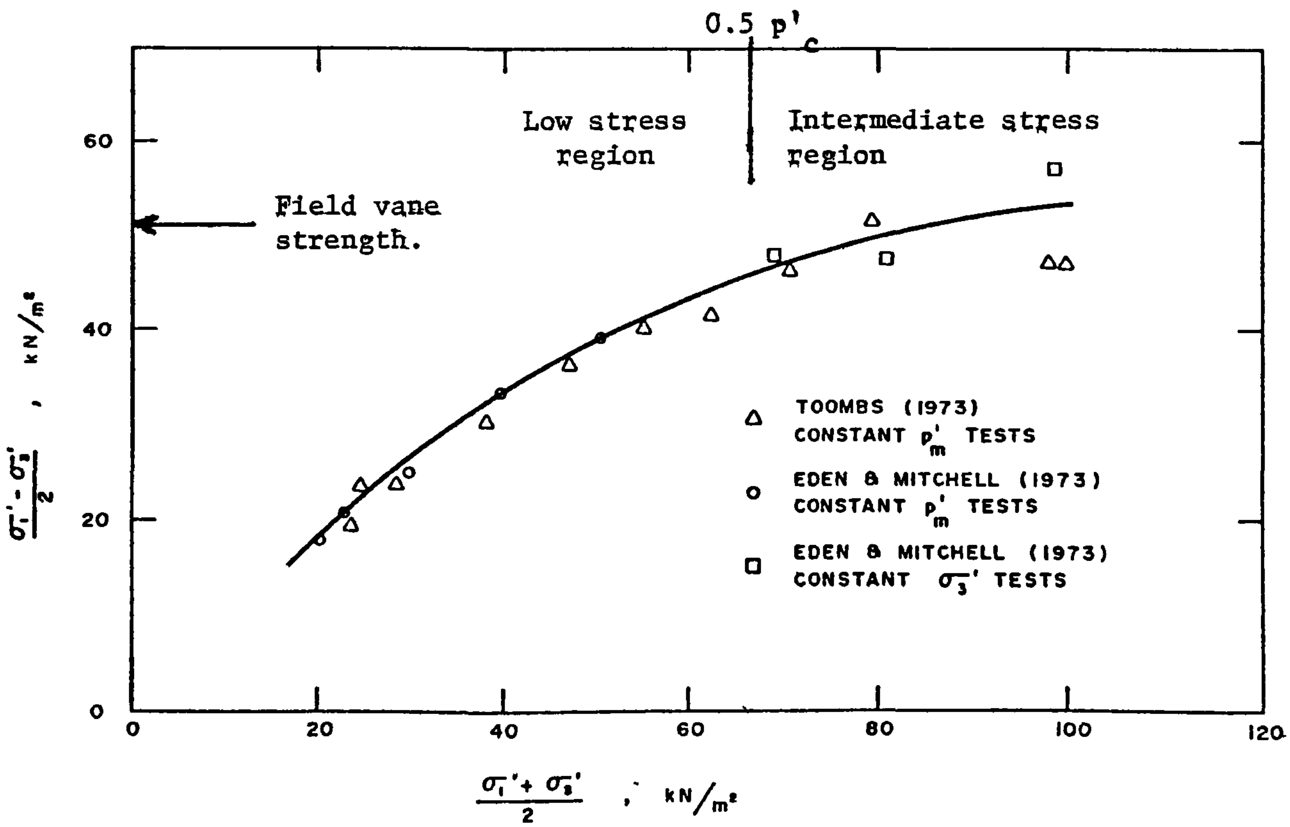


Figure 3.6.5 b South Nation River shear strength envelope. (after Scott et al, 1976)

However, the contribution of microfissures to dilation is questionable. Scott et al (1976) have shown that dilation occurs at low confining pressures during a constant p'_m test even in an apparently intact clay. Hence, it would appear that dilatancy is simply a result of shear under constant or decreasing effective normal stress and is not a direct result of microfissures.

Dilation of a microfissured clay during a constant p'_m triaxial test may permit a different mode of failure to develop than that occurring in a conventional CID triaxial test, where dilation generally does not occur. Under dilative, constant p'_m test conditions, microfissured clay behaves as a dense granular material with shear failure occurring as relative movement between clay particles. In a conventional CID triaxial test, shear failure takes place partly as a result of relative movement between clay particles and partly as a result of shear through the intact clay particles, because dilation is prevented. This results in a higher value of shear strength. Furthermore, dilation along the failure plane results in an increase in water content which subsequently reduces the strength of the clay.

Ladanyi (1970) has adapted shear strength envelopes for jointed rock masses to explain the effect of dilation and effective normal stress on the measured shear strength of a fissured clay.

It would be expected that in the intermediate stress range, where dilation is prevented, the stress path should not affect the measured shear strength results, since the mechanism of shear would be through the intact clay mass for either test procedure. This fact is illustrated in Figure 3.6.5b which illustrates the results of stress controlled constant p_m' triaxial tests and strain controlled conventional CID tests on fissured clay from the South Nation River. In the intermediate stress range, the results of three CID triaxial tests fall quite close to the values determined by constant p_m' triaxial tests.

Conventional CID triaxial test results may yield shear strength results similar to the results from constant p_m' tests, in the low effective stress range, if the strength of the intact clay mass is disturbed by sampling procedures, as might be the case with small diameter thin walled tube samples.

One important point to note from Figure 3.6.5a is that the residual strengths from conventional CID triaxial tests yield approximately the same shear strengths as constant p_m' triaxial tests. This suggests that residual strengths from conventional triaxial tests may be used to approximate the failure envelope determined from constant p_m' triaxial tests.

b) Intact clays

Few constant p_m' tests have been performed on samples of intact clay, hence it is difficult to arrive at any definite conclusions concerning the effect of different stress paths on intact clays. It would be anticipated that a dilatant mode of failure should not effect the shear strength envelope, since the mode of failure occurs through the intact clay mass whether the sample dilates or not.

Scott et al (1976) have indicated that both stress controlled, drained, constant p_m' tests and conventional strain controlled, drained

σ_3' = constant triaxial tests yield the same peak envelope for an intact clay from Hull, Quebec. These results are illustrated in Figure 3.6.5c. The constant p_m' tests exhibited greater dilatant tendencies but the dilatancy did not result in a reduction in shear strength as it would have for a microfissured clay.

It is also important to note from Figure 3.6.5c that the residual or post-peak failure envelope is not the same as the constant p_m' failure envelope, as it was for the microfissured clays.

Further stress and strain controlled, drained constant p_m' triaxial tests are required to fully investigate the effect of test procedures on the effective stress failure envelope of intact clays.

3.6.6 Progressive Failure

As discussed in Section 2.9, in order for progressive failure to occur, the material must possess a strain softening post-peak stress-strain relation. The stress-strain behaviour of an intact clay is that of a strain-softening material as illustrated by the results of Lefebvre and LaRochelle (1973), Lo (1972) and Lo and Morin (1972). Thus, the possibility of progressive failure is an important factor that must be considered in slope stability analyses of intact Leda clays. Substantial progressive failure may result in the post-peak shearing resistance being operative over a significant part of the failure surface in an intact clay.

The stress-strain behaviour of microfissured clays appears to be affected by test procedure. Both stress controlled and strain controlled constant p_m' triaxial test exhibit an artificial elastic-plastic

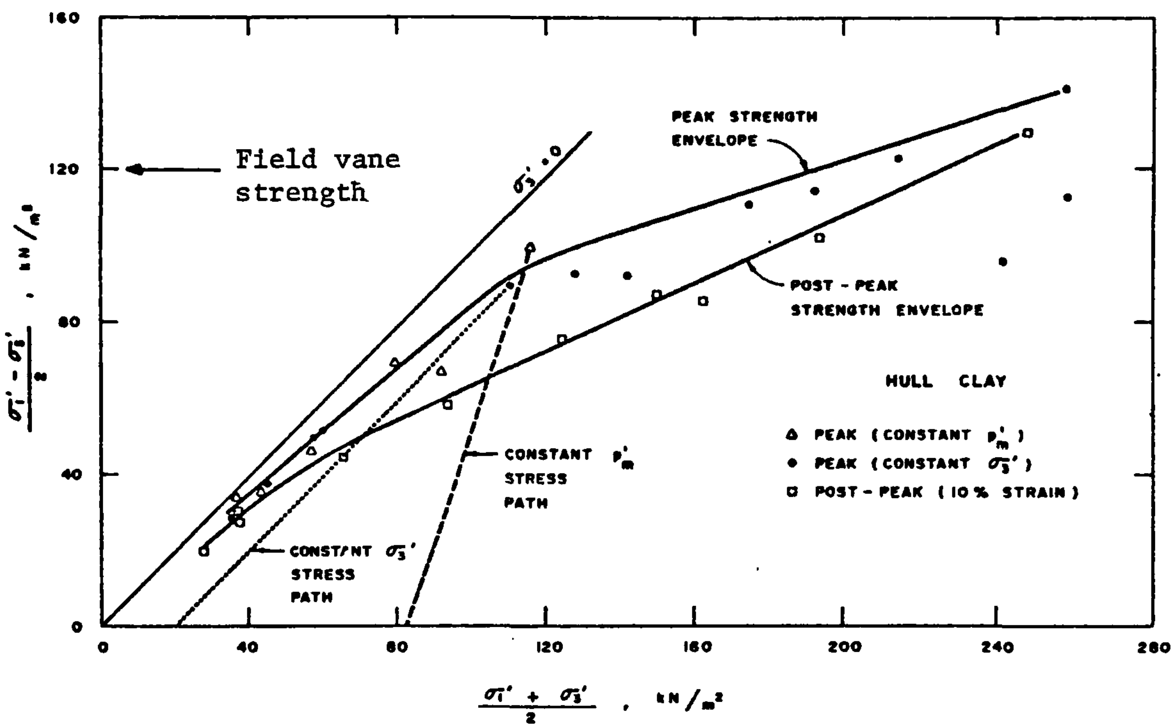


Figure 3.6.5 c Typical strength envelope for intact clay, from the corner of Jumonville and de la Verendrye Sts. in Hull, Que. (after Scott et al, 1976).

stress-strain behaviour. For conventional CID and CIU triaxial tests, both elastic-plastic and strain softening behaviour have been observed. However, it is considered that the strength of microfissured clays at large strains should be less than the peak value. Hence, progressive failure may also be a possibility in microfissured clays.

3.6.7 Discussion

This section has reviewed the effects of many factors on the effective stress failure envelope of Leda clay. Conflicting results have been reported for just about every aspect of shear strength of Leda clay. This has led to much confusion for the practicing geotechnical engineer. There are two interrelated reasons behind these many conflicting reports:

1) two different clays have been tested; one is microfissured and the other is intact,

2) different test procedures have been used in the literature to describe the behaviour of each of these materials.

Table 3.6.7 summarizes the conflicting results for the two clays as reported in the literature. This table clearly illustrates that the two clays behave apparently quite differently. However, it must be emphasized that the behaviour of the microfissured clays has been studied by the use of a stress controlled, constant p'_m triaxial test, while the intact clays have been studied by conventional CID and CIU triaxial tests. It is not known whether all the comments describing the distinct behaviour of microfissured clays in Table 3.6.7 would be applicable if conventional CID or CIU tests had been used to study its behaviour (i.e., the comments in Table 3.6.7 may be a function of test procedure).

Item	Microfissured Clays	Intact Clays
Commonly used test procedure in the literature	Stress controlled drained, constant p'_m triaxial tests	Conventional, strain controlled CIU or CID triaxial tests
Observed stress-strain behaviour using this test procedure	Elastic-plastic	Brittle, strain-softening
Characteristics of effective peak failure envelope	Low c' High ϕ'	Higher c' Lower ϕ'
Is effective peak failure envelope affected by*:		
Tube sampling	No	Yes
Sample size	No	No, only if macro-fissured
Time effects	No	Yes
Anisotropy	No	Yes
Progressive failure	No	Yes

*From the results reported in the literature using appropriate test procedure in the low stress range.

TABLE 3.6.7 Comparison of the Behaviour of Clay Types as Reported in the Literature

In retrospect, it would appear that much confusion still exists in the selection of the correct testing procedure to determine the shear strength of a clay, for the purpose of slope stability analyses. In addition, it appears that the choice of test procedure has a significant effect on the outcome of the test results, especially if the clay is microfissured.

To provide insight into the dilemma of whether the effective shear strength envelope of constant p'_m triaxial tests, or the peak or residual failure envelope of conventional CIU or CID triaxial tests should be used, it is beneficial to look at the results of stability analyses in Leda clay slopes using the results of different test procedures and different failure envelopes. This will be described in the next section.

3.7 Stability Analyses in Leda Clay Slopes

3.7.1 Total Stress Stability Analyses

Total stress stability analyses of Leda clay slopes have been found to yield unreliable factors of safety. For example, Eden and Jarrett (1971) report that a total stress analyses using undrained shear strengths determined by the field vane predicted an unrealistic factor of safety of 3.9 for a failed slope in Orleans, Ontario.

The principal reasons why total stress analyses provide unsatisfactory results in Leda clay are discussed below.

a) The field vane test measures the intact strength of the soil as explained in Section 3.6.1. Hence if the soil is fissured as at Orleans, the factor of safety will be overestimated.

b) Generally no account is made for the effects of anisotropy or the reduction in strength with time.

c) The effects of groundwater conditions are ignored.

Field observations of Leda clay slopes indicate that groundwater conditions are critical to the stability of a slope (Scott et al, 1976; LaRochelle, 1975) and that most slope failures occur in the spring and fall when groundwater conditions are most detrimental to the stability of a slope (Eden and Mitchell, 1970). Kenney (1975) estimates that seasonal fluctuations in groundwater pressures can change the shear strength of a soil by as much as ±10%. Thus, it would seem that any method that ignores such an important effect such as groundwater is bound to yield unreliable results.

Advanced forms of total stress analysis have been proposed by Ladd and Foott (1974) and Bjerrum (1973) that account for the affects of anisotropy. These methods apparently are successful for embankment design but are as yet of uncertain reliability for natural slopes and will not be discussed further.

The recognition of the fact that undrained behaviour of saturated clays is governed by effective stresses has led to the suggestion that the more theoretically correct effective stress method should be used for the stability analysis of all natural slopes (Janbu, 1975,1977; Schmertman, 1975). Only effective stress stability analysis of Leda slopes based upon determined groundwater conditions could then be considered acceptable and are discussed in the following section.

3.7.2 Effective Stress Stability Analyses

Mitchell (1970a), Eden and Jarrett (1971), Eden and Mitchell (1973) have analyzed many landslides in Ottawa area fissured clays using effective strength parameters determined from stress controlled, drained constant p'_m triaxial tests. They found factors of safety of close to unity in all cases assuming nearly full saturation of the slope. This indicates the success in using shear strength parameters determined from stress controlled, constant p'_m triaxial tests.

Lefebvre and LaRochelle (1973) analyzed slope failures in intact clays at St. Louis and St. Vallier using peak shear strength parameters determined from conventional CIU and CID tests and found a "gross overestimate of the factors of safety". When the post-peak or residual failure envelope was used, they found factors of safety close to unity.

Lo and Lee (1973) have analyzed well documented landslides at Breckenridge, Green Creek, Rockcliffe (Mitchell, 1970a) and the South Nation River (Eden et al, 1971) using residual strength parameters determined for the intact St. Vallier clay and report factors of safety close to unity. They suggest that these residual parameters can be used for the long term slope stability analysis of all Leda clays.

The success of Lo and Lee's analysis using residual factors of safety lies with the fact that at low effective stresses, both constant p'_m and conventional CID or CIU residual strength envelopes yield essentially the same effective stress shear strength parameters of c' and ϕ' for clays of similar strength and geotechnical properties.

It is apparent that peak shear strength parameters determined from CIU and CID tests on intact clay cannot be relied upon to provide reliable estimates of the factor of safety of a slope over the long term. If peak strengths were to be used, one would have to consider and correct the results for such important factors such as: anisotropy, reduction in strength with time and possibility of progressive failure.

Lo and Morin (1972) have shown experimentally that when the peak envelope is corrected for the effects of anisotropy and time effects, the resultant envelope falls near to the residual failure envelope, but slightly higher. This is illustrated in Figure 3.7.2a. It would be expected then that stability analyses using residual strength results would be slightly conservative.

The success with the use of residual strength parameters for the prediction of long term slope stability of intact clay slopes justifies their continued use, but does not prove that the concept of residual strengths existing in Leda clay slopes is correct from a theoretical standpoint. When Skempton (1964) proposed the use of residual strengths for the long term stability analysis of clay slopes, he did so based on the observation that slope failures of insensitive London clay were generally preceded by movement along a thin failure surface, in which the soil was strained to a post-peak or residual condition. Skempton specifically notes that his concepts of residual strength may not be applicable to the sensitive Leda clays of Eastern Canada.

Lo and Lee (1973a,b) propose a different mechanism to justify their use of residual strengths. By the results of a finite element program, which apparently simulates the geological development of a slope,

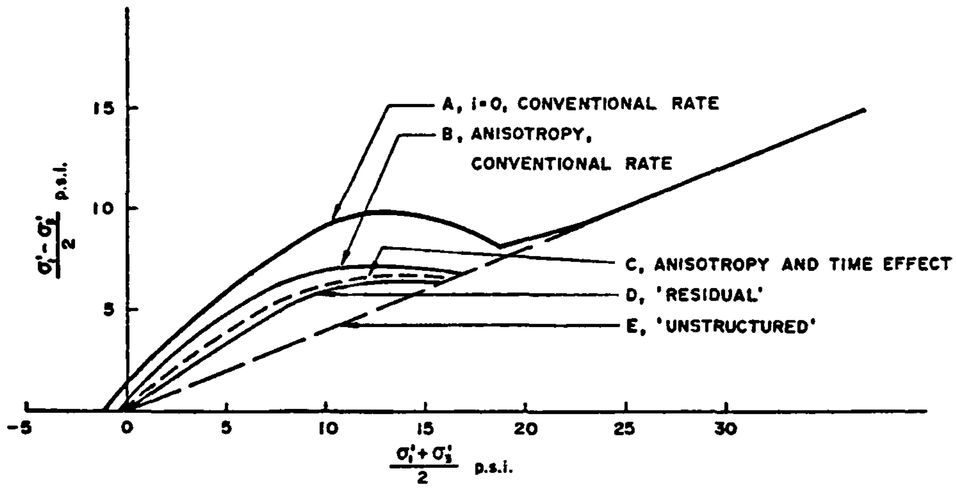


Figure 3.7.2 a Influence of anisotropy and time effects on the peak failure envelope of sensitive clays. (after Lo and Morin, 1972)

they indicate that a large zone of 'overstress' exists in natural slopes. Figure 3.7.2b illustrates these overstressed or local failure zones for two Leda clay slopes. Lo and Lee propose that in these large zones of 'overstress', the peak strength has been exceeded and only the residual strength remains to be mobilized at the time of failure. However, the validity of this concept is questionable. Leda clay is so brittle and sensitive (frequently in excess of 10), that once the peak strength is exceeded, especially over such a large area as that indicated by Lo and Lee (1973), dramatic failure of the slope should occur. Hence, it is not plausible that a slope would remain standing if such a large portion of it was in a residual strength condition.

It is postulated that due to the strength of their cementation bonds, Leda clays in slopes have maintained their brittle, sensitive, peak strength characteristics. At the time of incipient failure, a strength of a similar magnitude as the residual value is mobilized along the failure surface due to the effects of progressive failure, anisotropy and time. It is for this reason that residual strength parameters yield a good estimate of the stability of Leda clay slopes. Hence, the use of residual strength parameters is actually an empirical solution to the problem of slope stability analyses in Leda clay.

The important thing to realize, however, is that at the low effective stresses commonly determined for slope failures, both failure envelopes determined from stress controlled, drained, constant p'_m tests and the post-peak or residual failure envelope from conventional (σ_3 = constant) triaxial tests yield essentially the same effective shear strength parameters for microfissured clays (Mitchell, 1975). Since it

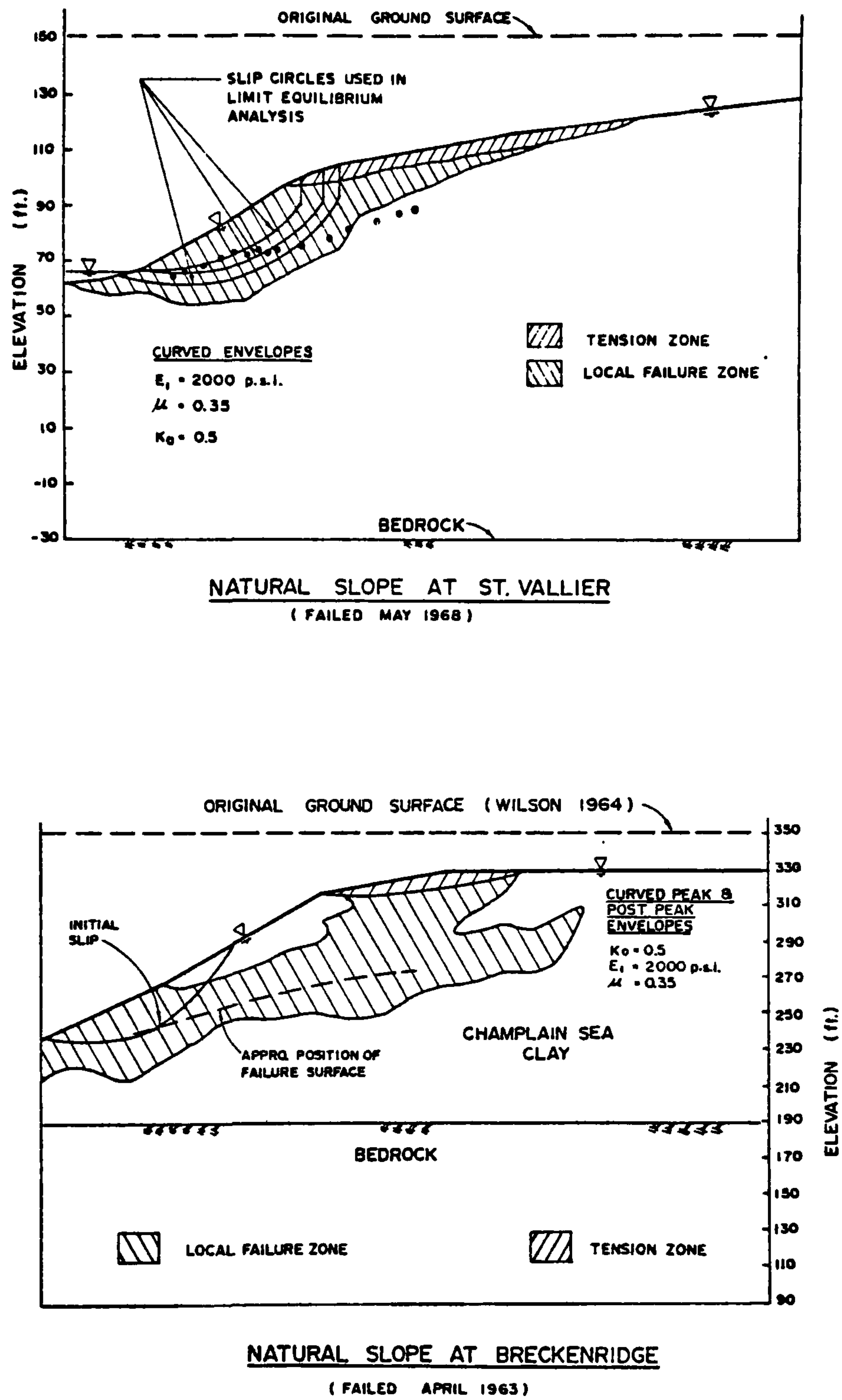


Figure 3.7.2 b Theoretical zones of 'overstress' or local failure in 2 natural slopes in Leda clay. (after Lo and Lee, 1973)

is evident from the results of stability analyses in the literature that these parameters yield reliable factors of safety, it is suggested that either the constant p'_m failure envelope or the conventional CID or CIU residual failure envelope may be used to estimate the factor of safety of a slope in a microfissured clay over the long term.

For intact clays, preliminary evidence indicates that stress controlled, drained, constant p'_m tests may yield the same failure envelope as the peak envelope from conventional (σ_3 = constant) tests. If peak strengths provide a gross overestimate of the factor of safety of an intact clay (Lefebvre and LaRochelle, 1973), then the use of the failure envelope from constant p'_m tests would also overestimate the factor of safety for intact clays. Hence, from the results reported in the literature, it would appear that only shear strengths similar in magnitude to post-peak or residual strengths may provide reliable factors of safety for intact clays.

One could then say that the use of the residual strength parameters defined by conventional (σ_3 = constant) tests would provide a satisfactory, although probably conservative estimate of the long term stability of either an intact or microfissured clay. It must be emphasized that such an approach is empirical and is based upon successful use as reported in the literature.

Lo and Morin (1972) have shown that the residual strength envelope is independent of: anisotropy, time effects, size effects, type of test and obviously progressive failure. Since the residual envelope is independent of the type of test, it is suggested that CIU tests would be adequate to define it. CIU tests are simple and rapid to perform and are easily achievable by most geotechnical consulting firms. Furthermore, it would appear that good tube samples would be adequate for testing purposes, although piston samples would be preferred. Block samples would not be needed since only residual strengths are required.

Since the use of residual strength parameters would probably result in slightly conservative results, high factors of safety would not be required for design.

3.8 Summary and Conclusions

This chapter has reviewed the basic soil properties, structural integrity, stress-strain behaviour and measured shear strengths of Leda clay. In addition, the effects of various factors such as sampling disturbance, sample size, time, anisotropy, stress path and progressive failure on the measured shear strengths were discussed. It was pointed out that much confusion exists today for the practicing geotechnical engineer concerned with slope stability analyses because of the conflicting results and opinions reported in the literature concerning just about every aspect of the shear strength of Leda clay.

The reason for the confusion in the literature was explained as a result of two interrelated factors: two structurally different clays have been tested, and different test procedures have been generally used for each of these clays. Stress controlled, drained constant p'_m triaxial tests have been generally used to study microfissured Leda clays while conventional strain controlled CIU or CID triaxial tests have been generally used to study intact Leda clays.

As a result of the different test procedures, one could use effective stress shear strength parameters of c' and ϕ' determined from the failure envelope of constant p'_m triaxial tests or from the peak or post-peak failure envelope from the conventional CIU or CID triaxial tests. A review of the reported results of stability analyses using the various shear strength envelopes has indicated that the use of the c' and ϕ' parameters, determined from the failure envelope of either constant p'_m triaxial tests or the residual strengths of conventional CIU or CID triaxial tests, yield satisfactory factors of safety in microfissured clays. However, only c' and ϕ' parameters similar in magnitude to those determined from the residual shear strength envelope, provide conservative factors of safety for intact clay slopes.

On the basis of conservative yet satisfactory estimates of factors of safety, it was suggested that residual strengths determined from conventional (σ_3 = constant) triaxial tests could be used as a preliminary estimate of the stability of slopes of either microfissured or intact clays. It must be noted however, that the justification for this approach is largely empirical and that the use of residual strengths is not conceptually correct.

CHAPTER 4

GENERAL PLEISTOCENE GEOLOGY OF THE OTTAWA RIVER VALLEY REGION

4.0 Introduction

The Ottawa River Valley region as it appears today has undergone a complex geological history that has involved glaciation, marine submergence, fluvial erosion and mass wasting. These geological processes have resulted in the deposition of a variety of sediments, each with distinct characteristics. The deposited clay in particular, is not unique throughout the region but exhibits considerable variability in geotechnical properties. In an effort to fully appreciate the heterogeneity of the deposits, the geological history of the region is briefly discussed below. Each class of sediment is discussed in detail, once the overall geologic picture is presented.

4.1 General

Recent evidence by Gadd (1975,1977) advocates the single advance of a Laurentide ice sheet of Wisconsin age throughout the region and the deposition of a single, thin (generally less than 3 m), discontinuous layer of till and some ice contact, sand and gravel over the bedrock. An ice dam in the vicinity of Quebec City allowed the formation of a glacial meltwater lake, following the retreat of the ice sheet from the Ottawa Valley.

A thin (2-8 metres) stratum of light and dark grey banded clay and silty clay varves was deposited over the till. By assuming each varve couple represents one year of deposition, Fransham (1978) estimates that in the vicinity of the Ottawa Valley, this glacial freshwater lake would have existed for about 600 years.

At approximately 12,800 years BP (before present) the ice dam near Quebec City was breached (Gadd, 1977) and the region was inundated with saline water. The mixture of saline and fresh water resulted in what is termed "brackish" water. This early stage of the "Champlain Sea" was about 150 metres deep (Fransham, 1978). Figure 4.1.1 illustrates the extent of the Champlain Sea throughout Eastern Canada.

Streams and rivers brought silt and clay size particles into this sea. The environment of deposition in the sea was quiescent due to the depth of water, allowing the very fine size fraction to settle out. For the next 2600 years [12,800 years; the date of marine submergence minus 10,200 years; the age of freshwater clams discovered in the Bear Brook locality (Gadd, 1975)], deep water marine conditions continued and enabled the deposition of a thick (3 to 30 metres) stratum of marine clay. During this interval, as the ice slowly retreated, the land rebounded isostatically relieved of the great weight of the ice. Sediments that were deposited near the sea shore during the quiescent deep water conditions were transported by streams towards the center of the basin. Thus a higher energy, deltaic type of environment gradually replaced the deep water environment of deposition.

The deltaic sediments consisted of sand size particles being deposited as topset beds and silt and clay size particles being

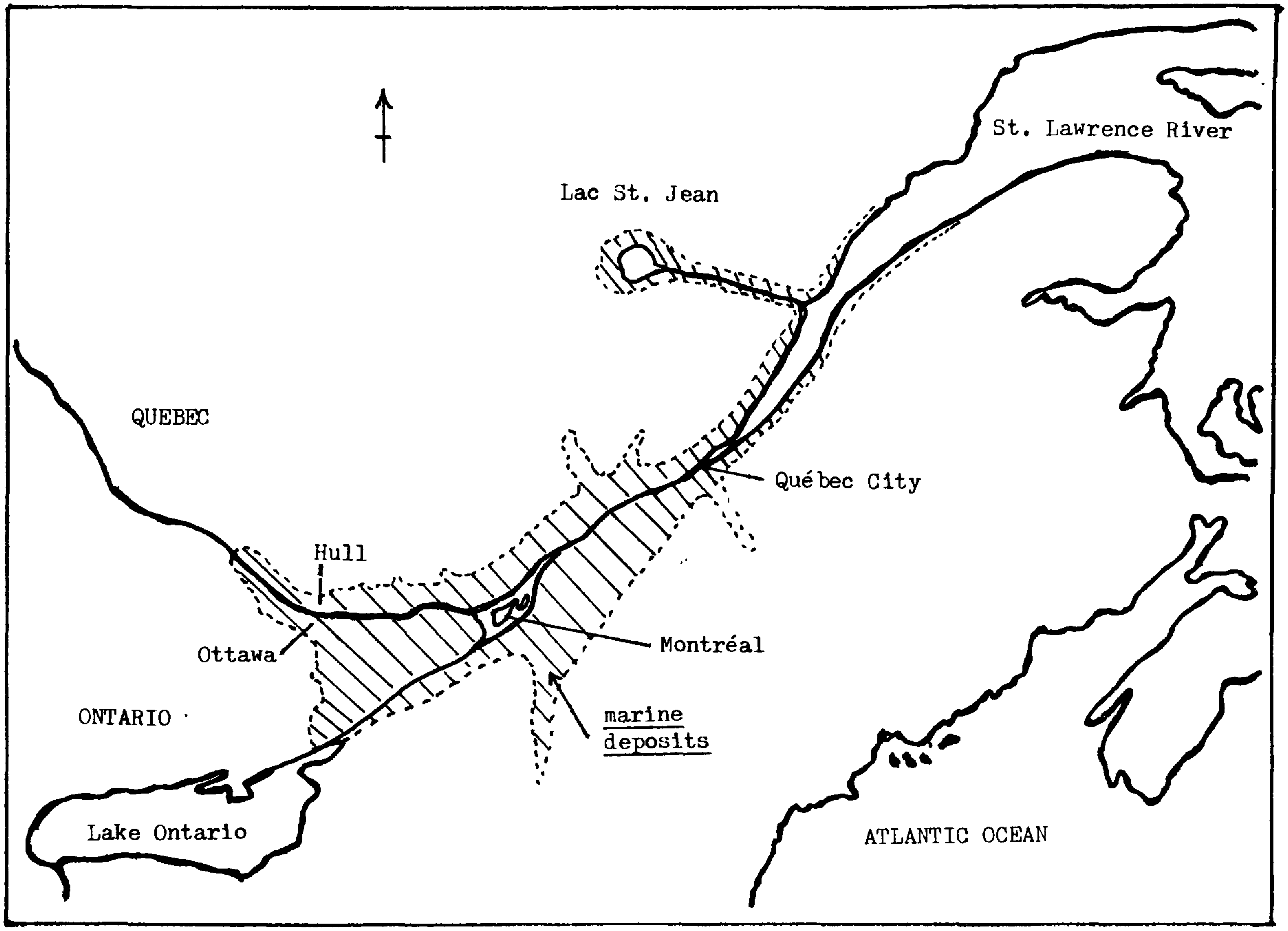


FIGURE 4.1.1 EXTENT OF THE CHAMPLAIN SEA THROUGHOUT EASTERN CANADA

deposited primarily as bottom set beds of a prograding delta. In this case, it is difficult to separately distinguish foreset beds since the slope angles are very small and the material being deposited is quite fine.

As the depth of the water further shallowed, a higher level energy environment occurred resulting in the cyclic deposition of layers of slightly more coarser clayey silt and fine sand. This environment of deposition is termed "shoaling prograding delta" by Fransham and Gadd (1977).

As the land continued to rise in elevation, the ancestral Ottawa River had increasing influence on the deposition and erosion of sediments. At about this time, large bodies of water such as the Great Lakes were draining into the ancestral Ottawa River which was then 5.6 km wide at a point just northwest of Ottawa, compared with 1.6 km today (Fransham, 1978). As this wide river was discharging into the regressing sea, a major deltaic and braided river channel stratum of uniform fine sand was deposited over the marine clay throughout most of the valley. This stratum is about 2-4 metres in thickness (Fransham and Gadd, 1977).

Approximately 10,000 years BP is generally accepted as marking the end of the marine era throughout most of the valley (Gadd, 1975). At this time the river gradient increased and freshwater fluvial drainage conditions predominated. Several principal river channels were cut along fault controlled lines at locations illustrated in Figure 4.1.2. The fault system provided zones of weakness along which glacial erosion scoured out depressions, only partly filled them with glacial debris

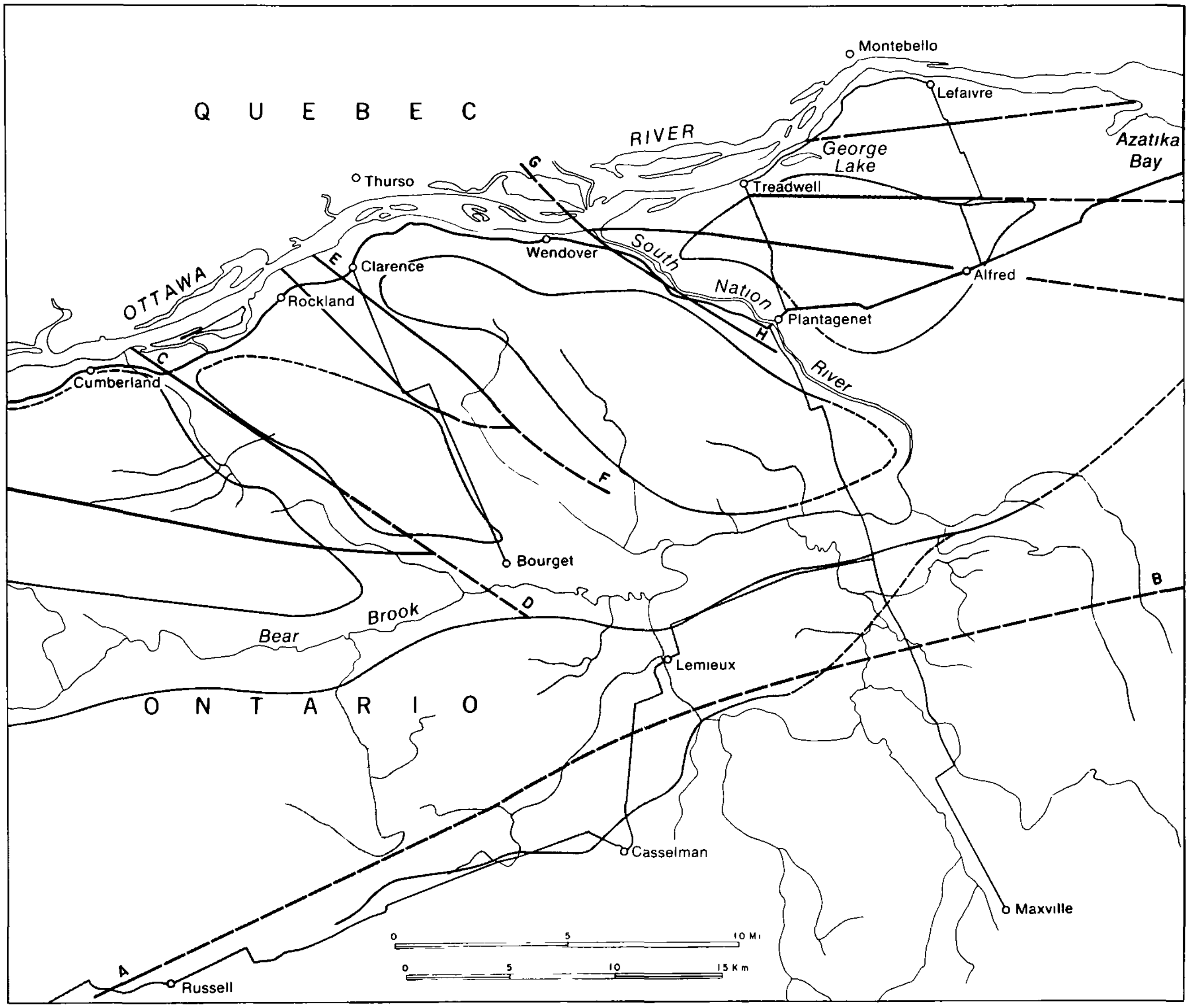


Figure 4.1.2 Location of faults (heavy black lines, some lettered AB, CD, EF, GH) in relation to abandoned channels (white) between 'islands' or erosional remnants of sand-capped marine clay (stippled). (After Gadd,1976.)

and allowed for the localized deposition of thick deposits of marine clay. Gadd (1976) postulates that the surface of the marine clay retained the shape of the fault controlled depressions and concludes that the fault system was the fundamental factor determining the pattern of postmarine surface drainage patterns. Thus, streams and rivers were able to easily incise channels as deep as 30 m (Gadd, 1976) because of the accumulation of fine, soft, deltaic sand and marine clay sediments in these depressions. This regional channelling left "islands" of fine deltaic sand overlying saturated sensitive clay. Because of the high susceptibility of this material to flowslides, the new fluvial channels widened very rapidly.

Smaller and younger tributary streams, which now define modern river courses such as the South Nation River, cut narrower channels since they had a smaller initial gradient and relatively coarse material in their bed. A narrow channel is a limiting factor controlling the size of flowslides, since flowslides are restricted to the capacity of the valley into which they flow. In contrast, the very wide, presently abandoned, river channels permitted the occurrence of very large flowslides, as evidenced by numerous landslide scars that have been mapped along these abandoned river channels (Gadd, 1976; Fransham et al, 1976). Gadd (1976) estimates that out of total of approximately 11,160 acres of landslide terrain in the Thurso-Russel map area, 11,160 acres (96%) are associated with the fault-controlled abandoned river channel system, whereas only about 480 acres (4%) are associated with modern streams.

According to Gadd (1962,1976), as the ancestral Ottawa River continued to meander, some of these channels were subsequently filled with clay that had been eroded upstream and redeposited under freshwater conditions, thus yielding what could be termed as another facies of Leda clay. However, as will be pointed out later, the actual occurrence of this freshwater redeposited material is quite rare and it merits no special discussion here. It will however, be briefly discussed in the next section. Finally, as the discharge of the Ottawa River decreased, these channels were abandoned in favour of the present day location of the river.

4.2 Individual Description of Each Facies of Leda Clay

Figure 4.2.1 is a stratigraphic column illustrating the sequence of sediment deposition in the Ottawa valley region. Of particular interest to this thesis is the group of deposits commonly identified as "Leda Clay" which consists of the deep water marine clay facies, the prograding delta clay facies and the shoaling prograding delta clay and silt facies. The underlying varved clay is not labelled as Leda Clay in this thesis since it was deposited in a freshwater glacial lake environment prior to inundation by the sea. Furthermore, surficial deposits of this material are rare and it is not involved in landslides.

The Leda clay deposits are discussed in more detail below. A brief description of the other deposits in the sedimentary suite is given on Figure 4.2.1 and is thought to be sufficient. For more details concerning these deposits the reader is referred to Gadd (1962, 1975, 1976, 1977); Fransham and Gadd (1977); Fransham (1978) and Karrow (1961).

Thickness Range m	Stratigraphic Unit	Description
Variable	BOG	Peat, Muck, Organics
2-4	Fluvial-Deltaic Sand	Uniform fine sand, poorly drained.
< 15	Shoaling Prograding Delta Clay and Silt	Commonly identified as 'Leda' clay. Discussed in detail in text.
13-50	Prograding Delta Clay	
3-30	Deep Water Marine Clay	
2-8	Varved clay	Freshwater glacial lake, banded deposit of thin (1.5 cm) dark grey clay and light grey silty clay layers. Surficial exposures of this material are rare.
0-3	Glacial till and some ice contact sand and gravel.	Till is in the range of sandy to silty with boulders. Sand and gravel are poorly sorted.
	Bedrock	Precambrian crystalline and gneissic rocks to the north. Folded Paleozoic meta-sedimentary rocks to the south, that are intruded by by younger volcanic rocks.

Figure 4.2.1 Stratigraphic column (Modified after Fransham and Gadd, 1977)

The term "Leda Clay" is theoretically a misnomer. This name was first applied to the clay when the most common fossil was erroneously determined to be "Leda glacialis". The fossil has since been correctly identified as "Portlandia artica" as stated in Gadd (1975). LaRochelle et al (1970) suggest the use of the term "Champlain Sea Clay". However, Gadd (1975) recommends the retention of the term "Leda Clay" for the following two reasons: "Leda Clay" has become widely known particularly among engineers, and it has wider scope than the alternative "Champlain Sea Clay".

Table 4.2.1 is a summary of the 'average' index properties for the varved clay and each of the Leda clay facies. These 'average' results are based upon limited data and do not reflect the variability associated with each facies. The reader should thus consider them as tentative until more data is available.

The clays consist primarily of rock flour eroded from the underlying bedrock by glaciers. They are low in expansive clay mineral content with a predominance of illite, chlorite and other micaceous minerals. They have an open "card house" type of structure due to the flocculation of the clay particles by the electrolytic effect of salt water during deposition. The following is a more detailed description of each of the Leda clay facies:

4.2.1 Deep water marine clay facies

The transition from freshwater varved clay to deep water marine clay occurs rather abruptly over a length of about 1 metre of sediments and reflects the rapid submergence of the area by the

Unit	Thickness (m)	Specific Gravity	Bulk Density kg/m^3	Natural Water Content %	Liquid Limit %	Plastic Limit %	Clay Size %	Silt Size %	Sand Size %
Varved clay facies	2-8	2.79	1920	36	33	18	32	45	23
Deep-water marine clay facies	3-30	2.79	1730	50	47	24	62	37	1
Prodelta facies	13-50	2.79	1720	55	53	24	70	30	0
Shoaling prograding delta-silty material	<15	2.79	1760				14	82	4
Shoaling prograding delta-clayey material	<15	2.80	1830	48	44	25	50	50	0

TABLE 4.2.1 Average Index Properties of Sensitive Marine Clay of the Champlain Sea. [After Fransham, P.B. and Gadd, N.R. (1977)].

Champlain Sea. The deep water marine clay is dark grey and massive and is frequently mottled with black material believed to be disseminated organic matter and sulphides. It is the most commonly occurring and therefore the best known facies of Leda clay. This facies frequently contains marine fossils.

4.2.2 Prograding delta facies

The transition from the deep water marine clay facies to the prograding delta clay facies is not abrupt and can range over several metres. It reflects the subtle change in energy levels from quiescent deep water conditions to slightly more turbulent and fluctuating conditions representing seasonal variations in current and material supply (Fransham and Gadd, 1977). The seasonal changes are displayed by a banded stratigraphy. The clay is either red and grey banded or light and dark grey banded. The red and dark grey layers contain more clay size material. Black organic mottling and silt seams occasionally occur.

The origin of the red colour has not yet been adequately explained, although some theories concerning it do exist. Fransham (1978) advances the hypothesis that the red colour results from the predominance of red potash feldspar eroded from the pink gneissic Precambrian rocks north of the Ottawa valley. These feldspars, being quite susceptible to mechanical weathering, would form a large portion of the fine fraction of suspended sediment and settle out during the quiescent winter freeze up conditions and form the red bands. However, no mineralogical evidence comparing the red and grey bands exists to reinforce this hypothesis. In fact, little mineralogical work has been published on this particular facies of Leda clay.

This facies is of particular interest since most of the extensive landslides in the area appear to involve this material (Fransham and Gadd, 1977).

4.2.3 Shoaling prograding delta silt and clay facies

According to Fransham and Gadd (1977) this is the last and uppermost of the Leda clay facies and reflects a higher energy level that existed when the marine environment was very shallow and was changing into a freshwater fluvial system. This is also a red and grey banded material and would appear at first to be quite similar to the prograding delta facies. The main difference lies in the increased coarser size fraction in both red and grey bands. The grey layers vary from clayey to sandy silt. This facies may be further distinguished from the prograding delta facies by the presence of distorted beds caused by slumping, channelling and redeposition of material.

According to Gadd (1967, 1976) one might believe that another distinctive facies occurs abundantly throughout the Ottawa Valley Region, specifically in the abandoned river channels of the ancestral Ottawa River. Gadd (1976) calls this material a freshwater fluvial silt and clay. It consists of eroded marine clay that has been redeposited in the abandoned river channels under freshwater conditions. He describes this material as a fine facies of river sediment characterized by laminated or at least colour banded silt and/or clay. The bands are composed of reddish brown silty clay and grey coarse silt that vary in thickness from 0.5 cm to 10-15 cm.

Fransham (personal communication) has pointed out that the occurence of a truly freshwater redeposited clay in the Ottawa Valley is in fact quite rare and would be limited to slack water fluvial deposition sites such as terraces, channel bars and spits in the abandoned river channels. Certainly, the occurrence of this material is not as abundant as Gadd (1962,1976) has indicated. Most abandoned river channels would have one or more of the previously described facies of Leda clay as the dominant material.

4.3 Fissuring

Eden and Jarrett (1971), Crawford and Eden (1967) and Scott et al (1976) have indicated that both micro and macrofissures frequently occur in a stratified (red and grey or light grey and dark grey) clay with silt layers. This description matches that of the prograding and shoaling prograding delta clay and silt facies. Sangrey and Paul (1972) indicate that macrofissuring takes place at an average spacing of 15-20 cm in a clay matching the description of the deep-water marine facies.

Fissures do not necessarily occur in all exposures of these facies throughout the Ottawa River Valley region. Intact clay deposits are known to occur throughout the region not only below the fissured clay but also in surface exposures at high elevations. Thus, it would appear that fissuring is not the direct result of the characteristics of a particular clay facies or the geologic conditions at the time of deposition. Rather, it would appear that fissuring is the result of a special set of conditions occurring in only some regions, at some time after the deposition of the suite of sediments.

As explained in Chapter 3, the exact causes of fissuring are unknown. However, if fissuring is related to weathering or stress relief due to removal of overburden or slope cutting, one might expect the overlying prograding delta clay facies to be more fissured than the underlying deep-water marine clay facies.

4.4 Conclusion

The Ottawa River Valley region has undergone a complex series of geological events. A wide variety of sediments have thus been deposited. Of particular interest to geotechnical engineers in the study of slope stability is the suite of sediments known as Leda clay. The geological origin and the basic geotechnical properties of the three facies of Leda clay have been discussed. The remaining chapters of this thesis describe in more detail the engineering behaviour of Leda clay with particular reference to slope stability.

CHAPTER 5

SITE DESCRIPTIONS

5.0 Introduction

High quality samples are required to obtain accurate and reliable laboratory measurements of the shear strength of sensitive cemented Leda clays. As discussed in Chapter 3, thin walled tube samples strain the soil and cause a breakdown of "cementation bonds" in intact clays. This results in a dramatic reduction of both total and effective shear strength parameters. Eden (1971a) states that large, undisturbed block samples yield the most reliable results for sensitive clays. It was therefore decided to attempt to obtain high quality block samples for use in this thesis research.

This chapter describes the location, recent geology and soil stratigraphy of five sites throughout the Ottawa Valley Region from which block samples were taken.

5.1 General

The following criteria were used in the initial selection of sites to be sampled:

a) It was desired to obtain samples of widely varying shear strength in order to be able to draw conclusions concerning their undrained and drained shear strength behaviour that would be applicable to most Ottawa Valley region clays.

b) It was desired to obtain samples from each of the previously discussed facies of Leda clay in order to perform a detailed analysis of the engineering characteristics of each.

c) As discussed in Chapter 3, Lo and Lee (1973) have proposed, on the basis of the results of a finite element program, that the geological process of slope formation would result in the development of a significant zone of 'overstress' in a slope. (i.e., a zone in which the peak strength has been exceeded and the only available strength would correspond to the post-peak or residual strength). If this analysis is correct, one would expect that samples obtained from this apparently overstressed zone would not exhibit a brittle type of behaviour with a characteristic peak strength, but rather only an elastic-plastic stress-strain behaviour with the maximum shear strength attained corresponding to the residual value.

It was desired to investigate the legitimacy of such a proposal. Therefore, an attempt was made to obtain samples from long standing natural slopes in areas of known instability.

Once these initial criteria were fulfilled, the following constraints also had to be satisfied prior to block sampling.

a) The site had to be closely accessible by vehicle to permit easy unloading and loading of equipment and block samples.

b) In order to investigate the stress-strain behaviour of the clay in a slope and determine the legitimacy of Lo and Lee's (1973) proposal that only residual strengths exist in the apparent overstressed zone, it was decided to take block samples near the bottom, or toe of the slope. This area is theoretically the most overstressed zone and should not exhibit any peak strength according to Lo and Lee (1973a). To obtain good block samples, the clay in this area had to be undisturbed, not slide debris, undesiccated and free of extensive fissuring.

c) The clay to be sampled had to be within a certain depth of the surface to enable exposure by hand excavation with shovels.

Due to the rigorous selection criteria and constraints, it was quite difficult to find sites for sampling. After discussion with several engineers and geologists working in the region, and several investigative field trips, the five sites, numbered from east to west, illustrated on Figure 5.1 were sampled. A complete description of each site follows.

5.2 Site No. 1, St. Bernadin

St. Bernadin is a small village located approximately 70 km east of Ottawa. The site for sampling was at the location of a small landslide on Caledonia Creek that took place in the spring of 1976 behind the church cemetry. The cause of the landslide is believed to have been erosion at the toe, steepening the slope beyond the critical height.

Figure 5.2.1 is a simplified composite of a series of seventeen maps compiled by Fransham et al (1976). The figure is a good illustration of the surficial geology of the Ottawa River Valley region that was previously discussed in Chapter 4. By looking at the figure, one can see that the St. Bernadin site is located on the eastern edge of one of the 'sand islands' described by Gadd (1976). The immediate area is covered

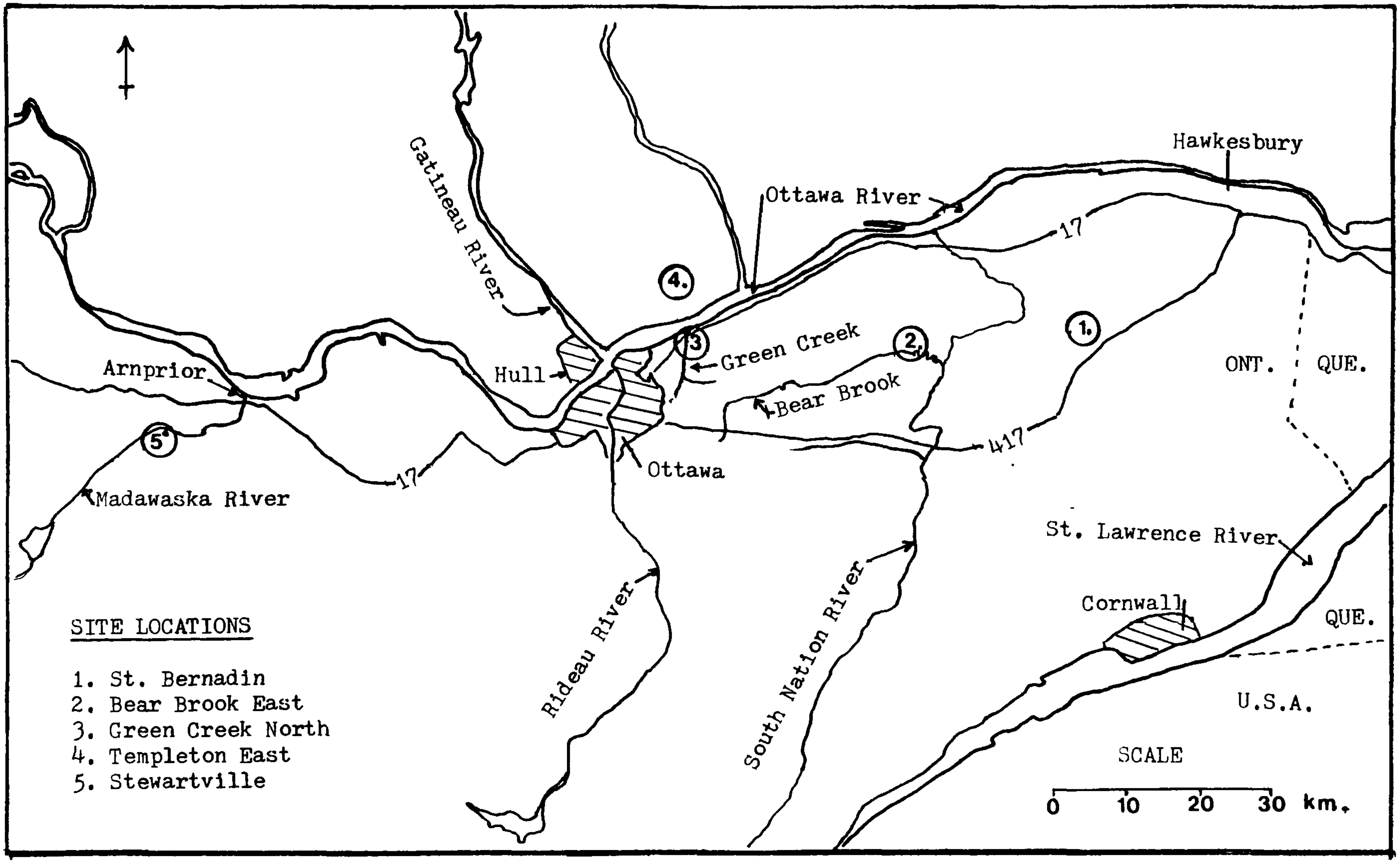


Figure 5.1 LOCATION OF SITES

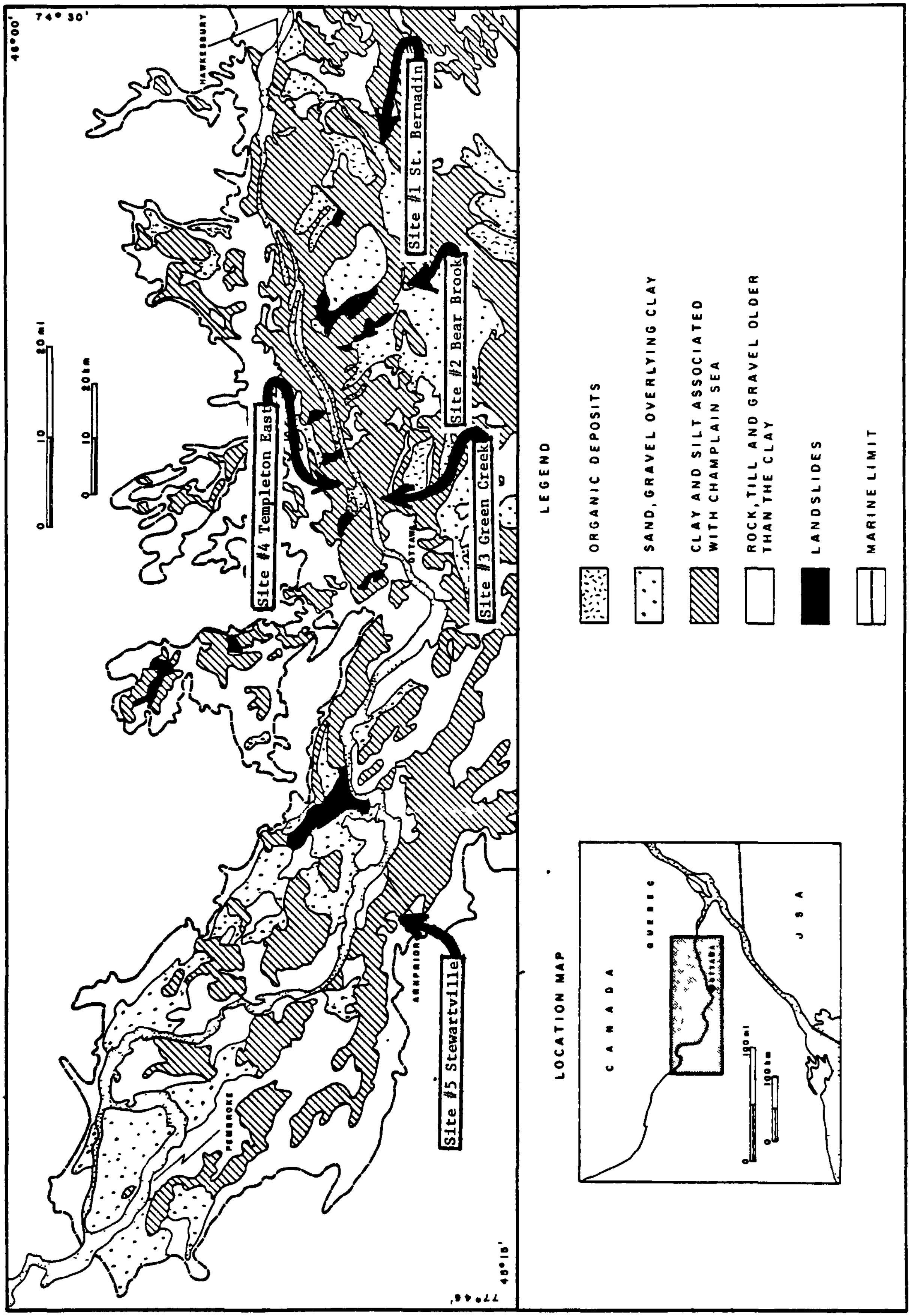


Figure 5.2.1 Pleistocene geology of the Ottawa River Valley Region. (From Fransham and Gadd, 1977)

with a layer of fluvial deltaic, fine sand with gravel whose exact thickness at the site is unknown but is in the vicinity of 2 metres. This sand was eroded to the south and east by the ancestral Ottawa River as it meandered and cut channels through the existing sedimentary suite, as described earlier in Chapter 4.

Underlying the sand, clay has been deposited in horizontal layers that according to visual and tactile examination consists of a repeating cycle of reddish-brown clay, a thin fine sand seam and then grey silty clay. In general, the red layers are about 3 cm thick, the sand seam about 0.1 to 0.5 cm, and the grey silty clay about 5 cm thick. The thickness of the individual layers varies frequently indicating a seasonal variation in current and volume of material available for deposition. The cycle thickness increases from about 8 cm to about 23 cm at the depth at which the block samples were taken. It is hypothesized that the red clay would have been deposited in quiescent winter conditions, the sand seam would be a fluvial deposit during the summer when the conditions would be more turbulent, and the grey silty clay would be a late autumn, early winter deposit when conditions became stagnant just prior to winter freeze-up. Occasional black mottling and disintegrating organic roots were observed in the grey silty clay layer. Occasional fissures were noticed parallel to the slope above the level at which block samples were taken. Based on the above visual identification information, the clay is classified as part of the prograding delta clay facies discussed in Chapter 4.

The photographs on Plate 5.1 show the excavation and the exposed soil stratigraphy and the location from which the block samples were taken in the backscarp of the landslide at St. Bernadin.

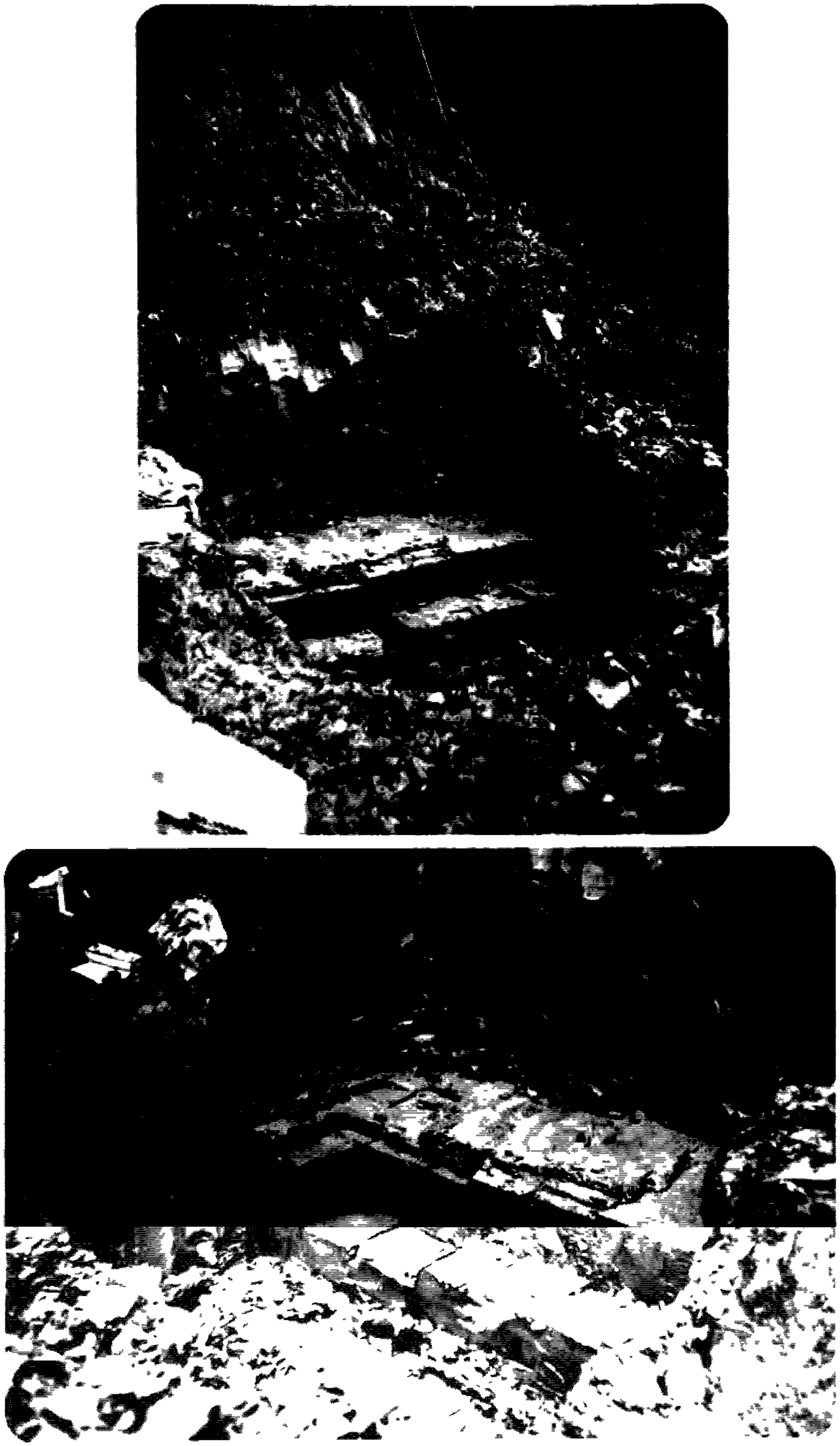


PLATE 5.1 Excavation, location of block samples, and exposed soil stratigraphy.
Site No. 1, St. Bernardin, Ont.

Figure 5.2.2 shows a cross-section through the slope, the location and elevation of the block samples and the terrace. It also shows the shear strength and water content profile with depth.

A summary table and discussion of the index properties for each site is presented in Chapter 7.

5.3 Site No. 2, Bear Brook East

Site No. 2 is located near the eastern end of Bear Brook close to Ettyville approximately 50 km east of Ottawa and about 4 km east-southeast of the town of Bourget. Bear Brook flows to the east about 3 km and then drains into the South Nation River, along which many large landslides have occurred.

The site is located in a small bay of an abandoned river channel of the ancestral Ottawa River (see Figure 5.2.1). This channel is now occupied by Cobbs Lake Creek and part of the South Nation River. The overlying fluvial-deltaic sand has been eroded and Leda clay lies exposed at the surface.

The Leda clay at the site consists of a repeating cycle of the following three materials:

1) a reddish brown silty clay layer about 8-13 cm thick,
2) a grey silty clay layer about 2-5 cm thick,
3) a grey clayey silt layer with sand about 8-13 cm thick.

The repeating cycle of layers at this site is different from that at St. Bernadin due to the presence of the thick layer of grey clayey silt. The trend of increasing grain size upwards in the annual

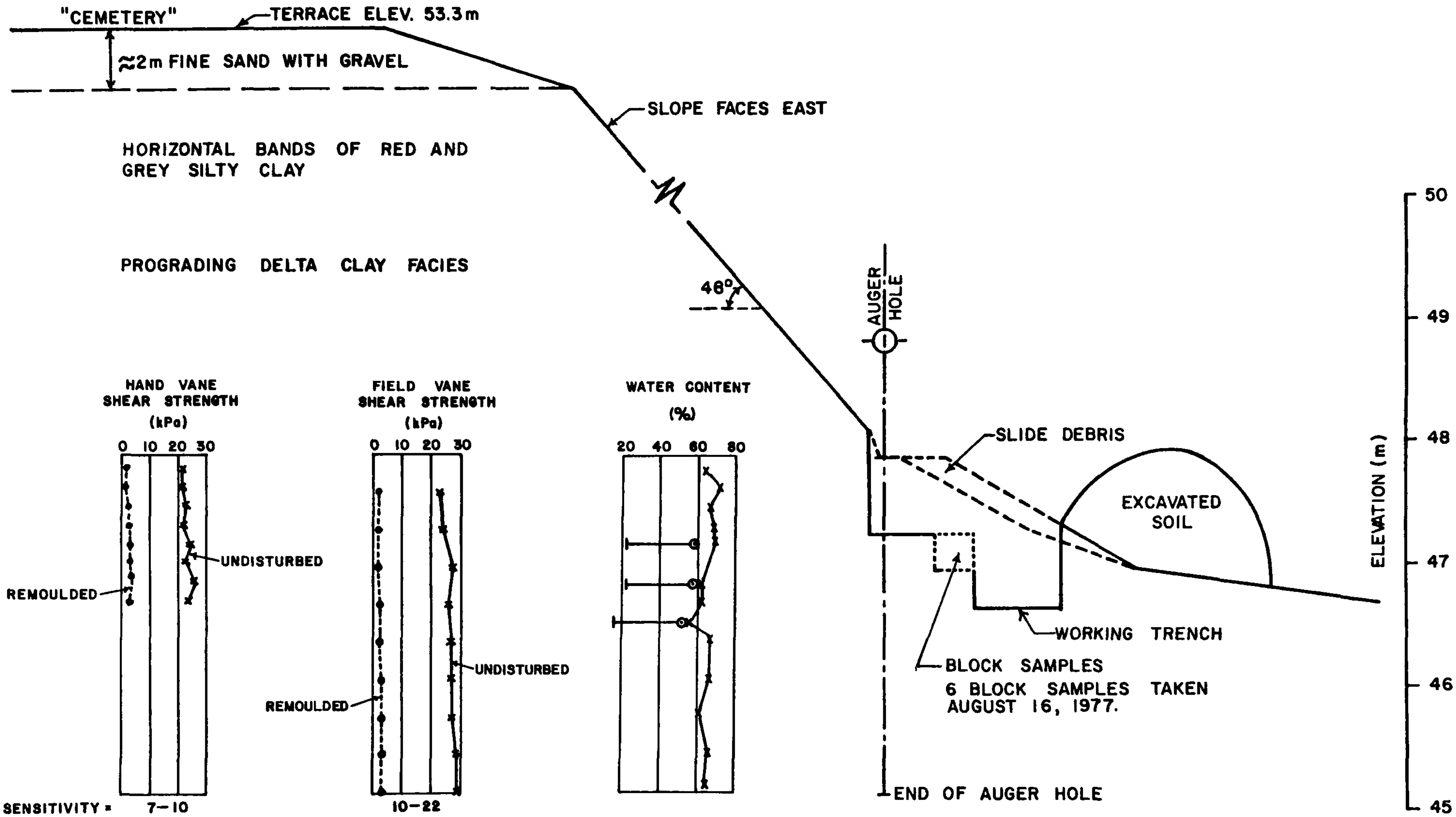


Figure 5.2.2 X-Section through slope.
Site No. 1, St. Bernadin

cycle (i.e., from red silty clay to grey clayey silt), indicates increasing energy in the environment of deposition as the seasons progress. It is postulated that the reddish brown clay would have been deposited during quiescent winter conditions, then spring flood conditions might create a short term deep water phase depositing the grey silty clay. As the water level would drop during the summer, there would be an increase in the energy level thereby depositing the coarser clayey silt with sand. The presence of this coarse silt layer indicates a different type of environment of deposition, one similar to that of a shallow water basin which is easily affected by seasonal variation in water level.

The stratigraphy at the site is actually quite complex and there is evidence of past slumping or faulting. Figure 5.3.1 is a sketch of the stratification after exposure by hand excavation. A distinct fault or slump can be seen by the differential movement between the reddish clay layers. Slumping occurs quite frequently in deltaic deposits since the material is deposited under water in a low state of relative density and the post depositional environment is often turbulent.

Photo No. 1 on Plate 5.2.1 is a close-up view of the bench excavated into the face of the slope at Bear Brook East just prior to taking block samples. Note the three distinct bands in each cycle. Photo No. 2 is a close-up of the back face of the excavation clearly showing the stratigraphy. Note the water percolating through the clayey silt layer. The photograph on Plate 5.2.2 shows an oblique view of block sample No.1 illustrating the distortion of the layers.

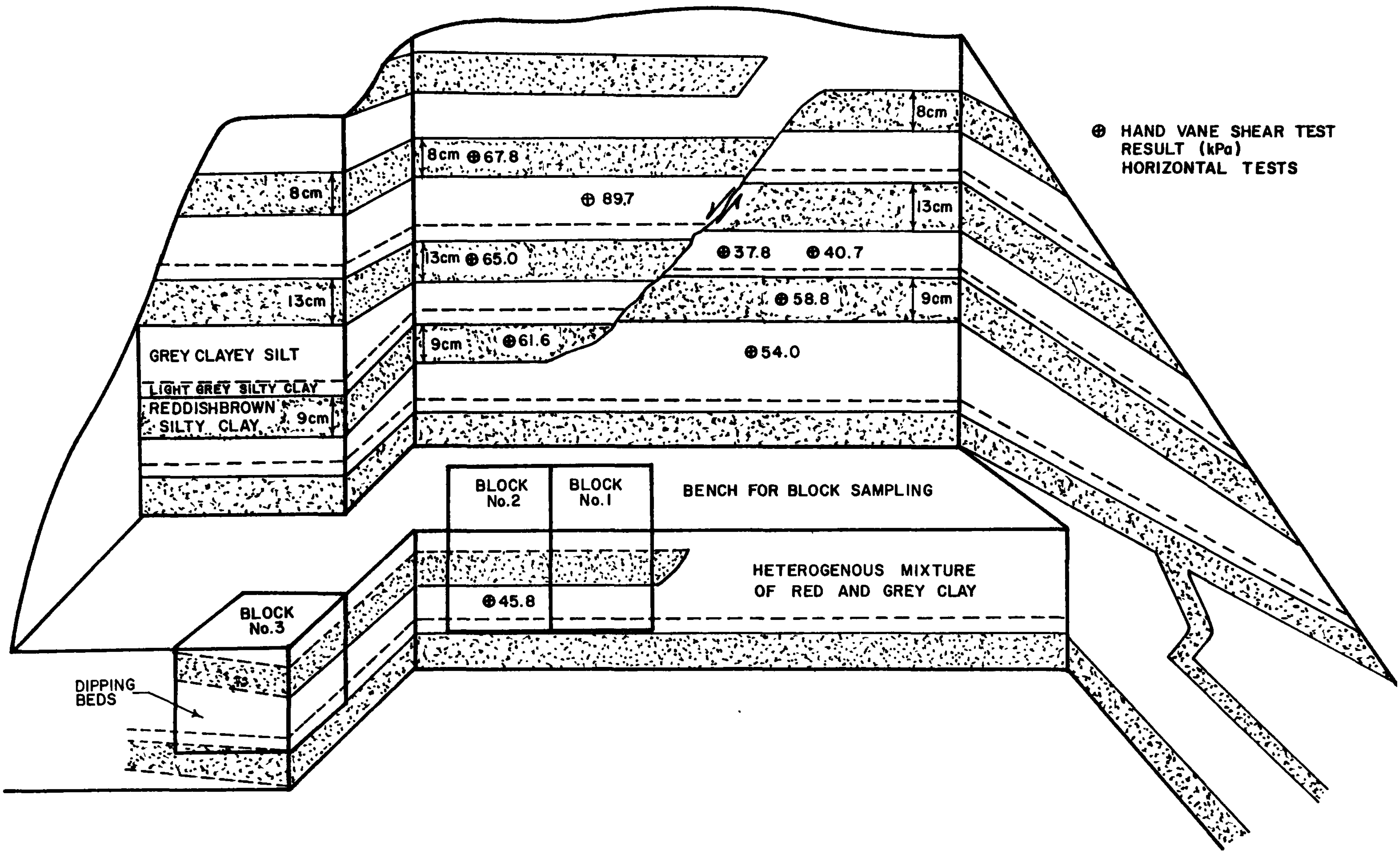


Figure 5.3.1 Sketch illustrating complex stratigraphy and location of block samples at Site No. 2, Bear Brook East.

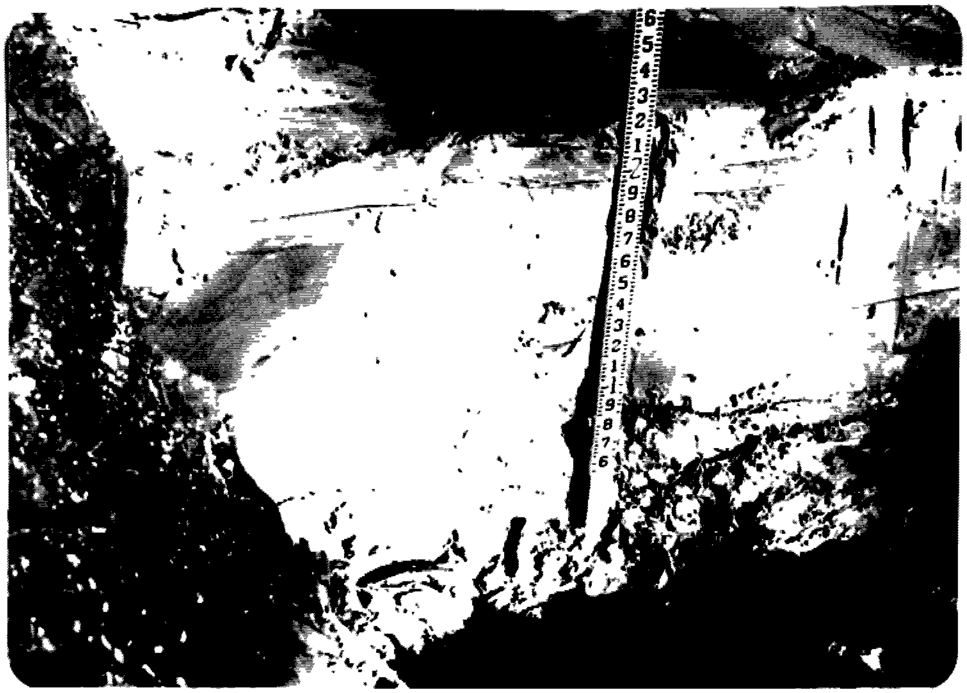


PHOTO NO. 1. Bench prior to taking block samples.

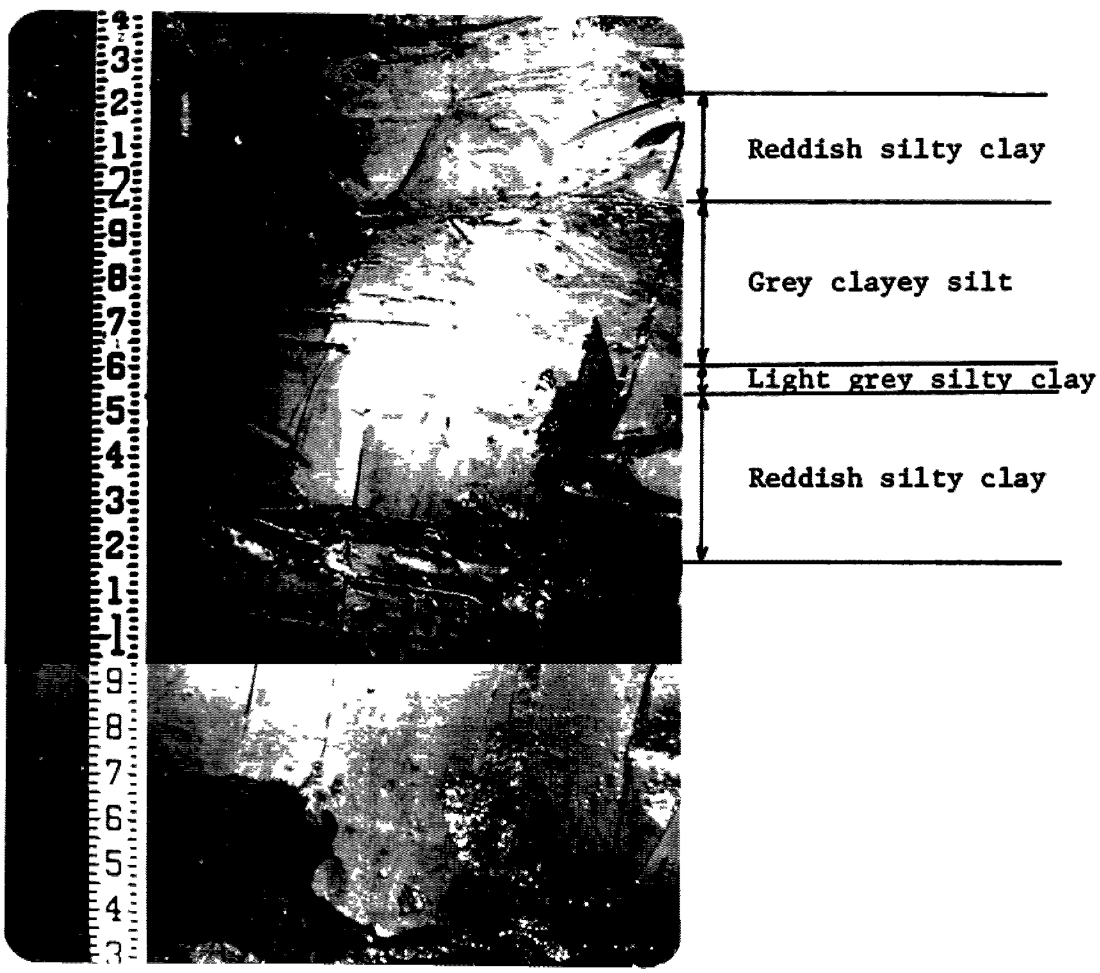


PHOTO NO. 2. Close up of back face of the excavation.

PLATE 5.2.1 Site No. 2, Bear Brook East.

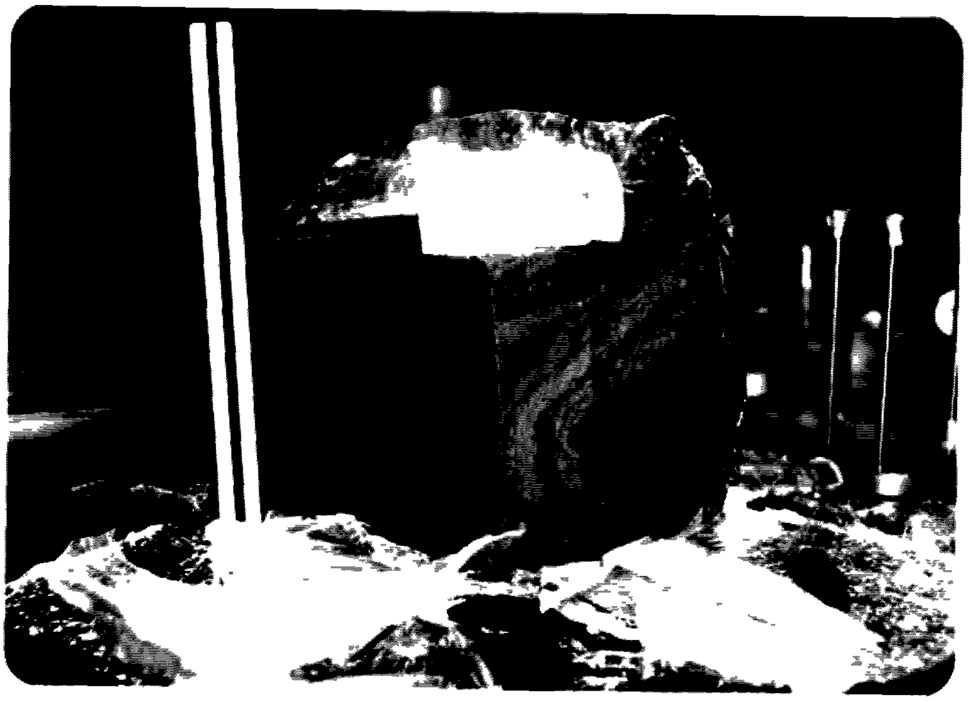


PLATE 5.2.2 Oblique view of Block Sample No. 1, Site No. 2, Bear Brook East. Note the distorted layers.

The photographs and sketch have shown the complex geology associated with this site which reflects the environment during and after deposition. Based on the visual and geologic evidence available, this site is classified as part of the shoaling prograding delta facies.

Figure 5.3.2 shows a cross-section through the slope, showing the location and elevation of the block samples, the water level in the brook and the clay terrace. It also shows the vane shear strength and water content profile with depth.

5.4 Site No. 3, Green Creek North

Site No. 3 is located on the north side of Highway No. 17 and about 300 metres east of Green Creek which is about 9 km east of downtown Ottawa. This site lies in what has been classified by Richard, Gadd and Vincent (1978) to be an abandoned channel of the ancestral Ottawa River. They describe the material as being a post-Champlain Sea deposit of silt and silty clay that is generally underlain at variable depth by the deep water marine facies of Leda clay.

It was decided to take samples from this site when the problem of excavation was solved due to the presence of a large power shovel provided by Beaver Construction Ontario Ltd..

Many landslides, both ancient and recent have taken place along Green Creek. In addition, a substantial amount of geotechnical investigations and analyses have been carried out by both the National Research Council and private consultants in the immediate area. This permits a comparison to be made between any results obtained from this research program with existing information.

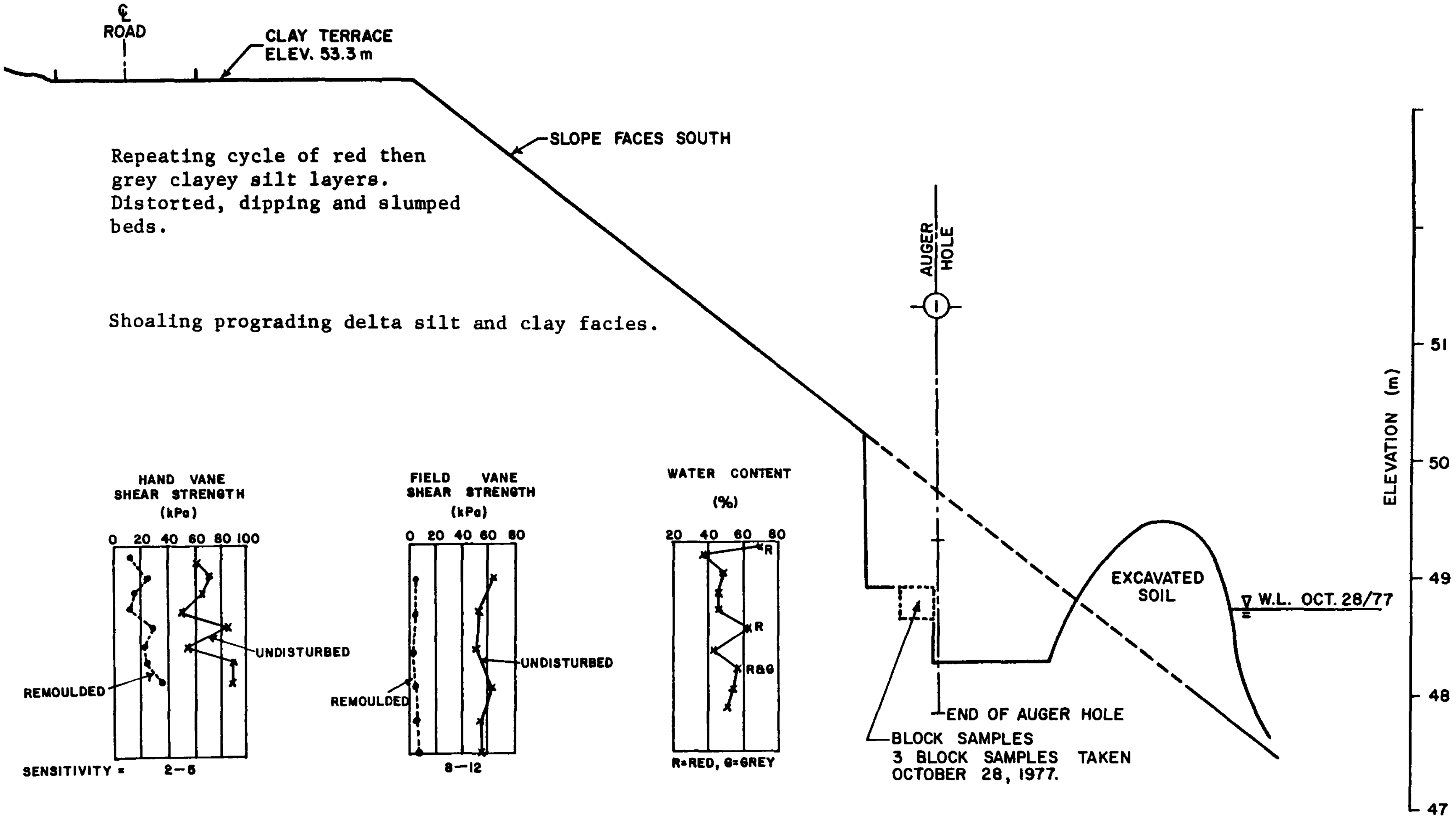


Figure 5.3.2 X-Section through slope.
Site No. 2, Bear Brook East.

An excavation was made by power shovel to a depth of about 4.5 metres (see the photograph on Plate 5.3.1). The clay is brownish grey and fissured forming a stiff crust to a depth of about 3.5 metres. Below this depth the clay is unoxidized, dark grey, but still contains vertical fissures at a spacing of about 15 to 30 cm.

An attempt was made to obtain block samples but the frequency of the fissures would not allow the cutting of block samples of specific dimensions and shape. The photograph on Plate 5.3.2 illustrates the extent of the fissuring in the excavation. Large pinnacles of clay would frequently collapse into the excavation. It was feared that the unsupported excavation would collapse and bury those trying to obtain the samples.

It was noticed that the large pinnacles of clay were relatively intact. It was decided to take chunk samples cut from these clay masses. This was the only way of obtaining samples from this site. Five such chunk samples were taken and the average size was only slightly smaller than the usual 25 cm cube blocks.

Prior to the refilling of the excavation a closer observation of the soil stratigraphy was made. This indicated that parts of the upper 3.5 metres were red and grey banded and that the last metre from 3.5 to 4.5 metres deep appeared to be generally grey and massive with the occasional indication of slight black mottling. Based on the visual and geologic information available, it would appear that from 0 to 3.5 metres depth, the clay is of the prograding delta clay facies. Below 3.5 metres, the clay appears to be of the deep water marine clay facies.

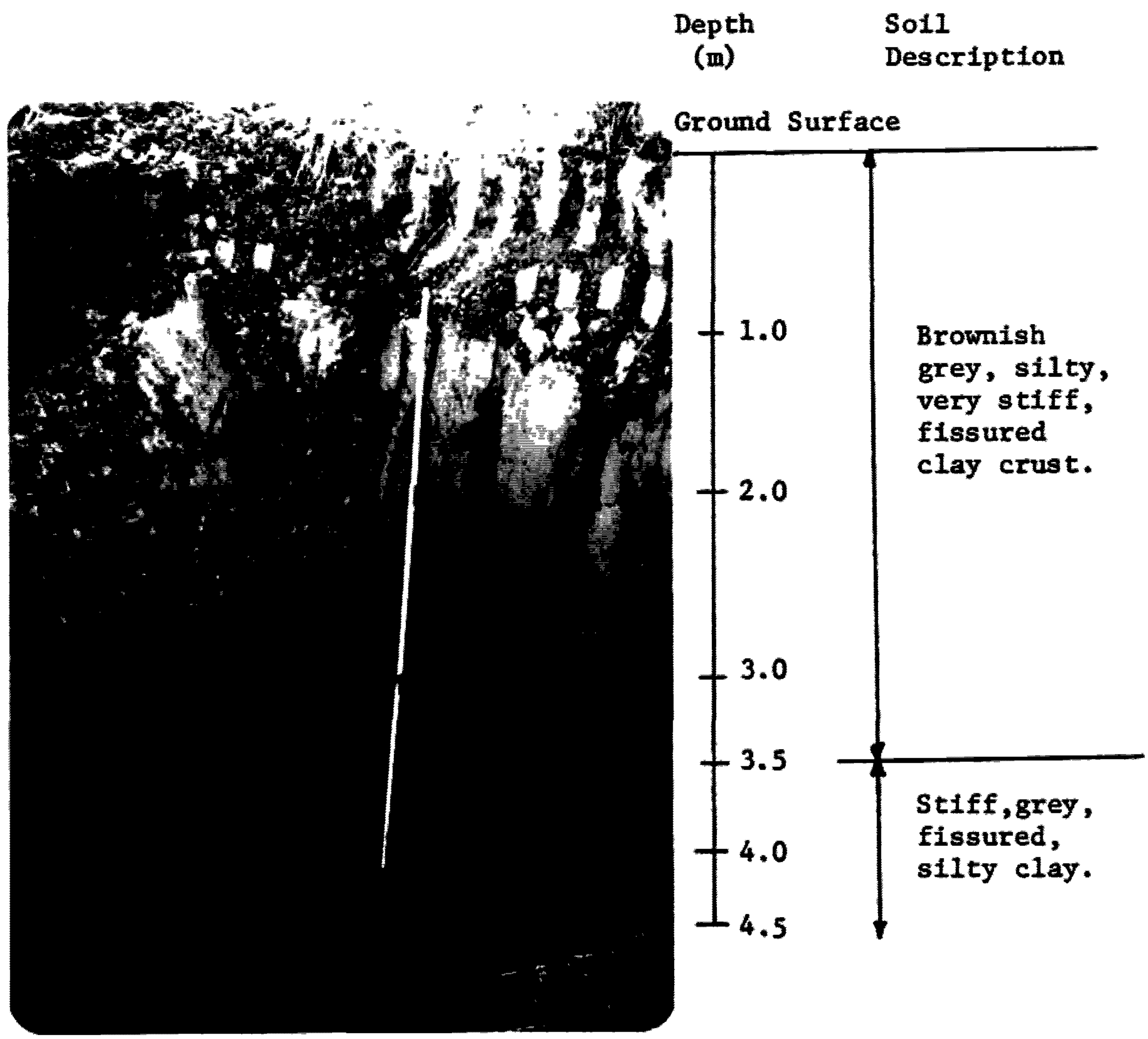


PLATE 5.3.1 **4.5 m deep excavation at Site No. 3, near Green Creek. White surveying rod is 12 feet (3.6 m) long.**

Note the colour change at 3.5 m.

PLATE 5.3.2 Extensive fissuring caused pinnacles of clay to collapse into excavation. Site No. 3, Green Creek North.

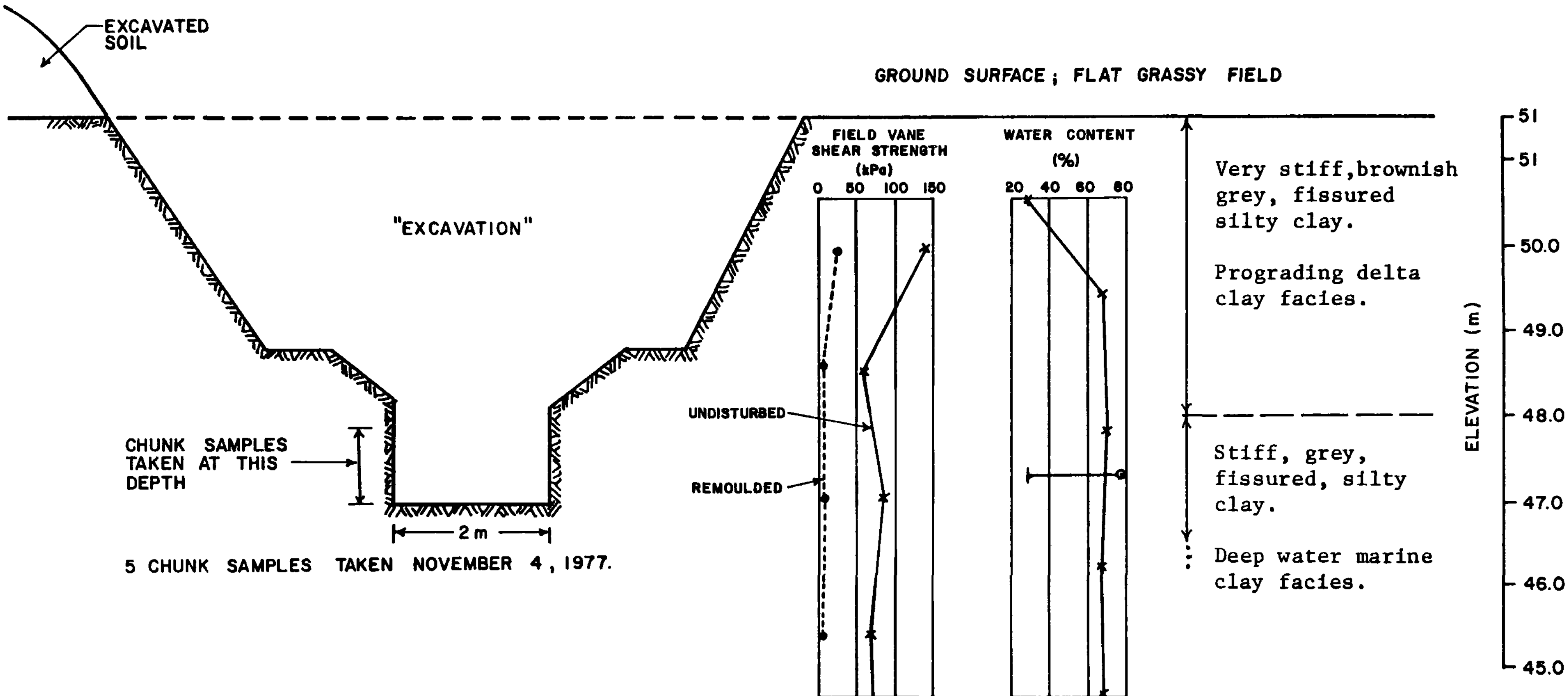
Unfortunately, it was not possible to perform field vane shear or hand vane shear tests in the excavation due to the danger of collapse. Fortunately, a local consulting geotechnical engineering firm had made several boreholes in the area of Green Creek, two of which are within 250 metres of the excavation. The field vane shear strength and water content profiles with depth of these two boreholes (No. B-3 and No. F-400, McRostie, Genest, Middlemiss & Associates Ltd., Report No. E2903) are quite similar and it is thought they adequately represent the shear strength at the site of the excavation.

Figure 5.4.1 shows a section through the excavation and presents the field vane shear strength and water content profile from the above boreholes.

5.5 Site No. 4, Templeton East

Site No. 4 is located approximately 70 km northeast of Ottawa in the Municipality of Templeton East, Quebec. It is situated along a small stream that is a tributary to the Petite Riviere Blanche. This small stream has been the scene of frequent landslide activity in the last few years. The site is the most northern of all the sites considered in this thesis. Bedrock outcrops less than 1 km to the north and the ancestral Ottawa River escarpment lies less than 1 km to the south.

Two facies of Leda clay are believed to have been exposed at this site. The uppermost facies is red and grey or light grey and dark grey banded closely fissured silty clay. This facies grades into a dark grey, silty, brittle, fossiliferous, sensitive clay.



NOTE: Field vane and water content profile data obtained from McRostie, Genest, Middlemiss and Ass. Ltd. Report No. E-2903.

Figure 5.4.1 X-Section through excavation. Site No. 3, Green Creek North.

The photograph on Plate 5.4.1 shows the excavation made at the site to obtain the block samples. The photograph on Plate 5.4.2 is a close-up of the exposed clay. Note the red banding in the clay above the bench and the single solid colour of the grey clay in the bench from which block samples were taken.

The dark grey silty clay contains frequent small intact shells and is classified as the deep water marine facies of Leda clay. Overlying this is the prograding delta clay facies. Fissures were discovered at close spacing in the prodelta clay, however only one fissure was determined in the deep water marine clay. Although the exact cause of fissuring in Leda clay is not precisely known, it may be of significance to note that the prograding delta clay facies appears to be more susceptible to fissure development. At the Templeton East site, water was found to be seeping freely through the fissures thus indicating the prodelta clay at this site had a high secondary permeability.

Figure 5.5.1 shows a section through the slope, the field vane shear strength and water content profiles with depth, and the elevation of block samples and the surrounding clay terrace. It is of interest to note the very high sensitivity that ranges from 20 to 132 for this clay as determined by the field vane test. In addition, the natural water content exceeds the liquid limit by 15%. Furthermore, the very brittle nature of the clay was observed by the fact that extensive radial cracking would occur upon insertion and rotation of the small hand vane into the bench while attempting to perform in-situ shear vane tests after the block samples were taken.

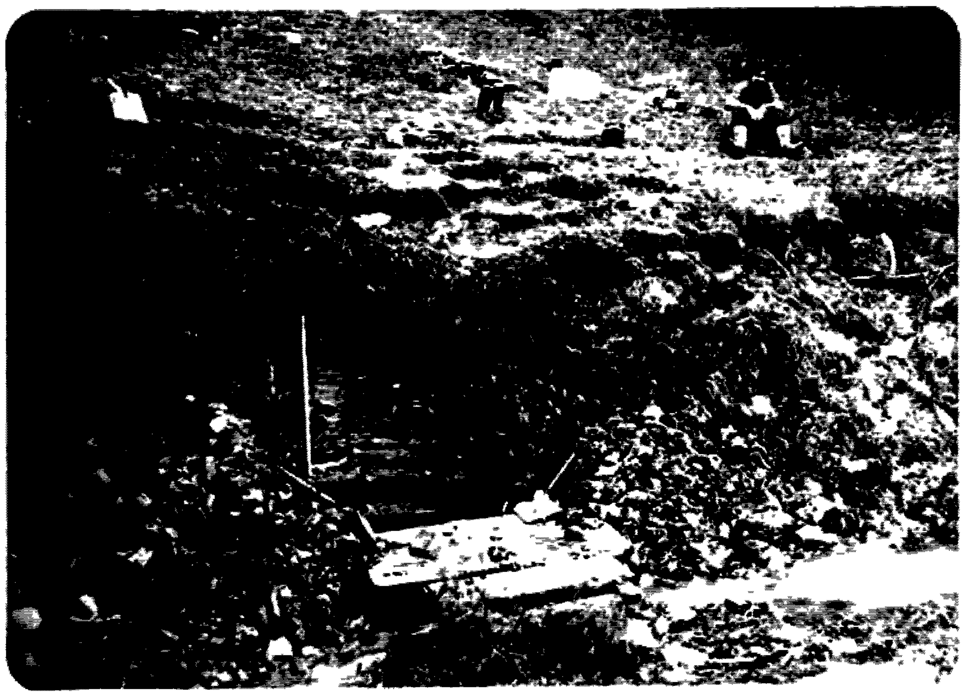


PLATE 5.4.1 Excavation at Site No. 4, Templeton East.

PLATE 5.4.2 Close up of the exposed clay at Site No. 4, Templeton East.

Note the extensively fissured, red and grey banded prograding delta clay above the bench. Also note the massive, grey, deep water marine clay in the bench.

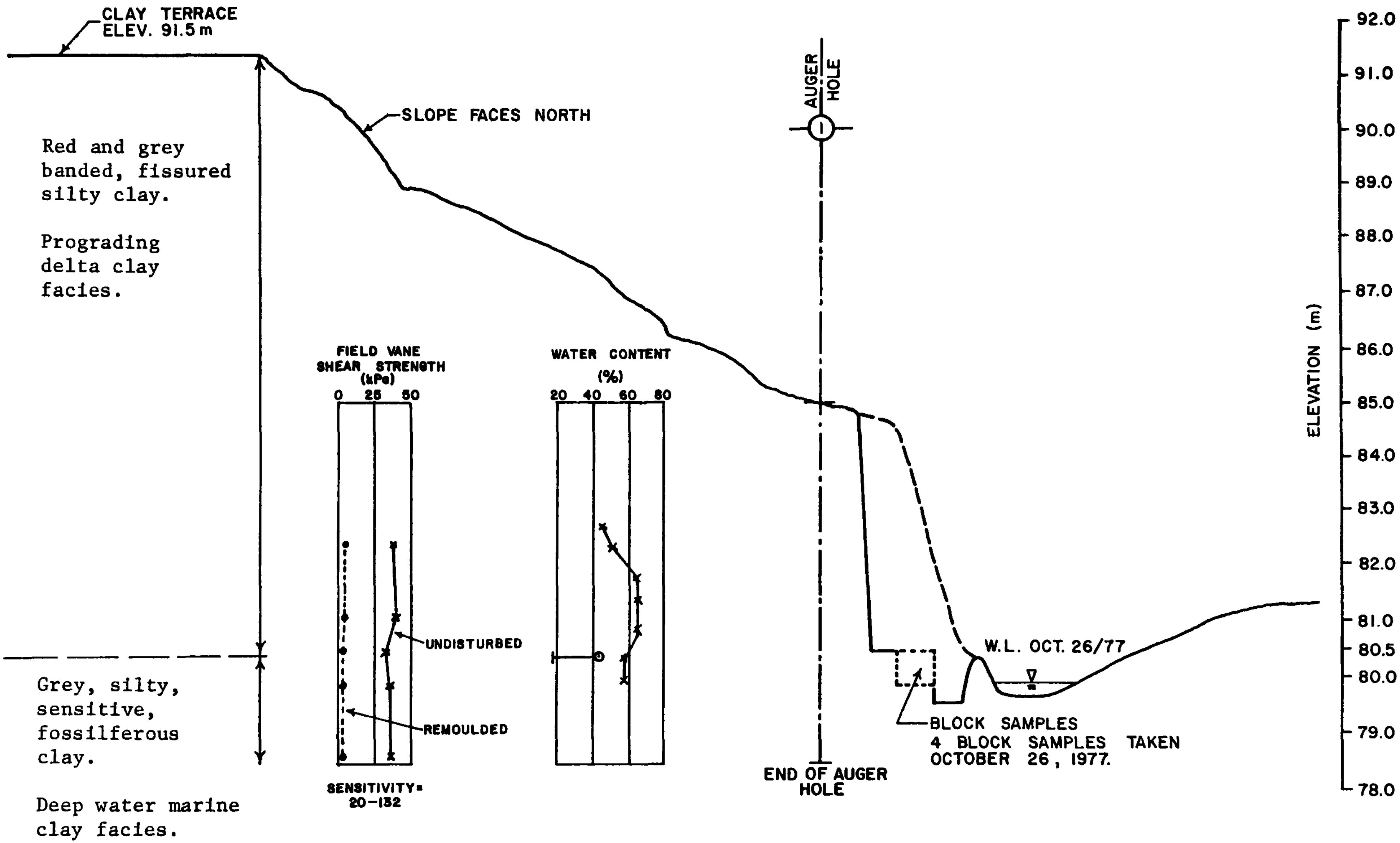


Figure 5.5.1 X-Section through slope.
Site No.4, Templeton East.

5.6 Site No. 5, Stewartville

Site No. 5 is located approximately 70 km west of Ottawa near the town of Stewartville, Ontario. More precisely, it is situated on the south shore of the Madawaska River about 370 metres downstream from the Stewartville Generating Station.

The regional geology near the site is quite complex. Precambrian igneous and metamorphic bedrock and Paleozoic sedimentary bedrock outcrops on the north shore. Thick deposits of predominantly sandy glacial till with boulders and fluvial sand and gravel cover the greater part of the land surface on the north and south shores of the river. Approximately 2 km downstream, there are extensive deposits of marine clay.

At the site, a grey, very stiff, undesiccated silty clay with occasional sand seams becoming more silty with increasing depth, according to tactile examination, was encountered below a thick deposit of sand and gravel with small boulders. This clay lies at about the same elevation (100 m) as that existing downstream and it would be logical to believe that they are the same deposit.

It is thought that this clay was deposited under marine conditions when the Champlain Sea covered most of this area. The clay was deposited in local depressions in the bedrock of the Madawaska Valley. The glacial till that had been deposited over the highland bedrock was not submerged by the Champlain Sea. As the land continued to rebound and the sea regressed, conditions gradually changed from marine to fluvial-deltaic and eroded the glacial till and deposited it as a thick stratum of sand and gravel overlying the clay. This stratum is as thick as 48 metres. Throughout the years, the Madawaska River has incised a channel through the sand and gravel and clay down to the bedrock, exposing the clay in the river banks as it exists today.

Due to the presence of the occasional silty seam and the general lack of fossils and organic mottling in the clay sampled, it is thought that the clay reflects an environment of deposition attributed to that of a prograding delta.

Plate 5.5.1 shows the sand and gravel layer overlying the clay. Plate 5.5.2 shows a view of the excavation into the clay just prior to block sampling. Figure 5.6.1 shows a section through the slope illustrating the stratigraphy, location of block samples, river water level and the water content profile with depth. This clay was too stiff to permit the insertion of the field vane. The clay at this site is, in fact, so hard that the block samples would actually emit a ringing sound when rapped by the knuckles.

5.7 Conclusion

High quality samples were obtained from five sites within a radius of 70 km of Ottawa. These samples vary in strength from soft to very hard and represent each of the three facies of Leda clay. The samples were generally obtained by hand excavation into the face at the bottom of a natural slope.

The location, local geology, site description and soil stratigraphy of each site have been discussed. Field vane shear strength and water content profiles with depth have been presented for all sites.

PLATE 5.5.1 Sand and gravel layer with small boulders overlying clay at Site No. 5, Stewartville.

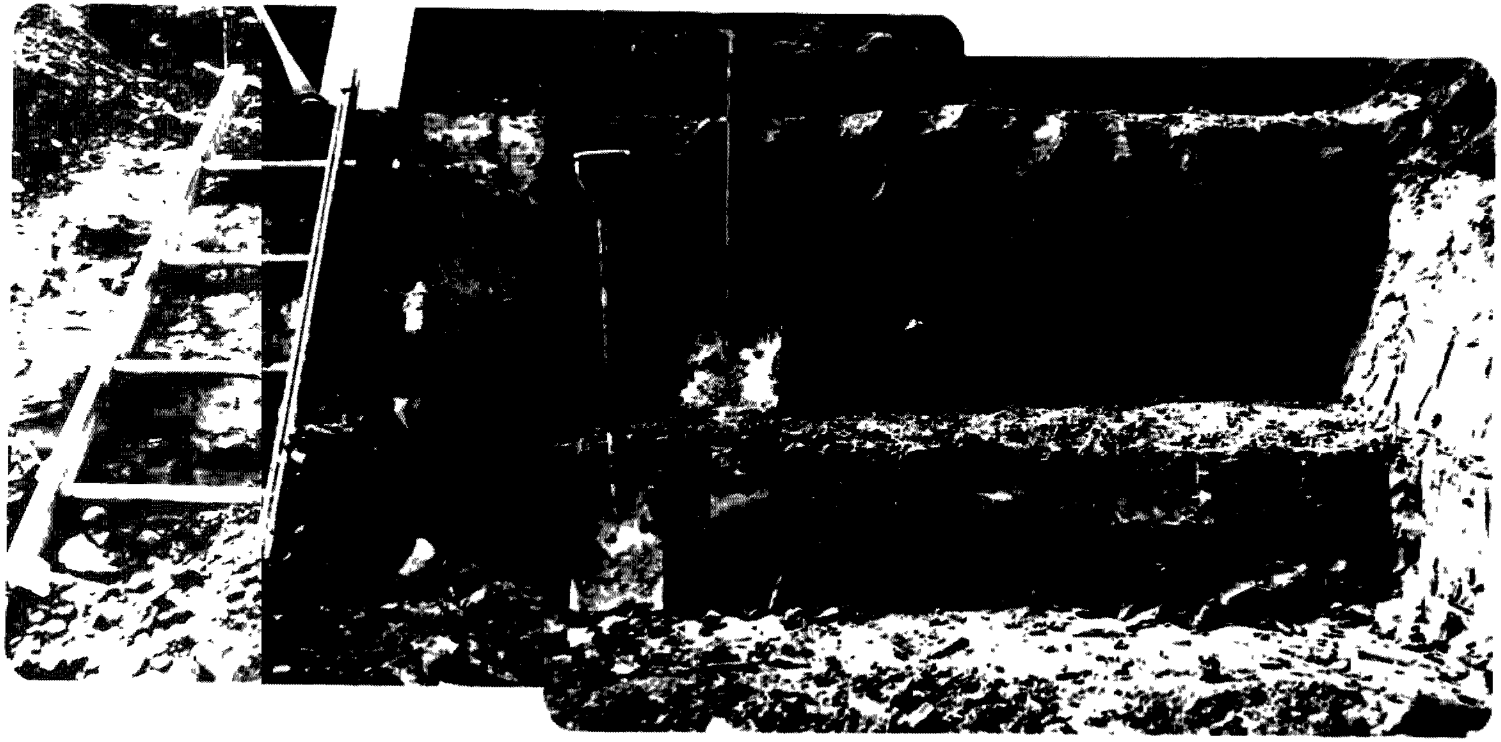


PLATE 5.5.2 Exposed clay in bench prior to block sampling at Site No. 5, Stewartville.

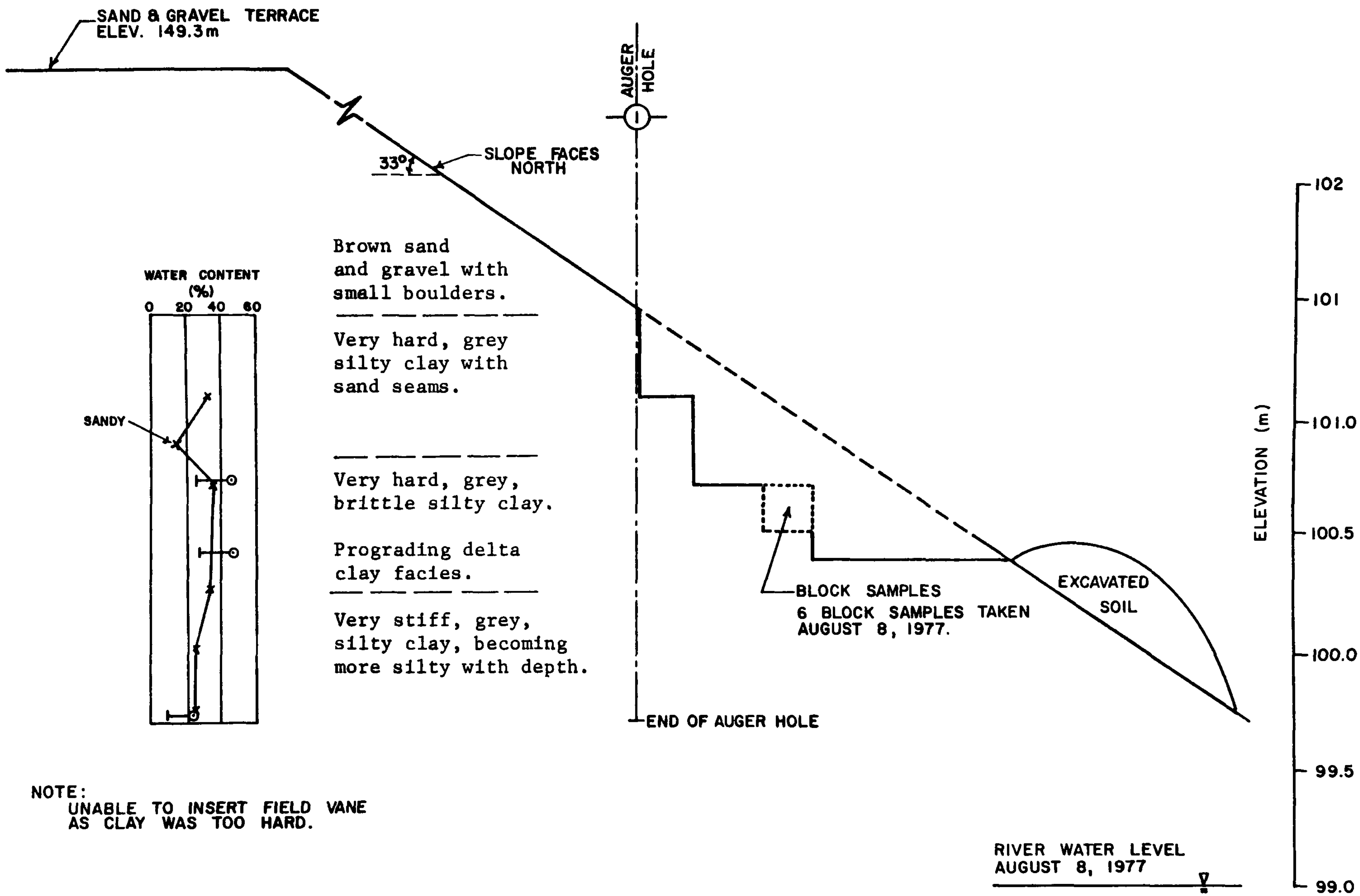


Figure 5.6.1 X-Section through slope.
Site No. 5, Stewartville.

CHAPTER 6

FIELD AND LABORATORY PROCEDURES

6.0 Introduction

Proper procedures are essential to ensure the accuracy, reliability and repeatability of test results. It is also essential for the reader to be familiar with the way in which tests were performed in order to assess their applicability to practical problems. This chapter describes the procedures followed in this thesis. The chapter is divided into two parts. Part A discusses the procedures followed in the field regarding block sampling and in-situ vane shear testing. Part B discusses the rationale and procedures for all the laboratory tests performed.

PART A - Procedures Followed in the Field

6.1 Site Investigation

Once a location was selected as a possible site for block sampling, according to the criteria and constraints that were outlined in Chapter 5, a preliminary soil investigation was carried out. This involved hand augering a 7.6 cm diameter borehole to a depth of about 3 m while performing in-situ field vane shear tests at 0.3 m intervals of depth. Soil samples were collected for identification, water content determination and Atterberg Limits at 0.15 m intervals of depth.

A four bladed field vane with tapered ends, similar to that described in ASTM D2573-72, was used to measure the in-situ undrained shear strength of the clay. This vane had a blade diameter of 6.7 cm, a straight length of 15 cm and the ends were tapered at 45 degree angles. The blade thickness was 3.3 mm. The vane was inserted manually into the borehole and the torque applied to cause failure was measured by tension pull scales attached at a specified radius (generally 30.5 cm) from the center of the rods. After failure, the vane was rotated ten complete revolutions and the remoulded shear strength was measured immediately.

The soil samples collected were brought to the lab and water contents were immediately measured for each sample. Atterberg limts were performed on selected samples.

Further in-situ vane shear tests were performed alongside the borehole or into the face of the slope at intervals of 10 or 15 cm using a small hand held vane shear tester. The hand vane does not have tapered ends, has a blade diameter of 1.9 cm, a height of 2.8 cm and a blade thickness of 1 mm. Because of the smaller blade dimensions and the smaller testing interval, the hand vane was helpful in the determination of whether there were strata of varying strength and to discover whether there were sand layers present that might prevent the possibility of obtaining high quality block samples. Sand layers are easily determined with a small vane since a substantial increase in resistance to penetration may be noticed when they are encountered.

If the clay at the site was undisturbed, undesiccated, free of sand layers and displayed a reasonably uniform water content and undrained shear strength profile over the range of depth considered for block sampling, the site was considered satisfactory and an attempt was made to take block samples. Occasionally, it was necessary to reject some sites, which appeared satisfactory after the preliminary soil investigation, because of the presence of closely spaced (less than 10 cm) macrofissures which became visible only while excavating.

6.2 Block Sampling

There exists no standard or accepted recommended practice on how block samples should be taken. In fact the methods by which block samples are obtained are generally only briefly described in the literature. The reason for this is that the practice of block sampling is a simple one usually involving ordinary hand tools. Any procedure that results in the obtention of undisturbed blocks of good size could be considered acceptable.

The procedure generally requires that an excavation be made into the desired deposit. A bench, from which the samples are to be taken, and a working space or trench, are cut by hand shovel to minimize disturbance. Obtaining a block of certain dimensions can then be a laborious task. LaRochelle and Lefebvre (1971) describe a procedure by which vertical slots were carefully cut around the blocks which were disengaged by many passes of a wire along the bottom of a 23 cm cube. It is presumed that these vertical slots would have been cut by a small spade or shovel. This procedure is time consuming and results in the wastage of a considerable part of the material in the bench, which necessitates additional excavation to obtain a specified number of blocks. What is required then is a procedure by which thin vertical slots could be quickly and easily cut around the blocks without disturbing it.

At four of the five sites, discussed in this thesis, block samples were obtained by the procedure described below. At Site No. 3, Green Creek North, 'chunk' samples were taken for the reasons described in Section 5.4.

As was explained in Chapter 5, it was desired to obtain block samples from the bottom face of the slope to study the stress-strain behaviour of the clay and thereby determine whether peak or post-peak strengths were operative. This zone of a slope is considered that most stressed and should only exhibit post-peak strength, according to theory.

Excavation into the face of the slope and to the depth at which block samples were to be taken was performed by hand shovel. A bench 30 cm high, 1.5 m wide and 60 cm deep into the face of the slope was cut with a sharp flat edge shovel. A gas powered chain saw was used to cut three grooves about 25 cm long and about 1 cm wide on the sides and back of each block. A short length of high strength piano wire was then placed in the side and back grooves to a depth of 25 cm and then pulled through the block with a sawing motion until the bottom had been completely cut. A 20 cm by 20 cm cutting blade was then inserted beneath the block along the bottom cut, to permit the block to be slipped out of the bench. This was not as easy a task as it sounds, since significant adhesion remained between the block and the remoulded material in the grooves which resisted movement. The 25 cm cubed block, weighing generally about 30 kilograms, was then slid onto a small wooden board that had been previously covered with a layer of Saran wrap. All remoulded clay was then removed from the sides, and then the block was completely sealed in 2 layers of Saran wrap and labelled. The block was then lifted up by the underlying board and placed in a cardboard box which had a 5 cm thick foam cushion on the bottom to dampen vibrations during transportation to the laboratory.

The chain saw was used in the block sampling procedure for all the sites at which block samples were taken. The clay at these sites varied from soft to very stiff. It is believed that this procedure is the most efficient and practical way to obtain good quality block samples. Disturbance to the block is at a minimum since the chain saw does not displace and consequently strain the soil, but rather cuts a groove and ejects the disturbed clay. Little vibration is imparted to the block since vibration is quickly damped by remoulded clay in the grooves.

Photo No. 1 on Plate 6.1 illustrates the chain saw in action. Note the sharp corners easily and rapidly obtainable. Photo No. 2 shows the block sealed in Saran wrap, labelled and sitting on the wood board, about to be placed into the cardboard box with the green foam bottom.

Upon arrival at the laboratory, the block was again sealed with another layer of Saran wrap, then covered with a double coat of aluminum foil. Cheese cloth was placed over this and then the entire block was covered with about a 0.5 cm thick layer of wax. The cheese cloth acts to reinforce the wax. A bottom layer of aluminum foil was then placed over the wax to prevent the wax from adhering to any surface upon which the block was placed.

The samples were then stored, until required for testing, in a cool (12°C), humid (100% relative humidity) storage area. After the blocks had been opened to obtain samples for testing, they were resealed, surrounded by wet rags and paper, and then stored in a large refrigerator at 10°C.

Photo No. 1 Chain saw in action.

Photo No. 2 Block sample sealed, labelled and ready to be placed in protective box. Block samples are from Site No. [illegible], Stewartville.

Plate 5.1 Block sampling procedures.

PART B - Procedures Followed in the Laboratory

6.3 Laboratory Samples

Samples for testing were cut from the block generally by a small electric sabre saw with a thin, 8 cm long blade. Little disturbance is caused to the sample by this method since the clay is quickly re-moulded and removed by the blade. The saw motor and housing was never allowed to come in contact with the soil and consequently little vibration was imparted to the sample. This method was thought to be best suited for trimming the samples since the size required could be rapidly and easily cut. The use of a knife blade or thin wire saw often stresses the block and leads to splitting and considerable sample wastage. Samples were always cut at least 1-2 cm larger than required in all dimensions and then trimmed to the correct size by using a very thin wire saw and holding frame.

The next section briefly describes the procedure followed for each laboratory test performed.

6.4 Atterberg Limits, Specific Gravity and Grain Size Distribution (Hydrometer Analysis)

ASTM procedures were followed for each of these basic tests. These procedures are described in ASTM designations: D423-66, D424-59, D854-58 and D422-63.

6.5 Laboratory Vane Tests

Stress controlled vane shear tests were performed on block samples in the laboratory using a Wykenham-Farrance motor driven, spring loaded, vane shear tester. The vane used had a diameter of 1.3 cm, a height of 1.9 cm and a blade thickness of 0.5 mm. The lab vane tests were performed directly on block samples with a small (1 to 2 kPa) confining pressure applied by means of bar clamps to prevent the block from splitting upon insertion of the vane. Values of torque were measured at small increments of angular deformation. The test duration ranged from 5 to 10 minutes, depending on sample strength and the spring used. Tests were performed at both vertical and horizontal orientation. The vane was inserted about 8 cm from either edge of the block to prevent splitting.

6.6 Unconfined Compression Tests

Unconfined compression tests were performed generally on samples trimmed to dimensions of 3.8 cm in diameter and 7.6 cm in length. The samples were placed in a standard triaxial cell apparatus with the enclosing cylindrical perspecs wall removed, to duplicate the loading conditions applied in the triaxial test. The samples were placed on oiled plastic platens to reduce end friction. The samples were loaded by a Wykenham-Farrance load press at a general rate of 0.076 cm/min which corresponds to a strain rate of about 1% per minute. A 668 N (150 lb) load ring was used to measure the applied load.

Water contents were measured on sample trimmings before the test and on the whole sample after the test. Failure modes were sketched. The effects of different strain rates and sample size were studied on samples from Site No. 4, Templeton East.

6.7 Consolidation Tests

6.7.1 General

Consolidation tests were performed on samples from each site. A Wykenham-Farrance fixed ring consolidometer was used. The sample dimensions were: 1.9 cm high and 6.3 cm in diameter, yielding a loaded surface area of 31.6 sq cm. The tests were carried out at a room temperature of about 23°C.

Much controversy exists concerning the proper procedure that should be followed to accurately determine the apparent preconsolidation pressure (Crawford, 1964, Jarrett, 1967). The preconsolidation pressure is termed 'apparent' since it does not always correspond to the maximum load applied on the soil tested during its geological history. This critical pressure is the point at which there is a sudden breakdown in the structure of the clay resulting in a large degree of deformation.

This apparent preconsolidation pressure is known to be significantly affected by load increment ratio, the rate of loading, wall friction on the consolidometer rings and several other factors. Hamilton and Crawford (1960), Crawford (1964,1965) and Jarrett (1967) have shown that the standard procedure of applying a load increment ratio of $\Delta p/p = 1$ to the test specimens produces rates of compression far exceeding those that occur in the field. This "shock" treatment destroys the soil structure, reducing the measured preconsolidation pressure. Several alternative consolidation tests have been proposed (Crawford, 1964; Smith and Wahls, 1969; Lowe et al, 1969; Aboshi et al, 1970) but none of these seem to have been fully accepted by the geotechnical community as a replacement for the standard incremental test for Leda clay. At present, it is believed that most commercial consulting geotechnical companies rely on the standard incremental consolidation tests with the incorporation of a smaller load increment ratio. This smaller load increment ratio reduces the "shock" effect on the soil specimen and also allows the more accurate determination of the apparent preconsolidation pressure since more data points are plotted.

It was desired to follow a loading procedure believed to be similar to that used by consultants in the region to enable comparisons to be made with existing data. Furthermore, one objective of the research is to enable the determined relationships between undrained shear strengths, preconsolidation pressures and drained shear strengths to be extrapolated to other sites by consultants. In order that this might be possible, it is required that similar testing procedures be followed. Thus, it was decided to use a loading procedure for the consolidation tests that would be similar to that commonly used by geotechnical consultants. This was considered more practical than trying to justify the use of one of the alternative types of consolidation tests.

6.7.2 Consolidation Test Procedure

The sample was placed into a ring that had been coated with Teflon and lightly smeared with a silicon base grease to reduce side friction. The sample was then placed in the consolidation cell and submerged in water. The sample was left overnight with no load to determine whether it would swell.

The preconsolidation pressure was then estimated on the basis of the Su/p'_c ratio where Su is the undrained shear strength and p'_c is the preconsolidation pressure. Crawford and Eden (1965) showed that for a large number of sites in the Ottawa region Su/p'_c varies from 0.24 to 0.40. This ratio is generally approximated as being 0.3.

With this estimate of the preconsolidation pressure, a loading sequence was chosen so as to have about 5 loads before the p'_c. Small load increments were chosen in the vicinity of the p'_c in order that it might be accurately determined. The loading sequence was generally quite similar for all the sites with the exception of the magnitude of the final loads required to fully define the consolidation curves for the stronger clays. The initial loads were changed quite rapidly after it became apparent that the sample had virtually stopped consolidating. When the load reached about 40% of the estimated preconsolidation pressure it was left on for a full 24 hours. A typical loading sequence is presented in Table 6.7.1. The load increment ratio $\Delta p/p$ was equal to 1 for the initial loads and then gradually reached a value of about 1/4 near the preconsolidation pressure and remained at this ratio until the end of the test.

The consolidation readings were corrected for machine deflection. The calibration curves for the oedometers are presented in Appendix D. Casagrande's method (1936) was used to determine the most probable preconsolidation pressure. In addition, the minimum and maximum preconsolidation pressures were also deduced according to the method presented by Bozozuk (1971).

Total Load p		Load Increment Δp		Increment Ratio Δp/p	
psf	kPa	psf	kPa		
250	12	250	12		
500	24	250	12	1.0	
1,000	48	500	24	1.0	
2,000	96	1,000	48	1.0	
3,000	144	1,000	48	0.5	
4,000	192	1,000	48	0.33	loading
5,000	239	1,000	48	0.25	
6,500	311	1,500	72	0.3	
8,000	383	1,500	72	0.23	
10,000	479	2,000	96	0.25	
12,000	548	2,000	96	0.20	
8,000	383	4,000	192	0.33	
3,000	144	5,000	239	0.62	unloading
1,000	48	2,000	96	1.5	
250	12	750	36	0.75	

TABLE 6.7.1 Typical Consolidation Loading Sequence

6.8 Triaxial Tests

6.8.1 General

Various types of triaxial texts have been used to define the shear strength envelope of Leda clay. Their merits and deficiencies have been described in Chapters 2 and 3. The selection of the appropriate triaxial test for use in this thesis was made based upon the discussion presented in Sections 3.7.2 and 3.8 and also upon the following objectives and criteria.

a) It was desired to study the relationship between undrained (total stress) and drained (effective stress) shear strength.

b) It would be necessary to perform many triaxial tests to adequately define the failure envelope for the five sites to be studied. Hence, an uncomplicated, quick yet accurate and reliable test would be preferred.

c) It would be desirable to use a test, that is similar to those reported in the literature, to enable a comparison to be made between the thesis and literature test results.

d) It was considered desirable to use a test procedure that could be or has been readily achievable by local geotechnical consultants. Thus, their past and future experience and knowledge of the clays in the area could be correlated with the results of the thesis.

A strain controlled, consolidated, undrained triaxial test with pore water pressure measurements was considered best suited to satisfy the objectives and criteria. This test enables the measurement of shear strength in terms of total and effective stresses. It is a simple, relatively rapid test that can be easily automated. Much work concerning slope stability analyses reported in the literature has been based upon the results of conventional CIU and CID triaxial tests. Thus, direct comparisons could be made between the thesis and literature test results. Most geotechnical consultants are equipped to perform this type of triaxial test and they have collected a significant amount of experience with it. Finally, as discussed in Chapter 3, a consolidated undrained triaxial test can easily and rapidly define the residual strength failure envelope, which appears to provide satisfactory estimates of the factor of safety in either fissured or intact clay slopes.

6.8.2 Triaxial consolidation

It is generally agreed that the process of sedimentation of clay particles takes place under an anisotropic stress system (i.e., the lateral stress is generally less than the vertical stress due to the overlying sediments). This results in anisotropic consolidation of the clay mass under at rest conditions. The ratio of the horizontal effective stress (σ_h') to the vertical effective stress (σ_v') under at rest conditions is termed K_o.

Prominent geotechnical research engineers recommend that soil samples be tested in the laboratory under the same effective stress that exist in-situ (Ladd et al, 1977; Bjerrum, 1973; Janbu, 1977). This logically implies then that the clay samples should be consolidated anisotropically according to the value of K_o for the clay. However, certain difficulties prevented adopting such a procedure and other reasons indicate that anisotropic consolidation is not always necessary.

Uncertainty exists as to the exact value of K_o for Leda clay.

A value of K_o varying from 0.5 to 0.75 is commonly assumed in the geotechnical literature (Bozozuk, 1977). However, work reported by Brooker and Ireland (1965) indicates values of K_o in excess of 1.0 for clays with an overconsolidation ratio greater than 4 and with a plasticity index greater than 28. An overconsolidation ratio of 4 is not infrequent to Leda clays. A value of K_o greater than 1.0 may occur due to excessive lateral pressures induced by geological processes or more commonly if sufficient removal of overburden occurs through erosion, so as to cause the lateral stresses to be greater than the vertical stresses. Thus, it appears difficult to correctly estimate the value of K_o for Leda clays to enable anisotropic consolidation.

All of the block samples tested in this thesis have been taken from the face of slopes as was illustrated in Chapter 5. At the face of a slope, the principal stresses are not oriented vertically and horizontally as is normally the case. The major principal stress is approximately parallel to the slope and the minor principal stress is approximately perpendicular to the face of the slope. Hence, the estimation of vertical and horizontal stresses for the purpose of anisotropic consolidation of vertically oriented samples would be difficult and probably inaccurate since the magnitude and direction of the principal stresses are not well known.

All the triaxial testing for this research was to be carried out in the low effective stress range discussed in Chapter 3. This meant that the consolidation pressures would not be in excess of 50% of the preconsolidation value determined in the oedometer test. Experience with isotropic consolidation of Leda clay samples in the low effective stress range for this thesis and for other projects, indicates that volume changes are quite small and in the order of 1 to 2%. It is thought that anisotropic consolidation to K_o stresses would not significantly affect these result.

Furthermore, Lo and Morin (1972) have shown that the residual strength failure envelope is independent of anisotropy. Hence, if the residual strength failure envelope is used in slope stability analyses, anisotropic consolidation would not be required.

With the above factors in mind, it was decided to consolidate the samples isotropically. Only the peak strenth failure envelope could possibly be slightly affected. However, it was considered that an error in the estimation of K_o would more severely affect the shear strength results than isotropic consolidation. As a final remark, isotropic consolidation is easier and less time consuming to perform.

6.8.3 Stress Range

It is generally accepted that the stress range of interest to the geotechnical engineer analyzing the stability of slopes is the low effective stress range, previously discussed in Chapter 3. The limits of this low effective stress range vary from site to site and depend upon the strength and degree of preconsolidation of the clay.

A review was made of published strength envelopes for the clay from several sites in the Ottawa region and the upper limit of the low effective stress range was defined in terms of the preconsolidation pressure of the clay. The low effective stress range was distinguished from the other stress ranges by the fact that in this range the Mohr-Coulomb failure envelope exhibits a small cohesion intercept and high friction angle. The intermediate stress range, on the other hand is the near horizontal part of the envelope and exhibits a high cohesion intercept and a low friction angle. In most cases, the authors had themselves defined the limit of the low effective stress range.

Table 6.8.1 presents the results of that review. From the table it would appear that, in general, one could expect the upper limit of the low effective stress range to be less than 50% of the preconsolidation pressure. Therefore, the range of stress used in the testing program varied from 0.0 to 0.5 p_c'. It is thought that such a stress range would adequately define the shear behaviour of the clay over a range of stress to which the clay in slopes approaching an unstable condition might be exposed.

Lefebvre and LaRochelle (1973) found that the part of the failure "circle involving effective normal stresses less than 10 kPa had a negligible influence on the factor of safety". Tensile testing, to define the behaviour of the clay below this value, is then not essential for slope stability analysis, and a stress range of 0.0 to 0.5 p_c' would appear to be adequate.

6.8.4 Description of Equipment

a) Samples

The samples used in the triaxial tests were 5 cm in diameter and 10 cm in length. They were surrounded laterally with filter paper

Site	Reference	Field Vane Strength (kPa)	p'_c (kPa)	Upper Limit of Low Stress Range, p'_L $p' = (\frac{\sigma'_1 + \sigma'_3}{2})$ (kPa)	$\frac{p'_L}{p'_c}$
Rockcliffe	Mitchell (1970a)	69	240	100	0.42
Breckenridge	Mitchell (1970a)	48	163	80	0.49
Orleans	Eden & Jarrett (1971)	67	215	100	0.47
Castor River	Toombs (1974)	38	110	57	0.52
Bear Brook	Toombs (1974)	115	297	115	0.39
South Nation	Toombs (1974)	48	129	80	0.62

Average = 0.49

TABLE 6.8.1 Limits of Low Stress Range in the Literature drains identical to those described by Bishop and Henkel (1962). The top and bottom of the sample was also covered with filter paper. One porous stone was used at the bottom of the sample to allow the measurement of pore water pressure. The top of the sample was loaded by a plastic platen. The samples were enclosed in a latex rubber membrane 0.4 mm thick.

b) Use of a back pressure

There are several important reasons for the use of a back pressure. These are:

1. To assure full saturation of the sample, porous stones and filter paper, necessary to ensure the accurate measurement of volume changes and pore water pressures.
2. To assure solution of air bubbles in the pressure lines so that there are fewer chances of pressure loss across air blockages.

The initial degree of saturation of the clay samples tested was found to be quite high and generally ranged from 97 to 100% according to calculations and the measured values of the pore pressure parameter B. Work reported by Toombs (1974) indicated that a back pressure of 200 kPa for a period of 16 to 24 hours would be adequate to ensure full saturation and solution of any small, undetected air bubbles in the pressure lines. The back pressure of 200 kPa used is twice that typically found in the literature (Lefebvre and LaRochelle, 1973). Careful surveillance of the pressure lines and triaxial cell indicated no air bubbles after saturation, prior to shearing. Measurement of the pore pressure parameter B and calculations indicated the degree of saturation to be in excess of 99% and generally 100% after saturation, prior to shearing.

c) Test equipment and set-up

The equipment used to carry out the triaxial tests is listed in Table 6.8.2. A photograph of the experimental set-up is presented on Plate 6.2.

Accurate measurement of pressures is very important in low stress range testing. The pressure gauges listed in Table 6.8.2 are accurate only to ±1 psi or (±7 kPa). To achieve better accuracy in the measurement of pressures, a mercury manometer was placed in parallel to the triaxial cell, so that it measured the absolute difference between the back pressure and the cell pressure. This manometer is accurate to 0.5 kPa.

The flow meter for volume change measurements during consolidation was placed in series between the manometer and the back pressure line into the triaxial cell. The flow meter is accurate to ±0.5 cc. In this way any volume change indicated on the flow meter was a measure of flow either into or out of the sample, and not flow in the pressure lines due to a fluctuating mercury manometer.

6.8.5 Test Procedure

The following procedure was followed for all of the CIU tests performed:

1. The sample was trimmed, sketched, weighed, surrounded by saturated filter paper and then the rubber membrane. Air between the membrane,

Piece	No. of Pieces	Manufacturer	Model
Rotating bushing triaxial cell for 1-1/2" or 2" diameter samples	2	Wykenham-Farrance	No. 1150
Flow measuring device	2	Wykenham-Farrance	No. P.3
Self compensating mercury pressure pot system	2		
Pressure measuring Control panel	4	Engineering Laboratory Equipment Limited	
Mercury manometer	2	Ottawa University	
Load cell	2		No. 45044, 72125
Porewater pressure transducer	2		No. 54427, 54428
Dial gauge	2		
Load press	2	Wykenham-Farrance	
Digital strain readout and printing device	1		

Note: Calibration curves for the load cells and porewater pressure transducers are presented in Appendix D.

TABLE 6.8.2 Triaxial Test Equipment

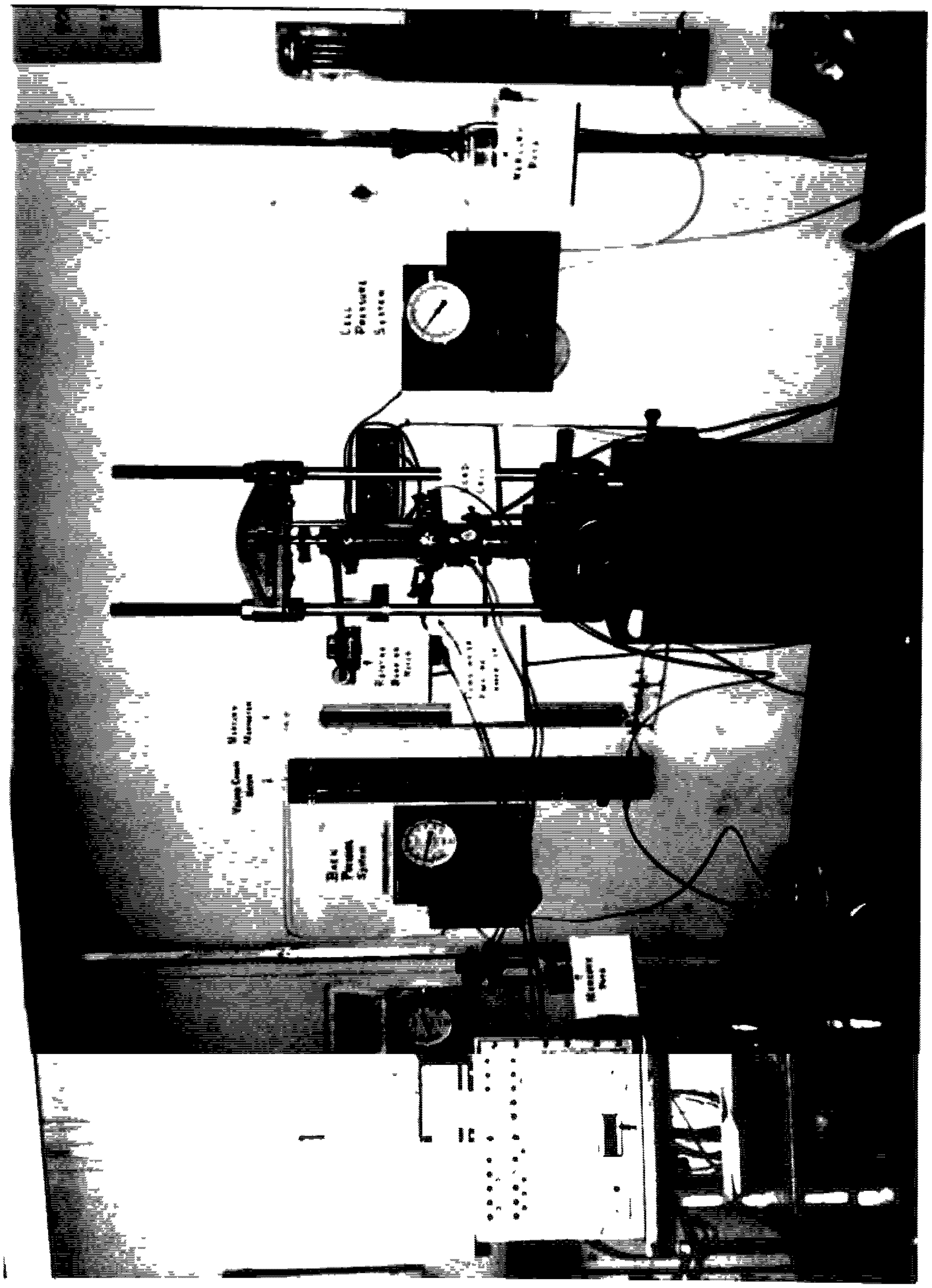


Plate 6.2 Experimental set up.

filter paper and sample was removed as much as possible by passing water up the sides of the sample. Care was taken to avoid sample disturbance.

2. The sample was then placed in the triaxial cell and a cell pressure slightly (1-3 kPa) in excess of a back pressure of 200 kPa was placed on the sample overnight (16 to 24) hours. The purpose of the excess cell pressure was to reduce the amount of water trapped between the sample and the membrane.

3. A predetermined all-round (isotropic) consolidation pressure was then applied. Under this consolidation pressure, a plot of change in volume vs. log time was made. The sample was allowed to consolidate until the curve had virtually flattened and consolidation had basically stopped.

4. The sample was then sheared at a constant rate of 0.0033 cm/min or approximately 1.8% strain per hour until a minimum strain of 6% was obtained. This strain rate is consistent with rates of 1 to 2% per hour reported in the literature (e.g., Bozozuk, 1977). During shear, values of load and pore water pressure were printed automatically on a paper tape. A short time interval between prints was used prior to and for a short time after failure yielding many data points. A longer time interval was used after failure since the load and porewater pressure were then changing slowly. Thus, it was not necessary to remain with the sample throughout the whole test taking readings. Strain readings measured by a dial gauge were recorded periodically throughout the tests in order to provide a check on the strain rate.

5. After the test was terminated, the apparatus was dismantled,

the sample was weighed and the mode of failure was sketched.

6. The readings of load and pore pressure taken at specific time intervals were transferred to a prepared data form amenable to punching up computer cards. This data form is presented in Appendix C.

7. A computer program, written by the author, was used to perform all the required calculations including the correction for the membrane and filter paper. In addition, the program would plot the stress-strain curve, variation of pore water pressure with strain, variation of principal effective ratio σ_1'/σ_3' with strain, variation of pore pressure parameter 'A' with strain and lastly the total and effective stress paths. The computer program, as well as an example of its output, are presented in Appendix C.

8. Two tests were initially performed for each site. One at a very low confining pressure of 0.7 kPa, and one at a higher confining pressure that would yield a failure point in the vicinity of $p' = \frac{\sigma_1' + \sigma_3'}{2}$ equal to 50% of the preconsolidation pressure (0.5 p_c'). Once these test results were computed and the resulting minimum and maximum failure points in the low stress range were plotted, it was a simple matter to choose other confining pressures that would yield failure points so as to fill in the space between the two limits. Five to six triaxial tests were performed for each site. The very low confining pressure of 0.7 kPa was chosen so as to enable the measurement of pore water pressures developed in a virtually unconfined test.

A few remarks concerning the test procedure and apparatus would seem appropriate at this time.

a) The use of the calibrated load cell, calibrated porewater pressure transducer and digital readout/printer allows for the complete automation of the test. Once started, the test will run on its own allowing one to accomplish other tasks during this period. Periodic checking ensures that all is going well.

b) It was possible to run two tests at the same time since two complete systems were available. The normal schedule allowed for the two tests to be consolidated in the morning and then sheared and dismantled in the afternoon.

c) The use of the load cell had the further advantage of ensuring a constant rate of strain throughout the test. Furthermore, its use was considered superior to a load ring which stores deformation as strain energy and then releases it suddenly and often dramatically when the sample actually fails. This disguises the true shape of the post-peak stress-strain curve.

d) The use of the computer program saved a considerable amount of time that would have been required for computation and plotting of the results.

6.8.6 Corrections to the Triaxial Test Results

a) Piston friction

If friction developed between the piston and the bushing to the top of the triaxial cell, greater loads than those imparted to the sample would be recorded. It is believed that possible piston friction was greatly reduced, if not eliminated, by the use of a rotating bushing at the top of the triaxial cell. Thus, no corrections were required to the triaxial test data to account for piston friction.

b) Change in sample area

As the samples strain, they deform and/or develop a shear plane. This results in a continuous change in area over which the sample is loaded. The very complex and varied mode by which samples fail makes it difficult to provide a valid correction applicable to all types of failure.

In this thesis, the common area correction based on the theory that the sample deforms as a right cylinder was used. Such a correction implies that the loaded area increases proportionately with strain. The new area at some value of unit strain (ε) is $A' = \frac{A_o}{1-\varepsilon}$ where A_o = the initial sample area at the start of the test.

c) Correction due to rubber membrane and filter paper drains

Restraint is imposed on the clay sample by the presence of the rubber membrane and filter paper drains. A correction must therefore be made to the measured stresses.

Corrections for the rubber membrane and filter paper drains have been proposed by Bishop and Henkel (1962), Duncan and Seed (1967) and LaRochelle (1967). However, the corrections suggested by Bishop and Henkel, and Duncan and Seed, are not applicable for values of stress less than 100 kPa. Since much of the testing in this thesis would be performed at confining pressures less than 100 kPa, these corrections were not considered acceptable. The correction proposed by LaRochelle (1967) was found to be not easily adaptable to computer techniques.

Gill (1968) has proposed a correction for membrane and filter paper drains in the low effective stress range. He based this correction on direct measurements of the increased deviator stress between samples with: a) two rubber membranes and a filter paper drain

b) only one rubber membrane.

He performed numerous CIU tests with σ_3 constant and σ_1 increasing at various confining pressures to adequately define the correction in the low and high effective stress ranges. His correction factors (C.F.), to be subtracted from the deviator stress, are detailed below and are applicable for strains from 0 to 10%.

For a consolidation pressure less than 70 kPa (10 psi):

$$\text{C.F.} = (0.70 + 0.167\ p) \times \frac{\text{strain \%}}{10}$$ where p = consolidation pressure in psi (1 psi = 6.895 kPa)

For a consolidation pressure greater than 70 kPa (10 psi):

$$\text{C.F.} = (2.20 + 0.017) \times \frac{\text{strain \%}}{10}$$

For strains greater than 10%, the multiplying factor outside of the brackets is removed.

Because of the similarity between test procedures and the ease with which these correction factors could be programmed, Gill's correction factors were adopted for use in this thesis.

It it interesting to note that for consolidation pressures greater than 70 kPa (10 psi) and strain greater than 10%, Gill's equation yields a correction of 16 kPa (2.37 psi) which is close to the general correction factor of Bishop and Henkel (1962) of 14 kPa (2.0 psi).

Typical correction factors at peak strength would generally be less than 1.5 kPa (0.2 psi) which is negligible. However, typical correction factors at residual strength at 6% strain would generally be about 5 to 7 kPa (0.75 to 1.0 psi) which is significant.

6.8.7 Choice of Failure Criterion

Two distinct failure criterion exist to define failure of triaxial test samples. These are the point of maximum principal stress difference ($\sigma_1 - \sigma_3$) and the point of maximum principal effective stress ratio (σ_1'/σ_3'). In drained tests, these two failure criteria occur simultaneously. However, in an undrained test they may occur at significantly different values of strain. Bjerrum and Simons (1960), Peck (1960) and Simons (1963) argue that the criterion of maximum principal effective stress ratio should be used since they found closer agreement between the angle of internal friction between drained (ϕ_d') and undrained test (ϕ_u') based on this criterion. Crawford (1960,1963) however, argues against the use of this criterion in Leda clay due to the very sensitive nature of the clay. He found that although the maximum deviator stress occurred at low strains, the porewater pressure increased with further strain. The increase in porewater pressure with strain causes the value of σ_3' to diminish rapidly, consequently the principal stress ratio increases with further strain. Crawford therefore discarded the criterion of maximum principal effective stress ratio since it did not reach a maximum within what was considered to be an acceptable degree of strain.

Experience with Leda clays tested in this thesis yielded similar results. Although the maximum deviator stress occurs at a strain of about 1%, the principal effective stress ratio in some cases did not reach a maximum even by 6% strain when the test was terminated. Thus, the criterion of maximum principal effective stress ratio does not always yield reasonable results for Leda clays. Hence, the criterion of maximum principal stress difference was adopted for use in this thesis.

6.9 Summary and Conclusions

This chapter has described in detail the procedures followed in the field and in the laboratory. In addition, the rationale for the choice of triaxial test, type of consolidation, stress range and failure criterion has been discussed. It was determined that a series of CIU triaxial tests, conducted in the low effective stress range with a failure criterion of maximum principal stress difference, would be appropriate to satisfy the objectives and constraints of the testing program. A computer program, to perform the calculations and plot the results of the triaxial tests, has also been presented.

Table 6.9.1 lists the numbers of each type of laboratory tests that were performed for this thesis.

Test	Number Performed
Atterberg Limits	10
Specific Gravity Determination	12
Grain Size Distribution, Hydrometer Method	6
Oedometer Tests	6
Laboratory Vane Shear Tests	15
Unconfined Compression Tests	22
CIU Triaxial Tests	35

TABLE 6.9.1 List of Laboratory Tests Performed

CHAPTER 7

PRESENTATION AND DISCUSSION OF THE LABORATORY TEST RESULTS

7.0 Introduction

This chapter presents the results of all the laboratory tests performed for each site. These test results include: basic identification tests, grain size analyses, oedometer tests, unconfined compression tests, laboratory vane tests and a series of consolidated isotropically, undrained triaxial tests. The significance of the test results are briefly discussed. A more detailed analyses and interpretation of the test results is presented in Chapter 8.

7.1 Basic Geotechnical Properties

Table 7.1.1 presents a summary of the basic index properties determined for the clay at each site. A brief review of this data indicates no succinct correlation between either elevation, bulk density void ratio, natural water content, Atterberg Limits and the clay facies. This would suggest that quite a range in basic index properties exists for each clay facies. The designation of mean index properties for each facies, as presentedby Fransham and Gadd (1977), might then be considered misleading.

Figure 7.1.1 is a plot of the Atterberg Limit results from each site on Casagrande's plasticity chart. Also plotted are Fransham and Gadd's average results, that were presented in Table 4.2.1. The results indicate that the deep water marine clay may vary from medium to very

Site No.	Location	Geological Facies of Leda Clay	Elevation of Block Samples (metres)*	Specific Gravity	Bulk Density (kN/m^3)	Void Ratio e_o	Avg Water Content %	Consistency Limits W_L (%)	I_p (%)	I_L (%)	Activity Ratio
1	St. Bernadin	Prograding Delta clay	47.2	2.79	15.35	1.98	66.	56.0	36.	1.3	.59
2	Bear Brook East	Shoaling Prograding									
		Delta silt	48.8	2.77	19.32	0.80	32.	32.0	16.5	1.18	.70
		Delta clay	48.8	2.79	15.85	1.70	64.	61.0	37.3	1.03	.44
3	Green Creek North	Deep water Marine clay	47.2	2.84	15.6	2.08	70.	78.0	49.0	0.9	.66
4	Templeton East	Deep water Marine clay	80.46	2.79	16.19	1.67	58.	41.0	21.	1.8	.38
5	Stewartville	Prograding Delta clay	100.	2.78	18.06	1.1	37.	47.0	28.	0.6	.51

*For most sites elevations have been determined with reference to topographic maps

TABLE 7.1.1 Summary Table of Average Index Properties

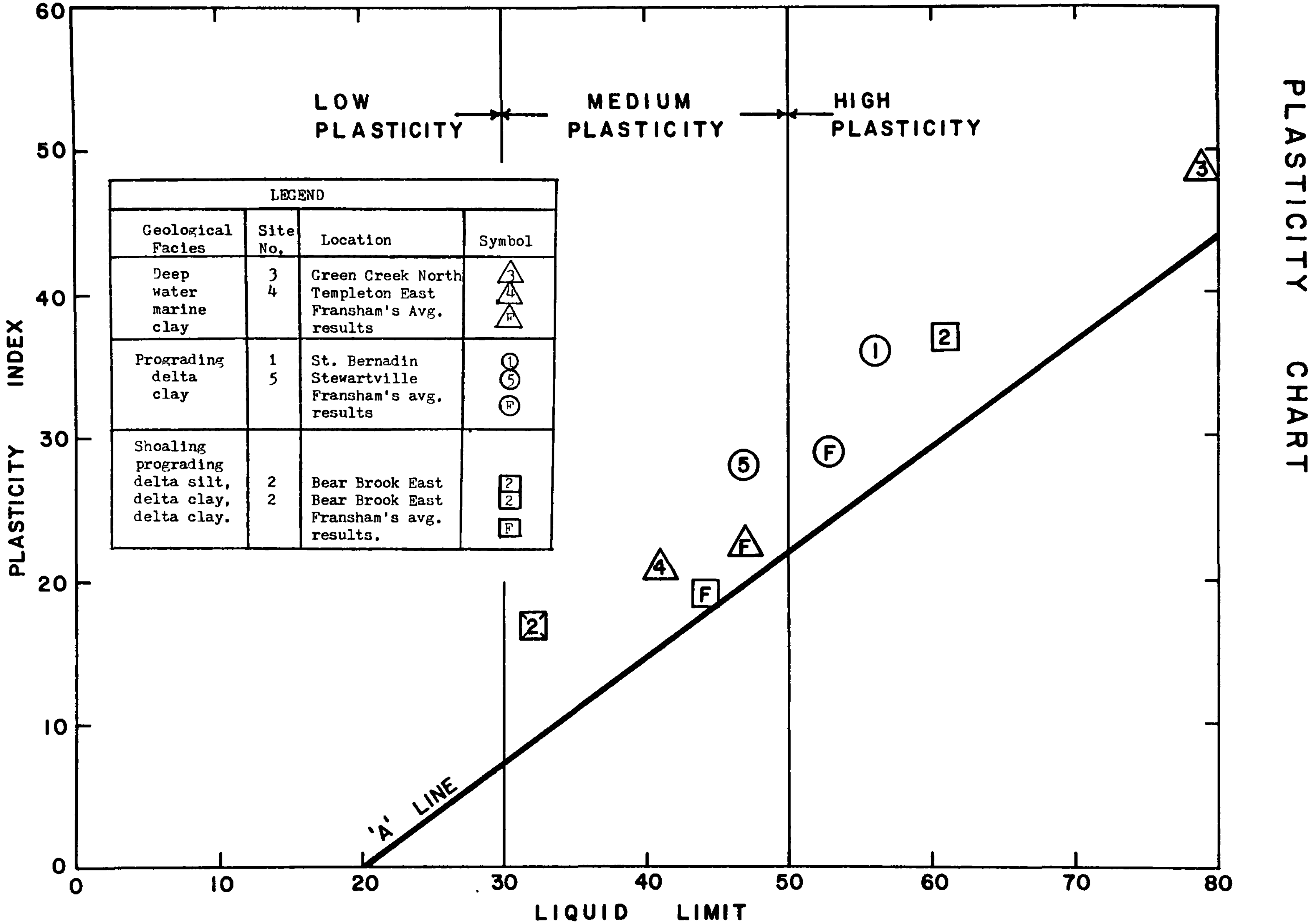


Figure 7.1.1 Plasticity chart

high plasticity, the prograding delta clay varies from medium to high plasticity and the shoaling prograding delta silt is in the lower range of the medium plasticity zone, while the shoaling prograding delta clay may be in the high plasticity zone.

Figure 7.1.2 is a plot of the grain size distribution of the material sampled at each site. Table 7.1.2 is a summary of the clay, silt and sand contents for each site.

Reviewing the clay contents for each site, one could say that both the deep water marine and the prograding delta clay have more or less the same range of clay size fraction, which is in accordance to what was reported by Fransham and Gadd (1977). The clay content for the shoaling prograding delta clay and silt varies over a very wide range depending upon which stratum is being sampled. This vertical anisotropy in grain size reflects a great seasonal variation in turbulence and material supply.

In summary, there exists a similar wide variation in the index properties of each geological facies of Leda clay. This is due to the effect of stratification in each facies. Because of this similar wide variation in index properties, there is no great difference between the clay facies from an engineering point of view. Attempts to classify each facies with average index properties are misleading. The only significant difference between the facies is that they become coarser the higher they are in the stratigraphic column. This reflects the general increase in the energy level in the environment of deposition.

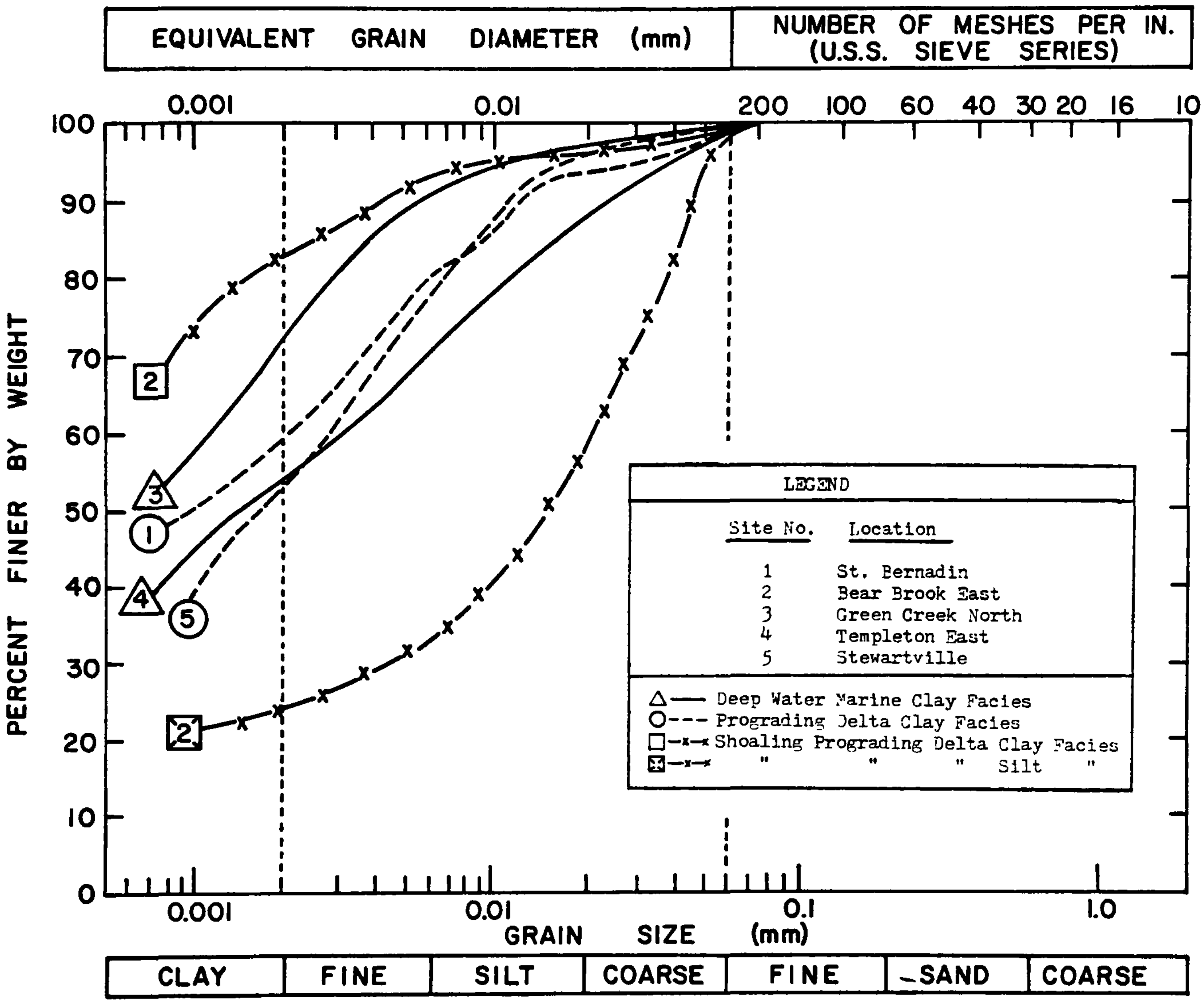


Figure 7.1.2 Grain size distributions

Site No.	Site Location	Geological Facies	Clay Size (%)	Silt Size (%)	Sand Size (%)	Remarks
1	St. Bernadin	Prograding Delta	61	38	1	
2	Bear Brook East	Shoaling Prograding Delta Silt	23.5	75.5	1	Sand fraction predominantly micaceous
2	Bear Brook East	Shoaling Prograding Delta Clay	84	15.5	0.5	Sand fraction predominantly quartz
3	Green Creek North	Deep water Marine clay	74	25.5	0.5	
4	Templeton East	Deep water Marine clay*	55	44	1	
5	Stewartville	Prograding Delta clay	55	45	1	

*Templeton site classified as deep water marine clay facies since it was quite fossiliferous, although % of clay size fraction is close to that of the prograding delta clay facies.

TABLE 7.1.2 Summary of Grain Size Distribution Data

7.2 Oedometer Test Results

Oedometer or consolidation tests were performed on samples from each site according to the procedure described in Chapter 6. Although the samples were left submerged overnight (16 hours) without load, no swelling was observed. Time-settlement curves under specific loads were typical of those reported in the literature for Leda clay (Crawford, 1964). The final test results are presented in the form of void ratio (e) versus log pressure curves on Figure 7.2.1.

All the curves indicate well defined breaks at what is termed the 'apparent' preconsolidation pressure; the maximum past overburden pressure to which the soil is thought to have been exposed. This break in the curve is termed 'apparent' because it is known to be affected by such factors as weathering, leaching, cementation, and fluctuating groundwater conditions and may not truly represent consolidation under previous overburden. It does represent, however, the point at which significant structural breakdown will occur under a certain imposed load. It can also be related empirically to the strength of the clay.

The probable preconsolidation pressures have been determined according to Casagrande's (1936) method and are presented in Table 7.2.1. In addition, the possible minimum and maximum values of the preconsolidation pressure, determined according to the method presented by Bozozuk (1971), are also presented.

Since the block samples were obtained from excavation into the face of natural slopes formed by streams and rivers, two values of effective overburden pressure (p_o') have been calculated and presented in Table 7.2.1.

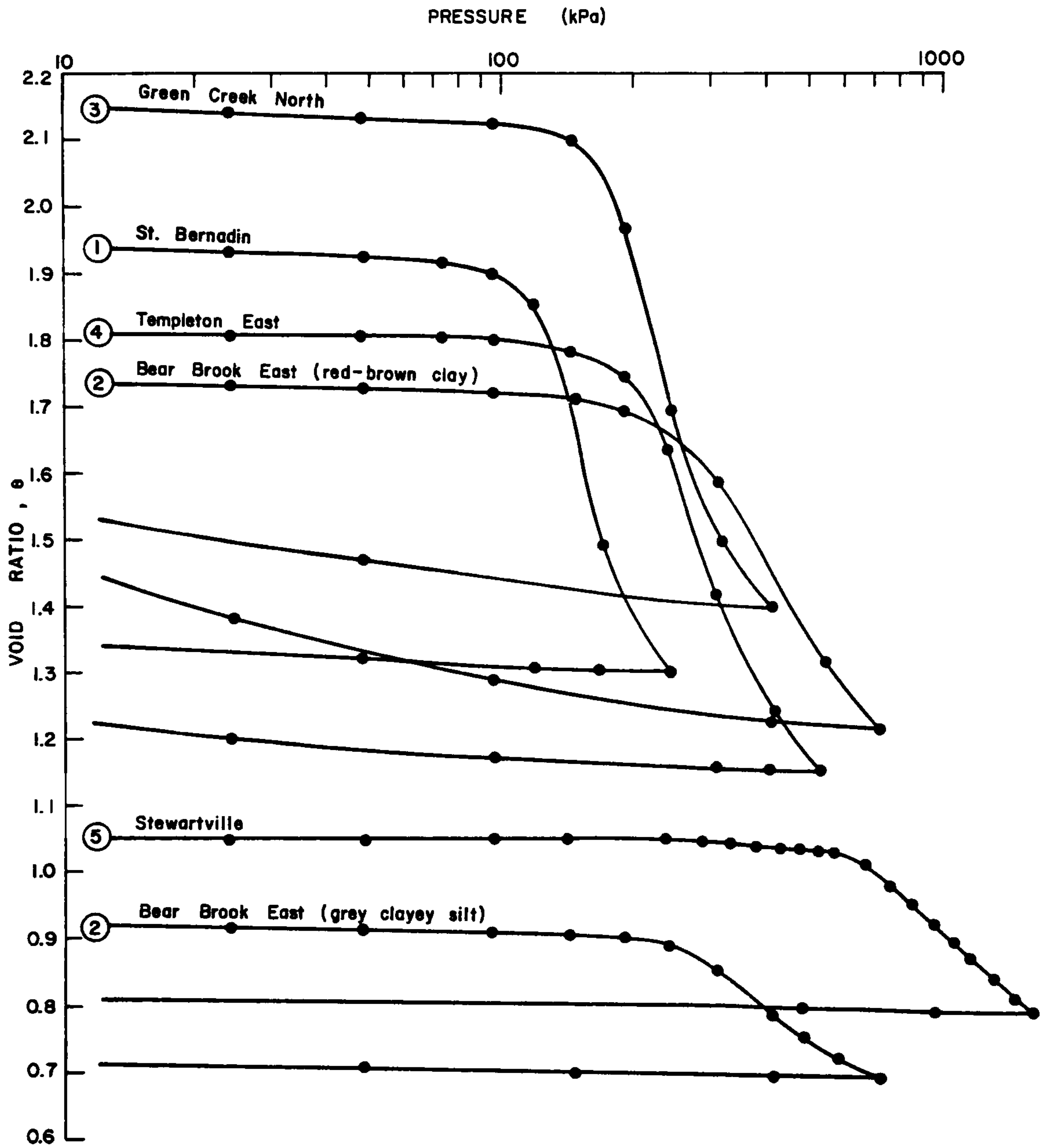


Figure 7.2.1 Oedometer test results

Site No.	Location	Geological Facies	Elevation of Block Samples m	p'_o (existing) (kPa)	p'_o (terrace) (kPa)	p'_c (min) (kPa)	p'_c (probable) (kPa)	p'_c (max) (kPa)
1	St. Bernadin	Prograding Delta clay	47.2	3.6	36	115	116	117
2	Bear Brook East	Shoaling Prograding Delta Silt	48.8	8.3	34.4	235	275	288
2	Bear Brook East	Shoaling Prograding Delta clay	48.8	8.3	34.4	225	265	282
3	Green Creek North	Deep water Marine clay	47.2	25	25	179	182	185
4	Templeton East	Deep water Marine clay	80.5	7	72.4	210	213	218
5	Stewartville	Prograding Delta clay	100.0	3.8	624	620	625	635

TABLE 7.2.1 Summary of Oedometer Test Results

These are:

a) The effective overburden pressure due to the depth of material existing on top of the block at the time samples were taken. These values are consequently very small.

b) The effective overburden pressure that would exist at the location of the block samples if the slope had not been cut by the river or stream. This value was calculated with reference to the existing terrace level at each site. The elevations of these terrace levels were indicated for each site in Chapter 5.

If the value of p_o', calculated from the terrace level, does not correspond with the measured preconsolidation pressures, then the terrace may be an erosional surface. Hence, the area was either covered with more soil at one time, or other special conditions are acting to explain the higher preconsolidation pressure.

Crawford and Eden (1965) found a correlation between elevation and preconsolidation pressure for the clay deposits they studied, situated near the Ottawa area. The limits of their data are illustrated in Figure 7.2.2. Also plotted on this figure are the preconsolidation values determined for the five sites studied in this thesis, as well as the results of three sites studied by Toombs (1974). As can be seen from this figure, many points lie outside the limits of data presented by Crawford and Eden.

This indicates that the relationship between preconsolidation pressure and elevation may be valid for only the limited geographical area studied by Crawford and Eden. Outside of this area, local factors

PRECONSOLIDATION PRESSURE AND ELEVATION

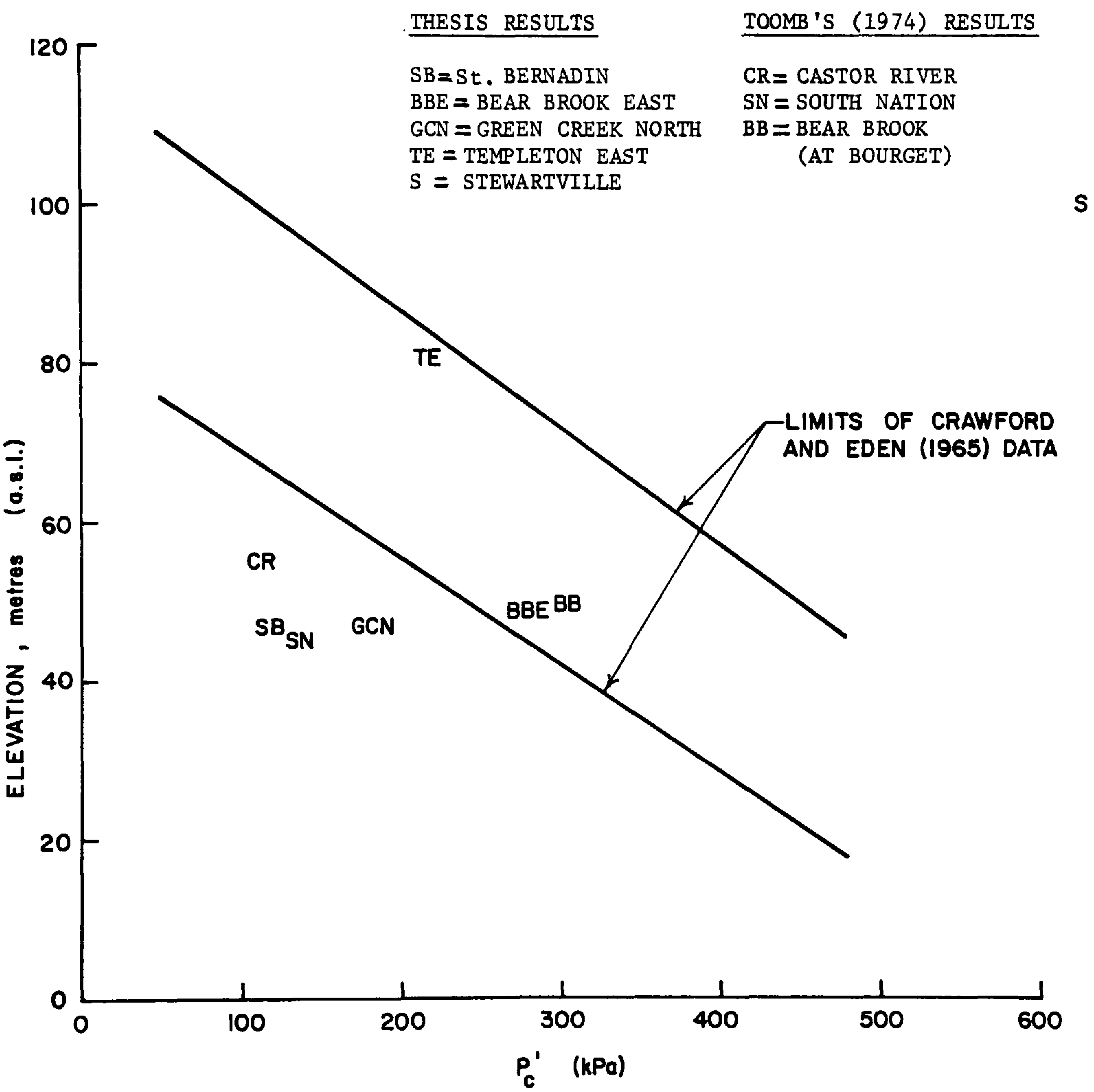


Figure 7.2.2

such as downward groundwater flow, weathering or leaching, cementation, greater deposition of post-marine sediments in certain areas and also the presence of high and low spots in the Champlain Sea basin, may have obscurred any correlation between elevation and preconsolidation pressure.

The very high preconsolidation pressure determined for the clay from Stewartville is the result of exceptional geologic circumstances. As discussed in Chapter 5, it is believed that as conditions in the Madawaska River valley changed from marine to fluvial, extensive deposits of glacial till deposited over the high bedrock to the north and south of the site were eroded and redeposited over the clay. A 48 m thick stratum of sand and gravel and small boulders was deposited and heavily loaded the underlying clay. The maximum loading condition on the clay probably occurred at the final deposition of the sand and gravel stratum. The water level in the sand and gravel stratum at this time can be considered to be at the top of the stratum, as this material must have been deposited under water. The clay was then progressively unloaded due to the cutting of the Madawaska River valley. If one considers the 48 m sand and gravel stratum to have a saturated density of 22 kN/m^3 (145 pcf), the computation of the effective overburden pressure would yield a value of 624 kPa which corresponds precisely with what was measured in the oedometer test. Such a high preconsolidation pressure is not considered representative of the prograding delta clay facies, but rather reflects exceptional geologic circumstances.

A surprisingly high preconsolidation pressure of 270 kPa was also recorded for the shoaling prograding delta silt and clay of Site No. 2. This is the uppermost of the Leda clay facies and should therefore exhibit the lowest amount of overconsolidation. However, as stated in Chapter 5, this site along Bear Brook, near Ettyville, lies in a small bay of an abandoned river channel. This area was originally overlain by a stratum of fine fluvial deltaic sand deposited by the ancestral Ottawa River and subsequently eroded when the Ottawa River started to incise channels into the sedimentary deposits. Uneroded deposits of the fine fluvial deltaic sand exist in the vicinity of this site, and by calculating the effective overburden pressure that would be exerted at the depth of block sampling, one can explain the high preconsolidation pressure determined by the oedometer tests at this site.

There exists at many locations around Ottawa a strong downward flow of groundwater through the Leda clay deposits to a more permeable sandy till or fractured bedrock (Jarrett and Eden, 1970; Fransham, 1978). This downward gradient causes an increase in effective stress on the clay and often results in higher than expected values of preconsolidation pressure, if hydrostatic conditions were assumed. A review of the bedrock and groundwater conditions in this area by Charron (1975) indicates however, that downward flow does not exist at Site No. 2. Therefore, the previous argument appears to be the most reasonable to explain the higher than expected value of preconsolidation pressure at this site.

It is of interest to note that oedometer tests were performed on samples from both the reddish-brown silty clay and the grey clayey silt from this site. Both tests yielded virtually the same value of preconsolidation pressure even though there was a great difference in void ratio (0.8) and water content (33%).

Considering the elevations at Sites No. 3 and 4, one would expect that the deep water marine clay at the Green Creek North site should exhibit a much higher preconsolidation pressure than the deep water marine clay at the Templeton East site, which is 33.3 m higher than the Green Creek North site. However, such is not the case. In fact, the Green Creek North clay exhibits a slightly lower preconsolidation pressure. This indicates the possibility of some unusual conditions at the Templeton East site that have caused a greater than expected value of preconsolidation at such a high elevation. Possible explanations for this occurrence are the following:

a) the clay may be highly cemented as indicated by its very brittle nature and high sensitivity,

b) a slightly greater thickness of sediments may have been deposited over the marine clay at the Templeton East site but at a significantly higher elevation than at the Green Creek North site. This is plausible since approximately the same thickness of sediments should be deposited at most locations in the basin of the Champlain Sea provided they are submerged. These sediments may have been deposited at a local high spot in the area of the Templeton East site. The high elevation of the clay at this site is probably due to the close proximity of the bedrock which outcrops less than 1 km to the north of the site.

In summary, the geological facies of Leda clay are not restricted to particular elevations. Deposits of the same facies may be found at significantly different elevations. Preconsolidation pressures can generally be explained as due to previous effective overburden pressures but may occasionally be the result of possible cementation or downward hydraulic gradients. Preconsolidation pressures may be related to elevation over a small area, but this is not true over a large area such as that studied in this thesis.

7.3 Laboratory Vane Shear Tests

Stress controlled laboratory vane shear tests described in Chapter 6 were performed on block samples from Sites 1, 4 and 5. It was not possible to perform lab vanes on material from Sites 2 and 3 due to the shortage of adequate samples. The results of these tests are summarized in Table 7.3.1 together with the results of field and hand vane shear tests performed in-situ at the depth of block sampling. In addition, Table 7.3.1 includes the results of unconfined compression tests and CIU triaxial tests carried out at very low (0.7 kPa) confining pressures. These tests will be discussed in the next section.

After block samples were obtained at each site, hand vane shear tests were performed in the face and bench of the excavation to provide a further check on the vane shear strength of the clay. Radial cracking was observed during insertion of the hand vane, or at very small angular rotations, at the Templeton East and Stewartville sites. The clay at both of these sites is very brittle and it is believed that the disturbance due to the insertion of the hand vane caused the clay to crack radially. No such radial cracking was observed during the laboratory vane shear tests. It is believed that the thin laboratory vane blades did not disturb the clay. Hence, the laboratory vane shear tests are considered to provide a reliable measure of the undrained shear strength.

Site No.	Site Location	Sensitivity Range[1]	Field Vane Shear Strength (kPa)	Hand Vane Shear Strength (kPa)	Lab Vane Shear Strength (kPa)	Unconfined[2] Shear Strength (kPa)	CIU Shear[3] Strength (kPa)	Remarks
1	St. Bernadin	7-10	27.	27.	30.6	27.5	26.9	
2	Bear Brook East (red-brown clay only)	8-12	58.5*	65.	-	71.0	21.9**	*Result of test in red-brown clay and grey silt **Low result due to premature failures in sample, caused by roots. Test at σ_3 = 40 kPa indicates strength of 69 kPa
3	Green Creek North	6-10	68	-	-	74	66.8	
4	Templeton East	20-132	32	42	56	77	53.5	
5	Stewartville	-	-[4]	-	164	184	193***	***High result due to sample variability Test at σ_3 = 50kPa indicates strength of 182 kPa.

1. Sensitivity was measured by the field vane.
2. Unconfined compression tests were generally performed on 3.8 cm by 7.6 cm samples at a strain rate of 1% per min.
3. CIU triaxial tests were generally performed on 5.0 cm by 10.0 cm samples at a strain rate of 0.03% per min.
4. No field vane tests performed at Stewartville site as clay was too hard to penetrate with the vane.

TABLE 7.3.1 Summary of Undrained Test Results

Plots of shear strength versus angular deformation for the lab vane tests are presented in Figures A1 to A3 of Appendix A. It is important to note that peak shearing resistance was mobilized at low angular deformation, generally 10° or less. After failure the vane blades rotated rapidly to 90° indicating that a complete cylindrical failure surface had developed. Tests were performed at both vertical and horizontal orientations. No significant difference in shear strength was noted for any site.

7.4 Unconfined Compression Tests

Strain controlled, unconfined compression tests were performed on samples cut from blocks for all sites. The procedure followed and equipment used for these tests has been described in Chapter 6. A minimum of two tests were performed at each site. The average shear strengths determined from the unconfined compression tests are summarized in Table 7.3.1. A detailed summary of all the test results including water contents, densities, strain at failure, mode of failure and remarks is presented in Table A-1 of Appendix A. Plots of shear stress versus strain for each site are presented on Figures A4 to A8 in Appendix A. All tests were performed on vertically oriented samples.

Unconfined compression tests on the grey clayey silt of Site No. 2, Bear Brook East, yielded shear strengths similar to the reddish-brown silty clay. Because of the similarity between the strength and the preconsolidation pressures of the two materials, it was decided to restrict further tests to the reddish-brown silty clay only.

The effect of sample size and rate of strain on the shear strength determined from the unconfined compression test was studied on samples of grey, very sensitive, silty clay from Site No. 4, Templeton East. The reader is referred to Table A1 in Appendix A for complete details. In brief, it appears that sample size and strain rate do affect the measured shear strength at this site. Samples 3.8 cm in diameter and 7.5 cm in height yielded a 14% increase in strength when compared to samples with dimensions of 5.0 cm by 10.0 cm. The larger samples are probably influenced by structural imperfections which would reduce their strength. A five-fold decrease in strain rate from the standard unconfined compression strain rate of 1% per minute to 0.2% per minute did not affect the measured shear strength. However, a thirty-three-fold reduction in strain rate to 0.03% per minute (1.8% per hour) reduced the measured shear strength by 10%. This agrees approximately with Bjerrum's (1971) estimation of 10 ± 5% reduction in strength per log cycle of strain rate for sensitive and plastic clays. Therefore, sample size and strain rate significantly affect the measured shear strength at this site.

The results of CIU triaxial tests performed on 5.0 by 10.0 cm samples from all sites at a rate of strain of 0.03% per minute (1.8% per hour) with a virtually negligible cell pressure of 0.7 kPa are also presented in Table 7.3.1.

Sample size and strain rate are not believed to significantly affect the measured shear strengths at Sites 1, 3 and 5 since relatively good agreement was obtained between the low confining pressure, CIU triaxial tests, and the standard unconfined compression tests performed on 3.8 cm by 7.6 cm samples at a rate of strain of 1.0% per minute.

At Site No. 2, good agreement was reached between the vane shear tests and the unconfined compression test. However, the CIU triaxial test yielded a much lower result. Strain rate and sample size may affect the measured shear strength at this site, however, it is not believed that they account for the large difference noted in the CIU test. It is believed that the presence of small roots in the center of this sample, causing a small fissure that was undetected during trimming, reduced the measured shear strength. It should be noted that another CIU test performed at a confining pressure of 40 kPa indicated a strength in accordance with the unconfined test result.

The poor agreement between most test results at Site No. 4, Templeton East, is the result of the very brittle, very sensitive nature of the clay deposit. The sensitivity as measured by the field vane at this site ranges from 20 to 132 which is considerably greater than the range of 6 to 12 recorded at the other sites. The plasticity index of this clay is also the lowest of all the sites.

The field vane test at this site yielded the lowest shear strength. This is the result of the disturbance caused during vane insertion into a brittle soil. The small hand vane causes less disturbance and yielded a slightly higher result. The smaller laboratory vane causes even less disturbance and yielded still higher strength. The lab vane result agrees well with the CIU test result and indicates that thin blade laboratory vane tests on block samples may provide the best estimation of the undrained shear strength of a clay, and yield the best correlation with the results of a series of CIU triaxial tests.

In summary, it appears that except for extra-sensitive, low plasticity, Leda clays, a reasonably good correlation exists between field, hand and laboratory vane tests, unconfined compression tests, and CIU triaxial tests at very low confining pressures.

The undrained shear strength of extrasensitive clays (sensitivity as measured by the field vane in excess of 10) should be verified by laboratory vane shear tests on high quality, large diameter tube or block samples. In addition, unconfined compression tests and CIU triaxial tests at very low confining pressures may be used to provide a further check on the undrained shear strength.

7.5 Isotropic Consolidation of Triaxial Test Samples

This section presents and discusses the results of isotropic consolidation of the triaxial test samples for all sites.

Before proceeding, it is first necessary to explain that a difficulty arises in the accurate measurement of volume changes during consolidation due to the procedure used in preparing the sample. Due to the desire to measure porewater pressures as accurately as possible and ensure full saturation of the sample, it is necessary to eliminate as much air as possible surrounding it. To accomplish this, water is passed through the bottom pedestal of the triaxial apparatus and up along the sides of the sample. Some of this water remains trapped between the membrane and the sample even though attempts are made to remove as much water as possible. Consolidation measurements of the volume of water extruded from the sample are therefore inaccurate since an unknown amount of water trapped next to the sample would be included. It is more accurate and reliable to compute the 'net' consolidation as the actual difference in weight before and after the test. Since the test is sheared undrained, this difference in weight represents the amount of water lost from the sample during the consolidation process.

Figures 7.5.1 to 7.5.5 are plots of net consolidation with time under various all round or isotropic consolidation pressures used in the triaxial tests for each site. As discussed in Chapter 6, the triaxial test procedure called for overnight saturation under a back pressure of 200 kPa. To balance this back pressure, it was necessary to apply an equal or slightly greater cell pressure. In general, an excess cell pressure or confining pressure of about 1-3 kPa was applied in an attempt to expel the excess water trapped between the sample and the membrane. Occasionally, the samples swelled slightly even against this confining pressure and this resulted in points above the 0.0% volume change line at a time equal to 0.1 minutes on the plots. The curve corresponding to a test performed at a pressure of 0.7 kPa represents swell that took place during the overnight saturation stage. These samples were sheared immediately after saturation with no consolidation. These curves are plotted merely to illustrate the amount of swell that took place.

Inspection of the consolidation with time plots indicates a general trend of increasing amount of consolidation with greater consolidation pressures, as would be expected. The amount of consolidation is quite small with a maximum of 2% recorded from the Green Creek North clay. More typically the maximum amount of consolidation was less than

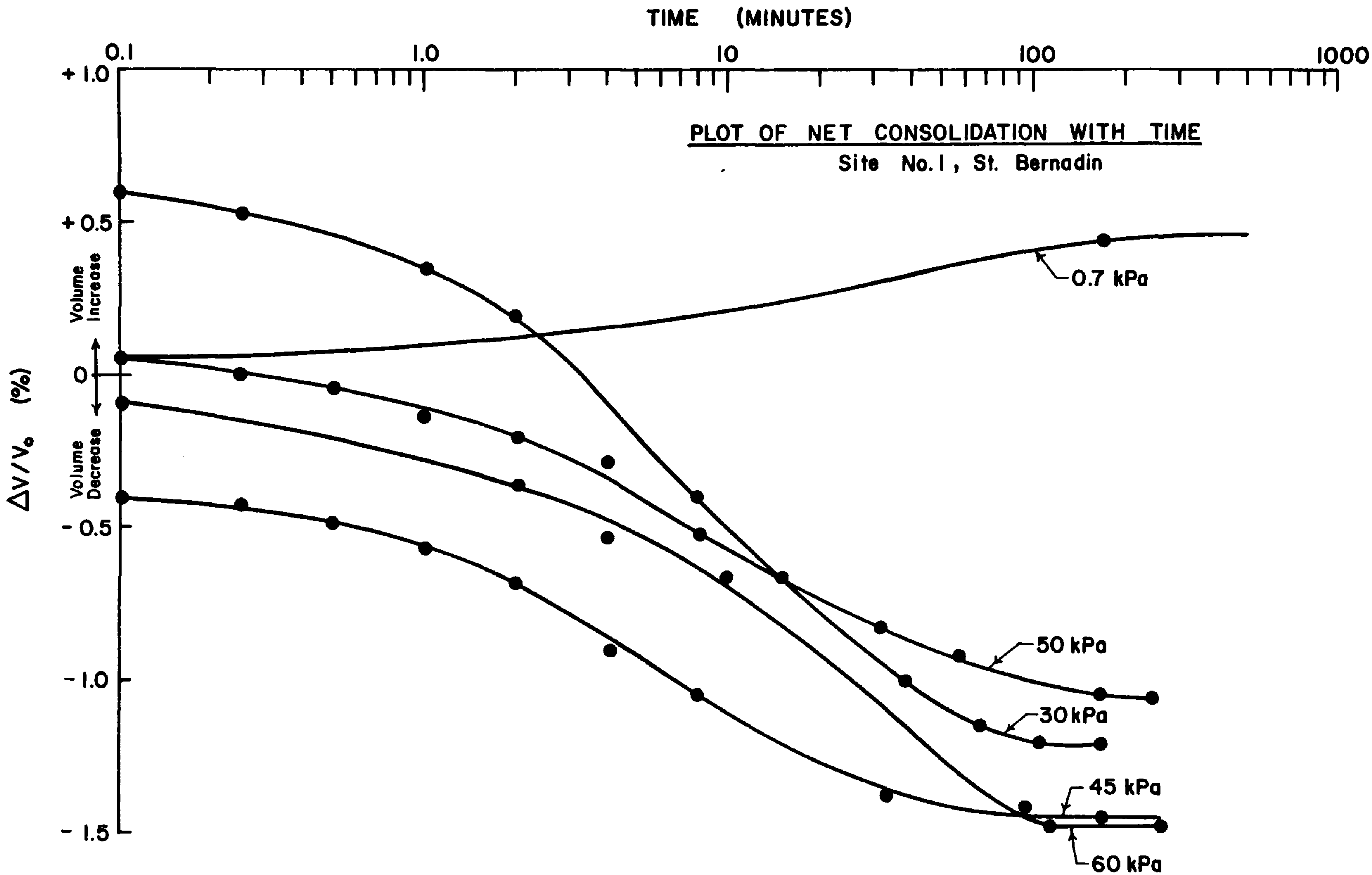

TIME (MINUTES)
0.1
1.0
10
100
1000
+ 1.0
+ 0.5
0
- 0.5
- 1.0
- 1.5
PLOT OF NET CONSOLIDATION WITH TIME
Site No. 1, St. Bernadin
Volume Increase
Volume Decrease
ΔV/V₀ (%)
0.7 kPa
50 kPa
30 kPa
45 kPa
60 kPa

Figure 7.5.1

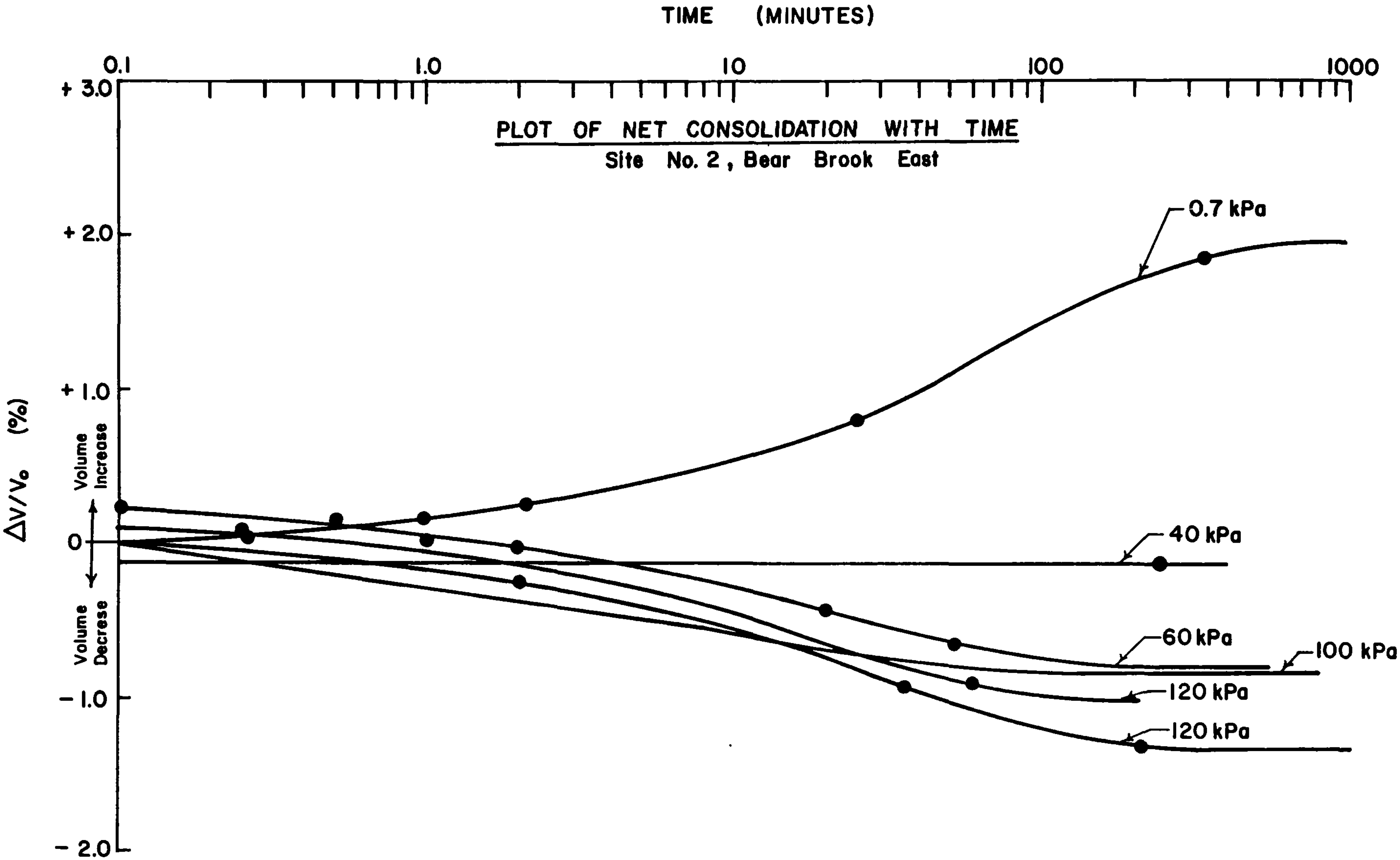

TIME (MINUTES)
0.1
1.0
10
100
1000
+ 3.0
+ 2.0
+ 1.0
0
− 1.0
− 2.0
PLOT OF NET CONSOLIDATION WITH TIME
Site No. 2, Bear Brook East
0.7 kPa
40 kPa
60 kPa
100 kPa
120 kPa
120 kPa
ΔV/Vo (%)
Volume Increase
Volume Decrease

Figure 7.5.2

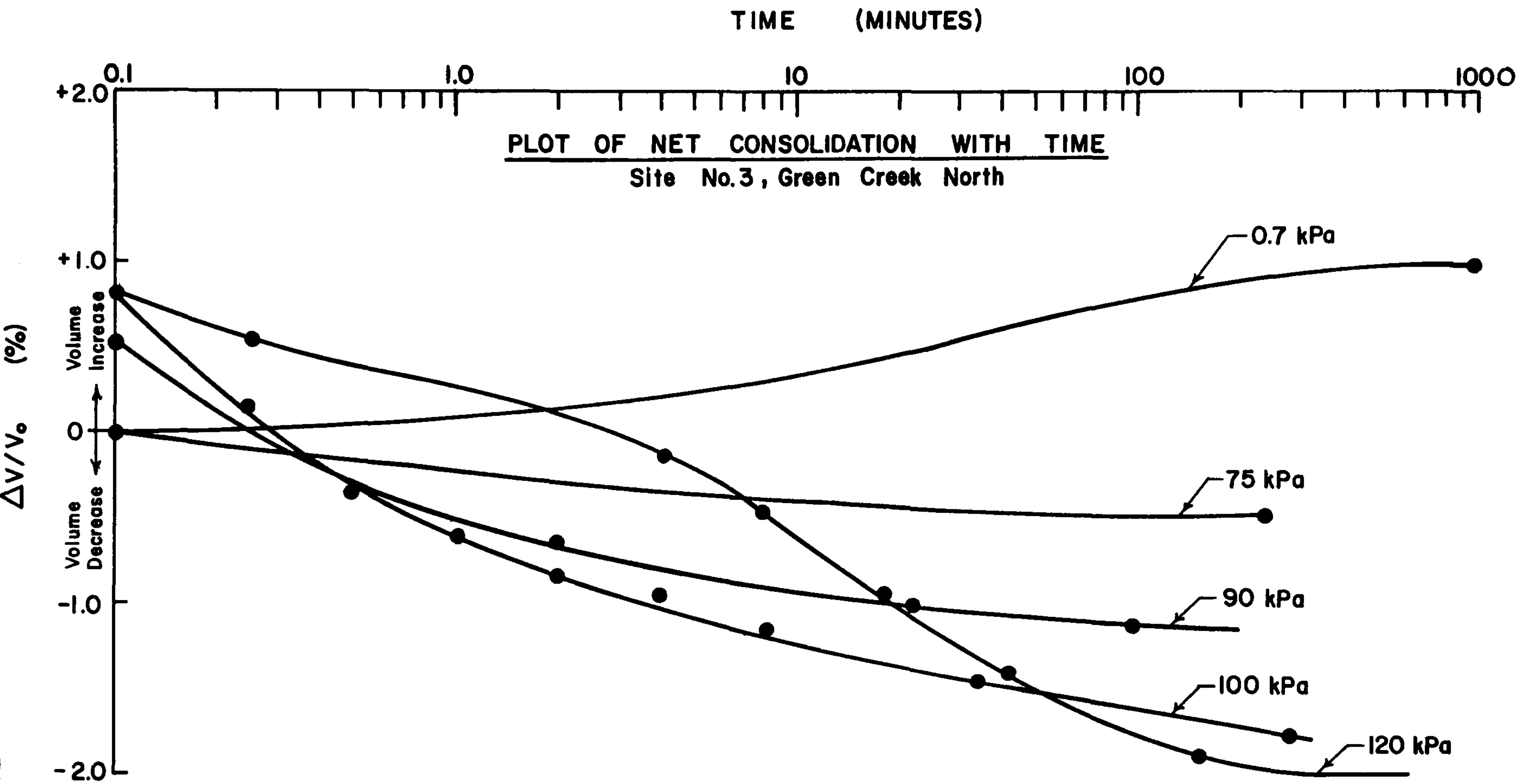

TIME (MINUTES)
0.1
1.0
10
100
1000
+2.0
+1.0
0
-1.0
-2.0
PLOT OF NET CONSOLIDATION WITH TIME
Site No.3, Green Creek North
$\Delta V / V_o$ (%)
Volume Increase
Volume Decrease
0.7 kPa
75 kPa
90 kPa
100 kPa
120 kPa

Figure 7.5.3

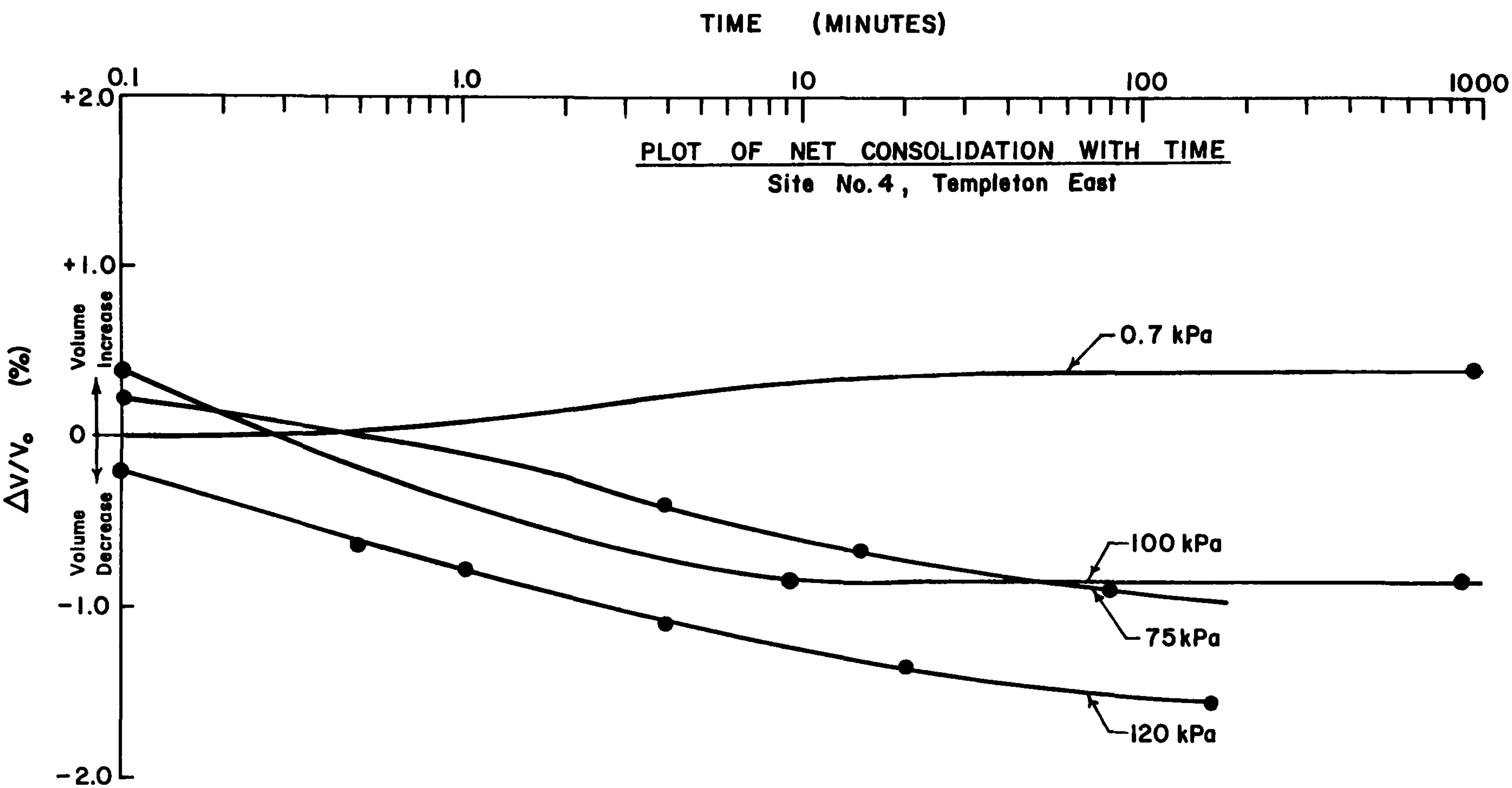

TIME (MINUTES)
0.1
1.0
10
100
1000
+2.0
+1.0
0
-1.0
-2.0
PLOT OF NET CONSOLIDATION WITH TIME
Site No. 4, Templeton East
ΔV/Vo (%)
Volume Increase
Volume Decrease
0.7 kPa
100 kPa
75 kPa
120 kPa

Figure 7.5.4

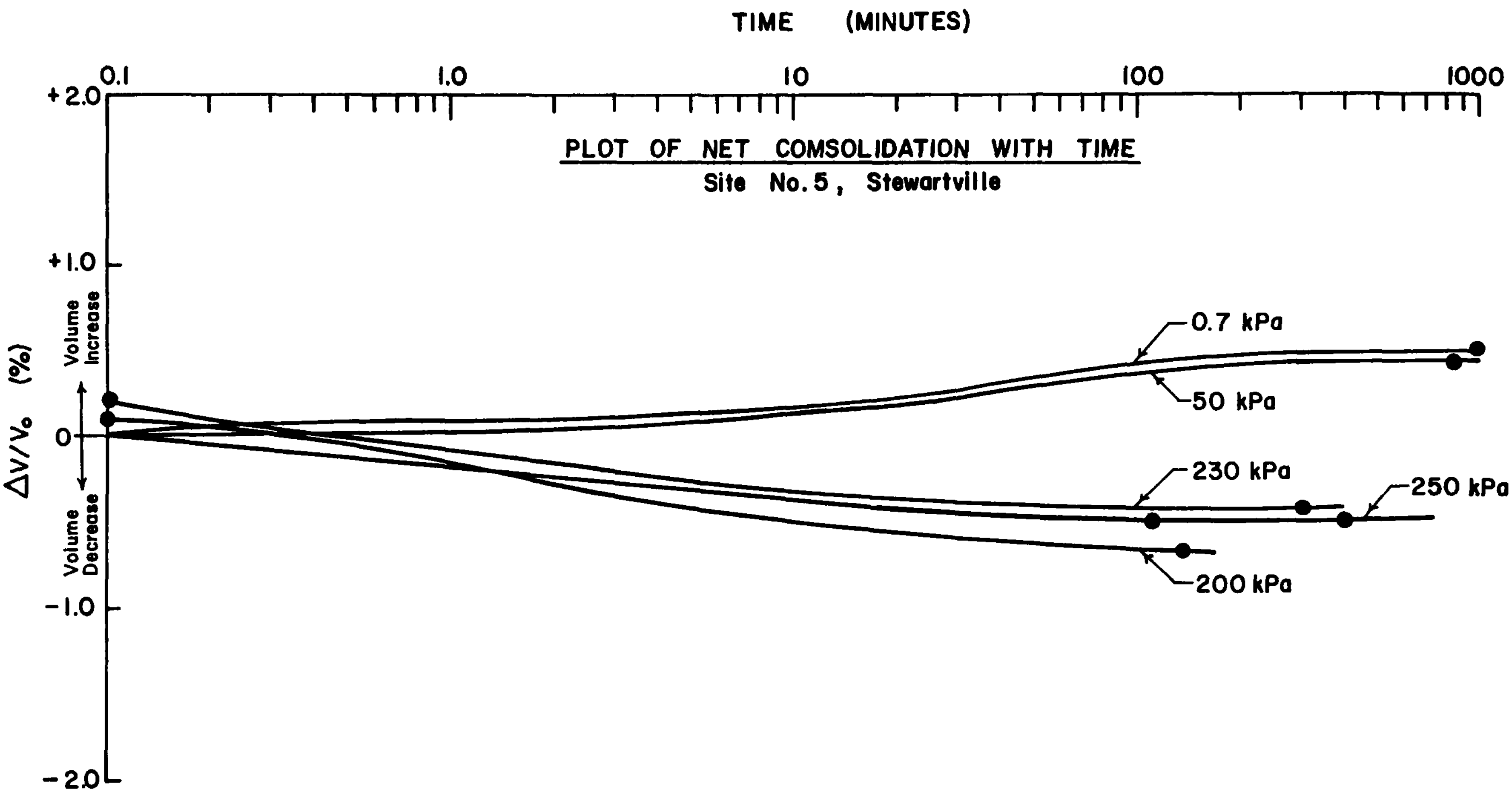

TIME (MINUTES)
0.1
1.0
10
100
1000
+2.0
+1.0
0
-1.0
-2.0
PLOT OF NET COMSOLIDATION WITH TIME
Site No. 5, Stewartville
$\Delta V/V_o$ (%)
Volume Increase
Volume Decrease
0.7 kPa
50 kPa
230 kPa
250 kPa
200 kPa

Figure 7.5.5

1.5% for most sites. This also was expected since all the tests were performed in the low effective normal stress range. Time to the end of consolidation generally ranged from 100 to 300 minutes, however one test of the Templeton East clay required 1400 minutes to complete consolidation.

Figure 7.5.6 summarizes the amount of net volume change versus isotropic consolidation pressure for each site. As can be seen from this figure, the data plots as reasonably smooth curves, although some scatter is evident. The curves appear to bend sharply downwards at a pressure equal to 50% of the preconsolidation value (indicated by a vertical arrow) for each site. It is suggested that a pressure of 50% of the preconsolidation value determined from the oedometer test represents a yield point under isotropic compression.

As far as isotropic swelling is concerned, it would be expected that the more overconsolidated a clay is, the more it would swell. However, this does not appear to be the case for the sites studied. In fact, the most heavily overconsolidated and the most lightly overconsolidated clays both swelled the same amount of 0.5%. A direct correlation does appear to exist however, between the magnitude of swell and the percent clay size fraction that was presented in Table 7.1.2. The higher the percent clay size fraction, the greater the swell.

In conclusion, since volume changes were essentially similar at all sites (generally less than 1.5% compression) it may be said that the consolidation pressures used were all lower than that which would cause any structural breakdown. Therefore, the proposed upper limit of

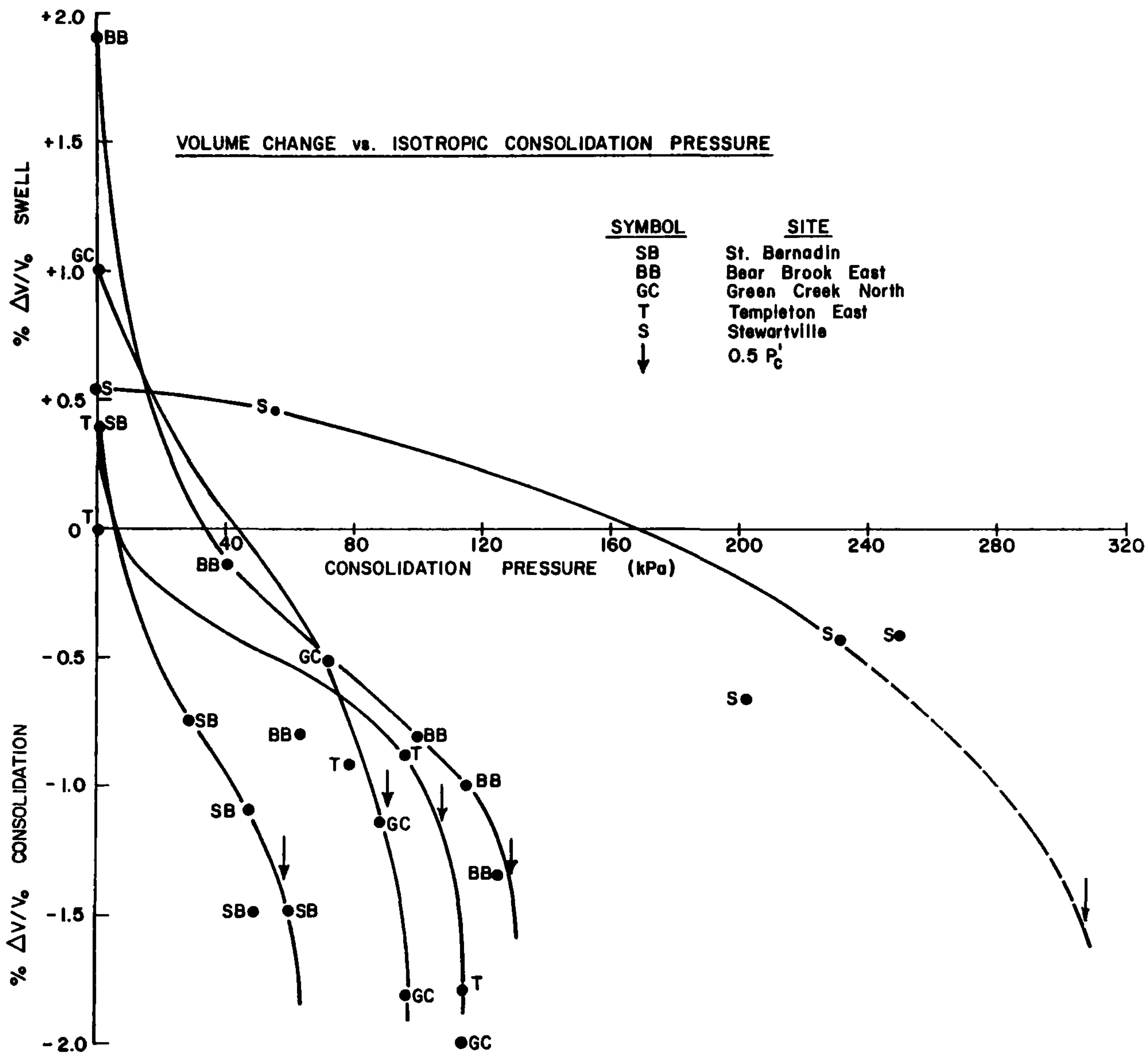

VOLUME CHANGE vs. ISOTROPIC CONSOLIDATION PRESSURE
SYMBOL SITE
SB St. Bernadin
BB Bear Brook East
GC Green Creek North
T Templeton East
S Stewartville
0.5 P_c'
% $\Delta V/V_o$ SWELL
% $\Delta V/V_o$ CONSOLIDATION
CONSOLIDATION PRESSURE (kPa)
+2.0
+1.5
+1.0
+0.5
0
-0.5
-1.0
-1.5
-2.0
40
80
120
160
200
240
280
320

Figure 7.5.6

consolidation pressures at 0.5 p'_c for a triaxial testing programme for the purpose of slope stability analyses, would seem quite satisfactory for Leda clays since no significant structural breakdown would occur.

Furthermore, it is suggested that a maximum volume compression of 1.5% during isotropic consolidation could be used as a guideline for further testing programmes. Any volume compression greater than 1.5% would indicate either excessive confining pressure or sample disturbance.

7.6 Shear Stress-Strain Behaviour

After isotropic consolidation to a specified pressure, the samples were sheared undrained at a rate of 1.8% per hour until a minimum strain of 6% was reached.

Figures 7.6.1 to 7.6.5 are the stress-strain curves so determined for all sites. Stress has been corrected for change in cross-sectional area and the effect of rubber membranes and filter paper drains according to the procedure described in Chapter 6. A complete summary of the CIU test data including: water contents, densities, void ratio, degree of saturation, peak and residual values of porewater pressures, strain and total and effective values of p and q, as well as mode of failure, is presented in Tables A-2 to A-6 in Appendix A.

As can be seen from Figures 7.6.1 to 7.6.5, the values of shear stress, q, peak at low strains of approximately 1% for most sites. This indicates the good quality of the block samples. After the peak there is generally a substantial reduction in shear stress with increasing strain until a residual value is reached close to 6% strain.

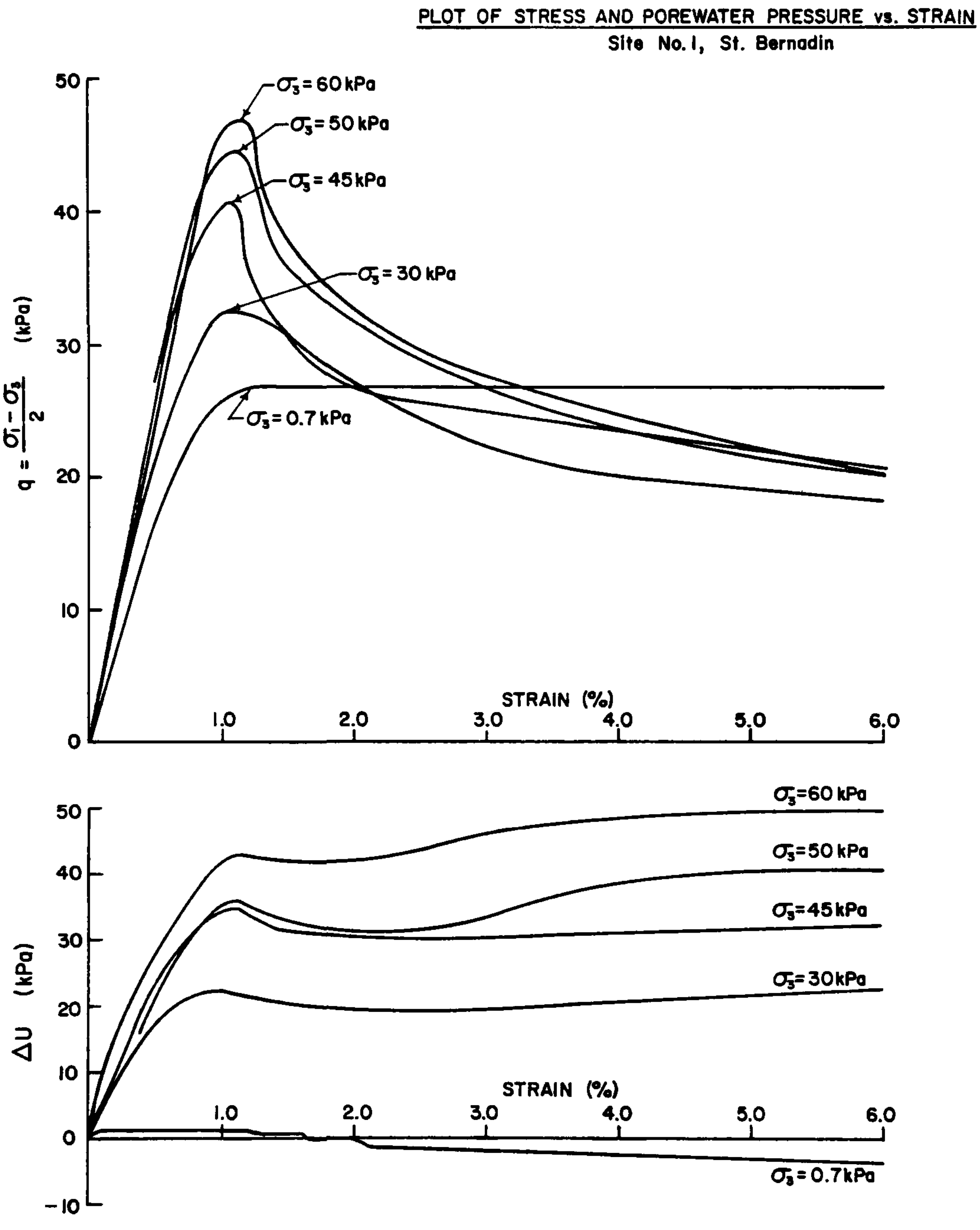

PLOT OF STRESS AND POREWATER PRESSURE vs. STRAIN
Site No. I, St. Bernadin
σ3 = 60 kPa
σ3 = 50 kPa
σ3 = 45 kPa
σ3 = 30 kPa
σ3 = 0.7 kPa
q = (σ1 − σ3)/2 (kPa)
STRAIN (%)
ΔU (kPa)

Figure 7.6.1

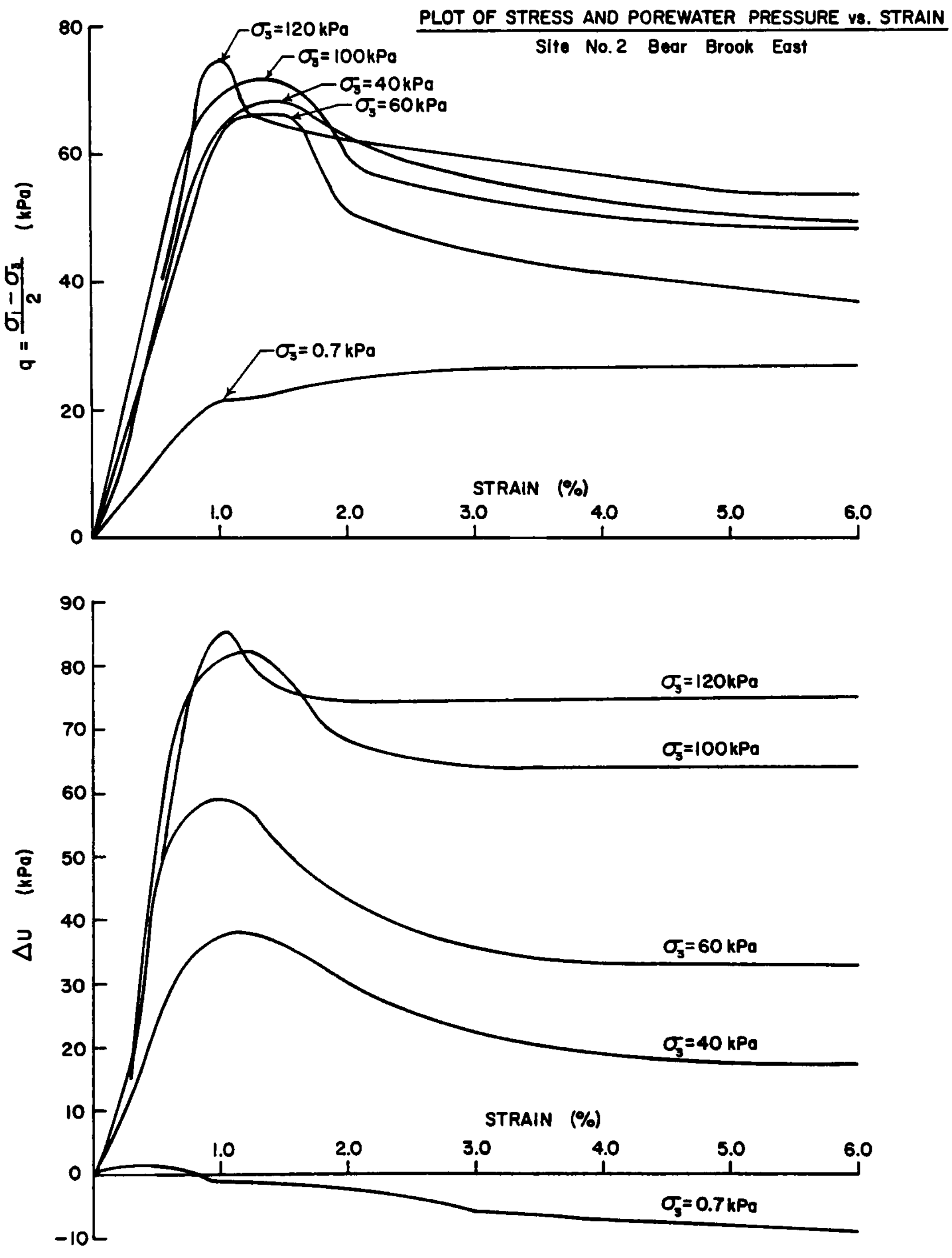

PLOT OF STRESS AND POREWATER PRESSURE vs. STRAIN
Site No. 2 Bear Brook East
σ3 = 120 kPa
σ3 = 100kPa
σ3 = 40 kPa
σ3 = 60 kPa
σ3 = 0.7 kPa
q = (σ1 − σ3)/2 (kPa)
STRAIN (%)
80
60
40
20
0
1.0
2.0
3.0
4.0
5.0
6.0
ΔU (kPa)
σ3 = 120kPa
σ3 = 100kPa
σ3 = 60 kPa
σ3 = 40 kPa
σ3 = 0.7 kPa
90
80
70
60
50
40
30
20
10
0
−10

Figure 7.6.2

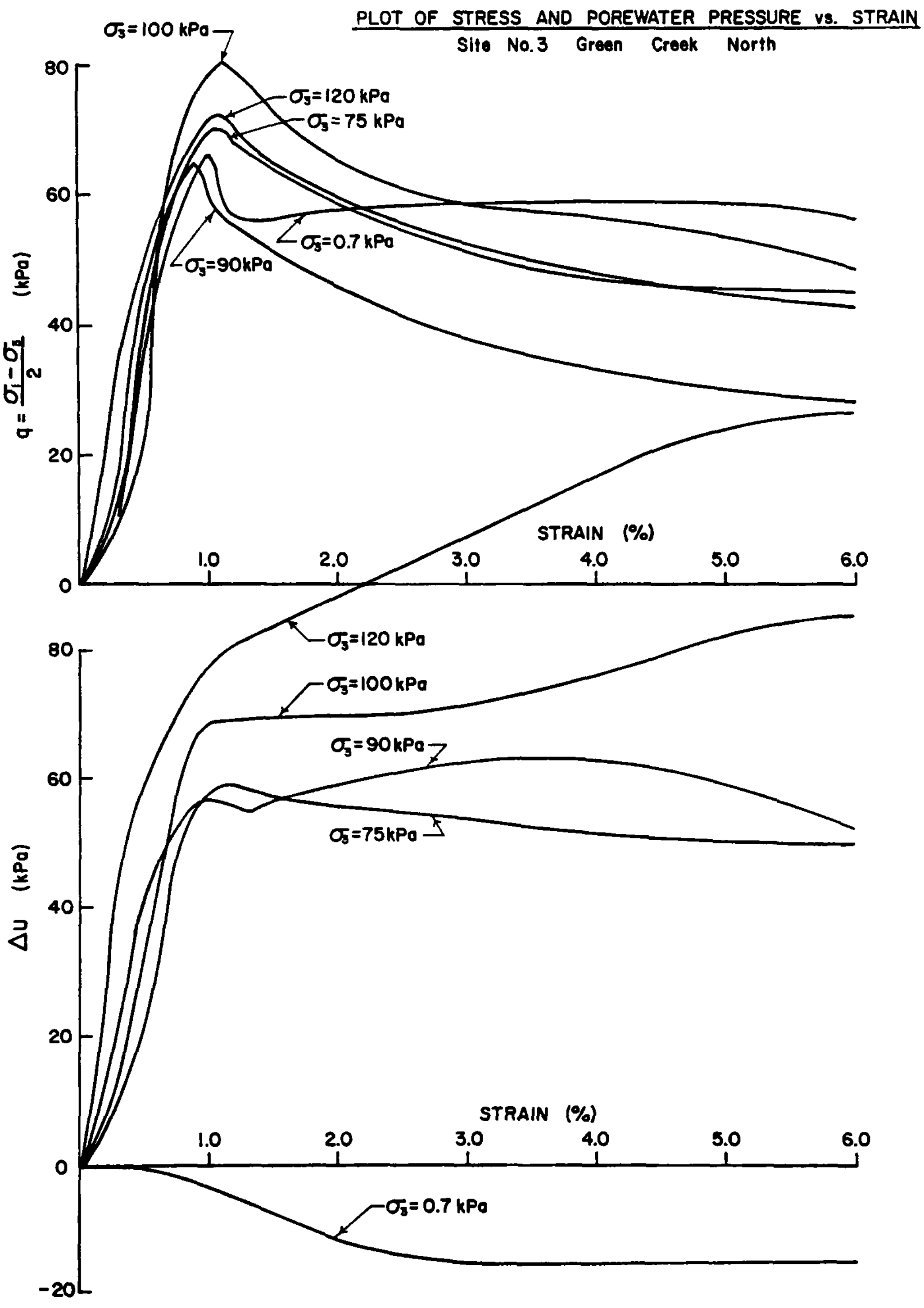

PLOT OF STRESS AND POREWATER PRESSURE vs. STRAIN
Site No. 3 Green Creek North
σ3 = 100 kPa
σ3 = 120 kPa
σ3 = 75 kPa
σ3 = 0.7 kPa
σ3 = 90 kPa
q = (σ1 − σ3)/2 (kPa)
STRAIN (%)
σ3 = 120 kPa
σ3 = 100 kPa
σ3 = 90 kPa
σ3 = 75 kPa
ΔU (kPa)
STRAIN (%)
σ3 = 0.7 kPa

Figure 7.6.3

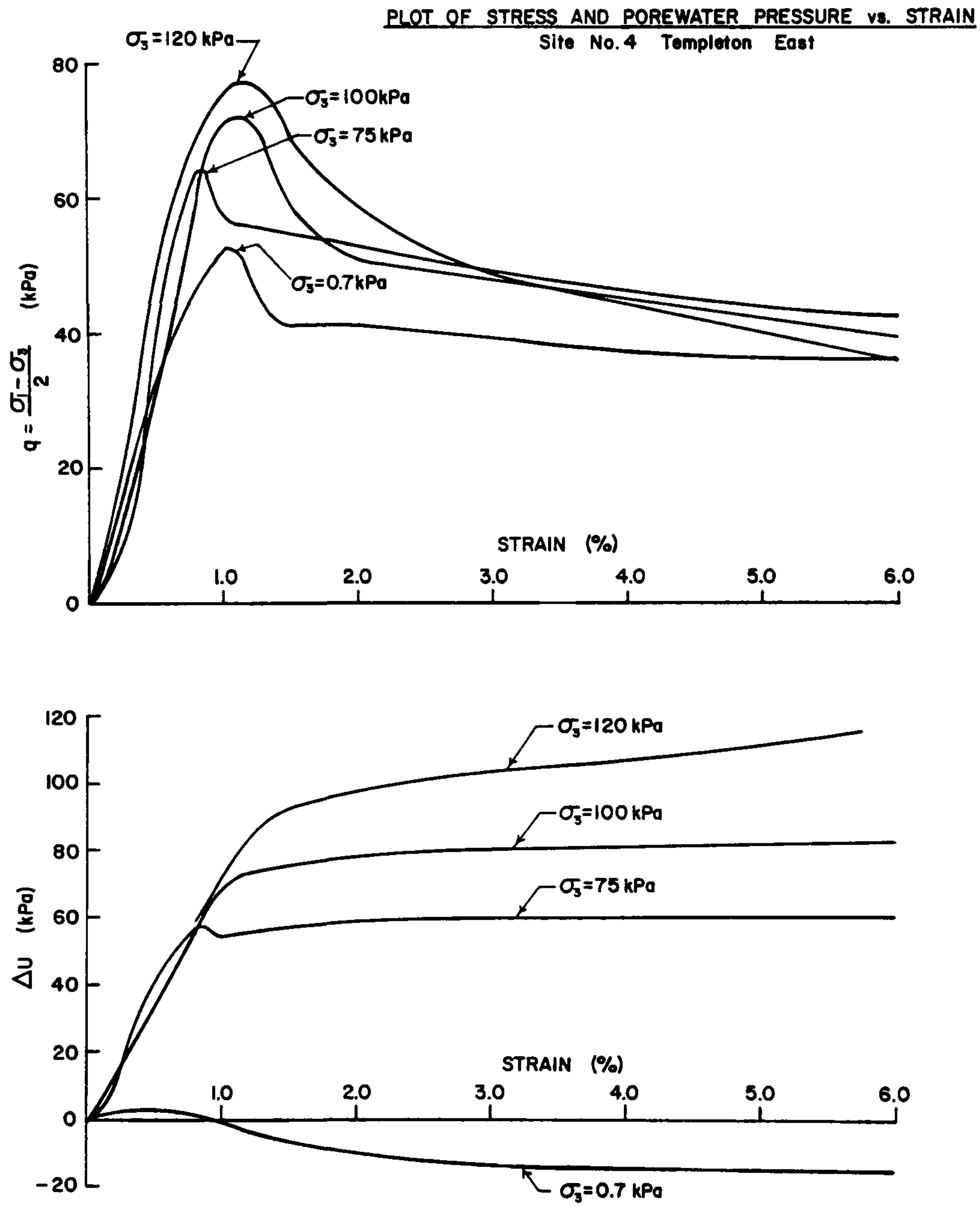

PLOT OF STRESS AND POREWATER PRESSURE vs. STRAIN
Site No. 4 Templeton East
σ_3=120 kPa
σ_3=100kPa
σ_3=75 kPa
σ_3=0.7kPa
$q = \frac{\sigma_1 - \sigma_3}{2}$ (kPa)
80
60
40
20
0
STRAIN (%)
1.0
2.0
3.0
4.0
5.0
6.0
σ_3=120 kPa
σ_3=100 kPa
σ_3=75 kPa
σ_3=0.7 kPa
ΔU (kPa)
120
100
80
60
40
20
0
-20
STRAIN (%)
1.0
2.0
3.0
4.0
5.0
6.0

Figure 7.6.4

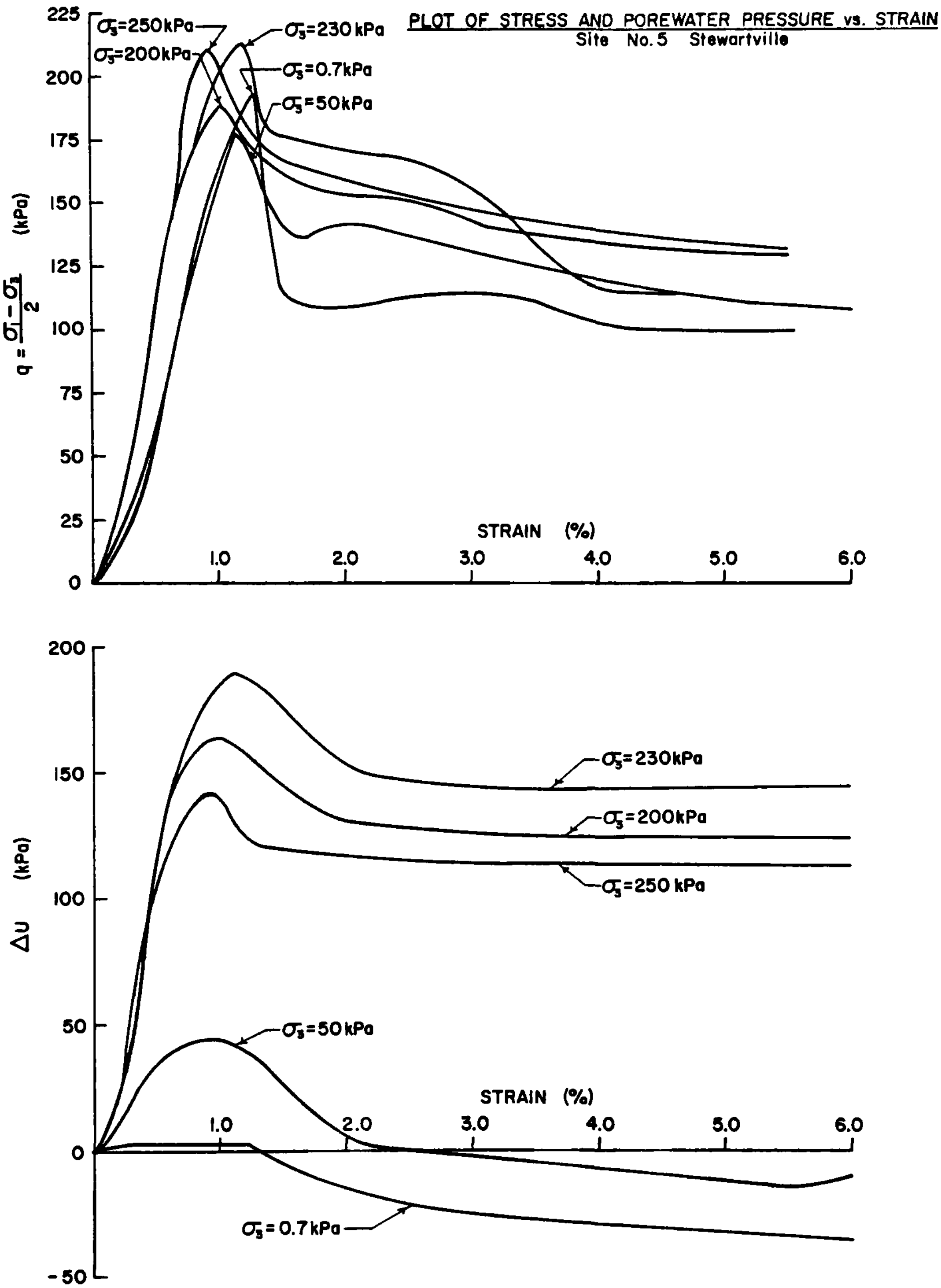

PLOT OF STRESS AND POREWATER PRESSURE vs. STRAIN
Site No. 5 Stewartville
σ_3=250kPa
σ_3=200kPa
σ_3=230 kPa
σ_3=0.7kPa
σ_3=50kPa
$q = \frac{\sigma_1 - \sigma_3}{2}$ (kPa)
STRAIN (%)
0
25
50
75
100
125
150
175
200
225
1.0
2.0
3.0
4.0
5.0
6.0
Δu (kPa)
σ_3=230kPa
σ_3=200kPa
σ_3=250 kPa
σ_3=50kPa
σ_3= 0.7kPa
-50
0
50
100
150
200

Figure 7.6.5

Many samples, but not all, were sheared to strains of 8 and 10%. These results indicated that 6% strain was sufficient to define the residual strength for the majority of tests. In brief, the stress-strain behaviour of these clays could be defined as brittle, strain softening.

There is a general increase in shear stress with increasing confining pressure as would be expected. However, there does appear to be some scatter in this regard, particularly for the Green Creek North and Stewartville clays, illustrated in Figures 7.6.3 and 7.6.5, respectively. This can be explained simply as due to sample variability. Although one would expect a uniform strength to exist for samples cut from the same, apparently homogeneous block, this may not necessarily be the case. A study of Orleans clay reported by Yong (1976), indicated a variation of unconfined compressive strength for samples cut from the same block in excess of ±20% compared to the average value. The water contents however, appeared quite constant. It is interesting to note that although wide variation was reported in the unconfined compressive strengths, they were normally distributed about the mean. If considerable scatter can occur for unconfined compression tests from the same block, it is not surprising that this scatter would also be evident in a series of CIU tests performed on samples from the same block. This scatter makes it difficult to draw a failure envelope through the points.

It is important at this time to comment upon the observed stress-strain behaviour and the concept of residual strengths being operative in slopes of Leda clay.

As discussed in Chapter 3, Lo and Lee (1973) have proposed, on the basis of a finite element program, that a significant zone of 'overstress' exists in a natural slope (see Figure 3.7.2b). In this zone of 'overstress', the peak strength is proposed to have been exceeded and only the residual strength remains to be mobilized. If a triaxial test was performed on material from this zone, an elasto-plastic stress-strain behaviour would be anticipated. The maximum value of shear stress attained would be equivalent to a residual strength value.

As was mentioned in Chapters 5 and 6, block samples for this thesis were obtained from the bottom or toe of natural slopes, except for Site No. 3, Green Creek North, where chunk samples were taken from an excavation in a flat, grassy field. According to Lo and Lee's (1973) concepts, the toe of a slope is the most overstressed zone, and one should not observe a brittle type behaviour.

Virtually all the test results illustrate a well defined peak with a significant reduction in strength after the peak, and thus cast a doubt on Lo and Lee's concept of residual strength existing in natural slopes of Leda clay. Furthermore, the stress-strain curves are quite similar to those from the Green Creek North site where no zone of 'overstress' can exist. The results clearly indicate, that even if the clay is in a highly stressed zone, it has retained its peak strength and sensitive, brittle characteristics. This is probably a result of "cementation" bonds in the clay.

7.7 Porewater Pressures Developed During Shear

In order to properly appreciate the shear stress-strain behaviour, it is necessary to consider the shear induced porewater pressures.

Figures 7.6.1 to 7.6.5 illustrate the change in porewater pressures (Δu) measured throughout the duration of the triaxial tests.

Tests performed at the very low confining pressure of 0.7 kPa indicate a very slight positive pore pressure up to failure and then become significantly negative with increasing strain. This indicates dilation with increasing strain after failure. The magnitude of this dilation appears to be related to preconsolidation pressure. In general, the greater the p'_c for a particular soil, the greater the magnitude of dilation at low confining pressures.

Tests performed at higher confining pressures all indicate increasing values of porewater pressure with increasing confining pressure, as expected. The Δu vs. strain curves appear to have a peak corresponding to the peak in shear stress, generally at about 1% strain.

It is interesting to note that for Sites 1, 3 and 4 the porewater pressure appears to increase with increasing strain after the peak at about 1% strain. For Sites 2 and 5 however, the porewater pressure seems to decrease after the peak at 1% and then reach a constant or residual value. This phenomenon can be explained by observing the effective stress path followed by the sample during shear. These stress paths are illustrated by the vertical dashed lines in Figures 7.8.1 to 7.8.5 of the next section. For Sites 2 and 5, Figures 7.8.2 and 7.8.5, respectively, it can be seen that no effective stress path goes beyond a value of $p' = 0.5\ p'_c$, indicated by a vertical arrow on the p' axis. However, for Sites 1, 3 and 4, Figures 7.8.1, 7.8.3 and 7.8.4, respectively, it can be seen that several stress paths either begin at or go beyond a value of

$p' = 0.5\ p'_c$. It is for these tests only, that an increase of porewater pressure with strain occurs after the initial peak at 1% strain. Tests conducted at a value of p' less than $0.5\ p'_c$ and whose stress paths remain below this range all demonstrate a porewater pressure that reaches a peak close to 1% and then either remains relatively constant or decreases to a residual value.

The reason for this occurrence is a result of the fact that during isotropic consolidation a yield point occurs at a value of $p'_m = 0.5\ p'_c$, indicating structural breakdown and causing significant volume changes. In isotropic consolidation $\sigma'_1 = \sigma'_2 = \sigma'_3$ and thus $\frac{\sigma'_1 + 2\sigma'_3}{3} = \frac{\sigma'_1 + \sigma'_3}{2}$, hence $p'_m = p' = \frac{\sigma'_1 + \sigma'_3}{2}$. Since the Mohr-Coulomb failure criterion is more commonly expressed in terms of $\sigma'_n = \frac{\sigma'_1 + \sigma'_3}{2}$, it is perhaps more convenient and appropriate to express this yield point as occurring when $p' \geq 0.5\ p'_c$. Therefore, when the value of p' exceeds $0.5\ p'_c$ during triaxial shear in a CIU test, yield or structural breakdown occurs that results in the development of high porewater pressures.

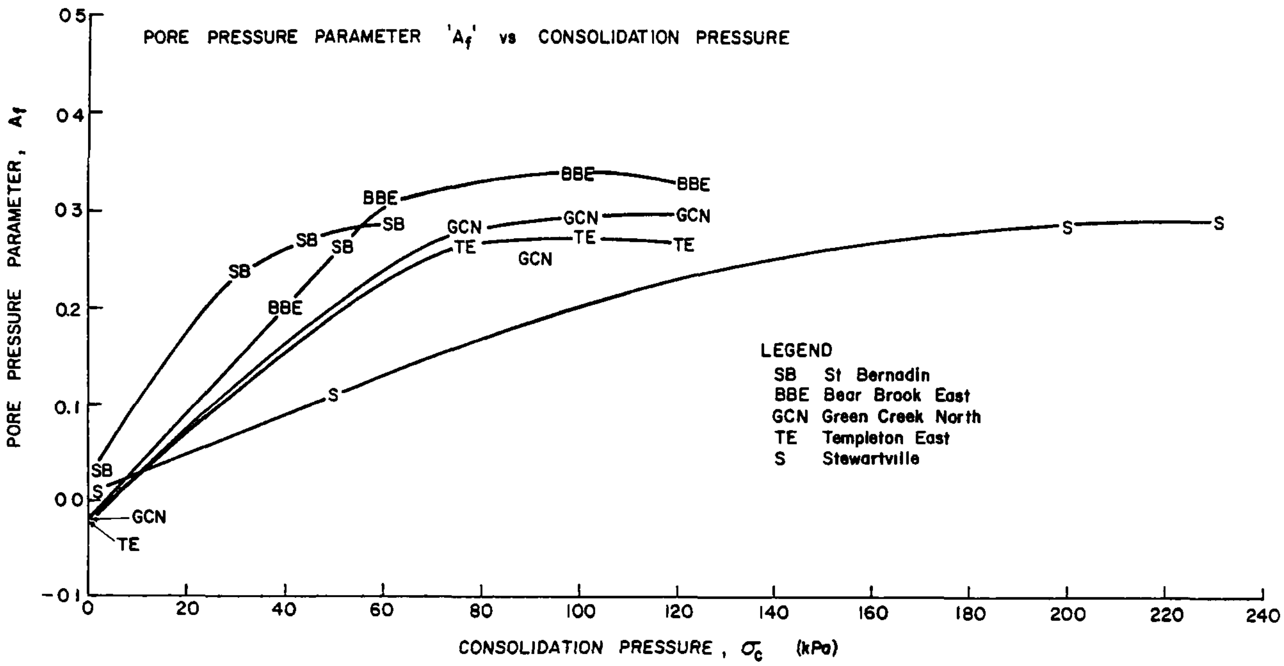
Briefly then, there appears to be a definite structural breakdown when the average effective confining pressure p' reaches a value close to $0.5\ p'_c$ measured in the oedometer test. The occurrence has been demonstrated by both isotropic consolidation and triaxial shear.

The pore pressure developed during shear significantly affects the observed strength of the clay. Tests performed at the low confining pressure of 0.7 kPa, that develop negative pore pressures with increasing strain, generally show only a small reduction in strength after peak and then either maintain a constant shear stress or show a slight increase in shear stress with further strain. However, tests performed at higher confining pressures, which develop higher pore pressures at failure, show a more marked reduction in shear stress after the peak. This is a direct result of the change in effective stress on the sample. When pore pressures grow negative, the effective stress increases and consequently results in an increase in shear strength. When the pore pressure goes positive, the effective stress decreases and consequently the shear strength is reduced.

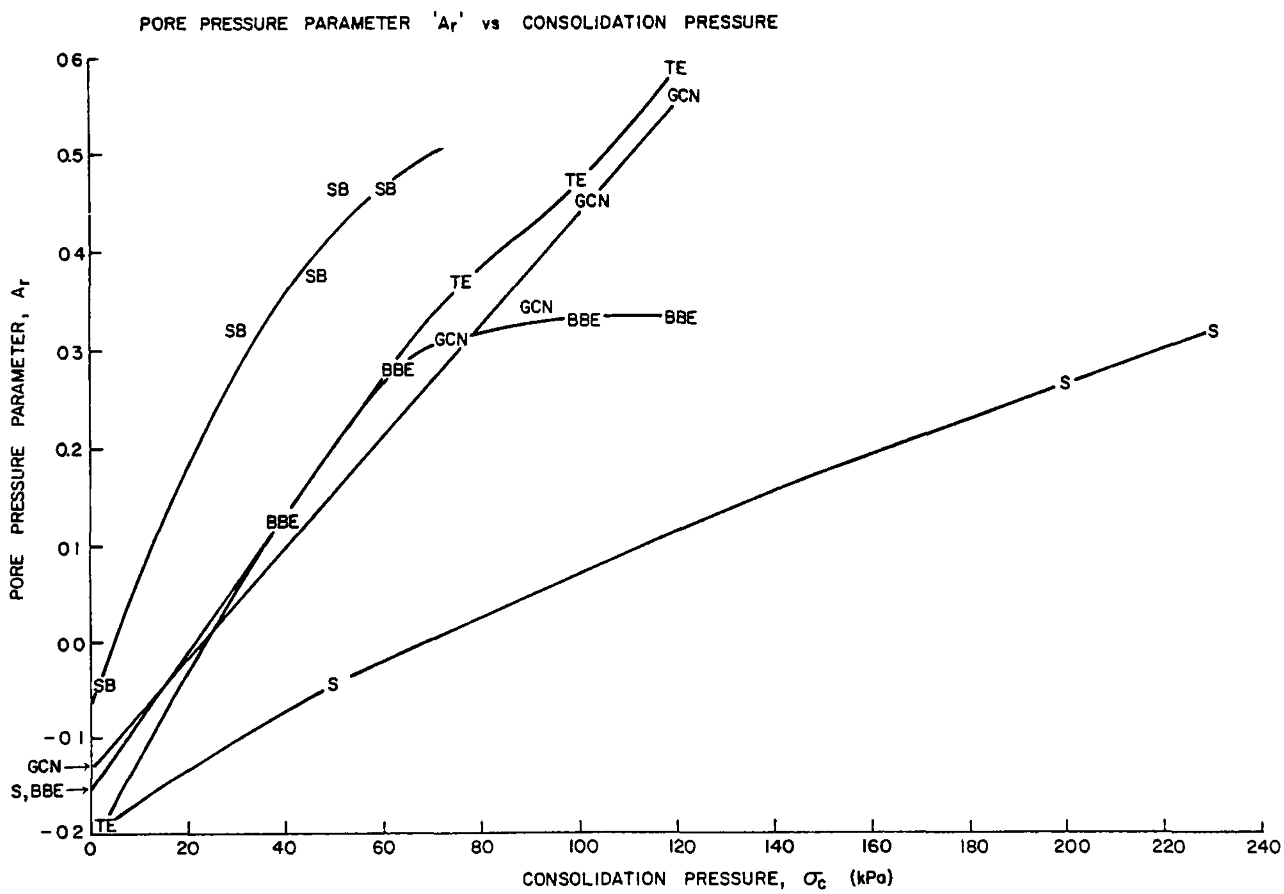
Values of the pore pressure parameter 'A' at failure (A_f) and at the residual strength (A_r) are plotted against the consolidation pressure at which the tests were performed for each site on Figure 7.7.1. The results of the plot of 'A_f' vs. consolidation pressure indicate that the value of A_f increases rapidly and linearly from 0.7 kPa consolidation pressure to about 25% of the preconsolidation pressure, then the values A_f slowly increase between 0.25 and 0.35. These values are typical for a lightly overconsolidated clay according to Lambe and Whitman (1969). The results of the plot of 'A_r' vs. consolidation pressure indicate a more or less linear increase in 'A_r' with consolidation pressure. However, the Bear Brook East clay appears to flatten out and maintain a value of A_r = 0.33.

7.8 Stress Paths and Strain Contours

Figures 7.8.1 to 7.8.5 are plots showing the effective stress paths (vertical dashed lines) the samples have followed until failure. Also plotted are contours of axial strain prior to failure. Examination of the contour spacing indicates yield begins to take place between 0.7 and 0.8% strain for most of the clays. Yield occurs when the stress-



PORE PRESSURE PARAMETER 'Af' vs CONSOLIDATION PRESSURE
PORE PRESSURE PARAMETER, Af
CONSOLIDATION PRESSURE, σc (kPa)
LEGEND
SB St Bernadin
BBE Bear Brook East
GCN Green Creek North
TE Templeton East
S Stewartville



PORE PRESSURE PARAMETER 'Ar' vs CONSOLIDATION PRESSURE
PORE PRESSURE PARAMETER, Ar
CONSOLIDATION PRESSURE, σc (kPa)

Figure 7.7.1

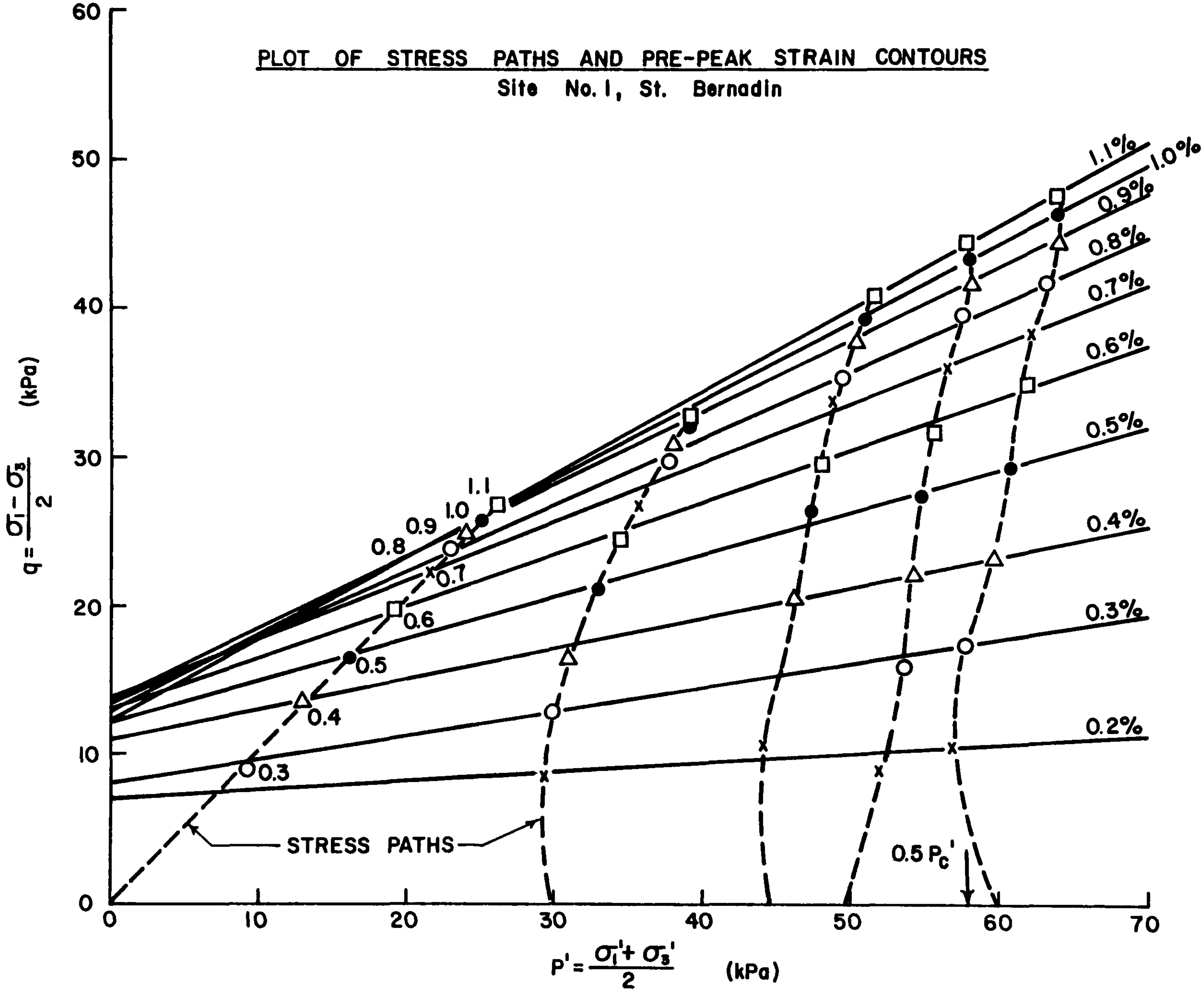

PLOT OF STRESS PATHS AND PRE-PEAK STRAIN CONTOURS
Site No. 1, St. Bernadin
q = (σ1 − σ3)/2 (kPa)
P' = (σ1' + σ3')/2 (kPa)
1.1%
1.0%
0.9%
0.8%
0.7%
0.6%
0.5%
0.4%
0.3%
0.2%
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
STRESS PATHS
0.5 Pc'
0
10
20
30
40
50
60
70

Figure 7.8.1

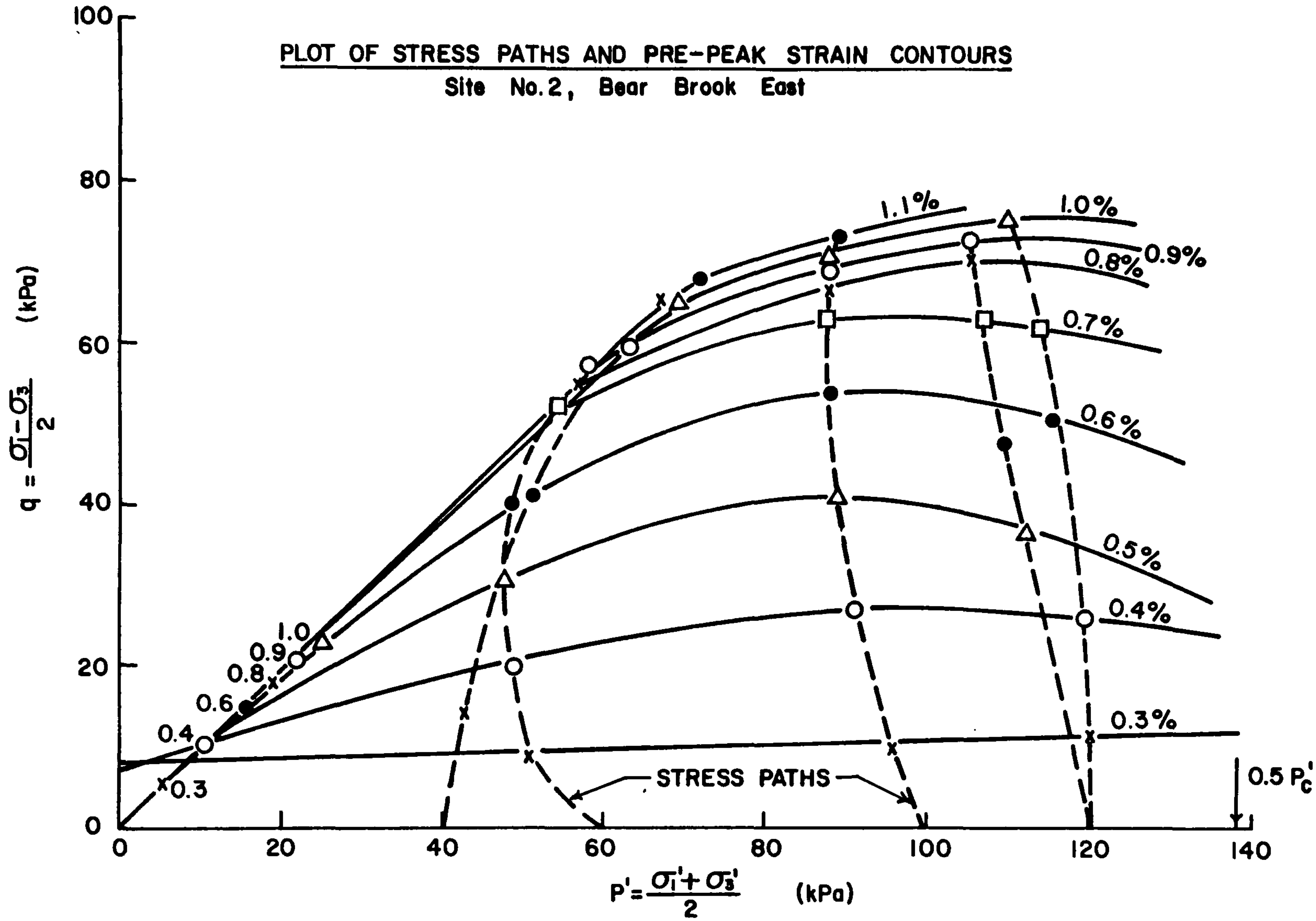

PLOT OF STRESS PATHS AND PRE-PEAK STRAIN CONTOURS
Site No. 2, Bear Brook East
$q = \frac{\sigma_1 - \sigma_3}{2}$ (kPa)
$P' = \frac{\sigma_1' + \sigma_3'}{2}$ (kPa)
0
20
40
60
80
100
120
140
1.1%
1.0%
0.9%
0.8%
0.7%
0.6%
0.5%
0.4%
0.3%
1.0
0.9
0.8
0.6
0.4
0.3
STRESS PATHS
$0.5 P_c'$

Figure 7.8.2

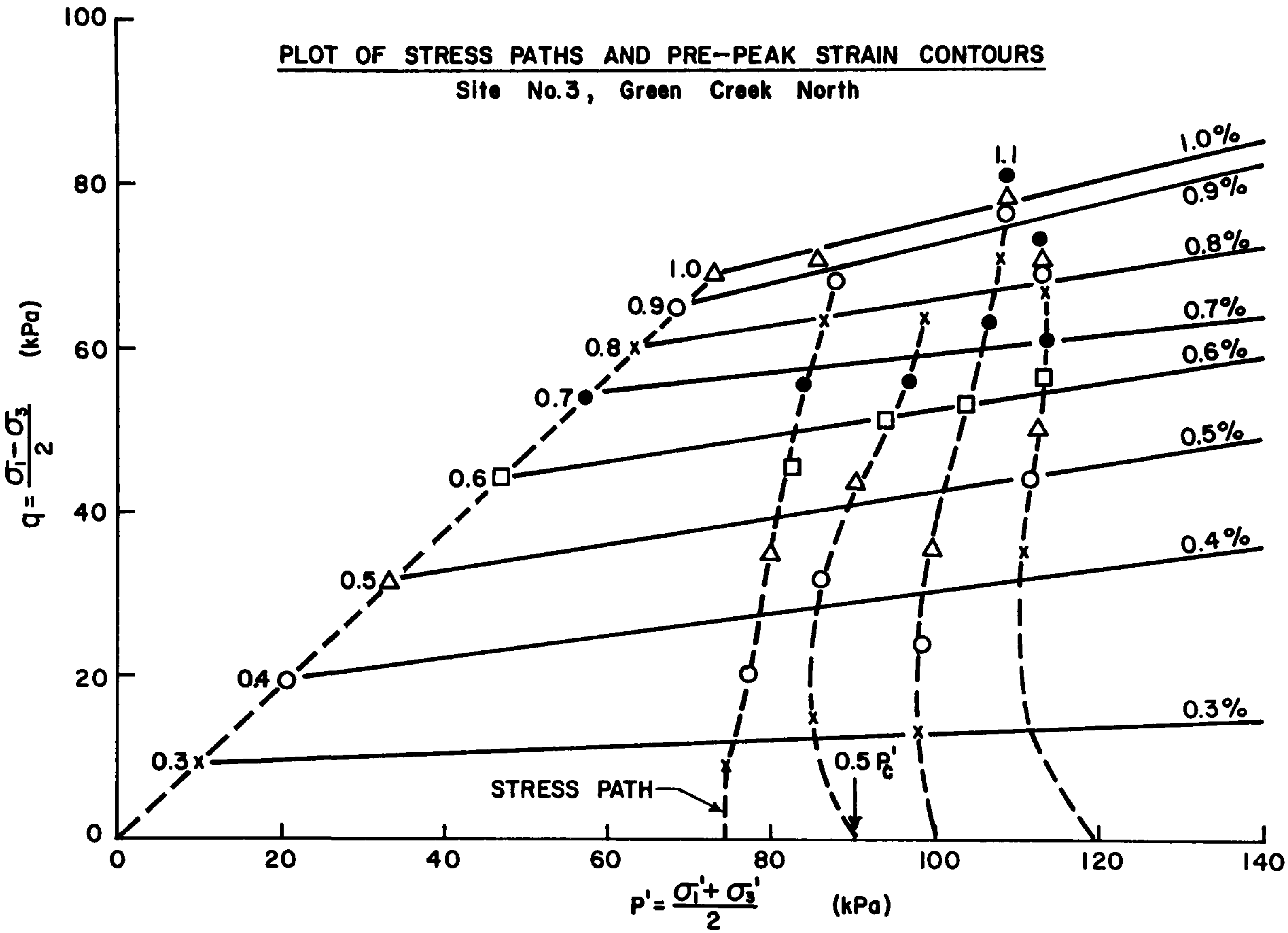

PLOT OF STRESS PATHS AND PRE-PEAK STRAIN CONTOURS
Site No.3, Green Creek North
1.0%
0.9%
0.8%
0.7%
0.6%
0.5%
0.4%
0.3%
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
STRESS PATH
0.5 P'c
q = (σ1 − σ3)/2 (kPa)
P' = (σ1' + σ3')/2 (kPa)
0
20
40
60
80
100
120
140

Figure 7.8.3

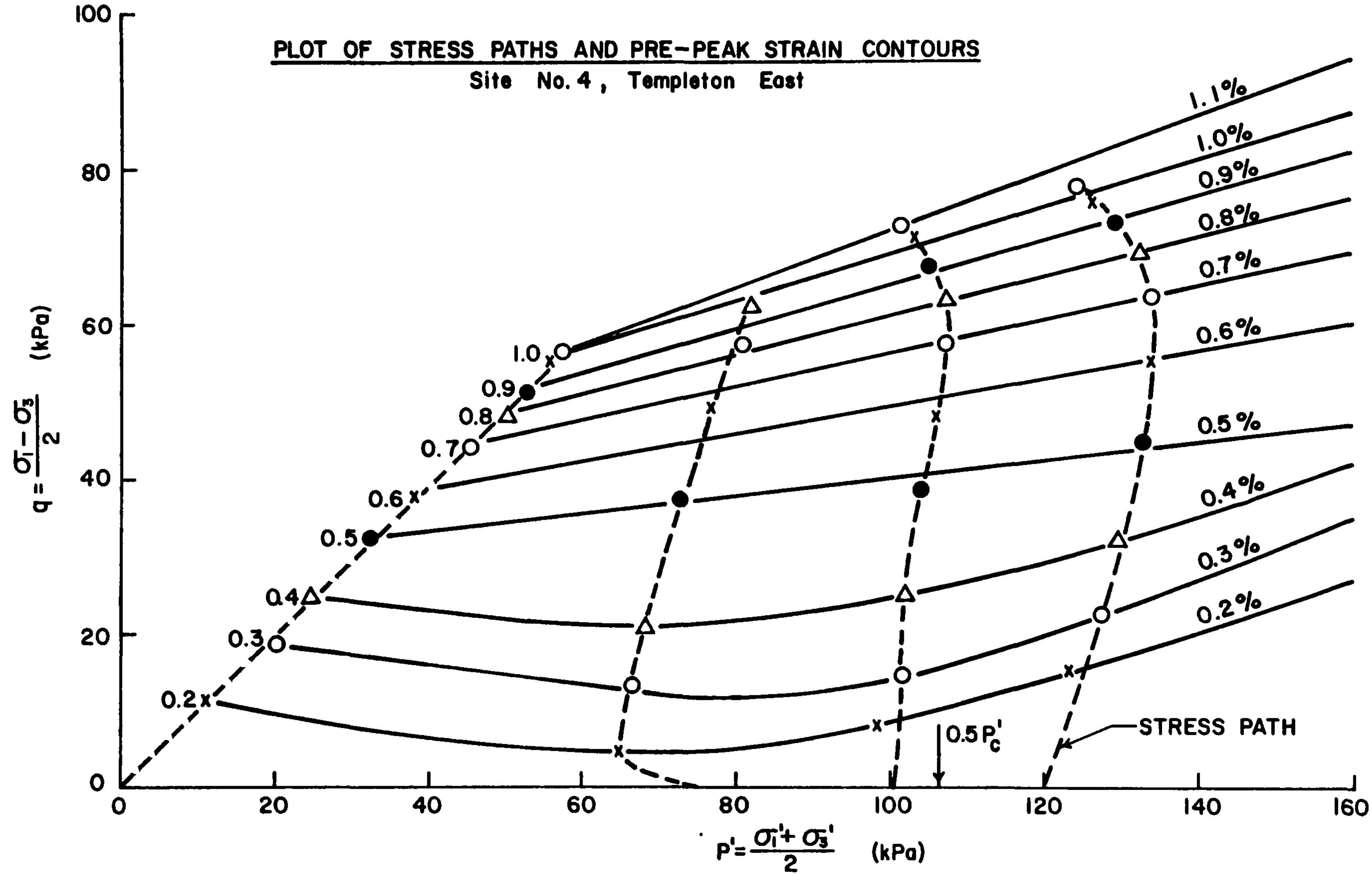

PLOT OF STRESS PATHS AND PRE-PEAK STRAIN CONTOURS
Site No. 4, Templeton East
1.1%
1.0%
0.9%
0.8%
0.7%
0.6%
0.5%
0.4%
0.3%
0.2%
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.5 P'c
STRESS PATH
q = (σ1 − σ3)/2 (kPa)
p' = (σ1' + σ3')/2 (kPa)
0
20
40
60
80
100
120
140
160

Figure 7.8.4

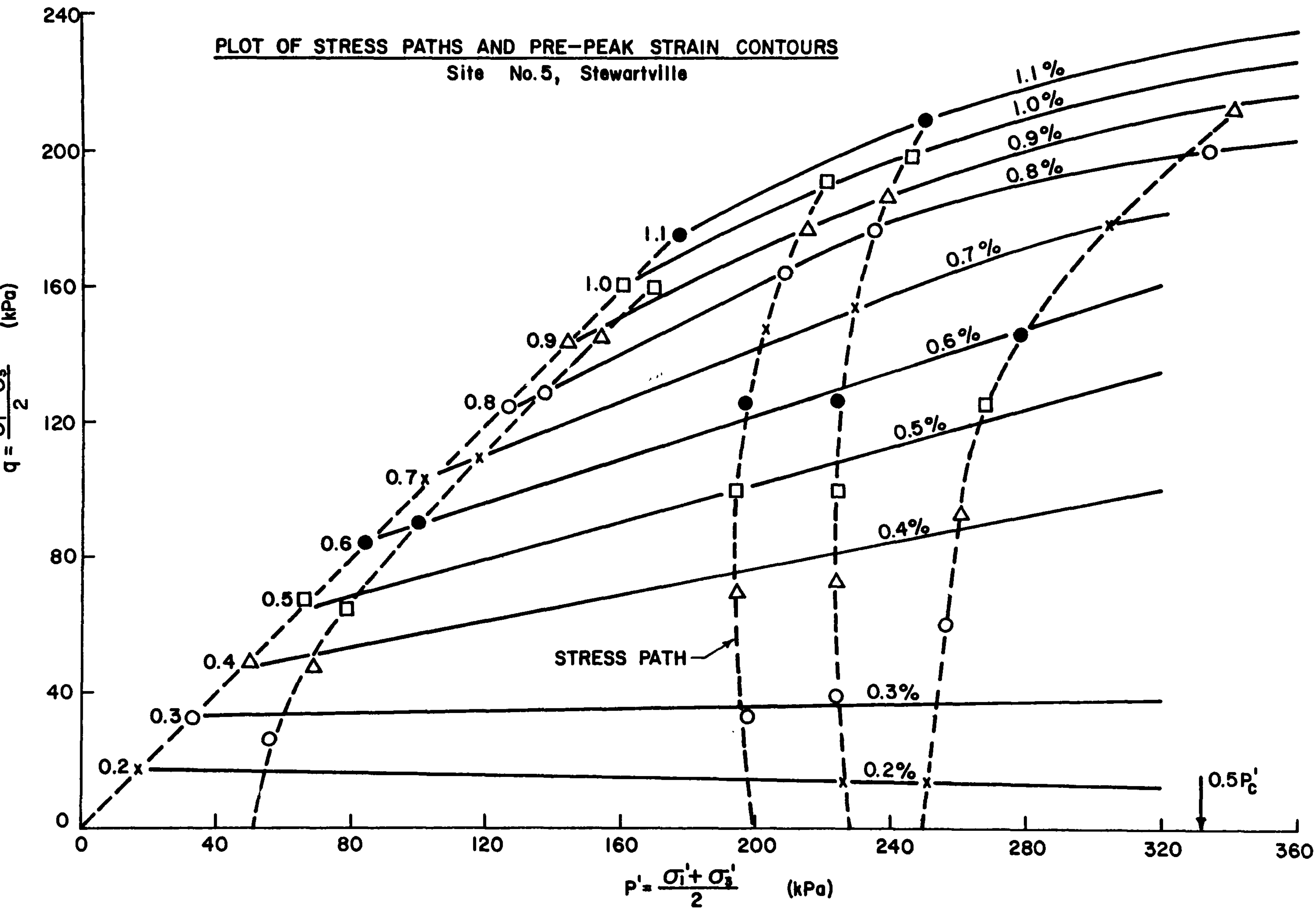

PLOT OF STRESS PATHS AND PRE-PEAK STRAIN CONTOURS
Site No. 5, Stewartville
q = (σ1 − σ3)/2 (kPa)
P' = (σ1' + σ3')/2 (kPa)
STRESS PATH
0.5P'c
1.1%
1.0%
0.9%
0.8%
0.7%
0.6%
0.5%
0.4%
0.3%
0.2%
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0
40
80
120
160
200
240
280
320
360

Figure 7.8.5

strain curve of the material begins to deviate from its initial tangent, indicating pronounced non-recoverable (plastic) straining under the applied load.

In general, the strain contours are linear in the low stress range (less than 0.5 p_c'), however they would be expected to flatten out in the intermediate stress range (greater than 0.5 p_c'). Most points of equal strain fall along the contour lines indicated, but the Green Creek North clay exhibits significant scatter (Figure 7.8.3). This scatter is not surprising when one considers that the contour interval is only 0.1% strain.

The usefulness of these strain contour plots will be discussed in the next section.

7.9 Mobilization of c' and ϕ' with Strain

By plotting the axial strain along stress paths and drawing contour lines, one can obtain vital and interesting information on the gradual development of friction and cohesion with increasing strain. The apparent cohesion mobilized at a particular strain is the intercept of the tangent to the strain contour on the shear stress axis at an effective normal stress equal to zero. The apparent angle of internal friction is the angle the tangent to the strain contour makes with the horizontal plane at this intercept. Since the points plotted on Figures 7.8.1 to 7.8.5 are p-q points, the intercept with the y axis is actually termed a' and the angle is termed α'. To convert to c' and ϕ' one can use these simple expressions, obtainable from the geometry of the problem:

$$\phi' = \sin^{-1} \tan\alpha' \qquad\qquad c' = \frac{a'}{\cos\phi'}$$

Figures 7.9.1 to 7.9.3 illustrate the mobilization of c' and ϕ' with strain for Sites 1, 4 and 5. Plots of this type were not made for Sites 2 and 3 due to scatter and non-linearity in the low stress range. The actual data used in the preparation of these figures is summarized in Tables A-7 to A-9 of Appendix A.

As can be seen from these figures, the mobilized c' generally peaks at a low strain between 0.8 and 1.1% and then drops dramatically. The residual value of c' at 6% strain is approximately equal to 50% of the peak value. The mobilized ϕ' generally peaks at greater strain, except for the very dense, hard clay of Site No. 5, Stewartville. The residual value of ϕ' is about 70 to 90% of the peak value.

The very brittle nature of the clay at Site No. 4, Templeton East, is illustrated by the dramatic (60%) reduction of mobilized c' from 1.1 to 1.5% strain. Thus, it can be readily seen that if the clay is strained beyond 1.1%, which is highly possible due to the insertion of the field vane or by taking tube samples, the strength measured would be substantially reduced from the peak value.

The previous figures were presented as a conceptual aid in the understanding of how cohesion and friction contribute towards the development of shear resistance. The stress-strain curves of Figures 7.6.1 to 7.6.5 peak at strains of about 1%, i.e., at the same strain that the mobilized c' peaks. After this point, further straining destroys the cementation bonds until only a certain residual value may be mobilized. After the peak occurs in c', ϕ' may increase slightly and then either remain relatively constant or decrease slowly. The shear strength after

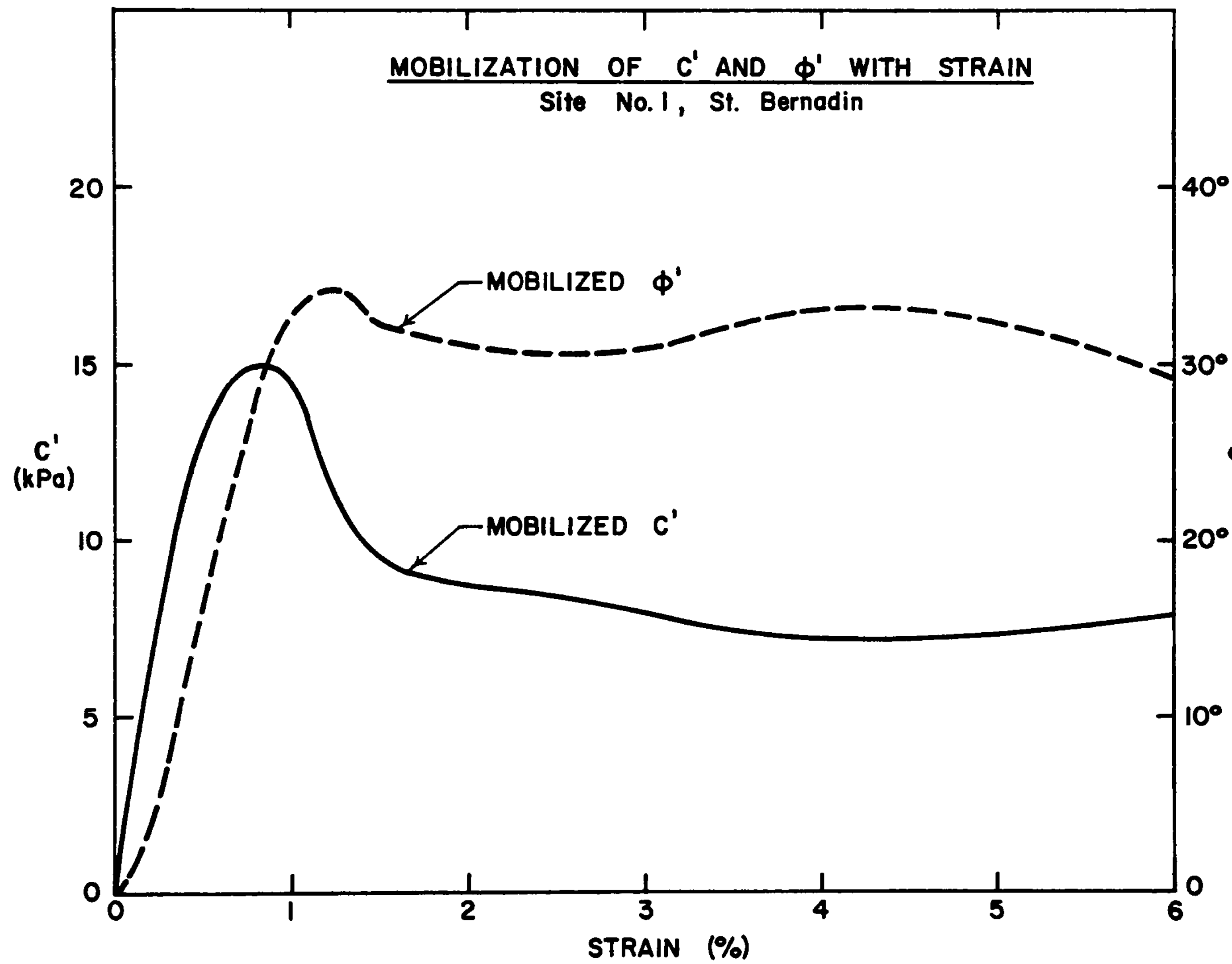

MOBILIZATION OF C' AND φ' WITH STRAIN
Site No. 1, St. Bernadin
MOBILIZED φ'
MOBILIZED C'
C' (kPa)
φ'
STRAIN (%)
0
5
10
15
20
0
10°
20°
30°
40°
0
1
2
3
4
5
6

Figure 7.9.1

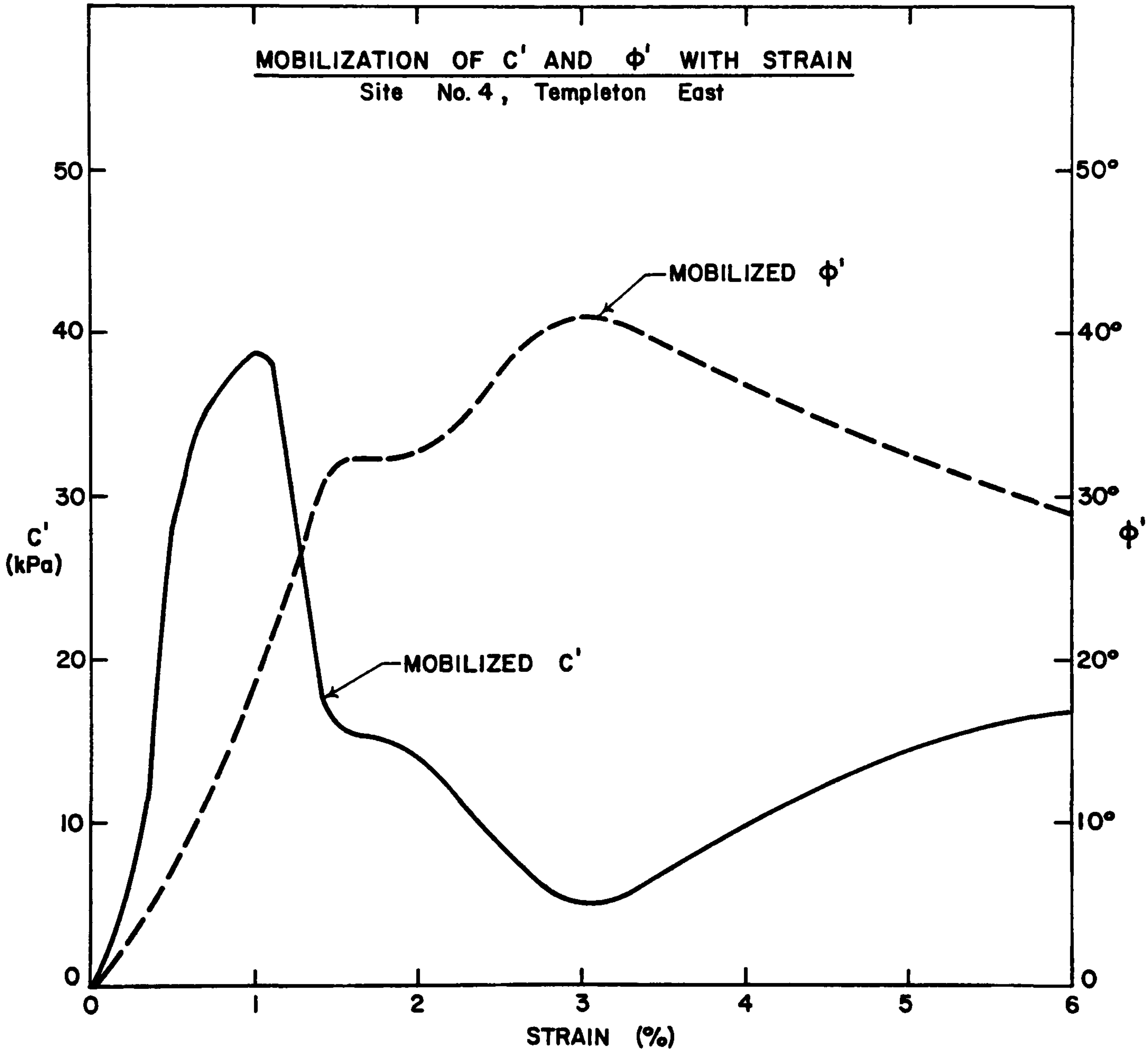

MOBILIZATION OF C' AND Φ' WITH STRAIN
Site No. 4, Templeton East
MOBILIZED Φ'
MOBILIZED C'
C' (kPa)
Φ'
STRAIN (%)
0
10
20
30
40
50
0
10°
20°
30°
40°
50°
0
1
2
3
4
5
6

Figure 7.9.2

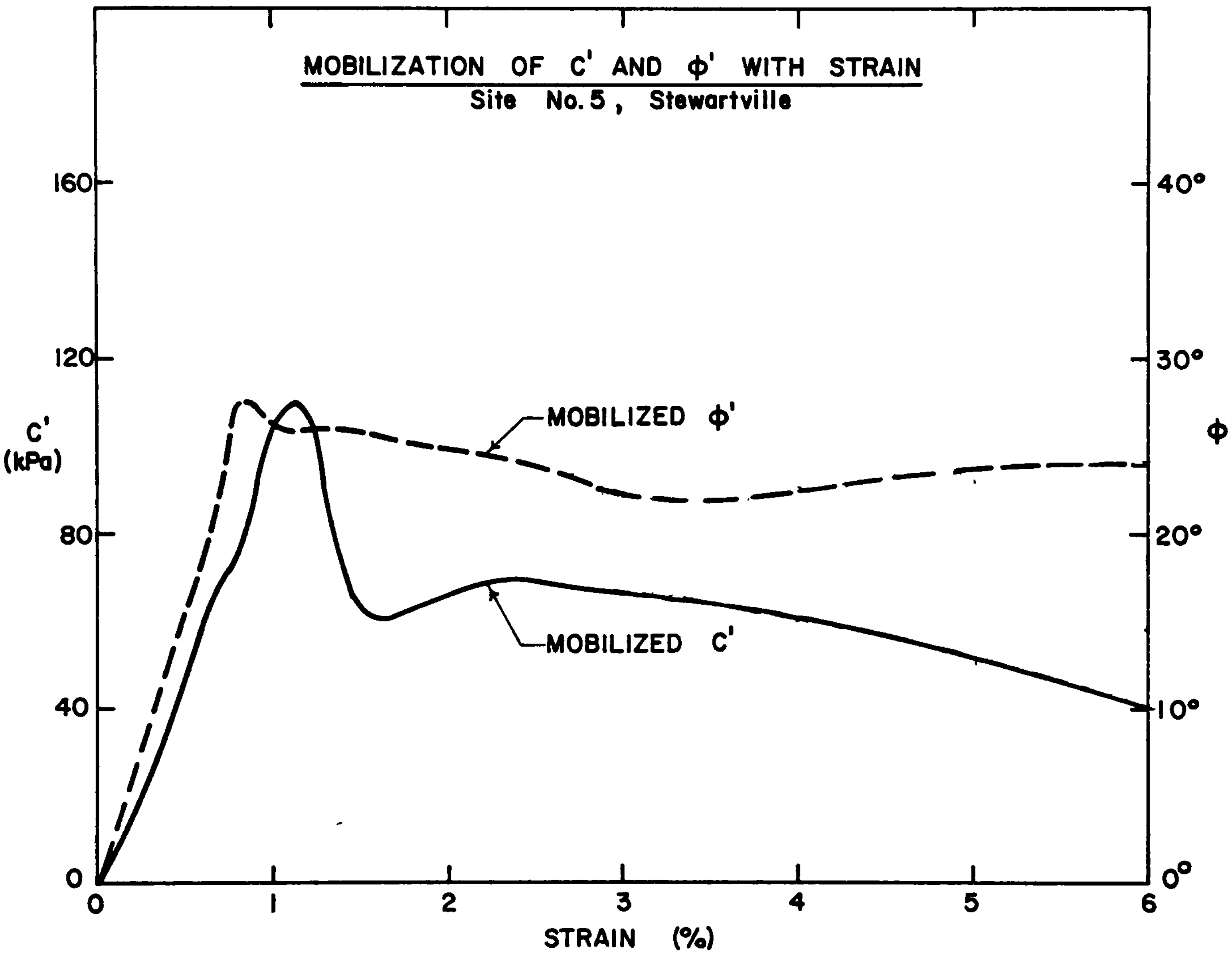

MOBILIZATION OF C' AND φ' WITH STRAIN
Site No. 5, Stewartville
C' (kPa)
φ'
MOBILIZED φ'
MOBILIZED C'
STRAIN (%)
0
40
80
120
160
0°
10°
20°
30°
40°
0
1
2
3
4
5
6

Figure 7.9.3

the peak mobilization of c', largely depends upon effective stresses, and therefore upon the effective confining pressure. As mentioned in Section 7.7, negative pore pressures are induced under low confining pressures. This increases the effective stress on the sample and therefore more frictional resistance is mobilized causing either the shear strength to increase or at least remain constant. Confining pressures greater than 0.5 p'_c cause a structural breakdown in the clay which induces large positive pore pressures.

This causes a reduction in effective stress. Thus, although the friction angle ϕ' may be increasing, the clay cannot take advantage of this frictional resistance due to the reducing effective stress.

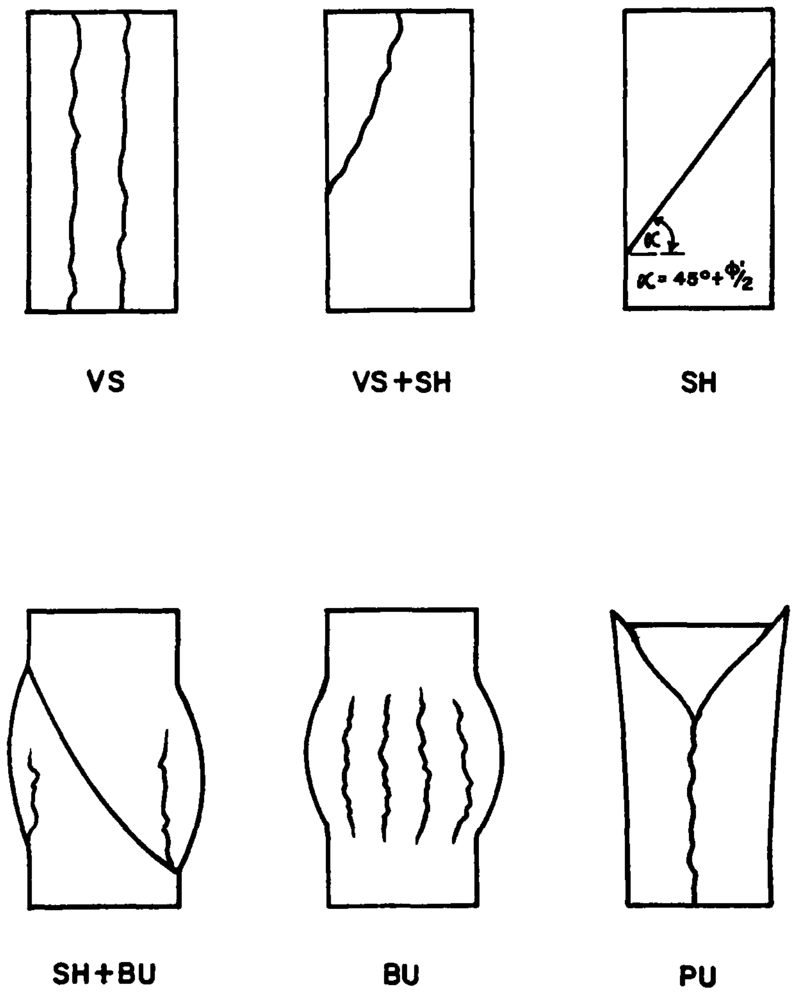
Thus, plotting both mobilized c' and ϕ' provides an interesting look into how these two parameters contribute towards shear strength.

7.10 Mode of Failure

Figure 7.10.1 presents sketches of the modes of failure observed throughout the testing program. A detailed account of modes of failure for each test performed is available in Tables A-1 to A-6 in Appendix A.

Toombs (1974) performed drained, stress controlled constant p'_m tests on fissured clay samples from the Ottawa area and found the mode of failure to be dependent on effective normal stress. Starting from the very lowest effective normal stress, the mode of failure was that of vertical splitting, progressing with increased effective normal stress to a combination of vertical splitting and shear, to shear, to a combination of shear and bulging or to only bulging.

OBSERVED FAILURE MODES



VS = VERTICAL SPLITTING

SH = SHEAR

BU = BULGING

PU = PUNCHING

Figure 7.10.1

For this thesis, strain controlled CIU tests were performed in the low normal effective stress range only. The predominant mode of failure was shear, although a small amount of vertical splitting occurred under very low confining pressures. Isolated cases of bulging or punching occurred under higher confining pressures.

Toombs reported that for samples tested in the low effective stress range, a typical failure surface was stepped or nodular due to dilation or volumetric expansion on the failure surface. Jarrett (1972) reports that for samples of fissured clay the failure surface is reduced to "a disintegrated, wet mass of small clay granules, ... similar to the condition of the soil debris left after landslides in this type of soil." Failure surfaces of samples tested undrained for this thesis were smooth, and not stepped or nodular.

Mohr-Coulomb theory predicts that the failure surface in the triaxial sample should occur at an angle $\alpha = 45° + \phi'/2$. Failure surfaces for this testing programme ranged from angles of 48° to 65° with an average of about 60°. This yields an average angle of internal friction of about 30° with a range of 6° to 40°. Typical values of ϕ' for fissured clays in the geotechnical literature range from 30° to 40°. Toombs (1974) and Lawrence (1969) have concluded that values of ϕ' measured from the shear plane do not agree with ϕ' measured from the Mohr envelope over the appropriate range of stress. Gill (1968) explains that this may be due to the effect of cementation bonds or anisotropy. It would appear that ϕ' can only be accurately determined by a series of triaxial tests at different confining pressures.

7.11 p-q Envelopes

Presented in Figures 7.11.1 to 7.11.5 is the locus or envelope of p-q points corresponding to peak failure and residual effective stress conditions, determined from the CIU tests for each site. Also presented is the peak total stress envelope and the average results of field, hand and laboratory vanes, plus the average shear strength from the unconfined compression tests.

As can be seen from the figures, the peak effective stress envelope is pronouncedly curved. The shapes of these peak envelopes are however, similar to the peak envelopes reported in the literature. The peak total stress envelope is best approximated by a straight line, inclined at a small angle to the horizontal. The residual effective stress envelope is also best represented with a straight line.

Some scatter is evident along the peak effective stress envelope, especially for Site No. 3, Green Creek North, but for the other sites the peak effective stress envelope is well defined. The reason for the significant scatter at Green Creek North was explained partially in Section 7.6 as sample variability and is also due to the lower quality of the "chunk" samples taken from this site as opposed to block samples from the others.

The scatter observed from some points along the residual strength failure envelope can be easily explained by the different modes of failure that occurred for these samples. The most common mode of failure observed in all the triaxial samples was that of a simple shear plane. However, as was noted in Section 7.10, vertical splitting occasionally occurred at very low confining pressures for some sites. At Site Nos. 1, 4 and 5 in particular, vertical splitting was observed for tests carried out at a cell

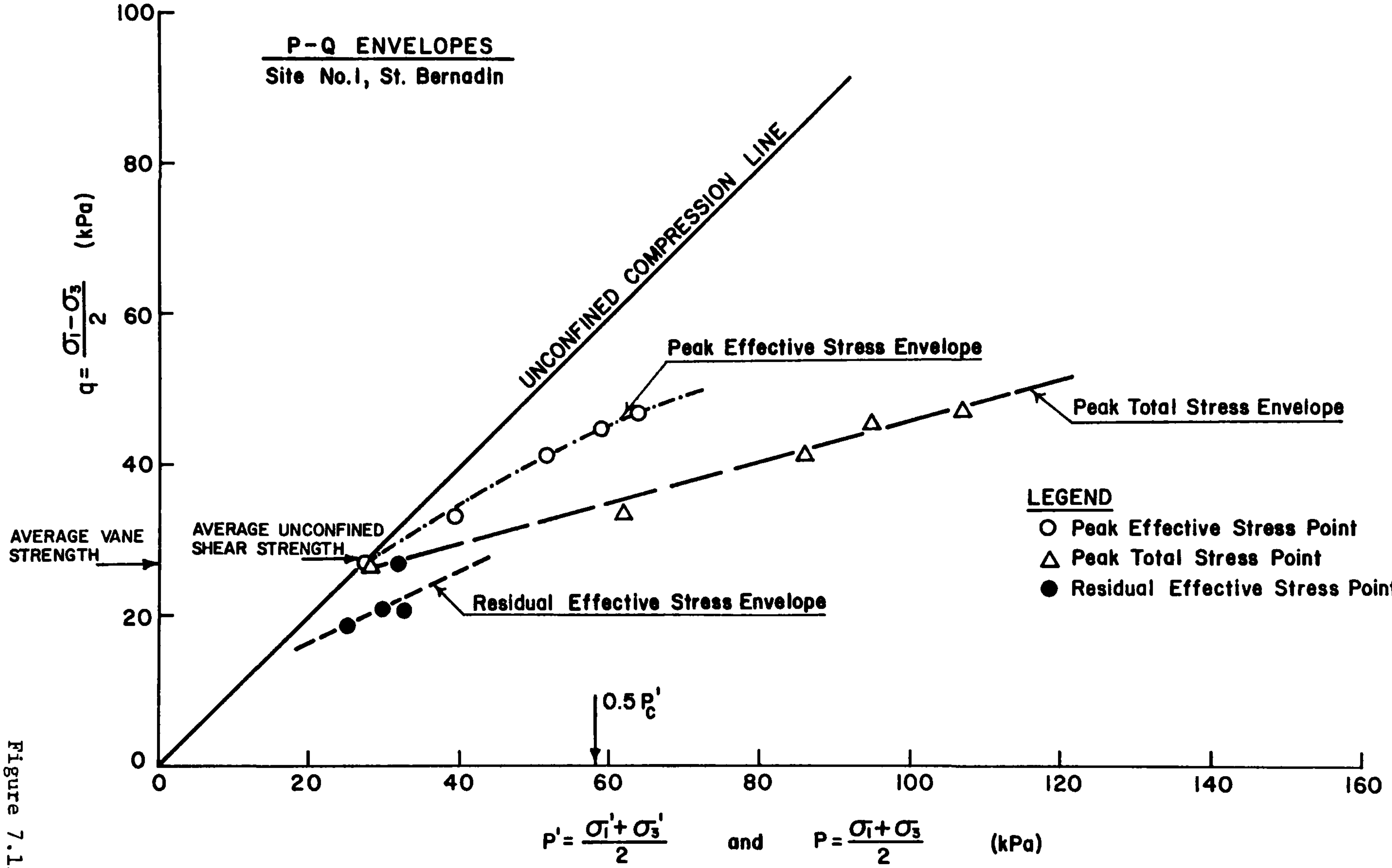

P-Q ENVELOPES
Site No.1, St. Bernadin
UNCONFINED COMPRESSION LINE
Peak Effective Stress Envelope
Peak Total Stress Envelope
Residual Effective Stress Envelope
AVERAGE VANE STRENGTH
AVERAGE UNCONFINED SHEAR STRENGTH
$0.5\,P_c'$
LEGEND
Peak Effective Stress Point
Peak Total Stress Point
Residual Effective Stress Point
$q = \frac{\sigma_1 - \sigma_3}{2}$ (kPa)
$P' = \frac{\sigma_1' + \sigma_3'}{2}$ and $P = \frac{\sigma_1 + \sigma_3}{2}$ (kPa)
0
20
40
60
80
100
120
140
160

Figure 7.11.1

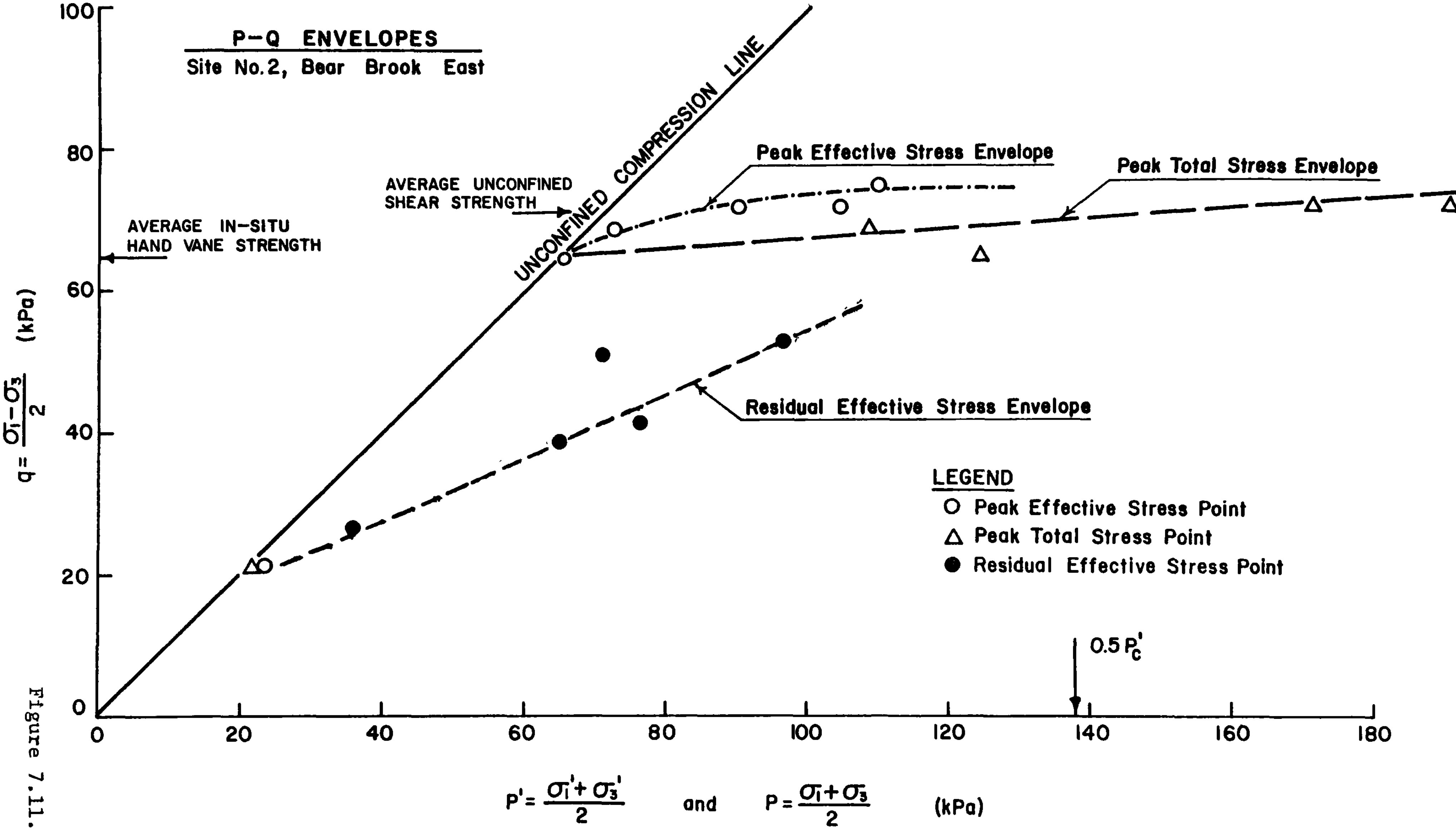
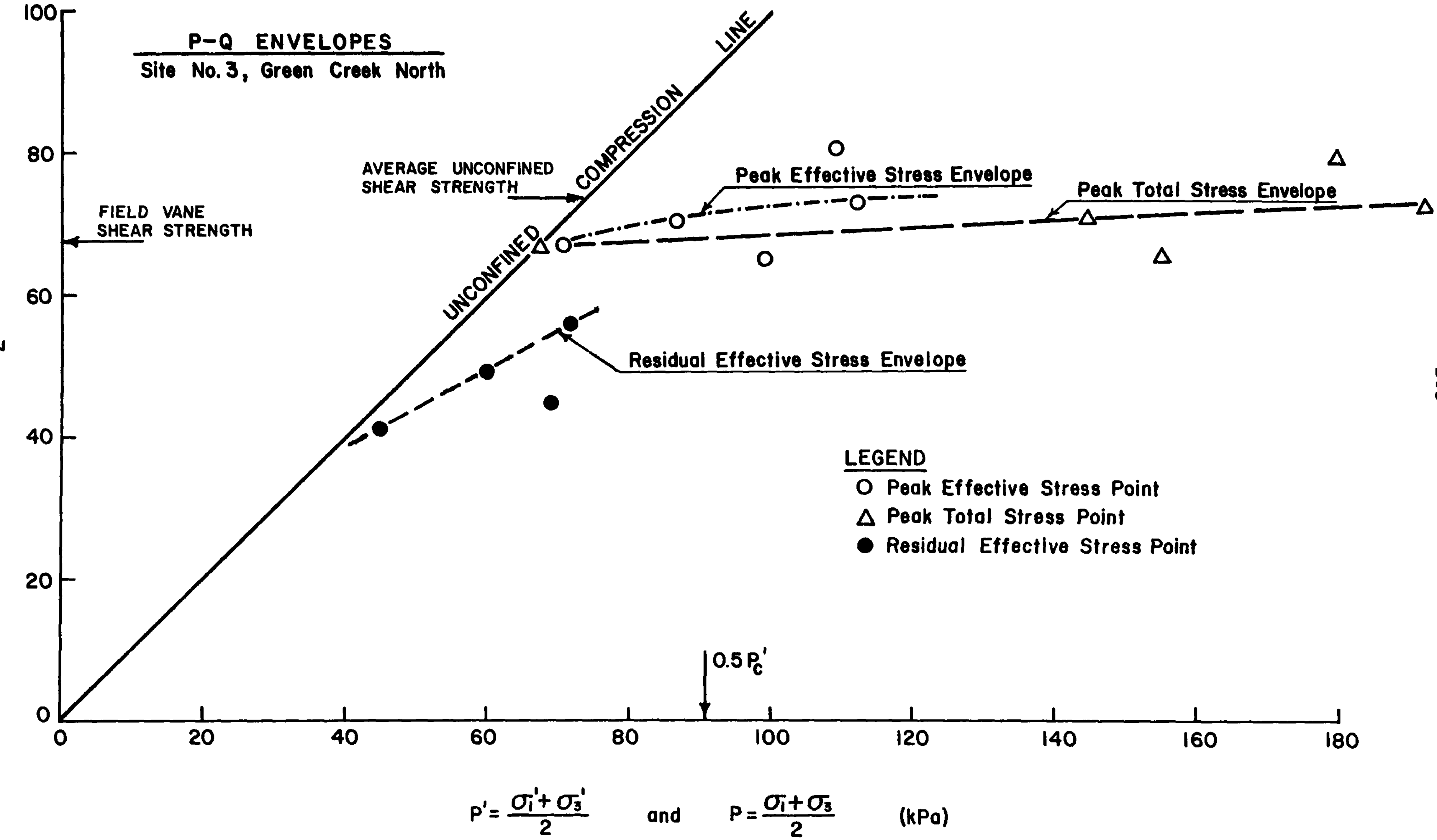
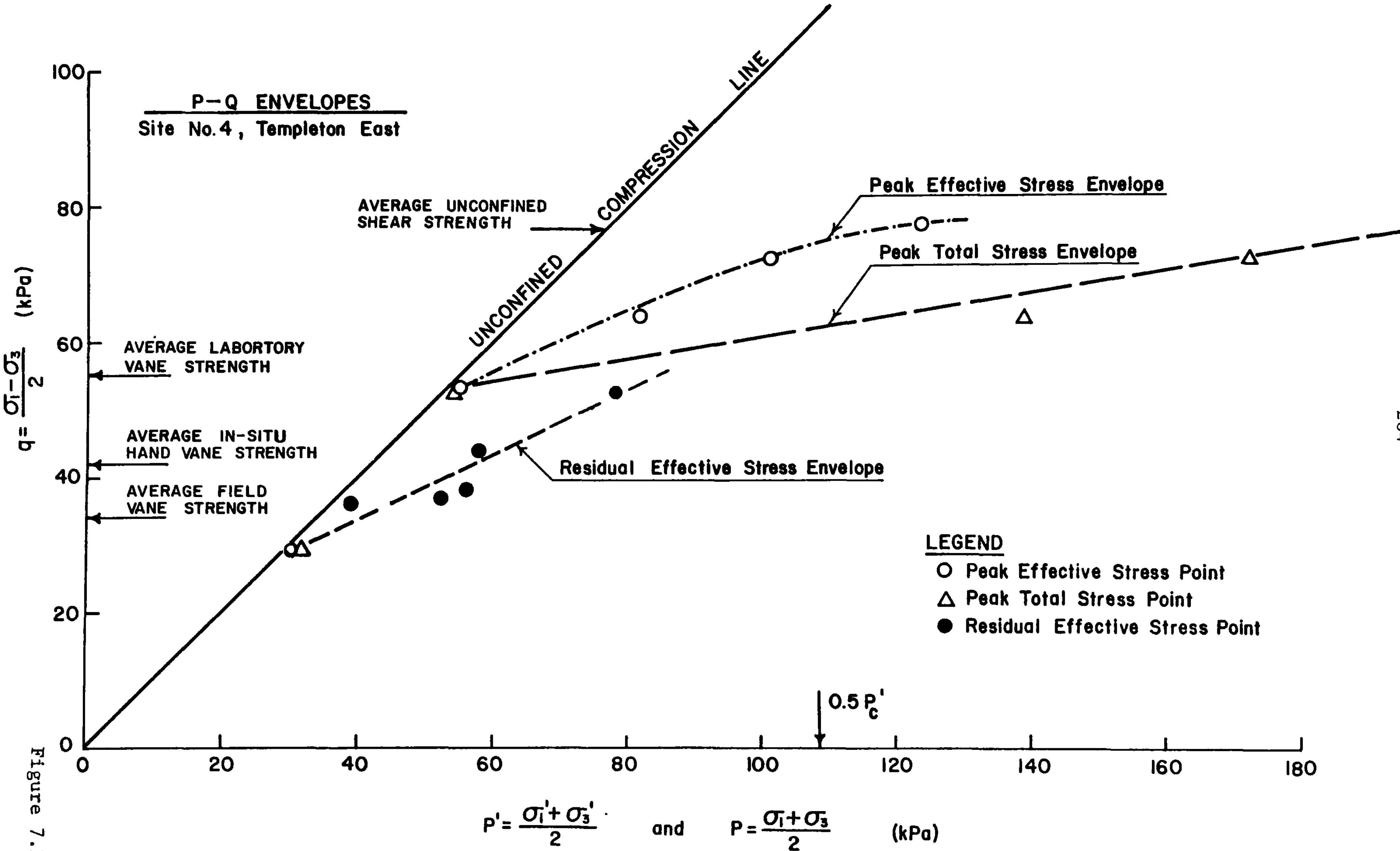

P-Q ENVELOPES
Site No. 2, Bear Brook East
UNCONFINED COMPRESSION LINE
Peak Effective Stress Envelope
Peak Total Stress Envelope
AVERAGE UNCONFINED SHEAR STRENGTH
AVERAGE IN-SITU HAND VANE STRENGTH
Residual Effective Stress Envelope
LEGEND
Peak Effective Stress Point
Peak Total Stress Point
Residual Effective Stress Point
0.5 P_c'
$q = \frac{\sigma_1 - \sigma_3}{2}$ (kPa)
$P' = \frac{\sigma_1' + \sigma_3'}{2}$ and $P = \frac{\sigma_1 + \sigma_3}{2}$ (kPa)
0
20
40
60
80
100
120
140
160
180

Figure 7.11.2



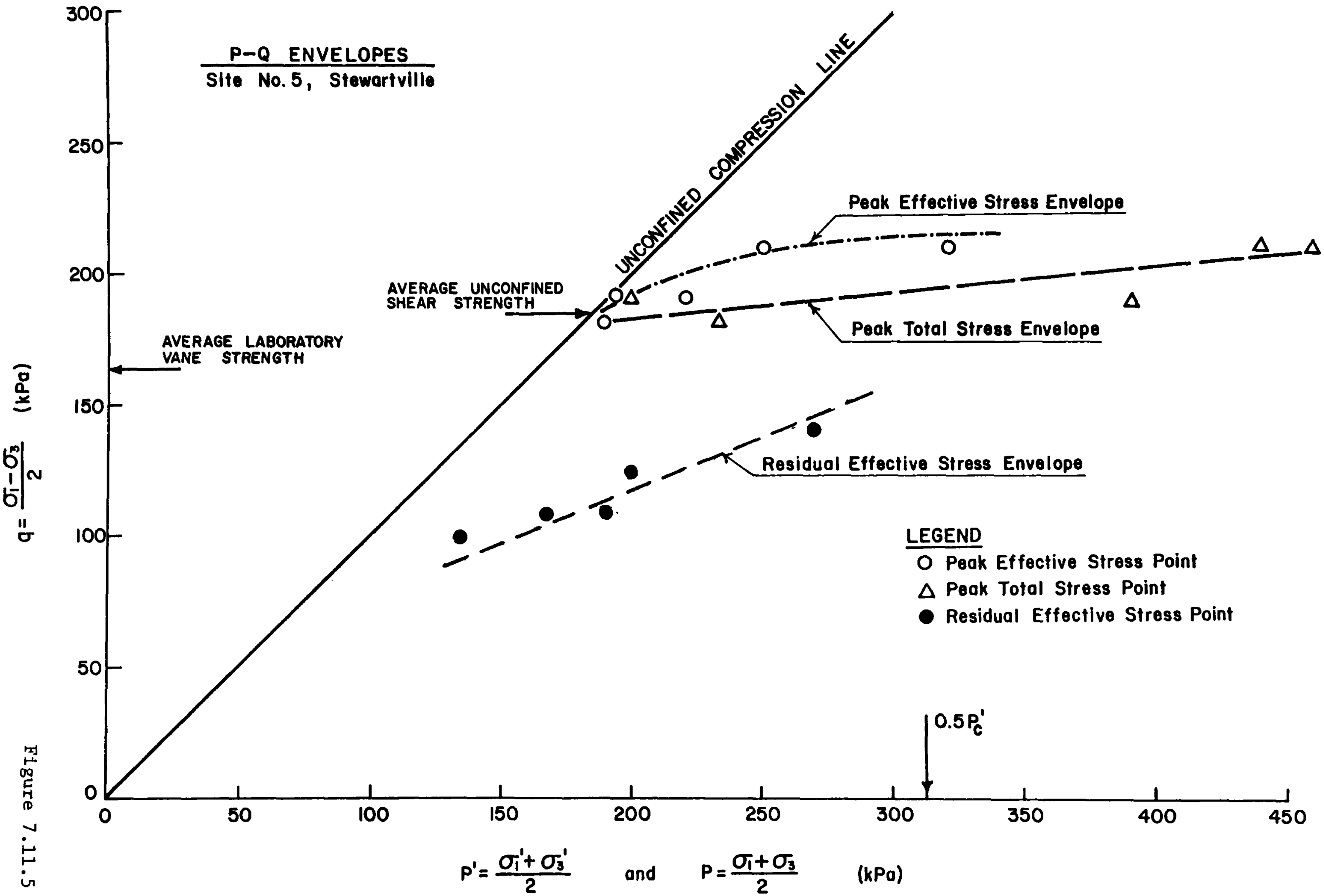
P-Q ENVELOPES
Site No. 3, Green Creek North
UNCONFINED COMPRESSION LINE
AVERAGE UNCONFINED SHEAR STRENGTH
FIELD VANE SHEAR STRENGTH
Peak Effective Stress Envelope
Peak Total Stress Envelope
Residual Effective Stress Envelope
LEGEND
Peak Effective Stress Point
Peak Total Stress Point
Residual Effective Stress Point
$0.5 P_c'$
$q = \frac{\sigma_1 - \sigma_3}{2}$ (kPa)
$P' = \frac{\sigma_1' + \sigma_3'}{2}$ and $P = \frac{\sigma_1 + \sigma_3}{2}$ (kPa)
0
20
40
60
80
100
120
140
160
180

Figure 7.11.3



P—Q ENVELOPES
Site No. 4, Templeton East
UNCONFINED COMPRESSION LINE
AVERAGE UNCONFINED SHEAR STRENGTH
Peak Effective Stress Envelope
Peak Total Stress Envelope
Residual Effective Stress Envelope
AVERAGE LABORTORY VANE STRENGTH
AVERAGE IN-SITU HAND VANE STRENGTH
AVERAGE FIELD VANE STRENGTH
LEGEND
Peak Effective Stress Point
Peak Total Stress Point
Residual Effective Stress Point
0.5 P'c
q = (σ1 − σ3)/2 (kPa)
P' = (σ1' + σ3')/2 and P = (σ1 + σ3)/2 (kPa)
0
20
40
60
80
100
120
140
160
180

Figure 7.11.4



P-Q ENVELOPES
Site No. 5, Stewartville
UNCONFINED COMPRESSION LINE
Peak Effective Stress Envelope
AVERAGE UNCONFINED SHEAR STRENGTH
Peak Total Stress Envelope
AVERAGE LABORATORY VANE STRENGTH
Residual Effective Stress Envelope
LEGEND
Peak Effective Stress Point
Peak Total Stress Point
Residual Effective Stress Point
0.5 P_c'
$q = \frac{\sigma_1 - \sigma_3}{2}$ (kPa)
$P' = \frac{\sigma_1' + \sigma_3'}{2}$ and $P = \frac{\sigma_1 + \sigma_3}{2}$ (kPa)
0
50
100
150
200
250
300
350
400
450

Figure 7.11.5

pressure of 0.7 kPa. This vertical splitting is believed to be the result of the effect of end restraint on the sample which at very low confining pressures induces a tensile stress at the center of the sample.

The residual failure envelope is a "best fit" through the data points for tests that exhibited shear failure. It is thought that this line best represents the true residual failure envelope for each site. The data points for tests with a vertical splitting mode of failure fall above the illustrated residual failure envelope.

At Site No. 2, the high point off the residual strength envelope is the result of a test on a shortened sample with a length to diameter ratio of only 1.7 as opposed to the normal ratio of 2.0. This reduced length has resulted in a higher than normal residual strength, although the peak strength apparently was not affected.

At Site No. 3, the point that falls below the residual strength envelope is the result of a peculiar mode of failure that developed after peak failure. Multiple shear surfaces were observed to develop in the lower part of the sample with increasing strain. This peculiar mode of failure was observed in only this sample and resulted in a lower than anticipated value of residual strength.

Thus, it is considered that the illustrated residual failure envelope is representative of each site. Any deviations from this line can generally be explained as the result of an unusual mode of failure.

It is of importance to note, from these figures, the relative location of the peak effective stress and the peak total stress envelope.

For all the sites, these two envelopes intersect at a point on the unconfined compression line. The shear stress at this point of intersection compares favourably with the results of vane strengths and unconfined compression shear strengths. In general, the unconfined compression shear strength slightly overestimates this shear stress level, but the vane strength estimates it very well.

At least one type of vane strength is indicated for each site. At some sites it was not possible to perform field, hand and lab vanes for various reasons described earlier in Chapter 5. At sites where more than one type of vane test was done, agreement was generally good except for Site No. 4, Templeton East. At this site the clay is very brittle and very sensitive to disturbance. It is thought that the large field vane, and to a lesser extent the hand vane, disturb the soil upon insertion and hence underestimate the undrained shear strength. The small lab vane however, appears to provide the best estimate of the undrained strength at this site.

In a vane shear test, the failure surface is in the vertical plane and the principal shear stress is in a horizontal direction. In a triaxial test, the principal shear stress is in a vertical direction, and the failure surface is theoretically oriented at an angle $\alpha = 45° + \phi/2$ from the horizontal. The fact that the vane shear strength correlates well with unconfined compression and CIU shear strengths indicates that the clay at most sites is not strongly anisotropic in strength. The poor correlation between most test results at Site No. 4, Templeton East, would indicate that the clay at this site may be anisotropic as might be expected due to the low plasticity index of the clay.

In Chapter 3, it was explained that the field vane measures the intact strength of a clay even if it is fissured (Lo, 1970). This is because the shape of the failure surface is predetermined and cylindrical, whereas most fissure surfaces are planar. Hence, the probability of containing fissures in the failure surface is small.

During sample trimming an occasional macrofissure was noticed in the block samples from Sites No. 2, 3 and 4. However, the samples were cut and trimmed so as to avoid any visible fissures. It is important to note from the results illustrated on the p-q envelopes of Figures 7.11.1 to 7.11.5, that no peak failure points fall below the stress level defined by the field vane strength and the intersection of the peak effective and total stress envelope, except for one sample from each of Sites Nos. 1 and 4, which will be discussed later. This indicates that all the clays tested in this thesis behave essentially as intact materials in the low effective stress range. This is in contrast to the microfissured behaviour of most Ottawa area clays as described by Eden and Mitchell (1970, 1973) and Mitchell (1975).

Microfissured clays exhibit a predominantly frictional, dilatant, straight line failure envelope in the low effective stress range. This failure envelope begins at a shear stress level far below that defined by the field vane shear strength as was illustrated in Figures 3.5.3, 3.6.1b and 3.6.5b. This type of failure envelope is attributed to the presence of closely spaced hairline fractures or 'closed fissures' (Eden and Mitchell, 1970).

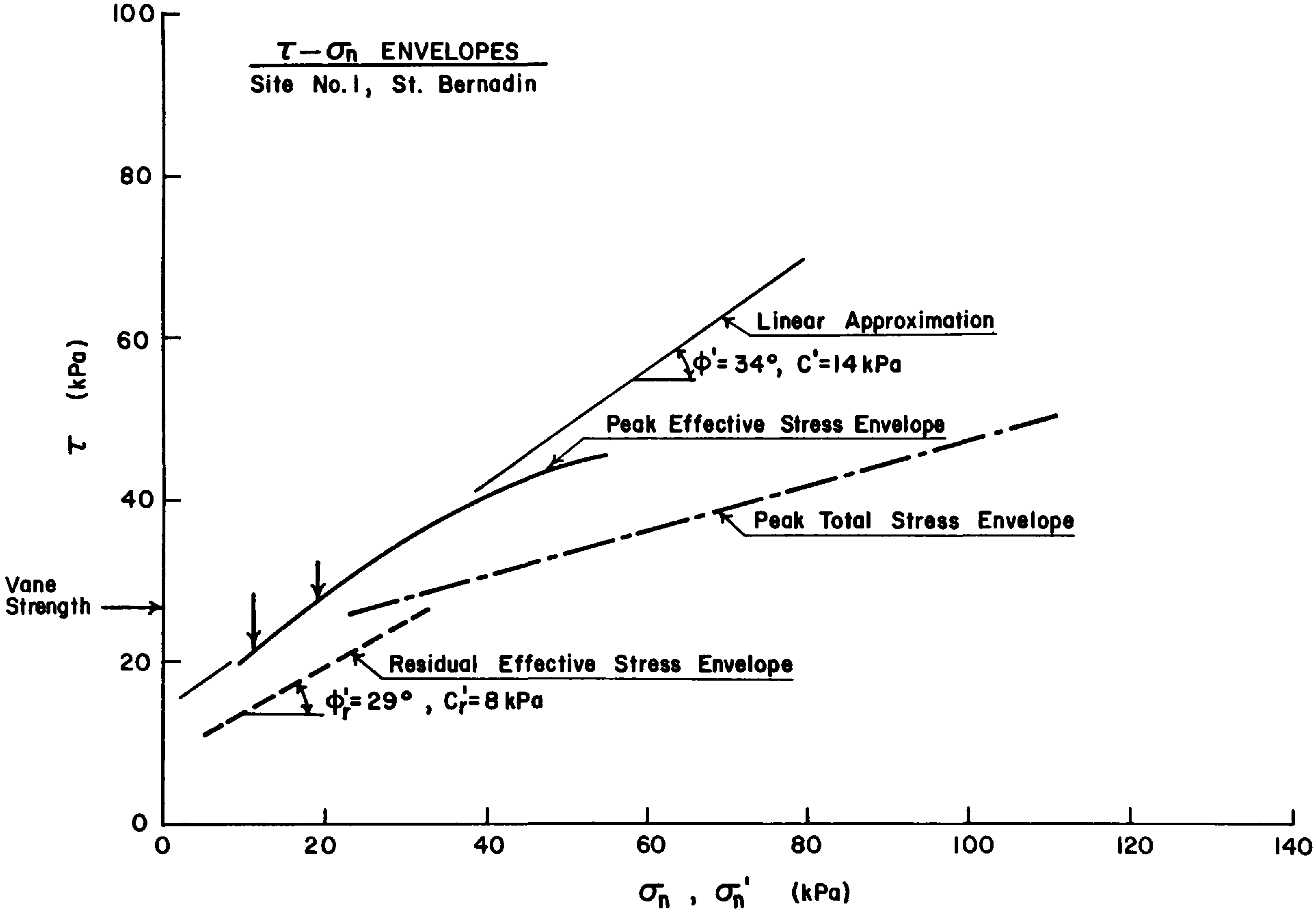
The one sample from Site No. 2 and the one sample from Site No.4 that had a peak failure point below the shear stress level defined by the field vane strength are believed to be the result of a macrofissure that was undetected during the trimming of the sample. Close examination of the sample from Bear Brook East revealed a root in the centre of the sample that may have caused the fissure. Both of these tests were carried out at a very low confining pressure of 0.7 kPa. Both failure points are coincident with the unconfined compression line.

In summary, although the clay mass may be macrofissured at some of the sites studied, the clay samples tested for this thesis behave essentially as an intact material in the low effective stress range of testing. This would suggest that sample size may be an important factor that must be considered when extrapolating apparently intact strength results to a clay mass that may be macrofissured in the field. The influence of these macrofissures on the strength mobilized in the field would have to be considered.

Finally, the peak effective and total stress envelope intersect at a point on the unconfined compression line. This point of intersection may be estimated by the shear stress determined from a vane shear test. This suggests that undrained field vane strength may be related to the peak effective stress (drained) failure envelope.

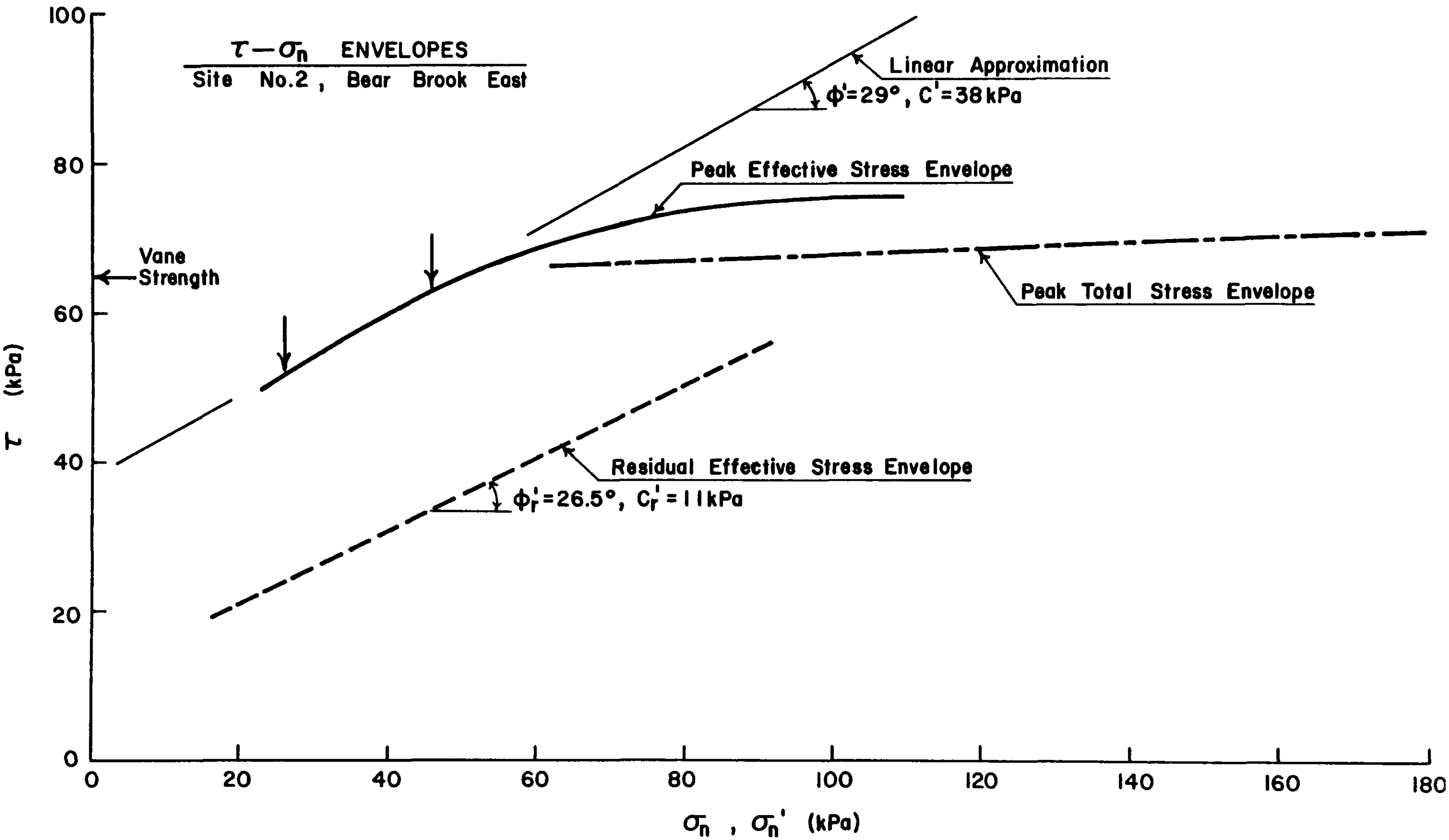
7.12 τ-σ_N Envelopes

Presented in Figures 7.12.1 to 7.12.5 are the shear stress vs. total and effective normal stress plots for each site. The p-q plots presented in Section 7.11 represent points located at the tops of the Mohr circles. The τ-σ_n plots represent the tangent to the family of Mohr circles and is thus displaced to the left.



τ – σn ENVELOPES
Site No. 1, St. Bernadin
Linear Approximation
φ' = 34°, C' = 14 kPa
Peak Effective Stress Envelope
Peak Total Stress Envelope
Vane Strength
Residual Effective Stress Envelope
φ'r = 29°, C'r = 8 kPa
τ (kPa)
σn, σ'n (kPa)
0
20
40
60
80
100
120
140

Figure 7.12.1



τ — σn ENVELOPES
Site No.2, Bear Brook East
Linear Approximation
φ'=29°, C'=38 kPa
Peak Effective Stress Envelope
Vane Strength
Peak Total Stress Envelope
Residual Effective Stress Envelope
φr'=26.5°, Cr'=11 kPa
τ (kPa)
σn , σn' (kPa)
0
20
40
60
80
100
120
140
160
180

Figure 7.12.2

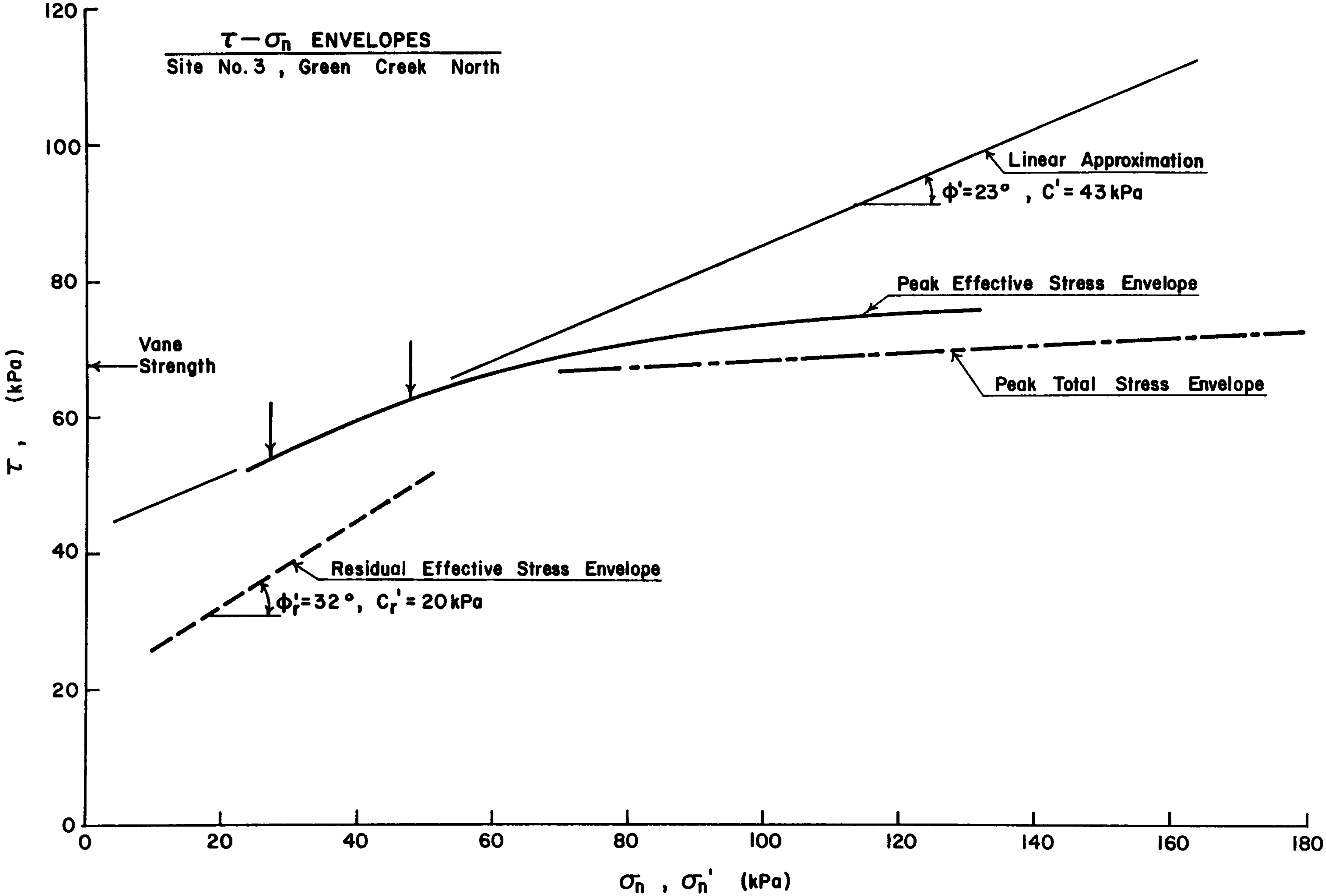

τ − σn ENVELOPES
Site No. 3 , Green Creek North
Linear Approximation
φ' = 23° , C' = 43 kPa
Peak Effective Stress Envelope
Peak Total Stress Envelope
Vane Strength
Residual Effective Stress Envelope
φr' = 32°, Cr' = 20 kPa
τ , (kPa)
σn , σn' (kPa)
0
20
40
60
80
100
120
140
160
180

Figure 7.12.3

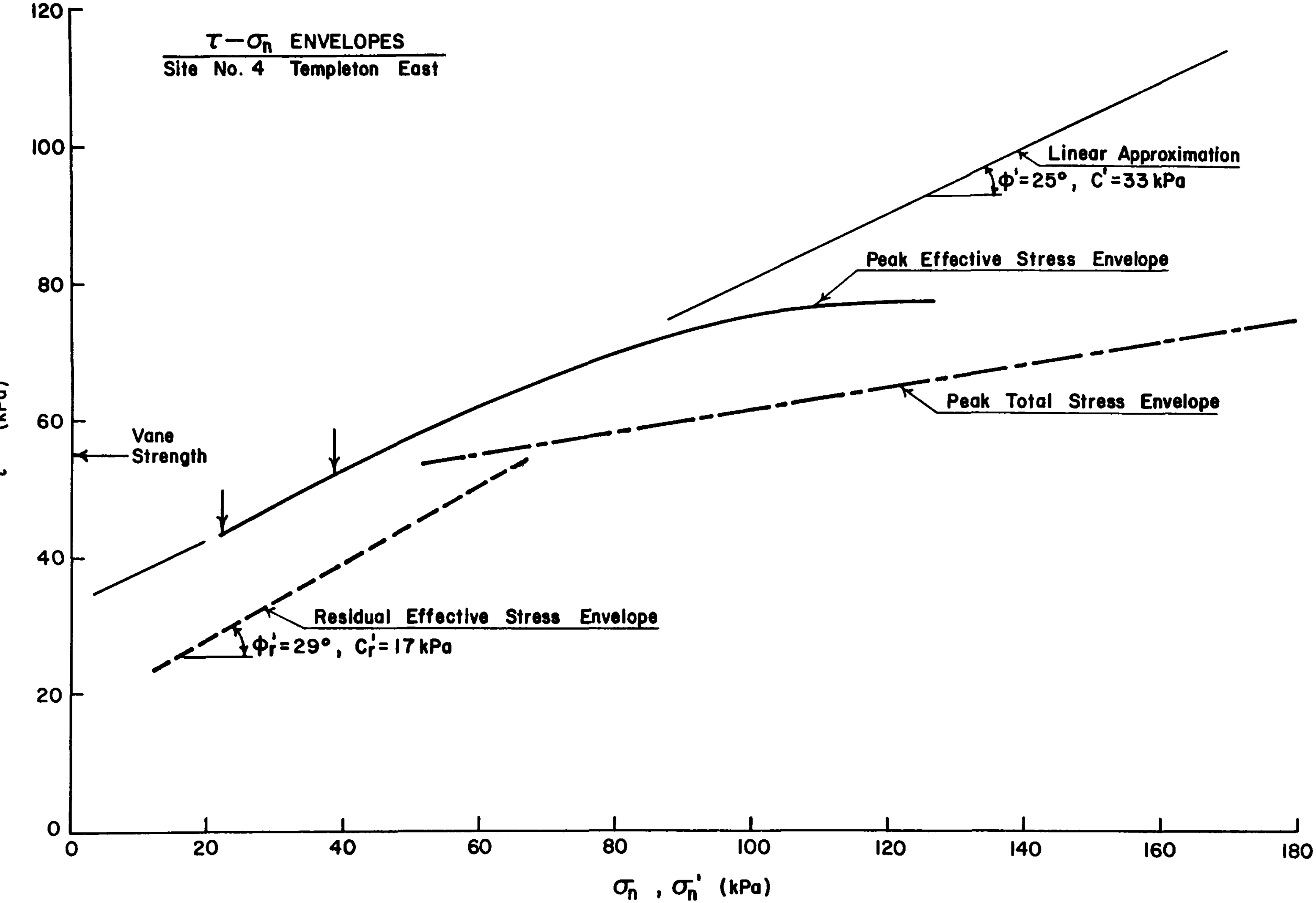

τ − σn ENVELOPES
Site No. 4 Templeton East
Linear Approximation
φ' = 25°, C' = 33 kPa
Peak Effective Stress Envelope
Peak Total Stress Envelope
Vane Strength
Residual Effective Stress Envelope
φr' = 29°, Cr' = 17 kPa
τ (kPa)
σn, σn' (kPa)
0
20
40
60
80
100
120
140
160
180

Figure 7.12.4

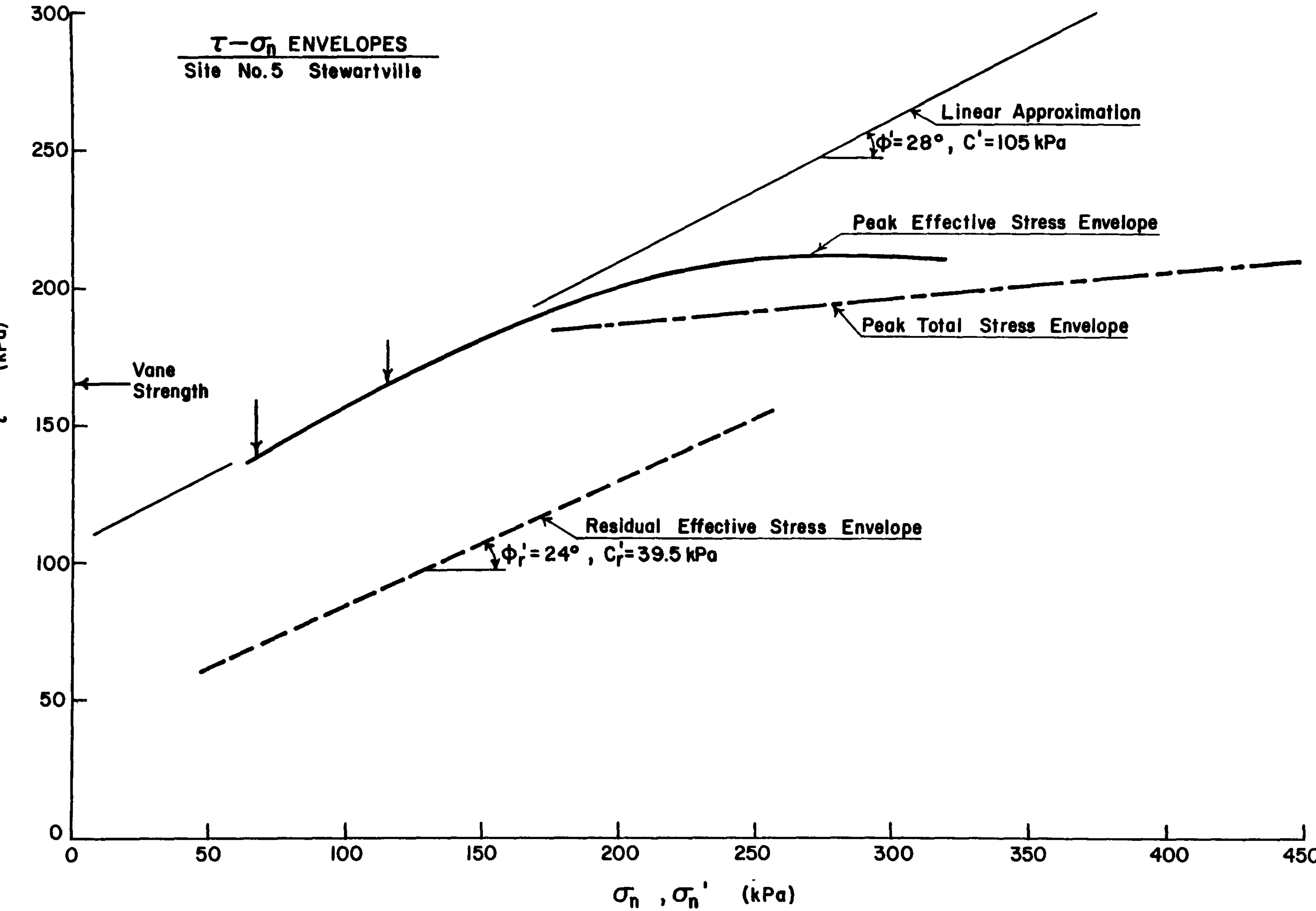

τ−σn ENVELOPES
Site No. 5 Stewartville
Linear Approximation
φ'=28°, C'=105 kPa
Peak Effective Stress Envelope
Peak Total Stress Envelope
Vane Strength
Residual Effective Stress Envelope
φr'=24°, Cr'=39.5 kPa
τ (kPa)
σn , σn' (kPa)
0
50
100
150
200
250
300
350
400
450

Figure 7.12.5

Transformation from p-q points can be performed mathematically as described by Toombs (1974) or graphically by the use of Mohr circles. The latter, more traditional approach was preferred in this thesis. The mathematical procedure is more time consuming and less certain when a curved failure envelope is involved.

Also presented in Figures 7.12.1 to 7.12.5 are the linear approximations to the curved peak failure envelope, assumed to be representative for the low effective stress range. The appropriate linear approximation of a curved failure envelope is often a matter open to debate and criticism. The necessity of using such a linear approximation for the shear strength envelope exposes a significant weakness in the present day method of slope stability analyses. It would be more logical and result in better accuracy if it was possible to input a curved failure envelope into the method of analyses, which is usually performed by computer. A computer could then easily calculate the known shear strength acting along the base of a slice according to the calculated effective normal pressure on the base of that slice. Such a computer program has been developed and discussed by Lo and Lee (1973). However, it has not been made available to the practicing engineer. Currently, work is being carried out at the University of Ottawa to develop a computer program to handle a curved failure envelope. However, until these programs become readily available and accepted, one must continue to use linear approximations.

In general, one would wish to use a linear approximation to the strength envelope over a range of normal effective stress that would be applicable to those existing in an unstable slope. Mitchell and Lawrence

(1973), Lo and Lee (1973), Lefebvre and LaRochelle (1973) and Scott et al (1976) have found that the average effective normal stress acting along a critical slip circle in a natural slope of Leda clay lies within the range of 15 to 70 kPa. However, we would naturally expect that the stronger the clay the higher the slope and consequently the higher the average effective normal stress.

The approach used to select the most appropriate linear approximation to the strength envelope for the clays tested in this thesis is as follows: A review was made of actual slope failures occurring in Leda clay and the average effective normal stress on the failure surface was noted or calculated assuming hydrostatic saturation. These average effective normal stresses acting along the failure surfaces were then related to the average field vane strength over the depth that sliding took place. It was preferred to compare the average normal effective stress on the plane of failure to the field vane strength rather than the preconsolidation pressure for these reasons: preconsolidation pressure varies with depth and an insufficient number of tests are reported in the literature to define an average value; the field vane shear strength generally does not vary significantly along the failure arc. Thus, an attempt was made to define the appropriate range of effective normal stress over which slope failures would most likely occur in Leda clays in relation to the field vane shear strength of the clay.

Table 7.12.1 presents the results of the literature survey. Although the literature survey is brief, it summarizes the results of landslides that have occurred in Leda clay with quite a range of shear strengths and throughout a wide geographic area within the former Champlain

Location	Reference	Date of Landslide	σ'_N (avg) Along Observed Failure Surface (kPa)	Su(Vane) (kPa)	$\frac{\sigma'_N \text{ (avg)}}{\text{Su(vane)}}$
Breckenridge	Lo & Lee (1973) Mitchell (1970) Crawford and Eden (1967)	April 1963	27	48	.56
St. Vallier	Lo & Lee (1973)	May 1968 May 1969 July 1969	33 24 24	43	.77 .45 .45
St. Louis	Lo & Lee (1973)	Spring 1968 Spring 1969	38 20	43	.88 .47
Tolunostoc River	Conlon (1966)		186	413	.45
Orleans	Eden & Jarrett (1971)	Fall 1965	20	67	.30
Rockcliffe	Mitchell (1970)	April 1967	38	70	.54
South Nation	Eden et al (1971) Lo & Lee (1973)	May 1971	27	43	.63

TABLE 7.12.1 Results of Literature Survey of Landslides in Leda Clay

Sea basin. The results indicate that ratio of the average normal effective stress to the undrained field vane shear strength may range from 0.3 to 0.9, with a mean of 0.55 and a standard deviation of 0.17. Thus, 68% of all the data reviewed would fall within a range of this ratio of 0.38 to 0.72. Simply, one could say that the expected average effective normal stress along a failure surface in an unstable slope of Leda clay would fall within a range of stress of 40 to 70% of the undrained field vane shear strength.

As a point of interest, if the ratio of field vane strength to preconsolidation pressure is approximately 0.3, then the expected range of effective normal stress in an unstable slope would be within a range of 0.12 to 0.21 of the preconsolidation pressure.

The linear approximation to the shear strength envelope over the applicable stress range of 40 to 70% of the vane strength is illustrated for each site in Figures 7.12.1 to 7.12.5. The computed appropriate stress range is illustrated by the two vertical arrows near the lower part of the peak effective stress envelope. The associated shear strength parameters of c' and ϕ' are relevant over a stress range generally quite larger than those given by the rule as is evident from the figures. Thus one could consider these parameters to be appropriate for most natural slope stability problems in Leda clay.

Figure 7.12.6 presents an overall view of the peak τ-σ_N envelopes for the clays tested. This figure provides one with a better appreciation of the possible range in strength envelopes in Leda clay.

Chapter 8 will discuss the significance of these results and compare them with results reported in the literature.

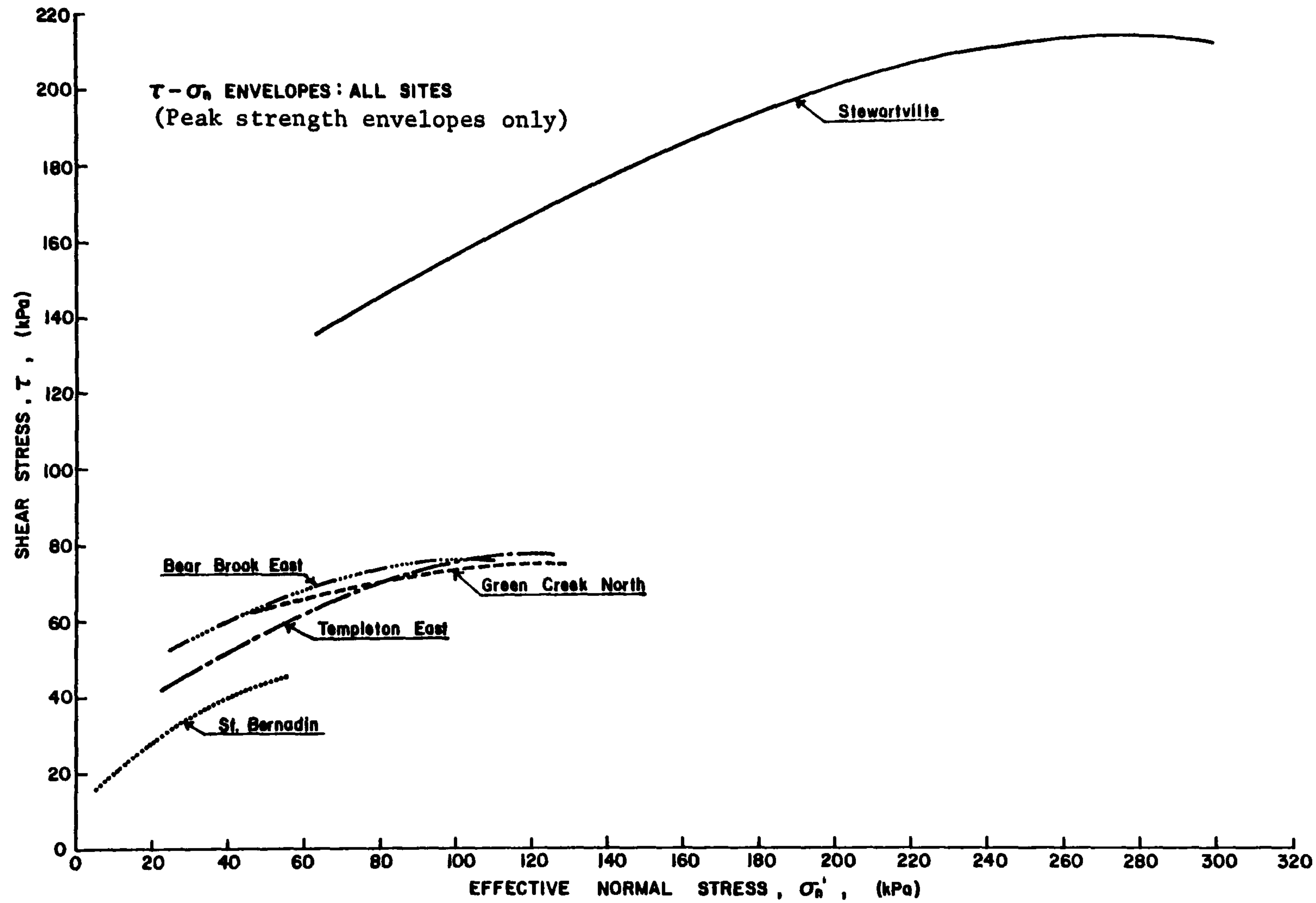

τ – σn ENVELOPES : ALL SITES
(Peak strength envelopes only)
Stewartville
Bear Brook East
Green Creek North
Templeton East
St. Bernadin
SHEAR STRESS, τ, (kPa)
EFFECTIVE NORMAL STRESS, σn', (kPa)
220
200
180
160
140
120
100
80
60
40
20
0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320

Figure 7.12.6

7.13 Summary and Conclusions

In this chapter, all the laboratory tests results performed on samples taken from the five sites described in Chapter 5, have been presented and briefly discussed. These samples represented the three different facies of Leda clay.

In summary, it may be said that although different geological processes have deposited three different facies of Leda clay, their basic geotechnical properties could be considered similar, with the exception of the shoaling prograding delta silt facies. No special distinction can be made between the basic geotechnical properties of each facies because of the similar wide range of values that may exist for each facies. A consideration of oedometer test results also revealed no special attributes to distinguish between facies.

Undrained strengths were measured by field, hand and laboratory vane tests and also by unconfined compression tests and CIU tests at negligible confining pressures. In general, good correlation was found for most test results, except for the extrasensitive, brittle clay of low plasticity from Templeton East. This clay is believed to be very easily disturbed because of its very brittle nature. The laboratory vane test provided a satisfactory estimate of the undrained shear strength of this brittle clay. Strain rate and sample size were found to affect the shear strength results at this site, but were not believed to have a significant effect at the other sites.

Isotropic consolidation of triaxial samples resulted in occasional swelling at very low confining pressures and small volume reductions (generally less than 1.5%) at higher confining pressures that were however, still in the low effective stress range. A definite yield point, at which structural breakdown and larger volume reductions occurs, was noted when the isotropic consolidation pressure reaches a value equal to approximately 50% of the preconsolidation pressure.

Stress-strain behaviour, observed for samples from all sites, can be described as being characteristic of a brittle strain softening material. A definite peak at low strains and significant reduction in shear stress at larger strains was observed. These results cast a doubt on the concept of residual strengths existing in 'overstressed' zones in natural slopes as proposed by Lo and Lee (1973).

Porewater pressures were observed to show a similar peak at low strains. Porewater pressures increased with further strain if the value of effective confining pressure p' was greater than the yield point of $p' = 0.5\ p'_c$. At this value of effective confining pressure, the structure of the clay breaks down and results in increasing porewater pressures.

By plotting increments of strain along the stress paths of a series of triaxial tests one can draw strain contour lines. Values of c' and ϕ' can be computed for individual strain contours and plots of the mobilization of c' and ϕ' parameters with strain can be made. These plots can be used to better comprehend the development of shear strength in a sample.

In general, the mode of failure most often observed was that of shear along a well defined failure plane. At very low confining pressures, vertical splitting occasionally occurred.

Although the clay mass at some sites was observed to have macrofissures, virtually all the samples tested behaved as intact materials in the low stress range. This is in contrast to the behaviour of microfissured clays as described by Eden and Mitchell (1970,1973). The peak effective and total stress envelopes intersect at a point on the unconfined compression line that may be estimated by the field vane strength. This suggests that the undrained field vane strength may be related to the peak effective stress (drained) failure envelope.

Shear stress vs. normal stress plots were presented and a linear approximation to the curved peak failure envelope was illustrated. The linear approximation was made over a range of stresses believed to be applicable to most slope failures in Leda clay, based upon the results determined from a literature survey of fåiled slopes. In addition, the residual effective stress failure envelope was also illustrated.

An examination of the behaviour of the different clay facies in isotropic consolidation and triaxial shear indicated no distinctive behaviour occurs for any particular facies of Leda clay. This fact, together with the fact that geotechnical properties of the different facies are similar in that each facies may show a wide variation of properties, suggests the conclusion that from an engineering point of view no special benefits result from distinguishing between the facies. Although it would be of interest to a geologist to classify each particular facies, a description of the material in standard engineering terms of colour, texture and strength, under the general name of Leda clay, should suffice for engineering purposes.

CHAPTER 8

ANALYSIS AND INTERPRETATION OF RESULTS

8.0 Introduction

The purpose of this chapter is to analyze and interpret the results of the shear strength tests presented in Chapter 7, in an attempt to provide a simple relationship between field vane shear strength and the effective shear strength parameters of c' and ϕ'. Such a relationship would enable land use planners and geotechnical engineers to estimate the c', ϕ' parameters for the purpose of a preliminary assessment of the stability of slopes. This estimate could be made on the basis of simple, undrained, in-situ, field vane shear tests without the need of expensive and time consuming laboratory tests. Such a method would be expected to yield more accurate and reliable results than assuming typical shear strength parameters presented in the literature with little regard to regional variations in clay strength.

This relationship however, would not eliminate the need for detailed site investigations by geotechnical consultants for important engineering projects. Detailed investigation and testing would be required to ensure that representative values of the shear strength parameters would be obtained and also to determine whether certain groundwater conditions, detrimental or beneficial to the stability of a slope, might be present (e.g., upwards or downwards hydraulic gradients).

8.1 Analysis of the Thesis Results

Table 8.1.1 summarizes the effective stress shear strength parameters that were determined from the τ-σ_N envelopes for each site, presented in Figures 7.12.1 to 7.12.5. The peak shear strength parameters were determined over a range of effective normal stress believed to be applicable to slope failures in Leda clay, according to the results of a literature survey discussed in Section 7.12. The post-peak or residual effective stress shear strength parameters apply over the full low effective normal stress range since the effective residual strength envelope is best represented by a straight line.

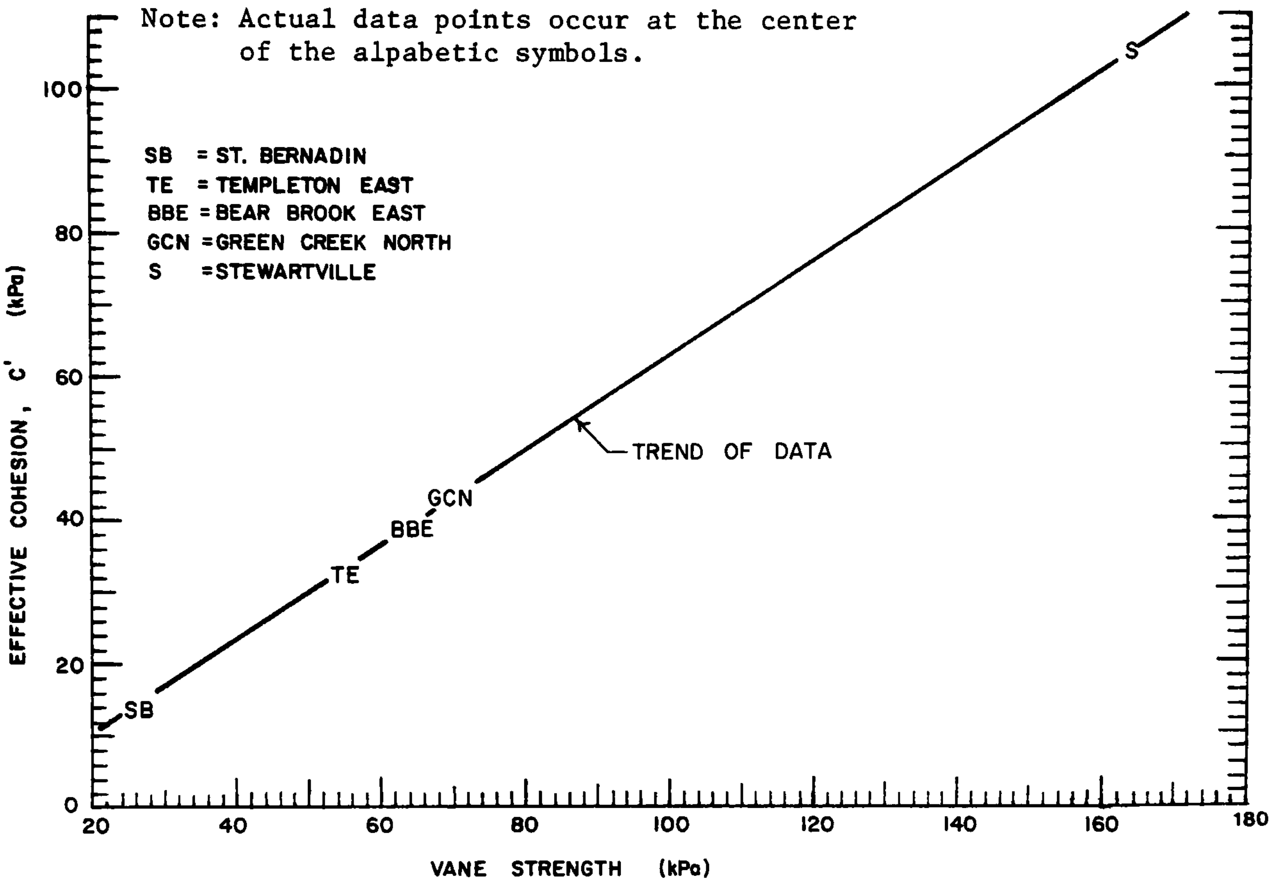
It was suggested in Chapter 7 that the undrained vane strength might be related to the effective stress or drained shear strength parameters of c' and ϕ'. Figure 8.1.1 illustrates that a relationship does indeed exist between the peak c', ϕ' parameters and the vane shear strength. The relationship between peak c' and vane strength appears to be quite linear for the five sites tested. The relationship between ϕ' and the vane strength exhibits more scatter but may be approximated by the trend line illustrated on the figure.

Figure 8.1.2 indicates that a distinct relationship also exists between the residual effective stress shear strength parameters of c'_r and ϕ'_r and vane strength. The relationship between c'_r and vane strength is linear, with only one data point falling off the line. There is a little scatter in the ϕ'_r vs. vane strength relationship, but the trend of the data appears to indicate a decrease in ϕ'_r with increasing vane strength.

Site No.	Site Name	Vane Strength (kPa)	$\frac{Su}{p'_c}$	Peak c' (kPa)	Peak ϕ' (degrees)	Residual c'_r (kPa)	Residual ϕ'_r (degrees)
1	St. Bernadin	27	.23	14	34	8	29
2	Bear Brook East	65	.25	38	29	11	26.5
3	Green Creek North	68	.37	43	23	20	32
4	Templeton East	56	.26	33	25	17	29
5	Stewartville	164	.26	105	28	39.5	24

TABLE 8.1.1 Vane Strengths and c', ϕ' Parameters for Sites Studies in this Thesis

PEAK C' vs. VANE STRENGTH (THESIS RESULTS ONLY)



PEAK Ø' vs. VANE STRENGTH (THESIS RESULTS ONLY)

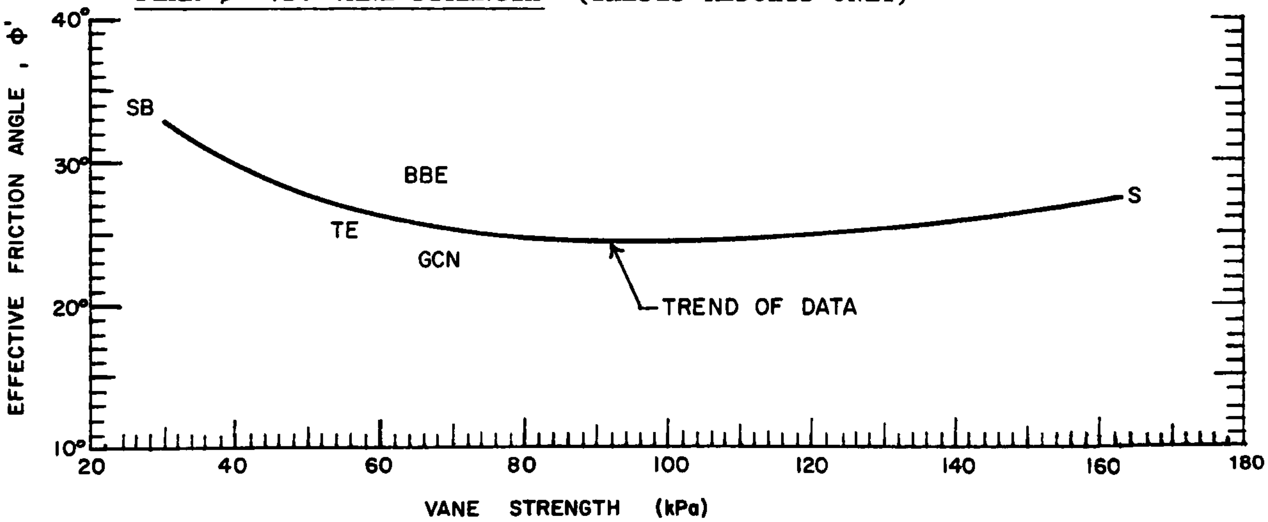
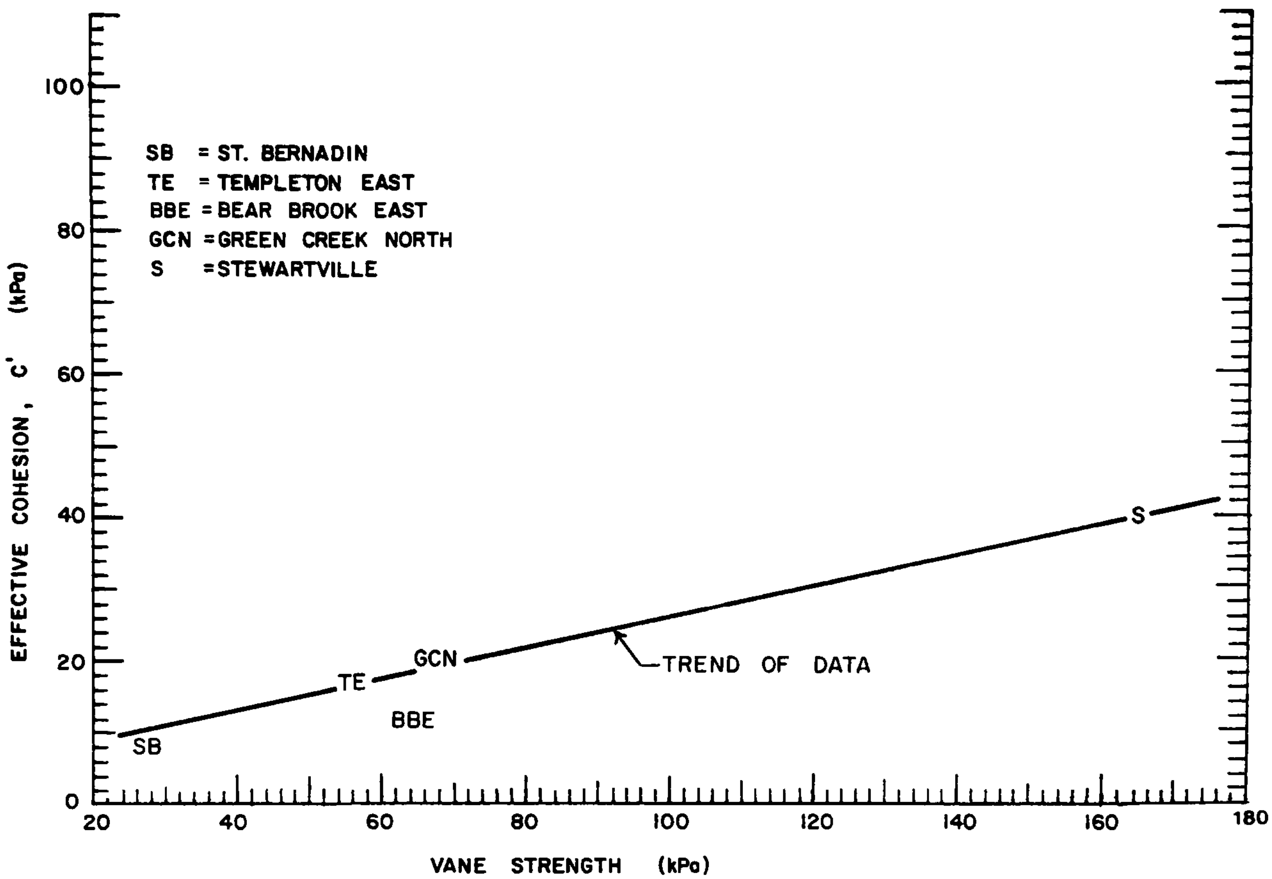


Figure 8.1.1

RESIDUAL C' vs. VANE STRENGTH (THESIS RESULTS ONLY)



RESIDUAL Ø' vs. VANE STRENGTH (THESIS RESULTS ONLY)

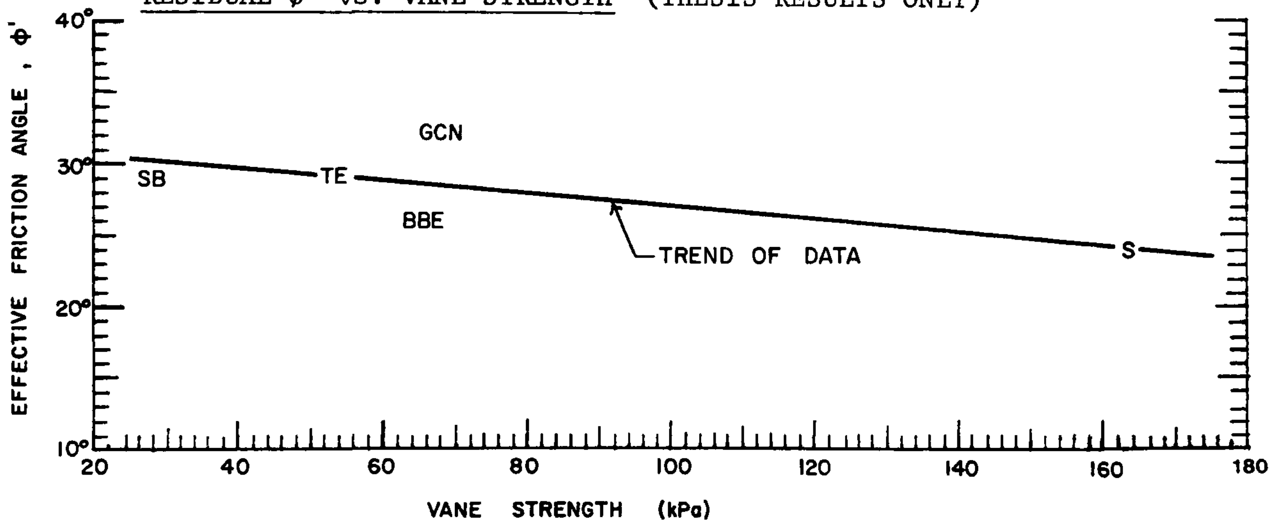


Figure 8.1.2

8.2 Comparison of the Observed Relationship from the Sites Tested in this Thesis with Other Results Reported in the Literature

As was thoroughly discussed in Chapter 3, it appears that Leda clays may be separated into two distinct types depending upon their structural integrity. These two types are intact and microfissured clays. The behaviour of these two different types of clays is different under low effective normal stresses. Microfissured clays generally have a low cohesion intercept and a high friction angle whereas intact clays generally have a higher cohesion intercept and a lower friction angle.

8.2.1 Intact Clays

In Section 7.11 it was mentioned, that although occasional macrofissures were observed in the clay mass of the sites tested, the behaviour of the triaxial samples under low effective stresses appeared to be that of an intact material. In order to provide a proper comparison of the observed relationship between vane strength and c', ϕ' parameters from the thesis test results, with other results reported in the literature or available to the author from other sources, it was necessary to choose sites where the clay was apparently intact.

Table 8.2.1 presents the c', ϕ' parameters determined from the effective stress failure envelope of six apparently intact clays from the Ottawa-Hull region and the Quebec City and Trois Rivieres locality. The references for each site are also given in the table. The effective stress failure envelope for each of these sites are presented in Figures B-1 to B-3 in Appendix B.

Site	Reference	Average Vane Strength (kPa)	$\frac{Su}{p'_c}$	Peak c' (kPa)	Peak ϕ' (degrees)	Residual c' (kPa)	Residual ϕ' (degrees)
St. Vallier Quebec	Lefebvre & LaRochelle (1973)	45	.23	40	23	7	31
St. Louis Quebec	Lefebvre & LaRochelle (1973)	45	.26	27	31	7	31
Hull, Quebec (Jumonville & DeLa Verendrye Sts)	Gourgon (1974) Scott et al (1976)	120	-	disturbed ?		21	26
Masson, Quebec (Autoroute 50)	Les Laboratoire Outaouais, Inc Report No.H-4303 (1976)	48	-	disturbed ?		10	37
Gloucester, Ont. (2.3 m to 5.3 m)	Law et al (1977)	20	.30	10	24	not available	
Kars, Ontario (7.0 m to 11.3 m)	Law et al (1977)	35	.25	14.5	25	not available	

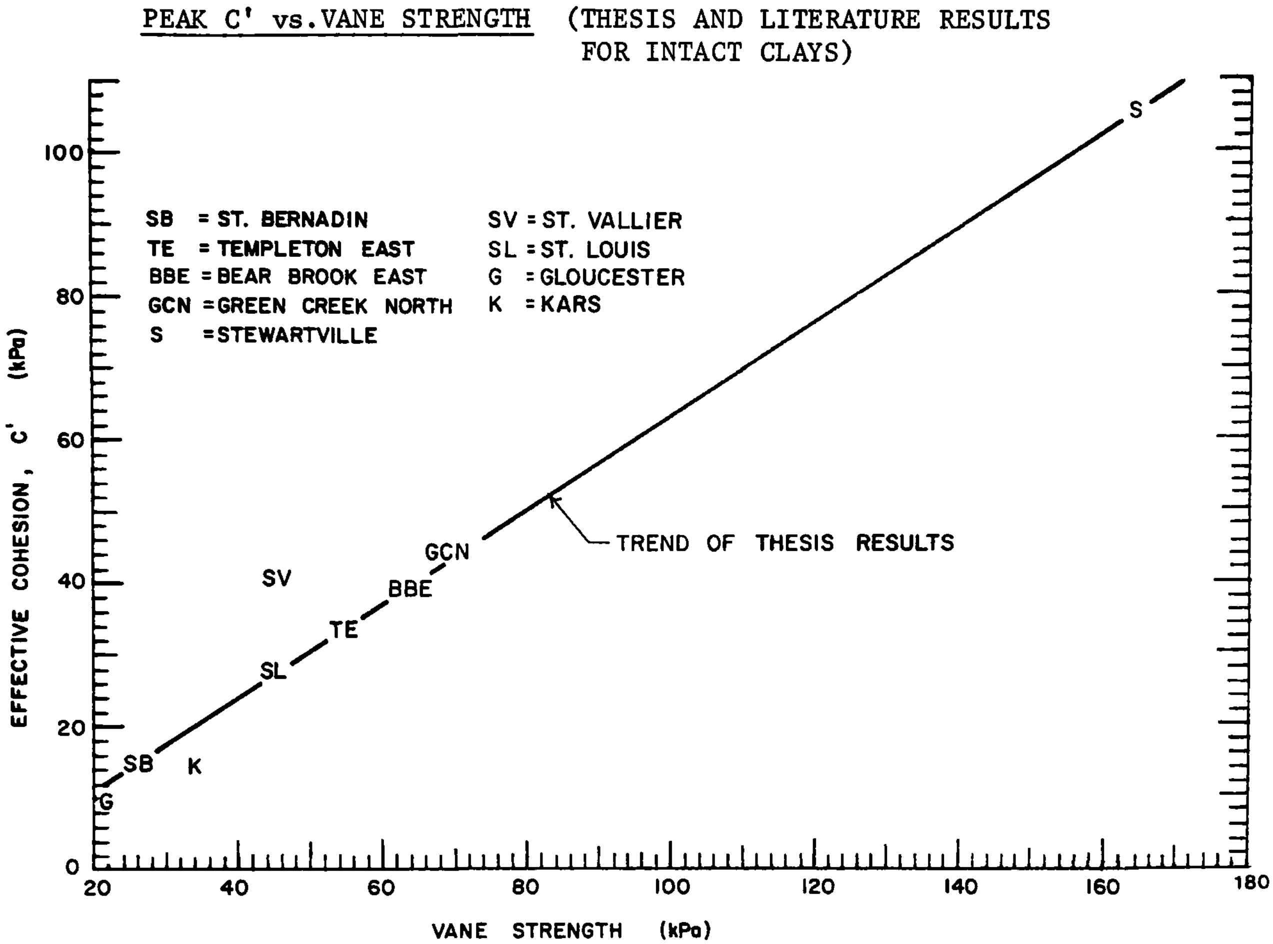
TABLE 8.2.1 Vane Strengths and c', ϕ' Parameters for Apparently Intact Clays

Figure 8.2.1 illustrates the comparison between the peak c', ϕ' parameters of the apparently intact clays tested for this thesis and from St. Vallier, St. Louis, Gloucester and Kars. All these test results were determined on samples either cut from blocks (St. Vallier and St. Louis) or good quality piston samples, 54 or 127 mm in diameter (Gloucester and Kars). The peak c', ϕ' parameters determined for the intact clay from the Hull and Masson sites were not plotted because the samples may have been disturbed since they generally were obtained by thin-walled Shelby tubes.

It would appear from Figure 8.2.1 that the peak c' vs. vane strength results reported in the literature agree quite well with the observed relationship from the thesis, plotted as a solid line. However, there is a large gap between vane strengths of 70 to 160 kPa where no data was available to confirm or negate the observed relationship.

There appears to be much more scatter in the peak ϕ' vs. vane strength relationship which may suggest that the two parameters are not clearly related, at least for peak ϕ' values.

Figure 8.2.2 illustrates the comparison of the residual c', ϕ' parameters vs. vane strength relationship of the thesis results (solid line) and the residual results reported in the literature. A glance at this figure indicates that the results reported in the literature generally fall below the observed relationship from the thesis results, but the trend of increasing c' with increasing vane strength remains the same. Also, it would appear that the residual ϕ' vs. vane strength values reported in the literature lie above the relationship observed in


PEAK C' vs.VANE STRENGTH (THESIS AND LITERATURE RESULTS FOR INTACT CLAYS)
SB = ST. BERNADIN
TE = TEMPLETON EAST
BBE = BEAR BROOK EAST
GCN = GREEN CREEK NORTH
S = STEWARTVILLE
SV = ST. VALLIER
SL = ST. LOUIS
G = GLOUCESTER
K = KARS
TREND OF THESIS RESULTS
EFFECTIVE COHESION, C' (kPa)
VANE STRENGTH (kPa)
0
20
40
60
80
100
120
140
160
180
S
GCN
BBE
SV
TE
SL
SB
K
G

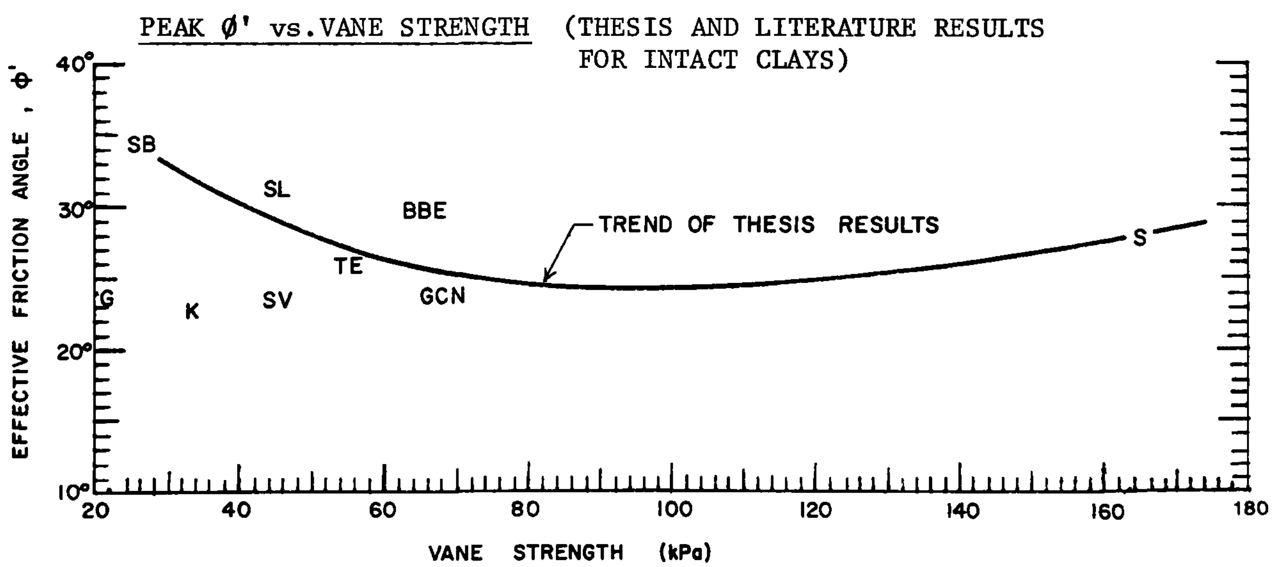
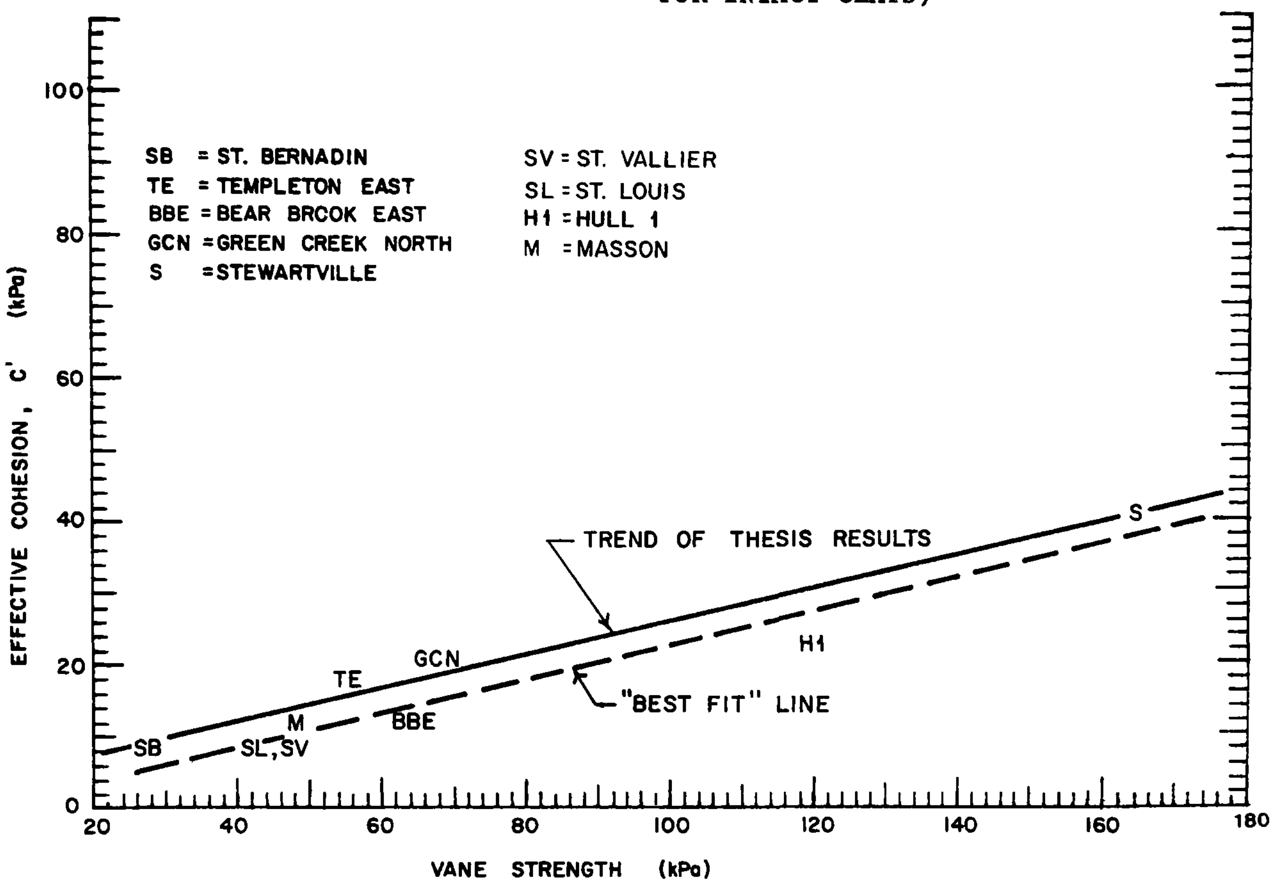

PEAK Ø' vs.VANE STRENGTH (THESIS AND LITERATURE RESULTS FOR INTACT CLAYS)
EFFECTIVE FRICTION ANGLE, Φ'
40°
30°
20°
10°
SB
SL
BBE
TREND OF THESIS RESULTS
S
TE
GCN
G
K
SV
20
40
60
80
100
120
140
160
180
VANE STRENGTH (kPa)

Figure 8.2.1

RESIDUAL C' vs. VANE STRENGTH (THESIS AND LITERATURE RESULTS FOR INTACT CLAYS)



RESIDUAL Ø' vs. VANE STRENGTH (THESIS AND LITERATURE RESULTS FOR INTACT CLAYS)

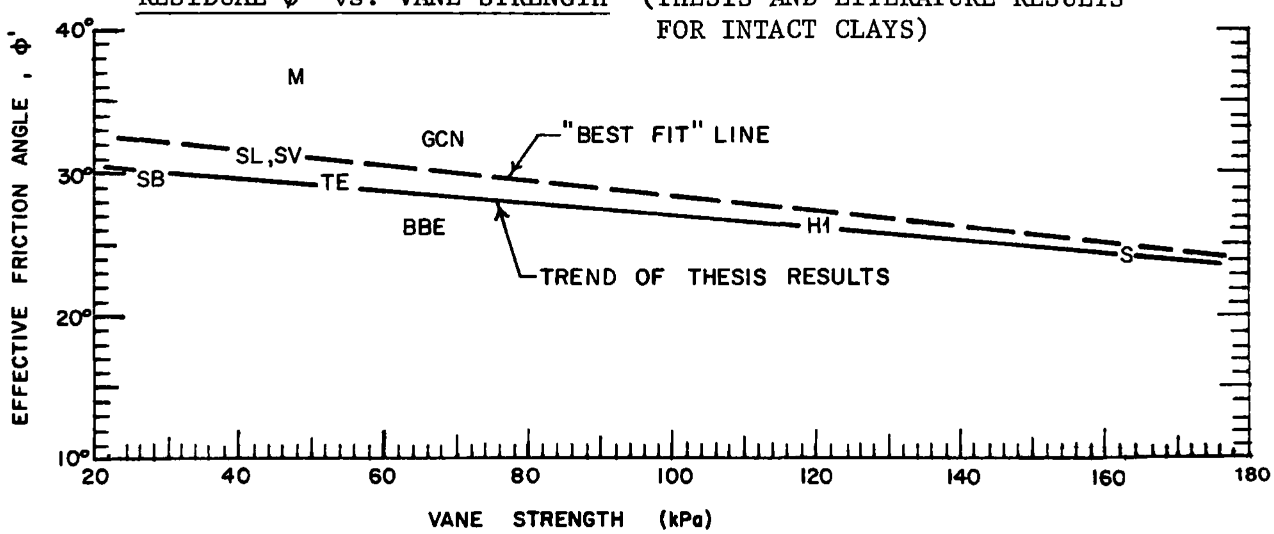


Figure 8.2.2

the thesis test results, but again the trend of decreasing ϕ' with increasing vane strength is quite evident. It would thus appear that the relationship from the thesis results could be modified to the dotted line, so as to include the results reported in the literature and reduce the scatter.

Such scatter is to be expected when one considers the significant variation in basic geotechnical properties, such as water content, grain size, density, etc., that exist between these clays from different sites and elevations. It would be expected that the more data one could obtain, the more confidence one would have in such a relationship.

An additional reason that could account for part of the discrepancy between the residual strength results from the thesis and from the literature, is that a correction factor was applied to the thesis shear strength results but not to all the literature results. This correction factor, which was discussed in Section 6.8.6c, subtracts the contribution of the filter paper and rubber membrane from the measured deviator strength. The effect of this correction factor, whose magnitude is dependent upon strain and confining pressure, is to slightly reduce the magnitude of the effective friction angle and consequently slightly increase the magnitude of the effective cohesion intercept. Thus the c' vs. vane strength relationship of the thesis results lies slightly above the apparent relationship from the literature results. The opposite is true for the ϕ' vs. vane strength relationship.

The previous discussion has indicated that for intact Leda clays there appears to be a reasonably good relationship between undrained vane shear strength and both the peak and residual effective shear strength parameters of c' and ϕ'. The review of effective stress stability analyses of Leda clay slopes in Section 3.7 indicated that the use of peak shear strength parameters results in a gross overestimate of the factor of safety of intact clay slopes. The use of residual strength parameters, however, is reported to result in satisfactory yet conservative estimates of the factor of safety [Lefebvre and LaRochelle (1973), Lo and Lee (1973)].

The relative success with the use of residual strength parameters for stability analyses in apparently intact clay slopes does not prove that only residual strengths exist in natural slopes. In fact, the test results from this thesis have indicated that even in the theoretically most overstressed zone at the toe of a natural slope, Leda clay may retain its brittle, sensitive strain softening behaviour. However, in Chapter 3 it was considered plausible to suggest that at the time of incipient failure, a strength similar in magnitude to the residual strength, defined by conventional CIU or CID triaxial tests, may be mobilized over a significant length of the failure surface due to the effects of progressive failure, anisotropy and rate of strain. It was thus concluded that since the use of residual effective stress shear strength parameters provide a conservative, yet satisfactory, estimate of the factor of safety for a preliminary analysis of the stability of a Leda clay slope, their use could be continued. The use of residual effective stress shear strength parameters is in fact an empirical solution to the problem of slope stability analyses.

Drained, stress controlled, constant p_m' triaxial tests on intact clays appear, from limited test results, to yield an effective stress failure envelope similar to the peak envelope of conventional triaxial tests. Hence, a stability analysis using the c', ϕ' parameters determined from a constant p_m' failure envelope on intact clays would also be expected to overestimate the factor of safety.

In conclusion, the relationship between vane shear strength and the residual c', ϕ' parameters of intact clays could be quite valuable to permit the rapid and easy estimation of the appropriate c' and ϕ' parameters to be used in a preliminary slope stability analysis.

8.2.2 Microfissured Clays

It was mentioned in Chapter 3 that the use of the c', ϕ' parameters, determined from the effective stress failure envelope defined by the peak strength of stress controlled, drained, constant p_m' triaxial tests, yield satisfactory estimates of the factor of safety of microfissured clay slopes. In addition, it was mentioned that, in the low effective stress range, the residual failure envelope from conventional CIU and CID triaxial tests on microfissured clays, appear to yield shear strength parameters similar to those determined from constant p_m' triaxial tests. Hence, they should also provide similarly acceptable factors of safety. Peak shear strengths from conventional CIU and CID triaxial tests on undisturbed microfissured clays yield a higher failure envelope and different c', ϕ' parameters that would not provide a realistic factor of safety.

In order to determine whether the effective shear strength parameters of c' and ϕ' for microfissured clays could also be estimated on the basis of vane shear strength, a review of the available literature was also carried out. Table 8.2.2 summarizes the values of c', ϕ' parameters determined from several effective stress failure envelopes of microfissured clays either reported in the literature or available to the author from other sources. These failure envelopes were defined by stress controlled, drained, constant p'_m triaxial tests. The references for each site are reported in the table. The effective stress failure envelope for each of these sites are presented in Figures B-4 to B-6 in Appendix B. Table 8.2.3 summarizes the values of c', ϕ' parameters determined from the effective failure envelope defined by residual strengths of conventional CIU and CID triaxial tests on microfissured clays. These failure envelopes are presented in Figures B-7 to B-8 in Appendix B.

Figure 8.2.3 illustrates the relationship of c', ϕ' parameters vs. vane strength for microfissured clays. This relationship has been determined from the effective stress shear strength envelopes of microfissured clays defined by both peak strengths from stress controlled, constant p'_m triaxial tests and residual strengths from conventional CIU and CID triaxial tests on microfissured clays.

As can be seen from the figure, the relationship between vane shear strength and the c', ϕ' parameters of the microfissured clays seems to be quite good. A definite increase in c', and decrease in ϕ', with increasing vane strength may be observed. This trend is similar to that observed for the residual c', ϕ' parameters of intact clays. The close fit of the residual strengths from CIU and CID tests on microfissured clays with the trend of the data gives further support to the proposal that in the low effective stress range the residual strengths from con-

Site	Reference	Vane Strength (kPa)	$\frac{Su}{p'_c}$	Constant p'_m c' (kPa)	Constant p'_m ϕ' (degrees)
Castor River Ontario	Toombs (1974)	38	.34	5	34
Breckenridge Quebec	Mitchell (1970a)	48	.29	5	35
South Nation River, Ont	Toombs (1974)	53	.40	10	33
Orleans Ontario	Eden and Jarrett (1971)	67	.31	11	34
Rockcliffe Ontario	Mitchell (1970a)	69	.29	12	33
Bear Brook Ontario (at Bourget)	Toombs (1974)	115	.39	23	26
Heron Road Ontario	Mitchell (1975) Michell & Wong (1973)	60	-	10	40.5

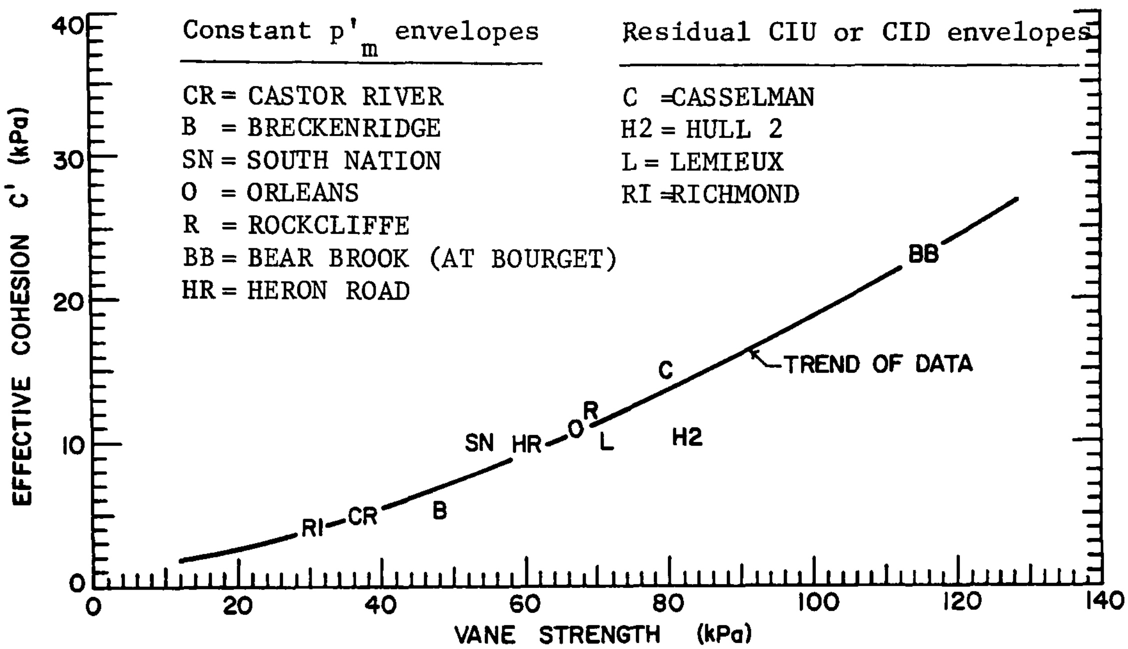
TABLE 8.2.2 Vane Strengths and Constant p'_m c', ϕ' Parameters for Microfissured Clays

Site	Reference	Vane Strength (kPa)	$\frac{Su}{p'_c}$	Residual c' (kPa)	Residual ϕ' (degrees)
Richmond, Ontario	Haile (1977)	30	0.3	4.0	37
Lemieux, Ontario	Fondex Ltd Report No. 3305-S 1974	71	-	10	34
Casselman, Ontario (Borehole 76-1-1 Sample 24)	Fransham (1978)	80	.23	14.5	26
Hull 2, Quebec (Place Concorde, Terrasse Louis Riel)	Fondex Ltd. Report No H-4082-S 1974	82	-	10	31

TABLE 8.2.3 Vane Strengths and Residual c', ϕ' Parameters for Microfissured Clays

C' vs. VANE STRENGTH (MICROFISSURED CLAYS)

Note: c', ϕ' parameters determined from effective stress envelopes defined by either peak strengths from stress controlled, constant p'_m triaxial tests or residual strengths of conventional CIU or CID triaxial tests on microfissured clays.



ϕ' vs. VANE STRENGTH (MICROFISSURED CLAYS)

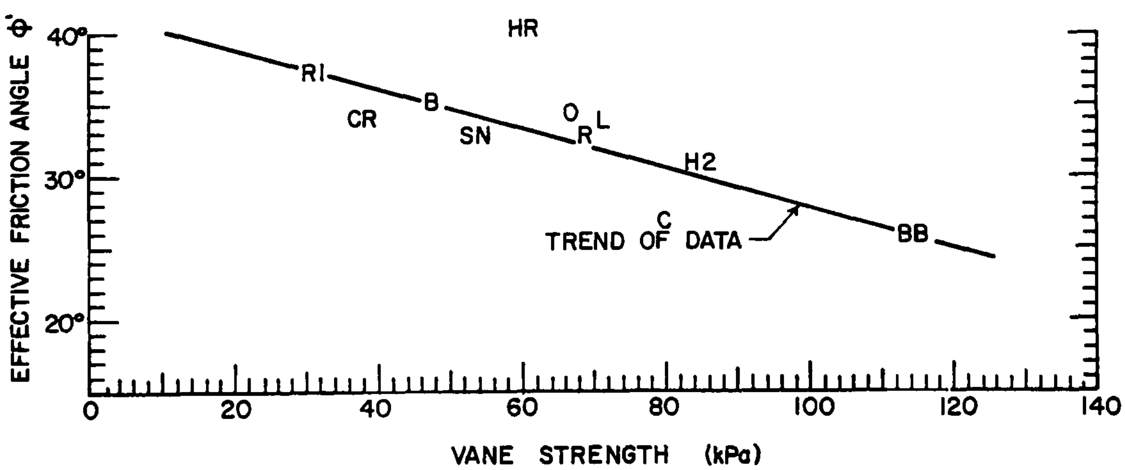


Figure 8.2.3

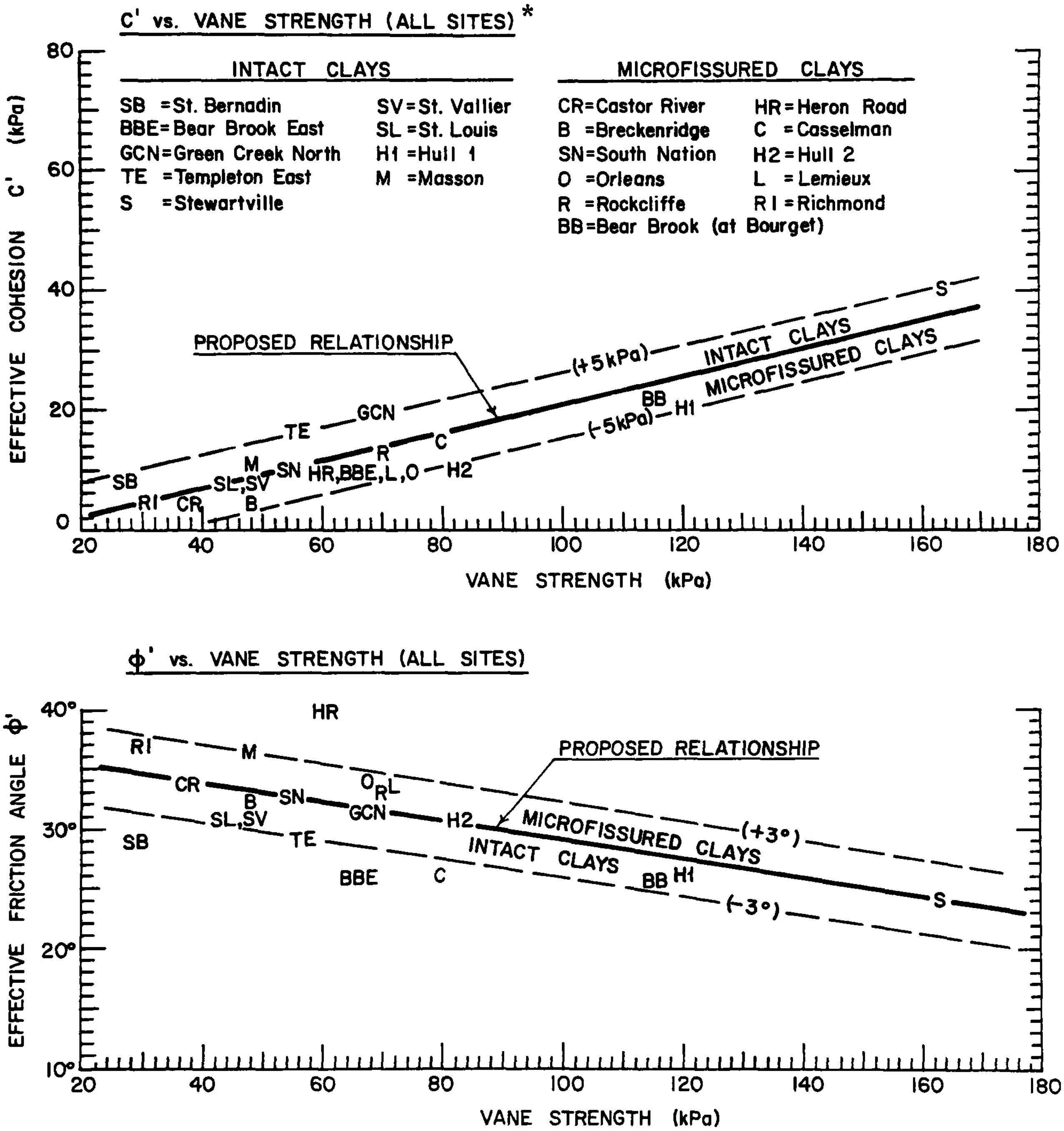
ventional CIU and CID triaxial tests yield c', ϕ' parameters similar to those of constant p'_m triaxial tests.

Up to this point, the shear strength behaviour of both microfissured and intact Leda clays have been treated separately. However, it was previously noted that the trend of the relationship between vane strength and c', ϕ' parameters appears to be similar for both microfissured and intact clays. Closer examination indicates that the magnitudes of the relationship are also similar. In the next section a relationship between vane strength and c', ϕ' parameters is proposed for Leda clay in general, based upon a combination of the c', ϕ' parameters for microfissured and intact clays of a particular vane shear strength.

8.3 Proposed Relationship Between Vane Shear Strength and the Effective Shear Strength Parameters of Leda Clay

Figure 8.3.1 combines both Figures 8.2.2 and 8.2.3. It presents the vane strength and c', ϕ' data points determined from effective stress failure envelope defined by: the peak strengths of stress controlled, drained constant p'_m triaxial tests on microfissured clays and the residual strengths of conventional CIU and CID triaxial tests on both microfissured and intact clays.

As can be seen from Figure 8.3.1, there is some scatter but the trend of the data is quite evident. The values of effective cohesion (c') increase with increasing vane strength and the values of effective friction angle (ϕ') decrease with increasing vane strength. Proposed relationships between each of the effective shear strength parameters of c' or ϕ' and vane strength are illustrated on this figure.



*Note: Actual data points occur at the center of the alphabetic symbols.

Figure 8.3.1

The proposed relationship between c' and vane strength was determined by a least squares linear regression analysis of all the data from 20 sites. The computed correlation coefficient was 0.91 which is quite acceptable when one considers the difference between intact and microfissured clays and also the variation in other basic geotechnical properties.

The proposed relationship appears to separate intact and microfissured clays, although there are exceptions to this general classification. As one would expect, the apparently intact clays exhibit higher values of effective cohesion and lower values of effective angle of internal friction. The reverse is true for microfissured clays.

Much more scatter is evident about the ϕ' vs. vane strength relationship. The proposed relationship is again a regression line, and appears to be a good fit, however the correlation coefficient of all the data is only -0.63, which indicates significant scatter. However, the results of a statistical analysis are often influenced by the extremes of the data and can yield misleading results. Confidence in the proposed relationship may be gained by a consideration of the fact that 16 out of a total of 20 sites (80%) fall within a range of 3 degrees of effective friction angle on either side of the proposed relationship. Since the results of stability analyses in Leda clay are known not to be very sensitive to small changes in friction angle the proposed relationship is considered to be quite acceptable.

The proposed relationship can be used if the investigator is uncertain whether the clay is intact or microfissured. This might often be the case if the stability of a slope is investigated by means of field vane tests only, without taking samples. If the investigator is certain of the structural integrity of the clay, he could choose values higher or lower than the proposed relationship based on personal judgement.

If the proposed relationship was used, then according to the data analyzed, a maximum error in estimation of c' of only 5 kPa (0.72 psi) would result, if the particular clay was at the limits of data for either microfissured or intact clays. A probable error in the estimation of ϕ' would be less than 3 degrees. These probable maximum errors are considered acceptable for the purposes of a preliminary estimation of the stability of a slope.

The proposed relationship yields values of c' and ϕ' that agree quite well with what is normally used in stability analyses for slopes in Leda clay. The normal range of field vane strength for most Leda clays subject to instability problems is from 30 to 80 kPa. Over this range of vane strength, the proposed relationship would yield values of c' ranging from 4 to 16 kPa and ϕ' values ranging from 34.5° to 31°. Since these typical values are known to yield satisfactory factors of safety according to reports in the literature, it is anticipated that the proposed relationship could be used with good success for most Leda clay slopes.

8.4 Conditions For Which the Proposed Relationship Will Not Apply

It is important to realize that the proposed relationship has been developed from tests performed on natural, undesiccated Leda clays whose strength is thought to be the result of overconsolidation and natural cementation. Attempts to use this relationship in a weathered, oxidized, stiff crust, where the strength has been altered by the desiccation process, may not provide satisfactory c', ϕ' parameters to be used in a stability analysis.

Two examples will illustrate this point. The first example is discussed by Lafleur (1978) who determined residual effective shear strength parameters from conventional CIU and CID triaxial tests of $c' = 2.8$ kPa and $\phi' = 40°$ on samples of a stiff (vane strength in excess of 70 kPa) brown, fissured Leda clay crust taken from a slope on Normandie Street in Hull, Quebec. The liquidity index in this material was about 0.4. The second example is described by McRostie and Associates, Report No. SF-1779, 1974, and concerns the Le Coteau slope in Gatineau, Quebec. They report effective shear strength parameters of $c' = 5$ kPa and $\phi' = 43°$ for a stiff to very stiff grey clay with some silt. The liquidity index in this material was about 0.6. Porewater analyses indicated that the material tested had a Na/Ca ratio of less than 5 which indicates that clay chemistry is quite different from the normal marine chemistry of most Leda clays (Sangrey and Paul,1971).The field vane strength is not precisely known but is believed to be in excess of 100 kPa.

When these examples are plotted on Figure 8.3.1 they fall significantly below the proposed relationship between c' and vane strength, and significantly above the proposed relationship between ϕ' and vane strength. The effect of weathering and desiccation appears to decrease the cohesion and increase the angle of internal friction, essentially causing the clay to behave as a granular material. Furthermore, vane shear tests in a stiff, desiccated crust are most probably unreliable since the material would probably not fail along a cylindrical surface as assumed.

Slope failures at these sites are different from those normally observed in Leda clay. These failures generally do not involve the toe and are not related to erosion by streams (Lefebvre et al,1976). They are generally quite shallow and are limited to the upper crustal material. The failures could simply be described as surficial sloughing of the stiff, macrofissured crust due to the steepness and height of the overall slope during conditions of high groundwater levels.

In summary, the proposed relationship cannot be used to estimate the c', ϕ' parameters of a stiff, desiccated, macrofissured crust because the vane shear test would not be reliable and the effective shear strength parameters have been altered from the natural state.

8.5 Recommended Procedures to be Followed for a Preliminary Estimate of the Stability of a Slope

This section outlines the procedures that should be followed to obtain an estimate of the stability of a slope, without the need for time consuming and expensive field work and laboratory tests. Such an estimate could be considered satisfactory for the purpose of land use planning and development or as a preliminary assessment of the stability of a slope for feasibility studies of engineering projects. However, the final assessment of the stability of a slope for important engineering projects should be based upon more detailed field work, laboratory testing and sophisticated forms of stability analyses.

8.5.1 Preliminary Research

Prior to any site specific work, the investigator should review all available documentation concerning the locality surrounding the slope or slopes in question. This documentation might include surficial and bedrock geology maps, hydrologic studies, aerial photographs and possibly geotechnical reports and publications. This preliminary information could be highly beneficial in determining the field and laboratory work to be carried out, factors influencing stability, and the best method of analysis.

8.5.2 Site Survey

A site survey should then be carried out to study the area in question. A careful investigator should search for evidence of previous slope failures, bedrock outcrops, water seeping out of the slope, fissures and sand or silt seams. If a stream or river is located at the toe, the investigator should study the possibility of erosion at the toe and look for signs of maximum and minimum water levels of the stream.

In addition the slope geometry should be measured and the optimum location of boreholes should be determined.

8.5.3 Field Work

A proper program of field work would involve the following:

a) At least one borehole should be put down. The most information on the soil stratigraphy of the slope would be obtained if this borehole was put down in the vicinity of the crest of the slope. However, this may result in a deep borehole that must first penetrate through overlying sediments or crustal material. Hence, a borehole at midslope or at the toe may be more economical.

The use of the relationship established for Leda clay between the effective shear strength parameters of c' and ϕ' and undrained vane shear strengths, measured at the locations from which the samples were taken for the laboratory determination of the effective shear strength parameters, does not require that the vane shear test be performed at a particular location in the slope.

Triaxial tests on block samples obtained from the toe of the slope for this thesis have indicated that the clay has retained its brittle, sensitive characteristics even in an apparently highly stressed zone of the slope. Hence, it would not be expected that a significant difference in vane strength, hence c', ϕ' parameters of a particular strata would result solely due to a difference in location in the slope at which the vane shear tests were performed.

b) These boreholes could be easily put down with a hand or motorized auger, that should preferably be at least 7 cm in diameter. They should penetrate at least 2 to 3 m below the toe of the slope.

c) Field vane shear tests should be performed at intervals of about 0.3 to 0.5 m according to ASTM Standards (D2573-72). Remoulded vane shear tests should also be performed, in the manner outlined in this designation, to determine the sensitivity of the clay. If the clay has a sensitivity greater than 10, good quality undisturbed samples, such as block or piston samples, will be required to verify the undrained shear strength of the clay in the laboratory.

d) Soil samples should be taken at about the same depth as the vane tests, and placed in air tight containers to prevent drying.

e) Piezometers should be inserted into the open boreholes to monitor groundwater levels. It would be advisable to place several piezometers, sealed at different elevations, in the deepest boreholes to check for downwards or upwards hydraulic gradients in the slope. It would also be advisable to place piezometers at other locations in the slope, possibly at the midslope and at the toe, if boreholes were put down at these locations. These additional piezometers would allow a flow net to be properly drawn. The water levels in these piezometers should be monitored through the most wet and most dry seasons.

8.5.4 Laboratory Work

The following laboratory tests could be performed on selected samples to determine the index properties of the clay: water contents, Atterberg limits, specific gravity and possibly grain size distribution.

If good quality tube or block samples were obtained, the undrained shear strength of the clay could be verified by means of laboratory vane tests, unconfined compression tests and CIU triaxial tests at low confining pressures.

8.5.5 Effective Stress Stability Analysis

Prior to performing an effective stress stability analysis, it is necessary to finalize the groundwater conditions and choose representative values of the effective stress shear strength parameters of c' and ϕ'.

The groundwater conditions can be easily finalized by drawing a flow net based upon the observed groundwater levels. The most detrimental conditions to the stability of the slope should be assumed. Representative values of c' and ϕ' can be determined using the proposed relationship of Figure 8.3.1 according to the following discussion.

If the undrained vane strength of the clay deposit at a particular site is relatively uniform with depth, then the average vane strength can be used to determine c', ϕ' parameters directly from the proposed relationship and then used in the stability analysis.

If the undrained vane strength profile increases linearly with depth, then several possibilities exist concerning the use of the proposed relationship. First, the minimum vane strength can be used to determine appropriate c', ϕ' parameters. If the factor of safety is satisfactory using minimum values then no further work need be done. If it is not satisfactory then an average value of vane strength may be used to estimate average c', ϕ' parameters and the stability evaluated. An alternative and probably more accurate approach would be to divide the soil into an arbitrary number of layers depending upon the range in vane strength and the anticipated depth of possible sliding, and then, by using average values of vane strength for each small layer, an estimate of average values of c', ϕ' parameters for each layer could be made and then used in the stability analysis.

The proposed relationship would be especially valuable for stability analysis of clay deposits of several different layers. Average values of undrained vane strength for each layer could be used to easily estimate c', ϕ' parameters for individual strata rather than performing numerous time consuming and expensive laboratory tests.

If the structural integrity of the clay at the site in question is not known, then the proposed relationship between vane strength and c', ϕ' parameters should be used. If the investigator is confident as to the structural integrity of the clay, he may choose appropriate values of c' and ϕ' based upon the range of values illustrated in Figure 8.3.1.

The stability analysis could be performed by using either chart solutions or computer techniques. In any case, it would be of interest to vary the c' and ϕ' parameters within the range of values illustrated in Figure 8.3.1, to study the variation of the factor of safety and ensure a realistic factor of safety has been obtained. In addition, the groundwater conditions could also be varied to study their affect on the factor of safety. From this small parametric study the investigator could obtain more confidence in a particular value of the factor of safety.

A probabilistic approach to the estimation of the factor of safety could also be easily performed by inputting the value of the proposed relationship as a mean value and using the range of possible values of c' and ϕ' illustrated in Figure 8.3.1 for a particular vane strength.

The c', ϕ' parameters used in the analysis as determined from the proposed relationship could be checked by back calculating the c', ϕ' parameters mobilized along any previous slope failures in the vicinity.

The choice of an adequate factor of safety for the classification of stable slopes depends upon the uncertainty in the knowledge and variation of the undrained shear strength and groundwater conditions. It also depends upon the confidence of the investigator in the proposed relationship. It is suggested that a minimum factor of safety of 1.2 could be used as a classification of stable slopes provided reasonable groundwater conditions and undrained shear strengths have been used.

8.6 Summary and Conclusion

Proposed then is a relationship by which a preliminary assessment of the stability of a slope can be made on the basis of simple field vane tests and measured or assumed groundwater conditions. The undrained shear strength measured by a properly performed field vane test can be used to estimate the effective stress shear strength parameters of c' and ϕ' required for a stability analysis.

The proposed relationship is based upon the use of c', ϕ' parameters determined from stress controlled, constant p'_m triaxial tests on microfissured clays and upon c', ϕ' parameters from the residual strengths of conventional CIU and CID triaxial tests on microfissured and intact clays. The proposed relationship however, does not require that a distinction be made between microfissured and intact Leda clays to provide a good estimate of the stability of a slope. This relationship yields c', ϕ' parameters that are typical of those commonly determined or assumed for most Leda clays and which are reported to yield satisfactory factors of safety.

This relationship may be used by the geotechnical consultant to provide a preliminary assessment of the stability of a slope. It may also be used by land use planners and developers to provide estimates of the stability of slopes of a more general nature carried out on a regional scale. Detailed in-situ soil investigations would still be required at sites of important engineering projects however, to determine the applicable shear strength parameters more precisely and also to determine the presence of detrimental or favourable groundwater conditions.

CHAPTER 9

CONCLUSIONS

9.0 Introduction

This chapter summarizes the accomplishments and lists the principal conclusions of this research project. Conclusions from the literature review and other less important but nonetheless noteworthy conclusions are also listed. Recommendations and suggestions for future work are also provided.

9.1 Summary of Accomplishments

1. A relationship has been proposed by which the effective stress shear strength parameters of c' and ϕ' may be estimated on the basis of simple, undrained field vane shear tests. It is believed that this relationship will prove useful in the preliminary assessment of the stability of a slope using either measured or assumed groundwater conditions.
2. The current literature concerning the aspects of shear strength and stability analysis of Leda clay slopes has been reviewed. An attempt has been made to explain the conflicting results and opinions reported in the literature concerning these aspects of the behaviour of Leda clay.
3. A review of the current literature concerning the Pleistocene geology of the Ottawa River Valley region has been provided. Special emphasis has been placed on the discussion of the geology of Leda clay.

4. The geotechnical properties and engineering behaviour of the different facies of Leda clay have been described and discussed.

5. General rules of thumb have been provided to estimate:

a) the limit of the low effective stress range for Leda clays, since testing above this stress range is not required for slope stability analyses,

b) the range of effective normal stress over which a linear approximation to a curved failure envelope may be made, in order that the determined strength parameters would be applicable to slope failures in Leda clay.

6. Five new effective stress failure envelopes have been added to the literature.

7. A computer program has been written to perform the calculations and plot the results of undrained and drained triaxial tests.

In brief, the objectives of the research, as originally outlined in Chapter 1, have been achieved.

9.2 Principal Conclusions of the Research Project

1. A relationship has been proposed by which the effective stress shear strength parameters of c' and ϕ' may be estimated on the basis of simple, undrained field vane shear tests. This relationship is based upon c', ϕ' parameters determined from effective stress failure envelopes that were defined by the peak strengths of stress controlled, drained constant p'_m triaxial tests on microfissured clays and the residual

strengths of conventional CIU and CID triaxial tests on microfissured and intact clays. However, the relationship does not require that a distinction be made as to whether the clay is intact or microfissured in order to provide a good estimate of the c' and ϕ' parameters. The data used to develop the relationship includes many sites where the c', ϕ' parameters have been used successfully in effective stress stability analyses.

2. At least three different facies of Leda clay have been identified in the Ottawa River Valley region that are of interest to geotechnical engineers. These facies are the result of different environments of deposition that occurred during the recent geological history of the Ottawa River Valley region. Each of these facies exhibits a similar wide range of basic geotechnical properties that prevents categorization of each facies by average values. The only significant difference between the facies is that there is a trend of increasing grain size upwards in the more coarse layer of each facies. This reflects the increasing energy level as the environment of deposition changed from quiescent deep water marine conditions to more turbulent fluivial-deltaic conditions. The behaviour of each facies in triaxial shear and one dimensional consolidation appears to be similar.

Thus, no average properties or distinctive behaviour can be attributed to any particular facies. Hence, it is not necessary for a geotechnical engineer to distinguish between Leda clay facies for the purpose of slope stability analyses. Description of the clay deposit in standard engineering terms should suffice.

3. High quality block samples taken at the bottom or toe of several natural slopes have indicated that the Leda clay sampled has retained its brittle, sensitive characteristics even though it is in a highly stressed zone. Well defined peak strengths with significant post-peak reduction in strength was observed. This casts a doubt on the concept that only residual strengths exist in the highly stressed zones of natural slopes.

However, it is still plausible to propose that a strength similar in magnitude to the residual strength, defined by conventional CIU and CID triaxial tests, is mobilized at the time of incipient failure in a slope. This might be the result of anisotropy, time effects and progressive failure.

4. A value of $p' = \frac{\sigma_1' + \sigma_3'}{2}$ equal to 50% of the preconsolidation pressure appears to define the boundary between the low effective stress region and the intermediate stress region for most microfissured clays, according to a review of effective stress failure envelopes in the literature. The same rule is believed to be appropriate for intact clays also, but the transition from the low to intermediate effective stress range is more difficult to define from the effective stress failure envelope of an intact clay.

When the value of $p' = 0.5\ p_c'$, significant structural breakdown occurs, as was observed during isotropic consolidation and triaxial shear. Thus, the point at which $p' = 0.5\ p_c'$ indicates the start of yield. During yielding, when $p' > 0.5\ p_c'$, significant volume reduction or increase in porewater pressure occurs.

It is therefore suggested that a value of $p' = 0.5\ p_c'$ should be considered as the upper limit of confining pressure required for a triaxial testing program for a slope stability analysis.

5. Approximately 70% of all slope failures in Leda clay appear to have an average value of normal effective stress on the surface of sliding that falls within a range of 40 to 70% of the undrained vane shear strength at the depth of sliding. This rule of thumb is the result of a review of a number of published reports of landslides in Leda clay. By using this rule of thumb, a linear approximation to a curved failure envelope may be drawn over a range of stresses believed to be applicable to most instability problems in Leda clay.

The range of normal effective stresses from 40 to 70% of the undrained vane strength would correspond to values of 0.12 to 0.21 of the preconsolidation pressure. Hence, an upper limit of effective confining pressure of 0.5 p_c' for a series of triaxial tests would appear adequate for the purpose of slope stability analyses in Leda clay.

6. Volume changes were observed to be less than 1.5% for all sites during isotropic consolidation at pressures less than 0.5 p_c'. It is suggested that similar volume changes should be expected in other testing programs. Volume changes in excess of 1.5% would probably either indicate that the effective confining pressure was in excess of 0.5 p_c' or that the sample may be disturbed.

7. It would appear that the field vane shear test provides a satisfactory measure of the undrained shear strength except for clays with a sensitivity in excess of 10. This is probably the result of disturbance during insertion of the field vane into the soil. For these

extrasensitive clays the undrained shear strength should be verified by laboratory vane tests using a thin bladed (less than 0.5 mm) vane and also by unconfined compression tests and CIU triaxial tests at low confining pressures.

8. The field vane shear strength indicates the stress level at which the peak effective stress envelope of an intact clay intersects the unconfined compression line. If the clay is microfissured, the field vane strength indicates the strength of the cementation bond in the intermediate stress region. If the clay is definitely intact and peak failure points fall below the field vane strength, sample disturbance is indicated.

9.3 Conclusions from the Literature

1. The many conflicting reports and opinions in the literature concerning the aspects of measured shear strength of Leda clay and its correct application to slope stability analyses appear to be the result of two major and interrelated factors:

a) Two structurally different clays have been tested and it is not correct to try and compare the results of each. One clay is apparently intact and the other is microfissured.

b) Two different test procedures have generally been used for each clay type. The test procedure may significantly affect the behaviour of the clay in shear and consequently the final measured shear strengths.

2. Effective stress stability analyses using c' and ϕ' parameters determined from an effective stress failure envelope, defined by stress

controlled, drained constant p'_m triaxial tests, yield factors of safety close to unity for failed slopes in microfissured clay.

The stress path followed by conventional CIU or CID triaxial tests seems to prevent the clay from dilating prior to failure and thereby results in a peak strength failure envelope that lies above the failure envelope determined from constant p'_m triaxial tests. Hence, stability analyses using c', ϕ' parameters defined by the peak strength failure envelope of conventional CIU or CID triaxial tests on microfissured clays would probably overestimate the factor of safety of a failed slope in this material.

The effective shear strength parameters of c' and ϕ' determined from the residual strength failure envelope of conventional CIU and CID triaxial tests are similar, in the low effective stress range, to the c', ϕ' parameters defined by the peak constant p'_m failure envelope. Consequently, it would be expected that the residual strength parameters would yield satisfactory, although probably conservative, factors of safety for microfissured clays.

3. The use of peak effective stress shear strength parameters of c' and ϕ' from conventional CIU and CID triaxial tests results in a gross overestimate of the factor of safety for failed slopes in intact clay. The use of residual strength parameters result in satisfactory factors of safety for failed slopes in intact clay.

Little information exists concerning constant p'_m triaxial tests on intact clays, but preliminary information indicates that stress controlled, drained constant p'_m tests define a failure envelope that is equivalent to the peak effective failure envelope. Hence, it would be expected that they also would result in an overestimate of the factor of safety of intact clay slopes.

4. It would appear that the use of c', ϕ' parameters determined from the residual effective stress envelope of conventional CIU and CID triaxial tests would provide a satisfactory preliminary estimate of the stability of a slope, although probably conservative, in either intact or microfissured clays. However, this does not imply that the use of residual strengths is theoretically correct. Residual strengths are rather an empirical solution to slope stability analyses in Leda clay.

5. The effects of sample size, sample disturbance, test procedure, anisotropy, time and progressive failure would have to be considered if the peak strength parameters from conventional CIU and CID triaxial tests were used in a slope stability analysis. Failure to do so may result in an unsafe design.

9.4 Additional Noteworthy Conclusions

1. Attempts to use the proposed relationship between vane strength and c', ϕ' parameters in a weathered oxidized, stiff crust may not result in satisfactory c', ϕ' parameters to be used in a stability analysis. This would be the result of the poor measurement of the undrained shear strength by the field vane test and also due to the alteration of the natural strength of Leda clay by oxidization, desiccation and possibly leaching.

2. CIU triaxial tests with porewater pressure measurements are

the best suited to obtain the residual strength failure envelope. In addition, CIU triaxial tests may also provide valuable peak strength information as well as total strength values. This type of test may be performed easily and rapidly. It is also amenable to automation and computer analysis of data.

3. If peak strengths are desired, a failure criterion must be chosen for undrained triaxial tests. The criterion of maximum principal stress difference is more feasible for Leda clays than the criterion of maximum principal effective stress ratio, which may not reach a maximum if porewater pressures increase with further strain after the peak. The criterion of maximum effective stress ratio could be used however, if the effective confining pressure in a test remains below a value of $p' = 0.5\ p'_c$.

4. The different geological facies of Leda clay are not restricted to certain elevations. Leda clays of the same facies may be found at significantly different elevations.

Preconsolidation pressures determined for samples from each facies are generally the result of previous overburden pressures but occasionally may be the result of downward hydraulic gradients or apparent cementation. Preconsolidation pressures are not related to elevation over a wide geographic area.

5. Total stress stability analyses in Leda clay are unreliable. This is primarily due to the inability to properly estimate the mobilized shear strength along a failure surface due to the effects of fissures, anisotropy, reduction in strength with time and progressive failure.

Furthermore, groundwater conditions which are known to have an integral effect on the stability of Leda clay slopes are ignored.

6. In general, the mode of failure observed in the CIU tests was that of shear along a well defined failure plane. At very low confining pressures, vertical splitting occasionally occurred and may have been the result of the effect of end restraint which can induce tensile stresses in the center of a specimen.

7. Plots of the mobilization of c' and ϕ' with strain are helpful in understanding the contribution of each of these parameters to the overall shear strength.

8. The procedure of taking block samples by the use of a chain saw as described in Chapter 6, appears to be the most practical way of obtaining high quality samples.

9.5 Recommendations and Suggestions

Following are some recommendations and suggestions based upon the results of this research project.

1. More results are required to verify the proposed relationship between vane strength and c', ϕ' parameters. The more results there are, the more confidence one would have concerning the reliability of the proposed relationship.

2. The applicability and value of the relationship needs to be verified by experience in attempting to estimate the stability of a slope based upon field vane strengths alone.

3. Improved methods of estimating groundwater conditions without

the need of an expensive piezometer installation or monitoring programs are required. Such an estimation of anticipated groundwater conditions might be possible by a consideration of stratigraphy, bedrock conditions and slope geometry. A better estimation of groundwater conditions would be expected to improve the degree of success of the proposed relationship.

4. More work needs to be done on the nature, cause and effect of both micro and macrofissures on the shear strength of Leda clay. It is quite important to determine whether microfissures actually exist in-situ or whether they are a product of the effects of sampling, stress relief, storage or induced shear forces.

5. More constant p'_m triaxial tests should be performed on samples of intact clay, to study the effect of test procedure on the measured shear strength and volume changes.

6. The present methods of slope stability analysis needs to be changed to incorporate a curved failure envelope. If a computer program is developed to handle the curved failure envelope, it should be made available to geotechnical consultants.

7. Further investigation into the concept of a yield point occurring at a value of $p' = \frac{\sigma'_1 + \sigma'_3}{2}$ = 50% of the preconsolidation pressure would be of interest.

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APPENDIX A

MISCELLANEOUS LABORATORY TEST RESULTS

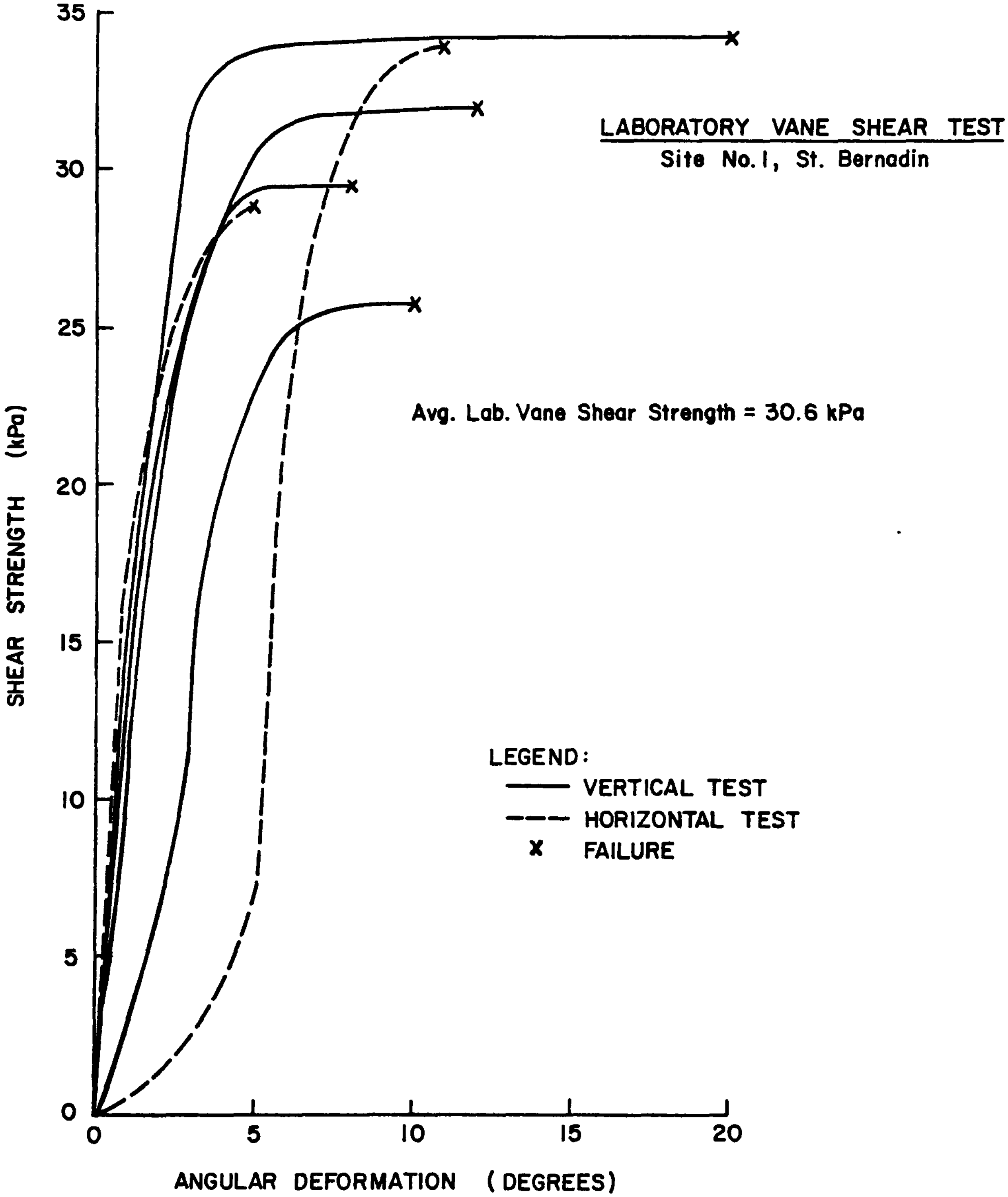

LABORATORY VANE SHEAR TEST
Site No. 1, St. Bernadin
Avg. Lab. Vane Shear Strength = 30.6 kPa
LEGEND:
VERTICAL TEST
HORIZONTAL TEST
X FAILURE
SHEAR STRENGTH (kPa)
ANGULAR DEFORMATION (DEGREES)
0
5
10
15
20
25
30
35
0
5
10
15
20

Figure A1

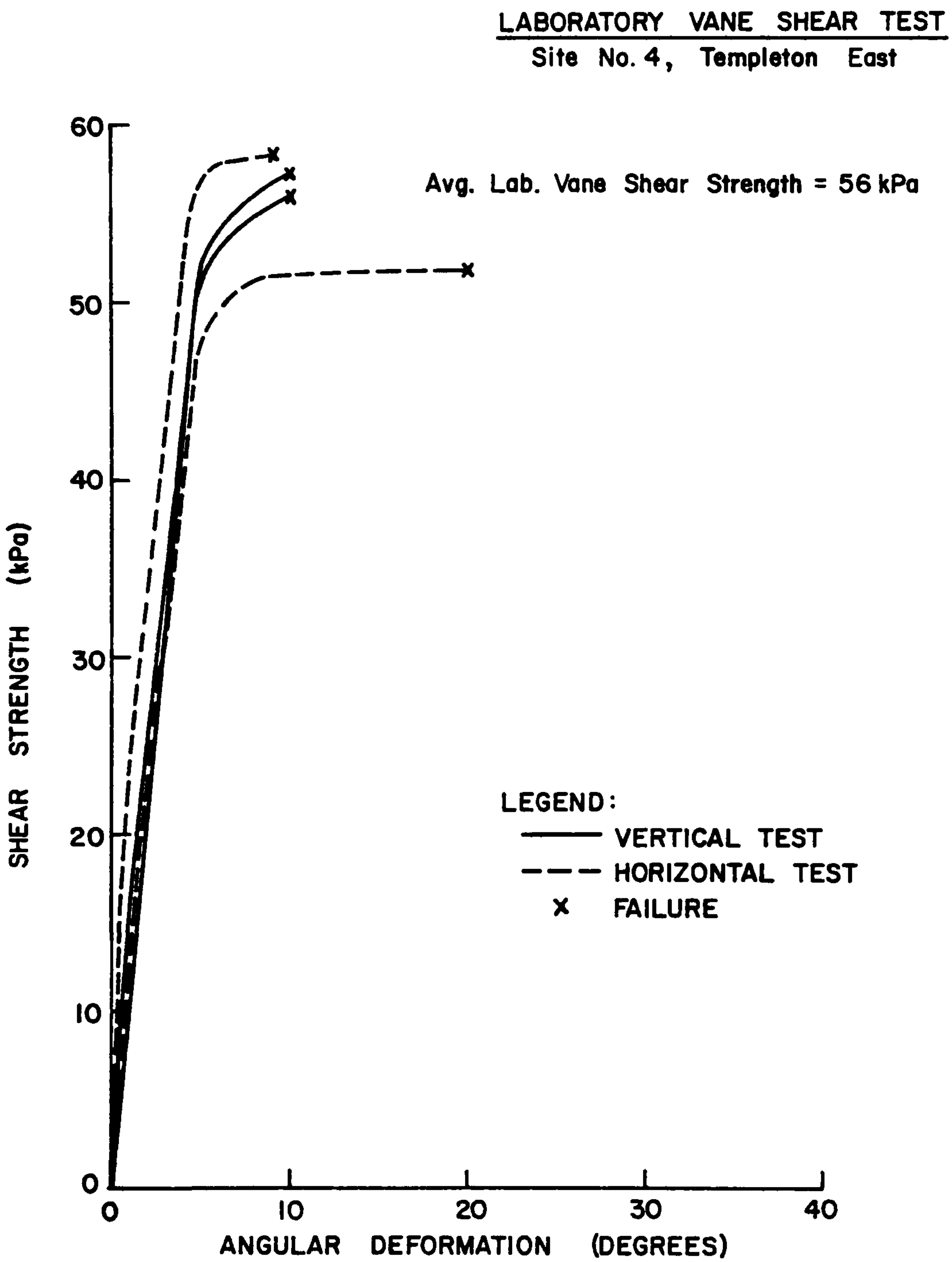

LABORATORY VANE SHEAR TEST
Site No. 4, Templeton East
Avg. Lab. Vane Shear Strength = 56 kPa
SHEAR STRENGTH (kPa)
60
50
40
30
20
10
0
0
10
20
30
40
ANGULAR DEFORMATION (DEGREES)
LEGEND:
VERTICAL TEST
HORIZONTAL TEST
x FAILURE

Figure A2

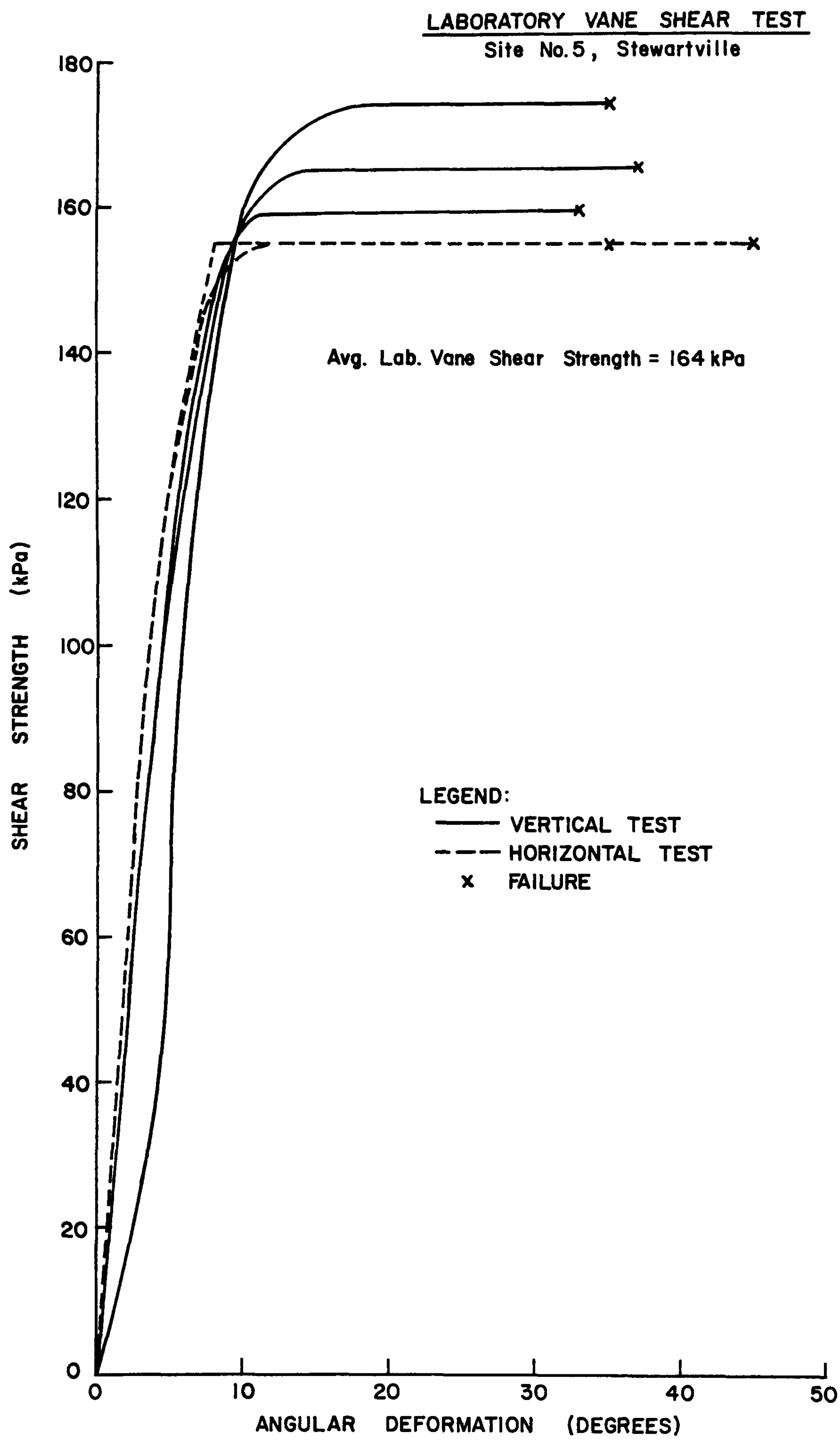

LABORATORY VANE SHEAR TEST
Site No. 5, Stewartville
Avg. Lab. Vane Shear Strength = 164 kPa
LEGEND:
VERTICAL TEST
HORIZONTAL TEST
x FAILURE
SHEAR STRENGTH (kPa)
ANGULAR DEFORMATION (DEGREES)
0
20
40
60
80
100
120
140
160
180
0
10
20
30
40
50

Figure A3

Site Name	UC Test Number	Sample Dimensions DxL (cm)	Rate of Strain (%/min)	Initial Density (kN/m^3)	Water Content (%)	$q_f = \sigma_1/2$ (kPa)	ε_f (%)	Mode of Failure	Remarks
St. Bernadin	1-1-3	5 x 10	0.4	15.33	65.8	29.2	1.0	VS	VS=vertical splitting
	1-1-4	5 x 10	0.4	15.67	66.0	26.3	1.1	VS	
				Average Strength =		27.5			
Bear Brook East	2-3-1	3.8 x 6.4	1.0	16.01	62.1	70.0	0.7	SH	Red silty clay
	2-3-3	3.8 x 7.6	1.0	16.01	62.8	72.0	1.0	SH	Red silty clay
				Average Strength =		71.0			
	2-3-2	3.8 x 7.6	1.0	19.15	32.5	94.1	1.2	SH	Grey clayey silt
	2-3-4	3.8 x 7.6	1.0	20.0	28.1	86.0	1.3	SH	Grey clayey silt
				Average Strength =		90.0			
Green Creek North	3-5-1	3.8 x 7.6	1.0	15.64	66.8	73.2	3.2	SH	SH = shear
	3-5-2	3.8 x 7.6	1.0	15.53	68.5	75.4	1.5	SH	
	3-5-3	3.8 x 7.6	1.0	15.59	70.0	73.7	1.7	SH	
				Average Strength =		74.1			

TABLE A-1 Summary of Unconfined Compression Tests

Site Name	UC Test Number	Sample Dimensions DxL (cm)	Rate of Strain (%/min)	Initial Density (kN/m^3)	Water Content (%)	$q_f = \sigma_{1/2}$ (kPa)	ε_f (%)	Mode of Failure	Remarks
Templeton East	4-1-1	3.8 x 7.6	0.4	16.47	58.3	91.4	1.76	SH	
	4-1-2	3.8 x 7.6	0.4	16.35	58.5	86.4	1.64	VS+SH	
	4-1-3	3.8 x 7.6	0.4	16.44	56.6	89.3	1.53	SH	
	4-2-1	3.8 x 7.6	0.4	16.45	57.7	89.6	1.2	SH	
	4-2-2	3.8 x 7.6	1.0	16.45	56.1	88.7	1.0	SH	Faster strain rate does not affect strength
	4-2-3	5.0 x 10	1.0	16.12	59.0	77.0	0.86	SH	Larger sample size reduces strength by 14%
	4-2-4	5.0 x 10	0.2	16.12	59.0	79.2	0.75	VS	Five-fold decrease in strain rate does not affect strength
	4-2-5	5.0 x 10	0.03	16.36	60.0	70.0	0.60	SH	Thirty-three fold decrease in strain rate reduces strength 10%
	4-2-6	5.0 x 10	0.3	17.05	54.4	57.6	1.4	VS	Low result probably due to sample variability or some microfissures.
Stewartville	5-1-1	3.8 x 7.6	0.5	19.0	36.7	215	1.0	VS	
	5-1-2	3.8 x 7.6	0.5	18.3	37.0	168	1.0	VS	
	5-1-3	3.8 x 7.6	0.5	18.2	37.2	166	0.7	VS	
	5-1-4	3.8 x 7.6	0.5	18.0	37.0	186	1.0	VS	
				Average Strength =		184			

TABLE A-1 (continued) Summary of Unconfined Compression Tests

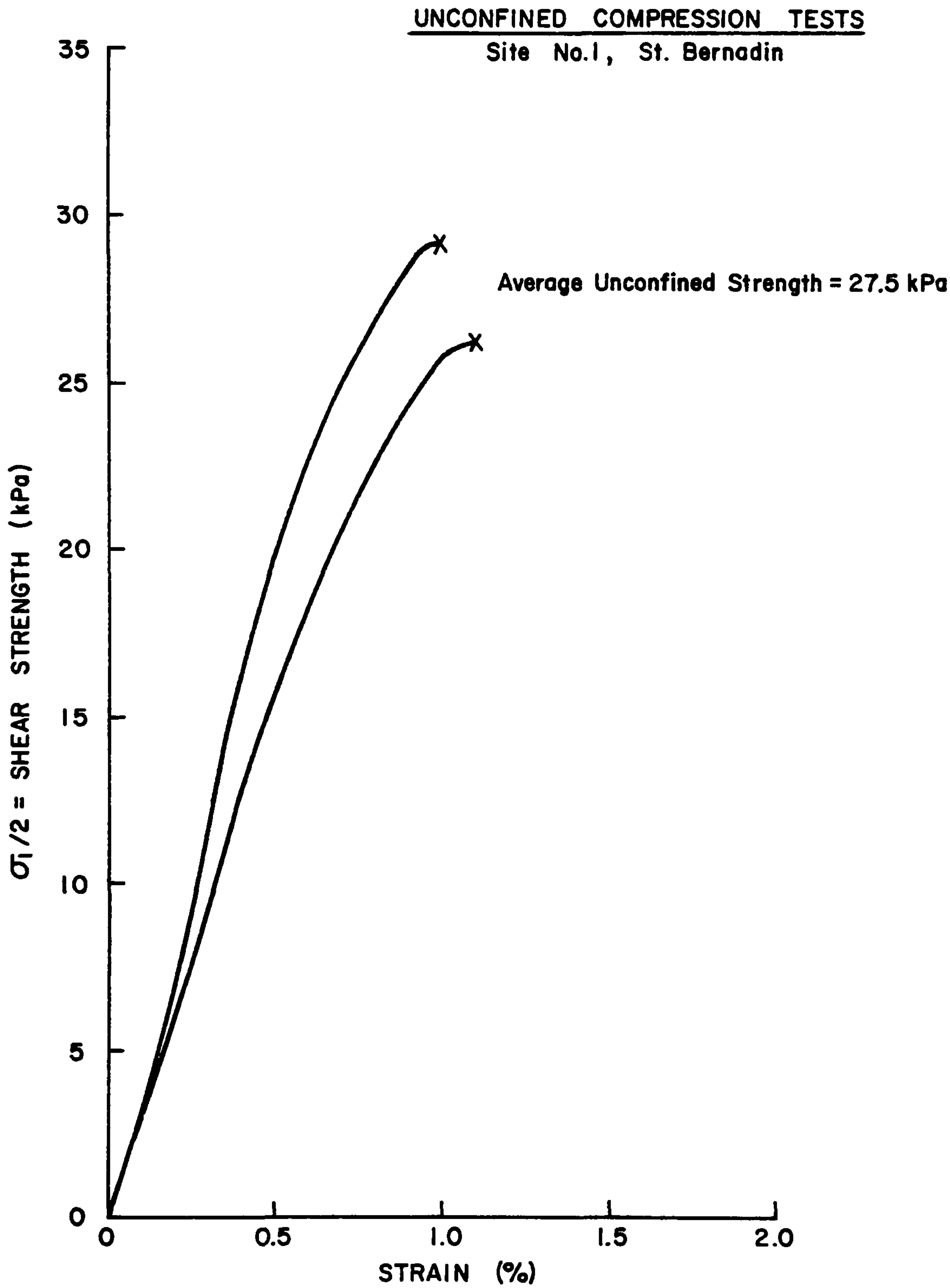

UNCONFINED COMPRESSION TESTS
Site No.1, St. Bernadin
Average Unconfined Strength = 27.5 kPa
σ1/2 = SHEAR STRENGTH (kPa)
35
30
25
20
15
10
5
0
0
0.5
1.0
1.5
2.0
STRAIN (%)

Figure A4

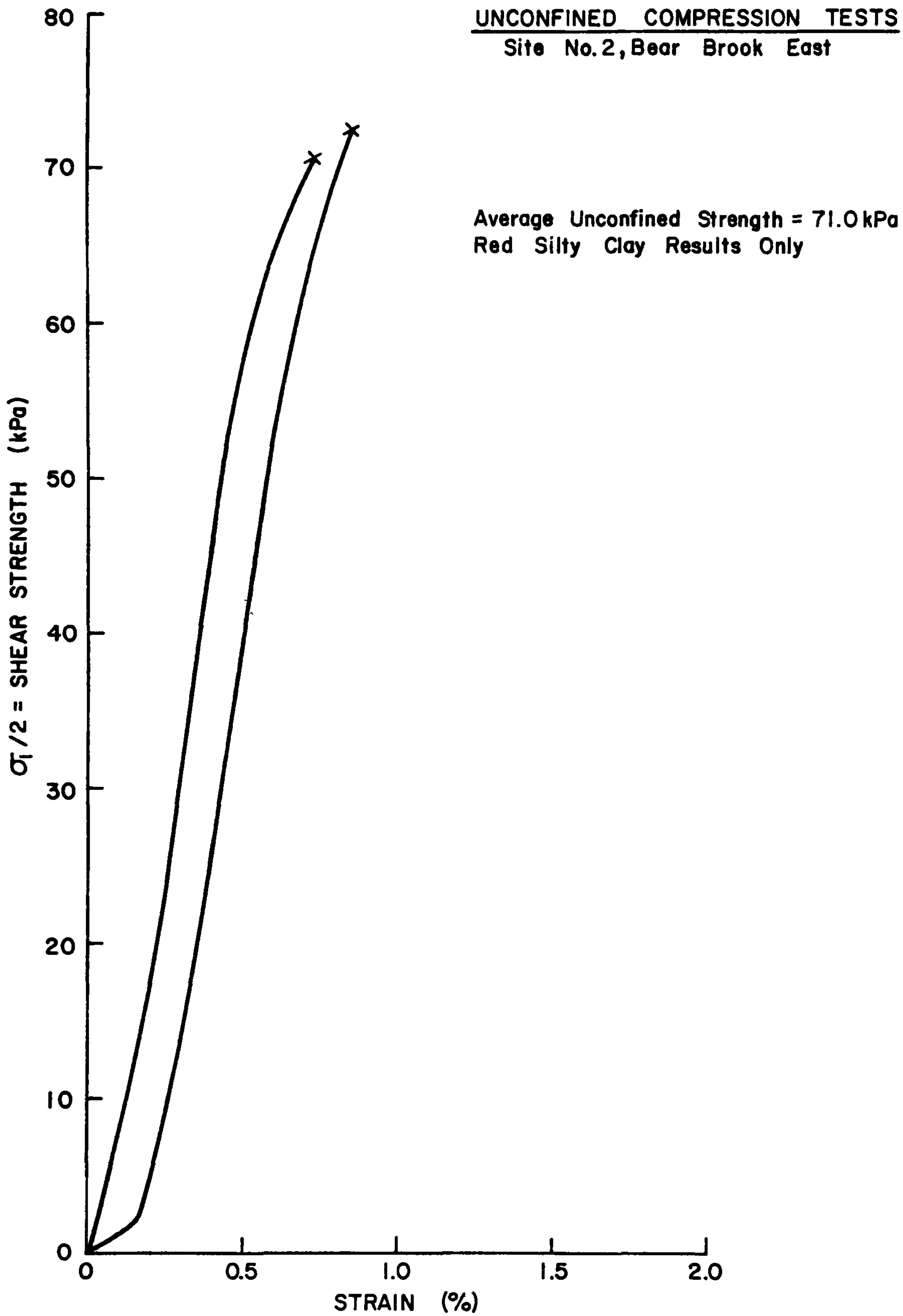

UNCONFINED COMPRESSION TESTS
Site No. 2, Bear Brook East
Average Unconfined Strength = 71.0 kPa
Red Silty Clay Results Only
$\sigma_1/2$ = SHEAR STRENGTH (kPa)
0
10
20
30
40
50
60
70
80
0
0.5
1.0
1.5
2.0
STRAIN (%)

Figure A5

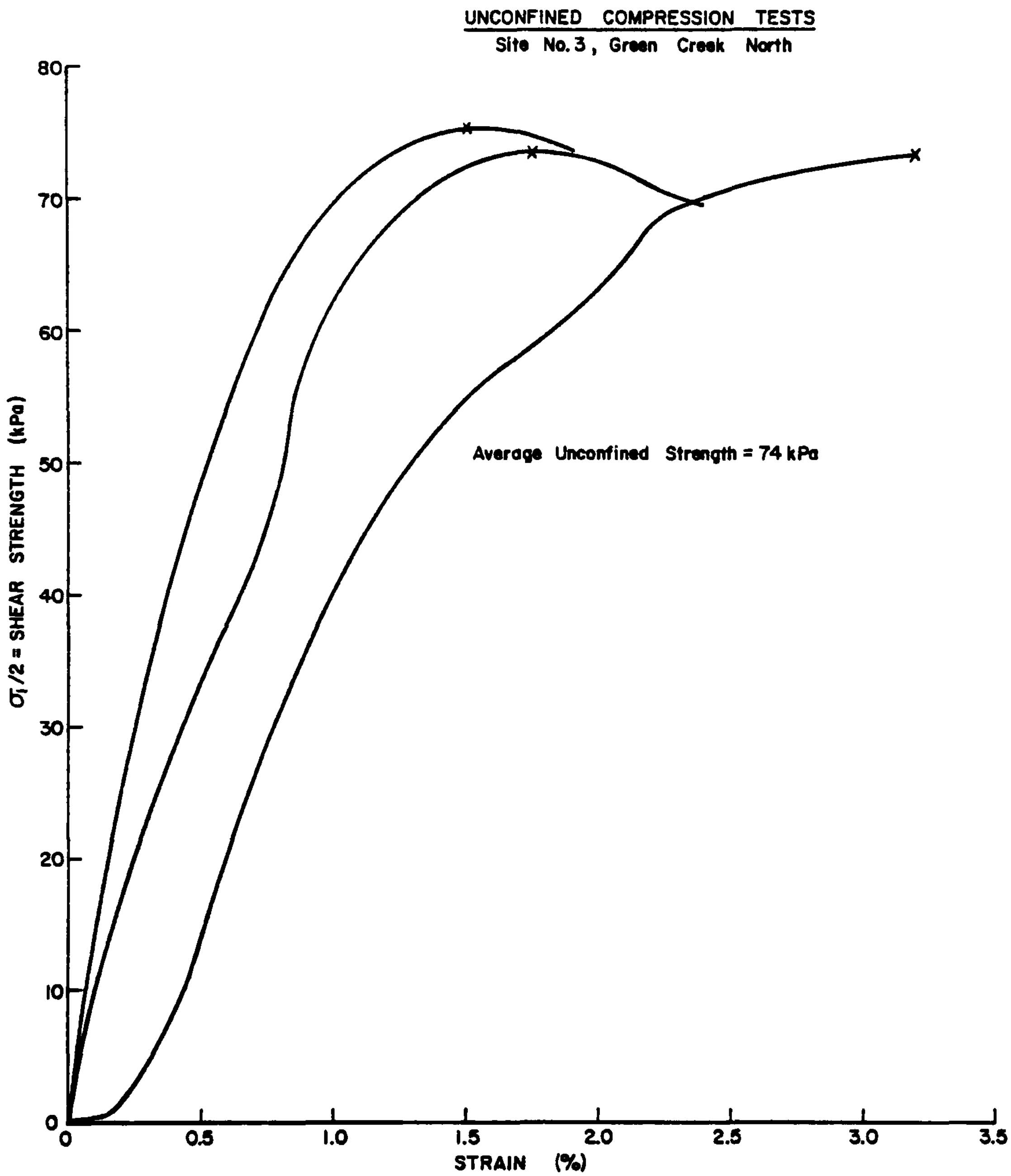
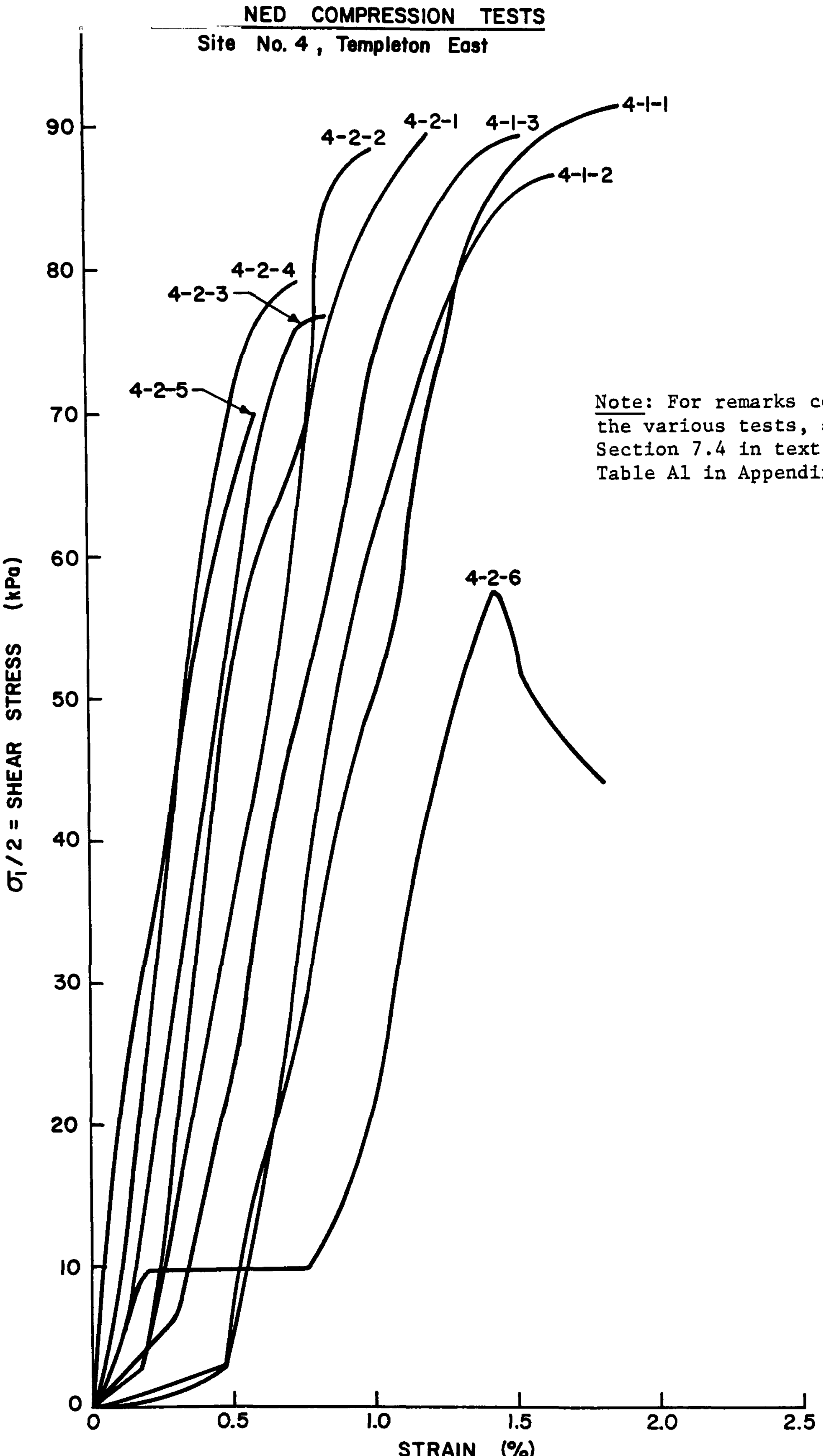

UNCONFINED COMPRESSION TESTS
Site No. 3, Green Creek North
Average Unconfined Strength = 74 kPa
$\sigma_1/2$ = SHEAR STRENGTH (kPa)
STRAIN (%)
0
10
20
30
40
50
60
70
80
0.5
1.0
1.5
2.0
2.5
3.0
3.5

Figure A6



Note: For remarks concerning the various tests, see Section 7.4 in text and Table A1 in Appendix A.

Figure A7

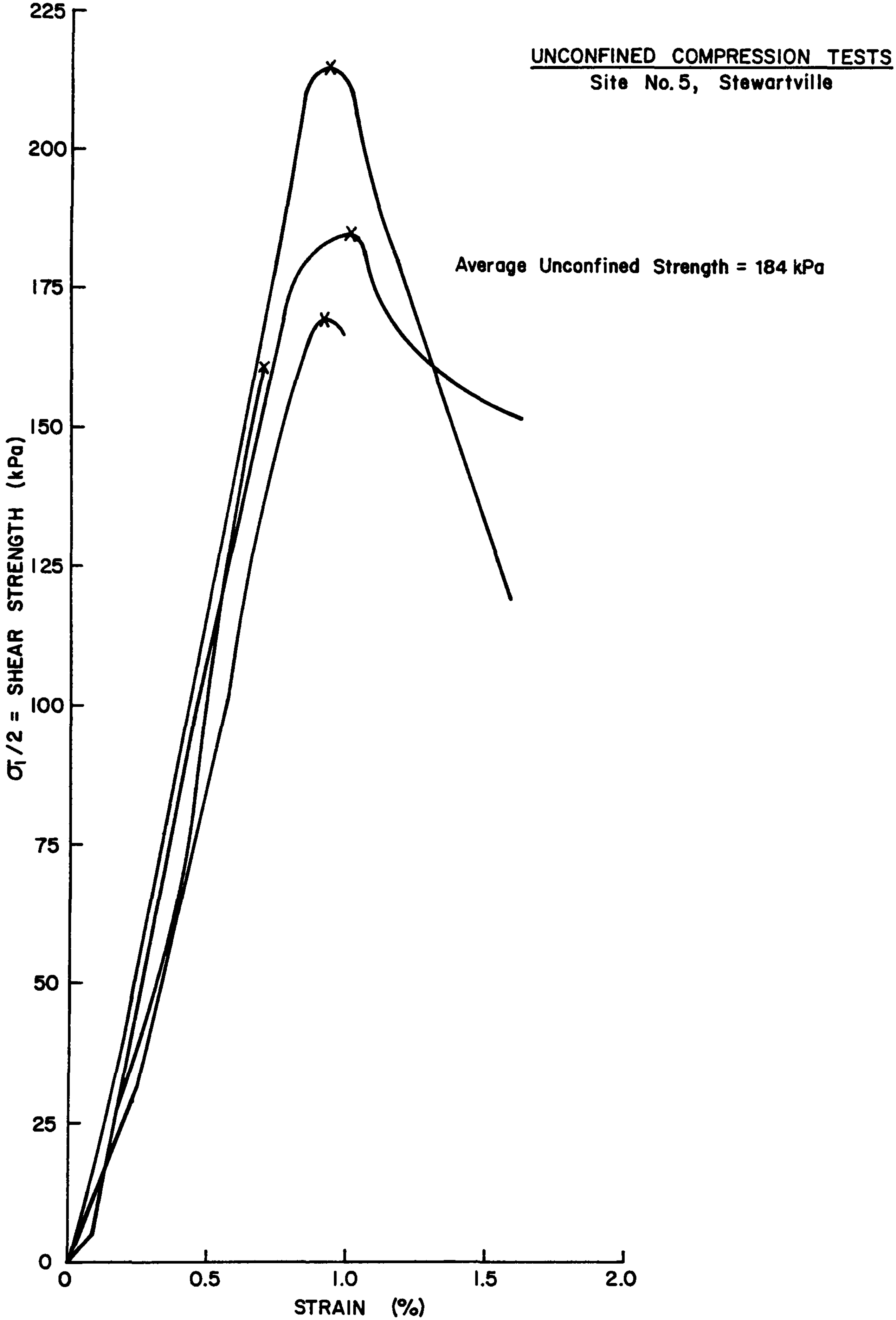

UNCONFINED COMPRESSION TESTS
Site No. 5, Stewartville
Average Unconfined Strength = 184 kPa
σ1/2 = SHEAR STRENGTH (kPa)
225
200
175
150
125
100
75
50
25
0
0
0.5
1.0
1.5
2.0
STRAIN (%)

Figure A8

Test No.	σ_{cons} (kPa)	Water Content (%)	$\gamma_{initial}$ (kN/m^3)	Net* Consolid. Volume Change $\Delta v/v_o$(%)	S_r (%)	Void Ratio		Peak Values						Residual Values ε = 6%				
						Initial	After Consolid	Δu_f (kPa)	P_{tot} (kPa)	p'_f (kPa)	q_f (kPa)	ε_f %	A_f	Δu_r (kPa)	p'_r (kPa)	q_r (kPa)	A_r	Mode of Failure
1-1-1	0.7	65.5	15.7	+0.4	99.0	1.87	1.88	1.50	27.6	26.9	26.9	1.1	.03	-3.0	31.5	26.9	-.05	VS
1-3-1	30	66.0	15.6	-0.8	100.0	1.99	1.96	22.4	62.1	39.7	32.5	1.0	.24	22.4	25.4	18.1	.32	SH
1-3-6	45	-	16.1	-1.5	-	-	-	34.8	86.25	51.5	41.2	1.1	.27	33.4	32.6	20.0	.38	SH
1-3-4	50	67.5	15.8	-1.1	99.0	2.10	2.07	36.3	94.8	58.5	44.8	1.1	.26	40.5	29.5	20.0	.46	SH+BU
1-3-3	60	66.8	15.9	-1.5	100.0	1.94	1.91	42.8	106.8	63.9	46.8	1.1	.28	49.6	30.	20.0	.45	SH

*Note: + equals volume increase, - equals volume decrease

TABLE A-2 Summary of CIU Triaxial Test Results

Site No. 1, St. Bernadin

Test No.	σ_{cons} (kPa)	Water Content (%)	$\gamma_{initial}$ (kN/m^3)	Net* Consolid. Volume Change $\Delta v/v_o$ (%)	S_r (%)	Void Ratio: Initial	Void Ratio: After Consolid	Peak Values: Δu_f (kPa)	Peak Values: P_{tot} (kPa)	Peak Values: p'_f (kPa)	Peak Values: q_f (kPa)	Peak Values: ε_f %	Peak Values: A_f	Residual Values ε = 6%: Δu_r (kPa)	Residual Values ε = 6%: p'_r (kPa)	Residual Values ε = 6%: q_r (kPa)	Residual Values ε = 6%: A_r	Mode of Failure
2-3-3 (red clay)	0.7	63.4	16.23	+1.97	100	1.68	1.73	-0.8	22.6	23.4	21.9	1.0	-0.02	-8.5	35.8	26.6	-0.16	VS
2-3-9[1]	40	60.9	16.09	-0.14	100	1.78	1.77	36.1	108.7	72.7	68.7	1.5	+0.20	20	71	51	+0.13	SH
2-3-5	60	60.1	16.61	-0.8	100	1.68	1.66	59.4	124.6	65.2	64.6	1.1	0.31	33.1	64.8	37.8	0.23	SH
2-3-6	100	60.9	16.3	-0.83	100	1.74	1.72	81.9	172.1	90.2	72.2	1.3	0.34	63.9	76.6	40.5	0.33	SH
2-2-1[2]	120	60.7	16.48	-1.0	99.6	1.72	1.69	84.9	195.0	110.1	75.1	1.0	0.31	76.3	96.5	52.5	0.33	SH
2-2-2[2]	120	60.6	17.12	-1.36	100	1.61	1.56	86.5	191.9	105.4	71.9	0.9	0.33	test stopped prematurely				SH

*Note: + equals volume increase, - equals volume decrease
1. 3.8 x 6.9 cm sample
2. 3.8 x 7.6 cm sample
All other samples 5.0 x 10.0 cm

TABLE A-3 Summary of CIU Triaxial Test Results

Site No. 2, Bear Brook East

Test No.	σ_{cons} (kPa)	Water Content (%)	$\gamma_{initial}$ (kN/m^3)	Net* Consolid. Volume Change $\Delta v/v_o$(%)	S_r (%)	Void Ratio: Initial	Void Ratio: After Consolid	Peak Values: Δu_f (kPa)	Peak Values: P_{tot} (kPa)	Peak Values: p'_f (kPa)	Peak Values: q_f (kPa)	Peak Values: ε_f %	Peak Values: A_f	Residual Values ε = 6%: Δu_r (kPa)	Residual Values ε = 6%: p'_r (kPa)	Residual Values ε = 6%: q_r (kPa)	Residual Values ε = 6%: A_r	Mode of Failure
3-2-2	0.7	70.3	15.9	+1.0	100	2.08	2.10	-3.01	67.4	70.3	66.8	.98	-0.02	-15.8	72.0	56.2	-0.13	SH+VS
3-2-3	75	74.5	15.0	-0.5	99	2.26	2.25	58.7	145.3	86.6	70.2	1.01	+0.27	50.2	69.5	44.5	0.31	multiple SH+VS
3-4-1	90	76.4	15.6	-1.15	99.7	2.13	2.10	55.6	154.7	99.1	64.7	0.83	0.25	48.8	68.9	27.8	0.34	PU
3-1-1	100	67.3	16.08	-1.84	100	1.88	1.86	70.6	180.5	109.8	80.5	1.06	0.27	88.7	60.0	48.6	0.45	SH+BU
3-1-2	120	68.1	15.2	-2.00	99	2.07	2.01	80.4	192.6	112.2	72.6	1.09	0.3	116.5	45.3	41.8	0.57	SH

Note: + equals volume increase, - equals volume decrease

TABLE A-4 Summary of CIU Triaxial Test Results

Site No. 3, Green Creek North

Test No.	σ_{cons} (kPa)	Water Content (%)	$\gamma_{initial}$ (kN/m^3)	Net* Consolid. Volume Change $\Delta v/v_o$ (%)	S_r (%)	Void Ratio		Peak Values						Residual Values ε = 6%				
						Initial	After Consolid	Δu_f (kPa)	p_{tot} (kPa)	p'_f (kPa)	q_f (kPa)	ε_f %	A_f	Δu_r (kPa)	p'_r (kPa)	q_r (kPa)	A_r	Mode of Failure
4-1-1	0.7	58.4	16.53	+0.4	99	1.66	1.67	-0.7	54.2	54.8	53.5	1.1	-0.01	-15.86	52.4	36.6	-0.2	VS
4-2-2	0.7	57.2	16.57	0	100	1.59	1.59	2.25	34.6	32.4	32.4	0.4	0.05	-25.6	78.7	52.5	-0.24	SH (fis-sured)
4-2-5	75	56.6	16.16	-0.93	99	1.67	1.64	57.12	139.1	82.	64.	0.85	0.28	60.1	59.0	43.9	0.37	SH
4-1-2	100	59.4	16.31	-0.84	99	1.72	1.69	71.8	172.6	100.9	72.6	1.1	0.29	81.9	56.1	38.0	0.47	SH
4-1-4	120	58.9	16.66	-1.77	100	-	-	73.6	197.6	124.0	77.7	1.1	0.27	117.2	38.7	35.9	0.61	BU

*Note: + equals volume increase, - equals volume decrease

TABLE A-5 Summary of CIU Triaxial Test Results

Site No. 4, Templeton East

Test No.	σ_{cons} (kPa)	Water Content (%)	$\gamma_{initial}$ (kN/m^3)	Net* Consolid. Volume Change $\Delta v/v_o$(%)	S_r (%)	Void Ratio		Peak Values						Residual Values ε = 6%				
						Initial	After Consolid	Δu_f (kPa)	P_{tot} (kPa)	p'_f (kPa)	q_f (kPa)	ε_f %	A_f	Δu_r (kPa)	p'_r (kPa)	q_r (kPa)	A_r	Mode of Failure
5-1-1	0.7	37.4	18.43	+0.5	99	1.07	1.08	3.8	197.3	193.5	193.5	1.26	0.01	-33.1	133.5	100.5	-0.16	VS
5-1-2	50	37.3	18.18	+0.5	100	1.08	1.09	43.6	232.2	188.6	182.2	1.16	0.11	- 9.8	167.8	108	-0.04	SH+VS
5-1-5	200	37.4	17.67	-0.67	100	1.15	1.13	169.1	390.1	220.9	190.1	1.02	0.29	125	200	125	0.27	SH+VS
5-1-6	230	37.3	17.67	-0.4	100	1.16	1.15	190.9	442.4	251.5	212.4	1.17	0.29	145.9	191	107	0.32	SH
5-1-4	250	37.2	18.49	-0.4	99	1.06	1.05	128.2	461.8	321.2	211.5	0.99	0.21	118.15	270.6	138.4	0.22	SH

*Note: + equals volume increase, - equals volume decrease

TABLE A-6 Summary of CIU Triaxial Test Results

Site No. 5, Stewartville

Strain %	Mobilized a' (kPa)	Mobilized α' (degrees)	Mobilized $\phi'=\sin^{-1}\tan\alpha$ (degrees)	Mobilized $c'=\frac{a'}{\cos\phi'}$ (kPa)
0.2	7.0	3.2	3.2	7.0
0.3	8.0	9.	9.1	8.1
0.4	11.0	12.	12.3	11.25
0.5	12.0	16.	16.6	12.5
0.6	13.0	19.	20.1	13.8
0.7	13.8	22.	23.8	15.0
0.8	13.2	25.	27.8	14.9
1.0	12.2	28.	32.1	14.4
1.1	11.3	29.5	34.2	13.6
1.5	8.0	28.	32.	9.4
3.0	7.0	27.0	30.6	8.1
4.0	6.0	29.0	33.6	7.2
6.0	7.0	26.0	29.2	8.

TABLE A-7 Mobilization of c' and ϕ' with Strain
Site No. 1, St. Bernadin

Data used for Figure 7.9.1

Strain %	Mobilized a' (kPa)	Mobilized α' (degrees)	Mobilized $\phi' = \sin^{-1}\tan\alpha$ (degrees)	Mobilized $c' = \frac{a'}{\cos\phi'}$ (kPa)
0.4	17.7	5.2	5.2	17.8
0.5	28	7	7.1	28.
0.6	31	10	10.2	31.5
0.7	34.5	12	12.3	35.3
0.8	35	14	14.4	36.1
0.9	36	16.5	17.2	37.7
1.0	37.5	17	17.8	39.3
1.1	36	19.5	20.7	38.5
1.5	12.6	30	35.3	15.4
2.0	11.96	30.0	35.3	14.6
3.0	3.67	34.0	42.4	5.0
4.0	8	31.0	36.9	10.0
6.0	15.0	26.0	29.2	17.0

TABLE A-8 Mobilization of c' and ϕ' with Strain
Site No. 4, Templeton East

Data used for Figure 7.9.2

Strain %	Mobilized a' (kPa)	Mobilized α' (degrees)	Mobilized $\phi'=\sin^{-1}\tan\alpha$ (degrees)	Mobilized $c'=\frac{a'}{\cos\phi'}$ (kPa)
0.2	16	0	0	16
0.3	36	2	2	36
0.4	38	11	11.2	38.7
0.5	45	15	15.5	46.7
0.6	57	17.5	18.4	60.1
0.7	62	21	22.6	67.1
0.8	64	25	27.8	72.34
0.9	76	25	27.8	85.9
1.0	88	24	26.4	98.
1.1	98	24	26	110
1.5	56	24	26	61
2.0	59	23	25.1	65
3.0	62	20.7	22.2	67
4.0	55	21.	22.5	60
5.0	46	21.5	23.	50
6.0	36.5	22.	24.	40

Note: mobilized a' and α' were determined over the 0 to 230 kPa stress range

TABLE A-9 Mobilization of c' and ϕ' with Strain
Site No. 5, Stewartville

Data used for Figure 7.9.3

APPENDIX B

EFFECTIVE STRESS FAILURE ENVELOPES

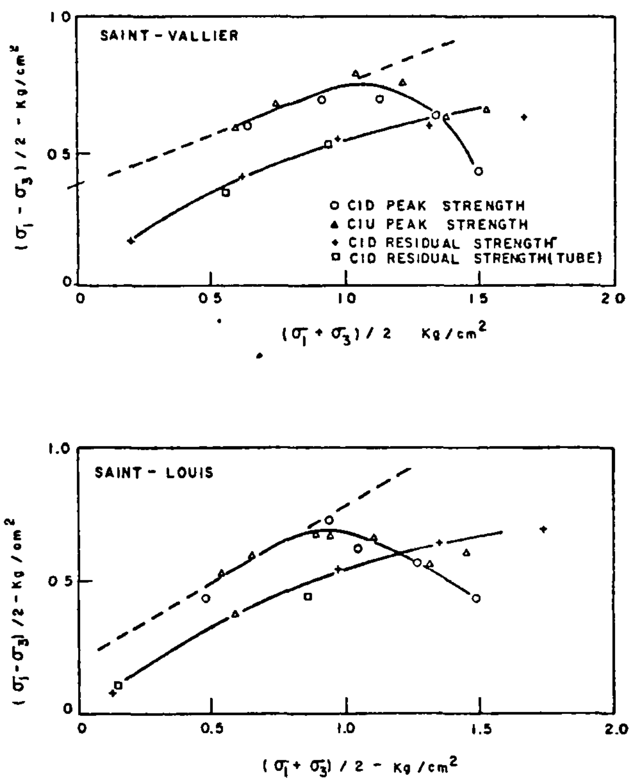
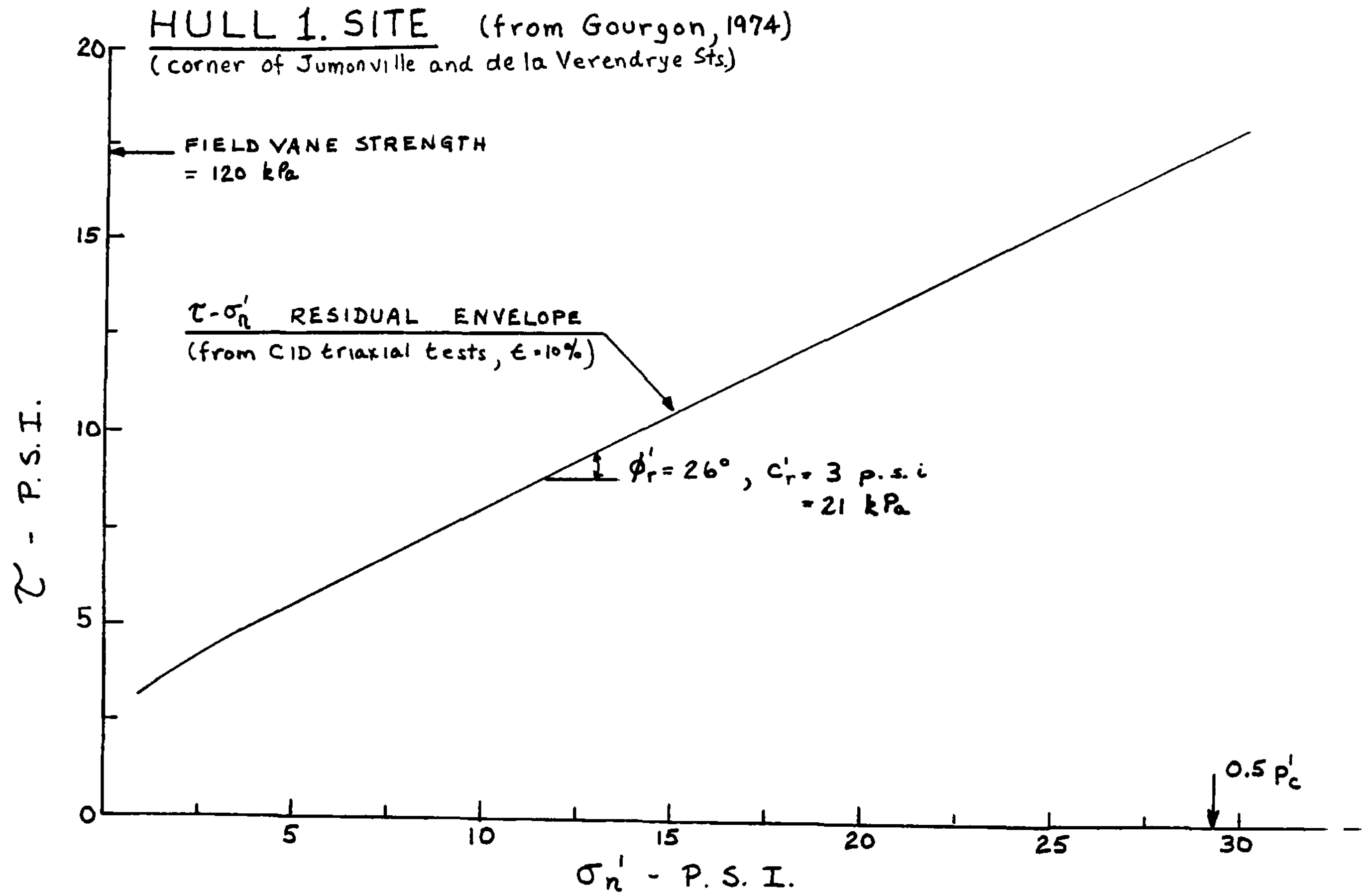
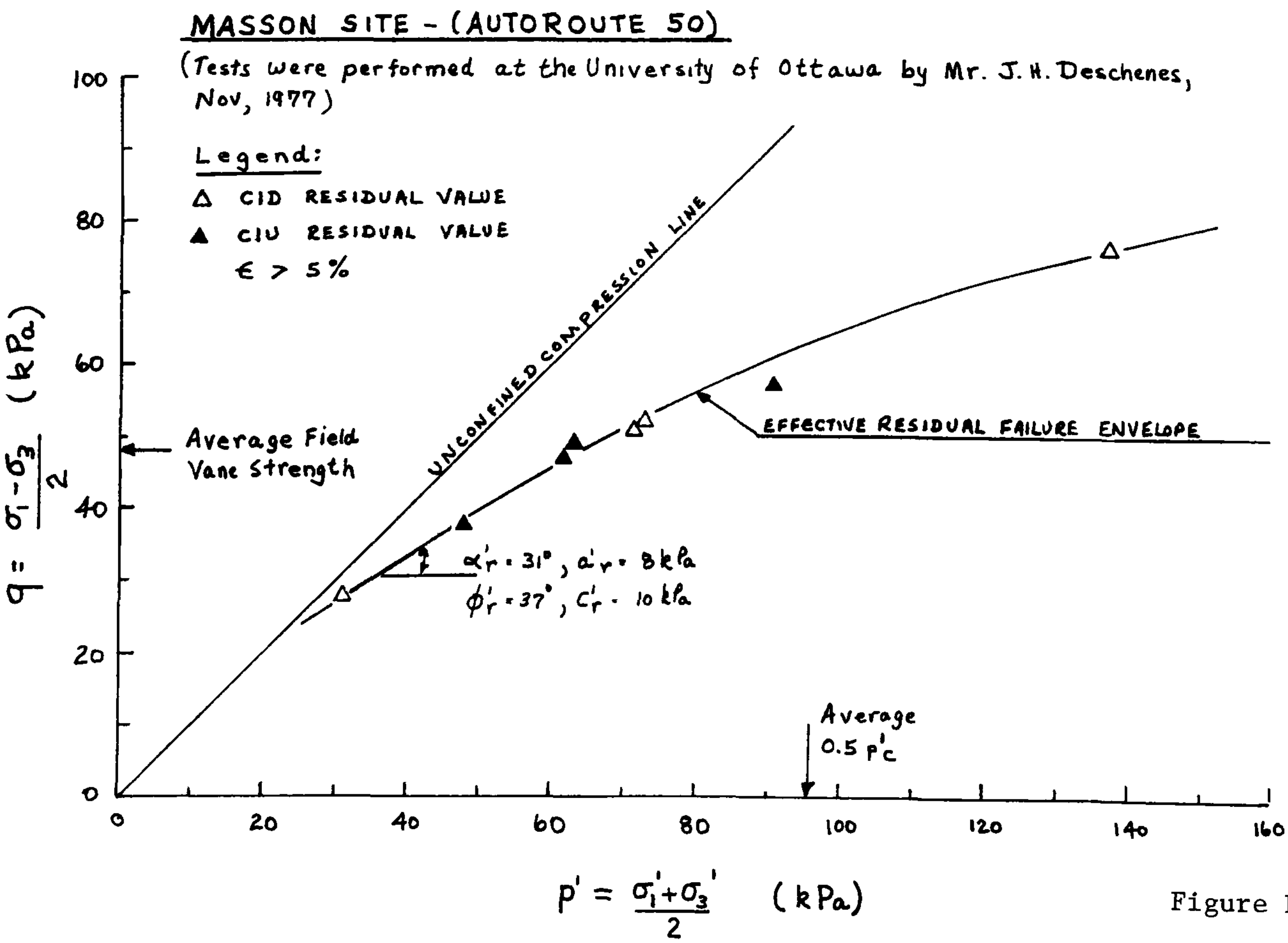


Figure B-1 Failure envelopes for St. Vallier and St. Louis (from Lefebvre and LaRochelle, 1973)

		St. Vallier	St. Louis
Peak strength envelope:	c' (kPa)	40	27
	ø'	23°	31°
Residual strength envelope:	c'_r (kPa)	7	7
	$ø'_r$	31°	31°
Field vane strength (kPa)		45	45

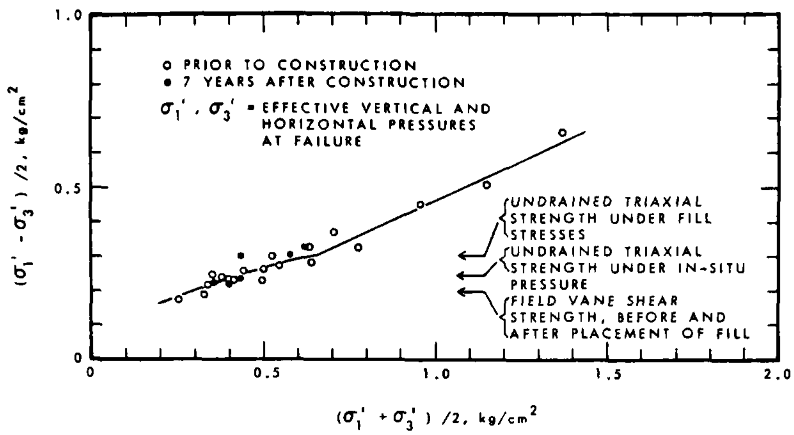


HULL 1. SITE (from Gourgon, 1974)
(corner of Jumonville and de la Verendrye Sts.)
FIELD VANE STRENGTH = 120 kPa
τ-σ'n RESIDUAL ENVELOPE
(from CID triaxial tests, ε = 10%)
φ'r = 26°, c'r = 3 p.s.i = 21 kPa
0.5 p'c
τ - P.S.I.
σ'n - P.S.I.
0
5
10
15
20
25
30

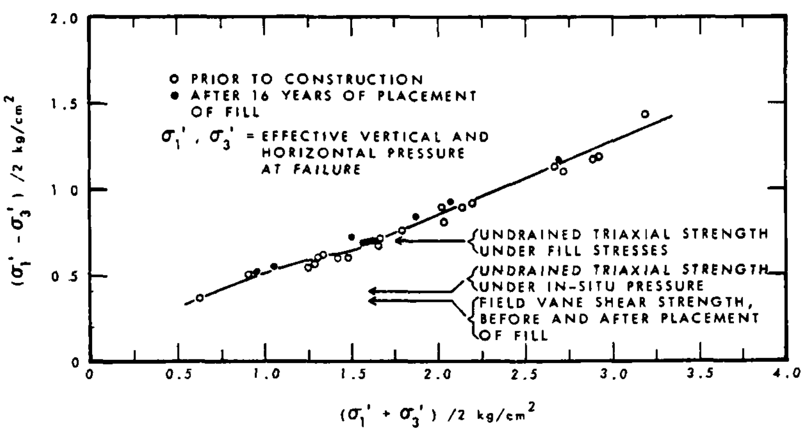


MASSON SITE - (AUTOROUTE 50)
(Tests were performed at the University of Ottawa by Mr. J.H. Deschenes, Nov, 1977)
Legend:
Δ CID RESIDUAL VALUE
▲ CIU RESIDUAL VALUE
ε > 5%
UNCONFINED COMPRESSION LINE
EFFECTIVE RESIDUAL FAILURE ENVELOPE
Average Field Vane Strength
α'r = 31°, a'r = 8 kPa
φ'r = 37°, c'r = 10 kPa
Average 0.5 p'c
q = (σ1 - σ3)/2 (kPa)
p' = (σ'1 + σ'3)/2 (kPa)
0
20
40
60
80
100
120
140
160

Figure B-2



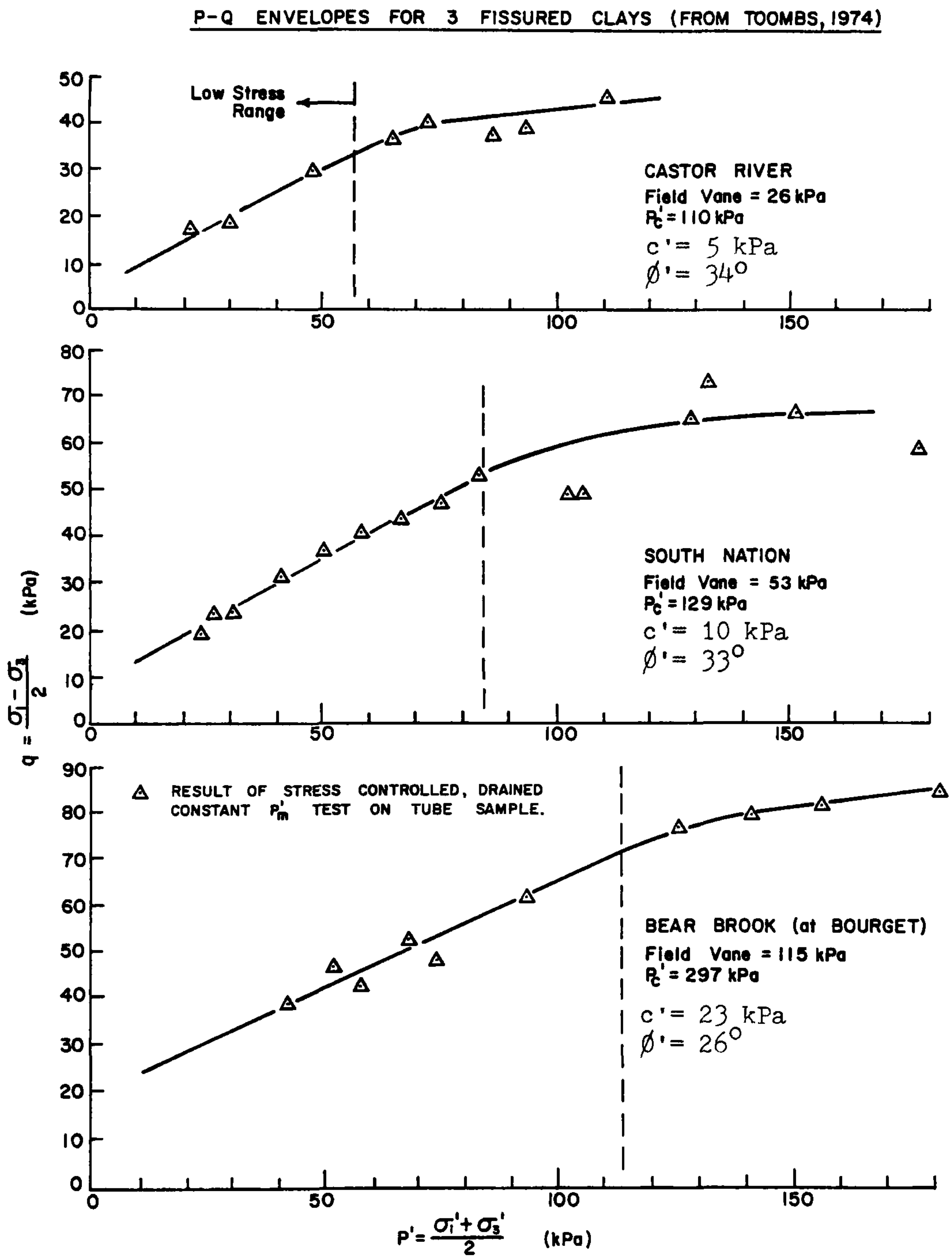
Peak Strength Envelope for Soil Layer Between Depths 2.3 m and 5.3 m (Gloucester)



Peak Strength Envelope for Soil Layer Between Depths 7.0 m and 11.3 m (Kars)

Figure B-3 (from Law et al, 1977)

		Gloucester	Kars
Peak strength envelope:	c' (kPa)	10	14.5
	ø'	24°	25°
Field vane strength (kPa)		20	35

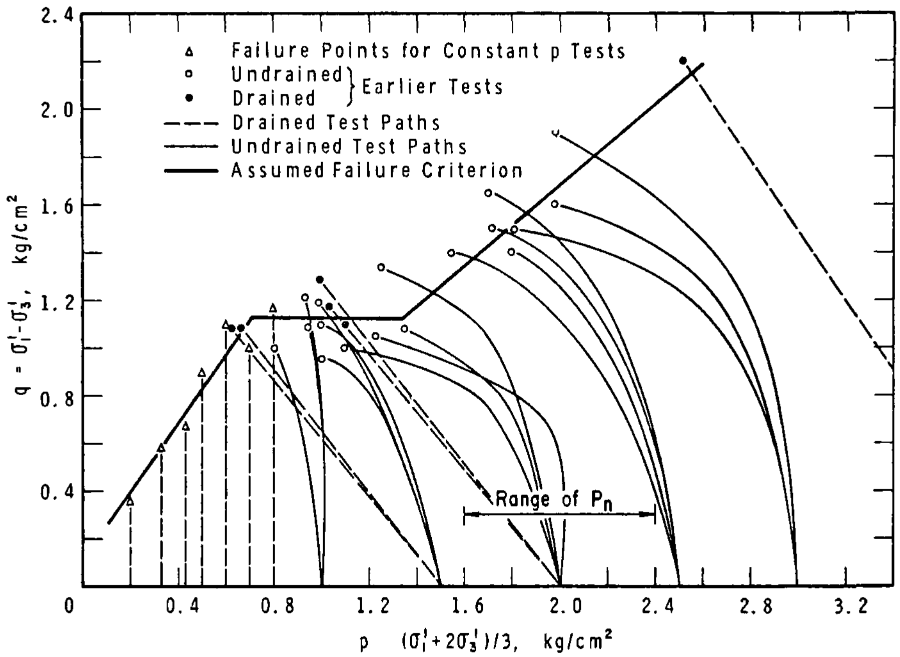


P-Q ENVELOPES FOR 3 FISSURED CLAYS (FROM TOOMBS, 1974)
Low Stress Range
CASTOR RIVER
Field Vane = 26 kPa
P_c' = 110 kPa
c' = 5 kPa
ø' = 34°
SOUTH NATION
Field Vane = 53 kPa
P_c' = 129 kPa
c' = 10 kPa
ø' = 33°
RESULT OF STRESS CONTROLLED, DRAINED CONSTANT P_m' TEST ON TUBE SAMPLE.
BEAR BROOK (at BOURGET)
Field Vane = 115 kPa
P_c' = 297 kPa
c' = 23 kPa
ø' = 26°
$q = \frac{\sigma_1 - \sigma_3}{2}$ (kPa)
$P' = \frac{\sigma_1' + \sigma_3'}{2}$ (kPa)

Figure B-4

BRECKENRIDGE (Mitchell,1970) c' = 5 kPa

f.v.=48 kPa p_c'=163 kPa ϕ' = 35°



ROCKCLIFFE (Mitchell,1970) c' = 12 kPa

f.v.=69 kPa p_c'=240 kPa ϕ' = 33°

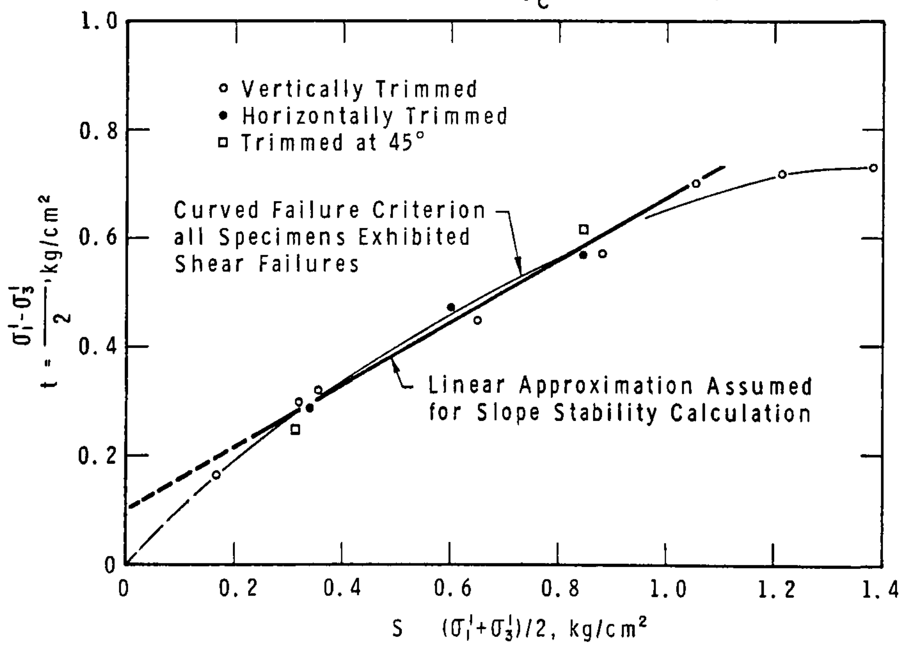


Figure B-5

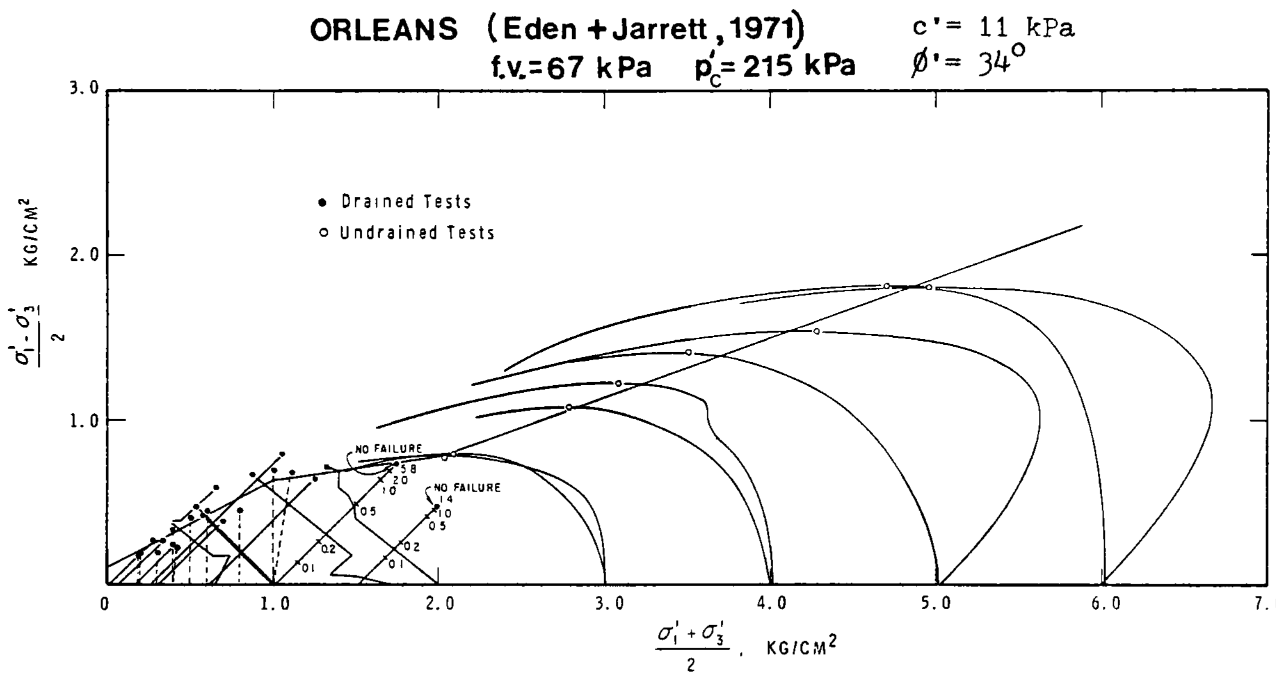
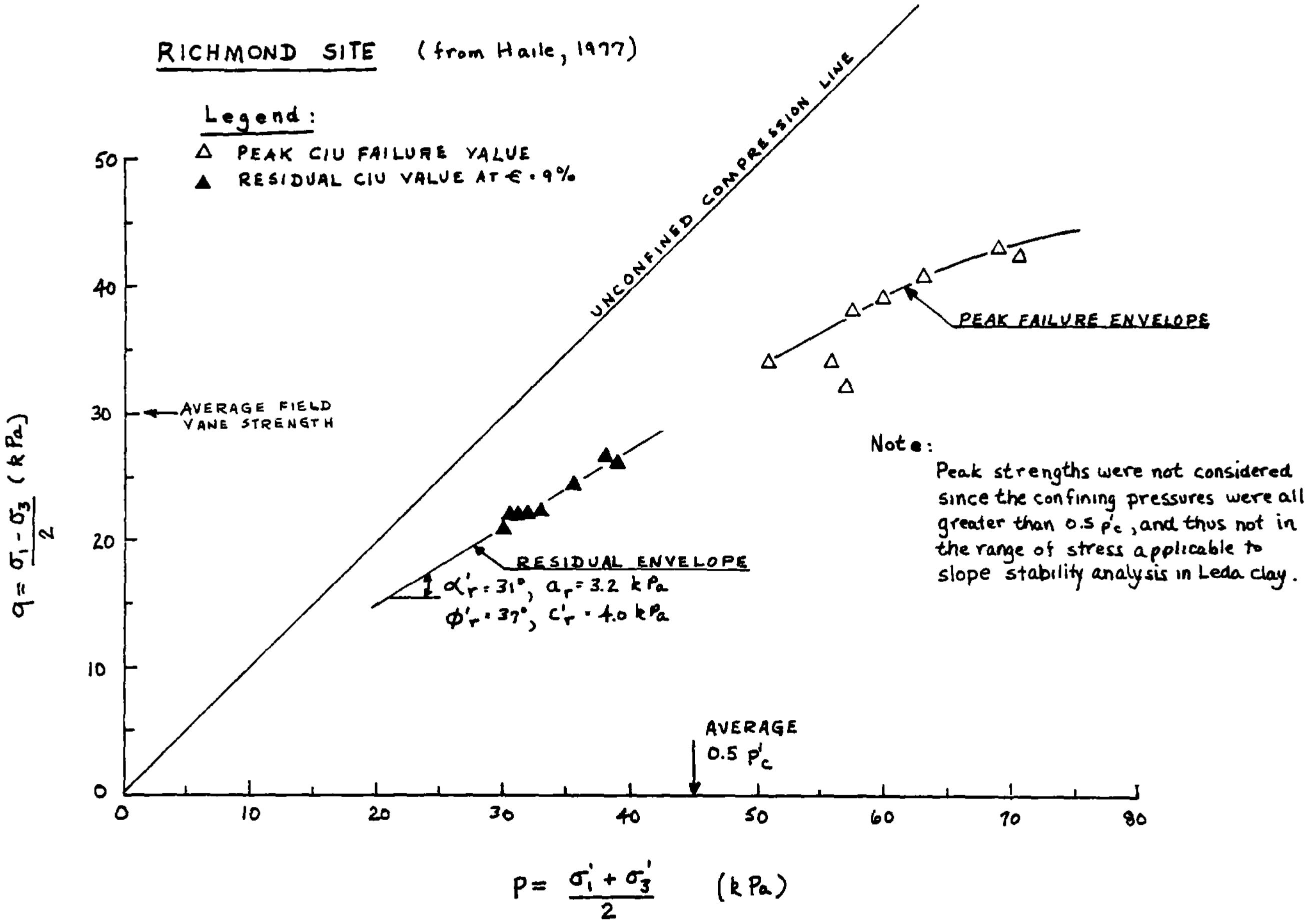

ORLEANS (Eden + Jarrett, 1971)
c' = 11 kPa
f.v.= 67 kPa
p'c = 215 kPa
ø' = 34°
Drained Tests
Undrained Tests
NO FAILURE
NO FAILURE
KG/CM²
KG/CM²
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0

Figure B-6


RICHMOND SITE (from Haile, 1977)
Legend:
△ PEAK CIU FAILURE VALUE
▲ RESIDUAL CIU VALUE AT ∈ = 9%
UNCONFINED COMPRESSION LINE
PEAK FAILURE ENVELOPE
AVERAGE FIELD VANE STRENGTH
RESIDUAL ENVELOPE
α'r = 31°, ar = 3.2 kPa
φ'r = 37°, c'r = 4.0 kPa
AVERAGE 0.5 p'c
Note:
Peak strengths were not considered since the confining pressures were all greater than 0.5 p'c, and thus not in the range of stress applicable to slope stability analysis in Leda clay.
q = (σ1 - σ3)/2 (kPa)
p = (σ'1 + σ'3)/2 (kPa)
0
10
20
30
40
50
60
70
80

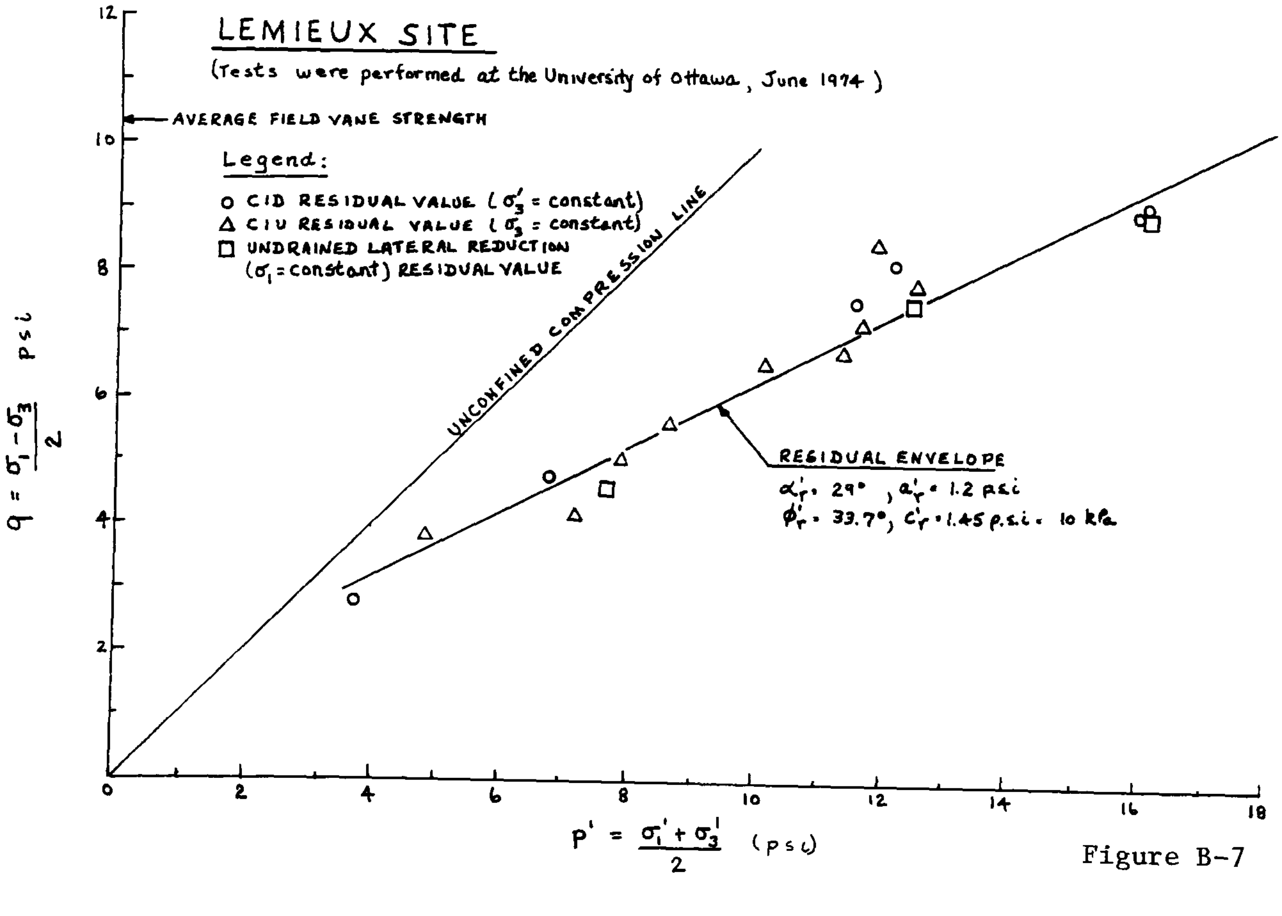
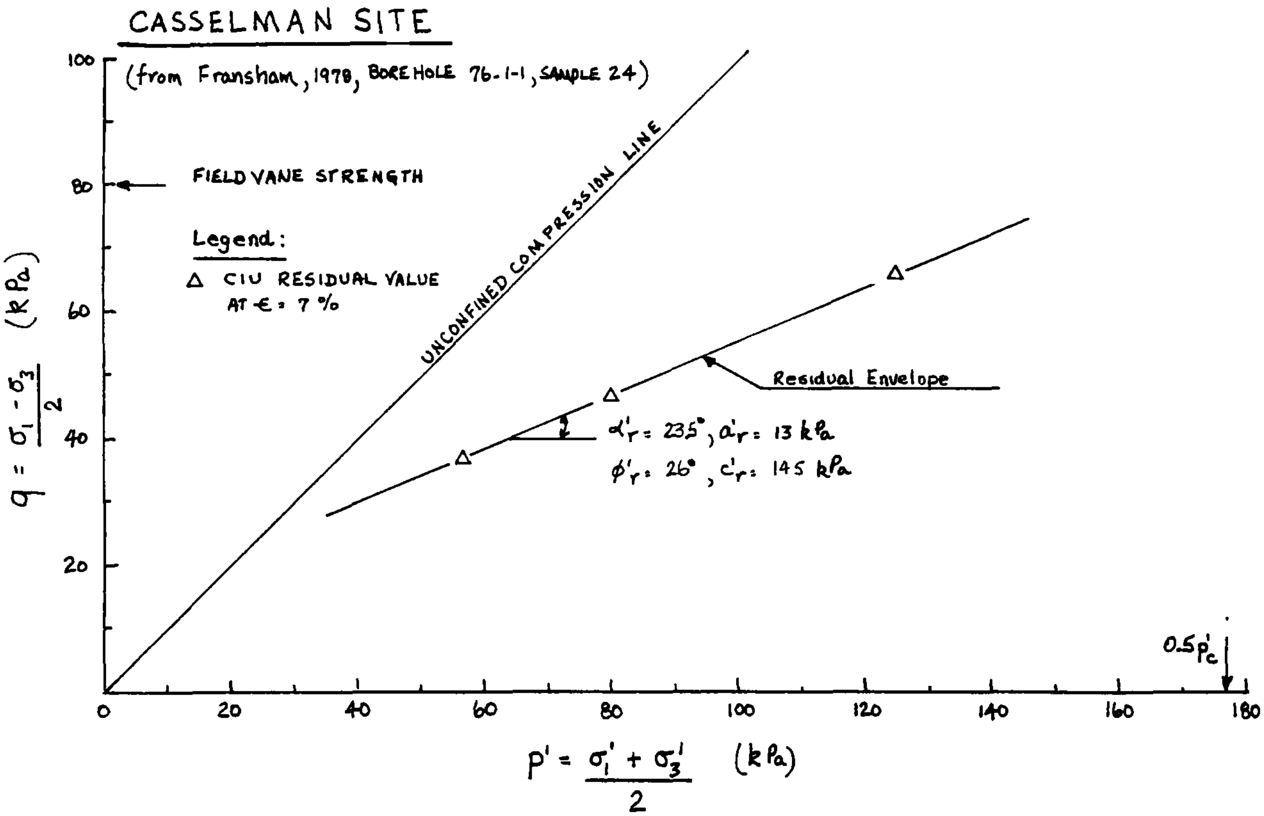

LEMIEUX SITE
(Tests were performed at the University of Ottawa, June 1974)
AVERAGE FIELD VANE STRENGTH
Legend:
○ CID RESIDUAL VALUE (σ'3 = constant)
△ CIU RESIDUAL VALUE (σ3 = constant)
□ UNDRAINED LATERAL REDUCTION (σ1 = constant) RESIDUAL VALUE
UNCONFINED COMPRESSION LINE
RESIDUAL ENVELOPE
α'r = 29°, a'r = 1.2 psi
φ'r = 33.7°, c'r = 1.45 p.s.i. = 10 kPa
q = (σ1 - σ3)/2 psi
p' = (σ'1 + σ'3)/2 (psi)
0
2
4
6
8
10
12
14
16
18

Figure B-7


CASSELMAN SITE
(from Fransham, 1978, BOREHOLE 76-1-1, SAMPLE 24)
FIELD VANE STRENGTH
Legend:
Δ CIU RESIDUAL VALUE AT ε = 7 %
UNCONFINED COMPRESSION LINE
Residual Envelope
α'r = 23.5°, a'r = 13 kPa
φ'r = 26°, c'r = 14.5 kPa
0.5 p'c
q = (σ1 - σ3)/2 (kPa)
p' = (σ'1 + σ'3)/2 (kPa)
0
20
40
60
80
100
120
140
160
180

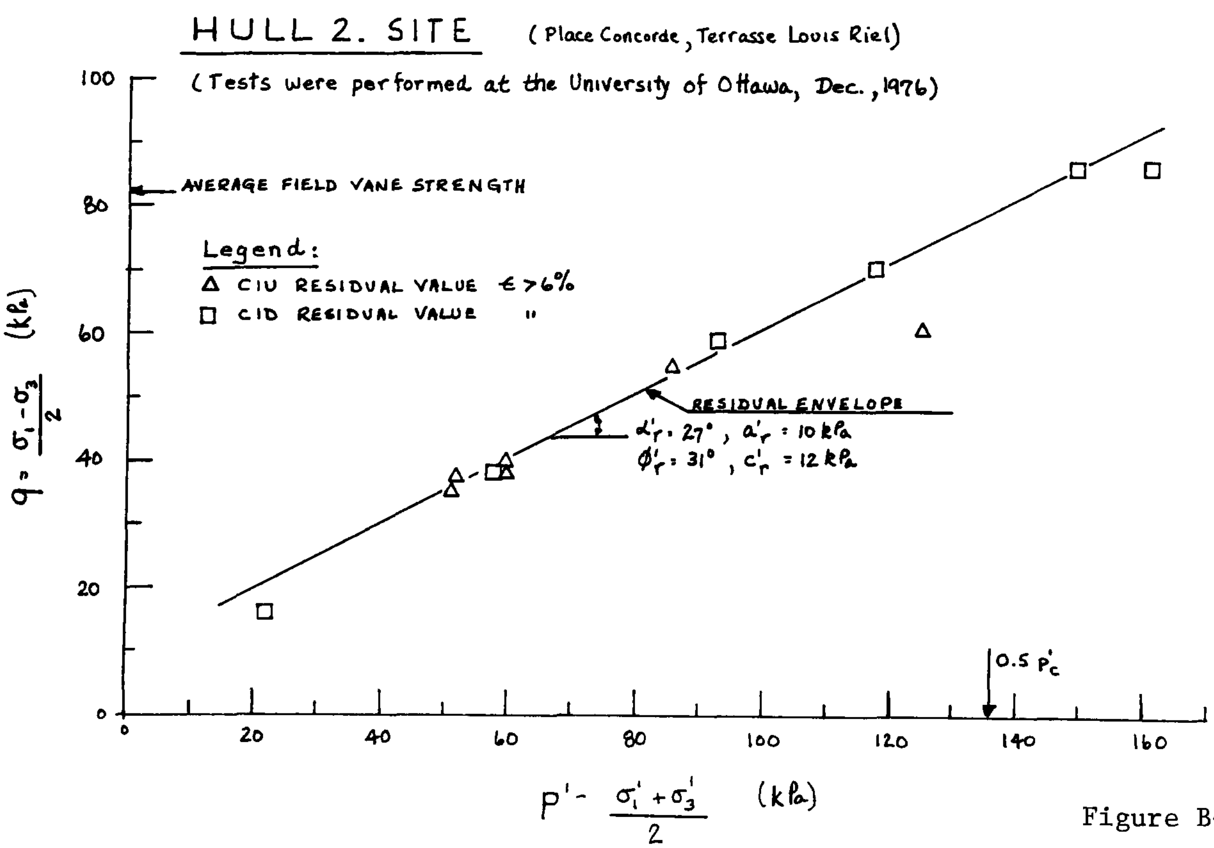

HULL 2. SITE (Place Concorde, Terrasse Louis Riel)
(Tests were performed at the University of Ottawa, Dec., 1976)
AVERAGE FIELD VANE STRENGTH
Legend:
Δ CIU RESIDUAL VALUE ε > 6%
□ CID RESIDUAL VALUE "
RESIDUAL ENVELOPE
α'r = 27°, a'r = 10 kPa
φ'r = 31°, c'r = 12 kPa
0.5 p'c
q = (σ1 - σ3)/2 (kPa)
p' = (σ'1 + σ'3)/2 (kPa)
0
20
40
60
80
100
120
140
160

Figure B-8

APPENDIX C

COMPUTER PROGRAM

DATA SHEET FOR PROGRAM TO PERFORM CALCULATIONS FOR CONSOLIDATED UNDRAINED OR DRAINED TRIAXIAL TESTS

SITE NAME, TEST TYPE AND NUMBER, DATE PERFORMED, SIGMA 3 =

CARD 1

INITIAL TEST DATA: FORMAT F10.3

CARD 2

DO - (CM.)	LO (CM)	CLDC - (CM)	CVDC - (CC.)	LCF	PPCF	STRATE - (IN./MIN)
1–10	11–20	21–30	31–40	41–50	51–60	61–70

CARD 3.

B.P. (PSI) F10.1	C.P. - (PSI) F10.1	SIG 3 - (PSI) F10.1	N I2
1–10	11–20	21–30	31–32

TEST DATA: FORMAT F10.2

DATA PT.	TIME - (MIN.)	LODCEL READOUT	POREPRESS. READOUT	VOLUME CHANGE (CC)	DATA PT.	TIME (MIN)	LODCEL READOUT	POREPRESS READOUT	VOLUME CHANGE - (CC)
	1–10	11–20	21–30	31–40		1–10	11–20	21–30	31–40
1					17				
2					18				
3					19				
4					20				
5					21				
6					22				
7					23				
8					24				
9					25				
10					26				
11					27				
12					28				
13					29				
14					30				
15					31				
16					32				

G.McE, 780120

Sample data form for computer program.

```
      C *************************************************************************
      C
      C COMPUTER PROGRAM TO PERFORM CALCULATIONS FOR CONSOLIDATED DRAINED AND
      C UNDRAINED TRIAXIAL TESTS AND TO PLOT THE RESULTS.
      C
      C
      C *************************************************************************
      C
0001        REAL LO,LODCEL(99),MQ(99),MP(99),MPTOT(99),LOAD(99),LENGTH,LACO,
           1LC(99),P(99),PTOT(99),PP(99),PPPA(99),LCF,PPCF,MSIG3,MSTRAT
0002        REAL MDELPP(99),MAXPP
0003        REAL NAME1(20),NAME2(20),NAME3(20),NAME4(20),NAME5(20),NAME6(20)
0004        DIMENSION X(100),TITLE(99),XP(3),Y(3)
0005        DIMENSION XV(25),YV(25),XH(20),YH(20)
0006        DIMENSION VC(99),A(99),STRAIN(99),VOLCH(99),DEVSTR(99),
           1SIG1(99),SIG1E(99),SIG3E(99),Q(99),DELPP(99),VOL(99),TIME(99),
           2APP(99),EFFRAT(99),TOTRAT(99),CF(99),TITLE2(99)
0007        DIMENSION VERTX(35),VERTY(35),HORX(35),HORY(35)
0008        CALL BPLT(4,2)
      C
      C READ TEST TYPE AND IDENTIFICATION
      C
0009        READ(5,2)(TITLE(I),I=1,20)
0010      2 FORMAT(20A4)
      C
      C READ IN INITIAL TEST DATA
      C
0011        READ(5,3)DO,LO,CLDC,CVDC,LCF,PPCF,STRATE
0012      3 FORMAT(6F10.3,F10.5)
0013        READ(5,4)BP,CP,SIG3,N
0014      4 FORMAT(3F10.2,I2)
      C DO IS THE ORIGINAL DIAMETER IN CM.
      C LO IS THE ORIGINAL LENGTH IN CM.
      C CLDC IS THE CHANGE IN LENGTH DUE TO CONSOLIDATION IN CM.
      C CVDC IS THE CHANGE IN VOLUME DUE TO CONSOLIDATION IN CC.
      C LCF IS THE LOAD CELL TRANSDUCER CALIBRATION FACTOR
      C PPCF IS THE PORE PRESSURE TRANSDUCER CALIBRATION FACTOR
      C STRATE IS THE STRAIN RATE IN INCHES PER MINUTE
      C BP = BACKPRESSURE IN PSI
      C CP = CELL PRESSURE IN PSI
      C SIG3 IS THE CONFINING CELL PRESSURE IN PSI
      C N IS THE NUMBER OF DATA POINTS TO BE READ AND CALCULATED
      C
0015        QPEAK=0.0
0016        MAXPP=0.0
      C
0017        READ(5,5)(TIME(I),LODCEL(I),PP(I),VC(I),I=1,N)
0018      5 FORMAT(4F10.2)
      C TIME IS THE TIME OF THE READING WHICH IS RELATED TO THE STRAIN
      C LODCEL IS THE LOAD RECORDED ON THE LOADCELL IN LBS. AT A PARTICULAR TIME.
      C PP IS THE PRESSURE RECORDED ON THE PORE PRESSURE TRANSDUCER IN PSI
      C VC IS THE VOLUME CHANGE, WHICH OCCURS ONLY IN A DRAINED TEST, IN CC.
      C
      C
      C
      C DETERMINE INITIAL AREA IN SQ. CM.
0019        AO=3.14159*(DO**2)/4.
      C DETERMINE INITIAL VOLUME IN CM CUBED.
0020        VO=LO*AO
      C DETERMINE LENGTH AFTER CONSOLIDATION (LACO) CM.
```

```
      LACO=(LO+CLDC)
C DETERMINE VOLUME AFTER CONSOLIDATION (VACO) CM. CUBED.
      VACO=VO+CVDC
C DETERMINE AREA AFTER CONSOLIDATION IN SQ. CM. (AACO)
      AACO=VACO/LACO
C
C
C PERFORM THE FOLLOWING CALCULATIONS FOR EACH DATA POINT
C
      DO 100 I=1,N
      LC(I)=TIME(I)*STRATE
C LC IS THE LENGTH CHANGE IN THE SAMPLE DUE TO LOADING IN INCHES
C DETERMINE THE NEW AREA AFTER EACH LOAD INCREMENT IN SQ. CM.
      A(I)=AACO*(1.-(VC(I)/VACO))/(1.-(LC(I)*2.54/LACO))
C CHANGE AREA FROM SQ. CM. TO SQ. IN. TO DO CALCULATIONS
      A(I)=A(I)/(2.54**2)
C CALCULATE LOAD ON SAMPLE IN LBS. AND STRAIN IN PERCENT
      LOAD(I)=LODCEL(I)*LCF
      STRAIN(I)=LC(I)*2.54*100./LACO
C CALCULATE VOLUME CHANGE (VOLCH) IN SAMPLE OCCURRING DURING SHEAR FOR
C DRAINED TEST ONLY , IN PERCENT
      VOLCH(I)=VC(I)*100./VACO
C APP IS THE ACTUAL POREPRESSURE IN THE SAMPLE= READOUT TIMES THE CALIBRATION
C FACTOR, IN PSI.
      APP(I) =PP(I)*PPCF
C DELPP IS THE CHANGE IN POREPRESSURE I.E. DELTA U
      DELPP(I)=APP(I)-APP(1)
C SIG3E IS THE EFFECTIVE CONFINING PRESSURE IN P.S.I.
      SIG3E(I)=SIG3-DELPP(I)
C DEVSTR IS THE DEVIATOR STRESS ON THE SAMPLE IN PSI.
      DEVSTR(I)=LOAD(I)/A(I)
C CORRECT THE DEVIATOR STRESS DUE TO THE STRENGTH OF THE MEMBRANE AND
C FILTER PAPER DRAINS
      CF(I)=(0.70+0.167*SIG3)*(STRAIN(I)/10.)
      IF(SIG3.GT.10.0) CF(I)=(2.20+0.017*SIG3)*(STRAIN(I)/10.)
      DEVSTR(I)=DEVSTR(I)-CF(I)
C COMPUTE SIG1 AND SIG1E IN P.S.I.
      SIG1(I)=DEVSTR(I)+SIG3
      SIG1E(I)=SIG1(I)-DELPP(I)
C CALCULATE Q AND P IN TOTAL AND EFFECTIVE STRESSES IN P.S.I.
C QTOT=QEFF
      Q(I)=(SIG1E(I)-SIG3E(I))/2.
      P(I)=(SIG1E(I)+SIG3E(I))/2.
      PTOT(I)=(SIG1(I)+SIG3)/2.
C CONVERT Q AND P TO S.I. UNITS I.E. KILOPASCALS (KPA)
      MQ(I)=Q(I)*6.895
      MP(I)=P(I)*6.895
      MPTOT(I)=PTOT(I)*6.895
      MDELPP(I)=DELPP(I)*6.895
C CALCULATE THE PRINCIPAL STRESS RATIO SIG1/SIG3, SIG1E/SIG3E
      TOTRAT(I)=SIG1(I)/SIG3
      EFFRAT(I)=SIG1E(I)/SIG3E(I)
C CALCULATE THE RATIO OF CHANGE IN POREPRESSURE TO CHANGE IN SIGMA 1
C     I.E. DELPP/SIG1. THIS IS  THE PORE PRESSURE PARAMETER A
      PPPA(I)=DELPP(I)/SIG1(I)
C CONVERT SIG3 FROM PSI TO KG./SQ. CM.
      MSIG3=SIG3*0.007
C CONVERT STRATE FROM IN./MIN. TO CM./MIN.
```

```
0051          MSTRAT=STRATE*2.54
       C DETERMINE THE PEAK VALUE OF Q'
0052          IF(MQ(I).GT.QPEAK)QPEAK=MQ(I)
       C DETERMINE THE MAXIMUM VALUE OF POREPRESSURE MAXPP.
0053          IF(MDELPP(I).GT.MAXPP)MAXPP=MDELPP(I)
0054      100 CONTINUE
0055          WRITE(6,7)(TITLE(I),I=1,20)
0056        7 FORMAT('1',///,' ',20A4,///)
0057          WRITE(6,8)DO,LO,AO,VO,CLDC,CVDC
0058        8 FORMAT('-','DO=',F5.3,'CM.',2X,'LO=',F7.3,'CM.',2X,'AO=',F7.3,
           1'SQ.CM.',2X,'VO=',F8.3,'C.C.',//,' THE CHANGE IN LENGTH DUE TO
           2CONSOLIDATION IS = ',F5.3,'CM.',/,' THE CHANGE IN VOLUME DUE TO
           3CONSOLIDATION IS = ',F7.3,'C.C.')
0059          WRITE(6,12)LACO,AACO,VACO
0060       12 FORMAT('0','THE SAMPLE LENGTH AFTER CONSOLIDATION IS = ',F7.3,
           1'CM. ',4X,'THE SAMPLE AREA AFTER CONSOLIDATION IS= ',F7.3,'SQ.CM.
           2',/,' THE SAMPLE VOLUME AFTER CONSOLIDATION IS= ',F7.3,' C.C.')
0061          WRITE(6,9)BP,CP,SIG3,MSIG3,STRATE,MSTRAT
0062        9 FORMAT('0','THE BACKPRESSURE WAS SET AT ',F5.1,' P.S.I.',4X,
           1'THE CELL PRESSURE WAS SET AT ',F5.1,'P.S.I',/,' THE CONFINING PRE
           2SSURE IS THEN =',F6.2,'PSI= ',F7.5,' KG./SQ.CM.',/,'0','THE RATE '
           3'OF STRAIN WAS= ',F8.6,' IN./MIN. = ',F7.5,'CM./MIN.')
0063          WRITE(6,10)LCF,PPCF
0064       10 FORMAT('0','THE LOAD CELL TRANSDUCER CALIBRATION FACTOR IS = ',
           1F5.3,4X,'THE PORE PRESSURE TRANSDUCER CALIBRATION FACTOR IS = ',
           2F5.3)
0065          WRITE(6,11)N
0066       11 FORMAT('0','THE NUMBER OF DATA POINTS IS ',I2)
0067          WRITE(6,14)
0068       14 FORMAT('0',//,' DATA',4X,'TIME',3X,'STRAIN',5X,'LOAD',3X,'AREA',
           14X,'DEVSTR',3X,'DELPP',5X,'VOLCH',4X,'SIG1',3X,'SIG3',2X,'SIG1E',
           23X,'SIG3E',3X,'  Q''',3X,'    P''',8X,'C.F.')
0069          WRITE(6,15)
0070       15 FORMAT(' ',1X,'PT.',4X,'MIN',3X,'PERCENT',6X,'LBS',2X,'SQ.IN.',
           14X,'PSI',7X,'PSI',5X,'PERCENT',3X,'PSI',4X,'PSI',4X,'PSI',4X,'PSI
           2',4X,'PSI',5X,'PSI',9X,'PSI')
0071          WRITE(6,16)( I ,TIME(I),STRAIN(I),LOAD(I),A(I),DEVSTR(I),DELPP(I),
           1VOLCH(I),SIG1(I),SIG3,SIG1E(I),SIG3E(I),Q(I),P(I),CF(I),I=1,N)
0072       16 FORMAT('0',2X,I2,2X,F5.1,4X,F5.2,4X,F6.2,2X,F5.3,3X,F5.2,4X,F6.2,
           14X,F6.2,3X,F5.2,2X,F5.2,2X,F5.2,3X,F5.2,3X,F5.2,3X,F5.2,6X,F5.2)
0073          WRITE(6,20)
0074       20 FORMAT('1')
0075          WRITE(6,7)(TITLE(I),I=1,20)
0076          WRITE(6,17)
0077       17 FORMAT('-',/,' DATA',2X,'STRAIN',5X,'DELPP',5X,'  Q''',9X,'  P'''
           1,7X,'PTOT',7X,'TOTRAT',5X,' EFFRAT',6X,'  A')
0078          WRITE(6,18)
0079       18 FORMAT(' ',1X,'PT.',1X,'PERCENT',6X,' KPA ',6X,'KPA',10X,'KPA',
           17X,'KPA',6X,'SIG1/SIG3',1X,'SIG1E/SIG3E',2X,'DELPP/SIG1')
0080          WRITE(6,19)(I,STRAIN(I),MDELPP(I),MQ(I),MP(I),MPTOT(I),TOTRAT(I),
           1EFFRAT(I),PPPA(I),I=1,N)
0081       19 FORMAT('0',2X,I2,2X,F5.2,6X,F6.2,4X,F6.2,7X,F6.2,4X,F6.2,3X,F10.2
           1,5X,F7.2,6X,F6.2)
0082          WRITE(6,24)QPEAK
0083       24 FORMAT('-','THE PEAK STRENGTH = ',F6.2,' KPA')
0084          WRITE(6,25)
0085       25 FORMAT('1')
       C
```

```
C THIS PART OF THE PROGRAM PLOTS THE RESULTS OF THE COMPUTATIONS
C
C PLOT THE AXES
      CALL ORG(-12.,-7.0)
      DO 30 J=1,3
      DO 29 I=1,2
      READ(5,98)XP(I),Y(I)
   98 FORMAT(2F10.1)
   29 CONTINUE
      CALL LIN(XP,Y,2)
   30 CONTINUE
C INCREMENT THE AXES IN UNITS OF 5  KPA FOR THE Y AXIS AND UNITS OF .5 PERCENT
C  FOR THE X AXIS
      DO 75 I=1,25
      YV(I)=I*1.0
      XV(I)=0.0
   75 CONTINUE
      CALL SLEW(0.0,0.0)
      CALL DOT(XV,YV,22)
      DO 77 I=1,8
      YV(I)=I*(-1.0)
      XV(I)=0.0
   77 CONTINUE
      CALL DOT(XV,YV,8)
      XV(I)=20.
      YV(1)=0.0
      DO 78 I=1,25
      YV(I)=I*1.0
      XV(I)=20.
   78 CONTINUE
      CALL DOT(XV,YV,22)
      DO 79 I=1,8
      YV(I)=I*(-1.0)
      XV(I)=20.
   79 CONTINUE
      CALL DOT(XV,YV,8)
      DO 76 I=1,20
      XH(I)=I*1.0
      YH(I)=0.0
   76 CONTINUE
      CALL SLEW(0.0,0.0)
      CALL DOT(XH,YH,20)
C READ AND PRINT THE NAMES OF THE AXES
      READ(5,33)(NAME1(I),I=1,5)
      READ(5,33)(NAME2(I),I=1,5)
      READ(5,33)(NAME3(I),I=1,5)
      READ(5,33)(NAME4(I),I=1,5)
      READ(5,33)(NAME5(I),I=1,5)
      READ(5,33)(NAME6(I),I=1,5)
   33 FORMAT(5A4)
C READ AND PRINT THE NAMES OF THE AXIS
      CALL SYM(NAME1,-2.0,10.0,-20)
      CALL SYM(NAME2,-2.0,-0.5,-13)
      CALL SYM(NAME3,6.0,1.0,20)
      CALL SYM(NAME4,22.0,3.0,-16)
      CALL SYM(NAME5,22.0,-2.0,-19)
C PRINT THE TEST TYPE AND IDENTIFICATION DATA
      CALL SYM(TITLE,3.0,20.0,80)
```

```
C  PLOT THE VARIATION OF Q WITH STRAIN
      CALL SLEW(STRAIN(1),MQ(1))
      CALL SCAL(0.5,5.0)
      IF(QPEAK.GT.115.0) CALL SCAL(0.5,10.0)
      CALL DOT(STRAIN,MQ,N)
C PLOT THE VARIATION OF EFFRAT WITH STRAIN
      CALL SLEW(STRAIN(1),EFFRAT(1))
      CALL SCAL(0.5,2.0)
      CALL LIN(STRAIN,EFFRAT,N)
      CALL SCAL(1.0,1.0)
      CALL ORG(-12.0,-15.0)
C PLOT THE VARIATION OF PORE PRESSURE PARAMATER   A  WITH STRAIN
      CALL SLEW(STRAIN(1),PPPA(1))
      CALL SCAL(0.5,0.1)
      CALL LIN(STRAIN,PPPA,N)
C PLOT THE VARIATION OF DELTA PORE PRESSURE WITH STRAIN
      CALL SLEW(STRAIN(1),MDELPP(1))
      CALL SCAL(0.5,5.0)
      IF(MAXPP.GT.40.0.AND.MAXPP.LT.80.0) CALL SCAL(0.5,10.0)
      IF(MAXPP.GT.80.0.AND.MAXPP.LT.160.) CALL SCAL(0.5,20.0)
      IF(MAXPP.GT.160..AND.MAXPP.LT.320.) CALL SCAL(0.5,40.0)
      CALL DOT(STRAIN,MDELPP,N)
      CALL EPLT
      CALL QUIT
C PLOT THE STRESS PATH FOLLOWED TO FAILURE
      CALL BPLT(4,2)
      CALL ORG(-12.0,-12.0)
C INCREMENT THE AXES IN UNITS OF 5 KPA
      DO 65 I=1,30
      VERTY(I)=I*1.0
      VERTX(I)=0.0
   65 CONTINUE
      DO 66 I=1,30
      HORX(I)=I*1.0
      HORY(I)=0.0
   66 CONTINUE
      CALL SLEW(0.000,0.000)
      CALL DOT(VERTX,VERTY,25)
      CALL SLEW(0.000,0.000)
      CALL DOT(HORX,HORY,25)
      CALL SYM(TITLE,3.0,20.0,80)
C PRINT THE TITLE
      READ(5,62)(TITLE2(I),I=1,20)
   62 FORMAT(20A4)
      CALL SYM(TITLE2,3.0,18.0,80)
      CALL SYM(NAME1,-2.0,9.0,-15)
      CALL SYM(NAME6,10.0,-2.0,20)
C PLOT THE EFFECTIVE STRESS PATH
      CALL SCAL(5.0,5.0)
      IF(QPEAK.GT.115.0) CALL SCAL(10.0,10.0)
      IF(SIG3.GE.14.5)CALL SCAL(10.0,10.0)
      IF(SIG3.GE.29.0)CALL SCAL(20.0,20.0)
      CALL SLEW(MP(1),MQ(1))
      CALL DOT(MP,MQ,N)
C PLOT THE TOTAL STRESS PATH
      CALL SLEW(MPTOT(1),MQ(1))
      CALL LIN(MPTOT,MQ,N)
      CALL EPLT
```

```
      CALL QUIT
      WRITE(6,329)
  329 FORMAT('1','THE SCALES USED IN THE PLOTS ARE ...')
      IF(QPEAK.LT.115.0) WRITE(6,330)
  330 FORMAT('0','Q'' SCALE...1 UNIT= 5 KPA')
      IF(QPEAK.GT.115.0) WRITE(6,331)
  331 FORMAT('0','Q'' SCALE...1 UNIT= 10 KPA')
      IF(MAXPP.LT.40.) WRITE(6,332)
      IF(MAXPP.GT.40.0.AND.MAXPP.LT.80.0) WRITE(6,333)
      IF(MAXPP.GT.80.0.AND.MAXPP.LT.160.) WRITE(6,334)
      IF(MAXPP.GT.160..AND.MAXPP.LT.320.) WRITE(6,335)
  332 FORMAT('0','PORE PRESSURE SCALE...1 UNIT= 5 KPA')
  333 FORMAT('0','PORE PRESSURE SCALE...1 UNIT= 10 KPA')
  334 FORMAT('0','PORE PRESSURE SCALE...1 UNIT= 20 KPA')
  335 FORMAT('0','PORE PRESSURE SCALE...1 UNIT= 40 KPA')
      WRITE(6,401)
  401 FORMAT('0','STRAIN SCALE...1 UNIT= 0.5 PERCENT',//,' EFFRAT SCALE.
     1..1 UNIT= 2 ',//,' PORE PRESSURE PARAMETER A SCALE...1 UNIT= 0.1')
      IF(SIG3.LT.14.5) WRITE(6,339)
      IF(SIG3.GE.14.5.AND.SIG3.LT.29.) WRITE(6,338)
  339 FORMAT('0','STRESS PATH SCALE...1 UNIT= 5 KPA')
      IF(QPEAK.GT.115..AND.SIG3.LT.29.) WRITE(6,338)
  338 FORMAT('0','STRESS PATH SCALE... 1 UNIT= 10 KPA')
      IF(SIG3.GE.29.0) WRITE(6,336)
  336 FORMAT('0','STRESS PATH SCALE...1 UNIT= 20 KPA')
      WRITE(6,20)
      RETURN
      END
```

STEWARTVILLE CIU 5-1-6 78- 04-01 SIGMA3= 230KPA

D0=5.050CM. L0= 10.100CM. A0= 20.030SQ.CM. V0= 202.299C.C.

THE CHANGE IN LENGTH DUE TO CONSOLIDATION IS = 0.0 CM.
THE CHANGE IN VOLUME DUE TO CONSOLIDATION IS = -0.850C.C.

THE SAMPLE LENGTH AFTER CONSOLIDATION IS = 10.100CM. THE SAMPLE AREA AFTER CONSOLIDATION IS= 19.945SQ.CM.
THE SAMPLE VOLUME AFTER CONSOLIDATION IS= 201.449 C.C.

THE BACKPRESSURE WAS SET AT 29.0 P.S.I. THE CELL PRESSURE WAS SET AT 62.4P.S.I
THE CONFINING PRESSURE IS THEN = 33.36PSI= 0.23352 KG./SQ.CM.

THE RATE OF STRAIN WAS= 0.001300 IN./MIN. = 0.00330CM./MIN.

THE LOAD CELL TRANSDUCER CALIBRATION FACTOR IS = 0.278 THE PORE PRESSURE TRANSDUCER CALIBRATION FACTOR IS = 1.090

THE NUMBER OF DATA POINTS IS 26

DATA PT.	TIME MIN	STRAIN PERCENT	LOAD LBS	AREA SQ.IN.	DEVSTR PSI	DELPP PSI	VOLCH PERCENT	SIG1 PSI	SIG3 PSI	SIG1E PSI	SIG3E PSI	Q' PSI	P' PSI	C.F. PSI
1	0.0	0.0	0.0	3.092	0.0	0.0	0.0	33.36	33.36	33.36	33.36	0.0	33.36	0.0
2	3.0	0.10	3.61	3.095	1.14	0.55	0.0	34.50	33.36	33.96	32.81	0.57	33.39	0.03
3	6.0	0.20	5.56	3.098	1.74	0.98	0.0	35.10	33.36	34.12	32.38	0.87	33.25	0.05
4	9.0	0.29	8.34	3.101	2.61	1.42	0.0	35.97	33.36	34.55	31.94	1.30	33.25	0.08
5	12.0	0.39	12.23	3.104	3.83	2.29	0.0	37.19	33.36	34.90	31.07	1.92	32.99	0.11
6	15.0	0.49	35.58	3.107	11.32	6.32	0.0	44.68	33.36	38.36	27.04	5.66	32.70	0.14
7	18.0	0.59	64.77	3.110	20.67	11.34	0.0	54.03	33.36	42.69	22.02	10.33	32.36	0.16
8	21.0	0.69	91.46	3.113	29.19	15.59	0.0	62.55	33.36	46.96	17.77	14.60	32.37	0.19
9	24.0	0.78	114.81	3.116	36.63	19.18	0.0	69.99	33.36	50.81	14.18	18.31	32.49	0.22
10	27.0	0.88	135.66	3.119	43.25	22.02	0.0	76.61	33.36	54.59	11.34	21.63	32.97	0.24
11	30.0	0.98	153.73	3.122	48.97	24.31	0.0	82.33	33.36	58.02	9.05	24.48	33.54	0.27
12	33.0	1.08	169.02	3.125	53.78	25.83	0.0	87.14	33.36	61.31	7.53	26.89	34.42	0.30
13	36.0	1.18	180.98	3.128	57.53	26.92	0.0	90.89	33.36	63.96	6.44	28.76	35.20	0.33
14	39.0	1.28	190.43	3.131	60.46	27.58	0.0	93.82	33.36	66.24	5.78	30.23	36.01	0.35
15	42.0	1.37	194.32	3.135	61.61	27.69	0.0	94.97	33.36	67.29	5.67	30.81	36.48	0.38
16	45.0	1.47	181.53	3.138	57.45	25.72	0.0	90.81	33.36	65.08	7.64	28.72	36.36	0.41
17	48.0	1.57	168.19	3.141	53.12	23.43	0.0	86.48	33.36	63.04	9.93	26.56	36.48	0.43
18	54.0	1.77	163.74	3.147	51.54	22.89	0.0	84.90	33.36	62.01	10.47	25.77	36.24	0.49
19	66.0	2.16	159.29	3.160	49.82	21.91	0.0	83.18	33.36	61.27	11.45	24.91	36.36	0.60
20	87.0	2.84	157.07	3.182	48.57	21.15	0.0	81.93	33.36	60.79	12.21	24.29	36.50	0.79
21	102.0	3.33	124.54	3.198	38.02	20.38	0.0	71.38	33.36	51.00	12.98	19.01	31.99	0.92
22	120.0	3.92	118.43	3.219	35.72	20.73	0.0	69.08	33.36	48.15	12.43	17.86	30.29	1.09
23	135.0	4.41	114.26	3.234	34.11	20.93	0.0	67.47	33.36	46.54	12.43	17.05	29.48	1.22
24	165.0	5.39	109.81	3.268	32.11	21.15	0.0	65.47	33.36	44.32	12.21	16.06	28.27	1.49
25	210.0	6.87	107.03	3.319	30.34	21.15	0.0	63.70	33.36	42.56	12.21	15.17	27.39	1.90
26	240.0	7.85	107.31	3.355	29.82	21.04	0.0	63.18	33.36	42.14	12.32	14.91	27.23	2.17

STEWARTVILLE CIU 5-1-6 78- 04-01 SIGMA3= 230KPA

DATA PT.	STRAIN PERCENT	DELPP KPA	Q' KPA	P' KPA	PTOT KPA	TOTRAT SIG1/SIG3	EFFRAT SIG1E/SIG3E	A DELPP/SIG1
1	0.0	0.0	0.0	230.02	230.02	1.00	1.00	0.0
2	0.10	3.76	3.93	230.19	233.95	1.03	1.03	0.02
3	0.20	6.76	6.00	229.25	236.02	1.05	1.05	0.03
4	0.29	9.77	8.99	229.24	239.01	1.08	1.08	0.04
5	0.39	15.78	13.21	227.45	243.23	1.11	1.12	0.06
6	0.49	43.59	39.02	225.45	269.04	1.34	1.42	0.14
7	0.59	78.16	71.25	223.10	301.26	1.62	1.94	0.21
8	0.69	107.47	100.64	223.18	330.65	1.88	2.64	0.25
9	0.78	132.27	126.28	224.02	356.30	2.10	3.58	0.27
10	0.88	151.81	149.11	227.31	379.12	2.30	4.81	0.29
11	0.98	167.60	168.82	231.24	398.83	2.47	6.41	0.30
12	1.08	178.12	185.42	237.32	415.44	2.61	8.15	0.30
13	1.18	185.63	198.32	242.70	428.33	2.72	9.94	0.30
14	1.28	190.14	208.43	248.31	438.45	2.81	11.45	0.29
15	1.37	190.89	212.41	251.53	442.43	2.85	11.86	0.29
16	1.47	177.37	198.05	250.70	428.07	2.72	8.52	0.28
17	1.57	161.58	183.11	251.55	413.13	2.59	6.35	0.27
18	1.77	157.83	177.69	249.89	407.70	2.54	5.92	0.27
19	2.16	151.06	171.74	250.70	401.76	2.49	5.35	0.26
20	2.84	145.80	167.46	251.67	397.48	2.46	4.98	0.26
21	3.33	140.54	131.07	220.55	361.09	2.14	3.93	0.29
22	3.92	144.30	123.14	208.86	353.16	2.07	3.87	0.30
23	4.41	144.30	117.58	203.30	347.60	2.02	3.74	0.31
24	5.39	145.80	110.70	194.92	340.72	1.96	3.63	0.32
25	6.87	145.80	104.61	188.82	334.63	1.91	3.48	0.33
26	7.85	145.05	102.79	187.76	332.81	1.89	3.42	0.33

THE PEAK STRENGTH = 212.41 KPA

```
THE SCALES USED IN THE PLOTS ARE ...
Q' SCALE...1 UNIT= 10 KPA
PORE PRESSURE SCALE...1 UNIT= 40 KPA
STRAIN SCALE...1 UNIT= 0.5 PERCENT
EFFRAT SCALE...1 UNIT= 2
PORE PRESSURE PARAMETER A SCALE...1 UNIT= 0.1
STRESS PATH SCALE...1 UNIT= 20 KPA
```

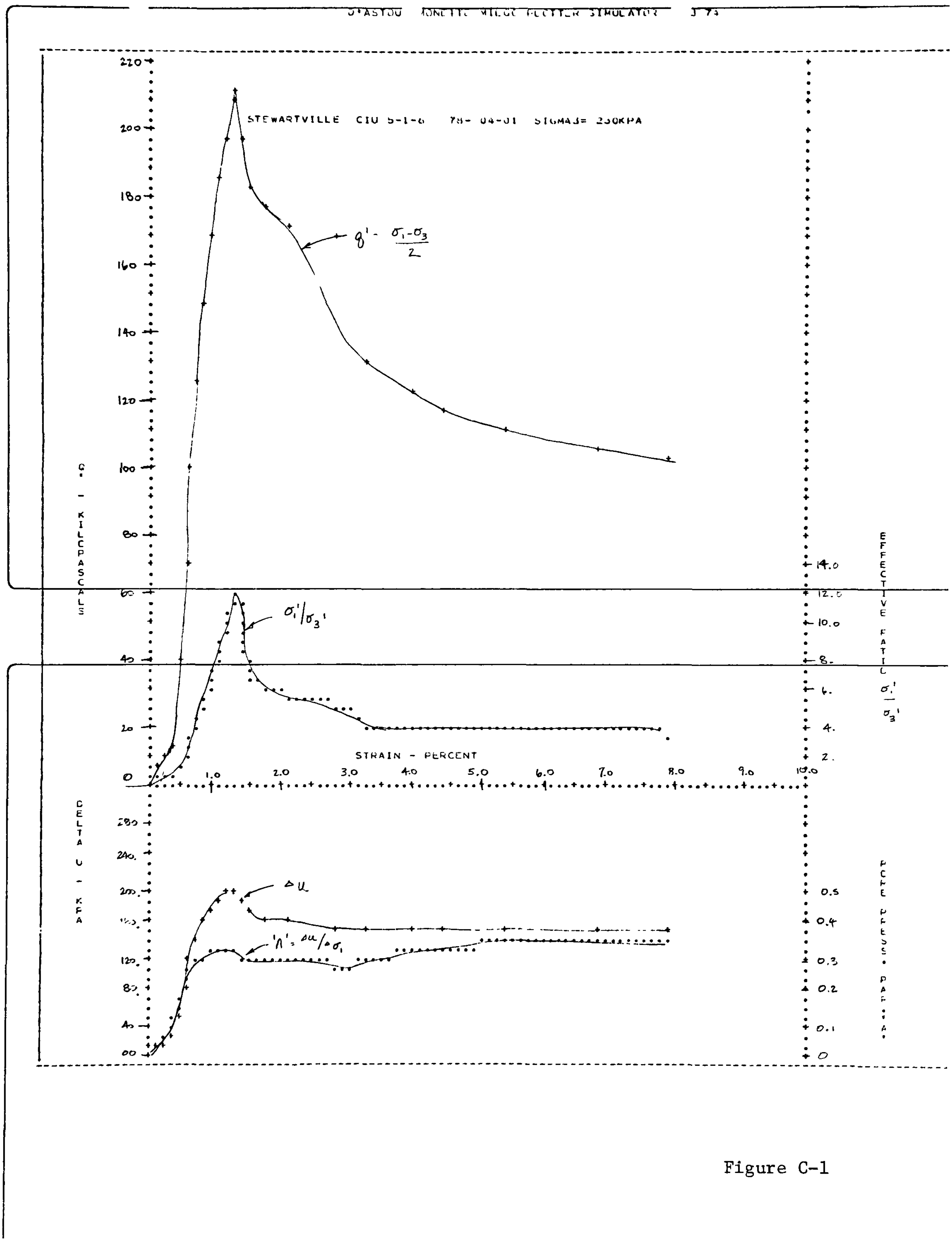

STEWARTVILLE CIU 5-1-6 78- 04-01 SIGMA3= 250KPA
q' - $\frac{\sigma_1-\sigma_3}{2}$
σ_1'/σ_3'
Δu
'A' = $\Delta u/\Delta\sigma_1$
STRAIN - PERCENT
Q - KILOPASCALS
DELTA U - KPA
EFFECTIVE RATIO σ_1'/σ_3'
PORE PRESS. PAR. A

Figure C-1

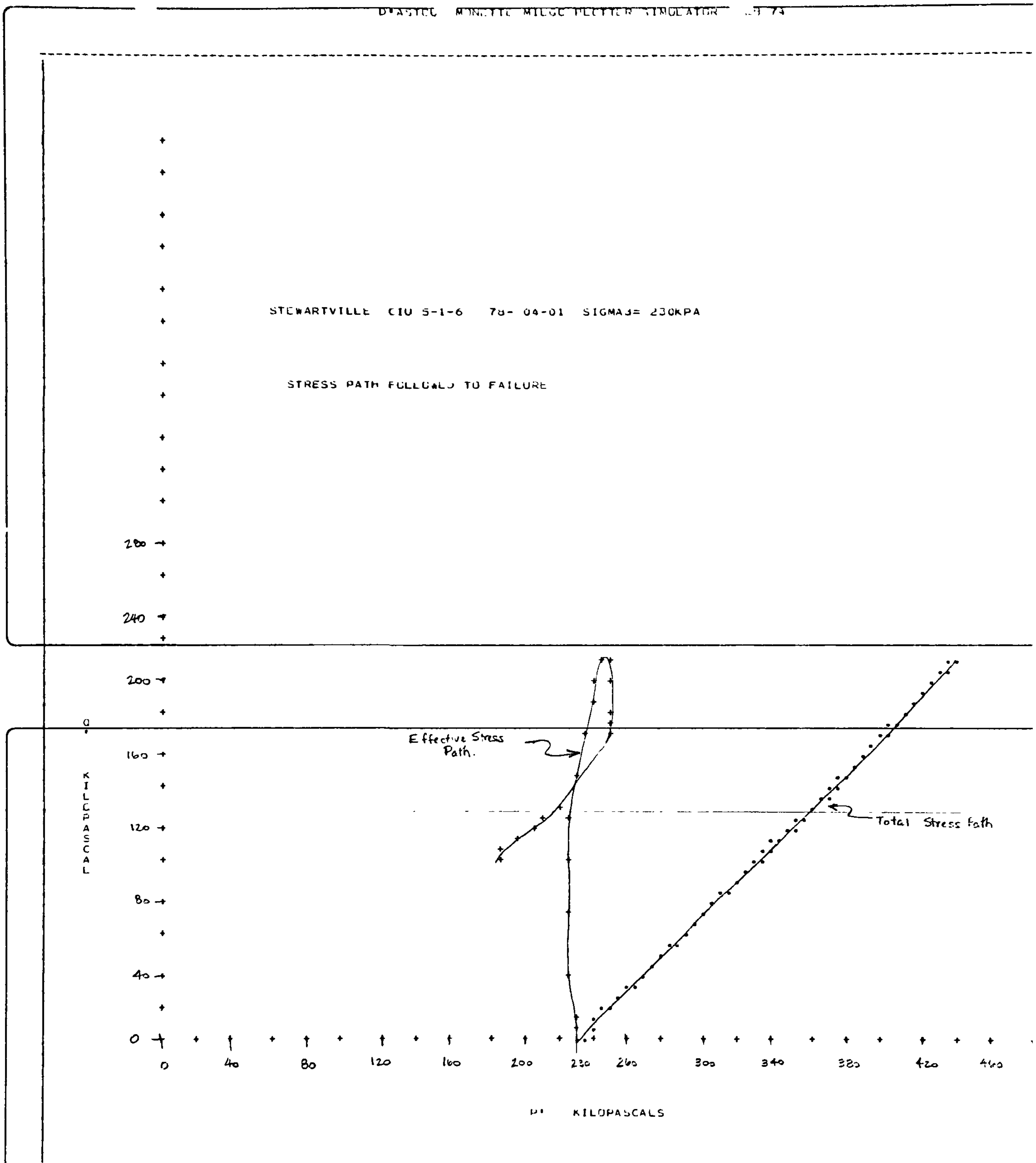
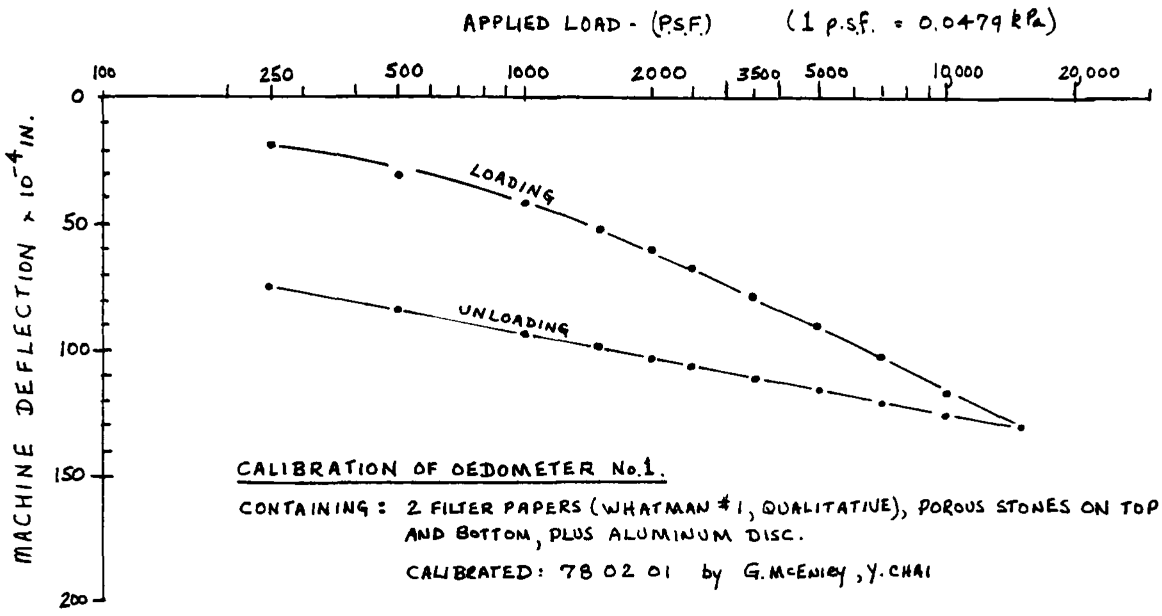

STEWARTVILLE CIU 5-1-6 78- 04-01 SIGMA3= 230KPA
STRESS PATH FOLLOWED TO FAILURE
Effective Stress Path.
Total Stress Path
q KILOPASCAL
p' KILOPASCALS

Figure C-2

APPENDIX D

CALIBRATIONS


APPLIED LOAD - (P.S.F.) (1 p.s.f. = 0.0479 kPa)
100
250
500
1000
2000
3500
5000
10,000
20,000
0
50
100
150
200
MACHINE DEFLECTION × 10^-4 IN.
LOADING
UNLOADING
CALIBRATION OF OEDOMETER No.1.
CONTAINING: 2 FILTER PAPERS (WHATMAN #1, QUALITATIVE), POROUS STONES ON TOP AND BOTTOM, PLUS ALUMINUM DISC.
CALIBRATED: 78 02 01 by G. McENIRY, Y. CHAI

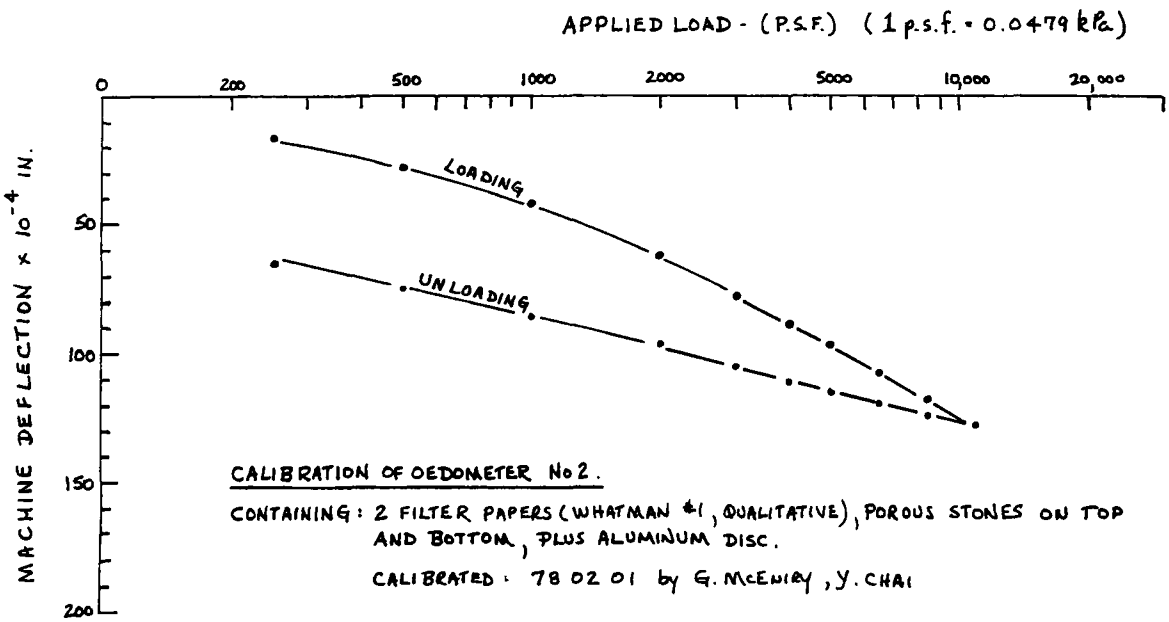
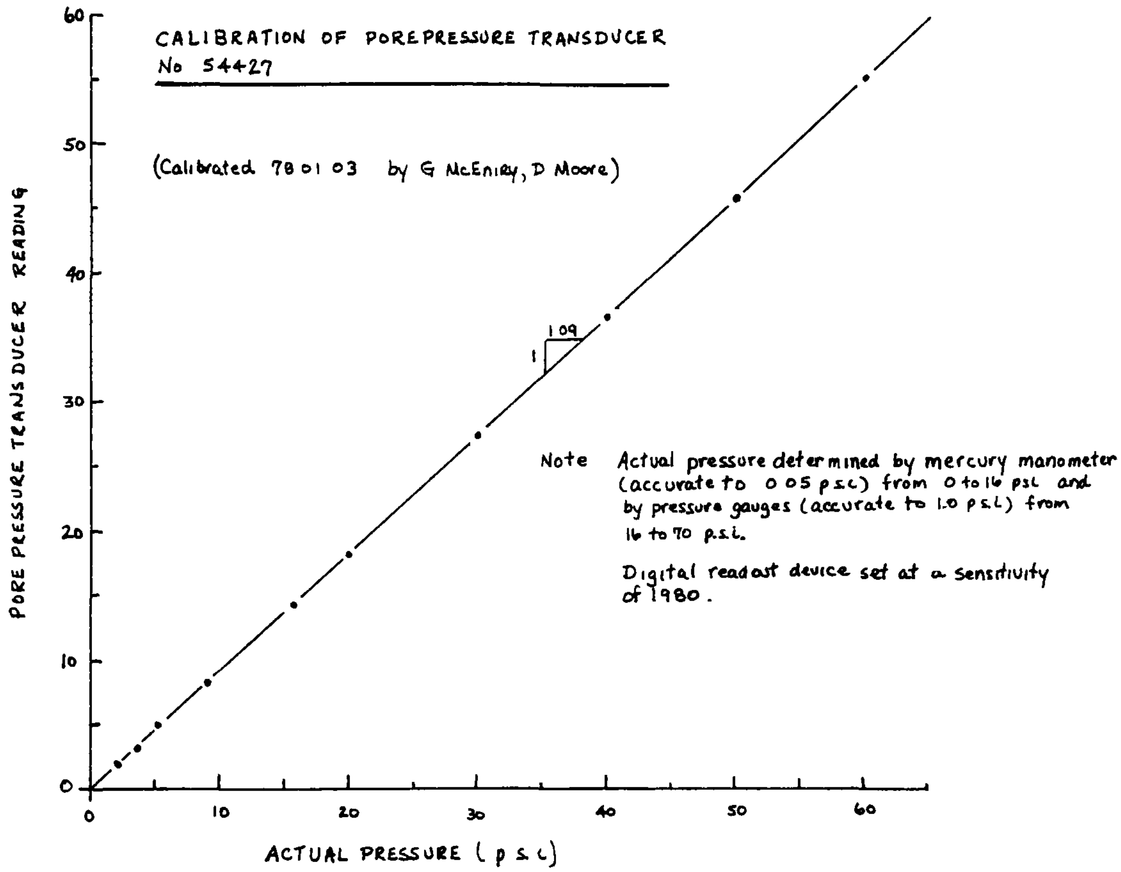

APPLIED LOAD - (P.S.F.) (1 p.s.f. = 0.0479 kPa)
0
200
500
1000
2000
5000
10,000
20,000
50
100
150
200
MACHINE DEFLECTION × 10^-4 IN.
LOADING
UNLOADING
CALIBRATION OF OEDOMETER No 2.
CONTAINING: 2 FILTER PAPERS (WHATMAN #1, QUALITATIVE), POROUS STONES ON TOP AND BOTTOM, PLUS ALUMINUM DISC.
CALIBRATED: 78 02 01 by G. McENIRY, Y. CHAI

Figure D-1


CALIBRATION OF POREPRESSURE TRANSDUCER
No 54427
(Calibrated 78 01 03 by G McEniry, D Moore)
Note Actual pressure determined by mercury manometer (accurate to 0 05 p.s.i.) from 0 to 16 psi and by pressure gauges (accurate to 1.0 p.s.i.) from 16 to 70 p.s.i.
Digital readout device set at a sensitivity of 1980.
1.09
1
PORE PRESSURE TRANSDUCER READING
ACTUAL PRESSURE (p.s.i)
0
10
20
30
40
50
60

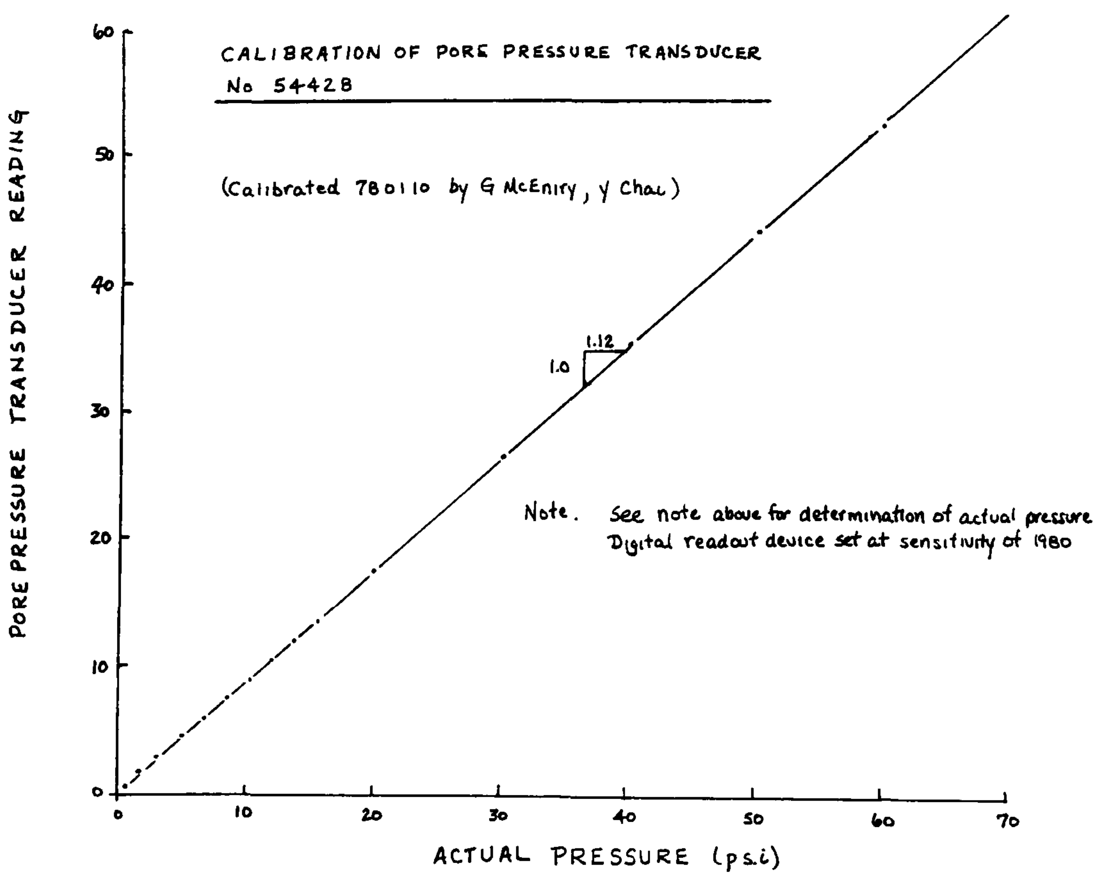

CALIBRATION OF PORE PRESSURE TRANSDUCER
No 54428
(Calibrated 780110 by G McEniry, Y Chau)
Note. See note above for determination of actual pressure
Digital readout device set at sensitivity of 1980
1.12
1.0
PORE PRESSURE TRANSDUCER READING
ACTUAL PRESSURE (p.s.i)
0
10
20
30
40
50
60
70

Figure D-2

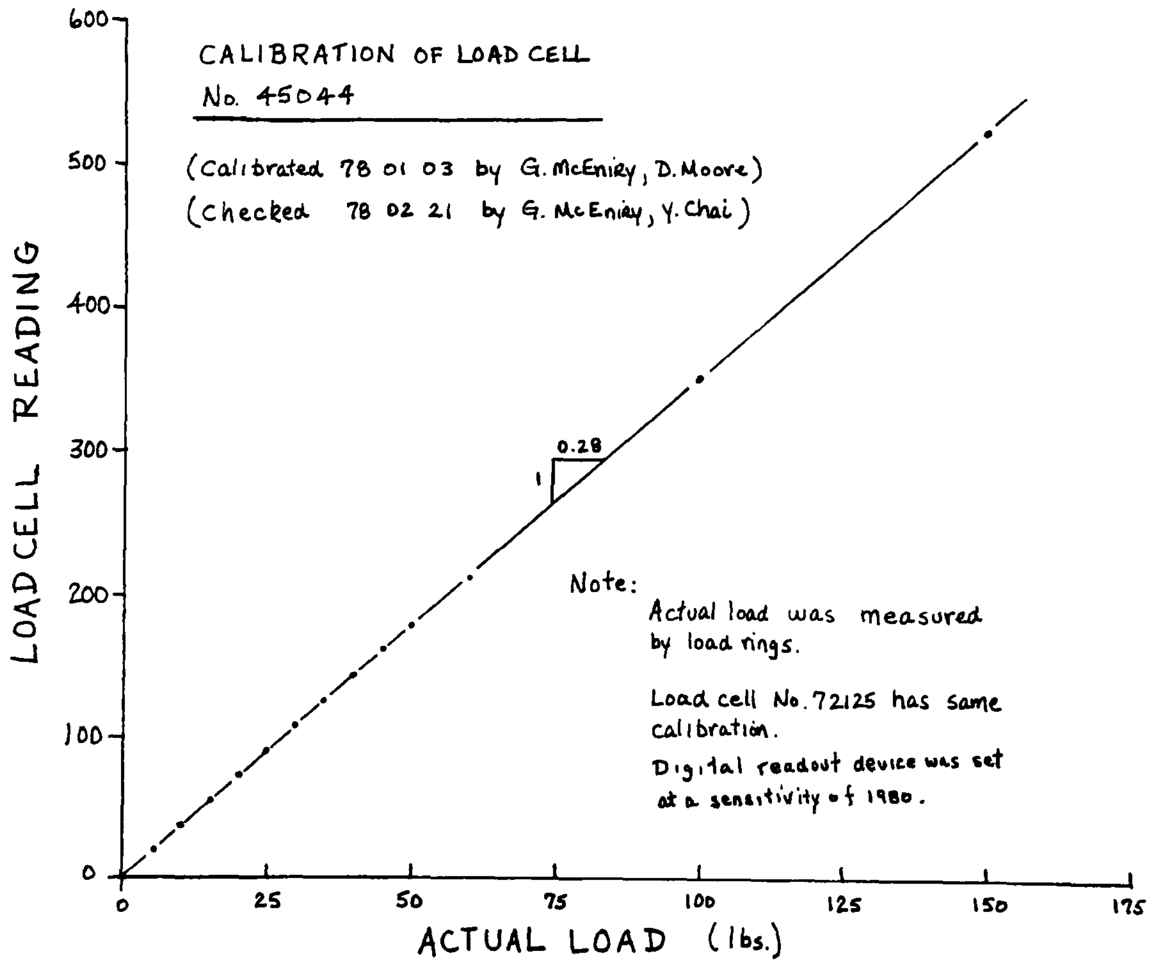

CALIBRATION OF LOAD CELL
No. 45044
(Calibrated 78 01 03 by G. McEniry, D. Moore)
(Checked 78 02 21 by G. McEniry, Y. Chai)
LOADCELL READING
ACTUAL LOAD (lbs.)
0.28
1
Note:
Actual load was measured by load rings.
Load cell No. 72125 has same calibration.
Digital readout device was set at a sensitivity of 1980.
0
100
200
300
400
500
600
25
50
75
100
125
150
175

Figure D-3