THE EFFECTIVE SHEAR STRENGTH BEHAVIOUR OF THREE FISSURED CLAYS

by

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SYNOPSIS

A better understanding of the behaviour of fissured clay of Eastern Ontario in shear is required, as well as the influence of bond strength upon its shear strength parameters.

As a limited amount of relevant literature was available, the behaviour in shear of three fissured clays was studied. Also investigated was the influence of clay strength upon such properties as volume change during shear and volume change during isotropic consolidation, strain during shear, field vane strength, and preconsolidation pressure. An empirical relationship between effective stress shear strength and field vane strength is proposed.

ACKNOWLEDGEMENTS

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LIST OF SYMBOLS

u	Pore pressure
u_w	Pore water pressure
u_a	Pore air pressure
σ_n	Normal stress
σ'_n	Effective normal stress
σ_1	Major principal stress
σ_2	Intermediate principal stress
σ_3	Minor principal stress
P_m	Constant mean normal stress $(\frac{\sigma'_1+2\sigma'_3}{3})$
τ	Shear stress
ε	Linear strain
γ	Unit weight of soil
γ_s	Unit weight of solid particles
γ_w	Unit weight of water
γ_d	Unit weight of dry soil
γ'	Unit weight of submerged soil
e	Void ratio
n	Porosity
w	Water content
S_t	Degree of saturation
w_L	Liquid limit
w_P	Plastic limit
I_P	Plasticity index

I_L	Liquidity index
c_v	Coefficient of consolidation
t	Consolidation time
τ_f	Shear strength
c'	Effective cohesion intercept
ϕ'	Apparent angle of internal friction
c_u	Apparent cohesion intercept
ϕ_u	Apparent angle of internal friction
S_{ℓ}	Sensitivity

CHAPTER 1

INTRODUCTION

1.1 General

Presented is a study of the behaviour of three Eastern Ontario fissured Leda clays in shear. The influence of bond strength upon volume change from consolidation and shear, strain from shear, field vane strength, and preconsolidation pressure is investigated.

At present information describing behaviour in shear for fissured clays of Eastern Ontario is limited. Deviator strength envelopes for a few fissured clays from the Ottawa area are available in the literature.

This research was carried out to achieve a better understanding of the behaviour in shear of these fissured clays.

1.2 Statement of Problem

A study of the behaviour of the Eastern Ontario clays in shear leads to three main questions.

1. How can the action of the clay in shear be best described?
2. Is there any relationship between effective stress shear strength and other properties of the fissured clays?
3. Do the shear strengths of the fissured clays tested differ from each other and also from those in the literature from the Ottawa area?

It was proposed to provide answers to these questions by investigating the behaviour of three fissured clays in shear.

1.3 Objectives of the Study

Little is known of the behaviour in shear of fissured clays of Eastern Ontario, except perhaps those in the Ottawa area. The purpose of this study was to improve that knowledge by investigating and describing the behaviour in shear of fissured clay deposits at Castor River, South Nation, and Bear Brook in Russell County, approximately 25 miles south-east of Ottawa. This was the primary objective.

An additional objective was to determine if there is a relationship between the effective stress shear strength and other more easily measured clay properties.

1.4 Outline of Thesis

Following is a chapter-by-chapter summary of thesis contents.

Chapter 2 - Introduced are the origins, properties, and the two major known classes of Champlain Sea Clay; fissured and intact.

Chapter 3 - Beginning with the classical Mohr-Coulomb failure envelope this Chapter follows the development of the effective shear strength envelope to one of three regions; and the shear mechanisms in each. Also discussed is the concept of peak and post-peak deviator strength.

Chapter 4 - Factors such as rubber membranes, piston friction and sample disturbance which may influence interpretation of triaxial deviator strength results are assessed.

Chapter 5 - Description of equipment is followed by choice of a stress path and selection of a working normal stress range for the testing programme. This is followed by a summary of test procedures employed.

Chapter 6 - Index properties, site locations, soil profiles and preconsolidation pressures for the fissured clays tested and some clays from the literature are discussed.

Chapter 7 - This Chapter deals with the behaviour during consolidation and shear of the three fissured clays tested. Also investigated is the influence of bond strength upon properties of the fissured clays. A yield curve is established and the shear strength envelopes from testing are presented.

Chapter 8 - Presented are other curves of shear strength from the literature for fissured clays. Properties of clays from the literature and the clays tested are compared. A correlation between field vane strength and the strength envelopes for the fissured clays is proposed.

Chapter 9 - Summarized are conclusions regarding the behaviour in shear for the three fissured clays tested, and also objectives achieved. Recommendations and suggestions for future work are also made.

CHAPTER 2

GEOLOGY AND CHAMPLAIN SEA CLAYS

2.1 General

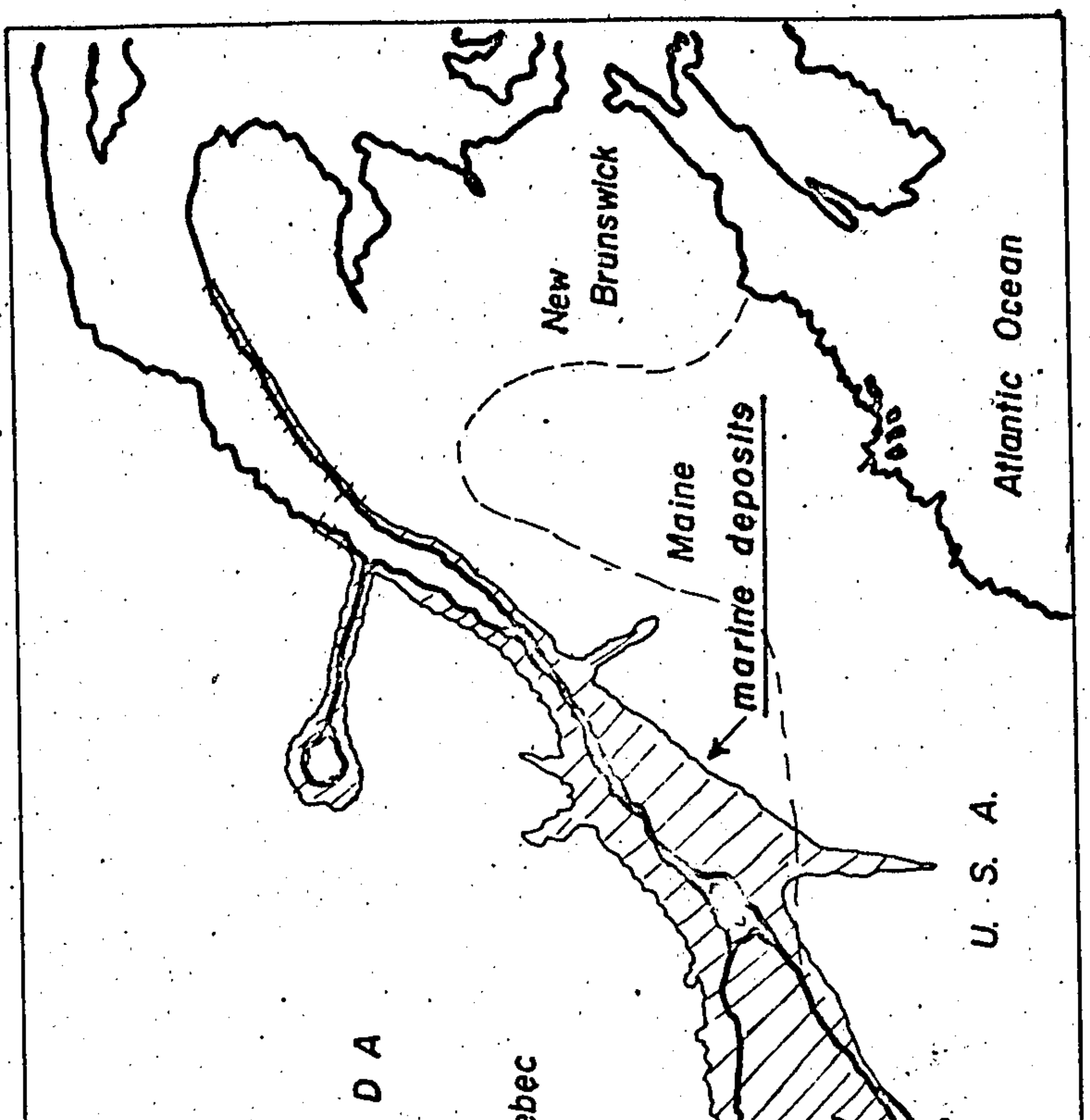
This Chapter deals with the origins, extent and general properties of the clays deposited in the Champlain Sea. It appears that there exist two different clay deposits. A Leda clay, thought to be fissured, is found predominantly at the surface in Eastern Ontario. A considerably less fissured or intact Champlain clay is usually encountered below this fissured clay in Ontario and at the surface in Quebec. Some of the properties of these two clays are discussed.

2.2 Geology

The reasons for variation in shear strength of the Marine clays of Eastern Canada lie in the geological history of the deposits, that is, in the environment of deposition and in changes subsequent to deposition.

At one time, most of the northern part of this continent was covered by the Wisconsin ice sheet, whose great weight depressed the land several hundred feet (Crawford, 1968). As this glacier retreated, the present St. Lawrence Valley was inundated by the marine waters of the Champlain Sea. The glacial melt waters, bearing sediments from till and bedrock, deposited a soft silty marine clay on the sea bed, eventually covering most topographical features (Gadd, 1962). This sea bed, at its greatest extent 12,000 years ago (Karrow, 1961), covered an area bounded by Pembroke, Kingston, Montreal, Quebec City, the Lake St. John District, and the shoreline of the lower St. Lawrence as far as Anticosti Island (Figure 2.2.1).

The land, relieved of the weight of retreating glacier, rebounded, and the Champlain Sea became shallower. Due to a sudden influx of fresh water, from either the Great Lakes or glacial sources, the salinity of the Champlain Sea was greatly reduced (Gadd, 1962). In some locations, the top of the original marine clay stratum was eroded and redeposited in this fresh water environment. The erosional agent is thought to have been turbulence from fresh water rivers flowing into the St. Lawrence Basin from the newly established drainage system in the surrounding lowlands (Gadd, 1962). Approximately 8,000 years ago, the bed of the Champlain Sea,



EASTERN CANADA AND THE CHAMPLAIN SEA

from Legget, Burn, & Bozozuk (1961)

figure 2.2.1.

rebounding after the melting of the glacier, rose above sea level, resulting in a clay plain.

The name "Leda Clay" to denote the clay of the St. Lawrence Basin is attributed (Karrow, 1961) to the nineteenth century geologist J. W. Dawson. La Rochelle et al (1970) suggest the name Champlain Clay.

Sangrey and Paul (1971) propose that there are two distinct clays in the Ottawa area, using geotechnical evidence to confirm Gadds geological findings. This seems to be confirmed by shear strength data and stability analyses of clays in the Ottawa area.

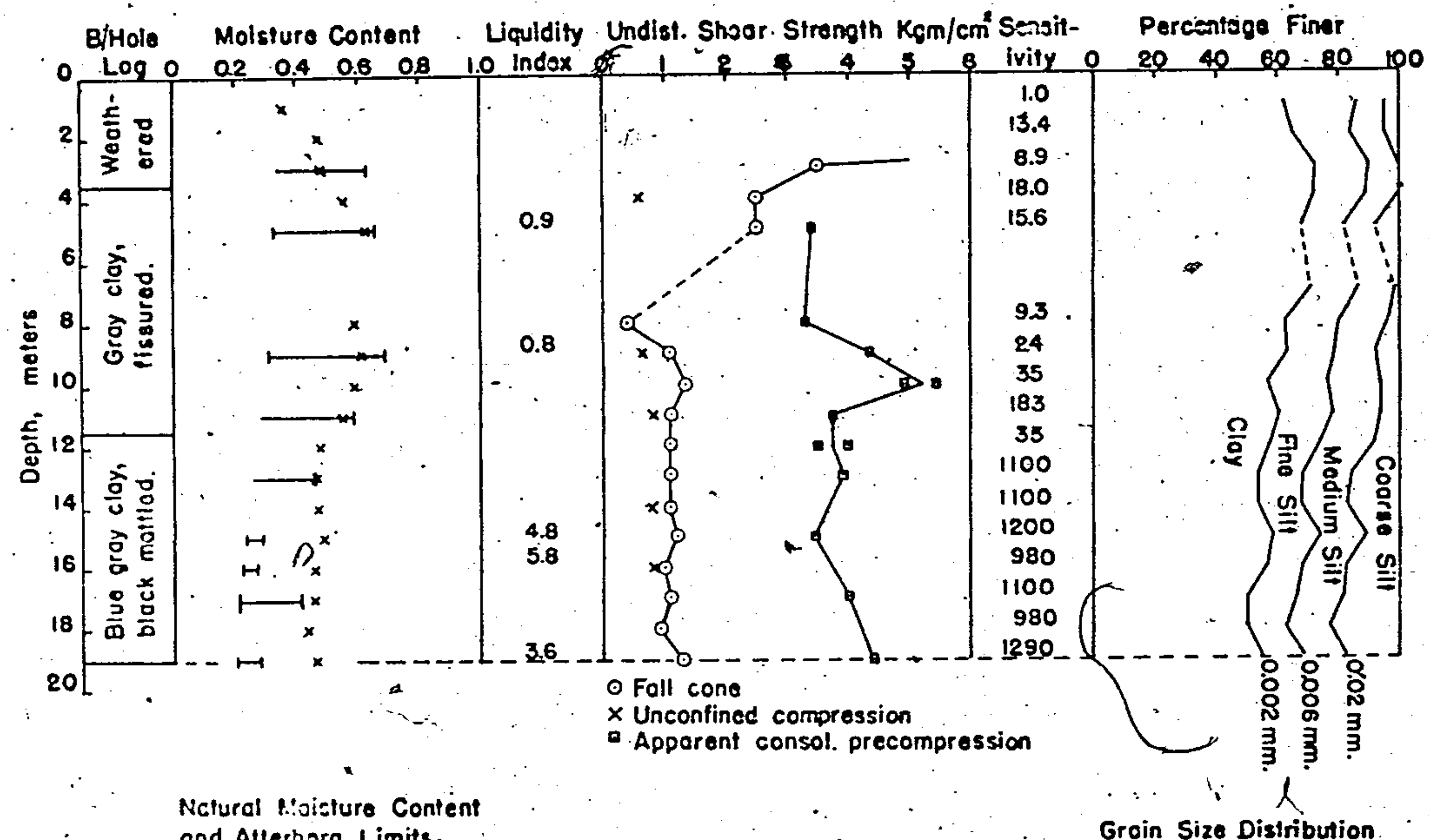
Since literature pertaining to local Ottawa clays has used the nomer of Leda, this will be adopted. The clays shall be described as intact and fissured Leda clays. Present evidence indicates that the lower clay (the original deposit) is an intact clay while the upper or redeposited clay contains micro-fissuring throughout its depth (Sangrey and Paul, 1971).

2.3 Fissured and Intact Champlain Clays

There are thought to be two clays of widely varying properties in the Ottawa area. A fissured fresh water clay overlies an intact saline clay. In some locations, this intact or considerably less fissured clay has not been deeply eroded and therefore occurs at or near the surface (notably in Hull, and at Greens Creek). This freshwater clay appears to be found mostly in Eastern Ontario, where salinity may have been reduced by incoming glacial melt water which eroded the intact marine clay. Further East, around Quebec City, currents from fresh incoming glacial melt water possibly had little ability to rework intact clay. Also water here may have been more saline, being closer to the ocean.

The intact clay in the Ottawa region is more silty and less plastic than the fissured clay. Intact clay can be visually distinguished from fissured clay by crumbling the clay between the fingers. Fissured clay will fracture into small nodules, less than 1 cm, whereas intact or considerably less fissured clays will tend to not crumble. Since both have similar water contents, the liquidity index of intact clays is higher (Figure 2.3.1). The upper clay generally is gray in colour and contains red or pink horizontal bands while the lower clay is a blue-grey colour with black mottling.

Although the undisturbed undrained strength of the two clays is essentially the same, the lower, intact clay is considerably more sensitive. Sangrey and Paul (1971) have noted that the intact clays have an effective fissure



Engineering index properties for the soil profile at Green Creek Golf Course slide.

from Sangrey and Paul (1971)

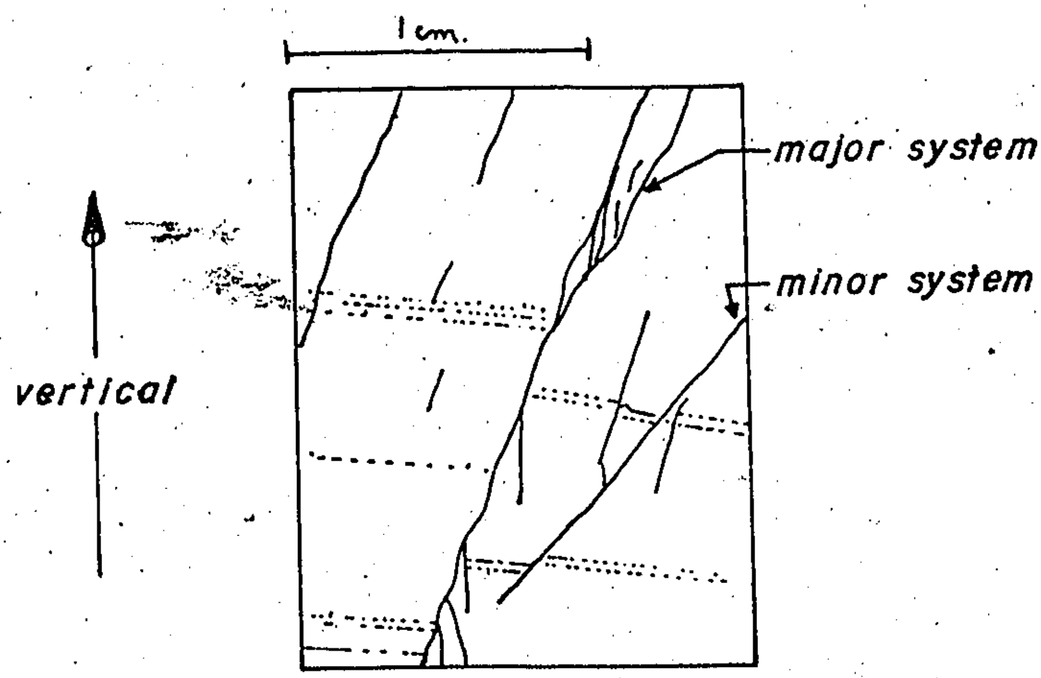
figure 2.3.1

spacing of 15-20 cm, and Rockcliffe fissured clay a spacing of less than 1 cm. Jarrett (1972) observed silt bands and fissures in an optically thin section of Rockcliffe Leda Clay (Figure 2.3.2). A system of fissures existed at 60° to 70° to the horizontal. The orientation of the clay particles in the fissured zones were found to lie in the direction of the fissures. Silt bands were observed to be vertically displaced across these fissures or rather lines. The effect of the closely spaced fissures in the upper clay is important in slope stability analysis. Typical shear strength envelopes for the two clays are shown in Figure 2.3.3.

Analysis of porewater has shown (Sangrey and Paul, 1971) the intact clay to have higher concentrations of sodium, iron and manganese; and the fissured clay to have higher portions of potassium, calcium and magnesium (Figure 2.3.4). It is the ratio of sodium to calcium that is often used to describe the clays. Sangrey and Paul suggest that flow sliding is restricted to fissured reworked clays (lower sodium to calcium ratio), however, massive flow slides have occurred further east in the intact Champlain clays.

What aotually has caused the fissures in fissured clay is not fully known. The older, intact, marine clay is fissure resistant, yet the younger, fissured

Fissures in
Leda Clay



Fissures and silt bands in an optical thin section of Rockcliffe Leda clay

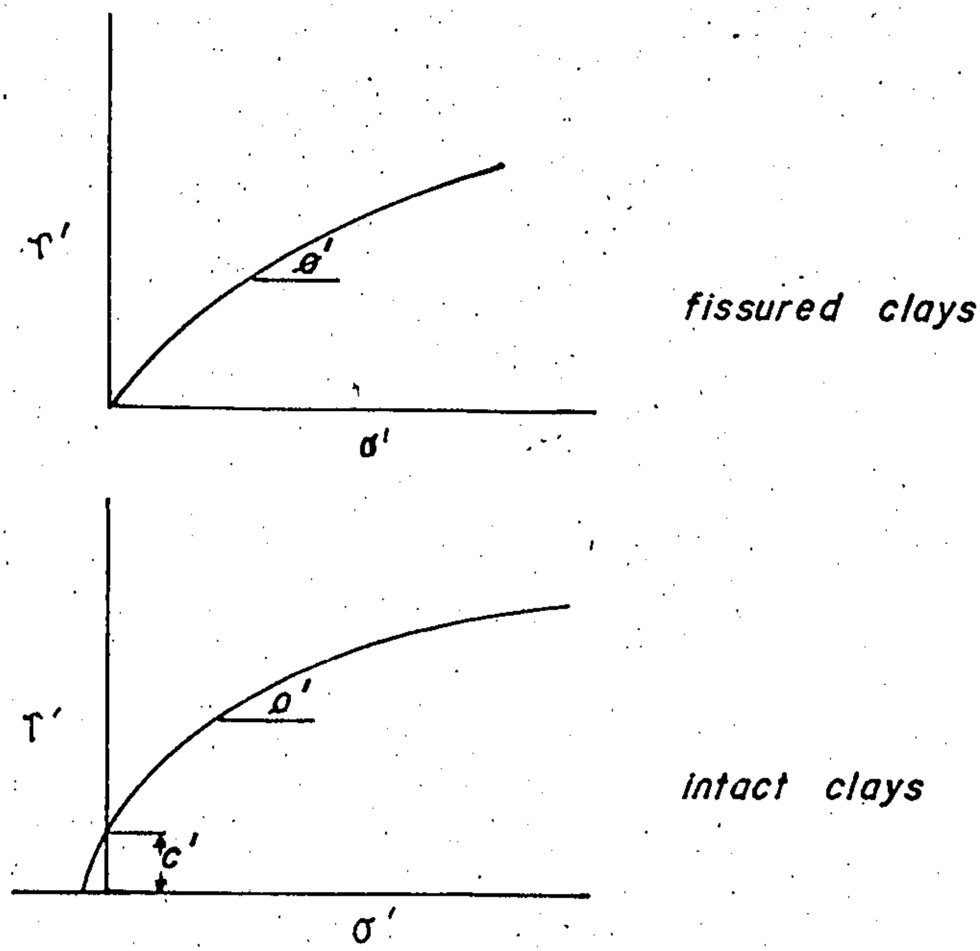
magnification of scale: 4x

····· silt bands
—— fissures

from Jarrett (1972)

figure 2.3.2

Representative Shear Strength Envelopes

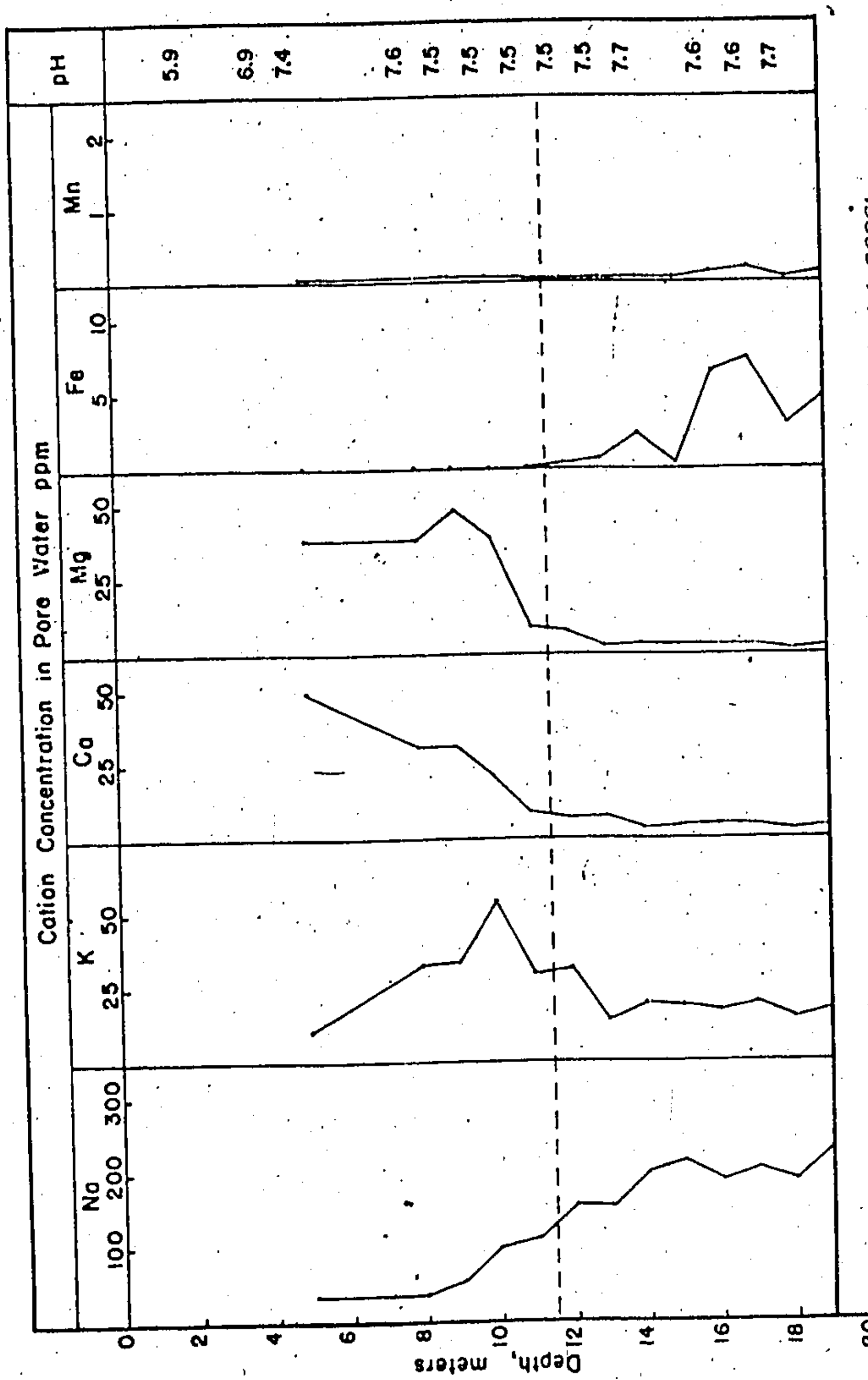


σ' = effective normal stress

τ' = effective shear strength

figure 2.3.3

Pore Water Cation Concentration



Cation content of the soil pore water from AA analysis, Green Creek Golf Course slide, borehole GCGC1.

from Sangrey and Paul (1971)

figure 2.3.4

fresh water clay is fissure susceptible. Mitchell (1970) postulates that the microfissures "may be a result of stress relief due to overburden removal and slope cutting. The formation of microfissures by this mechanism would depend primarily on the relationship between the swelling pressures and strength of cementation bonds in the clay." Another cause of fissuring or faulting in the fissured clays may be due to the cyclic fatiguing effect of seasonal changes in temperature and groundwater over long periods of time.

2.4 Properties of Leda Clays

Crawford (1968) in his paper "Quick Clays of Eastern Canada" has amply described Leda Clays in terms of composition, structure, sensitivity, compressibility, and over-consolidation.

His observations are mainly based on Leda clays of the Ottawa area (more likely fissured rather than intact), but are probably representative of the clay deposits studied in this thesis (Table 2.4.1).

The work by this writer has been confined to a study of the engineering properties of the soils; the index properties, vane strength, preconsolidation pressures, and mainly the effective stress shear strength properties.

Soil Properties Ottawa Leda Clay

SUMMARY OF SOIL PROPERTIES

	Location	Surface elev., ft	Depth, ft	p'	S_u	p_c	S_u/p_c	S_t	SC	Averages: w, %	I_p, %*	I_L	−2μ, %	γ, lb/cu.ft.	Std. error, kg/sq.cm.: p_c	S_u	Remarks
A	Breckenridge	330	20 100	.67 2.11	.36 .91	1.2 2.9	.30 .31	20- 150	0.4- 3.4	79	36	1.1	80	96	±.21	±.07	Uniform clay. Low carbonates. Drying crust extends to 20 ft. Extensive clay deposit to great depths.
B	Sewage plant	171	20 70	.38 1.41	1.00 1.00	3.0 4.1	.33 .25	25- 500+	.1- 2.1	53	19	2.0	66	105	—	±.15	Highly plastic clay to 50 ft, medium sensitivity. Below 50 ft, somewhat coarser, extremely sensitive.
BB	Sewage plant	171	30 70	.66 1.41	1.47 1.87	4.5 5.2	.33 .37								—	—	
C	Green Creek Fill	182	20 100	.68 2.38	.75 1.98	3.1 4.9	.21 .40	5- 25	.6- 13.7	66	40	0.0	70	101	±.44	±.20	Medium to highly plastic clay throughout. Salt content increases linearly from 0.6 grams/liter to 13.7 grams/liter. High carbonates at depth—same clay terrace as B.
D	Green Creek Slide	277	20 90	.69 2.02	.64 1.02	2.0 3.0	.32 .31	20- 500	.1- .5	69	34	1.7	77	101	±.34	±.12	Fine clay throughout. Drying crust to 20 ft. Extremely sensitive at greater depths. Higher terrace than B.
E	National Research	310	30 60	.83 1.40	.55 .77	1.8 2.3	.30 .33	15 500+	.1- 1.5	68	20	2.5	66	98	±.37	±.11	Clay throughout becoming siltier with depth. Drying crust extends to 30 ft. Extremely sensitive at depth. Isolated clay valley.
F	Main St.	222	15 65	.52 1.71	.57 1.03	2.3 3.9	.25 .26	15- 500+	.14 .6	51	22	1.7	50	112	±.19	±.15	Highly plastic clay at surface becoming siltier with depth. Extremely sensitive at depth. Isolated clay pocket.
G	National Museum	235	20 50	.86 1.15		1.6 2.1		10- 150	.1- 1.8	68	10	2.2	56	105	—	—	Same as for F.
H	Heron Rd.	250	20 90	.70 2.20	.58 .75			20- 70	.3- .5	49	11	2.1	51	108	—	±.11	Highly plastic clay becoming siltier and extremely sensitive with depth.
I	Gloucester	260	10 60	.16 .96	.21 .56	0.6 1.7	.35 .34	30 500+	.5- 2.1	72	23	2.0	71	99	±.15	±.06	Fine clay throughout. High water content, low plasticity, high sensitivity. Carbonates high. Extensive level clay plain. Drying crust to about 10 ft.
J	Kars	280	20 50	.38 .95	.29 .52	1.2 2.0	.21 .26	25- 500+	.2- .7	60	20	2.0	60	101	±.22	±.07	Extremely sensitive clay layer from 20 to 50 ft. Alluvium and drying crust to 20 ft.
K	Walkley Rd.	2[illegible]	15 45		.50 .70										—	±.07	Highly plastic clay—bedrock less than 50 ft.
KK	Walkley Rd.	255	33		.62	2.2	.28	50	1.7	58	28		62				

p' = effective overburden stress, kg/sq.cm.; S_u = undrained shear strength, kg/sq.cm.; p_c = preconsolidation pressure, kg/sq.cm.; SC = salt content grams/liter.

from Crawford and Eden (1966)

table 2.4.1

2.5 The Leda Clay Crust

It is known that the Leda clay crust, often highly overconsolidated by weathering, is much stronger than the unweathered clay beneath it. Webb (1970) has shown this for a clay in Ottawa South (Note Table 2.5.1). It appears that the crust at this location extends to about a 10 to 12 foot depth. The thickness of the Leda clay crust on a slope where the rate of erosion is approximately equal to the rate of crustal formation may be thin. Eden and Mitchell (1973) in analyzing failed slopes in the Ottawa region, found that the depth of initial failure "extended below the depth of active weathering" in most cases.

Although the crustal strength would have some influence on the stability of slopes, it is likely that a potential failure arc would develop mainly in the weaker stratum below the crust, where resistance to shear would be considerably less. It would appear that the predominate shearing resistance is provided by the unweathered clay beneath the crust.

TABLE 2.5.1

Applied effective normal stress (lb/in^2)	Depth of Sample (ft)	Measured effective shear strength (lb/in^2)
35	5	35
35	10	16
35	Greater Depth	11

CHAPTER 3

THE MOHR-COULOMB FAILURE ENVELOPE

3.1. General

This Chapter begins by discussing the classical Mohr-Coulomb failure envelope and its application to the shear strength of soils. The development of the Coulomb failure envelope for soils from a straight line to a curve approximated by three straight lines is outlined.

Following this is a note on the transformation of the deviator strength envelope to the shear strength envelope. The effect of anisotropy in a fissured clay, and peak deviator strength analysis is also discussed.

3.2 The Classical Mohr-Coulomb Failure Envelope

Generally, clay soils in terms of effective stresses are assigned properties of strength of a fixed cohesion, c', and a constant angle of internal friction, ϕ', represented by a Mohr-Coulomb failure envelope (Figure 3.2.1). This Mohr envelope is tangent to a series of Mohr circles. These Mohr circles represent limiting combinations of effective shear stress and effective normal stress. This straight line envelope of slope tan ϕ' intercepts the τ' axis at c' and is identified by the equation $\tau' = \sigma' \tan \phi' + c'$. All points on this line satisfy failure criteria.

3.3 The Failure Envelope of Leda Clay

It was recognized by Crawford and Eden (1967) that the use of the classical Mohr envelope derived from high stress range testing did not adequately describe the response of the Leda clay soil for the requirements of slope stability analysis. They considered that an extrapolation to a lower stress range of triaxial shear strength results, obtained using an effective normal stress range greater than the preconsolidation pressure, to be invalid. Bond breakdown caused by consolidation above P_c resulted in the clay behaving in a normally consolidated manner (Figure 3.3.1, Group 1 tests).

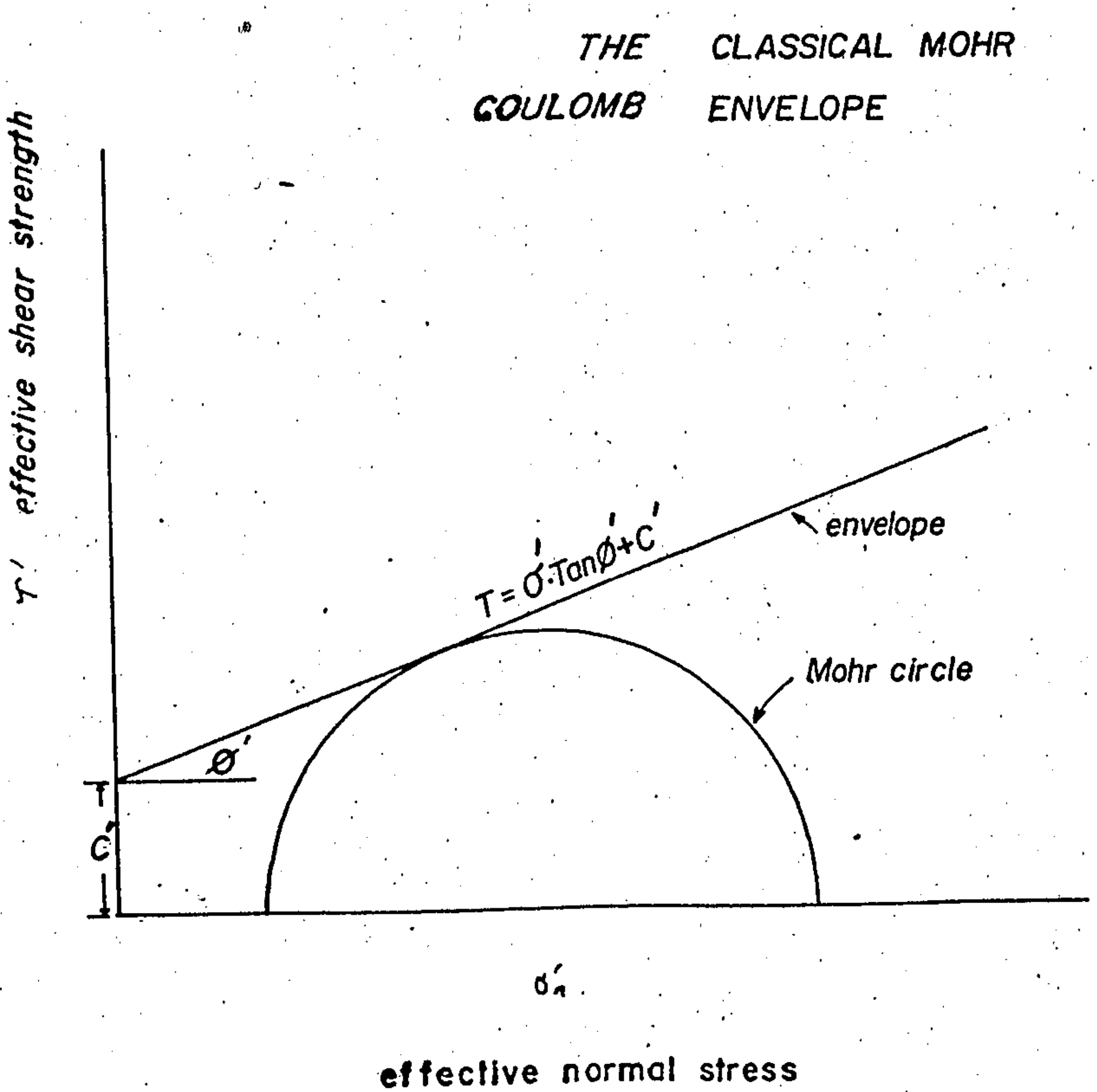
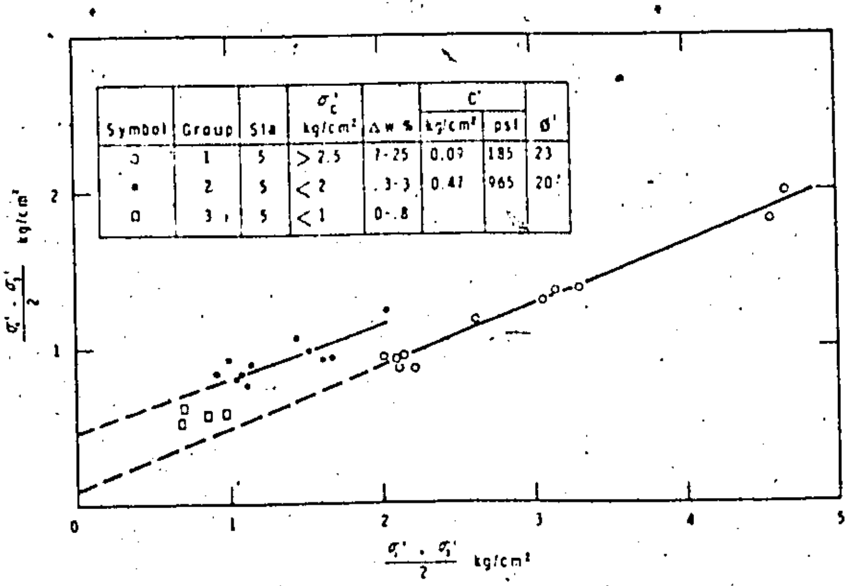

THE CLASSICAL MOHR
COULOMB ENVELOPE
τ' effective shear strength
T = σ'·Tanφ'+C'
envelope
Mohr circle
φ'
C'
σ'
effective normal stress

figure 3.2. I.

Shear Strength, Breckenridge



Symbol	Group	Sta	σ_c' kg/cm²	Δw %	C' kg/cm²	C' psi	Ø'
○	1	5	> 2.5	7-25	0.09	195	23
•	2	5	< 2	.3-3	0.47	965	20
□	3	5	< 1	0-.8			

STRENGTH TEST RESULTS, BRECKENRIDGE

from Crawford and Eden (1967)

figure 3.3.1

Using the same Breckenridge clay, they conducted two more test groups. One at effective normal pressures between 1.0 and 2.0 kg/cm^2, and the other between 0.6 and 1.0 kg/cm^2. Crawford and Eden discounted the validity of the low stress range tests because they postulated that failure was premature due to low lateral effective stresses. For purposes of stability analysis, they relied upon their group 2 tests shown in Figure 3.3.1 in the intermediate effective normal stress range.

This paper by Crawford and Eden was important in that it helped to initiate investigations into more justifiable methods of obtaining shear strength data for stability analysis. Townsend et al. (1969) continued investigating the effect of effective normal stress upon shear strength in the stress ranges just below the preconsolidation pressure and found that cementation dominates, and that shear strength was independent of effective normal stress. They proposed that the failure envelope at effective normal stresses immediately below the preconsolidation pressure was a straight, horizontal line (Note Figure 3.3.2).

Mitchell (1970b), testing Breckenridge clay in the effective stress range below that of

Intermediate Stress

Range Testing

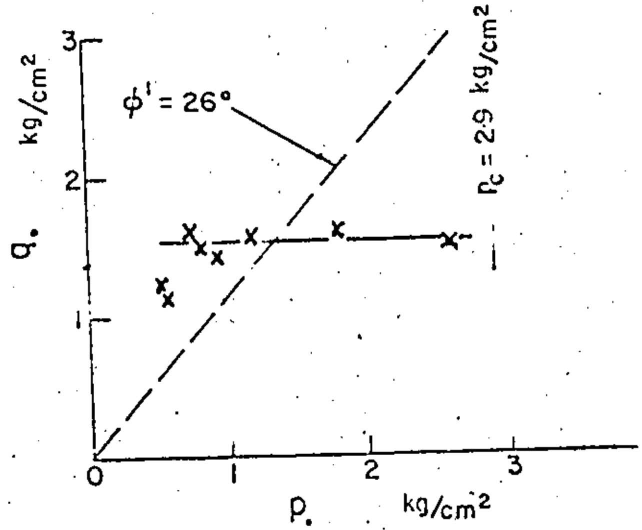


Figure 5 - The compressive strength of Leda clay.

$^*q_\circ = (\sigma_1' - \sigma_3')$
$p_\circ = (\sigma_1' + 2\sigma_3')/3$

x - *Results from strain controlled triaxial tests*

*from Townsend, Sangrey, & Walker (1969)

figure 3.3-2.

Townsend et al., found that shear strength in this low stress range was predominantly frictional (dependent on effective normal stress).

The shear strength test data from Crawford and Eden (1967) and Mitchell (1970b) have been combined and result in a failure envelope which may be divided into three regions according to effective normal stress (Figure 3.3.3). These three divisions (Figure 3.3.4) are denoted as low, intermediate, and high.

In the low stress range, the envelope is essentially frictional, with a high ϕ' and little or no cohesion. In the intermediate stress range, ϕ' is essentially zero, and cohesion is quite large. In the high stress range, above the preconsolidation pressure, the soil behaves frictionally in a normally consolidated manner. In this range ϕ' is about 2/3 of its value in the low stress range and the cohesion is small or zero.

Jarrett (1970) constructed a special testing cell, using the method of Bishop and Garga (1969), to test samples of Rockcliffe clay below an effective normal stress of about 1-1/2 lb/in^2. Due to the frictional nature of the soil he was not able to develop any tensile strength in the clay tested, in nine of the ten

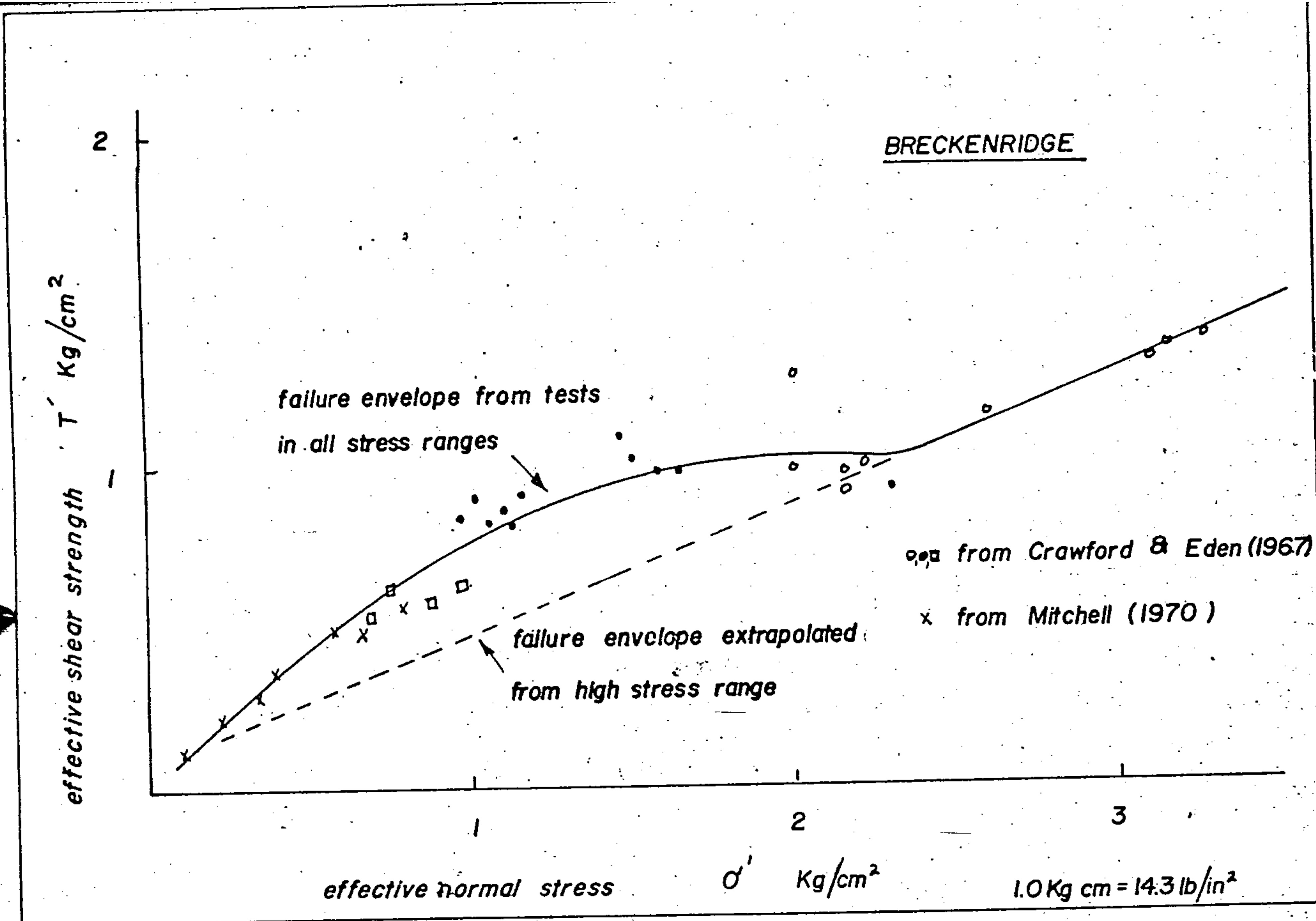

BRECKENRIDGE
2
1
effective shear strength τ' Kg/cm²
failure envelope from tests
in all stress ranges
failure envelope extrapolated
from high stress range
from Crawford & Eden (1967)
x from Mitchell (1970)
1
2
3
effective normal stress σ' Kg/cm²
1.0 Kg cm = 14.3 lb/in²

figure 3.3.3.

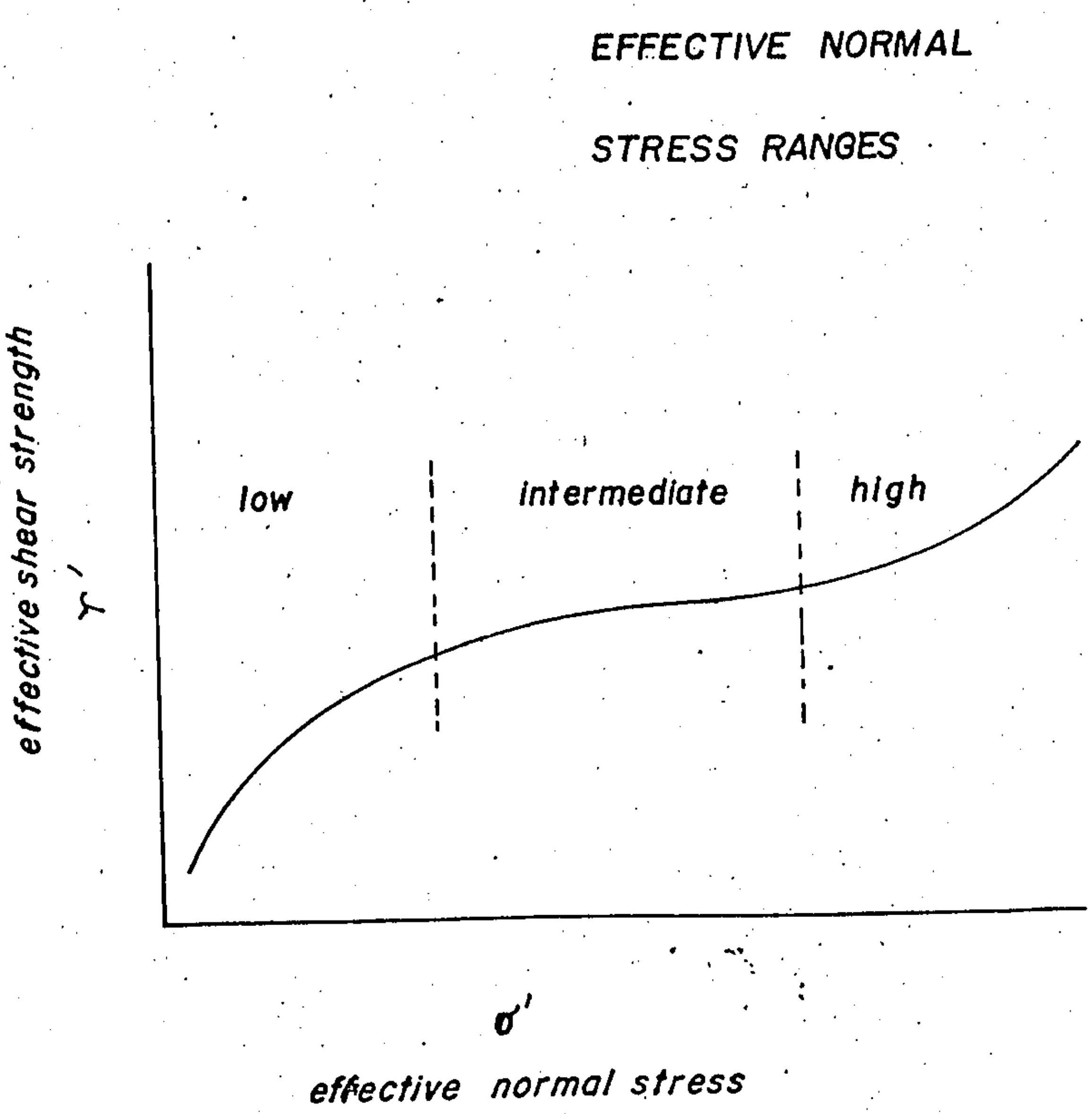

EFFECTIVE NORMAL
STRESS RANGES
low
intermediate
high
effective shear strength
τ'
σ'
effective normal stress

figure 3.3.4

samples (Note Figure 3.3.5). He also observed that although the mode of failure was influenced by anisotropy, there was no noticeable difference in strength for horizontally or vertically trimmed samples. There was no intercept of cohesion for Rockcliffe clay, which was considered to be fissured. Test results on the Rockcliffe clay at other normal stresses by Eden and Mitchell (1970) are also shown.

Conlon (1966) tested some Toulnustouc clay in an extension apparatus and found that this intact had a tensile strength of 2.4 lb/in^2 (Figure 3.3.6a). Lo and Morin (1972), using Brazilian tests, found for an intact St. Louis clay, a tensile strength along the stratification of 0.28 lb/in^2, and across the stratification 1.0 lb/in^2 (Figure 3.3.6b); and for an intact St. Vallier clay, a tensile strength across the stratification of 1.4 lb/in^2.

It appears that the failure envelopes for intact clays have a tensile strength, that is, a cohesion intercept. Fissured clays are essentially frictional, and have little or no cohesion intercept. Ladanyi (1970) has adapted shear strength envelopes for jointed rock masses to clays to explain the shear strength envelope for fissured clay (Figure 3.3.7).

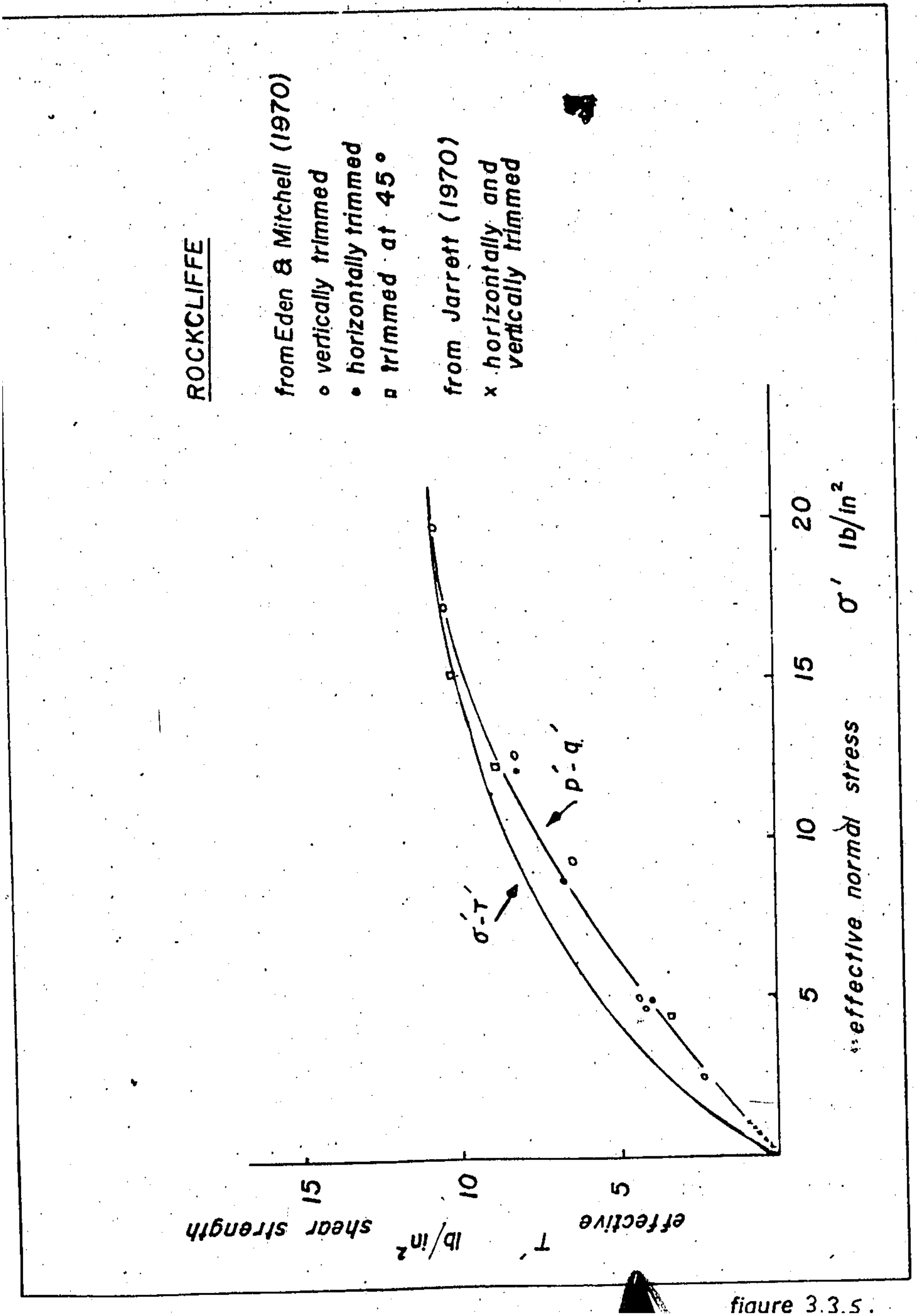
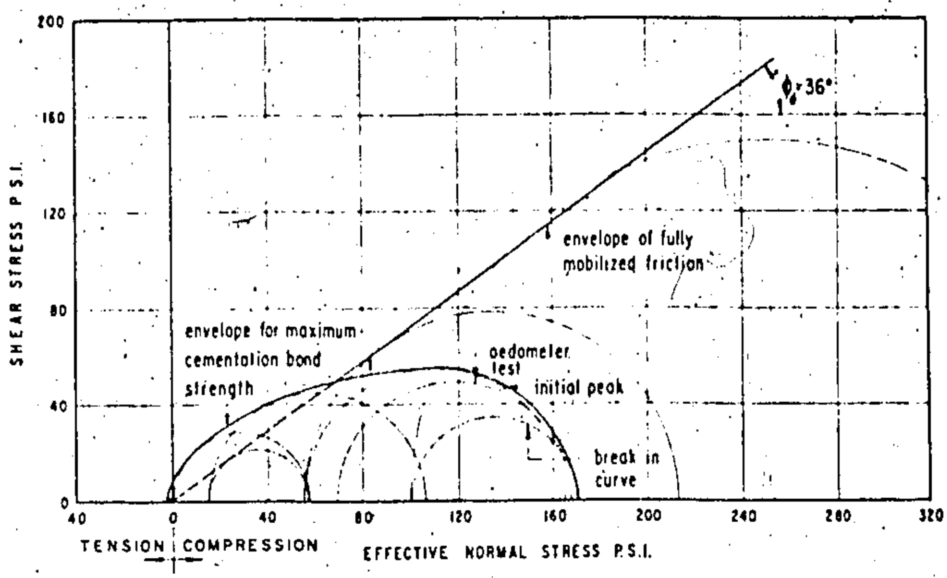

ROCKCLIFFE
fromEden & Mitchell (1970)
∘ vertically trimmed
• horizontally trimmed
▫ trimmed at 45°
from Jarrett (1970)
x horizontally and vertically trimmed
σ′-τ′
p′-q′
effective shear strength τ′ lb/in²
effective normal stress σ′ lb/in²
0
5
10
15
20

figure 3.3.5.

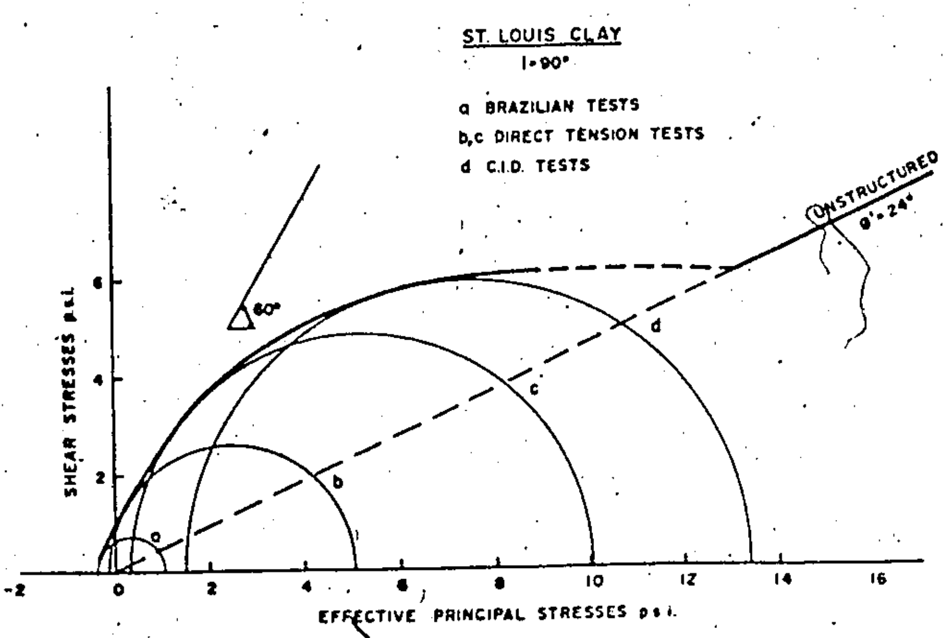
Intact Clays with Tensile Strength



Mohr-Coulomb plot for the consolidated drained triaxial tests

Toulnustouc clay, Conlon (1966)

(a)



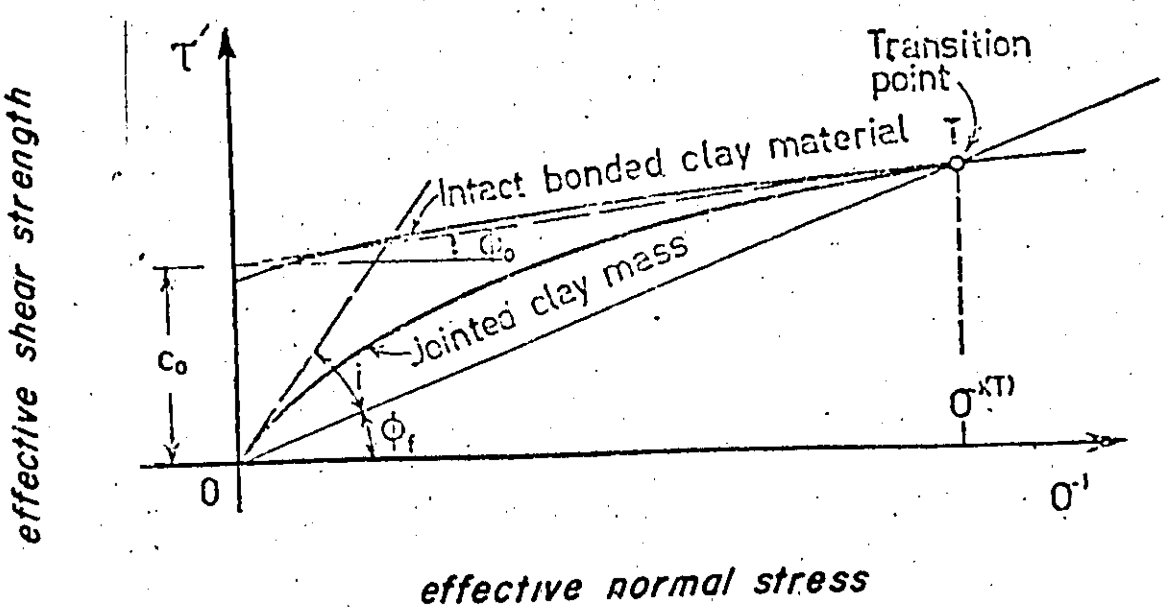
Drained failure envelope of St. Louis clay at $i = 90°$.

Saint Louis clay, Lo and Morin (1972)

(b)

figure 3.3.6.

Shear Strength of a Jointed Clay Mass



from Ladanyi (1970)

figure 3.3.7.

For engineering purposes, it is essential to know whether a clay is fissured or intact. A small cohesive strength in stability analysis can make a considerable difference in predicted factor of safety.

The development of the curved failure envelope having three stress ranges has been outlined; from the usual straight line envelope (Figure 3.2.1) to a curved envelope (Figure 3.3.4). This curved envelope can be reasonably represented by three straight lines (Figure 3.3.8) or by the use of a polynomial of the form:

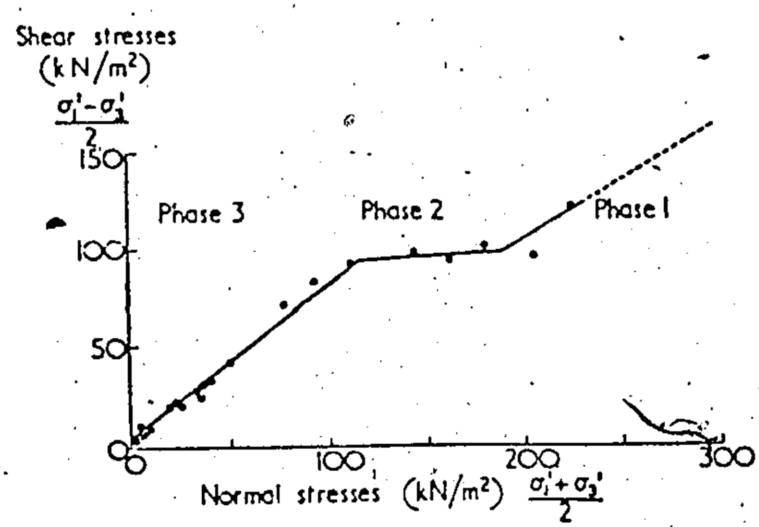
$$\sigma'_n = k_1(\tau')^2 + k_2(\tau') + k_3 \qquad (3.3.1)$$

where k_1, k_2 and k_3 are coefficients obtainable by curve fitting methods (Lo and Lee, 1973). This latter method has the advantage that three parameters, k_1, k_2 and k_3 can fully describe the shear strength envelope whose values of c' and ϕ' vary with effective normal stress. Thus it may be possible to describe mathematically an envelope in the low stress range if required.

3.4 p' - q' and the Mohr Envelope

Usually, in the literature, strength of

Fissured Clay



Failure envelope for samples of sensitive Leda clay from Rockcliffe airfield.

Rockcliffe clay, Jarrett (1972)

figure 3.3.8.

clays is reported in terms of p' and $\underline{q}'$, that is, $\frac{\sigma_1'+\sigma_3'}{2}$ and $\frac{\sigma_1'-\sigma_3'}{2}$. A curve through a series of p'-$\underline{q}'$ points would be a line through the tops of a number of Mohr circles, and not tangent to them as in the Mohr-Coulomb failure theory.

Transformation of the p'-$\underline{q}'$ envelope to the σ'-Γ' envelope may be performed graphically by the drawing of a curve tangent to the Mohr circles of several p'-$\underline{q}'$ points, or mathematically as outlined below.

Given in Figure 3.4.1 are two points $A(p',\underline{q}')$ and $B(\sigma_n',\tau')$. Point B is the transformation of point A from the p'-$\underline{q}'$ envelope to the σ_n'-τ' envelope. The p'-$\underline{q}'$ envelope is that which is obtained directly from experimental results as described above.

The transformation of the p'-$\underline{q}'$ envelope to the σ'-τ' envelope can be performed mathematically in the following manner:

A best fit curve is drawn through all of the plotted p'-$\underline{q}'$ points. The angle of a tangent to the p'-$\underline{q}'$ envelope from the horizontal, α_r' at point r, of the curve can be measured. Noting Figure 3.4.1;

$$\text{Sin } \phi_r' = \tan \alpha_r' \qquad (3.4.1)$$

Since
$$\text{Sin}^2 \phi_r' + \text{Cos}^2 \phi_r' = 1 \qquad (3.4.2)$$

..
$$\tan^2 \alpha_r' + \text{Cos}^2 \phi_r' = 1 \qquad (3.4.3)$$

and
$$\text{Cos } \phi_r' = + \sqrt{1 - \tan^2 \alpha_r} \qquad (3.4.4)$$

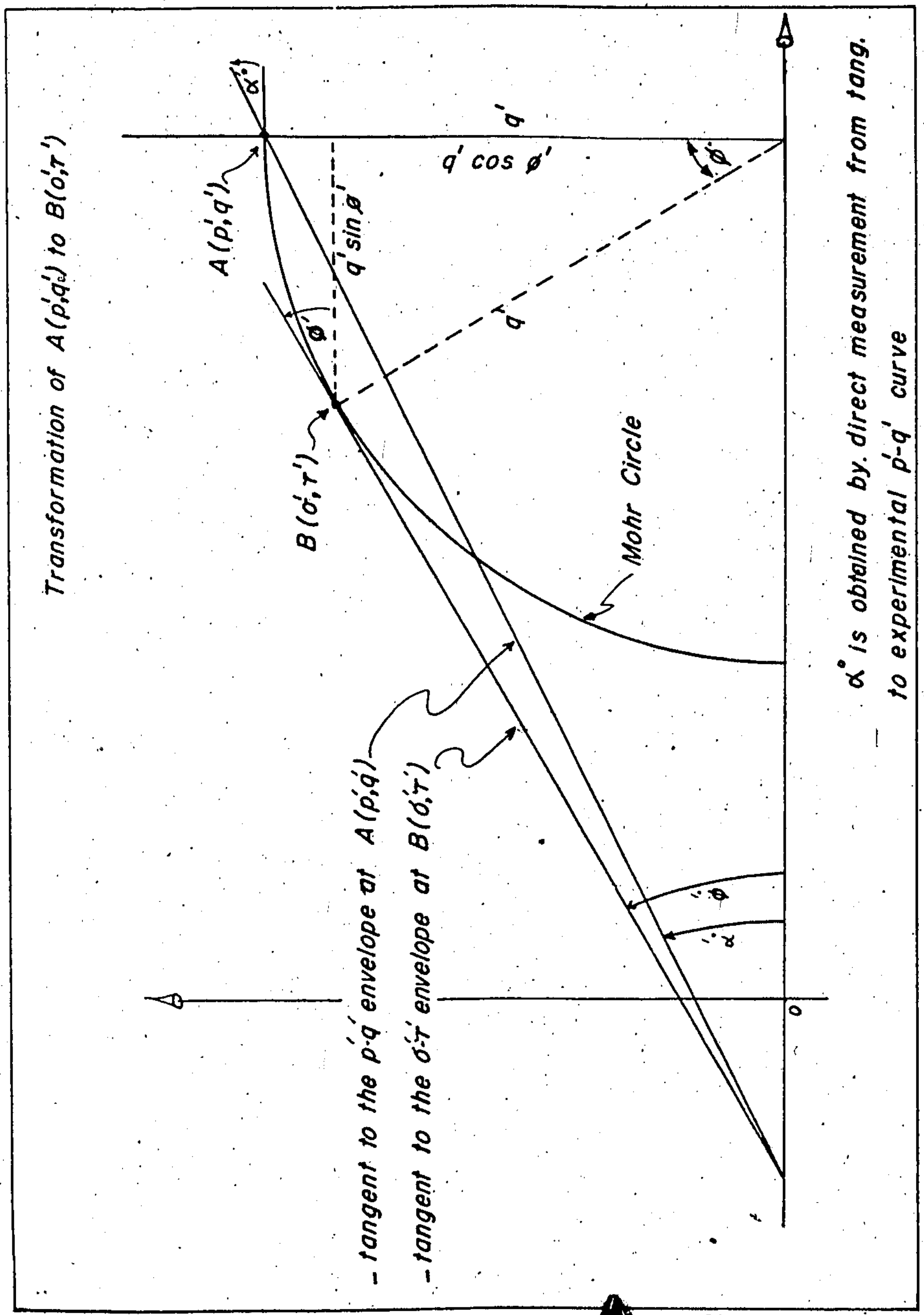

Transformation of A(p',q') to B(σ',τ')
A(p',q')
B(σ',τ')
q' cos φ'
q' sin φ'
q'
q'
φ'
φ'
α°
φ'
α'
Mohr Circle
0
– tangent to the p'-q' envelope at A(p',q')
– tangent to the σ'-τ' envelope at B(σ',τ')
α° is obtained by direct measurement from tang.
to experimental p'-q' curve

figure 3.4.1.

$$\tau'_r = q'_r \text{ Cos } \phi_r = q_r \sqrt{1 - \tan^2 \alpha_r} \qquad (3.4.5)$$

$$\sigma'_r = p'_r - q'_r \text{ Sin } \phi'_r = p'_r - q'_r \tan \alpha'_r \qquad 3.4.6)$$

Repetition of this operation for several points on the p'-q' curve results in a series of σ', τ' points and a σ'-τ' envelope from which values of c' and φ' can be readily obtained for any desired normal stress.

Comparison of the mathematically determined σ'-τ' curve to the graphically determined σ'-τ' curve shows that, for Castor River, the latter envelope is consistently 0.03 lb/in^2 below the former in terms of effective shear strength with a curved envelope. The equations may not be exactly correct.

Noting Equations 3.4.5 and 3.4.6 it is apparent that accuracy of the mathematical method also relies upon accuracte measurement of angle α, between the horizontal and the tangent to the p'-q' curve. Although the mathematical method appears to yield reasonable results, the graphical method is preferred, and this latter method was used.

3.5 Bonding and the Failure Envelope

Soil structure may be divided into the two components of fabric (particle geometry) and cementation bonding. Leda clay is thought to have a cardhouse type of structure (Quigley and Thompson, 1966 from Lambe, 1957) which undergoes particle orientation when sheared or consolidated.

There is evidence that strong cementation bonds exist at the points of contact between soil particles. These cementation bonds are thought to be a result of the deposition of Leda clay (Crawford, 1963). The level of stress under shear or consolidation at which these bonds begin to break down varies from site to site. It depends upon the nature of the bonds which has been determined by the environment of deposition of the clay and the geological history of the deposit following deposition.

Shear failure of a triaxial sample can occur in any one of three manners, depending on whether or not the effective stress at failure is in the low, intermediate or high stress region.

In the high stress ranges, due to the application of a consolidation pressure in excess of the preconsolidation pressure, samples tested undergo large volume reductions, as the bonded structure has been destroyed by these high consolidation pressures. The shearing stage exhibits further large volume reductions and large strains to failure as the bond strength has been destroyed and only frictional resistance is mobilized. Failure is usually by bulging of the sample with no distinct failure surface apparent.

Behaviour in the intermediate effective normal stress range (horizontal failure envelope) is controlled by the strength of cementation bonding. During both consolidation and shear very small volume reductions are observed. Strain at failure is moderate. The samples remain basically intact at failure, except for the formation of single or multiple failure planes. As the shear stresses became sufficient to destroy cementation bonds, the clay structure begins to consolidate, but cannot mobilize enough shear resistance to prevent failure.

In the low stress range under initial consolidation, very little volume reduction takes place. The sample may possibly swell. Upon shearing, a definite failure surface developes associated with a dilation or volumetric expansion and failure at a small strain. The sample at 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" (Jarrett, 1972).

During shear in the low stress range, the tendency of the clay between micro-fissures to dilate is resisted by the cementation bonds. In the very low stress ranges, dilatancy along micro-fissures during shear is pronounced, and shear resistance is essentially frictional. "As the effective stress level increases the micro-fissured material is held more firmly together with a concomitant strength increase. At the boundary between the low and intermediate stress ranges, an equality is reached between the strength along any planes of weakness and that of the cemented matrix" (Jarrett, 1972).

The observed physical and unobserved structural responses in each of the three stress regions has been outlined above. The stress range of most significance to slope stability analysis is the low effective normal stress range. Jarrett (1972) analyzed the Rockcliffe slide (Jarrett, 1971; Eden and Mitchell, 1970) using effective low stress parameters (Figure 3.3.6) and probable ground water conditions. He obtained shallow critical circles passing near the toe and crest of the slope with factors of safety close to unity. Analysis of the Rockcliffe slope using strength data from the intermediate stress region resulted in a large factor of safety indicating a stable slope with a critical, deep failure circle passing outside of the observed zone of failed material. This is also what Crawford and Eden (1967) found in their analysis of the Breckenridge slide.

Low stress range behaviour is the key to stability analysis of fissured Leda clays (Jarrett, 1972). Lo and Lee (1973), analyzing the stability of the Breckenridge landslide, determined that the average effective normal stress on a critical failure arc was just under 4 lb/in^2. "Most of the landslides that occur in the redeposited clays show a dilitant frictional type of shear behaviour under low normal stress levels. This shear mechanism results from inherent defects or planes of weakness, hence a curved failure envelope" (Eden and Mitchell, 1973).

3.6 Anisotropy.

Since all samples tested in this thesis have been tube samples, and hence tested vertically, the effect of anisotropy on the shear strengths obtained may be of importance in terms of slope stability analysis, since the failure arc varies from 0^o to 90^o from the horizontal.

Jarrett (1972) discusses his tests where he used the method of Bishop and Garga (1969) to test fissured Rockcliffe samples in the very low stress range (below unconfined compression) in tension. He trimmed his samples so that the minor principal stress was at 0^o and 90^o to the in situ horizon. He noted a difference in mode of shear failure but not in strength. His stress range was from 0 to 1-1/2 lb/in^2 effective normal stress. Anisotropy had no effect upon measured shear strength. Mitchell (1970), testing fissured Rockcliffe clay at i = 0; 45°, 90° from the horizontal, found shear strength to be essentially independent of anisotropy in the low stress region from 0.15 kg/cm^2 (2.0 lb/in^2) to 1.0 kg/cm^2 (14.3 lb/in^2) effective normal stress, but dependent on anisotropy in the intermediate stress region above 1.0 kg/cm^2 (14.3 lb/in^2).

It appears then that using vertical samples, and not testing for anisotropy is justified for slope stability analysis in fissured clays in the low stress range.

3.7 <u>Peak and Post-Peak Deviator Strength</u>

As discussed in Chapter 2, both intact and fissured Champlain Sea clay deposits exist. The fact that many authors refer to either or both as merely Leda Clay or Champlain Clay without considering the clay structure can be misleading.

Lefebvre and LaRochelle (1973) have found that for slope stability analyses of the sensitive Champlain Clays of the St. Laurent Lowlands, peak strength analysis provided "a gross overestimate of the factors of safety." However, using residual strength parameters for St. Louis and St. Vallier clays, these provided "a fairly accurate assessment of stability," with "calculated factors of safety close to unity."

Lo and Lee (1973) have taken Lefebvre and LaRochelle's post-peak or residual failure envelope for intact clays, and applied it to the analysis of fissured clays, such as Breckenridge, Greens Creek, Rockcliffe and South Nation. Although Lo and Lee obtain factors of safety close to 1.0, it would appear to be merely fortuitous, and not a valid method of analysis for fissured clays.

Mitchell and Lawrence (1973) "question the applicability of this" (residual) "analysis .... to the problem of long-term slope stability in" (fissured) "Ottawa area clays". Skempton (1964), when he presented his residual concepts of shear strength and slope stability analysis, noted as an exception, the Leda Clays of Eastern Canada. In view of the Leda clay literature published to 1964, Skempton was most probably referring to the fissured clays of Eastern Ontario. Mitchell (1970) carried out several strain controlled tests on Rockcliffe fissured clay and found that there was only a slight reduction (10%) in shearing strength at large strains (6%). That is, a residual strength very close to peak strength.

Using peak shear strength data from ten locations of landslides (in fissured clays) Eden and Mitchell (1973) found slope stability analyses yielded factors of safety of 1.00 ± .05 and that "these data suggest that the long term factor of safety of a slope in the Ottawa area" (fissured clays) "can be calculated with some confidence" using peak strength in the low normal stress range.

Scott (1973), analyzing the slope stability of a fissured Leda Clay at Castor River using peak strength, found a factor of safety of about 1. Small landslides have recently occurred close to the slope analyzed.

In the Ottawa-Hull area a few outcrops of intact clay occur, which undergo a large drop in shear strength from peak to post-peak values (Eden and Mitchell, 1973). This is similar to the test response of the St. Vallier and St. Louis intact clays.

Summarizing for this section, on the basis of current limited information, it appears that

1. Peak and post-peak strength envelopes may be quite different for intact clays.
2. Peak and post-peak strength envelopes may be essentially similar for fissured clays.
3. The question of how to obtain effective shear strength parameters for slope stability analysis in Champlain Sea Clays is complicated. Considerable further work is required to resolve this problem.

CHAPTER 4

FACTORS INFLUENCING THE INTERPRETATION OF TRIAXIAL TEST RESULTS

4.1 General

Discussed in this Chapter is the influence of such factors as piston friction, rubber membranes, paper drains, end platens, temperature and sample storage on the interpretation of deviator strength test results from samples.

4.2 Piston Friction

In a stress controlled test, such as used in this work, loads (weights) are applied to the sample by means of a hanger bearing upon a piston. This piston extends through a bushing into the cell to the top for the sample. Any friction between the cell bushing and the piston will result in a load loss to the sample.

When low cell pressures are used in triaxial testing, soil strengths are small, and any error due to piston friction can be important.

Piston friction can be essentially eliminated, however, if the triaxial cell is equipped with a rotating bushing. As the cells used were so equipped, no correction for piston friction have been applied to experimentally obtained triaxial shear strengths.

4.3 Rubber Membranes and Paper Drains

Failure to account for the effect of rubber membranes and paper drains upon the measured shear strength from a triaxial test may result in an over-estimate of shear strength. The following relationship should be considered:

measured =	"true" +	correction for
deviator	deviator	boundary conditions
strength	strength	of triaxial test.

Duncan and Seeds' (1967) and Henkel's (1962) corrections for filter paper drains and rubber membranes are for tests conducted in the high stress ranges and amount to about 2 lb/in^2.

Gill (1968) undertook determination of the effects of membrane and filter paper drain corrections in the low effective normal stress regions using consolidated undrained (CIU) tests with σ_3' constant and σ_1' increasing. Although the constant "p" tests (to be discussed later) are consolidated drained (CID) and σ_3' is not constant, but slowly and incrementally decreasing, the volume changes are quite small. It is felt that Gill's corrections are applicable to the constant P_m' test.

His correction factors (C.F.) to be subtracted from the deviator stress due to the effects of membrane, filter paper, and piston friction are:

1) $\text{C.F.} = (0.70 + 0.167\,\sigma_3') \times \%\varepsilon/10\%\ \text{lb/in}^2 \quad 0<\sigma_3'<10\ \text{lb/in}^2,\ 0<\varepsilon<10\%$ (4.3.1

2) $\text{C.F.} = (0.70 + 0.167\,\sigma_3')\ \text{lb/in}^2 \quad 0<\sigma_3'<10\ \text{lb/in}^2,\ \varepsilon>10\%$ (4.3.2

3) $\text{C.F.} = (2.20 + 0.017\,\sigma_3')\ \text{lb/in}^2 \quad \sigma_3'>10\ \text{lb/in}^2$ (4.3.3

Therefore the corrections required in this thesis have been examined as follows. A typical clay sample tested may have, at failure, a strain of 3% and an effective cell pressure, σ_3', of 1.0 lb/in^2.

$\therefore$ by equation 4.3.1) of Gill (1968)

$$\text{C.F.} = (0.70 + 0.167) \times 3\%/10\% = 0.26\ \text{lb/in}^2$$

Then $\underline{q}' = \dfrac{\sigma_1 - \sigma_3'}{2}$ would be reduced by $\dfrac{0.26}{2}$ or 0.13 lb/in^2 or .01 kg/cm^2, which is negligible.

These findings are confirmed by Lefebvre and LaRochelle (1973): "in view of the small strains at failure and the low cell pressures used during the (St. Vallier, St. Louis) tests, no correction factor was applied for the filter paper side drains or the rubber membranes." Lo and Morin (1972) make the same observations, and also report that they used a rubber membrane 0.012" thick (compared to membranes 0.010"-0,015" used in tri-axial tests by this writer).

Therefore, no correction was applied for effects due to membranes and drains in this testing program, as volume change, strain, and effective confining pressure were small in the low effective normal stress range.

4.4 Membrane Leakage

In drained tests of long duration, leakage passed or through the membrane into the sample can be of importance in assessing volume change. Rowe and Barden (1964) found that the bulk of the leakage (using "O"-rings and standard membranes) was not through the membrane, but passed the "O"-rings at a rate of up to 0.05 cc/hour under an effective cell pressure of 10 lb/in^2. In this work, silicone grease was carefully applied between the caps and membrane and twice the usual number of "O"-rings was used.

Therefore, it was felt that under the low cell pressures employed, that leakage was not an important factor.

4.5 Frictionless End-Platens

Rowe and Barden (1964) propose the use of frictionless end platens for undrained triaxial tests to reduce the effect of non-uniformity of stress and deformation. These non-uniformities result in a redistribution of moisture content (Crawford, 1961). In drained tests, end friction can reduce the volume changes near the ends of the samples and by preventing shear strains lead to an over-estimation of the strength of the sample.

However, in triaxial tests such as performed here, where the sample height to diameter ratio was 2:1, measured axial deformation and volume changes in the low effective normal stresses were found to be small. Hence friction or shearing stresses between the end platens and the soil sample were considered to be negligible.

As this investigation was mainly interested in low effective normal stress range behaviour, frictionless end platens were not employed. It is apparent, from data presented in Chapter 7, that samples tested triaxially in shear in the intermediate to high stress ranges did undergo some degree of bulging during shear, when strain and volume change at failure were relatively large. In these cases, frictionless end platens would have been of value.

4.6 Temperature

All testing of samples for the three clays has been carried out at room temperature ($68^{\circ}F$ to $72^{\circ}F$).

Campanella and Mitchell (1968), testing with illite have found that "an increase in temperature causes a decrease in the shearing strength of individual particle contacts. This decrease in the interparticle bond strength may be considered to result from the increase in thermal energy which acts in conjunction with the shear force at the interparticle contacts to increase the probability of bond slippage or failure." As a result there is a partial collapse of soil structure and a decrease in void ratio. This is a temperature induced secondary consolidation. "From a practical standpoint, pore pressure changes due to temperature variations may be significant. When saturated soil is removed from the field and taken to the laboratory, the temperature of the soil will usually increase since the field temperature of the soil is less than the normal laboratory temperature. The temperature increase will cause an increase in pore pressure and a decrease in effective stress."

It would appear that storage and testing of a clay at 68°F may not yield the same effective shear strength envelope as samples stored and tested at in situ ground temperatures. It is possible that sub-crustal ground temperature in the clays of Eastern Ontario range from 40° to 50°F.

Although the effect of temperature on the shear strength results is not known it may have had some influence. It is recommended that testing and storage be at or near ground temperature in the future.

4.7 Sample Storage

Bjerrum, (1971) in the Norwegian Geotechnical Institute Report No. 85 has proposed that the strength of the upper crust of Norwegian clays is due to the bonding effect caused by precipitated iron and possibly also aluminum. Berre, Schjetre and Lolle, in the same Report observed a brownish layer of clay adhering to the wall of a shelby tube after extrusion of the sample. It appears that iron ions from the shelby affected the sample.

All samples tested were wrapped in

aluminum foil and waxed for storage. Some samples from Bear Brook were poorly waxed. As a result the aluminum adjacent to the clay had disintegrated. The clay had assumed a brownish colour as opposed to its original gray-brown. Such samples were rejected and not tested.

It is proposed that the method of preparation of samples for storage exclude the use of aluminum foil adjacent the sample in the future, until more is known about the effect of such ions as aluminum and iron on bond strength.

Samples should be stored in a climate controlled room at a temperature close to the in situ ground temperature, probably 40° to 50°F. Relative humidity should be maintained close to 100%. As it is not possible to duplicate in situ conditions for the sample on the shelf, time before use should be kept to a minimum. Bozozuk (1971) has found that measured preconsolidation decreases with time for shelf samples.

4.8 Sample Disturbance

Tube rather than block samples have been used in the investigation of shear strength of the three Russell County clays.

Justification from the literature for the acceptability of tube samples will be presented below, followed by an examination of test data for signs of sample disturbance.

Sample disturbance is of course unavoidable when testing soil specimens from the field in the laboratory. It is related to gas expansion on stress release, sample taking and trimming operations, and of course handling before testing (Quigley and Thompson, 1966). It is then necessary to determine the relative degree of disturbance of the samples employed.

This writer used shelby tube samples obtained by standard field sampling methods. Recent literature (Quigley and Thompson, 1966; DeLory and Salvas, 1969; Bozozuk, 1971; Raymond et al., 1971; LaRochelle and Lefebvre, 1971) suggested that tube samples are inferior to block samples and may lead to erroneous test results in the laboratory.

Berre, Schijtre, and Sollie (N.G.I. Technical Report No. 85) note that "when a clay is stored several months in steel sampling tubes, the force required to extrude the sample is several times higher than if the sample were extruded shortly after sampling." However, it was found that the extrusion of fissured Leda clays from shelby tubes immediately after sampling results in minimal disturbance to the sample due to the presence of a very thin remoulded layer of this sensitive clay between the shelby tube and the intact sample.

Mitchel and Lawrence (1973), have found that for the frictional Leda clays of Eastern Ontario, "good tube samples give the same strength as block samples in the low stress range as the dilative-frictional strength attributed to those imperfections in the material is not sensitive to disturbance."

All of the samples tested have been from 2-inch diameter shelby tubes. Mitchell (1970) found little effect due to sample size (1.5, 2.5, 4.0 inch diameter specimens) on measured shear strength for fissured clay of Eastern Ontario.

For these reasons, it is felt that the samples used in this testing program in the low stress range are relatively undisturbed and have given valid peak deviator strengths when tested in the laboratory. Test results appear to confirm this assumption because of the uniformity and pattern of the test results.

Plotted in Fig. 4.8.1 are p'-q' points for South Nation and also for the South Nation of Eden and Mitchell (1973).

Twenty-one points are shown, of which eight are from Eden and Mitchell (1973) and the remainder from this testing programme.

The first five "dot" points from the left are from Eden and Mitchell (Fig. 4.8.1). They are the results of stress controlled constant P'_m testing on a block sample from a depth of 13 meters (43 feet) below the plain elevation. This block sample was obtained from a back scarp of the South Nation landslide. The last three "dot" points are tube samples from a depth of 16 meters (53 feet) and are strain controlled (constant cell pressure, increasing deviator strength) tests (Mitchell, 1973).

All samples shown as "triangles" in Fig. 4.8.1 are constant P'_m stress controlled results from shelby tubes. As Eden and Mitchell's clays are a few hundred yards downstream from the South Nation clays of this series, the clays would likely be reasonable similar.

South Nation - two deviator strength envelopes

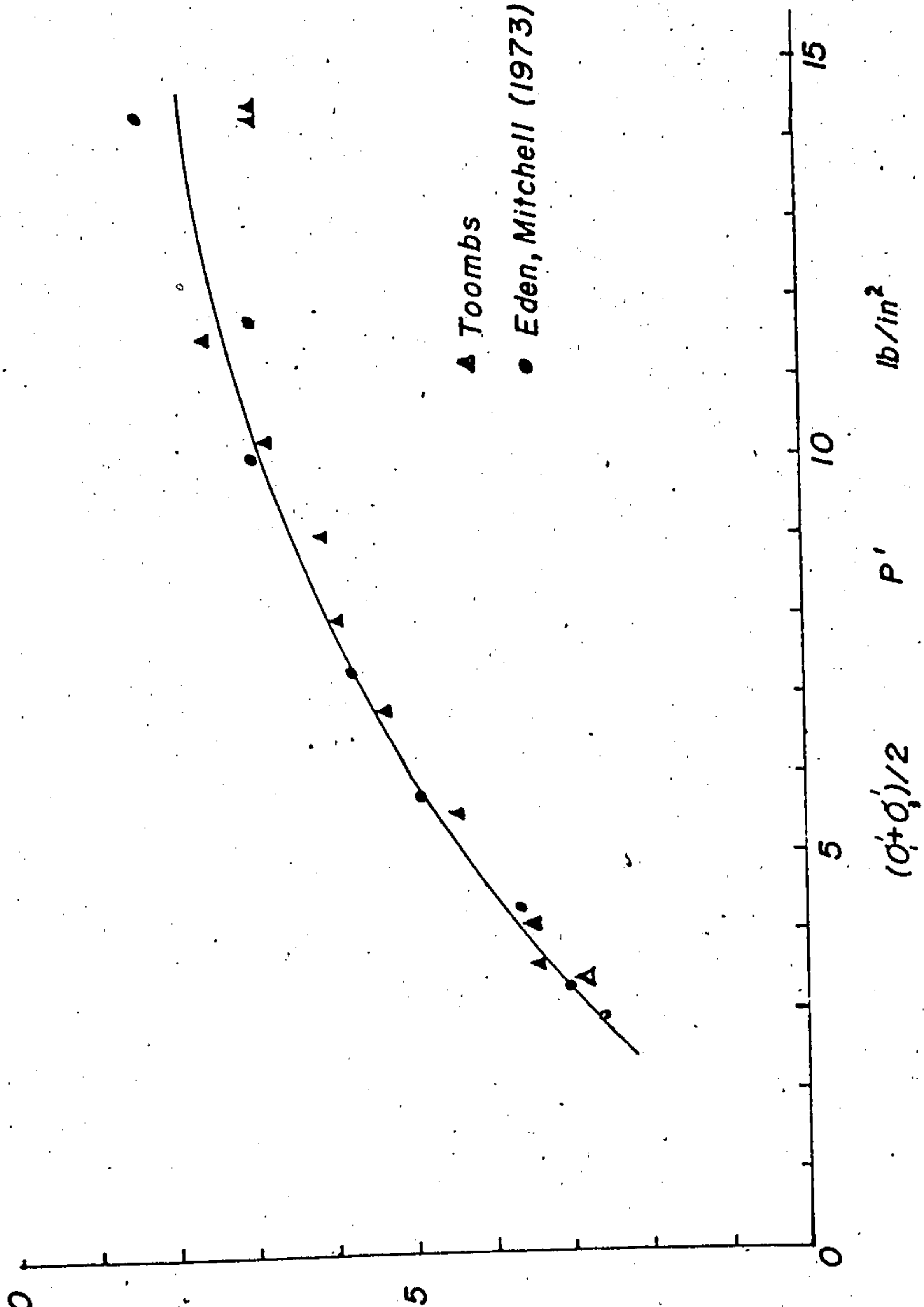


figure 4.8.1

Evidence as shown in Fig. 4.8.1 would indicate that good shelby tube samples are equivalent to block samples.

The consolidation test curves of void ratio vs. log pressure (Fig. 4.8.2) for Bear Brook, South Nation, and Castor River show well defined breaks in the curves, allowing Pc to be determined with reasonable confidence, and indicating that sample disturbance is not too severe.

The loads or pressures employed in the oedometer test were successively increased by an amount equal to 10 to 50% of the previous load. Increment duration was determined in the standard fashion from the deformation per increment-log time plot. In this manner, estimated preconsolidation pressures for the three clays tested triaxially were obtained.

Comparison of these laboratory preconsolidation pressures to corresponding field vane strengths resulted in a vane to preconsolidation ratio ranging from 0.26 to 0.34. Examination of these values indicates that disturbance is probably not too severe. One exception may be the second Castor River preconsolidation test (2700 lb/in^2). It is discounted as it was made on a sample under an eighteen foot fill. The surcharge associated with this fill was very close to the measured preconsolidation pressure of the first sample, which may have affected bond strength. An excellent indicator of sample disturbance is a measure of $\%\Delta V/V$ during sample

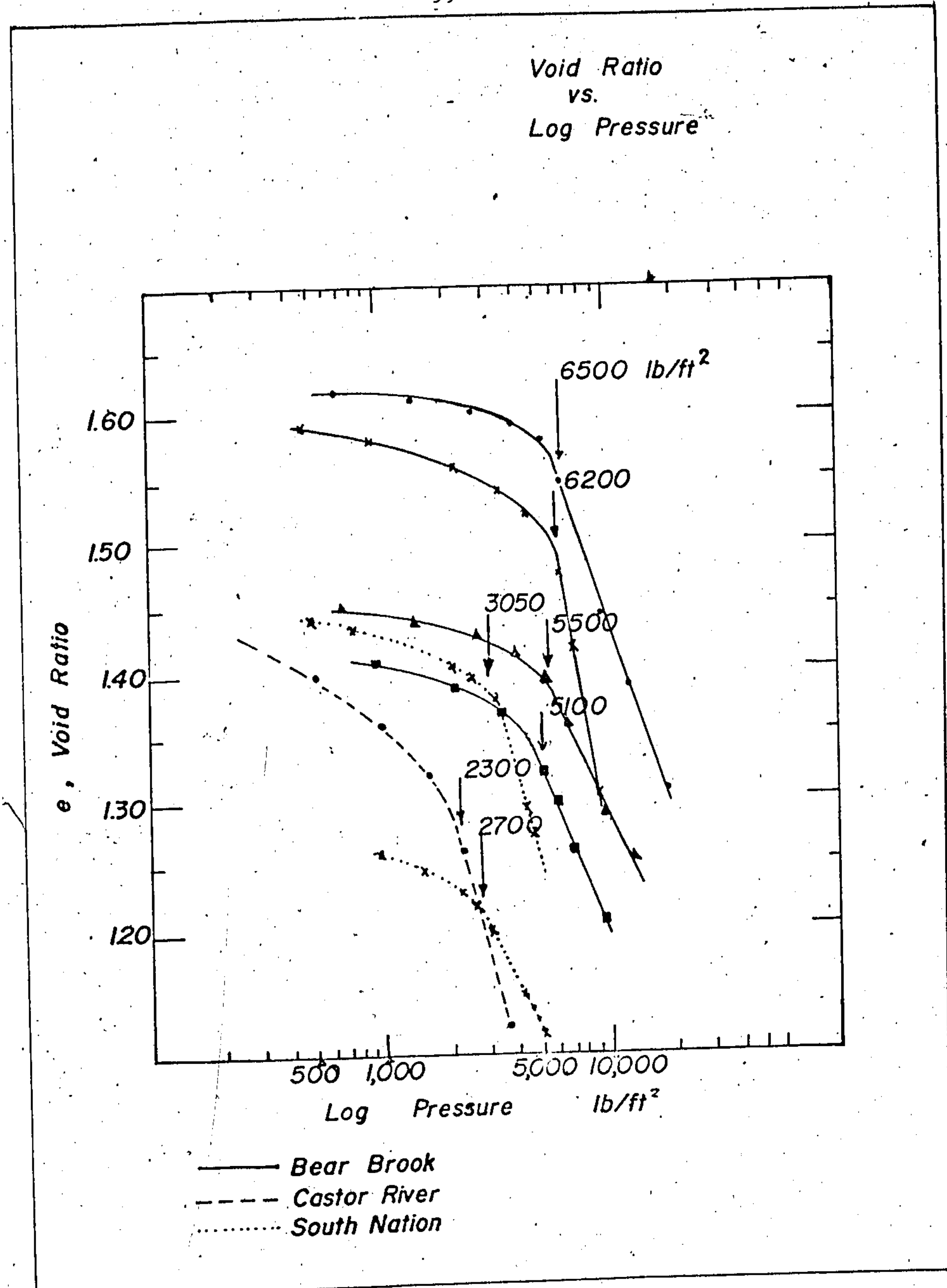

Void Ratio
vs.
Log Pressure
6500 lb/ft²
6200
3050
5500
5100
2300
2700
1.60
1.50
1.40
1.30
1.20
e, Void Ratio
500
1,000
5,000
10,000
Log Pressure lb/ft²
Bear Brook
Castor River
South Nation

figure 4.8.2

consolidation at all-round pressures in the low stress ranges. Should the bonds be intact, $\%\Delta V/V$ should be small. For South Nation, consolidation of samples to an all-round consolidation pressure 8 lb/in^2 resulted in an average $\%\Delta V/V$ of 1% compression. Although the mode of testing for some samples at Castor River and Bear Brook (consolidation to pressures above P'_m) resulted in larger volumes of pore fluid expelled, examination of these tests consolidated to P'_m only, resulted in a $\%\Delta V/V$ compression of about 2% for Bear Brook and 1% for Castor River.

Plots of the log of all-round consolidation pressure vs. volume change or volumetric strain for samples of widely varying void ratios (Figs. 7.3.6, 7.3.7, 7.3.8) indicate a pattern. It would appear therefore that bond strength has not been markedly affected by disturbance. If sample disturbance is a factor, considerable more scatter would be expected in the test results.

4.9 Sample Selection

Although all samples used in this series are from shelby tubes and therefore from different depths, some effort was made to select samples on the basis of similarity of water content for Castor River and Bear Brook soils. Samples from the South Nation River were chosen on a random basis, with only a visual estimate of suitability.

If it is assumed that all samples are saturated and that the soil solids have the same specific gravity, then saturated density would be a measure of water content. Careful monitoring of water flow into the sample during establishment of back pressure over a sufficient period of time indicated that the samples tested triaxially were saturated. This is confirmed by comparing initial pre-test sample weight to post test sample weight taking into account measured volume changes during consolidation and shear.

Samples were chosen before testing by weighing them and measuring their volumes to calculate their saturated unit weight. The measured saturated unit weights and the actual moisture contents obtained after testing are shown in Fig. 4.9.1. Also shown are the theoretical specific gravity curves for saturated samples. The scatter about the specific gravity line for samples from one site may be a reflection of not only a variation in specific gravity but also a variation in the degree of saturation. The moisture contents were obtained by weighing the entire sample to obtain the wet and dry weights and is therefore a measure of the average water content throughout the sample.

Although the water contents or void ratios of the samples tested for all three sites varied markedly (as tabulated in Section 7.2), their variation did not appear to affect the shear strength test results.

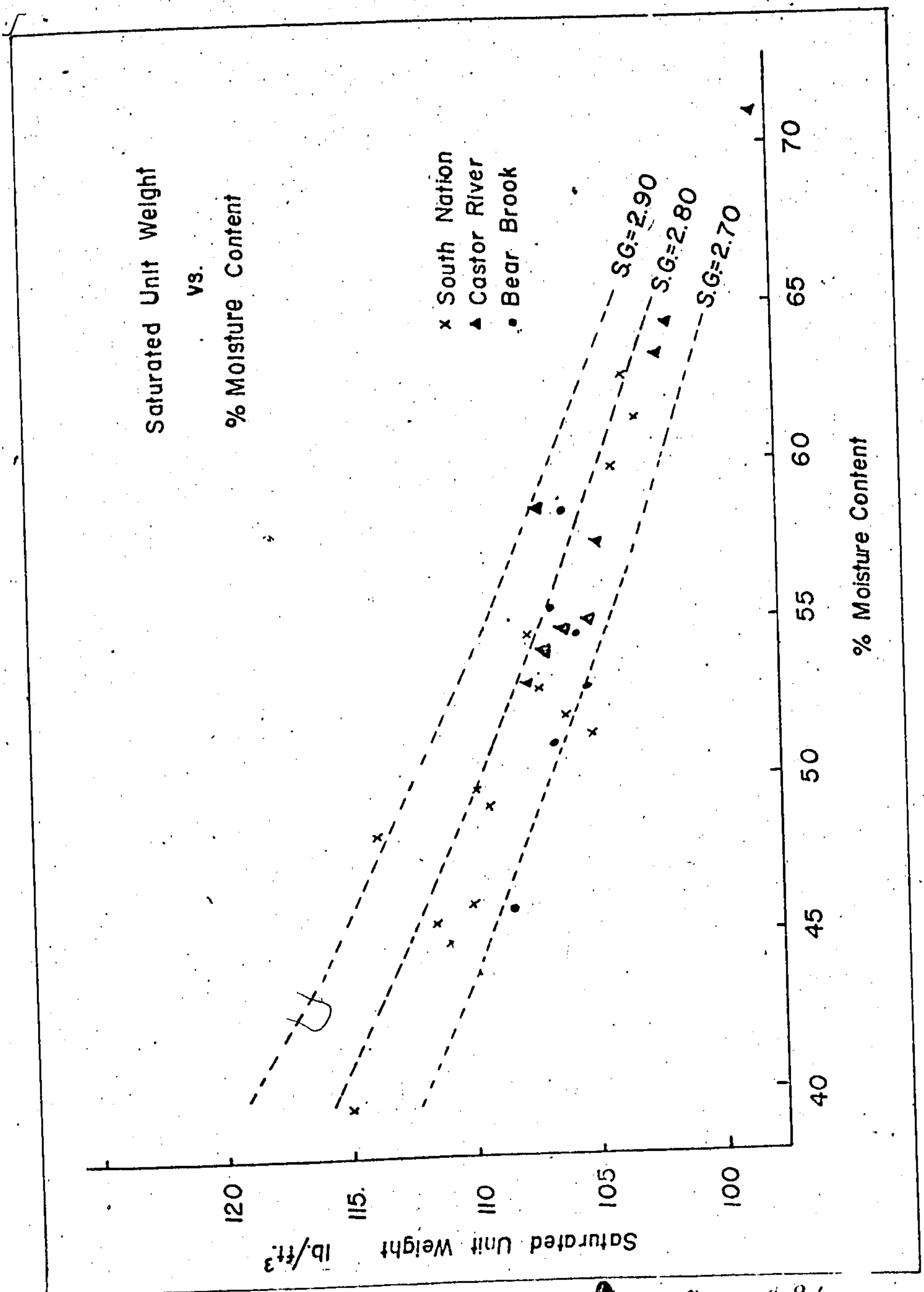

Saturated Unit Weight
vs.
% Moisture Content
x South Nation
▲ Castor River
• Bear Brook
S.G.=2.90
S.G.=2.80
S.G.=2.70
40
45
50
55
60
65
70
% Moisture Content
100
105
110
115
120
Saturated Unit Weight lb./ft.³

figure 4.9.1

CHAPTER 5

TESTING PROGRAMME - EQUIPMENT AND PROCEDURE

5.1 General

Included in this Chapter is a description of the equipment used, its schematic and physical set-up; use of a suitable back pressure; choice of a triaxial stress path; and test procedure. The method of sample selection for triaxial testing is also discussed.

5.2 Description of Equipment

The pressure supply system is that which is well described by Bishop and Henkel (1962). It is a self compensating mercury pot system. The equipment is listed below.

TABLE 5.2.1 EQUIPMENT

Equipment	Manufacturer	Model No.
Rotating Bushing Cell, 1-1/2", 2" Diameter Samples	Wykenham - Farrance	No. 1150
Flow Measuring Device	Wykenham - Farrance	No. P-3
Pressure Measuring Control Panel	Engineering Laboratory Equipment Limited	-

Accurate measurement of pressure is of extreme importance in low stress range testing. Since the pressure gauges are approximately ±1 lb/in^2 in accuracy 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 to the nearest 1/4 cm or .05 lb/in^2 (Note Fig. 5.2.1). The flow meter, also shown in Fig. 5.2.1, was placed in series between the manometer and the back pressure into the triaxial cell. In this way any volume change in the flow meter was representative of the flow in or out of the sample, and not of flow in the lines due to a fluctuating mercury manometer. Note Figs. 5.2.2, 5.2.3 and 5.2.4, photographs of the experimental set-up.

5.3 Use of a Back Pressure

There are several important reasons for employing a back pressure. These are:

1. To assure full saturation of the sample.
2. To assure full saturation of filter stones and paper drains for accurate volume change measurement and pressure application.

Experimental Setup - Schematic

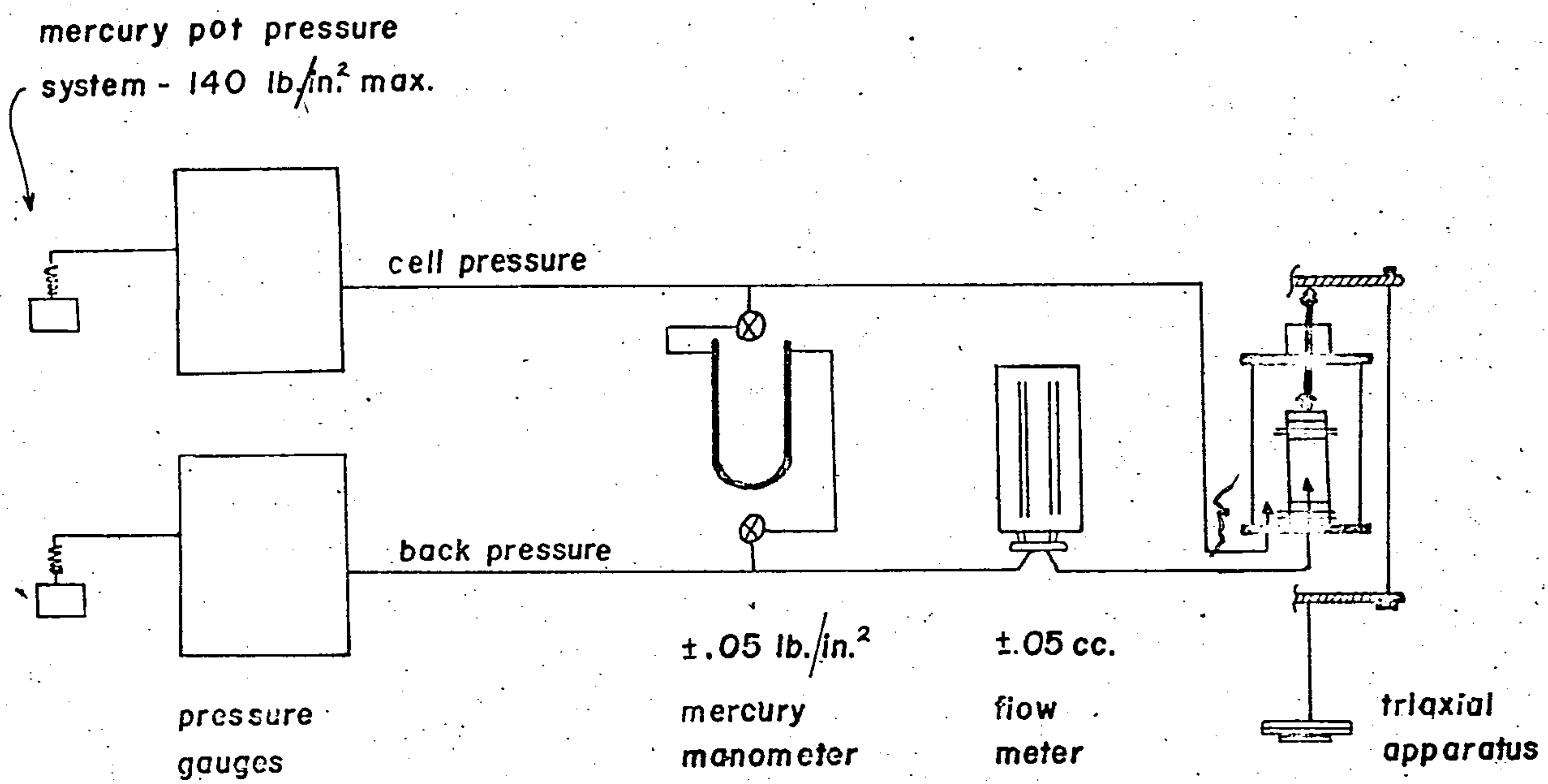
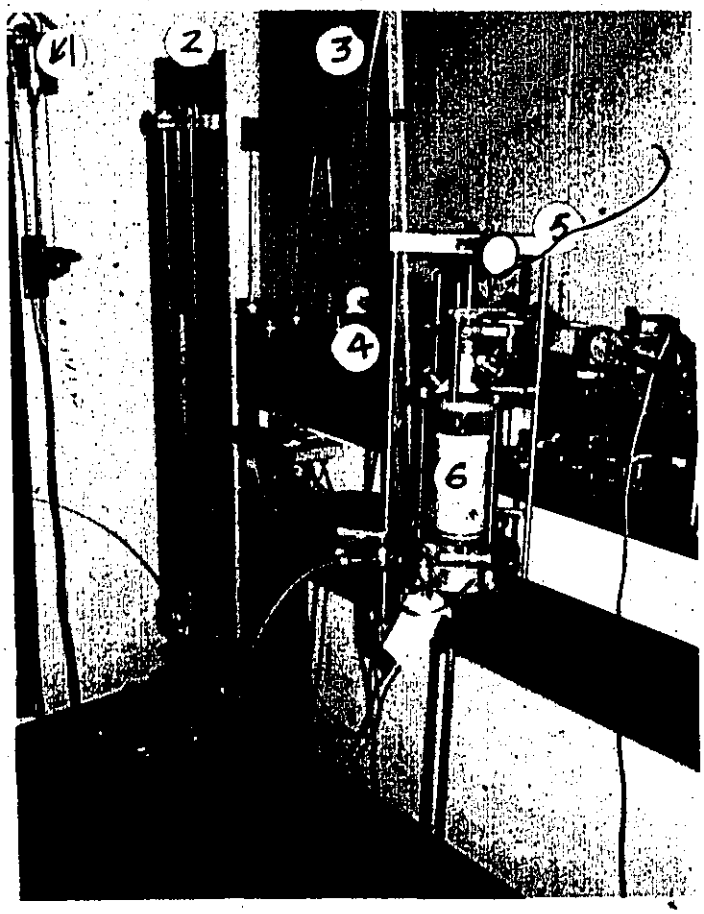
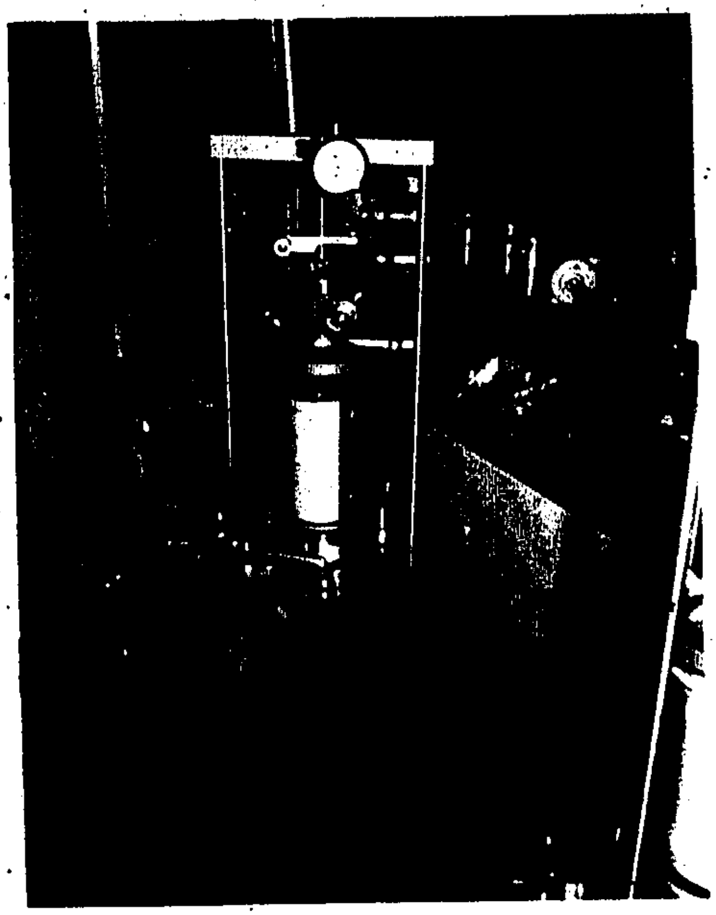


figure 5.21

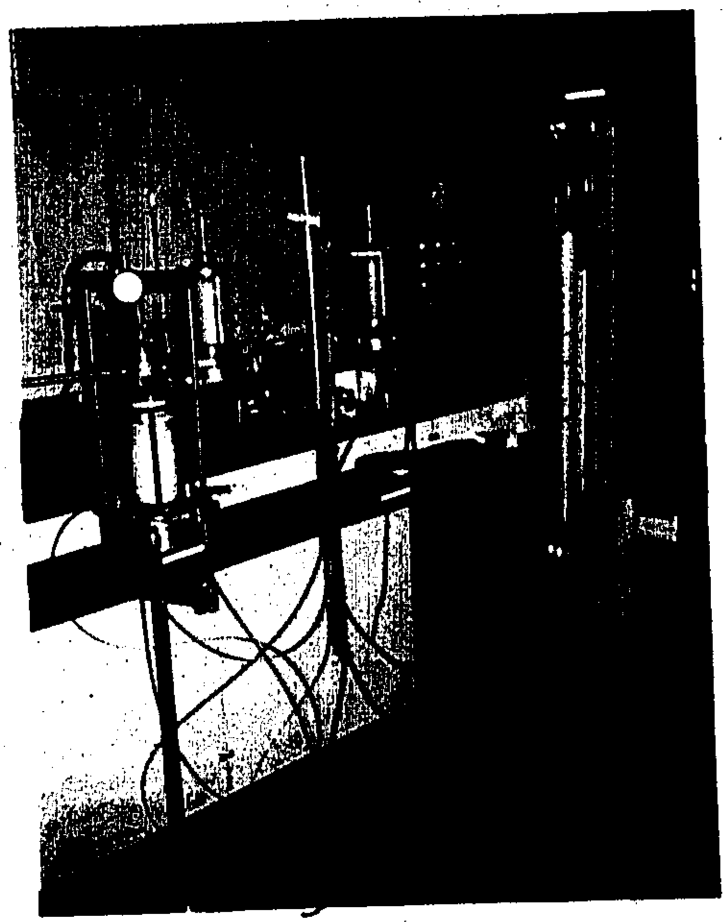
EXPERIMENTAL SETUP - PHYSICAL



1. Self compensating mercury pot system.
2. Flow meter.
3. Mercury manometer.
4. Bushing motor (behind manometer)
5. Dead load hangar
6. Cell containing sample under consolidation.



STATION ONE



STATION TWO

figure 5.24

3. To assure solution of air bubbles in pressure lines so that there are fewer chances of pressure loss across air blockages.

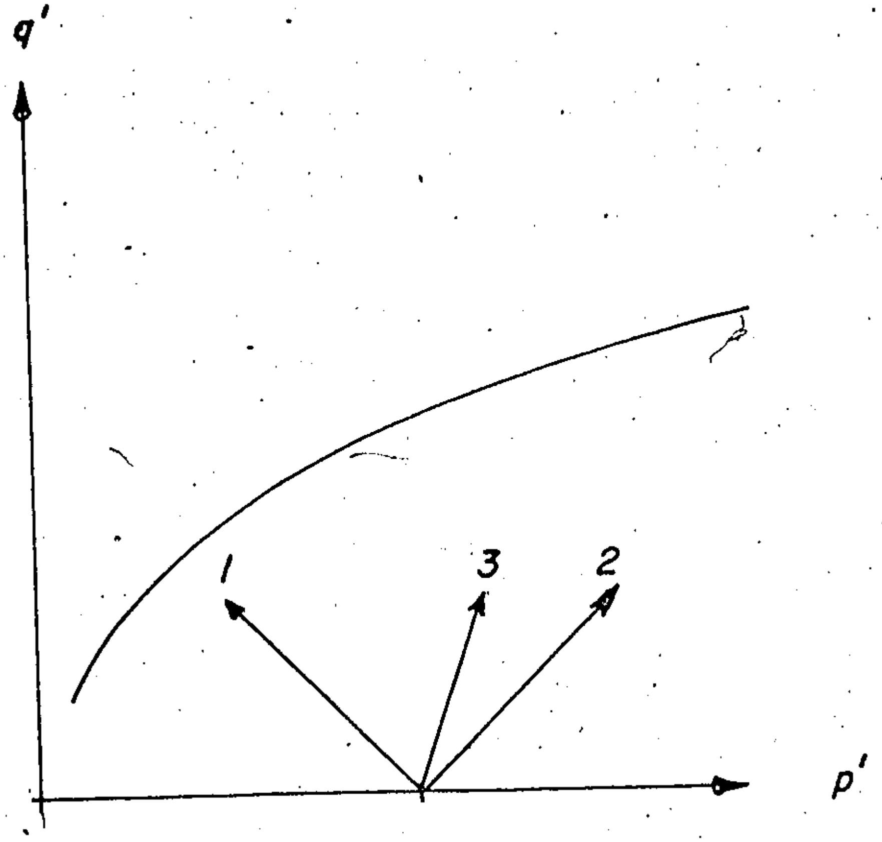
Lee and Black (1972) present tables for times of solution of bubbles of measurable dimensions in small tubes, large tubes and cells, under pressure. Black and Lee (1973) discuss time to full saturation (99% acceptable for all but stiffest soils) for various soils under given pressures. From the above references it was concluded that a back pressure of 50 psi was more than adequate to ensure full and rapid solution of air bubbles. A typical back pressure from the literature is about 1.0 kg/cm^2 (14.3 lb/in^2)(Lefebvre and LaRochelle, 1973).

5.4 Triaxial Stress Path

For triaxial testing purposes, any one of a number of triaxial stress paths are available (Note Fig. 5.4.1). Mitchell (1970) has reported that "the failure envelope appears to be independent of the stress path (for triaxial boundary conditions), providing that

Stress Paths

for Drained Triaxial Tests



1. $\sigma_1' = k$, σ_3' decreasing

2. $\sigma_3' = k$, σ_1' increasing

3. $(\sigma_1' + 2\sigma_3')/3 = k$

figure 5.4.1

stress path lies wholly within the yield curve." By Mitchell's definition of the yield curve this would include any stress path shown in Fig. 5.4.1 in the low to low-intermediate effective normal stress regions.

The stress path chosen was one of constant mean normal stress ($P'_m = (\sigma'_1 + 2\sigma'_3)/3$) during the shearing stage of the test. The advantages are:

1. The principal stresses are low and are representative of stress levels in the field.
2. Since mean normal stress is constant, any measured volume change is due to change in deviator stress and volume changes due to changes in the mean normal stress are eliminated.

Mitchell (1970) has reported that, for a fissured clay, he achieved the same shear strength data using stress controlled and strain controlled tests.

Constant monitoring of cell pressure would be required to run a P'_m = constant strain controlled test. Hence a stress controlled or load increment test is preferred and has been employed in this series of experiments.

There are certain limitations involved in triaxial testing, irregardless of the stress path employed.

If a sample is sheared under conditions of σ_1' increasing, and σ_3' equal to zero (stress path 2 starting from the origin, Fig. 5.4.1) then the sample stress path will follow what can be defined as the unconfined compression line of Fig. 5.4.2 until failure. Repeating the stress path 2 conditions for other samples, but increasing σ_3' each time will result in a series of p'-q' failure points to the right of the unconfined compression line, that is a deviator strength envelope.

This same envelope can be equally well reproduced in the low normal stress ranges by stress paths 1 and 3 (Fig. 5.4.1).

If stress paths 1 and 3 are employed in the very low normal stress ranges, it is possible they may intercept the unconfined compression line instead of the failure envelope. When this happens, σ_3' has become zero and cannot be further reduced. The sample is still intact and not yet failed. Failure can only be achieved by altering stress path 1 or 3 to stress path 2 (holding σ_3' constant, i.e. zero and increasing σ_1') until this stress path 2 reaches the intersection of the unconfined compression line and the deviator strength failure envelope.

It thus can be seen that no standard triaxial stress path can provide a failure envelope to the left of the unconfined compression line (Fig. 5.4.2). To extend the failure envelope into this region requires a tensile type test procedure (Lawrence, 1969) which was beyond the scope of this thesis.

Deviator strength envelopes of intact clays appear to intercept the unconfined compression line at much higher deviator strengths than fissured clays and therefore tensile tests appear to be necessary to adequately define their normal stress - shear strength relationship. Deviator strength envelopes of fissured clays, however, appear to intercept or become asymtopic to the unconfined compression line at low normal stresses and tensile tests are not necessary.

5.5 Effective Normal Stress Range

When defining a field problem, such as slope stability analysis, the importance of "tailoring the

Testing in the Low Normal Stress Range

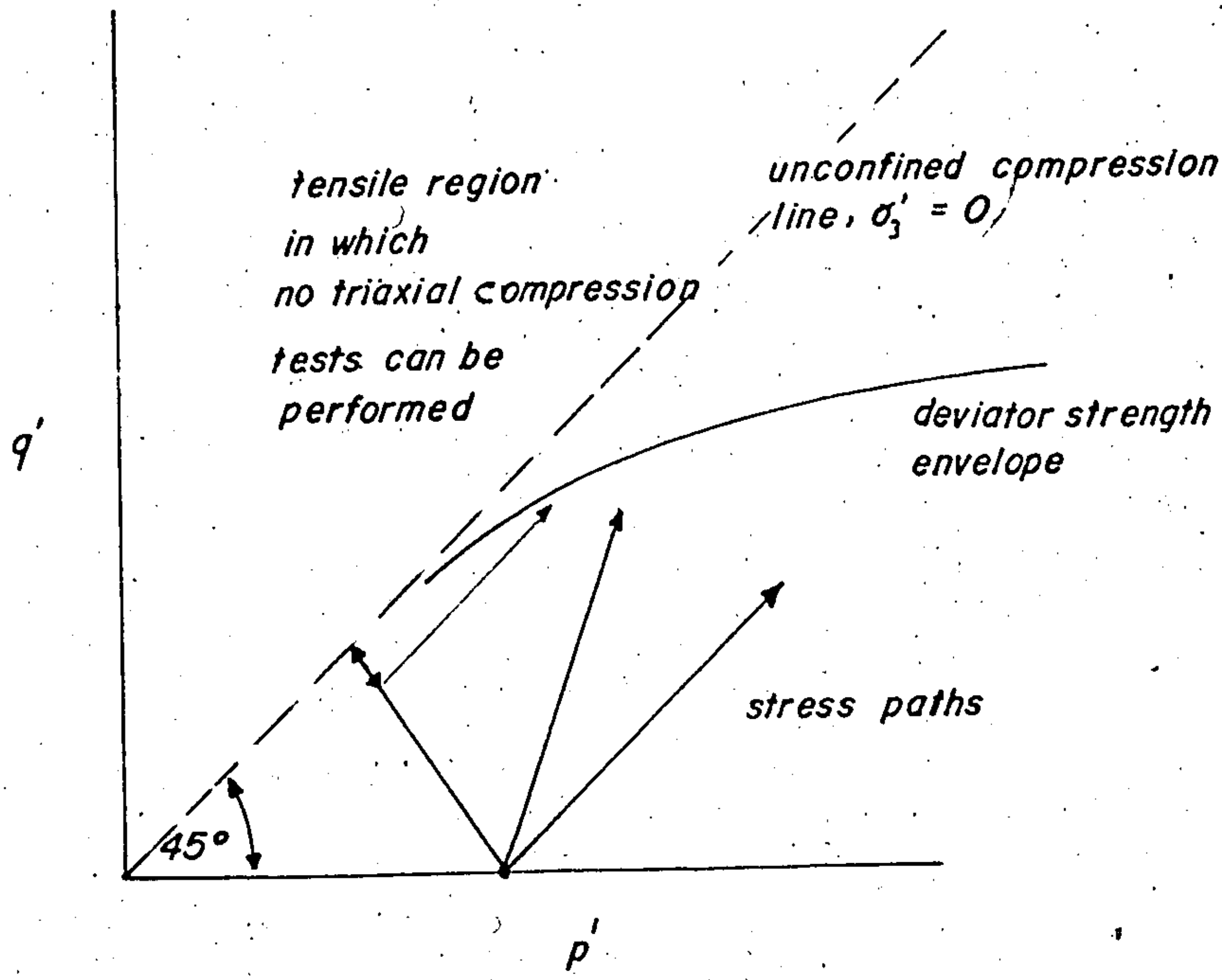


figure 5.4.2

laboratory testing accordingly" (Jarrett, 1972) cannot be over-emphasized. Such experimenters as Mitchell and Lawrence (1973), Lo and Lee (1973), and Lefebvre and LaRochelle (1973) have found that the average effective normal stress acting along a critical slip circle of a natural slope in fissured, redeposited clay, usually ranges from 2 to 10 lb/in^2 (0.15 to 0.75 kg/cm^2). Lefebvre and LaRochelle (1973) found that part of the failure "circle involving effective normal stresses less than 0.1 kg/cm^2 (1.4 lb/in^2) had a negligible influence upon the calculated factor of safety." Testing then, for fissured clays below the unconfined compression line is not essential for slope stability analysis. Shear testing for the three clays in this series was therefore performed between effective normal stresses of 2 and 15 lb/in^2 approximately.

5.6 Test Procedure

Two different test procedures were employed and are described below.

Test Procedure I

1. A back pressure (50 to 54 lb/in^2) was established over night (18 to 24 hours).
2. An all-round consolidation pressure equal to $P'_m = (\sigma'_1 + 2\sigma'_3)/3$ was applied,
3. Under this consolidation pressure, a plot of change in volume vs. log time was made, and an estimate to time to full failure for a drained strain controlled test was estimated by the method outlined in Bishop and Henkel (1962).
4. Under a stress controlled test, Bishop and Henkel (1962) recommend 8 to 10 increments to failure. Incremental loading was then chosen to obtain at least 8 load increments to failure.

5a. Time to failure of section 3) was applied not to the whole test, but rather to each load increment.

5b. If at the end of the calculated time for one increment, the rate of axial strain was greater than 2% per day, then the sample under this load increment was permitted to strain until its rate was less than 2% per day.

Test Procedure II

1. Back pressure applied as in Test Procedure I.
2. Preconsolidation pressure P'_c, was previously determined from an oedometer test.
3. If the chosen mean normal stress, $P'_m = (\sigma'_1 + 2\sigma'_3)/3$ was less than $P'_c/2$, the sample was isotropically consolidated to $P'_c/2$ and allowed to swell isotropically to the designated P'_m, prior to shearing.
4. If the chosen mean normal stress, P'_m, was greater than $P'_c/2$, the sample was isotropically consolidated to the mean normal stress, P'_m, prior to shear.
5. The same procedure was followed for shearing as in Test Procedure I.

The purpose of consolidating all samples to $P'_c/2$ when P'_m was less than $P'_c/2$ in Procedure II was to ensure that all samples tested were of a common consolidation history, as samples were obtained from varying depths, and was adopted in part for the last two series. Its influence upon test results has been discussed in Appendix A. The use of Procedure II is not recommended.

Samples were tested according to the following procedure:

Location	Procedure
South Nation	I
Castor River	I and II
Bear Brook	I and II

Some comments on the test procedure which are felt to be appropriate follow.

The rate of strain criteria of a maximum of 2 percent per day during shear appears to be reasonable. According to Lo and Morin (1972) an appropriate rate of testing for a CID test was 0.1 percent per hour or 2.4 percent per day. In the low normal stress range, Mitchell (1970) noted that "failure is not affected by rates of testing ranging from 3 hours to one week." Mitchell and Wong (1973) performed stress controlled constant P'_m tests "with stress increments of 0.20 kg/cm^2", changing load increments only when strain had decreased to a rate of 0.1% per day.

During shear, a maximum rate of strain of 2 percent per day was allowed prior to changing an increment. On this basis, the initial load increments were as short as an hour, and the final load increments sometimes as long as 4 to 15 hours in any one test.

During shear, the earlier increments of $\Delta q = \Delta(\sigma'_1 - \sigma'_3)/2$ were as large as 0.125 lb/in^2 ($\Delta\sigma'_3$ = 1.0 lb/in^2), but for the final increments, Δq, was usually reduced to 0.026 kg/cm^2 or 0.375 lb/in^2 ($\Delta\sigma'_3$ = 0.25 lb/in^2). Because of this low magnitude for the final load increment, the upper bound solution of the p'-$\underline{q}$' envelope was taken through the final increment point of each test.

CHAPTER 6

THREE LEDA CLAYS OF EASTERN ONTARIO

6.1 General

This Chapter deals with the site locations, soil profiles, index properties, vane strengths, and measured preconsolidation pressures of the three clays at South Nation, Castor River, and Bear Brook.

6.2 Site Locations and Soil Profiles

The geographical location of the South Nation, Castor River, and Bear Brook sites is shown in Fig. 6.2.1. Due to the level nature of the terrain, it would appear that all of the clay in the region would have similar properties. The approximate elevations of the plains in which the South Nation, Castor River, and Bear Brook valleys are cut are 188', 215', and 190' above sea level, respectively. All clay deposits are covered by 7 to 10 feet of fluvial and/or alluvial sand.

At the South Nation site, the interface between fissured clay and till is at about 123' (Fig. 6.2.2).

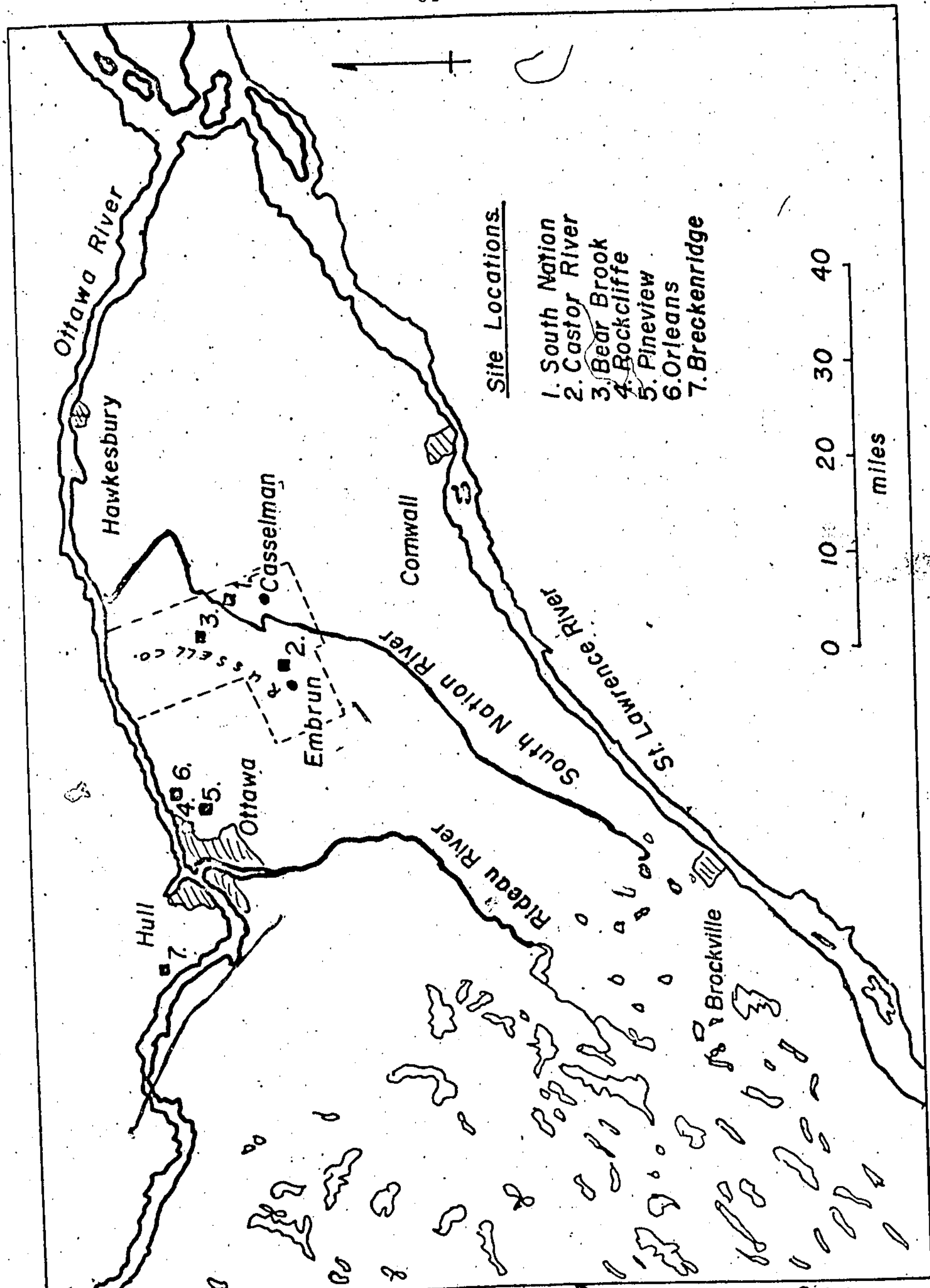

Ottawa River
Hawkesbury
Casselman
Cornwall
Embrun
Ottawa
Hull
Brockville
South Nation River
Rideau River
St. Lawrence River
Site Locations.
1. South Nation
2. Castor River
3. Bear Brook
4. Rockcliffe
5. Pineview
6. Orleans
7. Breckenridge
0
10
20
30
40
miles

figure 6.2.1.

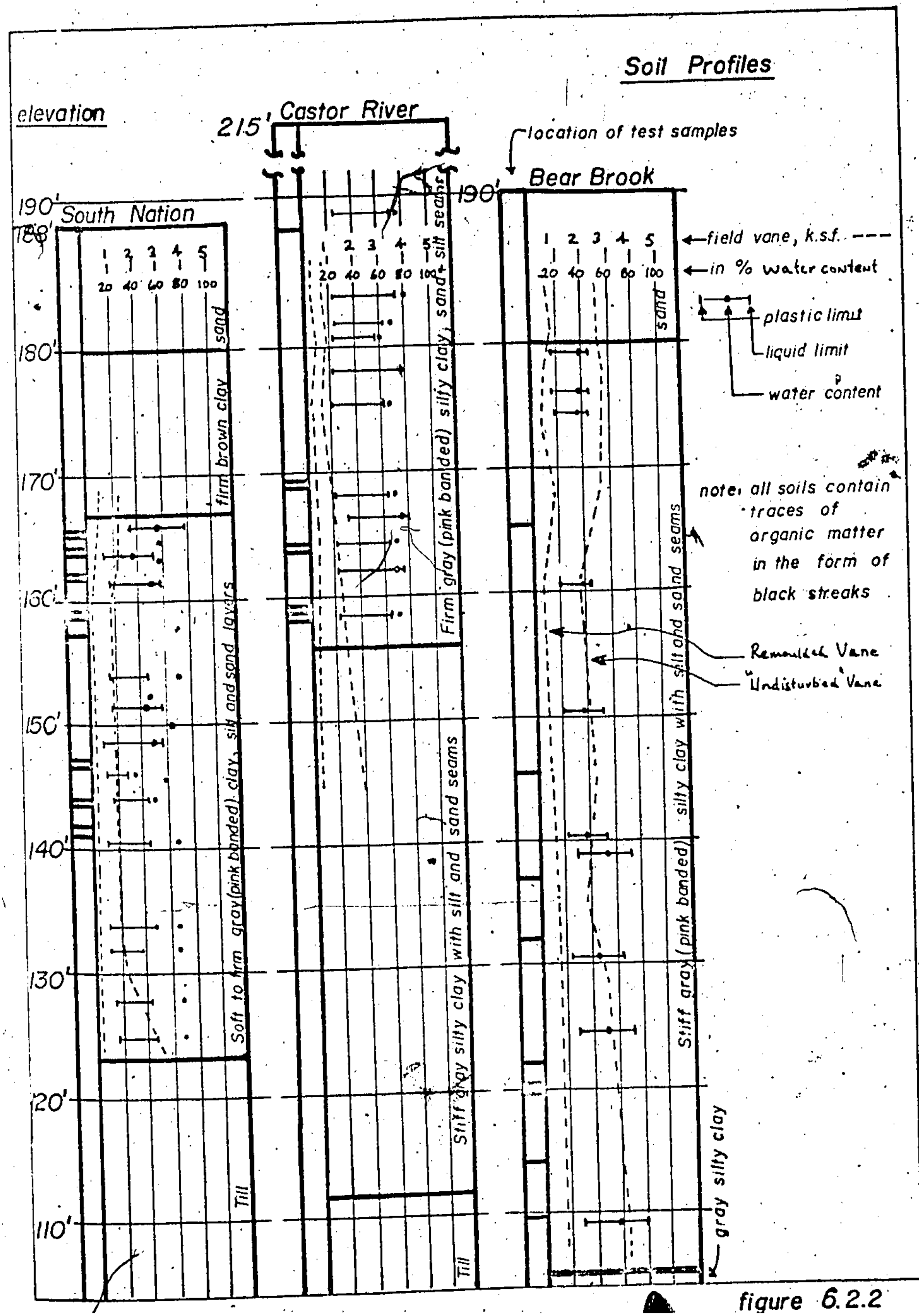

Soil Profiles
elevation
215'
Castor River
location of test samples
190'
Bear Brook
190'
South Nation
188'
field vane, k.s.f.
in % water content
plastic limit
liquid limit
water content
180'
170'
160'
150'
140'
130'
120'
110'
sand
firm brown clay
Soft to firm gray (pink banded) clay, silt and sand layers
Till
sand + silt seams
Firm gray (pink banded) silty clay, sand + silt seams
Stiff gray silty clay with silt and sand seams
Till
sand
Stiff gray (pink banded) silty clay with silt and sand seams
gray silty clay
note: all soils contain traces of organic matter in the form of black streaks
Remoulded Vane
"Undisturbed" Vane

figure 6.2.2

At the Castor River and Bear Brook sites, the interface between fissured and intact clay is at 156' and 108', respectively. Further details of the soil profile may be noted in Fig. 6.2.2.

Geologically speaking, these three clays are only similar in that they were deposited by similar natural processes at approximately the same time.

6.3 Index Properties

Index properties of the three fissured clays tested vary markedly, even though the three sites are quite close together as noted in Fig. 6.2.1.

Atterberg limits are given in Table 6.3.1. As noted in Fig. 6.2.2, all of the clays tested are pink-banded clays containing silt and sand seams.

The method of obtaining Atterberg limits for South Nation clays was to use different slices for the liquid limit, plastic limit, and the natural water content. As the pink banded gray clay has silt (observed when testing samples) seams, one would tend to treat the liquidity indices from the South Nation with caution. An average water content of 46.5 ± 11.3%[1] and a liquidity index of .694 ± .21[1] were obtained.

[1] One standard deviation.

TABLE 6.3.1

ATTERBERG LIMITS

												Mean, one Standard Deviation
Castor River												
Plastic Limit	(%)	27	23	23	22	22	23	26				
Liquid Limit	(%)	80	74	64	66	68	70	66				57±4.8
Moisture Content	(%)	75	72	62	63	64	66	67				
Plasticity Index	(%)	53	51	41	44	46	47	40				.932±042
Liquidity Index		.91	.96	.95	.93	.93	.91	1.02				
South Nation[1]												
Plastic Limit	(%)	27	18	16	32	18	18	15	18			
Liquid Limit	(%)	80	56	58	78	68	58	32	42			46.5±11.3
Moisture Content	(%)	55	57	43	58	46	48	23	42			
Plasticity Index	(%)	53	38	42	46	50	40	17	24			.694±.21
Liquidity Index		.53	1.08	.64	.57	.56	.75	.47	1.00			
Bear Brook												
Plastic Limit	(%)	32	20	28	30	30	20	20	28	25	21	
Liquid Limit	(%)	31	48	71	79	76	41	41	24	61	44	44.7±9.5
Moisture Content	(%)	47	39	51	54	59	32	35	53	43	34	
Plasticity Index	(%)	49	28	43	49	46	21	21	46	36	23	.50±.13
Liquidity Index		.31	.68	.54	.49	.63	.57	.72	.54	.48	.57	

[1] Sample depths presented in Figure 6.2.2.

Atterberg limits for Castor River and Bear Brook were obtained by slicing shelby tube samples vertically into three sections, thus obtaining composite silt and clay sections for average water content, liquid limit and plastic limit. Mean moisture constants and liquidity indices were 67 ± 4.8%, 0.932 ± .042, and 44.7 ± 9.5%, 0.50±0.13% for Castor River and Bear Brook, respectively. It is possible to say that for the soils tested, the Castor River soil is probably more uniform than the Bear Brook soil by noting the standard deviations given in Table 6.3.1. It is difficult to interpret the significance of the standard deviation for the liquidity index for the South Nation River deposit because of the test method.

Presented in Table 6.3.2 are Atterberg limits for some Ottawa area clays.

The plasticity chart of Fig. 6.3.1 shows that the three clays tested are of medium to high plasticity. The plasticity chart of Fig. 6.3.2 compares the tested clays to Ottawa region clays which are also clays of medium to high plasticity. Figure 6.3.3 shows a plasticity chart for five clays of Quebec. The sixth, from Hawkesbury, a clay of high plasticity is possibly 7

TABLE 6.3.2

ATTERBERG LIMITS

										Mean and One Standard Deviation
Orleans[1]										
Plastic Limit	(%)	29	28	28	27	20	22			
Liquid Limit	(%)	70	65	71	68	42	51			60.1 ±9.6%
Moisture Content	(%)	68	64	70	61	45	53			
Plasticity Index	(%)	41	37	43	41	22	29			
Liquidity Index		.95	.97	.97	.83	.88	.90			.916± .055
Breckenridge[2]										
Plastic Limit	(%)	30	28	27	28	28	27	28	27	
Liquid Limit	(%)	68	68	60	70	68	66	70	62	
Moisture Content	(%)	80	82	81	80	81	81	80	80	80.6 ±1.05%
Plasticity Index	(%)	38	40	33	42	40	39	40	35	
Liquidity Index		1.32	1.35	1.61	1.24	1.33	1.38	1.20	1.51	1.37 ± .14
Rockcliffe[3]										
Plastic Limit	(%)	22	28	24	28	28	28			
Liquid Limit	(%)	47	78	65	68	57	62			
Moisture Content	(%)	55	70	66	67	68	69			65.8 ±5.5
Plasticity Index	(%)	25	50	41	40	29	34			
Liquidity Index		1.32	0.84	1.02	.98	1.38	1.20			1.123± .19

[1] Orleans - Sample Depths (17'-35'), Elevations (235'-217')

[2] Breckenridge - Sample Depths (15'-60'), Elevations (315'-270')

[3] Rockcliffe - Sample Depths (11'-40'), Elevations (160'-130')

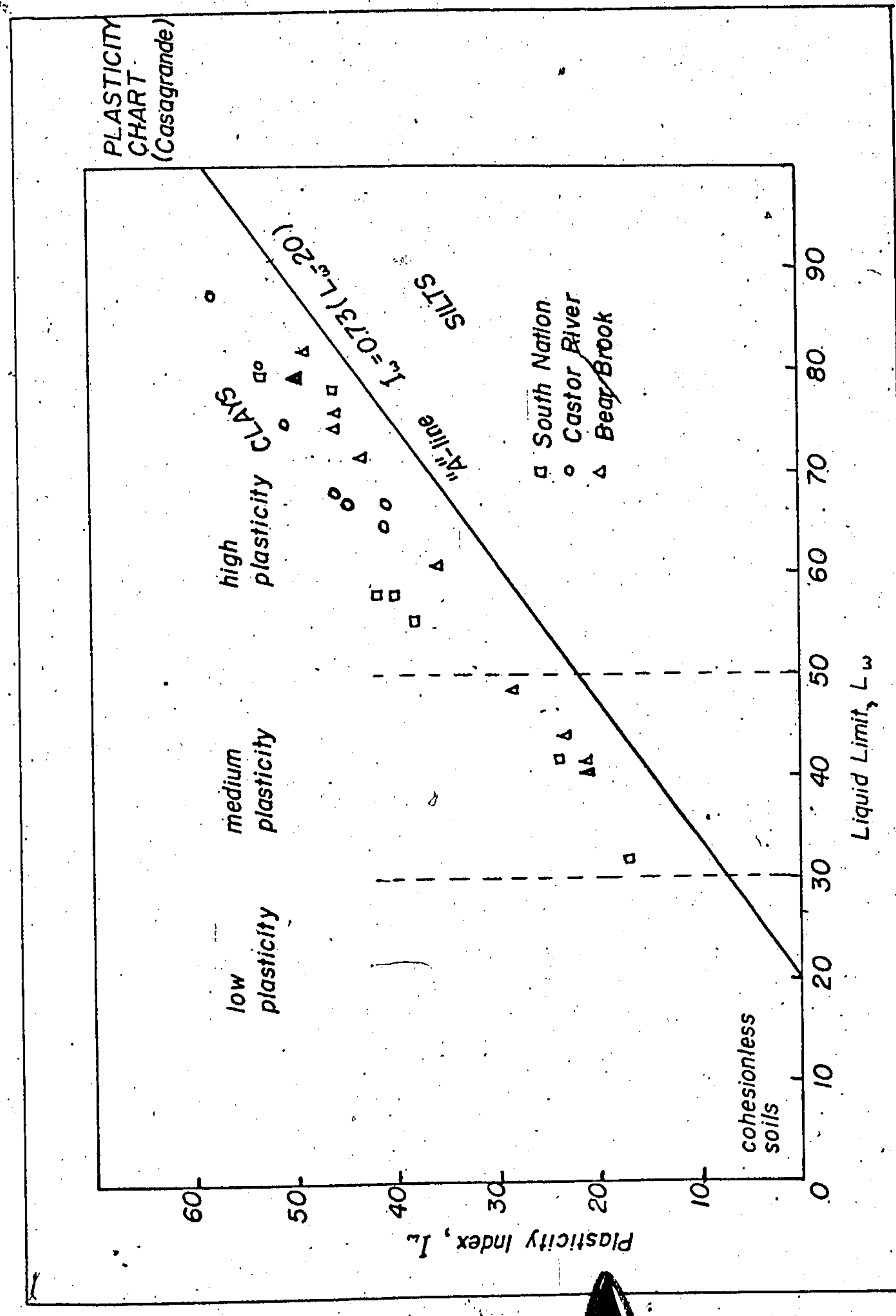
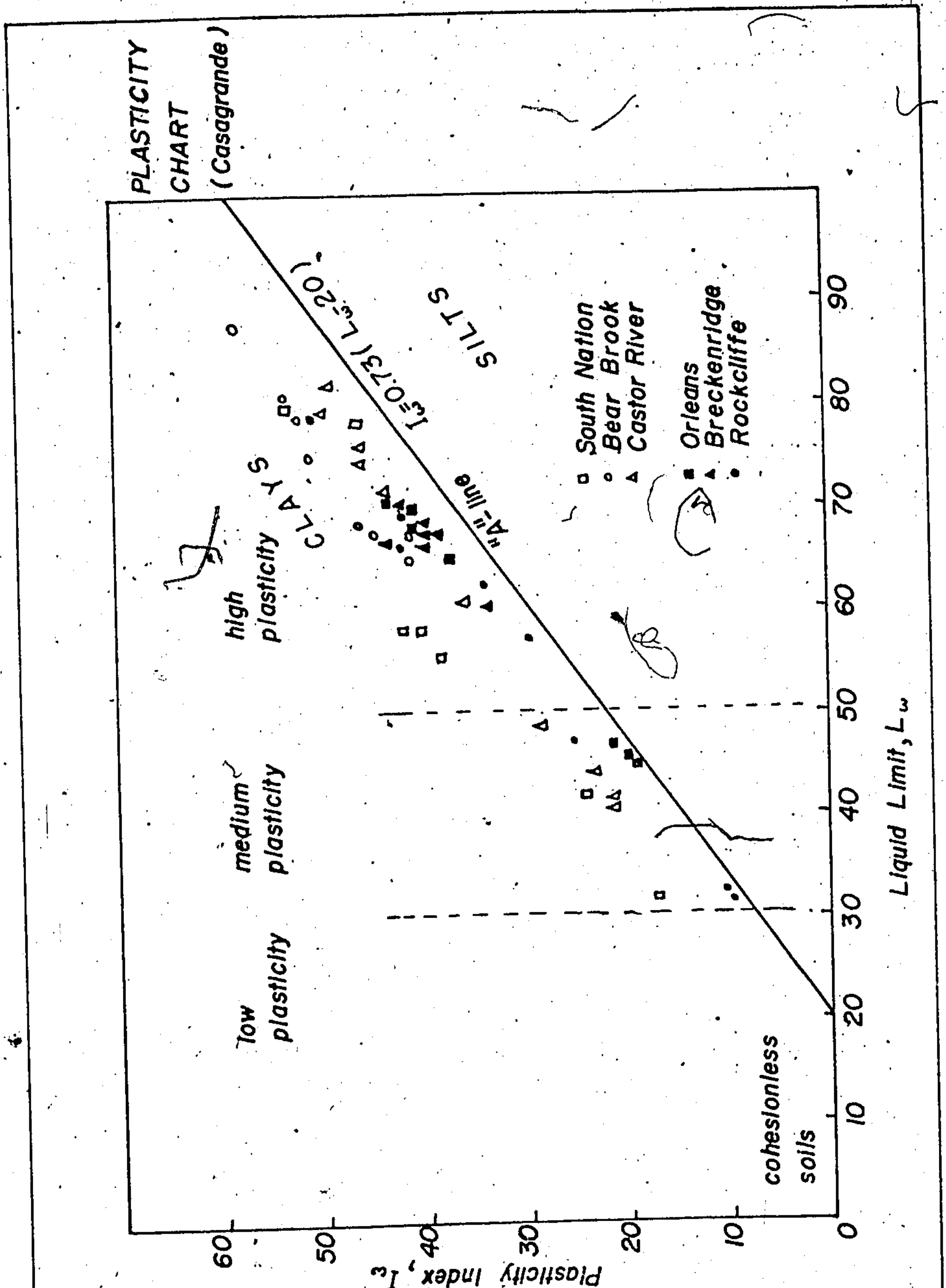

PLASTICITY CHART (Casagrande)
low plasticity
medium plasticity
high plasticity
CLAYS
SILTS
"A"-line
$I_w = 0.73(L_w - 20)$
South Nation
Castor River
Bear Brook
cohesionless soils
Liquid Limit, L_w
Plasticity Index, I_w
0
10
20
30
40
50
60
70
80
90

figure 6.3.1



PLASTICITY CHART (Casagrande)
Plasticity Index, I_ω
Liquid Limit, L_ω
"A" line
$I_\omega = 0.73(L_\omega - 20)$
CLAYS
SILTS
high plasticity
medium plasticity
low plasticity
cohesionless soils
South Nation
Bear Brook
Castor River
Orleans
Breckenridge
Rockcliffe
0
10
20
30
40
50
60
70
80
90

figure 6.3.2

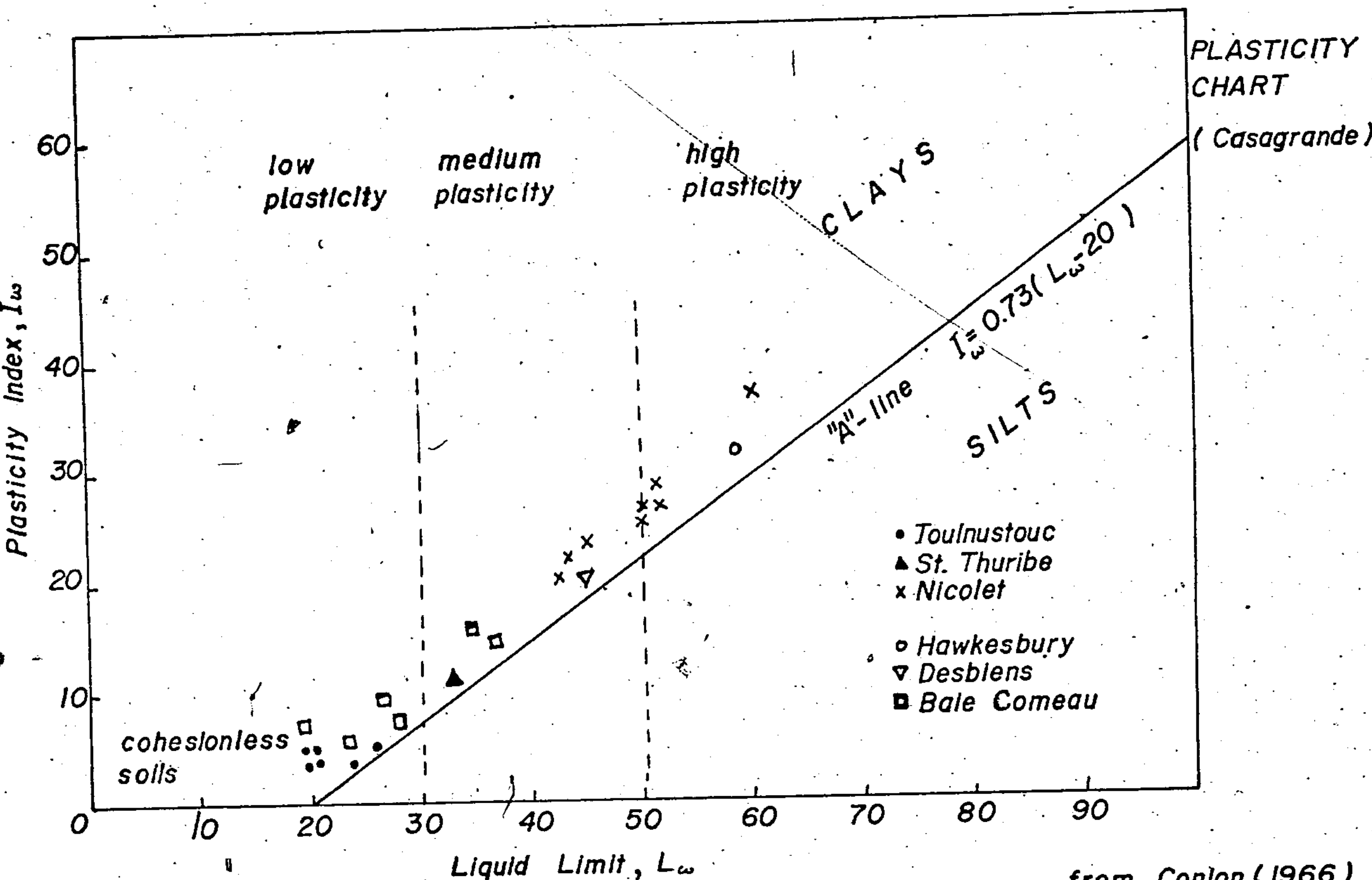

PLASTICITY CHART
(Casagrande)
low plasticity
medium plasticity
high plasticity
CLAYS
SILTS
$I_\omega = 0.73(L_\omega - 20)$
"A"-line
cohesionless soils
• Toulnustouc
▲ St. Thuribe
x Nicolet
o Hawkesbury
▽ Desbiens
◘ Baie Comeau
Plasticity Index, I_ω
Liquid Limit, L_ω
0
10
20
30
40
50
60
70
80
90
from Conlon (1966)

figure 6.3.3.

fissured. Toulnoustuc is definitely an intact clay (Conlon, 1966). The remainder are possibly intact, and are of low to medium plasticity.

Table 6.3.3 shows some properties of the fissured clays studied and of some fissured Ottawa area clays. Differences in properties between the intact Pineview clay and the other clays are not apparent. Pineview has a relatively large peak shear strength envelope for a given effective normal stress when compared to fissured clays. This difference is shear strength is not reflected by the other soil properties.

6.4 Vane Strength, Preconsolidation Pressure, and Elevation

Shown in Fig. 6.4.1 is a plot of all field vane strengths available for the three sites. It is apparent, by the curves drawn through the points, that field vane strength tends to increase with depth below the weathered crust, and to increase with elevation within the crust. The average field vane strengths at elevation 165 feet are 900 lb/lb^2, 1,100 lb/lb^2, 1,700 lb/lb^2 for

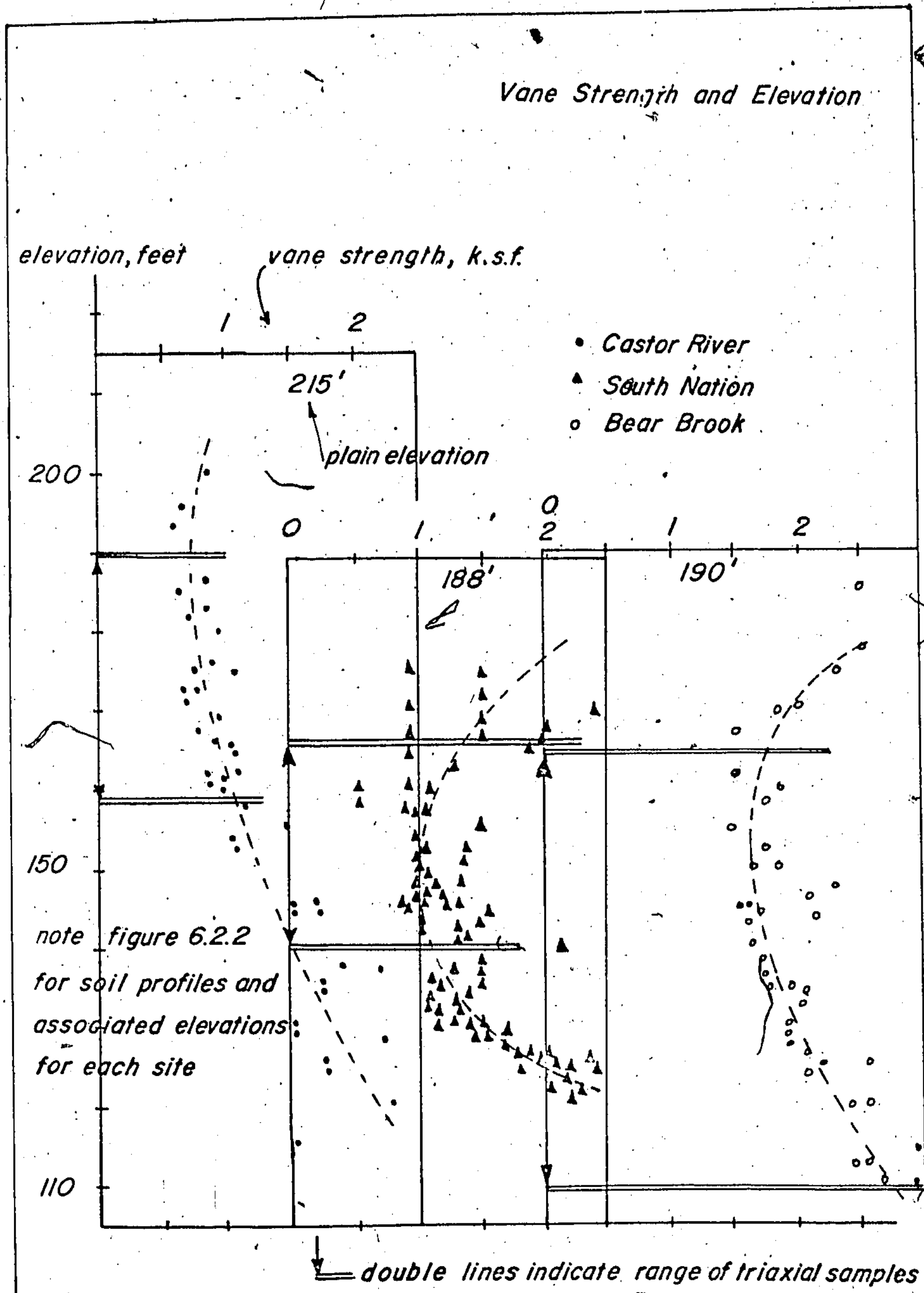
TABLE 6.3.3

EASTERN ONTARIO CLAYS*

	Field Vane lb/ft^2	Moisture Content %	Liquidity Index	Sensitivity	% Clay Size	Unit Weight lb/ft^3	Slope Height ft	Slope Angle o	Upper Plain Elevation ft
Castor River	800**	57±4.8	.93±.04	4-10	-	101-108	30	25	215
Breckenridge	1000**	80.0±1.5	1.37±.04	12-150	82	110	90	17	330
South Nation	1100**	46.5±11.3	.69±.21	8-11	50	103-115	70	10	188
Orleans	1400**	60.1±9.6	.92±.06	-	-	-	33	35	240
Rockcliffe	1450	65.8±5.5	1.12±.19	35	54	100	40	24	180
Pineview	1500	60-80	-	18	60	102	62	18	-
Bear Brook	2400**	44.7±9.5	.50±.13	3-6	-	104-108	38	22	190

*Sites from the literature identified in literature survey, Mitchell and Markell (1973)

**Field vane strengths associated with samples tested for shear strength envelopes.



Vane Strength and Elevation
elevation, feet
vane strength, k.s.f.
1
2
215'
plain elevation
• Castor River
▲ South Nation
o Bear Brook
200
0
1
2
0
2
1
2
188'
190'
150
note figure 6.2.2
for soil profiles and
associated elevations
for each site
110
double lines indicate range of triaxial samples

figure 6.4.1

Castor River, South Nation, and Bear Brook, respectively. The double horizontal lines on the figure show the range of depths from which the samples tested were obtained.

Table 6.4.1 compares field vane strength and preconsolidation pressure. Ratios of field vane strength to preconsolidation pressure ranging from 0.26 to 0.34 indicate a relationship between these parameters. The sample from B.H. No. 3, Castor River shows a ratio of 0.58. The clay from which this oedometer sample was taken has been subjected to a fill surcharge nearly equal to the preconsolidation value for several years. The confining effect of the fill may tend to increase vane strength and the measured preconsolidation pressure may be decreased due to disturbance of the bonded structure.

Crawford and Eden (1966) for Ottawa area clays, plotted elevation against preconsolidation pressure and field vane strength (Fig. 6.4.2). The top left of any line represented vane strength or consolidation in the soil just below the crust. The bottom right of any one line represented vane strength or consolidation in the bottom of the borehole.

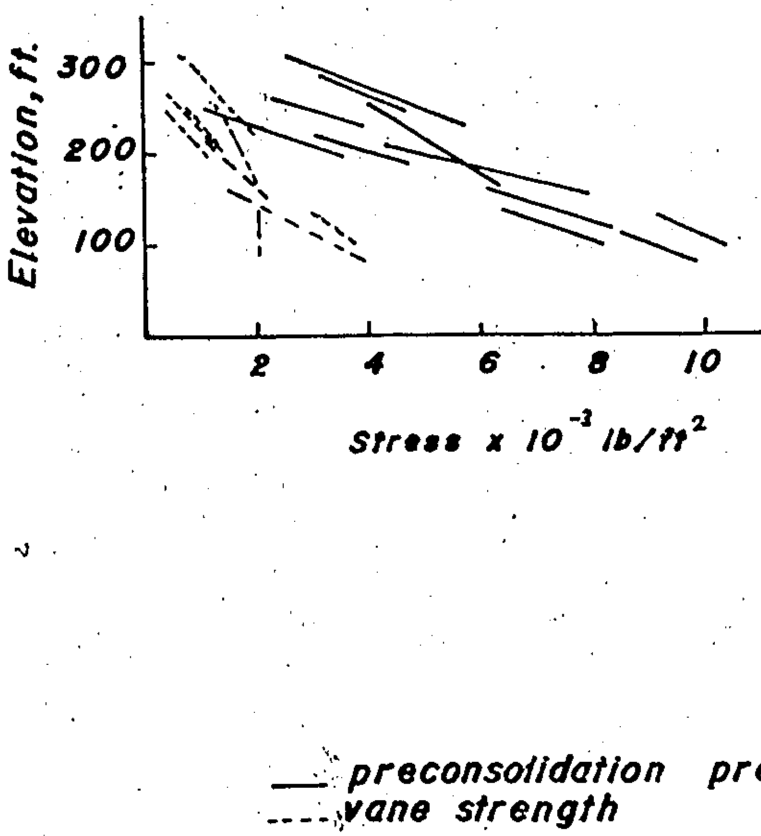
Upper and lower limits of Crawford and Eden's preconsolidation with elevation have been transposed to Fig. 6.4.3. Values of elevation and pre-

TABLE 6.4.1

FIELD VANE AND PRECONSOLIDATION

Location	Borehole	Elevation (feet)	Field Vane (lb/in^2)	Preconsolidation (lb/in^2)	Vane / Preconsolidation
Castor River	2	174	750	2,300	0.33
Castor River	3	178	1,100	1,900	0.58
South Nation	3	147	900	2,700	0.33
South Nation	4	156	900	3,050	0.30
Bear Brook	5	139	1,700	5,100	0.33
Bear Brook	5	151	1,700	6,500	0.26
Bear Brook	5	160	1,500	5,500	0.275
Bear Brook	5	173	2,000	6,200	0.32

Preconsolidation Pressure
Field Vane and
Elevation

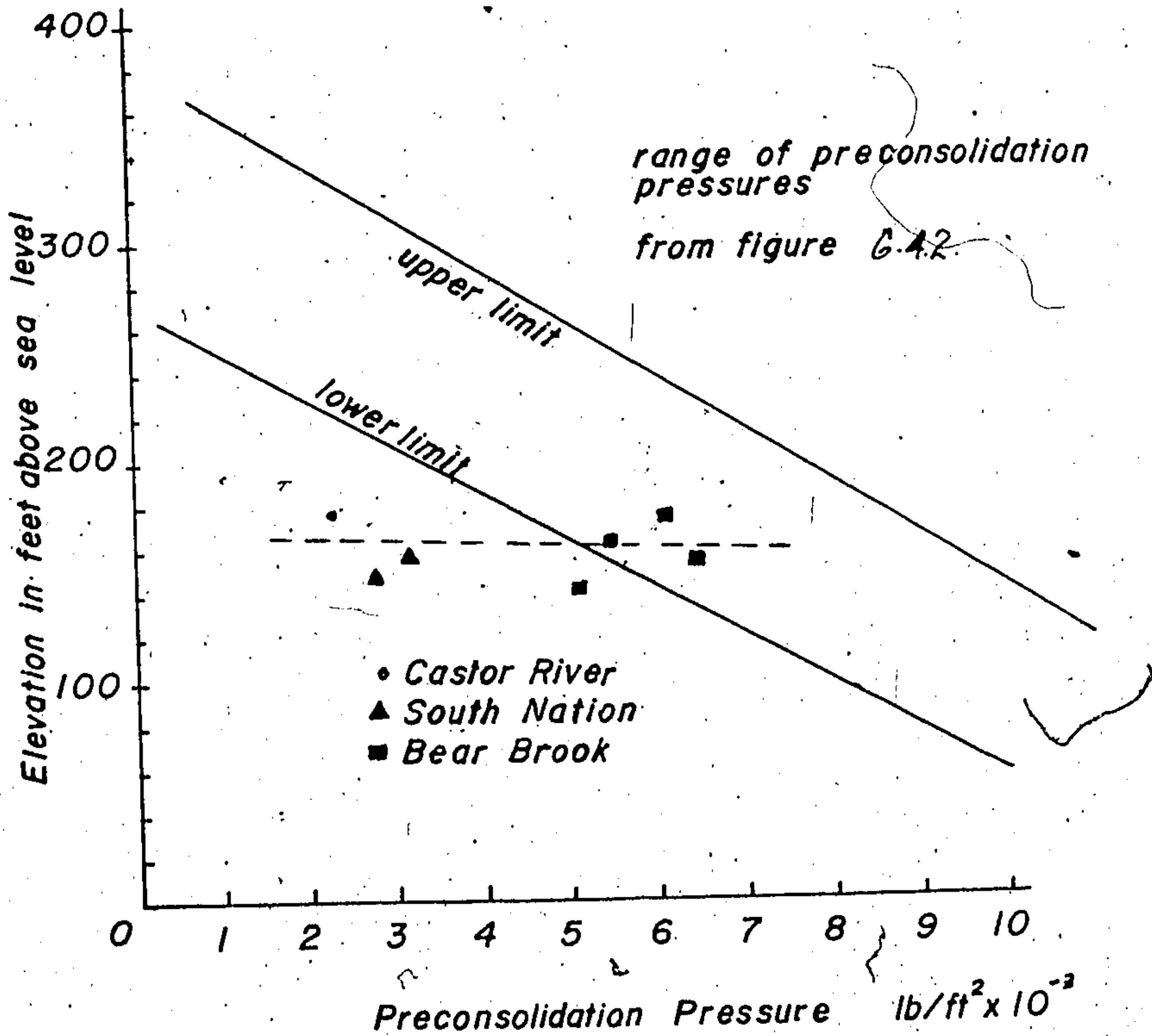


from Crawford & Eden (1966)

figure 6.4.2

Elevation and

Preconsolidation Pressure



$1000\ lb/ft^2 = 6.9\ lb/in^2 = 0.49\ Kg/cm^2$ $1\ lb/in^2 = 144\ lb/ft^2$

figure 6.4.3

preconsolidation for the three sites tested have been plotted.

From Crawford and Eden's analysis of Ottawa fissured clays, it would appear that measured preconsolidation at a given elevation across a number of sites would be approximately the same. This relationship does not appear to apply to the three fissured clays tested (Fig. 6.4.3). Confirmation of this trend contrary to Crawford and Eden exhibited by the seven preconsolidation tests can be noted in Fig. 6.4.1 where a great number of field vane strengths vary significantly at any given elevation below the crust across all three sites.

CHAPTER 7

PRESENTATION AND DISCUSSION

OF TEST RESULTS

7.1 General

This Chapter discusses volume change during consolidation and swelling, strain during strain, and yielding. The mode of failure in terms of effective normal stress is followed by the presentation of shear strength envelopes. An estimation of the effective normal stress at the boundaries between the stress ranges is given. With the exception of the section dealing with isotropic swelling, all descriptive sections are divided into three parts, i), ii) and iii). Each part represents a grouping of responses to consolidation and shear stresses.

It is proposed that all group i) summaries of experimental data are representative of the low normal stress range, ii) of the intermediate, and iii) of the high.

7.2 Initial Conditions

Following is Table 7.2.1 which list sample location, borehole number, elevation, depth below surface, in situ unit weight, and moisture content, as well as void ratio. These may be considered as initial conditions. Other data, such as void ratio and volume change during consolidation, swelling, and shear; values of p and q at failure and strain (axial) at failure are also listed.

Since the clay is banded with silt seams, the average void ratio for all clays varies widely, as summarized below:

	No. of Samples	Range of Void Ratio	Mean and One Standard Deviation
South Nation	17	1.07-1.78	1.42±.20
Castor River	8	1.36-1.76	1.51±.14
Bear Brook	10	1.08-1.71	1.44±.22

Since the samples used in testing are from shelby tubes from various elevations it would be expected that the void ratio would vary. However, as shown later, the large variations in void ratio did not appear to cause significant variation in the test results. It would seem that the behaviour of the clay in consolidation and shear was

TABLE 7.2.1 Properties of Samples Tested

Test No.	B.H. No.	S.A. No.	Depth	Elevation	γ in situ	Natural Moisture Content	Void Ratios: Initial	Void Ratios: Post Consolidation	Void Ratios: Post Swelling	Void Ratios: Post Shear	Volume Changes: Post Consolidation	Volume Changes: Post Swelling	Volume Changes: Post Shear	Consolidation Pressure	Constant "p_m" $P_m = \frac{\sigma_1'+2\sigma_3'}{3}$	Strain at Failure	$p = \frac{\sigma_1' + \sigma_3'}{2}$	$q' = \frac{\sigma_1' - \sigma_3'}{2}$
			ft	ft	lb/ft^3	%					$\%\frac{\Delta V}{V}$	$\%\frac{\Delta V}{V}$	$\%\frac{\Delta V}{V}$	lb/in^2	lb/in^2	%	lb/in^2	lb/in^2
SOUTH NATION RIVER																		
17	4	1	21'	160'	107.5	55	1.54	1.56	-	1.57	-.75	-	-.60	2.3	2.3	2.18	3.20	2.75
17	4	3	24'	157'	112.0	45	1.24	1.24	-	-	-.25	-	-	2.5	2.5	2.06	3.63	3.38
15	4	2	23'	158'	105	51.5	1.17	1.16	-	1.16	-.5	-	-.01	3.0	3.0	2.09	4.13	3.38
14	3	13	19'	161'	111.5	46.	1.38	1.36	-	1.37	-.8	-	+.35	4.0	4.0	4.96	5.50	4.50
13	3	12	18'	162'	115.	39.	1.09	1.07	-	1.08	-.8	-	+.35	5.0	5.0	2.93	6.75	5.25
12	3	11	16'	164'	110.	46.	1.35	1.32	-	1.32	-.75	-		6.0	6.0	5.50	7.94	5.81
11	3	12	18'	162'	109.5	49.	1.34	1.32	-	1.32	-1.20	-	-.05	7.0	7.0	2.0	9.00	6.00
9	3	11	16'	164'	107.0	52.	1.40	1.37	-	1.37	-1.40	-	-.05	8.0	8.0	3.00	10.25	6.75
10	3	13	19'	161'	108.5	48.	1.46	1.42	-	1.42	-1.65	-	-.05	9.0	9.0	2.0	11.50	7.50
24	3	26	36'	144'	107.5	66.2	1.47	1.38	-	1.29	3.5	-	-1.25	12.0	12.0	4.0	14.25	6.75
20	3	26	36'	144'	103.	67.8	1.68	1.60	-	1.57	-3.0	-	-1.30	12.0	12.0	4.0	14.25	6.75
18	3	24	33'	147'	126.	34.	1.06	1.00	-	0.93	-2.5	-	-2.9	15.0	15.0	12.6	18.50	10.50
25	3	29	40'	140'	104.	58.	1.62	1.54	-	1.47	-3.1	-	-2.6	17.0	17.0	6.76	18.0	9.0
21	3	26	36'	144'	104.	59.8	1.67	1.52	-	1.33	-5.25	-	-6.9	20.0	20.0	13.6	21.13	9.38
22	3	24	33'	147'	105.	61.0	1.78	1.64	-	1.54	-4.0	-	-3.90	21.0	21.0	8.25	23.75	8.25
23	3	28	39'	141'	109.5	49.6	1.40	1.24	-	1.04	-6.5	-	-8.6	27.0	27.0	18.3	22.25	9.75

Table 7.2.1 - continued

Test No.	B.H. No.	S.A. No.	Depth	Elevation	γ in situ	Natural Moisture Content	Void Ratios: Initial	Void Ratios: Post Consolidation	Void Ratios: Post Swelling	Void Ratios: Post Shear	Volume Changes: Post Consolidation	Volume Changes: Post Swelling	Volume Changes: Post Shear	Consolidation Pressure	Constant "p"$_m$ $p_m = \frac{\sigma_1'+2\sigma_3'}{3}$	Strain at Failure	$p = \frac{\sigma_1' + \sigma_3'}{2}$	$q' = \frac{\sigma_1' - \sigma_3'}{2}$
			ft	ft	lb/ft^3	%					$\%\frac{\Delta V}{V}$	$\%\frac{\Delta V}{V}$	$\%\frac{\Delta V}{V}$	lb/in^2	lb/in^2	%	lb/in^2	lb/in^2
CASTOR RIVER																		
44	4	6	25'	187'	108	53.	1.49	1.47	-	1.30	-.75	-	+1.20	2.0	2.0	2.7	2.813	2.563
31	3	13	38'	172'	107	58.5	1.74	1.71	-	1.69	-1.0	-	- .67	3.0	3.0	5.1	3.88	2.63
30	3	15	45'	165'	105	55.0	1.47	1.36	1.38	1.38	-5.0	+0.5	- .17	10.0	5.0	4.0	6.38	4.13
27	3	16	51'	159'	106	53.8	1.61	1.36	1.36	1.34	-4.0	+0.2	-0.47	10.0	7.0	6.35	8.75	5.25
28	3	16	51'	159'	106	57.4	1.55	1.46	1.47	1.44	-3.5	+0.2	-1.17	10.0	8.0	6.55	9.88	5.63
29	3	15	45'	165'	101	64.4	1.76	1.64	1.64	1.61	-4.5	+0.2	-0.9	10.0	10.0	5.6	11.75	5.25
32	3	13	38'	172'	102	63.5	1.74	1.67	-	1.62	-2.5	-	-1.9	11.0	11.0	4.0	12.88	5.63
33	3	16	51'	159'	107	54.0	1.51	1.42	-	1.80	-1.6	-	-2.8	13.0	13.0	10.0	15.13	6.38
BEAR BROOK																		
37	3	10	45'	145'	104.0	48.	1.34	-	-	-	-	-	+ .12	20.0	4.0	2.00	5.87	5.62
40	1	11	60'	114'	108.0	43.0	1.08	1.04	1.05	1.07	-1.5	+0.5	+ .8	10.0	5.0	2.22	7.13	6.38
35	3	6	25'	165'	106.	52.9	1.47	-	-	-	-	-	-	20.0	6.0	4.15	8.00	6.00
42	2	9	40'	132'	106.	58.4	1.68	1.62	1.63	1.63	-2.0	+0.4	-.17	10.0	7.0	1.90	9.50	7.50
34	2	8	35'	137'	106.5	56.	1.51	-	-	-	-	-	-.8	20.0	8.0	3.13	10.25	6.75

Table 7.2.1 - concluded

Test No.	B.H. No.	S.A. No.	Depth	Elevation	γ in situ	Natural Moisture Content	Void Ratios: Initial	Void Ratios: Post Consolidation	Void Ratios: Post Swelling	Void Ratios: Post Shear	Volume Changes: Post Consolidation	Volume Changes: Post Swelling	Volume Changes: Post Shear	Consolidation Pressure	Constant "p_m" $p_m = \frac{\sigma_1' + 2\sigma_3'}{3}$	Strain at Failure	$p = \frac{\sigma_1' + \sigma_3'}{2}$	$q' = \frac{\sigma_1' - \sigma_3'}{2}$
			ft	ft	lb/ft^3	%					$\%\frac{\Delta V}{V}$	$\%\frac{\Delta V}{V}$	$\%\frac{\Delta V}{V}$	lb/in^2	lb/in^2	%	lb/in^2	lb/in^2
36	3	14	80'	110'	106.5	51.1	1.36	1.25	1.28	1.29	-3.5	+.8	+.4	20.0	10.0	2.68	12.88	8.63
39	2	12	60'	112'	106.	54.5	1.45	1.40	-	1.43	-2.0	-	+.8	14.0	14.0	2.30	17.63	10.88
43	2	13	70'	102'	107.	55.4	1.71	1.62	-	1.71	-3.2	-	+3.2	16.0	16.0	7.10	19.75	11.25
41	2	13	70'	102'	105	52.9	1.37	1.30	-	1.26	+3.0	-	-1.7	18.0	18.0	5.90	21.88	11.63
38	2	11	50'	122'	105.	54	1.51		-	-	-	-	+1.20	22.0	22.0	3.20	26.0	12.00

almost completely controlled by the bonding and fissures and that the initial void ratio had little effect.

7.3 Volume Change from Consolidation and Swelling

In this section is a summary of response to consolidation and swelling under all-around pressure in a triaxial cell for the clays of Castor River, South Nation, and Bear Brook. Presentation is divided into subsections of consolidation, swelling and magnitude of volume change.

Isotropic Consolidation

Castor River. Isotropic consolidation (Fig. 7.3.1) $\%\Delta V/V$ vs. log time.

*i) For P'_m of 2.0 and 3.0 lb/in^2, the two curves were sequential, and $\%\Delta V/V$ was found to be 1/3 and 1% respectively. Time to full consolidation varied from 50 to 100 minutes.

*ii) For P'_m of 10 or 11 lb/in^2 (6 curves), $\%\Delta V/V$ ranged from 2% to 6%. Time to full consolidation was from 100 to 300 minutes.

*i), ii) refer to the low and intermediate normal stress ranges respectively.

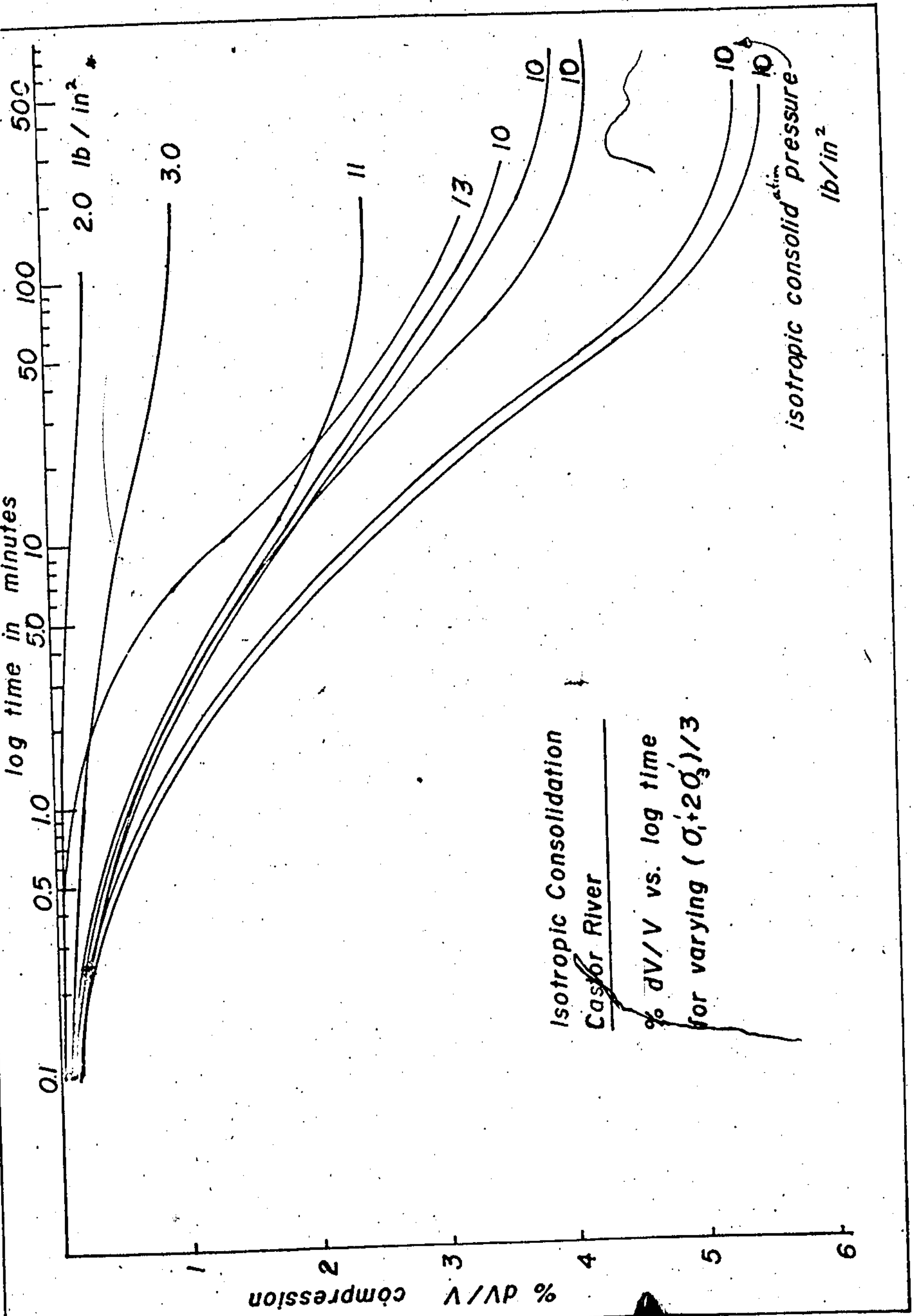

log time in minutes
0.1
0.5
1.0
5.0
10
50
100
500
2.0 lb/in²
3.0
11
13
10
10
10
10
10
isotropic consolidatim pressure lb/in²
Isotropic Consolidation
Castor River
% dV/V vs. log time
for varying $(\sigma_1' + 2\sigma_3')/3$
% dV/V compression
1
2
3
4
5
6

figure 7.3.1.

*iii) For P'_m of 13 lb/in^2 consolidation was terminated at about 200 minutes and it would appear that this was close to 100% consolidation.

South Nation. Isotropic consolidation (Fig. 7.3.2) %ΔV/V vs. log time.

i) For P'_m of 2.3 lb/in^2 to 9.0 lb/in^2, the curves were in exact sequence, %ΔV/V reached a maximum value of 1-1/2% at 9 lb/in^2 P'_m. Time to full consolidation was from 5 to 20 minutes.

ii) For P'_m of 12.0 lb/in^2 to 17.0 or 21.0 lb/in^2, the curves were in approximate sequence. %ΔV/V ranges from 2-1/2 to 3-1/2 percent. Time to consolidation is from 50 to 100 minutes.

iii) For P'_m of 20.0 and 27.0 lb/in^2 the curves reached consolidation at about 400 minutes, and %ΔV/V was 5 to 6%. The consolidation for P'_m = 27 lb/in^2 was carried on for 1300 minutes and showed a linear secondary consolidation to this time.

Bear Brook. Isotropic consolidation (Fig. 7.3.3) %ΔV/V vs. log time.

---

*iii) refers to the high normal stress range.

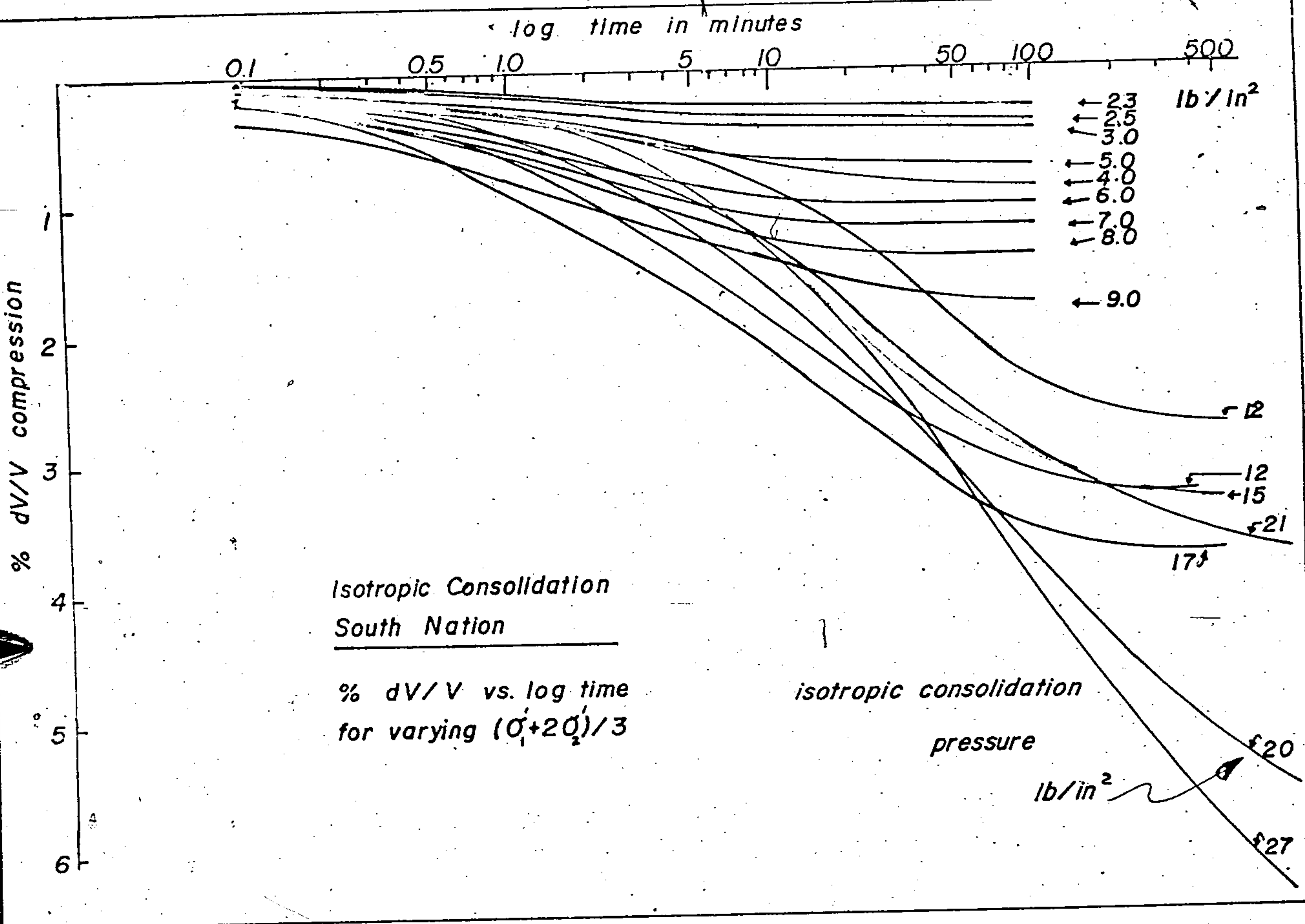

log time in minutes
0.1
0.5
1.0
5
10
50
100
500
lb/in²
2.3
2.5
3.0
5.0
4.0
6.0
7.0
8.0
9.0
12
12
15
21
17
20
27
% dV/V compression
1
2
3
4
5
6
Isotropic Consolidation
South Nation
% dV/V vs. log time
for varying (σ'₁+2σ'₂)/3
isotropic consolidation
pressure
lb/in²

figure 7.3.2

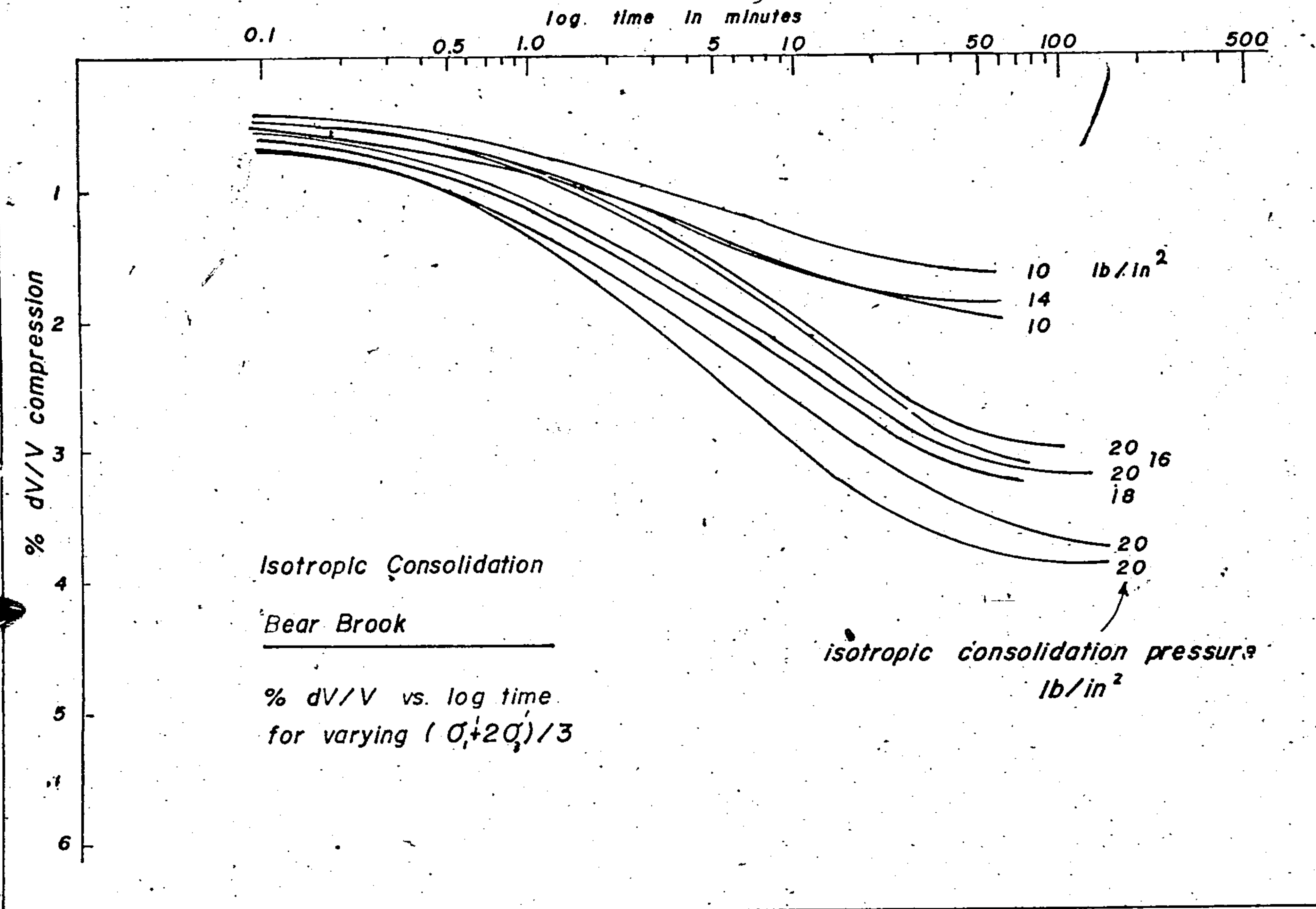

log. time in minutes
0.1
0.5
1.0
5
10
50
100
500
% dV/V compression
1
2
3
4
5
6
10 lb/in^2
14
10
20
16
20
18
20
20
isotropic consolidation pressure lb/in^2
Isotropic Consolidation
Bear Brook
% dV/V vs. log time
for varying $(\sigma_1' + 2\sigma_3')/3$

figure 7.3.3.

All low P'_m for Bear Brook have been consolidated to pressures greater than P'_m.

i) For P'_m of 10, 10, and 14 lb/in^2, the %ΔV/V ranged from 1-1/2 to 2%. Time to full consolidation was from 30 to 40 minutes.

ii) For P'_m of 16, 18, 20, 20, 20, 20 lb/in^2, the curves are not in sequence, but grouped. %ΔV/V ranged from 3 to 4%. Time to full consolidation was from 60 to 90 minutes.

iii) No tests in this range.

Test procedure II (consolidating samples under all-around pressure greater than P'_m, of 10 or 20 lb/in^2, then swelling to P'_m) was followed for most but not all tests in the very low stress range at Castor River and Bear Brook. For South Nation, test procedure I was followed.

Inspection of Fig. 7.3.2 for South Nation indicates a definite pattern as described above. It is probable that excess porewater pressures due to all-around consolidation pressures up to at least 9 lb/in^2 are dissipated through internal drainage paths provided by fissures. Consolidation at a cell pressure of 12 lb/in^2 appears to close up fissures, hence increasing time to full porewater pressure dissipation to approximately 200 minutes from about 20 minutes for 9 lb/in^2. The influence upon time to consolidation by length of the average drainage path has been discussed by Bishop and Henkel (1962).

Curves for consolidation at pressures of 12, 15, 17, and 21 lb/in^2 suggest a gradual breakdown of bonds with increasing cell pressure. The curves of 20 and 27 lb/in^2 did not appear to be approaching the end of volume change at 400 and 1300 minutes respectively, indicating a plastic yielding.

Although most tests were consolidated above P'_m at the remaining two sites, it would appear that the fissures close up at between 14 (15 minutes to full consolidation) and 16 lb/in^2 (50 minutes to full consolidation) for Bear Brook and between 3 (30 minutes to full consolidation and 10 lb/in^2 (200 minutes to full consolidation) at Castor River. Plastic yield under all-around consolidation ~~is~~ not apparent at the two latter sites for the ranges of isotropic consolidation employed.

Isotropic Swelling

<u>Castor River</u>. Isotropic swelling (10 lb/in^2)

(Fig. 7.3.4) $\%\Delta V/V$ vs. log time.

For P_m' of 5, 7, 8 lb/in^2 the curves were in sequence $\%\Delta V/V$ was a maximum for 5 lb/in^2 (0.5%) since the change in pressure at beginning of swelling was the greatest, 5 lb/in^2. Time to full rebound ranged from 10 to 100 minutes.

Bear Brook. Isotropic swelling (20 lb/in^2) (Fig. 7.3.4) $\%\Delta V/V$ vs. log time.

For P_m' of 4, 6, 8, and 10 lb/in^2 the curves were nearly in sequence. $\%\Delta V/V$ ranged from 0.5 to 1.5%. Time to full rebound ranged from 30 to 90 minutes.

Bear Brook. Isotropic swelling (10 lb/in^2) (Fig. 7.3.4) $\%\Delta V/V$ vs. log time.

For P_m' of 3 and 5 lb/in^2 the curves were in sequence. $\%\Delta V/V$ ranged from 0.4 to 0.6%. Time to full rebound was from 20 to 30 minutes.

Only tests from Castor River and Bear Brook were consolidated to 10 and 10 or 20 lb/in^2 respectively and allowed to swell back to the value of P_m'. Time to complete full rebound to P_m' was found to be of the order of about 20 to 50 minutes in most instances. It is possible that fissures reopened under the reduced

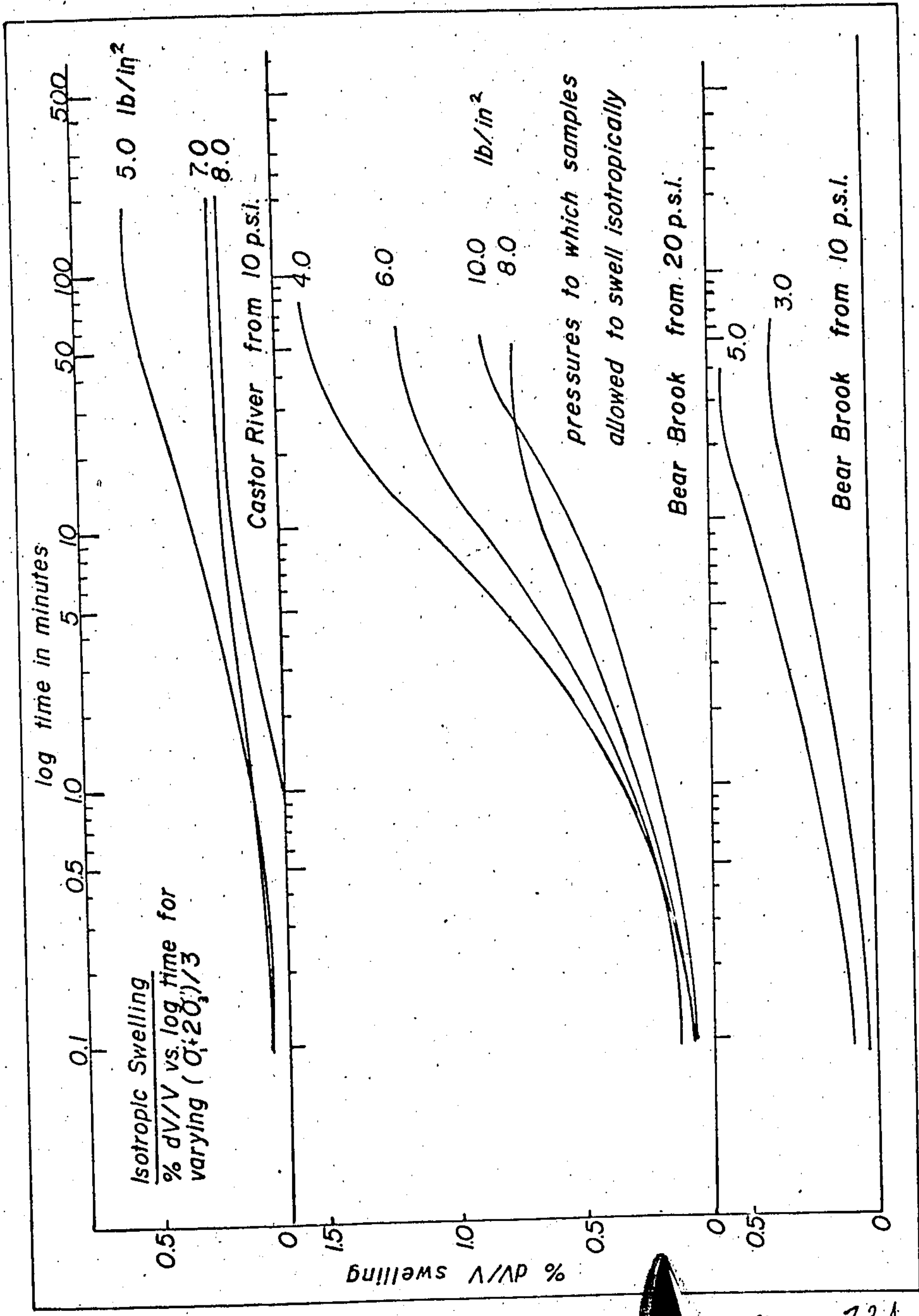

Isotropic Swelling
% dV/V vs. log time for
varying $(\sigma_1 + 2\sigma_3)/3$
log time in minutes
0.1
0.5
1.0
5
10
50
100
500
5.0 lb/in²
7.0
8.0
Castor River from 10 p.s.i.
4.0
6.0
10.0
8.0
lb/in²
pressures to which samples
allowed to swell isotropically
Bear Brook from 20 p.s.i.
5.0
3.0
Bear Brook from 10 p.s.i.
% dV/V swelling
0.5
0
1.5
1.0
0.5
0
0.5
0

figure 7.3.4.

all around confining pressure, resulting in a shorter path for water to flow to satisfy swelling.

The presence of these fissures both in swelling and in consolidation is of considerable importance. It is not known if these fissures are horizontally, vertically or randomly orientated. Drainage of fissures into sand and silt seams known to exist in situ in these three deposits may greatly affect permeability. Further investigation of the influence of varying effective normal stress upon permeability would be of interest.

Magnitude of Volume Changes

Although the samples are of varying void ratio (Section 7.2), it would seem that compression under all around consolidation is a function of the consolidation pressure (Fig. 7.3.5). The best example of this phenomenon is South Nation. It is possible that volume change is independent of initial void ratio and dependent upon the bonded structure. This bonded structure appears to be uniform over the depth tested. Possibly then, it is not necessary to use clay of the same average void ratio from block samples in testing programs.

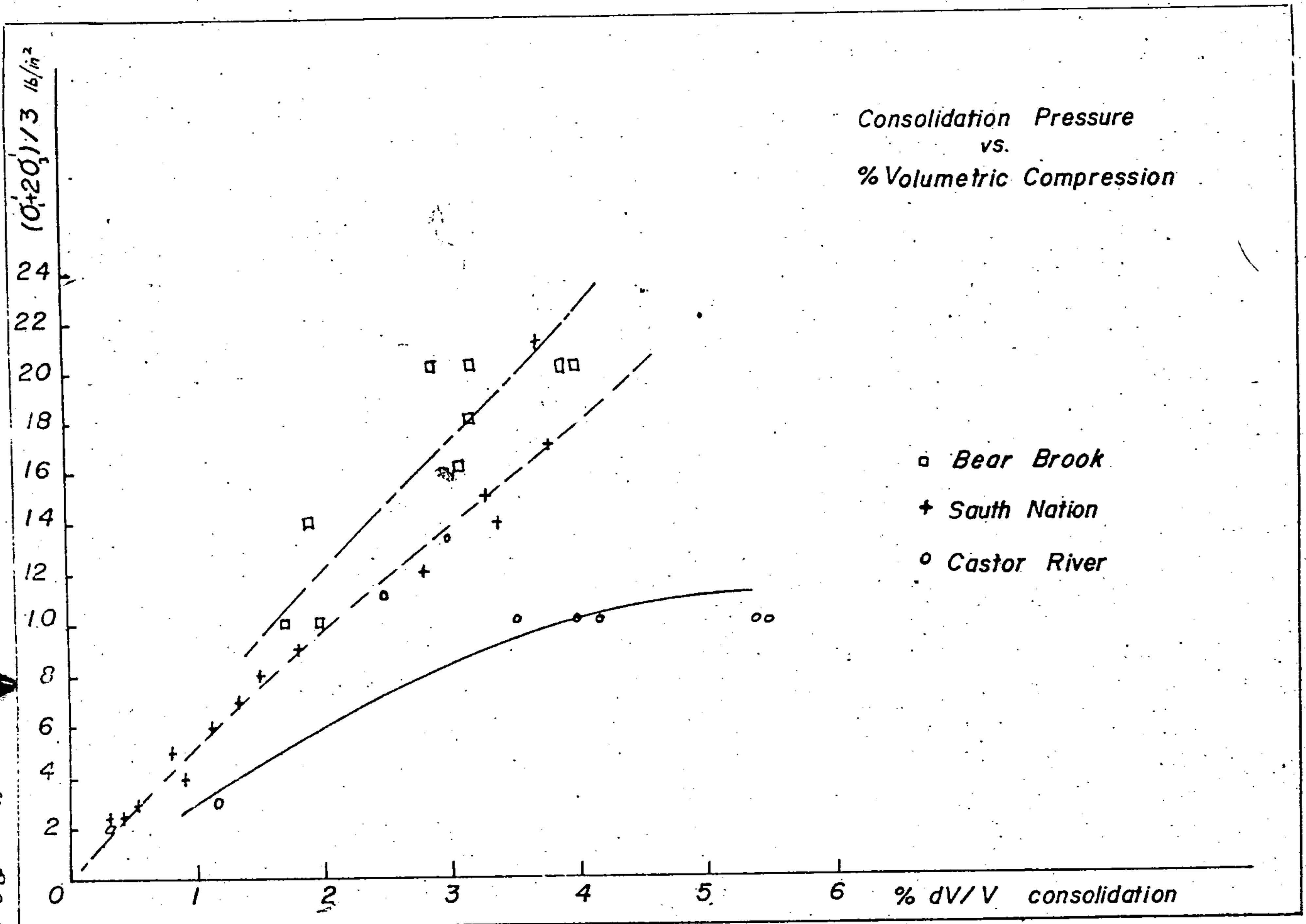

Consolidation Pressure
vs.
% Volumetric Compression
Bear Brook
Sauth Nation
Castor River
$(\sigma_1'+2\sigma_3')/3$ lb/in²
24
22
20
18
16
14
12
10
8
6
4
2
0
1
2
3
4
5
6
% dV/V consolidation

figure 7.3.5

The arithmetically plotted data for the three sites is represented in the form of log pressure vs. %dV/V in Figs. 7.3.6, 7.3.7, and 7.3.8 for Castor River, South Nation, and Bear Brook. The curves, as a function of all around consolidation pressure, $\sigma_1' = \sigma_2' = \sigma_3'$, are represented by solid triangles. Also shown in these figures are representative oedometer consolidation curves. In these cases, pressure is the major principal stress, σ_1', and points on these curves are represented by crosses. It should be noted that the points under isotropic triaxial consolidation are from different samples which vary widely in void ratio. Points on the oedometer curve are from successive loads on one sample. Investigation of the triaxial curve for South Nation (Fig. 7.3.7) indicates a definite break at 7.6 lb/in^2. It would appear that the combination of principal stresses are inducing this break in the curve and not simply the magnitude of σ_1', the major principal stress.

The average value of σ_1' at P_c is about 20 lb/in^2 from the consolidation tests. If the magnitude of the average principal stress controlled this break in the curve of the triaxial tests,

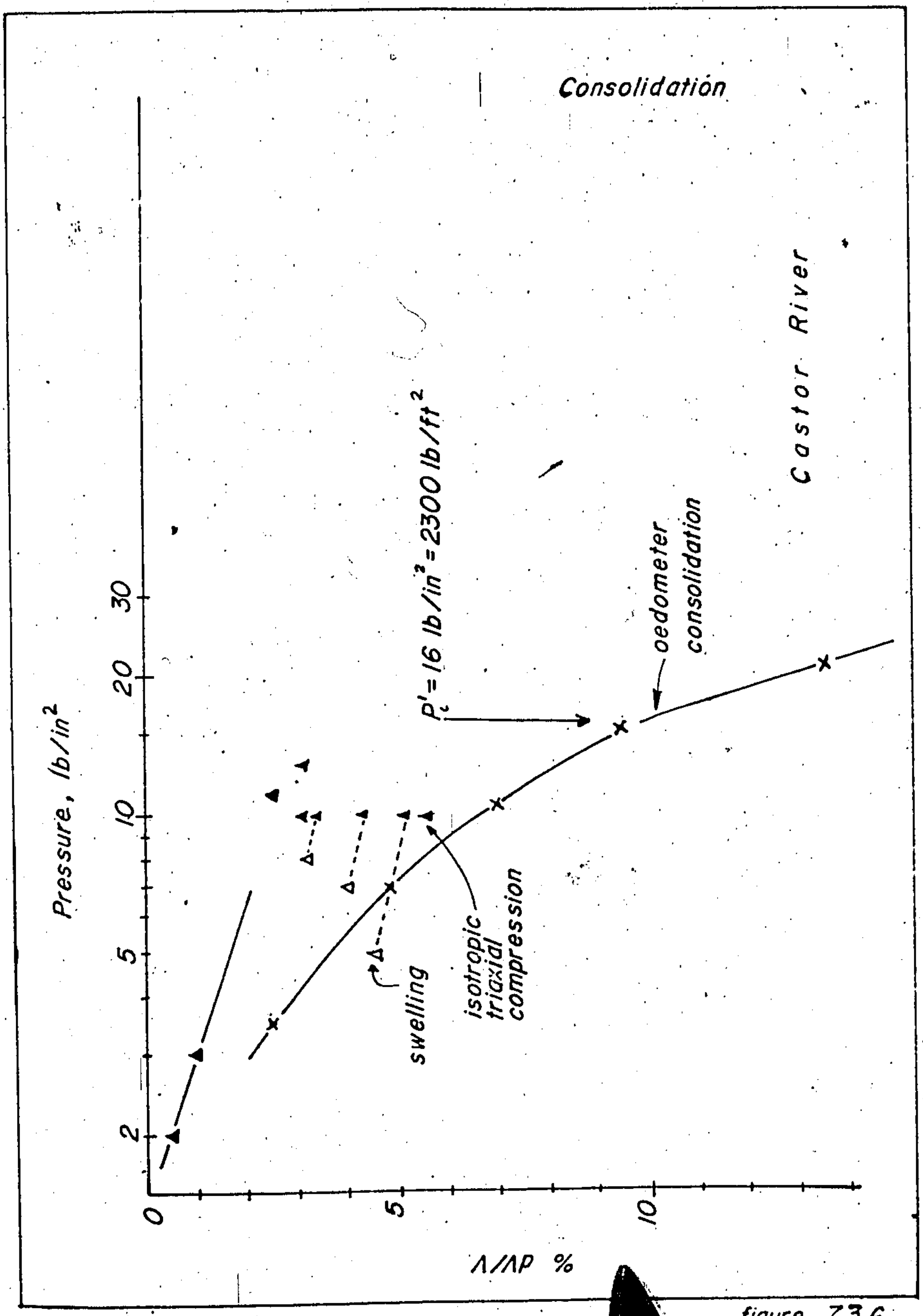

Consolidation
Castor River
P'_c = 16 lb/in² = 2300 lb/ft²
oedometer consolidation
isotropic triaxial compression
swelling
Pressure, lb/in²

figure 7.3.6

Consolidation

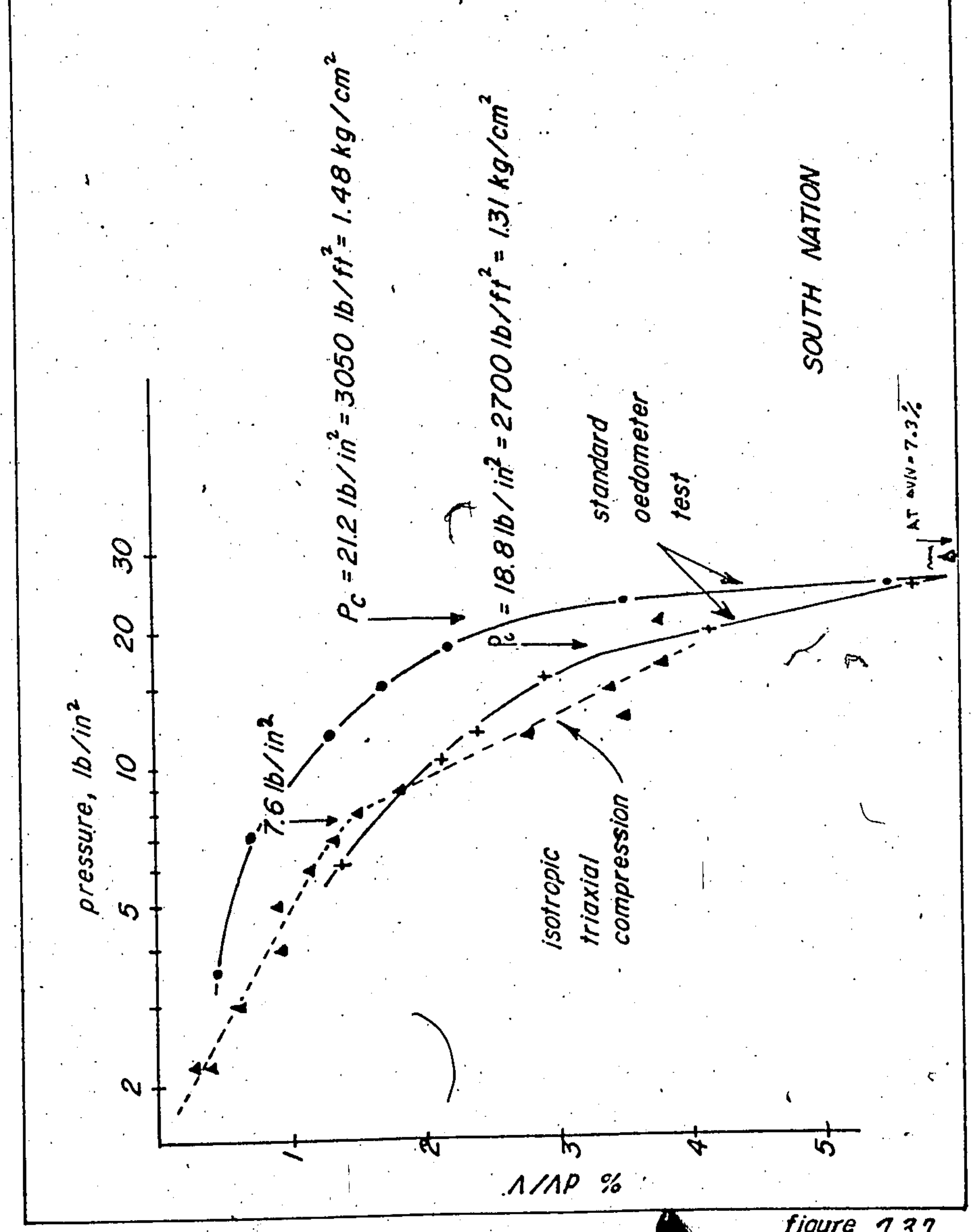


figure 7.3.7

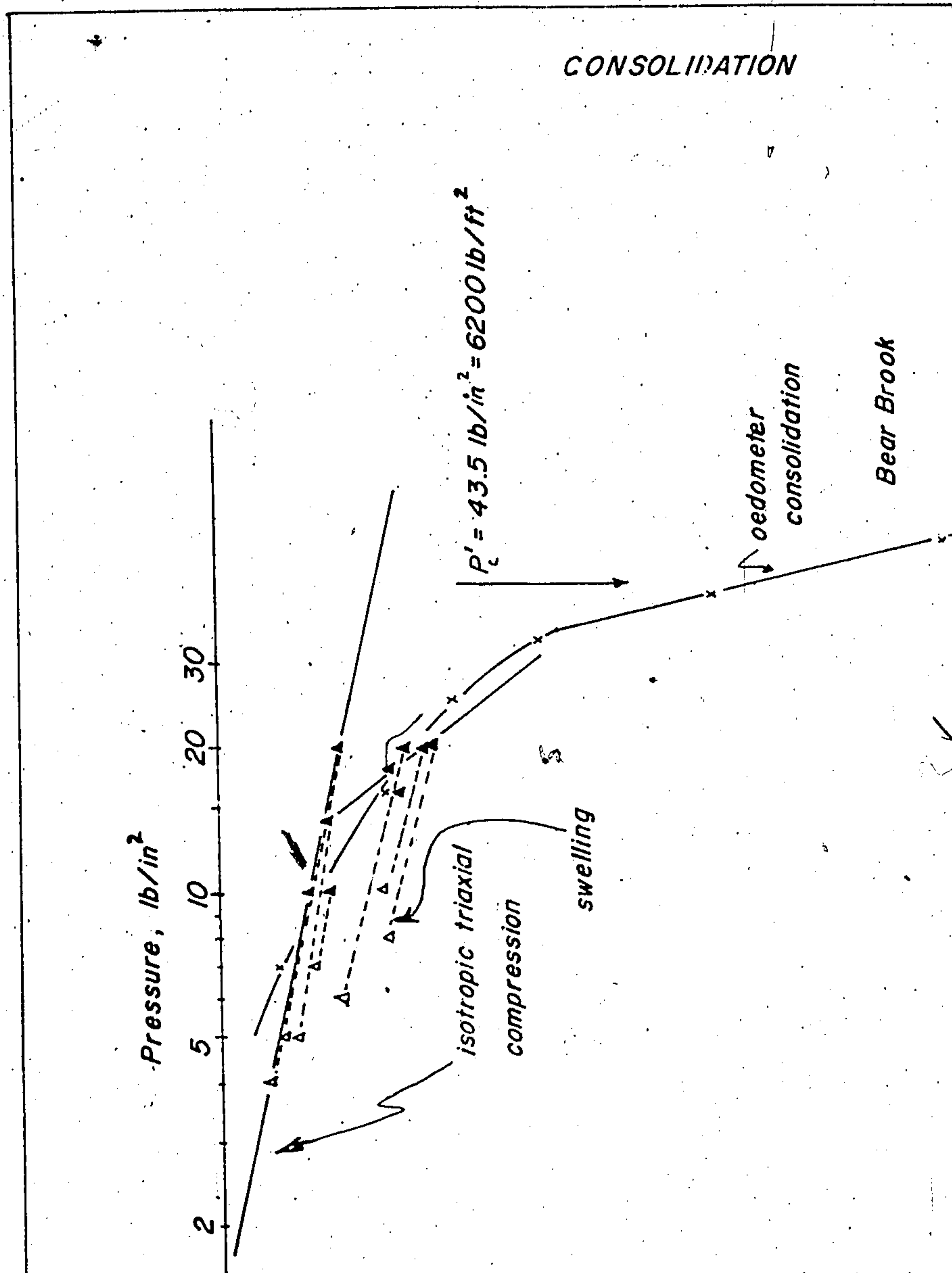

CONSOLIDATION
P'_c = 43.5 lb/in² = 6200 lb/ft²
oedometer consolidation
Bear Brook
swelling
isotropic triaxial compression
Pressure, lb/in²
2
5
10
20
30
0
5
10
Λ/ΛP %

figure 7.3.8

then $\frac{\sigma_1 + 2\sigma_3'}{3} = 7.6$ or $\sigma_1' + 2\sigma_3' = 22.8$ lb/in^2

if $\sigma_1 \approx 20$, then $\sigma_3' \approx 1.4$ lb/in^2 in the consolidation tests

then K, the coefficient of lateral pressure in the consolidation tests would be equal to 1.4/20 or .07 ≈ 0.1. This value for K seems unrealistically low.

It is possible that because of the anisotropic nature of Leda clay, the resistance to deformation under horizontal stress is less than resistance of deformation under vertical stress, there is a yield at σ_3' considerably less than σ_1'.

On the other hand, this break in the compression curve may be partially artificial, being caused by test procedure. Lawrence (1973) noted that "for a possibly cemented soil a macroscopically uniform hydrostatic confining pressure may well destroy some of the bonds due to the uneven microscopic application of pressure."

The curve composed of many different tests under triaxial isotropic compression at South Nation is approximately similar to that of oedometric compression having a P_c = 18.8 lb/in^2.

Similar curves for the Castor River are shown in Fig. 7.3.6. There is no apparent break in the curve as there was at South Nation for the log of all-around consolidation pressure vs. %dV/V. Insufficient data below $P'_m = 10\ lb/in^2$ is not available as many samples were consolidated above P'_m to $10\ lb/in^2$ and permitted to rebound or swell back to P'_m. It is of interest to note that these lines of swelling are approximately parallel to the initial part of the curve. There is a great variation in %dV/V at P'_m of $10\ lb/in^2$ for Castor River. It would appear that the bond strength of the clay at Castor River is more influenced by all around confining pressures at $10\ lb/in^2$ than South Nation. It is difficult to say whether or not the oedometer consolidation curve ($P_c = 16\ lb/in^2$) has anything in common in terms of %dV/V at a given pressure with the isotropic compression curve for all three sites.

Similarly for Bear Brook, (Fig. 7.3.8), the slopes of swelling were parallel to the initial slope of the log pressure ($\sigma'_1 = \sigma'_2 = \sigma'_3$) vs. %dV/V for isotropic compression. Apart from the sample consolidated to $21\ lb/in^2$, the isotropic triaxial compression curve approximates the oedometer consolidation curve ($P'_c = 43.5\ lb/in^2$).

On the basis of limited data, there appears to be a break in the isotropic triaxial compression curve at about 14 lb/in^2, though this is open to question.

In summation, insufficient data is available from the Castor River to estimate accurately a beginning of yield under isotropic consolidation.

On the basis of only a small amount of data, the ratios of vane to P'_m at the beginning of yield for South Nation and Bear Brook are 0.99 and 0.85 respectively (Table 7.3.1). An empirical yielding criteria of P'_m not greater than the vane strength may be reasonable. It should be realized that P'_m is an equal all round pressure and is not equivalent to a vertical stress in the field for foundation design.

TABLE 7.3.1 Beginning of Yield Under Isotropic Consolidation

	Beginning of Yield P'_m lb/in^2	Vane Strength lb/in^2	$\frac{\text{Vane}}{P'_m \text{ Yield}}$
Castor River	<10	5.5	-
South Nation	7.6	7.7	0.99
Bear Brook	14	16.5	0.85

7.4 Volume Change From Shear

The volume change from shear are summarized below.

Castor River. Fig. 7.4.1 $(\sigma_1' - \sigma_3')/2$ vs. $\%\Delta V/V$

i) For P_m' of 2.0, 3.0, and 5.0 lb/in^2, the samples compressed then dilated during shear.

ii) For P_m' of 7, 8, 10, 11 lb/in^2 the samples compressed progressively during shear ranging from 1/2 to 2% change in volume.

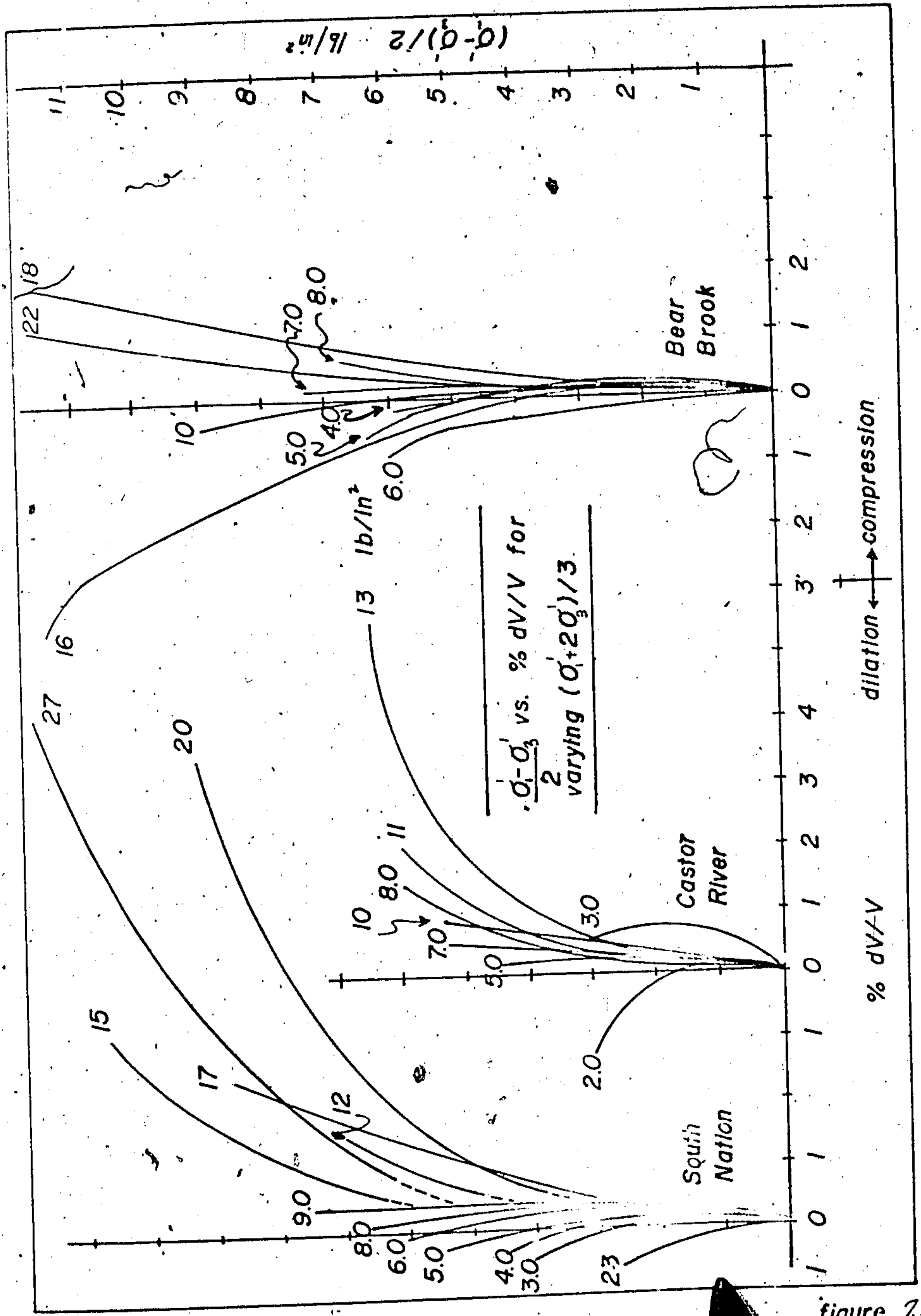
iii) For P_m' of 13 lb/in^2, the sample compressed progressively to 6% during shear.

South Nation. Fig. 7.4.1 $(\sigma_1' - \sigma_3')/2$ vs. $\%\Delta V/V$

i) From 2.3 to 9.0 lb/in^2 P_m' the curves were in exact sequence. In the early stages of shear $\%\Delta V/V$ was compressive, then as failure approached, the samples began to dilate.

ii) For P_m' of 12, 15, 17 lb/in^2 the samples underwent progressive compression to about 2% change in volume.

iii) For P_m' of 20, 27 lb/in^2 the samples underwent about 7 to 8% change in volume.



$(\sigma_1' - \sigma_3')/2$ lb/in²
$\frac{\sigma_1' - \sigma_3'}{2}$ vs. % dV/V for varying $(\sigma_1' + 2\sigma_3')/3$
lb/in²
Bear Brook
Castor River
South Nation
% dV/V
dilation
compression

figure 7.1.1.

<u>Bear Brook</u>. Fig. 7.4.1 $(\sigma_1' - \sigma_3')/2$ vs. $\%\Delta V/V$

The Bear Brook samples are more difficult to interpret, although they do tend to present a pattern.

i) For samples of $P_m' = 4, 5, 6, 7, 10, 16\ lb/in^2$ the samples compressed initially, then dilated during shear.

ii) Samples of P_m' equal to 8, 18, and 22 lb/in^2 compressed progressively during shear, in the order of 1 to 2% change in volume.

iii) No samples tested in this range.

Plotted in Fig. 7.4.2 are the curves from each site representing volume change at failure as a function of effective normal stress. The three sites, in order of increasing bond strength are Castor River, South Nation, and Bear Brook. The curves for Castor River and South Nation are much better defined than those for Bear Brook.

At failure, volume changes are dilitant below normal stresses of 4.5, 6 and approximately 8 lb/in^2 for Castor River, South Nation and Bear Brook respectively. This figure also shows that the greater the bond strength of the clay, the less the percentage volume decrease or compression at failure in the higher normal stress ranges and

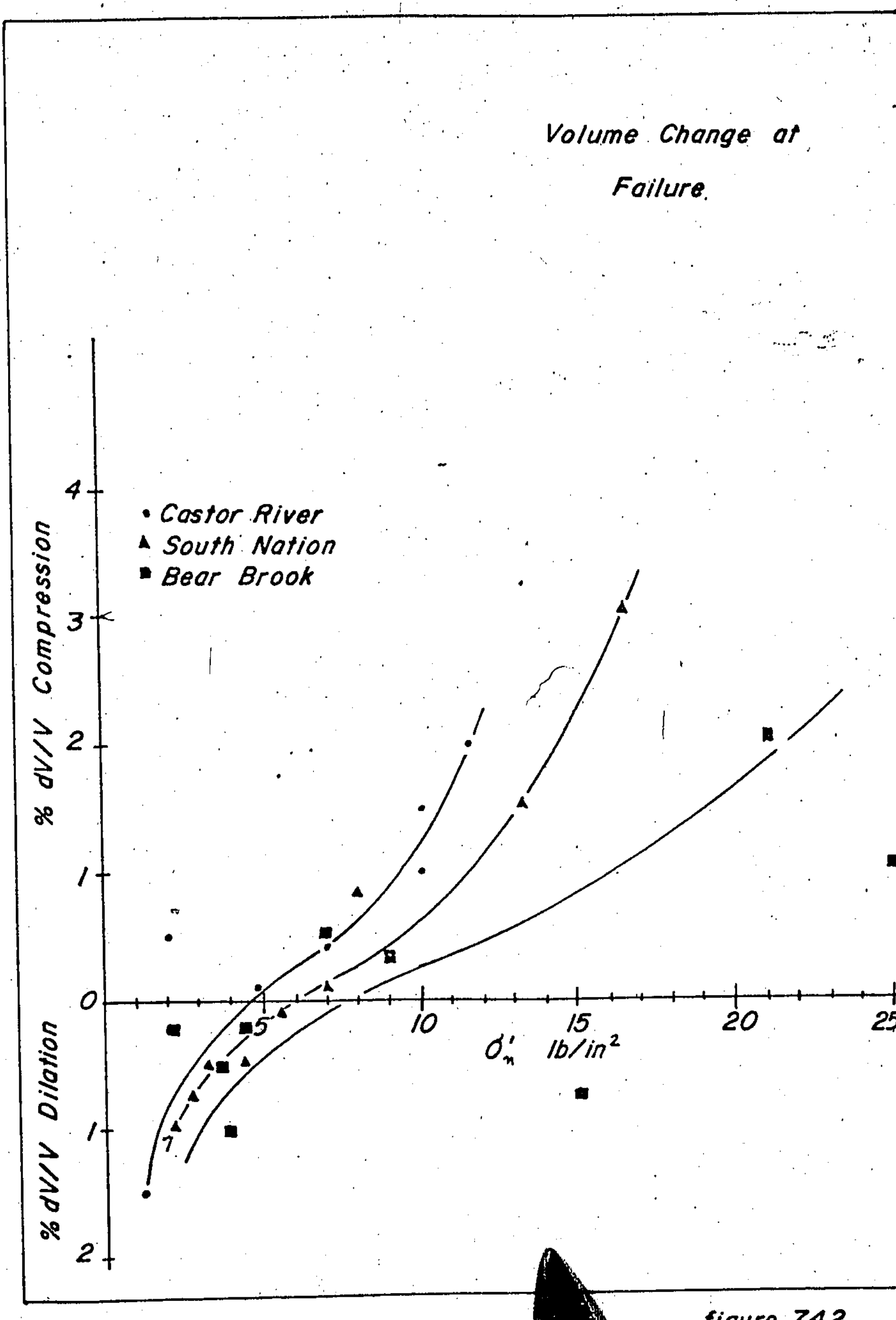

Volume Change at
Failure.
• Castor River
▲ South Nation
■ Bear Brook
% dV/V Compression
% dV/V Dilation
4
3
2
1
0
1
2
5
10
15
20
25
σ'ₙ lb/in²

figure 7.4.2

the greater the dilation or percentage volume increase at failure in lower normal stress ranges.

At Castor River, during shear, samples tested at normal effective stresses below 4-1/2 lb/in^2 undergo dilation at failure. Slope stability analyses at Castor River have shown the maximum effective normal stress on critical failure arcs to be below 4 lb/in^2 with an average value of approximately 2 lb/in^2 (Scott, 1973). Thus negative porewater pressures would be built up during shear unless there was sufficient drainage and time to satisfy dilation.

The greater tendency for dilation at failure for South Nation and Bear Brook clays would cause even more negative porewater pressures at these sites during undrained shear.

7.5 Strain from Shear

Following is a summary of axial strain during shear for the three clays tested.

South Nation River. (Fig. 7.5.1) $(\sigma_1' - \sigma_3')/2$ vs. $\%\varepsilon$ axial

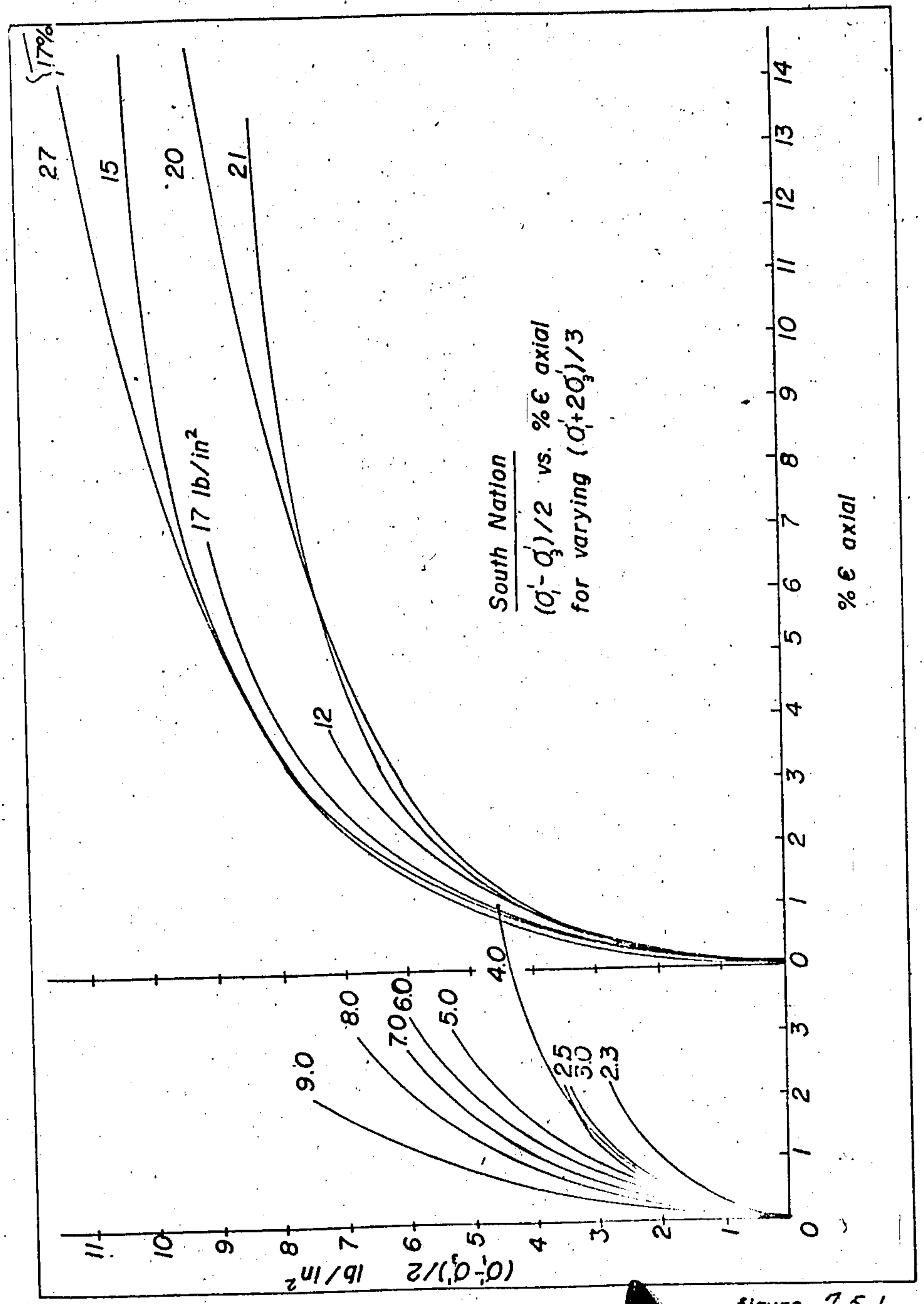

South Nation
$(\sigma_1' - \sigma_3')/2$ vs. % ϵ axial
for varying $(\sigma_1' + 2\sigma_3')/3$
17 lb/in²
% ϵ axial
$(\sigma_1' - \sigma_3')/2$ lb/in²
17%

figure 7.5.1

i) Strain from 2.0 to 3.0 % for P'_m from 2.3 to 9.0 lb/in^2. Curves in nearly consecutive sequence.

ii) Strain was 4 and 7% for P'_m of 12 and 17 lb/in^2, respectively.

iii) Strain from 12 to 17% for P'_m of 15, 20, 21, 27 lb/in^2.

<u>Castor River</u>. (Fig. 7.5.2) $(\sigma'_1 - \sigma'_3)/2$ vs. $\%\varepsilon$axial

i) Strain from 1-1/2 to 4-1/2% for P'_m of 2, 3, 5 lb/in^2.

ii) Strain from 4-1/2 to 6-1/2% for P'_m of 7, 8, 10, 11 lb/in^2.

iii) Strain of 10% for P'_m of 13 lb/in^2.

<u>Bear Brook</u>. (Fig. 7.5.2) $(\sigma'_1 - \sigma'_3)/2$ vs. $\%\varepsilon$axial

i) Strain from 2 to 3% for P'_m from 4 to 14 lb/in^2.

ii) Strain from 3 to 8% for P'_m of 6, 16, 18, 22 lb/in^2.

iii) No samples tested in this range.

Examination of Fig. 7.5.1 for South Nation indicates that between 9 and 12 lb/in^2 axial deformation increases markedly, and between 12 and 15 lb/in^2 it becomes plastic. Castor River, in Fig. 7.5.2 undergoes relatively large axial deformations at low values of P'_m. Bear Brook, a much stronger clay, undergoes relatively little axial deformation.

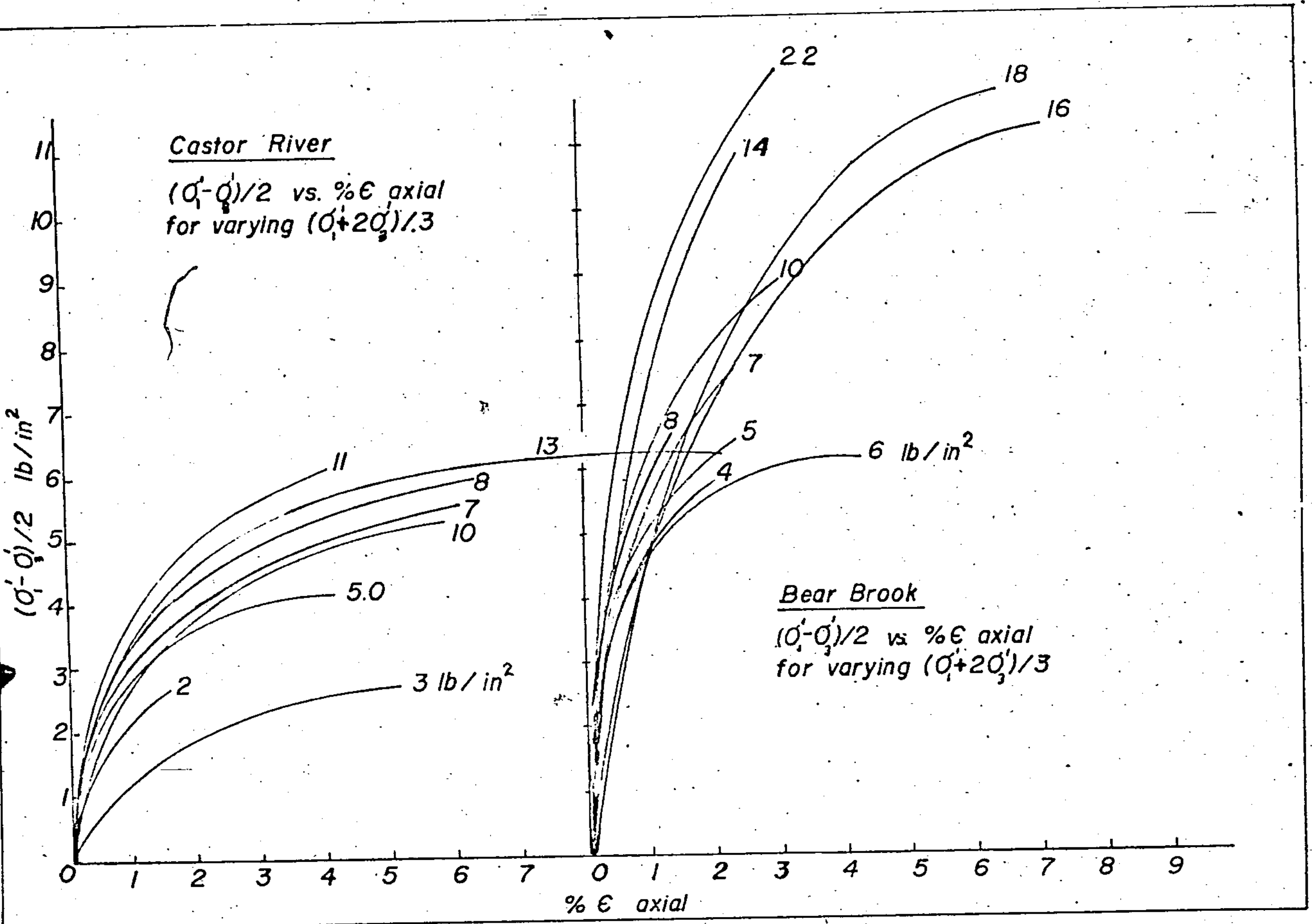

Castor River
$(\sigma_1' - \sigma_3')/2$ vs. $\%\epsilon$ axial
for varying $(\sigma_1' + 2\sigma_3')/3$
Bear Brook
$(\sigma_1' - \sigma_3')/2$ vs. $\%\epsilon$ axial
for varying $(\sigma_1' + 2\sigma_3')/3$
$(\sigma_1' - \sigma_3')/2$ lb / in^2
% ϵ axial
11
8
7
10
5.0
2
3 lb / in^2
13
22
14
18
16
10
7
8
5
4
6 lb / in^2
0
1
2
3
4
5
6
7
8
9
10
11

figure 7.5.2

Shown in Fig. 7.5.3 are plots of axial strain at failure vs. effective normal stress at failure for Castor River, South Nation, and Bear Brook.

It would appear that, for any normal stress, the greater the strength of the soil, the less axial strain at failure. South Nation values show less variation than those of Castor River, and particularly Bear Brook.

Insufficient data is available to postulate an explanation of the patterns of axial strain shown in Fig. 7.5.3. Axial strain is, of course, not only a function of the shear strain on a failure surface but also of the volume change in the sample. Both of these parameters however, are related to the normal stress.

Strain and Volume Change During Increment Loading

Since mean normal stress is held constant, measured volume change is due solely to the change in deviator stress. Volume changes due to changes in mean normal stress are eliminated.

Behaviour in shear in terms of measured volume change for $P'_m = 15\ lb/in^2$ appears to be considerably different than that at $P'_m = 5\ lb/in^2$. For the

Influence of Normal Stress

on

Axial Strain in Shear

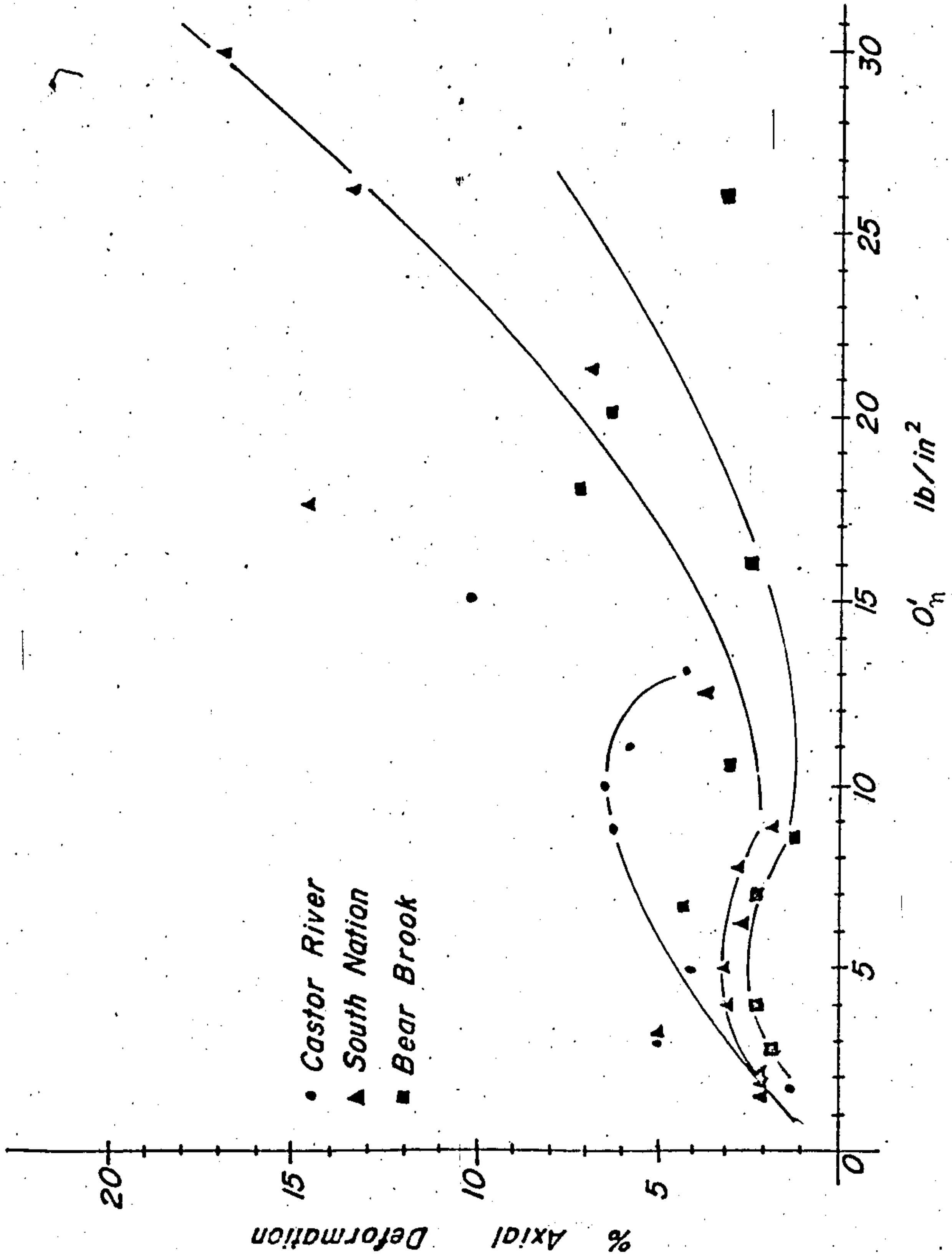


figure 7.5.3

South Nation clays, a P'_m of 5 lb/in^2 would be considered to lie in the low effective normal stress range and one of 15 lb/in^2 would be considered to lie in the intermediate effective normal stress range.

The deviatoric volume changes for P'_m of 5 lb/in^2 (Fig. 7.5.4) for the critical load increments are compressive, then become increasingly dilitant as the deviator strength approaches failure. As the deviator stress increments approach ultimate strength, strain associated with each increment becomes progressively greater (Fig. 7.5.5). The deviatoric volume changes for P'_m of 15 lb/in^2 for all load increments are compressive (Fig. 7.5.6). As the deviator stress increments approach ultimate strength, both volume change and strain (Fig. 7.5.7) become progressively greater.

From the data in Figs. 7.5.4 to 7.5.7, it would appear that there is a noticeable change in behaviour when the ratio of the applied deviator stress to the ultimate deviator strength at failure, $\underline{q}/\underline{q}_f$, approaches 0.92 and 0.85 for P'_m of 5 and 15 lb/in^2 respectively for South Nation fissured clay.

Estimated time to full primary consolidation or dilation from volume change during shear is plotted

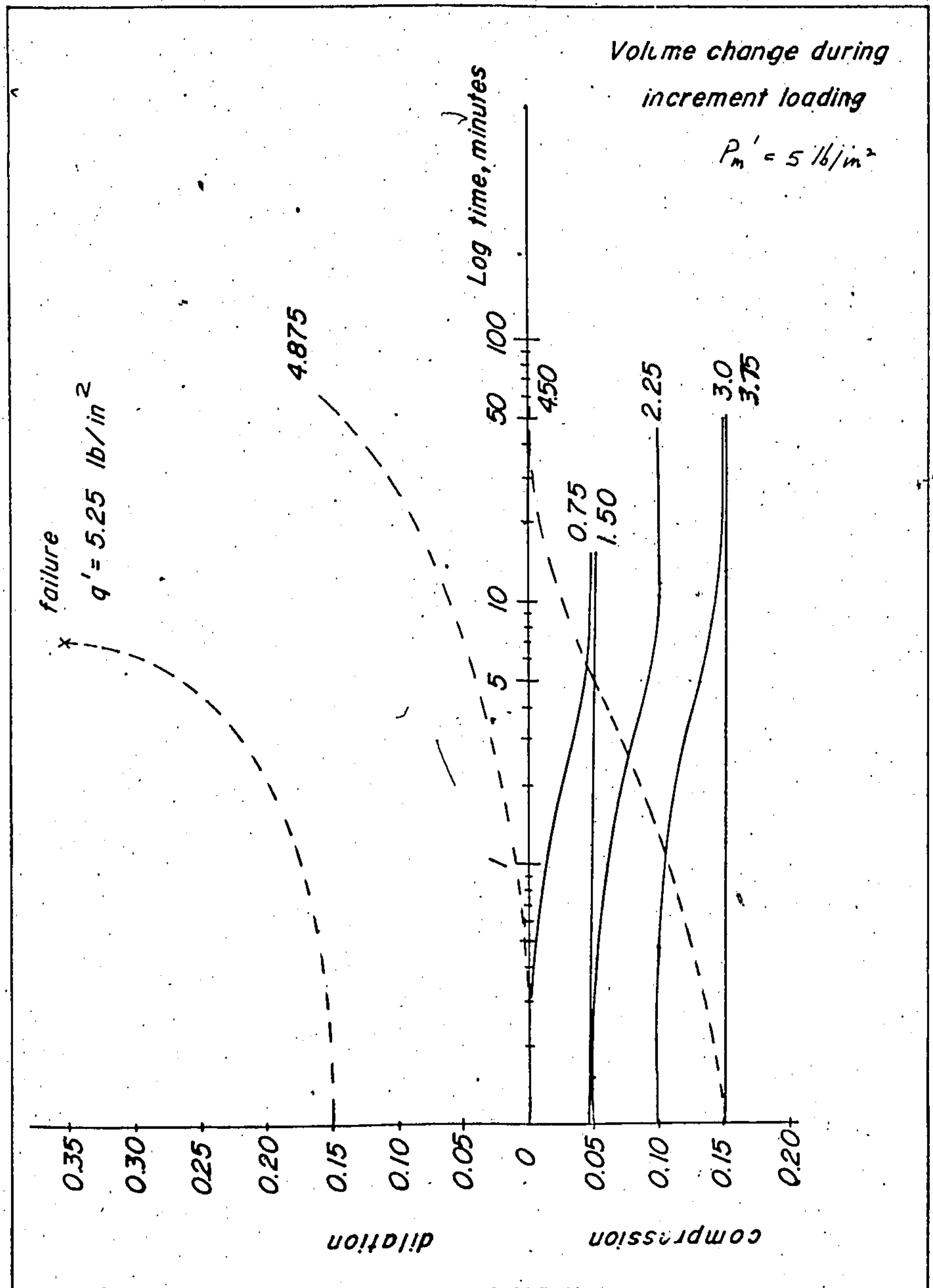

Volume change during
increment loading
Pm' = 5 lb/in²
Log time, minutes
100
50
10
5
1
4.875
4.50
0.75
1.50
2.25
3.0
3.75
failure
q' = 5.25 lb/in²
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0
0.05
0.10
0.15
0.20
dilation
compression
dV/Vp %

figure 7.5.4

Strain during increment loading

$P_m' = 5$ lb/in²

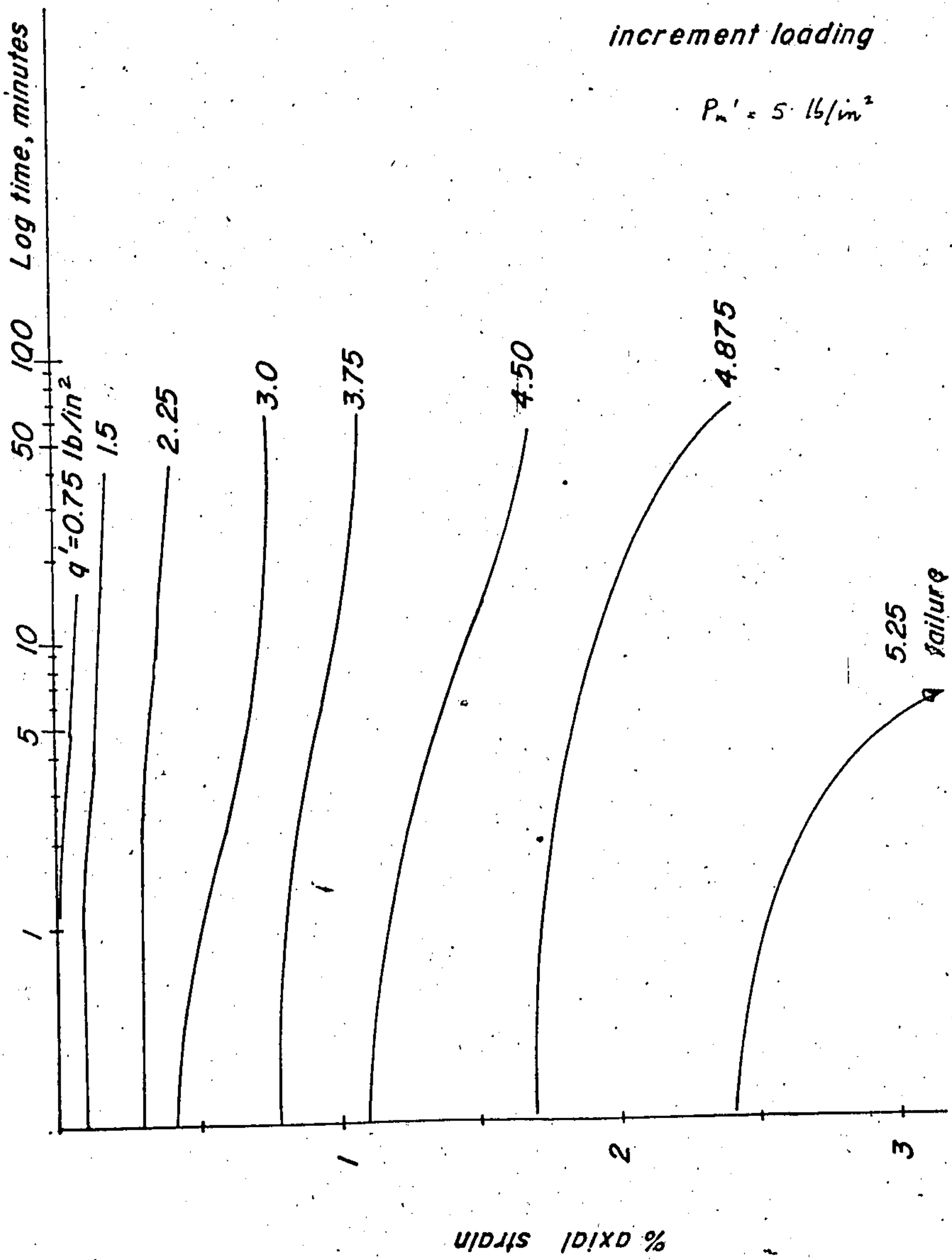


figure 7.55

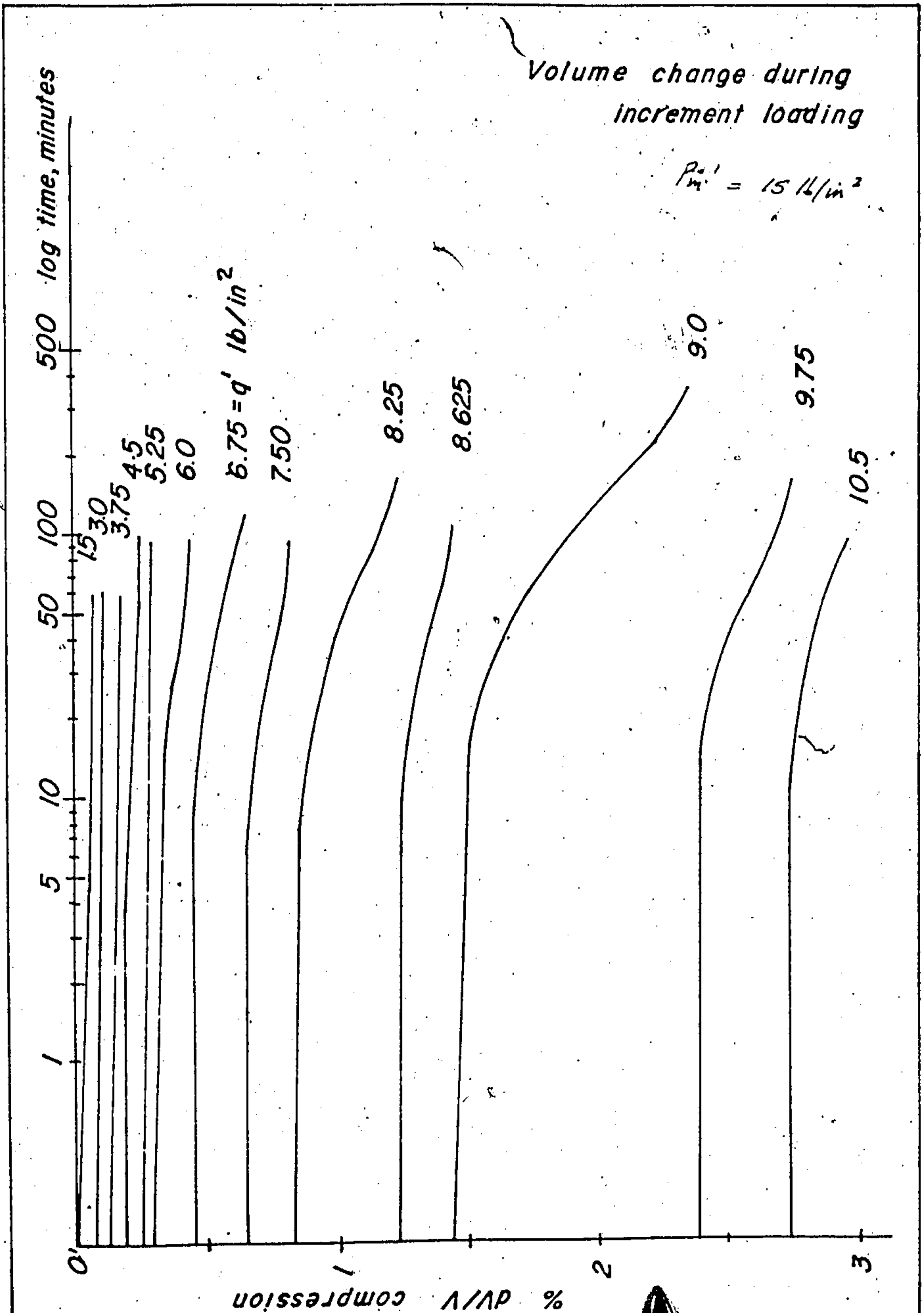

Volume change during increment loading
$p_m' = 15$ lb/in²
log time, minutes
500
100
50
10
5
1
1.5
3.0
3.75
4.5
5.25
6.0
6.75 = q' lb/in²
7.50
8.25
8.625
9.0
9.75
10.5
0
1
2
3
% dV/V compression

figure 7.5.6

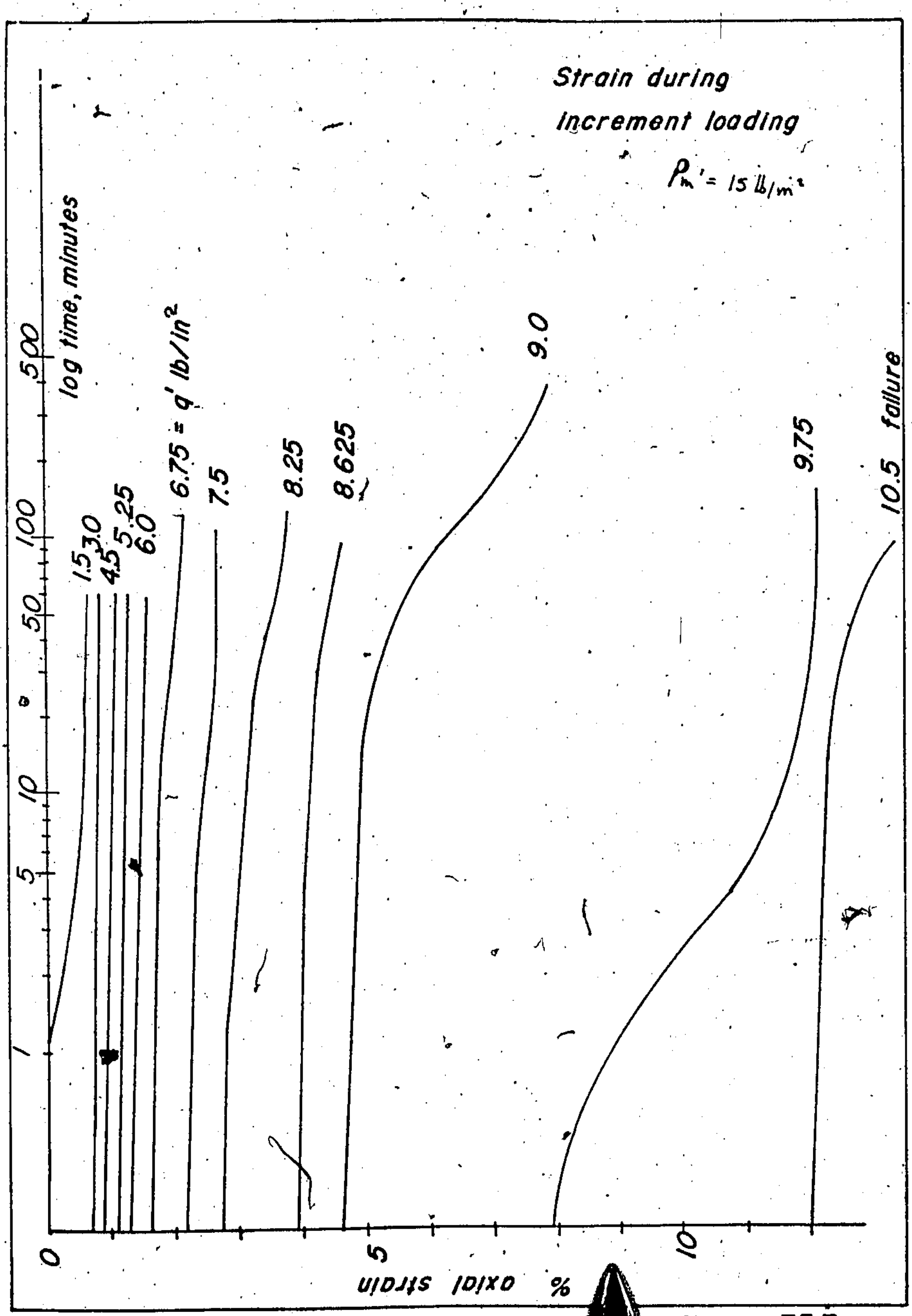

Strain during
increment loading
$p_m' = 15$ lb/in²
log time, minutes
500
100
50
10
5
1
6.75 = q' lb/in²
1.5
3.0
4.5
5.25
6.0
7.5
8.25
8.625
9.0
9.75
10.5 failure
0
5
10
% axial strain

figure 7.5.7

against the ratio of q/q_f for the P'_m tests of 5 and 15 lb/in^2 (Fig. 7.5.8). These South Nation samples have void ratios of 1.09 and 1.06 respectively.

Estimates of time to full primary consolidation are possible for P'_m of 15 lb/in^2 up to about q/q_f = 0.85. Beyond this ratio, volume change (consolidation) during shear appears to be of a plastic nature. Also plotted are points showing estimates of time to full primary consolidation or dilation for P'_m of 5 lb/in^2. Beyond q/q_f = 0.92 volume change appears to be increasingly plastic and not possible to estimate in terms of primary dissipation of excess pore water pressures. However, trends are indicated in Fig. 7.5.8 for these two tests by the two curves shown.

It would appear that the greater the value of P'_m during shear, the greater the time required to satisfy porewater porewater pressures during shear. Estimation of strength in the field during shear may require consideration of porewater pressures generated by shear as well as those due to the position of the ground water table.

A considerably greater time is required for dissipation of shear induced porewater pressures for clays

Dissipation of porewater pressures in shear

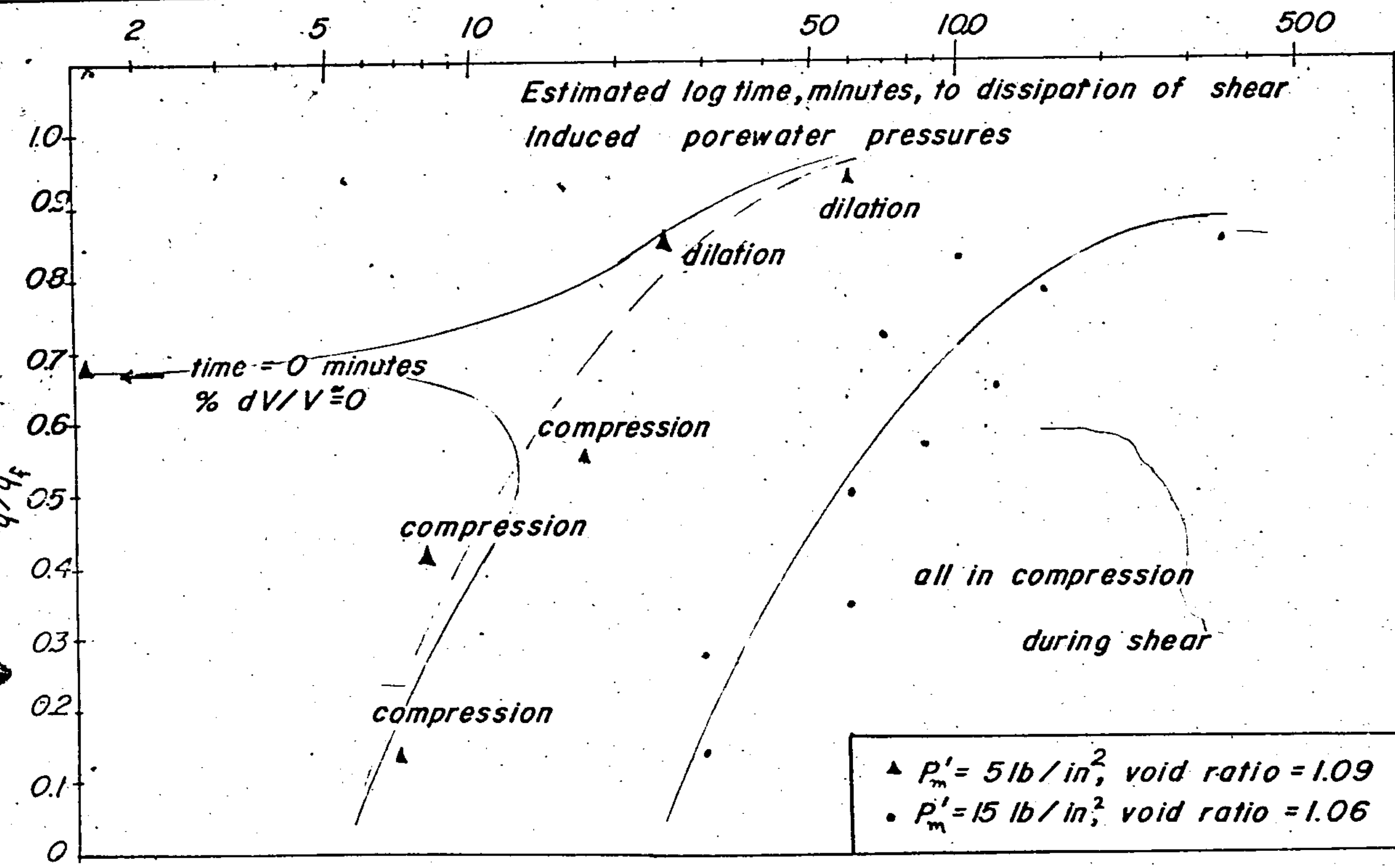


figure 7.5.8

sheared under higher effective normal stresses. It is possible that porewater pressure dissipation due to shear in the lower effective normal stress regions is more rapid due to drainage along fissures.

As shear induced porewater pressures in the lower stress ranges are negative, failure to consider them along with pressures due to ground water may be slightly conservative, although it appears that these negative porewater pressures due to shear are fairly quickly satisfied. On the other hand, dissipation of positive porewater pressures at higher effective normal stresses appears to be considerably slower. In the field, failure to consider these positive shear induced porewater pressures along with those induced by ground water may be unsafe.

It would appear that effective stress analysis of in situ shear strength would be incomplete without considering positive and negative porewater pressures developed during shear, the time available for dissipation of these pore pressures, and the associated effective normal stress ranges.

7.6 Yield from Shear Deformation

Presented in Figs. 7.6.1 to 7.6.3 are plots showing contours of axial strain. Examination of the contour spacing shows that a plastic yield under axial deformation begins at approximately 3% strain for all three clays.

Failure during shear is abrupt in the low normal stress region as shown by the contour spacing with little or no plastic yielding occurring until failure is imminent. In the intermediate stress region failure is preceded by an increasing amount of yield. In the high stress region yield is achieved relatively early in the test. Excessive strain accompanies each local increment until failure after yielding has been reached.

A yield curve for shear deformations is drawn at approximately 3% axial strain in the p'-q' space of Figs. 7.6.1 to 7.6.3. These 3% yield curves appear to indicate at what combination of stresses bond breakdown from shear deformation may begin. Trends shown by these curves appear to indicate that during shear the stronger the clay the less the strain at failure for any given low effective normal stress range. Also under the same conditions the stronger clays tend to exhibit more of a brittle than a plastic failure in strain during shear. This would seem to indicate that the clays having the greater strength are more strongly bonded.

Contours of Equal Axial Strain

CASTOR RIVER

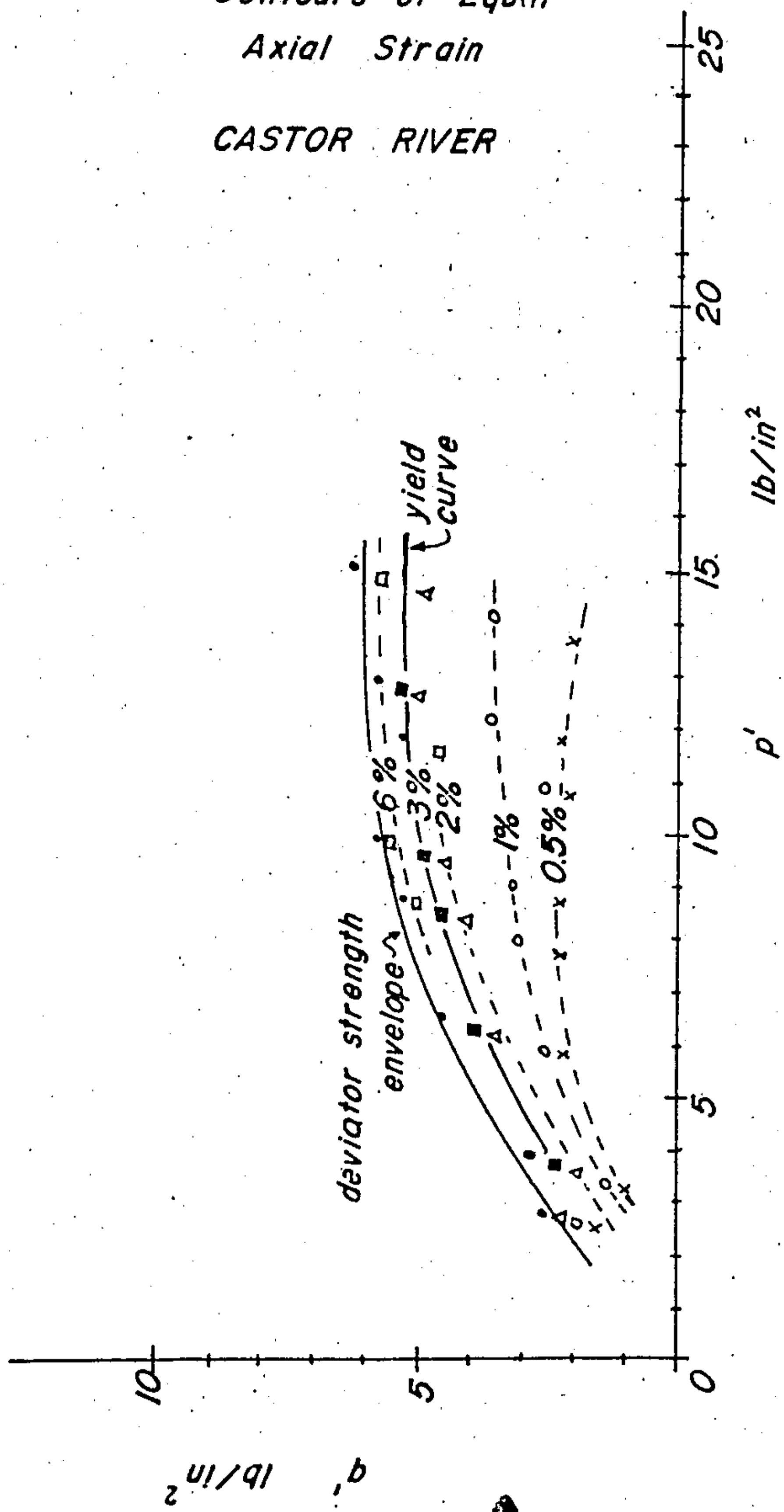


figure 7.6.1

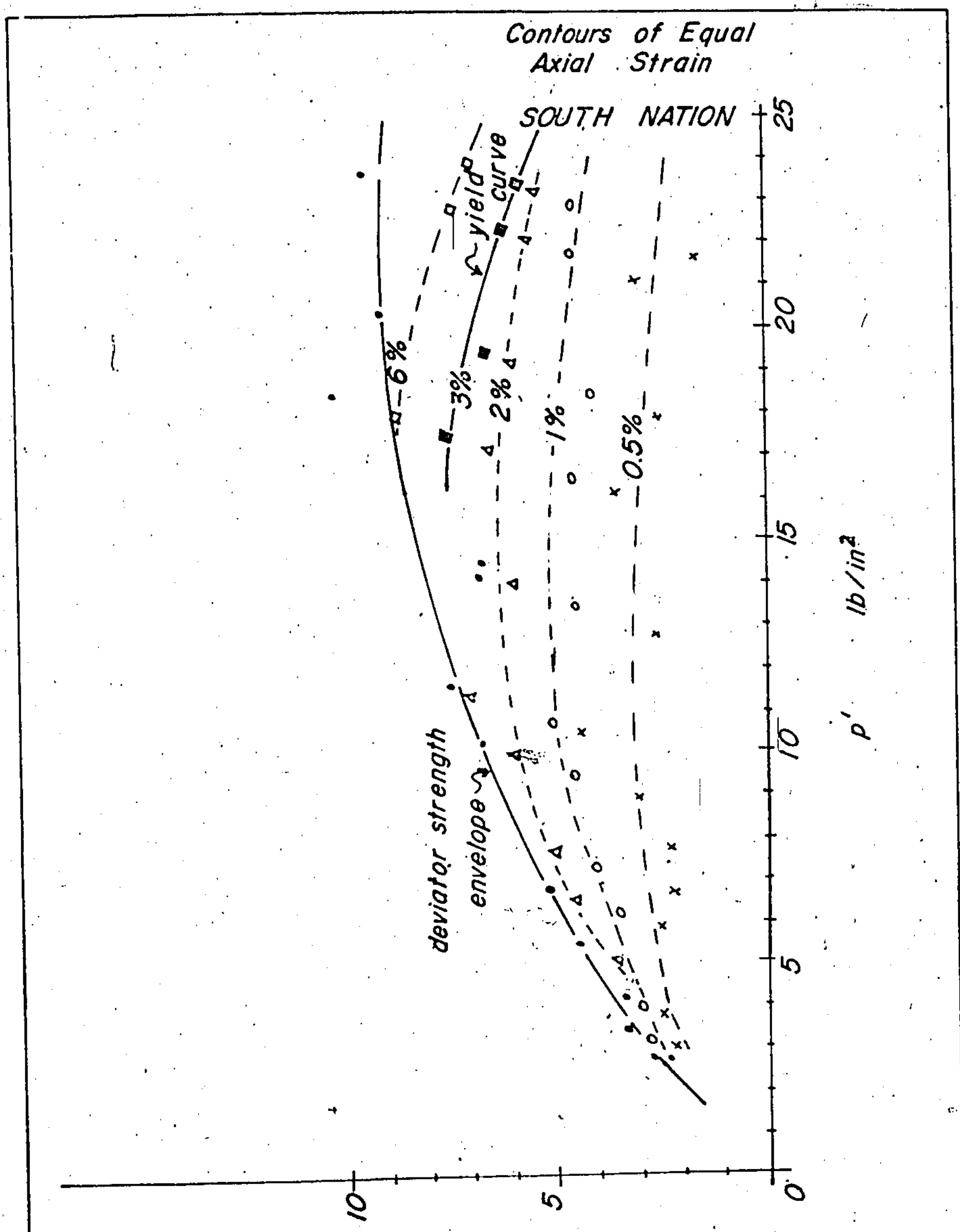
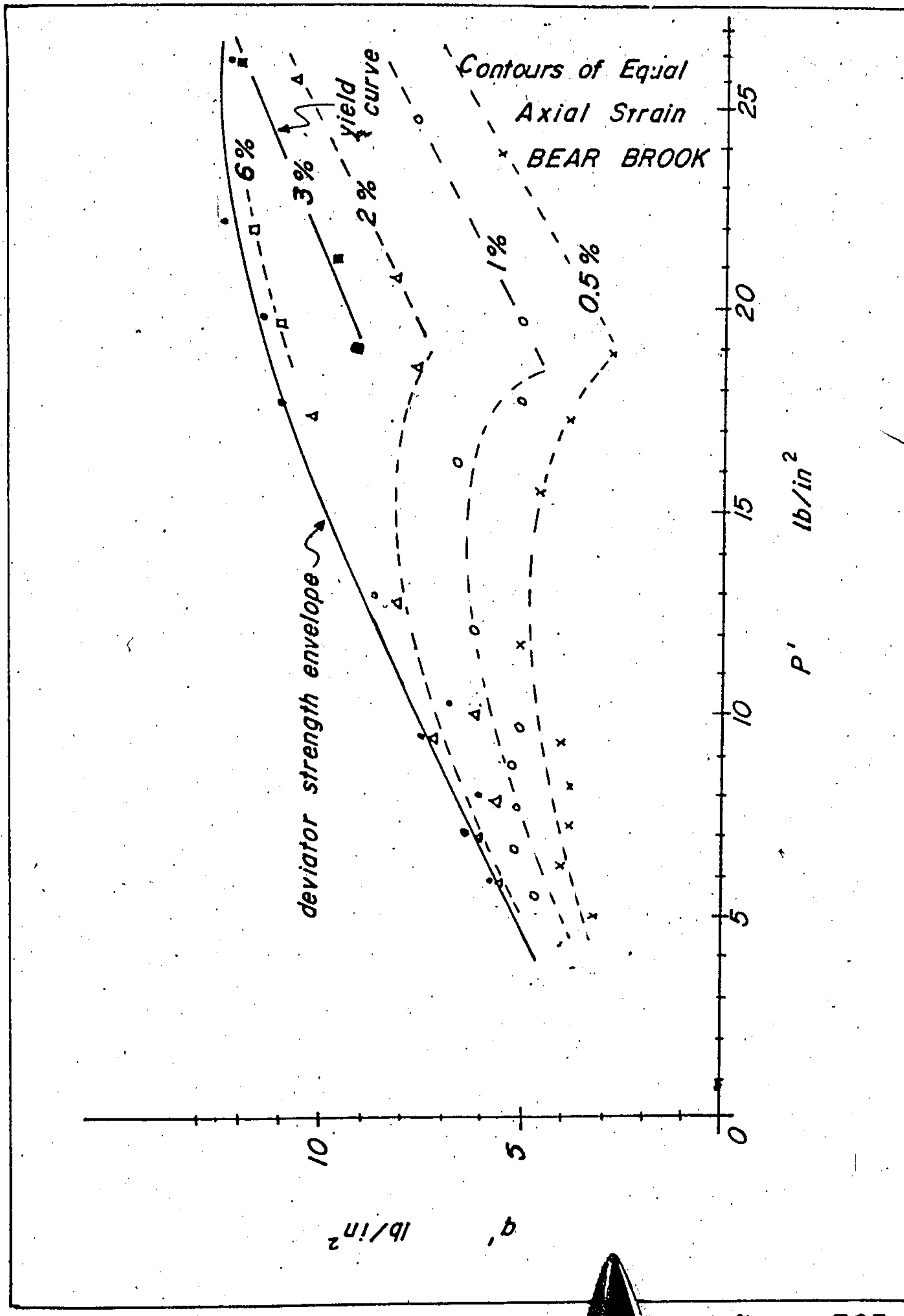

Contours of Equal
Axial Strain
SOUTH NATION
yield curve
deviator strength envelope
6%
3%
2%
1%
0.5%
0
5
10
15
20
25
p'
lb/in²
0
5
10
q'
lb/in²

figure 7.6.2



Contours of Equal
Axial Strain
BEAR BROOK
yield curve
6%
3%
2%
1%
0.5%
deviator strength envelope
0
5
10
15
20
25
P'
lb/in^2
5
10
q'
lb/in^2

figure 7.6.3

Mitchell (1970), although his yield criteria are based on volumetric and distortional strain, notes that the "failure envelope appears to be independent of stress path." It would appear that any portion of these deviator strength envelopes within the yield envelopes would be reproducible by any triaxial stress path.

The yield curves or envelopes may indicate at what combination of stresses a sample will begin to undergo bond breakdown by shear deformations no matter what stress path the shear test follows.

7.7 Mode of Failure and Effective Normal Stress

Mode of Failure

The mode of failure was found to be dependent upon the effective normal stress. There appeared to be a pattern evident in the three series tested. 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.

Typical of shear in a fissured clay is the formation of a nodular or stepped failure surface as shown for some modes of failure of a fissured clay in Fig. 7.7.1.

Failure of Fissured Clays

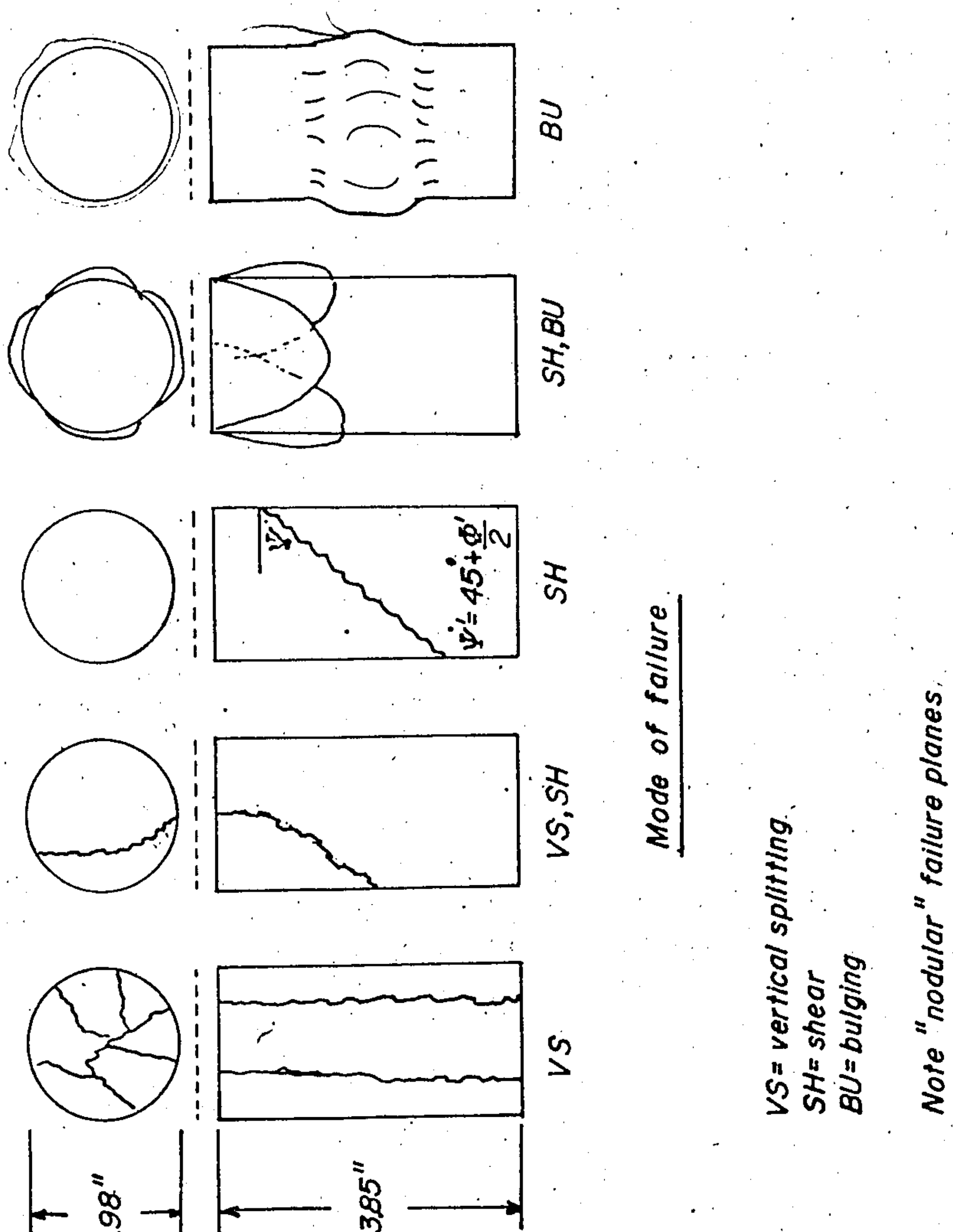


figure 7.7.1

In contrast, the shear plane of Hull intact clay is very sharp, as if cleanly cut with a knife (Fig. 7.7.2).

Contained in Table 7.7.1 for each triaxial test is the mode of failure (as drawn in Fig. 7.7.1), $p' = \frac{\sigma'_1+\sigma'_3}{2}$, and effective normal stress, σ'_n.

Measured and Calculated ϕ' Angles

Listed in Table 7.7.1 is the computed (where possible) angle of shear by the relationship

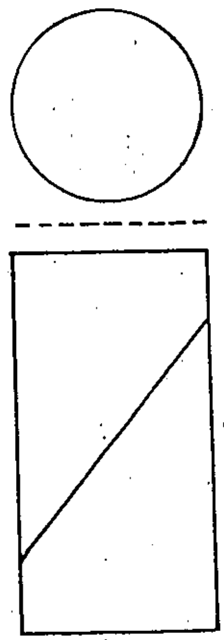
$$\psi = 45^{\circ} + \phi'/2 \qquad (7.7.1)$$

where ψ' = angle from horizontal for the shear plane (Fig. 7.7.1). This ϕ' is compared to the ϕ' measured from the shear strength envelope in terms of effective normal stress for each of the three sites (Figs. 7.7.3 to 7.7.5).

It would appear that the Mohr-Coulomb theory does not adequately describe the position of the failure plane in triaxial laboratory testing, although it does seem adequate to describe the shear strength of the soil in the field. Eden and Mitchell (1973) have found that a ϕ' ranging from 30° to 40° in the low-normal stress ranges is applicable to landsliding in fissured Ottawa area clays and this range of ϕ' with that obtained by the Mohr-Coulomb theory.

Lawrence (1969) reported that for low normal strength testing of a Leda clay, he found that ϕ',

Shear Failure
of an
Intact Clay



Hull clay

The failure plane is smooth, as shown

figure 7.7.2

TABLE 7.7.1 Mode of Failure and Measured ϕ'

Test No.	Location	P' lb/in^2	σ'_n lb/in^2	Mode of* Failure	ϕ' shear degrees
17	South Nation	3.30	1.70	VS,SH	30-1/2°
16	"	3.63	2.0	VS,SH	39°
15	"	4.13	2.4	VS,SH	5
14	"	5.50	3.4	VS,SH	28
13	"	6.75	4.5	VS,SH	16
12	"	7.94	5.0	SH	21
11	"	9.00	5.9	SH	15
9	"	10.25	8.0	SH	12
10	"	11.50	9.2	SH	21
24	"	14.25	-	SH,BU	-
20	"	14.25	-	SH,BU	-
18	"	18.50	-	SH,BU	-
25	"	18.00	-	SH,BU	-
21	"	21.13	-	SH,BU	-
22	"	23.75	-	SH,BU	-
23	"	27.25	-	SH,BU	-
44	Castor River	2.81	-	VS,SH	-
31	"	3.88	-	VS,SH	-
30	"	6.38	4.8	SH	31°
27	"	8.75	8.5	SH	30°
28	"	9.88	10.0	SH	15°
29	"	11.75	-	BU	-
32	"	12.88	-	SH,BU	-
33	"	15.13	-	BU	-
37	Bear Brook	5.87	-	VS	-
40	"	7.13	-	VS,SH	-
35	"	8.00	-	SH	13°
42	"	8.5	-	SH	17°
34	"	10.25	-	BU	-
36	"	12.88	-	BU	-
39	"	17.63	-	BU	-
43	"	19.75	-	BU	-
41	"	21.88	-	SH	-
38	"	26.00	-	-	-

*VS - vertical splitting
SH - shear
BU - bulging

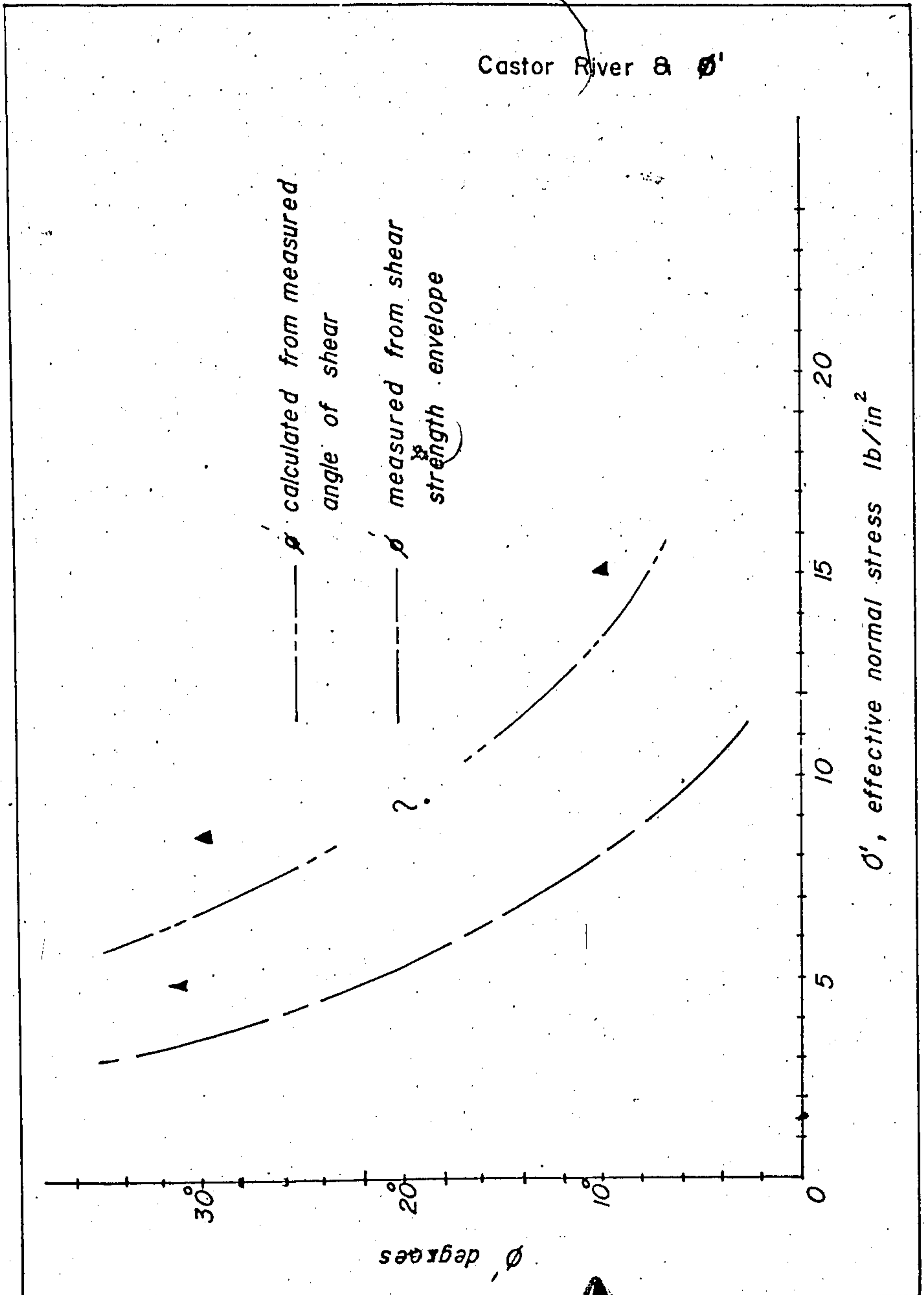

Castor River & ø'
ø' calculated from measured angle of shear
ø' measured from shear strength envelope
σ', effective normal stress lb/in²
ø' degrees
0
5
10
15
20
10°
20°
30°
?

figure 7.7.3.

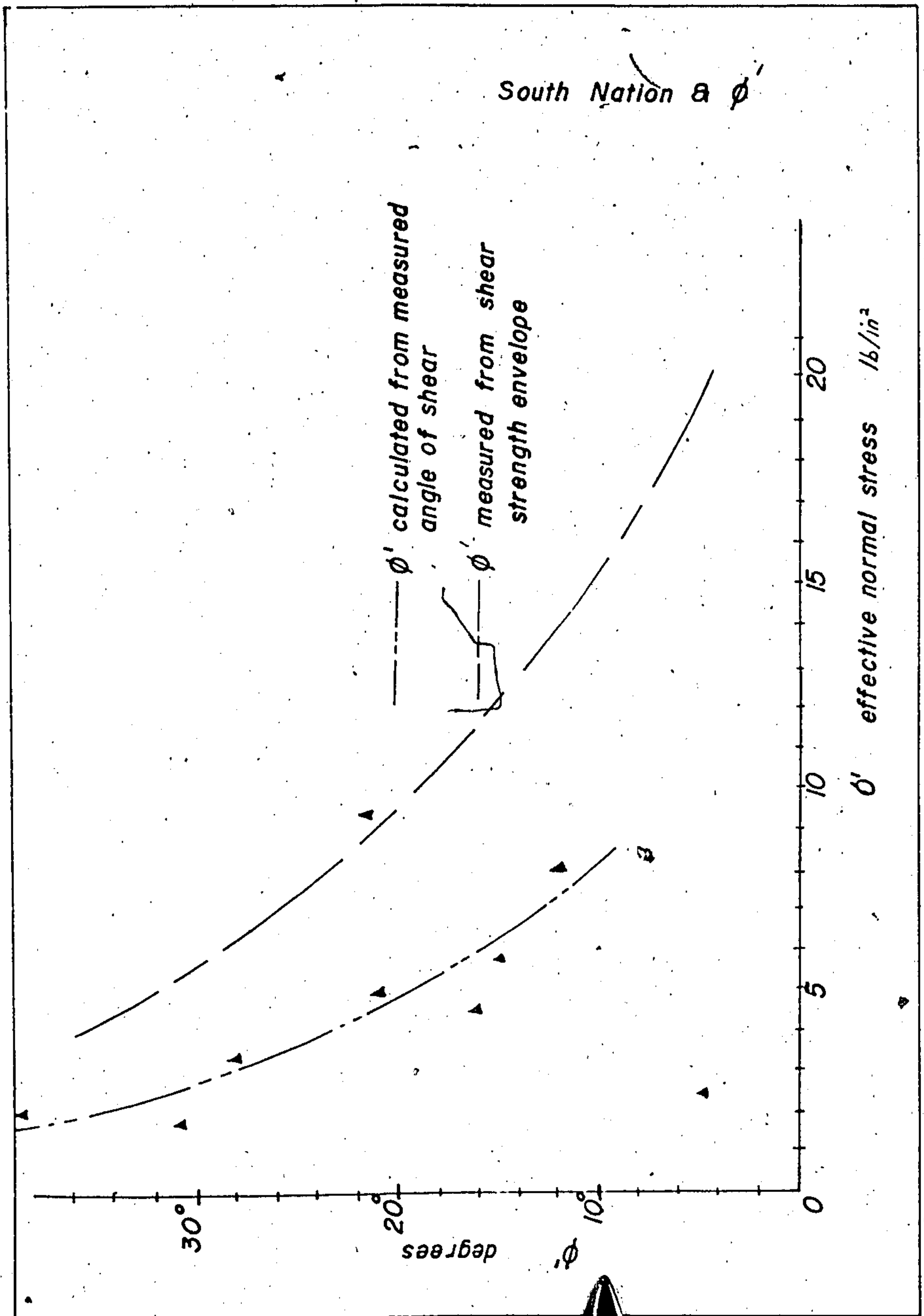

South Nation a φ'
φ' calculated from measured angle of shear
φ' measured from shear strength envelope
effective normal stress σ' lb/in²
φ' degrees
30°
20°
10°
0
5
10
15
20

figure 7.2.4

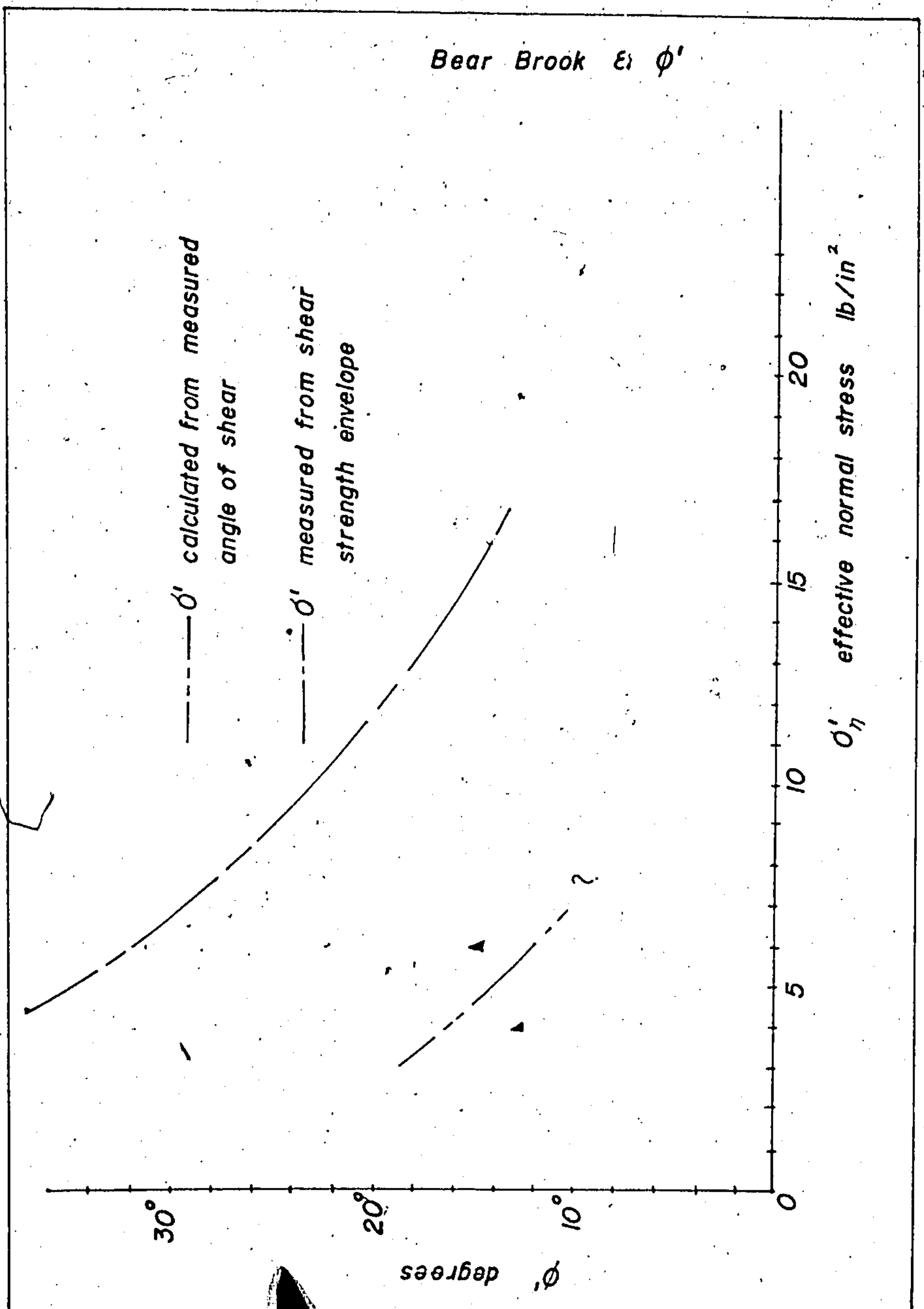

Bear Brook ε φ'
φ' calculated from measured angle of shear
φ' measured from shear strength envelope
0
5
10
15
20
σ'_n effective normal stress lb/in²
10°
20°
30°
φ' degrees

figure 7.7.5.

calculated from the angle of the failure plane from the horizontal, ranged from 10° to 30°. Examination of Fig. 7.6.4 at South Nation indicates that ϕ' (from the measured angle of shear) also ranges from 10° to 30°. Insufficient data is available from the Bear Brook and Castor River sites to draw any comparisons.

Values of ϕ' from measured shear plane do not agree with ϕ' from the Mohr envelope in terms of normal stress. This may be because the clay is non-homogeneous (fissured, and with silt seams) and anisotropic. Lawrence, referring to Jaegar, notes that compressive stress (i.e., cell pressure greater than zero) can alter this failure surface.

7.8 Summary of Test Results

On the basis of response during consolidation and shear as presented earlier in this Chapter, each of the three sites tested are subdivided into regions of low, intermediate and high effective normal stress (Table 7.8.1).

The characteristics during isotropic consolidation (volume change) and shear (volume change and

TABLE 7.8.1 NORMAL STRESS RANGES

Location	Low Normal Stress Range lb/in^2	Intermediate Normal Stress Range lb/in^2	High Normal Stress Range lb/in^2
Castor River	< 6	6-13	>13
South Nation	<12	12-20	>20
Bear Brook	<16	16-	-

and strain at failure) in the low, intermediate and high normal stress ranges for the three clays tested are shown in Table 7.8.2.. Also shown are the values of effective shear strength for these ranges.

Volume change in consolidation and shear, and strain in shear do not appear to show any trend in terms of strength of the clays. The only trend that is apparent is that associated with stress range. The possibility that each stress range has unique boundaries of volume change and strain independent of bond strength may be useful. Comparison of test results during shear and consolidation on any fissured clay of unknown strength to Table 7.8.2 (using a stress controlled constant P'_m stress path) may determine the stress range in which the sample has been tested. This information may not only be valuable in indicating relative strength, but also provides an indication of at what stress levels further tests should be performed to complete a strength envelope.

7.9 Shear Strength Envelopes

Shear strength in the high stress region is of "relatively little practical significance," ..... "the intermediate stress region is associated with loading

TABLE 7.8.2 SUMMARY OF TEST RESULTS BY STRESS RANGE

Property	Location	Stress Range Low	Intermediate	High
Isotropic Consolidation %ΔV/V	Castor River	<2	2-3	>4
	South Nation	<2	2-3-1/2	>3-1/2
	Bear Brook	<1-1/2	1-1/2-	-
Shear $(\%\Delta V/V)_f$ D=Dilation C=Compression	Castor River	D-1/2C	1/2C-2-1/2C	>2-1/2C
	South Nation	D-2C	2C-4C	>4C
	Bear Brook	D-1C	1C-	-
Shear $(\%\varepsilon axial)_f$	Castor River	<5-1/2	5-1/2-7	>7
	South Nation	<2-1/2	2-1/2-9	>9
	Bear Brook	<5	5-	-
τ Failure lb/in^2	Castor River	<5	5-6	>6
	South Nation	<8	8-9-1/2	>9-1/2
	Bear Brook	<11	11-	-

the soil to stress levels above in situ stresses, i.e., earth embankments" (Mitchell, 1970). For slope stability analysis, we are primarily interested in the low stress range. Most of the shear strength testing has been in the low and intermediate stress ranges.

Presented in Fig. 7.9.1 are the $p'-q'$ and the graphically obtained $\sigma'-\tau'$ envelopes for South Nation, Castor River, and Bear Brook respectively. A proposal of why some of the Bear Brook and Castor River $p'-q'$ results are lower than would be expected is presented in Appendix A. Transformation of the $p'-q'$ envelope to the $\sigma'-\tau'$ envelope has been discussed in Section 3.4.

Typical of fissured clays, these curves appear to have a very small or no cohesion intercept (Eden and Mitchell, 1973) when σ'_n equals zero.

In order of increasing shear strength for a given effective normal stress they are:

1) Castor River
2) South Nation
3) Bear Brook

From Table 7.8.1, the low, intermediate and high normal stress ranges are superimposed upon the $\sigma'-\tau'$ envelopes obtained from Fig. 7.9.1 (Note Fig. 7.9.2).

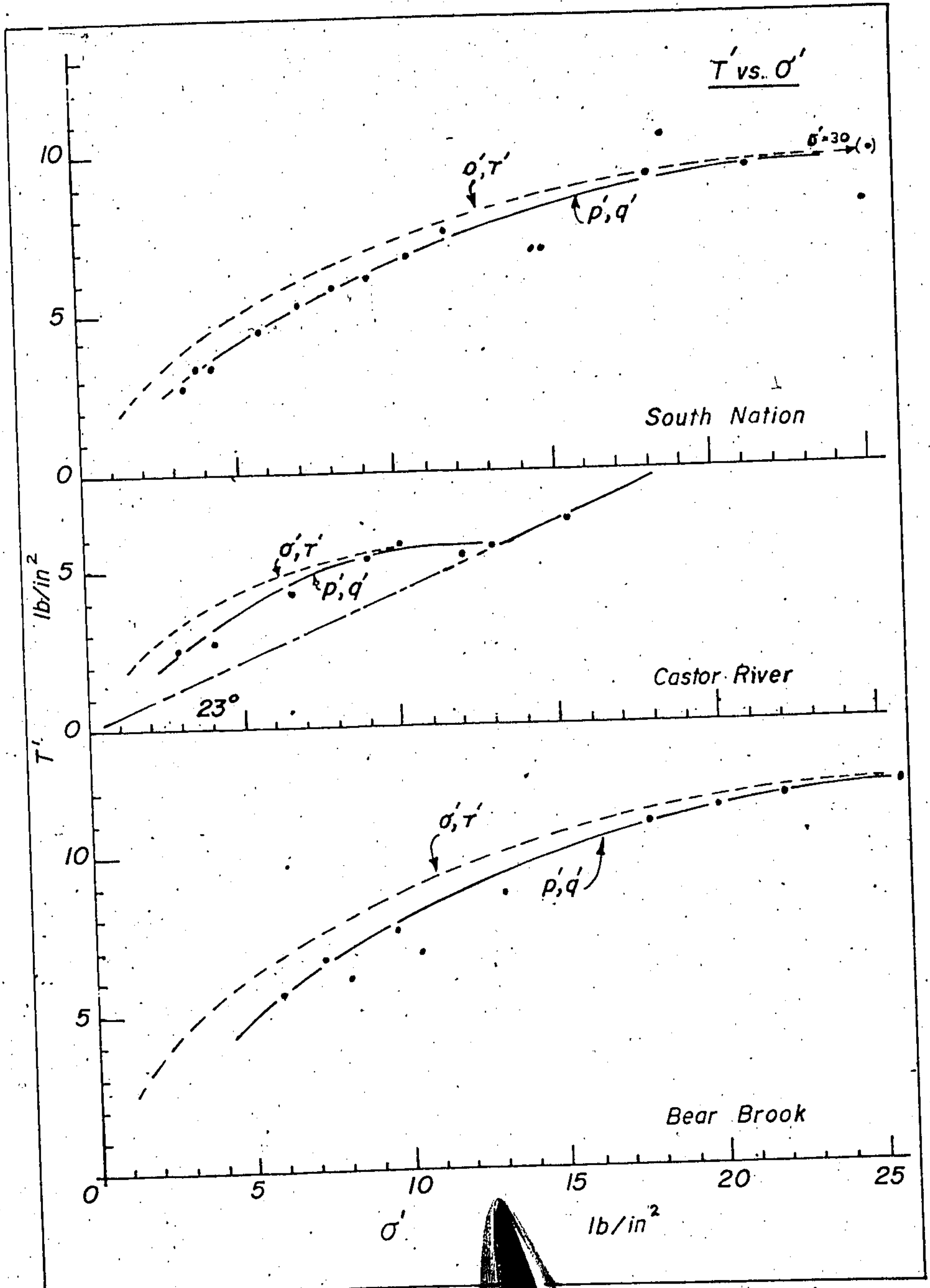
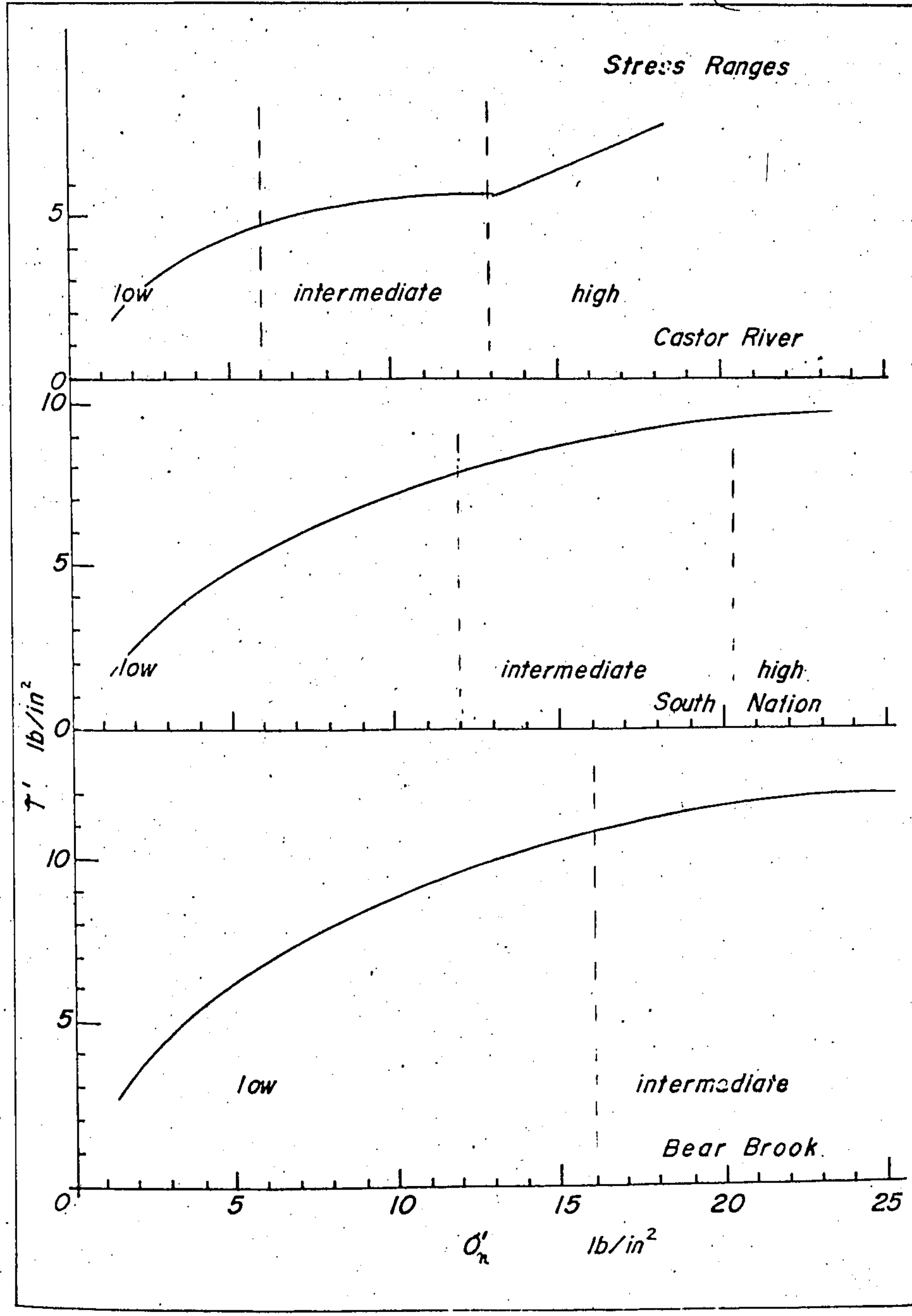

T' vs. σ'
10
σ'=30 (•)
σ', τ'
p', q'
5
South Nation
0
σ', τ'
5
p', q'
Castor River
23°
0
τ'
lb/in²
σ', τ'
10
p', q'
5
Bear Brook
0
5
10
15
20
25
σ'
lb/in²

figure 7.9.1.



Stress Ranges
low
intermediate
high
Castor River
low
intermediate
high
South Nation
low
intermediate
Bear Brook
0
5
10
0
5
10
0
5
10
0
5
10
15
20
25
τ' lb/in²
σ'n lb/in²

figure 7.92

CHAPTER 8

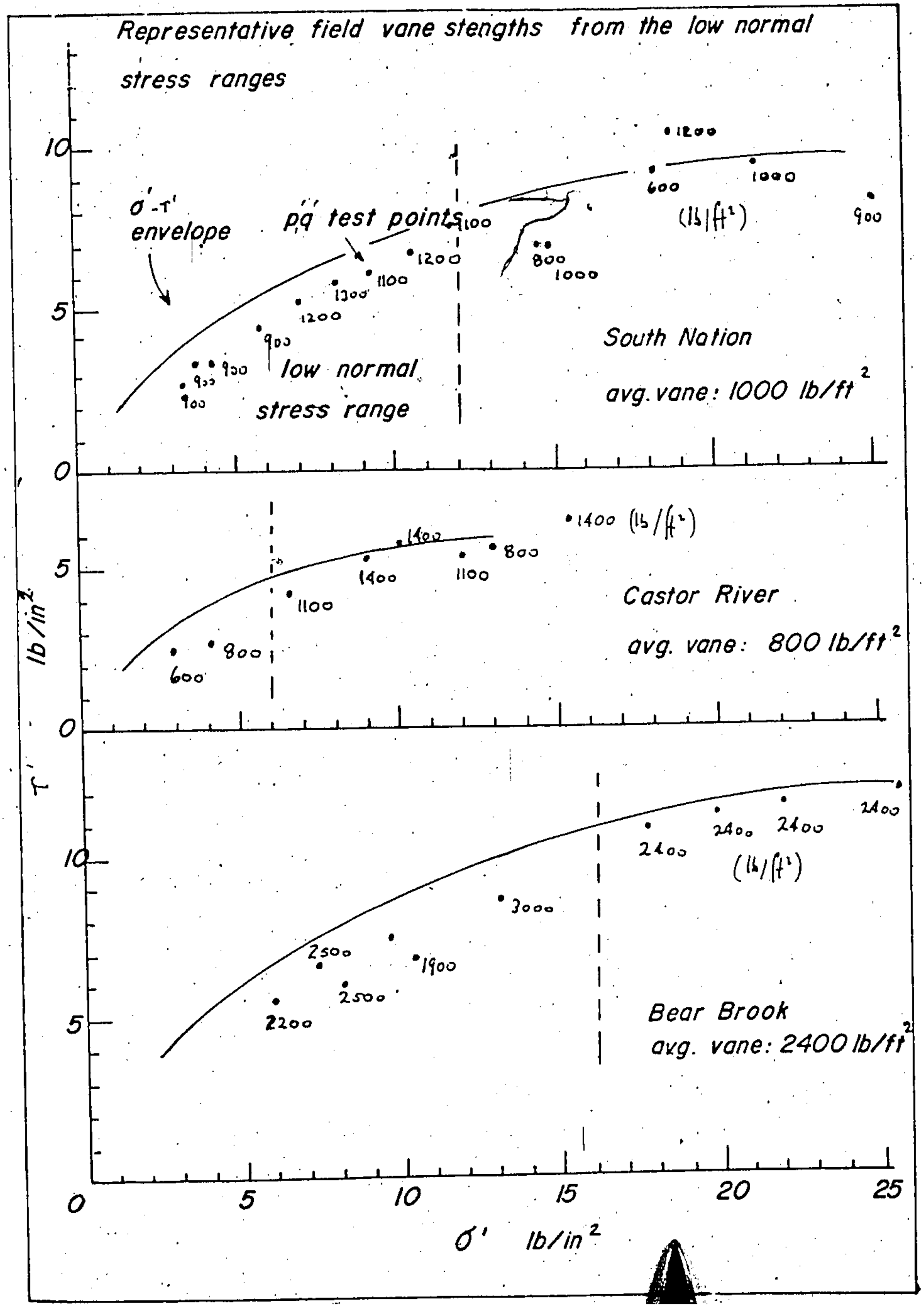
SHEAR STRENGTH AND EASTERN ONTARIO FISSURED CLAYS

8.1 General

There appears to be an important relationship between bond strength and such factors investigated as volume change from shear and isotropic consolidation, preconsolidation pressure, and field vane strength, as well as the effective stress shear strength envelopes.

It was decided to compare effective stress shear strength to an easily measured property. Field vane strength was chosen because it is an easily performed routine test upon relatively undisturbed material.

A field vane strength was estimated by interpolation from borehole logs for each sample tested triaxially. Each of the Castor River, South Nation, and Bear Brook sites were assigned on average field vane strength based upon the field vane strength from the same borehole and close to the same elevation of the samples tested triaxially (Fig. 8.1.1). The average vane strengths were 800, 1000, and 2400 lb/ft^2 respectively for samples tested in the low effective normal stress ranges. Field


Representative field vane stengths from the low normal stress ranges
σ'-τ' envelope
p'q' test points
low normal stress range
(lb/ft²)
South Nation
avg. vane: 1000 lb/ft²
Castor River
avg. vane: 800 lb/ft²
Bear Brook
avg. vane: 2400 lb/ft²
τ' lb/in²
σ' lb/in²

vane strengths for the range of samples tested at Breckenridge (Mitchell, 1970) and Orleans (Eden and Jarrett, 1971) are from the literature. Field vane strength for Rockcliffe clay (Mitchell, 1970) is from borehole logs provided by Mr. G. C. McRostie of Ottawa. No value of field vane strength was available (Mitchell, 1973) for the South Nation clay of Eden and Mitchell (1973).

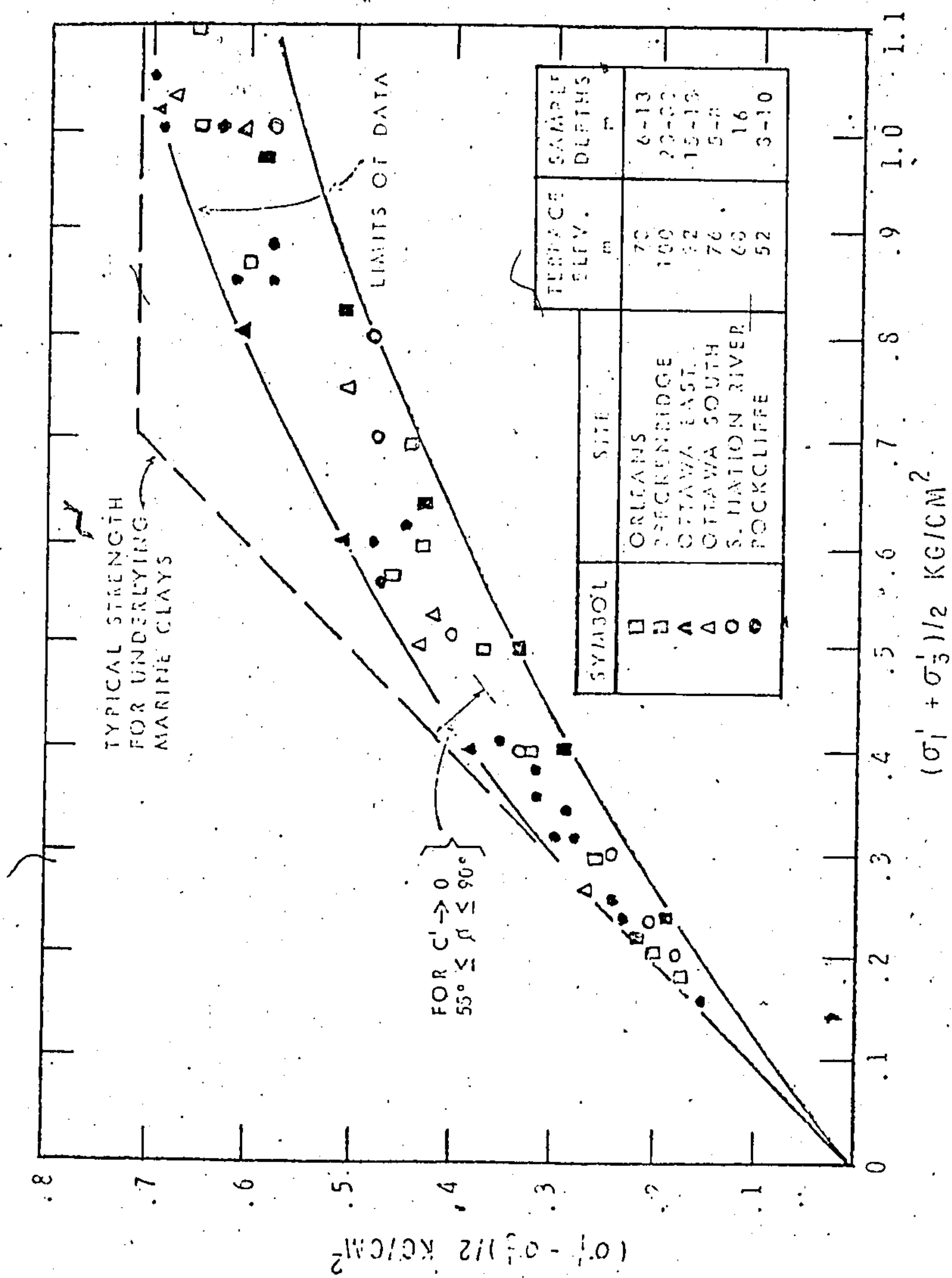
This chapter represents the deviator strength envelopes for six fissured clays in terms of field vane strength and proposes an empirical relationship. Due to the influence of anisotropy, membrane corrections, and end friction during shear, the relationship is more relevant for clays tested in the low effective normal stress ranges.

8.2 Effective Shear Strength Envelopes

A detailed search of the literature for deviator strength envelopes has been undertaken. Shown in Fig. 8.2.1 is a standard method of presentation of shear strength data from Eden and Mitchell (1973).

Deviator strength envelopes for Breckenridge (Mitchell, 1970), Rockcliffe (Eden and Mitchell, 1970; Jarrett, 1970), Orleans (Eden and Jarrett, 1971),

Strength Data for Several Sites in the Ottawa Area



from Eden and Mitchell (1973)

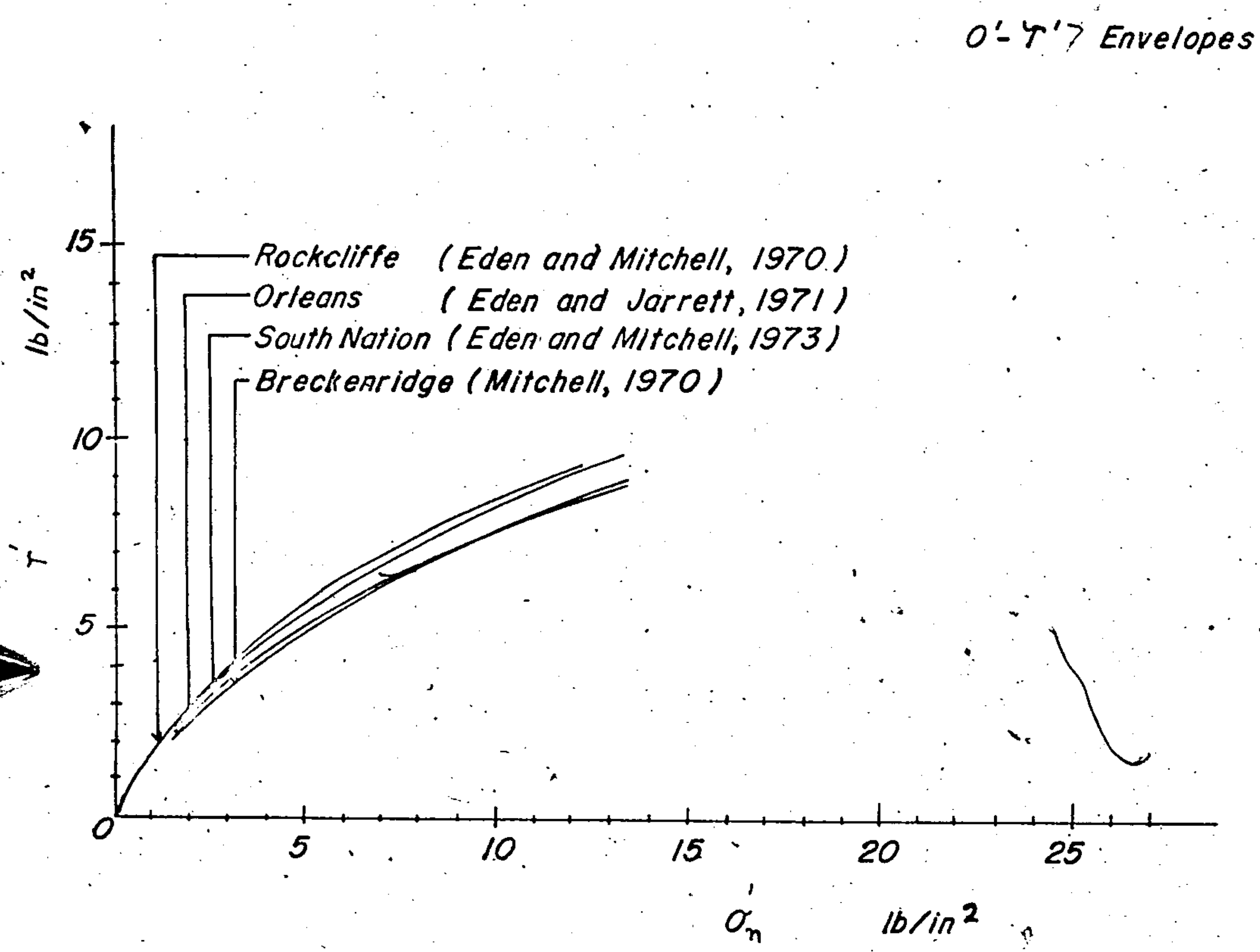
figure 8.2.1

and South Nation (Eden and Mitchell, 1973) have been transferred graphically to effective shear strength envelopes and are presented in Fig. 8.2.2.

Of these fissured clays, that of Rockcliffe is the only one that has tensile strength testing to complete the entire strength envelope. The shear strength envelope for Rockcliffe passes through the origin at $\sigma'_n = 0$. The other clays shown in Fig. 8.2.2 are fissured and may or may not have shear strength envelopes which pass through the origin.

Presented in Fig. 8.2.3 are shear strength envelopes for Bear Brook, South Nation, and Castor River from Section 7.9. No testing has been performed in the very low stress range where tensile tests are required. It is not known whether these envelopes have a zero cohesion intercept or not when σ'_n equals zero.

Table 8.2.1 lists various values of q' for fixed values of p' and associated field vane strengths for the three clays tested as well as for Orleans, Rockcliffe and Breckenridge. It should be noted that Breckenridge is a fissured clay not of Eastern Ontario, but rather of across the Ottawa River in Quebec (Fig. 6.2.1).


O'-T'7 Envelopes
Rockcliffe (Eden and Mitchell, 1970)
Orleans (Eden and Jarrett, 1971)
South Nation (Eden and Mitchell, 1973)
Breckenridge (Mitchell, 1970)
lb/in²
τ'
15
10
5
0
5
10
15
20
25
σ'n
lb/in²

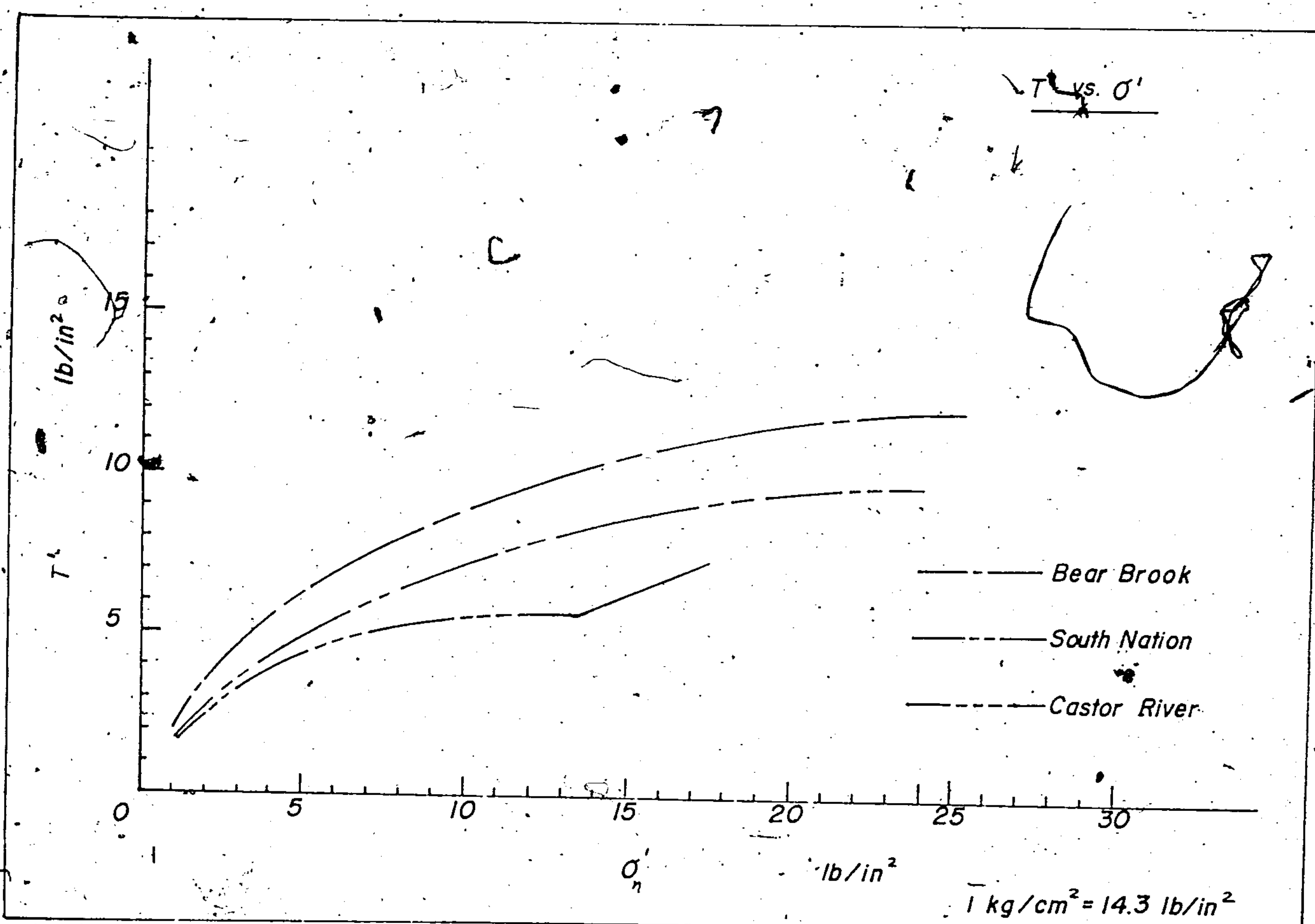

τ vs. σ'
Bear Brook
South Nation
Castor River
τ
lb/in²
0
5
10
15
20
25
30
σ'ₙ
lb/in²
1 kg/cm² = 14.3 lb/in²

TABLE 8.2.1

DEVIATOR STRENGTH ENVELOPE DATA

	p'=3* q'	p'=5 q'	p'=7 q'	p'=9 q'	p'=11 q'	Field Vane lb/ft^2
Castor River	2.5*	3.7	4.75	5.4	6.1	800
Breckenridge	2.2	3.6	4.35	6.05	7.1	1000
South Nation	2.7	4.05	5.3	6.3	7.3	1100
Orleans	2.9	4.4	5.6	6.6	7.7	1400
Rockcliffe	3.0	4.4	5.6	6.65	8.0	1450
Bear Brook	-	5.0	6.5	7.6	8.7	2400

*lb/in^2

Some properties of these clays are presented in Table 8.2.2. Preconsolidation pressure and vane strength are the only properties that show a trend with the magnitude of the effective stress shear strength envelopes.

8.3 Deviator Strength Envelopes and Field Vane Strength

The deviator strength envelopes and the field vane shear strengths from the six sites are compared. Each of these deviator strength envelopes, or p'-q' curves have been represented by five (p',q') points in Table 8.2.1. The table lists, for constant values of p', associated values of q' for all envelopes, and also average field vane strengths associated with each envelope. Table 8.2.1 is presented graphically in Fig. 8.3.1 as a plot of q' vs. field vane strength for different values of p'. This permits the comparison of deviator strength envelopes from different sites on the basis of vane strength.

For the six clays presented in Fig. 8.3.1, there appears to be a good relationship between field vane strength and the envelopes of effective deviator (or normal shear) strength. This graphical figure may be useful for estimating deviator strength (and hence

TABLE 8.2.2

PROPERTIES OF SOME FISSURED CLAYS

	Preconsolidation Pressure lb/ft^2	Field Vane lb/ft^2	Vane Preconsolidation Ratio	Moisture Content %	Liquidity Index %	Sensitivity	% Clay Size	Unit Weight lb/ft^3	Upper Plain Elevation ft
Castor River	2300*	750*	0.33	57±4.8	.93±.04	4-10	-	101-108	215
Breckenridge	-	1000	-	80.0±1.05	1.37±.16	12-150	82	110	330
South Nation	2700	900	0.33	46.5±11.3	0.69±.21	8-11	50	103-115	188
Orleans	4500	1400	0.31	60.1±9.6	0.92±.06	-	-	-	240
Rockcliffe	5000	1450	0.30	65.8±5.3	1.12±.19	35	54	100	180
Bear Brook	6200	2000	0.32	44.7±9.5	0.50±.13	3-6	-	104-108	190

*Specific, not necessarily average values in all cases

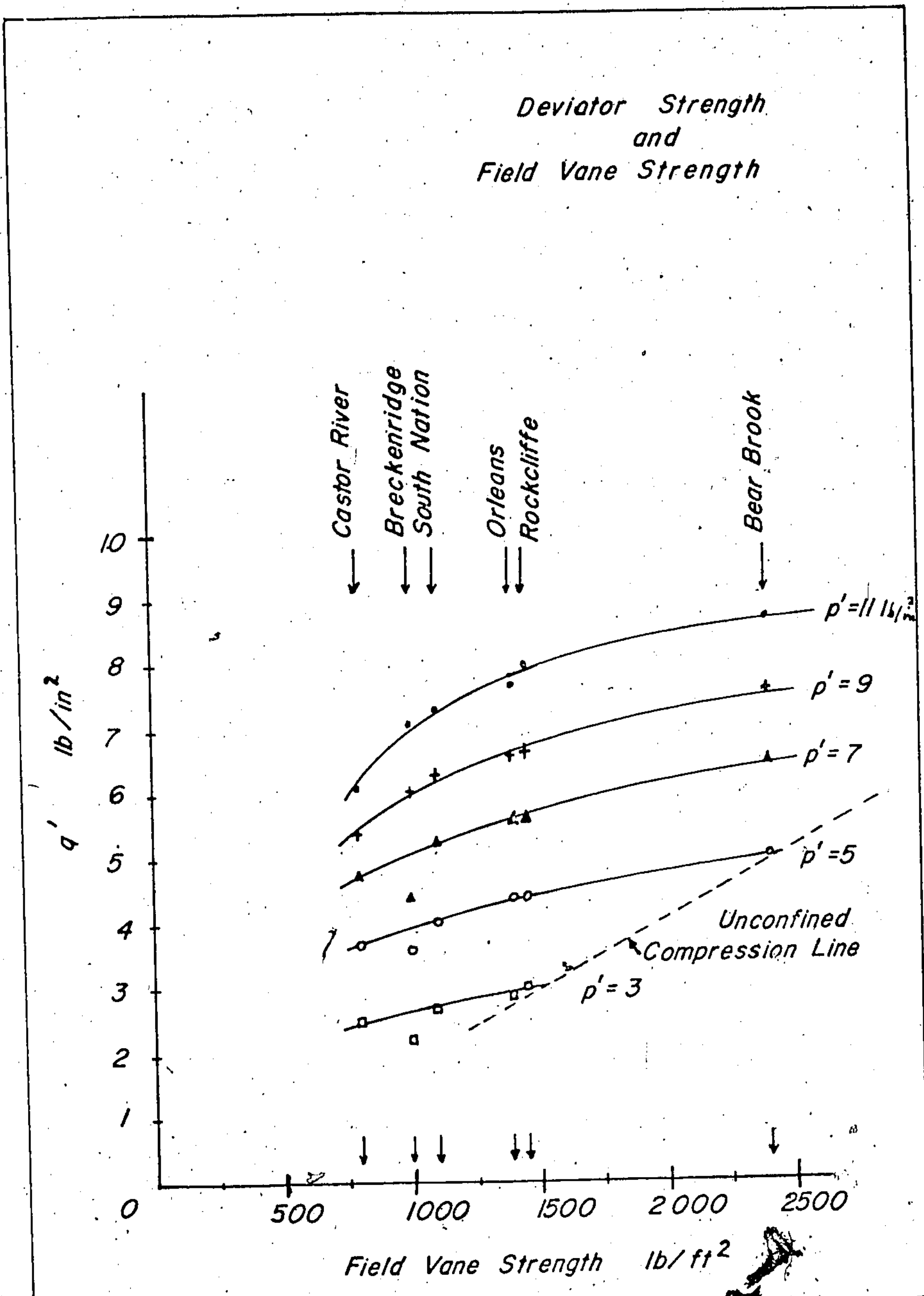

Deviator Strength
and
Field Vane Strength
Castor River
Breckenridge
South Nation
Orleans
Rockcliffe
Bear Brook
p' = 11 lb/in²
p' = 9
p' = 7
p' = 5
p' = 3
Unconfined
Compression Line
q' lb/in²
10
9
8
7
6
5
4
3
2
1
0
500
1000
1500
2000
2500
Field Vane Strength lb/ft²

Figure 8.3.1.

effective shear strength) envelopes for clays of Eastern Ontario on the basis of the simple field vane test.

It may be possible to estimate the effective stress shear strength by means of a series of field vane tests in the area to be investigated. For slope stability analysis it is recommended that these vane tests be taken at the elevation of potential failure surfaces. One or more series of field vane strengths chosen as representative of the area being studied may be selected, resulting in one or more effective shear strength envelopes. Further investigation to substantiate this relationship between the deviator (hence effective shear) strength envelopes in the low normal stress range and field vane strength for fissured clays of Eastern Ontario is required.

CHAPTER 9

CONCLUSIONS

9.1 General

Summarized in this Chapter are accomplishments, conclusions, as well as recommendations and suggestions for future work.

9.2 Summary of Accomplishments

The effective shear strength behaviour of three fissured Leda clays has been investigated. Some of the conclusions from this investigation have been summarized in the following section. Their actions during shear and consolidation have been described in Chapter 7. Relationships between these actions and clay strength have been given.

Three more shear strength envelopes for fissured clays have been added to the literature. It is apparent that shear strengths of Eastern Ontario fissured clays vary widely. This has been demonstrated by the range of the shear strengths found for the three clays from Castor River, South Nation and Bear Brook, and those in the literature.

A relationship between shear strength and field vane strength has been proposed, based on data available.

9.3 Important Conclusions

Following is a summary of the conclusions of this study.

1. There appears to be a relationship between bond strength and such factors as preconsolidation pressure, field vane strength, volume change from isotropic consolidation and shear, and strain from shear.
2. When the behaviour during shear and isotropic consolidation have been expressed in terms of low, intermediate and high normal stress ranges for the three fissured clays tested, it would appear that there is no trend in terms of shear strength. Behaviour in each normal stress range appears to be about the same for all three clays. Comparison of constant P'_m drained

stress controlled data for any fissured clay to data from the three above tests may aid in determining the stress region in which a fissured leda clay of unknown strength was tested.

3. Resistance to deformation under horizontal stress appears to be much less than resistance under vertical stress during isotropic consolidation. This is of importance when considering the anisotropic nature of the clay for design purposes.
4. Strongly bonded fissured clays in shear undergo greater dilation in the low normal stress ranges and less compression in the high normal stress ranges than less strongly bonded fissured clays. It would appear that for fissured clays, porewater pressures induced by shear at low normal stress comparable to in situ conditions for slope stability analysis are negative.
5. The influence of fissures on drainage paths and time to full primary consolidation has been investigated, although it is not known if fissures actually do exist in situ, or are a product of stress release during sampling, their effect upon time to primary consolidation during isotropic consolidation and dissipation of porewater pressure induced by shear is considerable.

6. It would appear that effective stress analysis of in situ shear strength would be incomplete without considering positive and negative porewater pressures developed during shear, time for dissipation of these pressures, and the associated effective normal stress ranges.
7. Volume change from shear and isotropic consolidation appears to be independent of initial void ratios ranging from 1.1 to 1.8.
8. The use of randomly selected samples of widely varying void ratio and depth (below the crust) does not appear to introduce excessive scatter of deviator strength data.
9. The 3% axial strain yield curve envelope may indicate at what combination of stresses a sample may begin to undergo bond breakdown during shear.
10. The Mohr-Coulomb theory does not adequately predict the inclination from the horizontal of the triaxial shear plane, but it appears quite adequate to describe in situ shear strength on the basis of laboratory tests in the low stress ranges for the fissured clays tested.
11. Fissured leda clays of Eastern Ontario vary considerably in strength. Comparison of field vane strengths to

shear strengths has led to an empirical relationship based on limited data by which a deviator strength envelope (or shear strength envelope) may be reproduced with a single or average field vane strength.

12. Field vane to preconsolidation pressure ratios of the three clays tested ranged from 0.26 to 0.34, in agreement with other fissured clays of the Ottawa area.

13. Crawford and Eden (1966) found for the Ottawa area fissured clays that there appears to be a distinct relationship between elevation and preconsolidation pressure across a number of sites. Clays at the three sites were tested for preconsolidation pressure at approximately the same elevation. They exhibited preconsolidation pressures ranging from 2300 to 6500 lb/ft^2. There does not appear to be any relationship in this area between a given elevation and preconsolidation pressure for the three sites tested.

14. p'-q' points, which constitute the deviator strength envelope should be obtained by means of test procedure I. Employment of test procedure II involves isotropic consolidation to pressures above P'_m. This results in a small reduction in measured strength in some cases.

15. Good shelby tube samples appear to yield the same effective shear strength results as block samples in

the low to intermediate normal stress ranges for South Nation fissured clays. It is possible that tube samples from the other two sites tested are adequate.

9.4 Recommendations and Suggestions

Following are some recommendations and suggestions based upon testing results of the programme.

1. Further experimentation into the behaviour of fissured clays during consolidation and shear in each of the low, intermediate and high stress regions is required.
2. Further investigation of the apparently anisotropic bond strength exhibited by the clays tested under isotropic triaxial compression is of interest.
3. Further investigation into the origin and extent of fissures both in laboratory samples and in situ, and their influence upon permeability is required.
4. More research into a better understanding of bond strength is required. Possibly an investigation into physio-chemical properties of the leda clay would be of considerable interest.
5. A better understanding of how strength appears to control volume change during isotropic consolidation and shear irrespective of void ratio is of interest.

6. A testing programme looking into post-peak residual strength of fissured leda clays of Eastern Ontario would further describe behaviour in shear for these clays.
7. Confirmation of the proposed relationship between shear strength and field vane strength by further testing is required.
8. In the future, testing and storage of samples should be at or near ground in situ temperatures. This ground temperature should be recorded when sampling in the field.
9. During testing by the P'_m stress path, samples should not be consolidated above P'_m, as this may disturb bonding. Test procedure I should be followed.
10. Contrary to the standard backpressure of 15 lb/in^2 employed in the literature, a greater value should be used in order to ensure a sufficient degree of saturation without waiting for an excessive length of time.

APPENDIX

A. Effect of Test Procedure on Measured p' and q'

As discussed in Section 5.6 on Test Procedure, some samples in the Castor River and Bear Brook series were consolidated to pressures in excess of P_m' and then permitted to swell at P_m', prior to shearing.

It appears that this procedure may have reduced the measured deviator strength slightly in some cases. Samples which were consolidated isotropically to pressures above P_m' are indicated (along with isotropic consolidation pressures) in Figure A.1. Numbering from the left, the p'-q' points affected by isotropic consolidation to pressures above P_m' are the third (10 lb/in^2), fourth (10 lb/in^2) and fifth (10 lb/in^2) for Castor River and the first (20 lb/in^2), second (10 lb/in^2), third (20 lb/in^2), fourth (10 lb/in^2), fifth (20 lb/in^2), and sixth (20 lb/in^2) for Bear Brook. At Castor River only the p'-q' point consolidated to 10 lb/in^2, having a P_m' of 5 lb/in^2 appears to have been slightly disturbed. At Bear Brook, points of P_m' of 6, 8, 10 lb/in^2 consolidated to 20 lb/in^2 and allowed to swell to P_m' appear to have slightly reduced deviator strengths. The deviator strength associated with

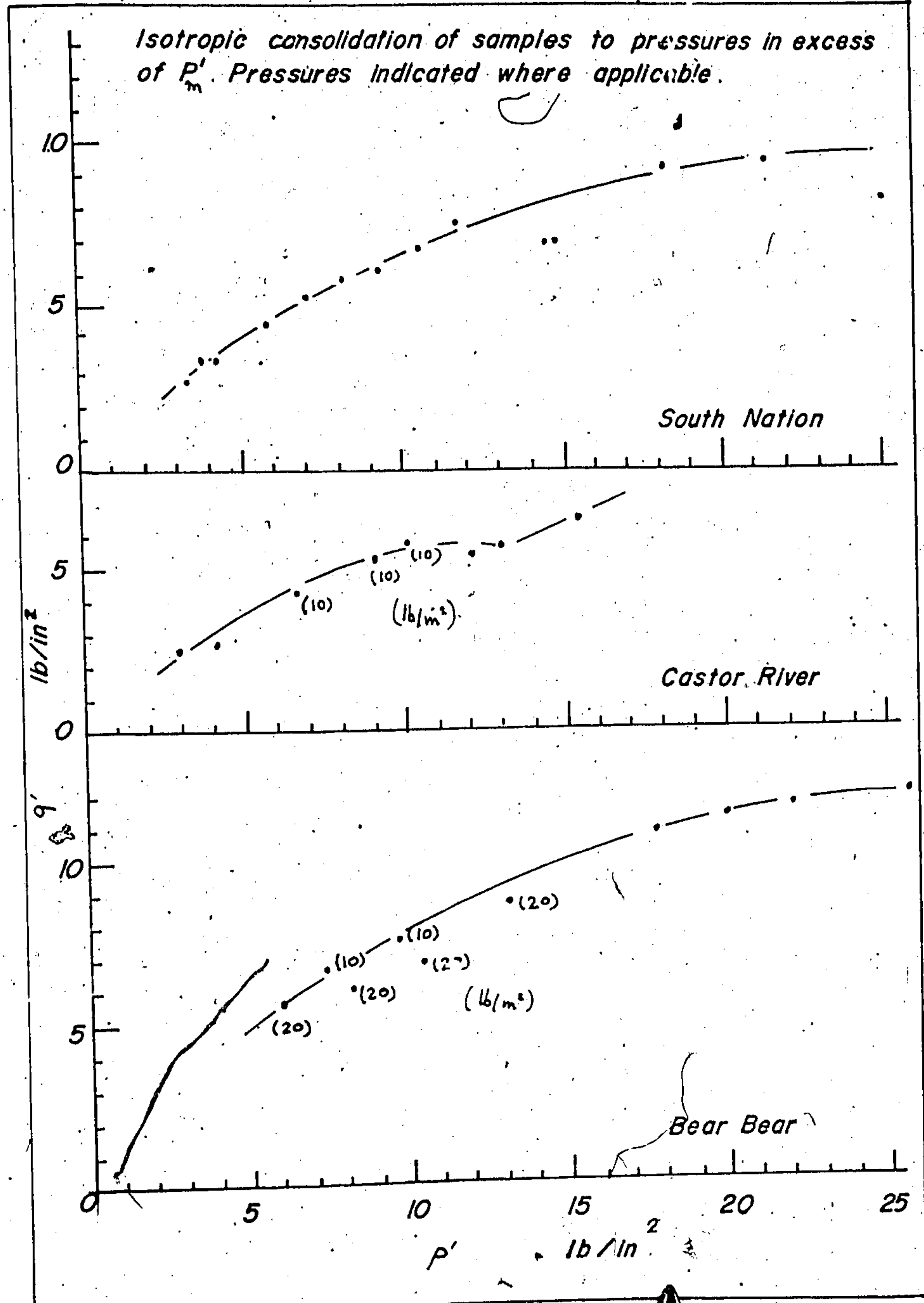

Isotropic consolidation of samples to pressures in excess of P'_m. Pressures indicated where applicable.
South Nation
Castor River
Bear Bear
(10)
(20)
(lb/in²)
q' lb/in²
p' lb/in²
0
5
10
15
20
25

figure A.1

P'_m of 4 lb/in^2 consolidated to 20 lb/in^2 does not appear to have been reduced by this test procedure. This can be noted in Fig. 7.3.8 where isotropic consolidation to 20 lb/in^2 in this instance has not resulted in excessive percentage volume change.

Consolidation of Bear Brook samples of P'_m of 5 and 7 lb/in^2 to 10 lb/in^2, prior to swelling and shear, appears to have little effect upon measured strength. It would appear that any advantage obtained by consolidating samples to pressures in excess of P'_m in order to reduce influence of sampling and handling upon measured deviator strength is outweighed by a slight reduction in deviator strength caused by this isotropic consolidation of samples to pressures in excess of P'_m.

It is recommended that, in future testing, samples be consolidated to pressures not in excess of P'_m.

BIBLIOGRAPHY

Berre, T. (1971) Schjetne, K. Sollie, S. "Sampling Disturbance of Soft Marine Clays." Norwegian Geotechnical Institute Report #85.

Bishop, A.W. (1969) Garga, "Drained Tension Tests on London Clay," Technical Note, Vol. 19, Geotechnique, 1969.

Bishop, A.W. (1962) "The measurement of soil properties in the triaxial test", Published by Edward Arnold, London, 1962.

Bjerrum, L. (1971) "Weathering of Marine Clays in Temperate Climates", Norwegian Geotechnical Institute Report #85.

Black, D.K. (1973) Lee, K.L. "Saturating Laboratory Samples by Backpressure", ASCE, JSMFD, 99, 1, p.75, 1973.

Bozozuk, M. (1971) "Effect of Sampling, Size, and Storage on test results for Marine Clay", ASTM, STP #483, 1971, pp.121-131.

Campanella, R.G. (1968) "Influence of Temperature Variations on Soil Behaviour", JSMFD, ASCE, 94, 3, 709.

Crawford, C.B. (1961) "The influence of strain on the shearing resistance of sensitive clay", Proc. ASTM, Vol. 61, 1961, pp.1250-1276.

Crawford, C.B. (1963) "Cohesion in an undisturbed Sensitive Clay", Geotechnique 13:2:132-136, 1963.

Crawford, C.B. (1968) "Quick Clays of Eastern Canada", Engin. Geol. V. 2, No. 4, pp.239-265, 1968.

Crawford, C.B. (1963) Eden, W.J. "A Comparison of Laboratory Results with in-situ properties of Leda Clay", Proc. #6 Int. Conf. Soil Mech. Found. Eng., Vol. 1, pp.31-35, 1965.

Crawford, C.B. (1965) Eden, W.J. "A Comparison of Laboratory Results with in-situ properties of Leda Clay", Proc. 6th Int. Conf. Soil Mech. Found. Eng., Vol. 1, pp.31-35, September, 1965.

Crawford, C.B. (1967) "Stability of Natural Slopes in Sensitive Clay", JSMFD, ASCE, Vol. 93, No. SM4, pp.419-436, July, 1967.

Crawford, C.B. (1969) Closure to "Stability of Natural Slopes in Sensitive Clay", Proc. ASCE, Conf. Stability and Performance of Slopes and Embankments, Berkeley, California, August, 1969.

Conlon, R.J. (1966) "Landslide on the Toulnustouc River, Quebec", Canadian Geotechnical Journal, Vol. 3, No. 3, 1966.

De Lory, F.A. (1968) Salvas, R.J. "Some Observations on the Undrained shearing strength used to analyze a failure", 21st Canadian Soil Mechanics Conference, Winnipeg, Manitoba, September, 1968.

Duncan, J.M. (1967) Seed, H.B. "Corrections for Strength test data", JSMFD, ASCE, Vol. ~~93~~, SM5, pp.121-137, 1967.

Eden, W.J. (1973) "Some observations at LeCoteau landslide, Gatineau, Quebec", Canadian Geotechnical Journal, Vol. 9, No. 4, November 1972.

Eden, W.J. (1970) "The Mechanics of Landslides in Leda Clay", Canadian Geotechnical Journal, 7:285, 1970.

Eden, W.J. (1971) Jarrett, P.M. "Landslide at Orleans, Ontario", NRCC Publication No. 11856, March, 1971.

Gadd, N.R. (1962) "Surficial Geology of Ottawa Map-Area, Ontario & Quebec", Geol. Survey Can., Paper 62 (16).

Gill, A.S. (1968) "A study of Cohesion in terms of Effective stresses for some Ontario clay", Ph.D. Thesis, U. of Toronto, 1968.

Henkel, D.J. (1952)
Gilbert, G.D. "The effect of the rubber membrane on the measured triaxial compression strength of clay samples", Geotechnique, Vol. 3, pp.20-29, 1952-3.

Holubec, I. (1965)
Scott, J.D. "Transducers and piston friction in Triaxial tests", University of Waterloo.

Jarrett, P.M. (1970) Discussion to Eden & Mitchell, 1970, Canadian Geotechnical Journal, 7, 504, 1970.

Jarrett, P.M. (1972) "The effects of soil structure on the engineering behaviour of sensitive clay", Q. Jour. Eng. Geol., Vol. 5, 1972.

Karrow, P.F. (1961) "The Champlain Sea and its Sediments", in "Soils in Canada", R.F. Legget, Editor, University of Toronto Press, Royal Society of Canada, 1961.

Kenney, T.C. (1968)
Moum, J.
Berre, T. "An Experimental Study of Bonds in a Natural Clay", Proc. Oslo Geot. Conf., 1967.

Ladanyi, B. (1970) Discussion to Eden & Mitchell, 1970, Canadian Geotechnical Journal 7, 506, 1970.

Lambe, T.W. (1957) "Composition and Engineering Properties of Soil", V. Proj. Highway Research Board, No. 36.

LaRochelle, P. (1970)
Chagnon, J.Y.
Lefebvre, G. "Regional Geology and Landslides in the Marine Clay Deposits of Eastern Canada", Canadian Geotechnical Journal, 7, 145, (1970).

LaRochelle, P. (1971)
Lefebvre, G.
"Sampling Disturbance in Champlain Clays", ASTM, STP, No. 483, 1971.

Lawrence, V.M. (1969)
"The Failure Behaviour of Naturally Cemented Soils", M.Sc. Thesis, Queen's University, 1969, Kingston.

Lee, K.L. (1972)
Black, D.K.
"Time to Dissolve air bubble in drain line", ASCE, JSMFD, 98, 2, 181, 1972.

Lefebvre, G. (1973)
LaRochelle, P.
"The Analysis of slope failures in cemented Champlain Clays", 26th Canadian Geotechnical Conference, Toronto, October, 1973.

Leggett, R.F. (1961)
"Three Buildings on Floating Foundations in Ottawa, Canada", Journ. Nat. Bldgs. Org., Vol. 6, No. 1, January, 1961.

Lo, K.Y. (1973)
Lee, C.F.
"Stress Analysis and Slope Stability in Strain Softening Materials", Geotechnique 23, No. 1, 1-23, (1973).

Lo, K.Y. (1973)
Lee, C.F.
"Analysis of progressive failure in clay slopes, Moscow, International Conference on Soil Mechanics and Foundation Engineering, August, 1973.

Lo, K.Y. (1973)
Lee, C.F.
"An evaluation of the stability of natural slopes in Plastic Champlain Clays", 26th Canadian Geotechnical Conference, Toronto, October, 1973.

Lo, K.Y. (1972)
Morin, J.P.
"Strength anisotropy and time effects of two sensitive clays", Canadian Geotechnical Journal, Vol. 9, No. 3, August, 1972.

Mitchell, R.J. (1970)
"Landslides at Breckenridge, Pineview Golf Club and Rockcliffe", NRCC No. 11536, DBR TP No. 322, 1970.

Mitchell, R.J. (1970)
"On the Yielding and Mechanical Strength of Leda Clay", Canadian Geotechnical Journal, 7:297:1970.

Mitchell, R.J. (1973)
Private Communication.

Mitchell, J.K. (1968) Campanella, R.G. Singh, A. — "Soil creep as a rate process", JSMFD, ASCE, 94, 1, 231.

Mitchell, R.J. (1973) Lawrence, V.M. — An Approach to the problem of progressive failure: Discussion Canadian Geotechnical Journal, Vol. 10, No. 3, 1973.

Mitchell, R.J. (1973) Markell, A.R. — "Flowsliding in sensitive clays", 26th Canadian Geotechnical Conference, Toronto, October 1973.

Mitchell, R.J. (1973) Wong, P.K.K. — "The Generalized Failure of an Ottawa Valley-Champlain Sea Clay", Canadian Geotechnical Journal, Vol. 10, No. 4, 1973.

Patton, F.D. (1971) — "The determination of Shear Strength of Rock Masses", from "The analysis and Design of Rock Slopes", Dept. of Civil Engineering and Dept. of Extension, University of Alberta, Edmonton, August 23-27, 1971.

Paul, M.J. (1971) — "The Mechanics of Landslides in Leda Clay", Canadian Geotechnical Journal, 8, 143 (1971).

Penner, E. ~~(1963)~~ — "Sensitivity in Leda Clay", Nature, V. 197, p.347, 1963.

Penner, E. (1965) — "Studies of Sensitivity and Electro-Kinetic Potential in Leda Clay", Nature, V. 204, pp.808-809, November, 1964.

Quigley, R.M. (1966) Thompson, C.D. — "The Fabric of Anisotropically Consolidated Sensitive Marine Clay", Canadian Geotechnical Journal, V. 3, No. 2.

Raymond, G.P. (1971) Townsend, D.L. Lojkasek, M.J. — "The effect of sampling on the Undrained soil properties of a Leda Soil", Canadian Geotechnical Journal, 8, 4, 546, November, 1971.

Rowe, P.W. (1964) Barden, L. "Importance of free ends in triaxial testing", JSMFD, ASCE, Vol. 90, SM1, 1964.

Rowe, P.W. (1964) Barden, L. Lee, I.K. "Energy Components during the Triaxial Cell and Direct Shear Tests", Geotechnique 14, 247-261, 1964.

Sangrey, D.A. (1969) Townsend, D.L. "Characteristics of Three Sensitive Canadian Clays", C.E. Research Report No. 63, Department of Civil Engineering, Queen's University, Kingston, Ontario.

Schmertmann, J.H. (1953) "Estimating the true consolidation behaviour of clay from laboratory test results", Proc. ASCE, Vol. 79, Separate 311, October 1953.

Scott, J.D. (1973) Private Communication.

Scott, R.F. (1969) "Stress deformation and strength characteristics, State of the Art Volume", Seventh International Soil Mechanics Conference, Mexico, 1969.

Skempton, A.W. (1964) "Long term stability of clay slopes", Fourth Rankin Lecture, Geotechnique 14, 75-102, 1964.

Soderman, L. (1965) "Geotechnical Properties of Three Ontario Clays", Canadian Geotechnical Journal, V. 2, No. 2, p.167, 1965.

Tavenas, F.A. (1973) Roy, M. LaRochelle, P. "An artificial material for simulating Champlain Clays", Canadian Geotechnical Journal, Vol. 10, No. 3, August, 1973.

Townsend, D.L. (1969) Sangrey, D.A. Walker, I.K. "The Brittle Behaviour of Naturally Cemented Soils", Proc. 5th Int. Conf. Soil Mech. Found. Eng., Mexico City, 1969.

Webb, G.S. (1970) Title unknown, Strength of Crustal Leda Clay, M.A.Sc. Thesis, Queen's University, Kingston, 1970.

Youssef, M.S. (1961) et al. "Temperature changes and their effects on some physical properties of Soils", Proceedings 5th Int. Conf. Soil Mech. Found. Eng., Paris, 1961.