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THE INFLUENCE OF MICROFABRIC ON THE ENGINEERING PROPERTIES OF GLACIAL TILLS

by

RALPH JAMES WITTEBOLLE

A THESIS
SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
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David Sego

Supervisor

Supervisor

Date October 29, 1982

To Susan, my best friend.

Abstract

This research program arose from an interest in the effects of soil microfabric upon the engineering behavior of glacial tills. Tills from sites in Nipawin, Saskatchewan and Edmonton, Alberta were examined during the study. In addition, a series of reconstituted samples were prepared.

A scanning electron microscope study was undertaken to provide a detailed record of the microfabric. The intact fabrics of the intact Nipawin and Edmonton tills have an open structure which is best portrayed by the fabric of the reconstituted from slurry samples. Reconstitution by remolding produces a closed fabric.

A comprehensive laboratory testing program involving index tests, consolidation tests, static and cyclic triaxial tests allowed an assessment of the influence of the micro fabric upon the engineering properties of the intact and reconstituted tills.

The research program indicates that the microfabric of matrix dominated tills can be divided into two assemblages: the matrix and soil peds. The soil peds form a structural framework within the soil. When a load is applied under low confining pressures, the highly consolidated peds act as individual particles and dilation occurs. The amount of dilation is dependent on the degree to which the soil peds developed during both primary and secondary consolidation. At high confining stresses, the soil peds are not allowed to dilate and the strength of the soil is controlled by the

clayey silt matrix.

The results show that the confining stress has a significant effect upon the strength of a till. Due to the ped framework, the angle of frictional resistance is highly dependent on the effective confining pressure or the normal stress on the failure plane. When heavier loads are anticipated, the decrease in the strength parameter should be considered.

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1. INTRODUCTION

1.1 General

The safety and economic design of heavy foundations on glacial tills depends on accurately estimating the in-situ engineering properties. Often discontinuities and clasts within the till limit the number of samples suitable for laboratory testing, therefore design is based on a relatively few laboratory and small scale in-situ tests.

During the site investigation for the Nipawin dam site, it was noted that the foundation soils closely resembled the glacial tills in the Edmonton area. The comparison was based on grain size distribution, Atterberg limits and bulk densities. As a result of these similarities, the preliminary design at Nipawin was based on the strength properties of the Edmonton till as reported by Medeiros (1979).

Subsequent laboratory testing of the Nipawin Till samples showed a considerable difference between its strength properties and those of the Edmonton Till. One explanation for the difference in shear behavior between the two tills was offered after examining the contact and distribution of the particles within the till materials with a scanning electron microscope. Sego (1980) reports:

> "The micrographs show two striking differences in the character of the tills. The Edmonton Till is characterized by a structure arrangement where

> coarse grained particles are in contact. The void space is infilled with fine grained clay particles which form granulated lumps. The till from Axis 5 site (Nipawin Dam Site) shows that the coarse grained particles have fewer contacts with other similar sized particles. It is characterized instead by the coarse particles being surrounded by fine grained clay size particles."

These observations were provided by May (1980). It was reasoned that the lack of coarse grained contacts within the Nipawin Till inhibited the material from dilating, hence a lower strength was measured.

The importance of soil microfabric and macrofabric features on the large scale strength, deformation, and drainage characteristics of soils has long been realized (Marsland et. al., 1982). While the macrofabric features are expected to have a dominating influence on the engineering properties of glacial tills, Sego (1980) has shown that microfabric features also can have a significant effect. Thus the effect of microfabric features on engineering properties such as compressibility and strength must be studied.

1.2 Scope of Research

This work has been undertaken to quantify the effect of various microfabric features on the engineering behavior of two tills, namely Nipawin and Edmonton. In addition, the microfabrics and engineering behavior of three reconstituted tills were studied.

Index tests were carried out on the Edmonton and Nipawin tills in order to classify the deposits. The index tests included sieve analysis, Atterberg limits, density and X-ray diffraction tests.

In order to study the engineering behavior of the tills, triaxial compression tests were carried out over a range of confining pressures. The till samples were subjected to both static and cyclic loadings.

Index and strength tests were also performed on the reconstituted samples. Dissimilarities in the engineering behavior of the reconstituted samples were noted and the intact till behavior was compared to the reconstituted behavior to see if the strength parameters could be related.

A scanning electron microscope study was undertaken to provide a detailed record of the microfabric. Samples from both intact and reconstituted tills were examined and their fabric compared. The samples were examined under low to high levels of magnification (Collins and McGown, 1974; Derbyshire, 1978), in order that adequate descriptions could be made for a wide range of features, particle groupings, and orientations.

2. GENERAL GEOLOGY

2.1 Introduction

A study of the engineering behavior of glacial tills requires an adequate knowledge of the sequences of glacial deposition. The first part of this chapter summarizes the past and present models of glacial deposition as proposed by various authors. The second part of the chapter describes the geologic stratigraphy for the tills studied.

2.2 Classification of Glacial Deposits

2.2.1 Historical Classification

Deposits of glacial till are perhaps the most widespread products of glaciation in the provinces of Alberta and Saskatchewan. Till has been defined as:

> "an unstratified glacial deposit of clay, silt,
> sand, gravel and boulders"

(Terzaghi and Peck, 1948). Flint (1957) classified till according to its depositional history. Lodgement till was defined as being deposited from glacial material in transport near the base of the glacier and it tends to be very dense. Ablation till, on the other hand, was deposited from material transported within or upon the "dead ice" area of a melting glacier and it is less dense than lodgement till.

The conceptual mode of glacial sedimentation most commonly referred to is one of a dense subglacial lodgement till with a thin capping of relatively loose ablation till. Interbedded sequences of till with stratified sediment are thus taken to represent a history of glacier advance and retreat.

2.2.2 Recent Observations

The classification of tills has undergone a rapid evolution from the basic subdivisions of ablation and lodgement tills. Recent observations on modern glaciers have greatly enhanced the understanding of glacial depositional processes (Boulton, 1972; Shaw, 1977; Eyles, 1979). Three depositional models are used to explain the complexity of various glacial sediments and land forms (Boulton and Paul, 1976).

The first model is concerned with deposition in alpine areas. This model can be used to interpret glacial sequences in modern valley glaciers. It cannot be used, however, to interpret glacial sequences of the large continental ice sheets present during periods of glaciation.

In lowland areas two characteristic sedimentary models are suggested: subglacial/proglacial sediment association and supraglacial sediment association (Figure 2.1). The subglacial/proglacial model is associated with temperate glaciers which transport the majority of their debris load near the glacier sole. Large volumes of debris are accumulated due to the high rate of basal ice melt. The actual magnitude of till deposition depends on many factors including water pressure at the base, ice velocity, basal ice temperature, particle size and the shape, roughness and permeability of the underlying bed (Boulton, 1975). Subglacial tills are commonly indentified by factors such as high bulk density, mechanical composition, and clast fabric. Generally the sediments exhibit strongly developed fabrics and smoothed pebbles with minor breakages. In addition, Kruger (1979) suggests that the following features also indicate subglacial deposition: lenses of sorted material, smudges, small scale deformations of till matrix and smudges by clasts, clasts consistently striated and clasts with stoss and lee sides in relation to glacier movement direction.

In contrast, subpolar and polar glaciers transport debris through a considerable thickness of basal ice. Upon stagnation and downwasting of these ice masses, supraglacial sediments are produced. Supraglacial sediments commonly have weakly developed, inconsistent fabrics and mostly angular and broken pebbles (Flint, 1971). Two forms of supraglacial sediments are flow and melt out tills.

Flow tills are formed when ice rich glacial debris melts and the debris flows downslope under the influence of gravity. Two types of flow till have been distinguished. The first occurs as debris moves in a slurry in which particles are independently mobile. In the second, mass sliding occurs and the particles are restrained by their neighbours.

Melt out tills are similar in origin to flow tills, however, the slurry does not flow and the till may retain some englacial features. The presence of meltwater ensures that both tills are deposited at water contents well above their liquid limits and have high initial void ratios.

Supraglacial debris accumulating on the ice surface by ablation inhibits the melt-out process by acting as an insulator to the underlying ice. When the thickness of the debris zone is equal to or greater than the thickness of the active layer no further surface melting will occur. As a consequence, till depositional processes in cold regions could be dominated by melt out from an isothermal basal zone at the 0°C isotherm.

Considering the changing temperature regimes during the different stages of glaciation, it is likely that a single till deposit could possibly consist of both supraglacial and subglacial deposits. Classification of a till deposit then depends on the exposure being examined. A deposit could exhibit both supra and subglacial features. It is evident that conflicting theories on the origin of a certain till deposit could arise. To fully appreciate the complexities of a specific till deposit, the total extent of the deposit must be examined. Unfortunately, a total section is rarely exposed.

2.3 General Area Geology

2.3.1 Nipawin Site - Quaternary Geology

2.3.1.1 Geomorphology

The general surface topography in the Nipawin area is predominantly composed of proglacial landforms which are associated with a lacustrine environment. The area was inundated by Lake Aggasiz and till outcrops are absent except for occurrences along the North Saskatchewan River. Subsequent erosion and some fluvial deposition has modified the surficial topography.

Christiansen (1979) postulates that during the latter stages of the Wisconsin, a major meltwater channel, seemingly following the course of the present river, formed the Saskatchewan River Delta in this area. Presently, the river has eroded through the deltaic sands and gravels forming erosional terraces on both sides of the valley. Where the groundwater table nears the surface, peat bogs have formed.

2.3.1.2 Stratigraphy

The geology of the area has been summarized by Christiansen and Menely (1969). In ascending order, the major stratigraphic units in a typical Nipawin section are:

1. Bedrock
2. Lower till

3. Middle till
4. Upper till
5. Proglacial sediments

The bedrock in the vicinity consists of the Ashville Formation and the Swan River-Manville Gro The Ashville Formation is composed of Lower Cretac clay shales, silts and very fine sands. The Swan River-Manville Group includes interbedded quartzos fine to coarse sand, silts and clay shales with th occasional limestone stringers and carbonaceous be

The lower till is a dense, grey, calcareous, clayey silt. The unit forms the lower part of the wall and underlies the valley fill in the river bot X-ray diffraction patterns indicate that illite and kaolinite are the dominant clay minerals. The till composed of approximately 51% silt and clay, 44% sa and 5% gravel.

The middle till is a dense, grey, calcareous, clayey silt. It is distinguished from the lower til appreciable amounts of expandable clay minerals. Th normal composition is approximately 64% silts and c 31% sand, and 5% gravel. The majority of the gravel clasts are subangular.

The upper till is a dense, grey, clayey silty The clay minerals have a low activity suggesting th lack of expandable clay minerals.

The proglacial sediments are primarily sands and silts which are predominately deltaic in origin. The till sheets are, in most locations, separated by fluvial deposits which range from clays to gravels. In addition, an erosional boulder pavement exists at certain locations between the upper and middle tills. The upper till is thought to belong to the Battleford Formation whereas the middle and lower tills have been correlated with the Floral Formation (Christiansen, 1979). The Battleford and Floral Formations represent two distinct glacial sequences where

> "the hiatus between the deposition of the Floral and Battleford Formations was a brief one"

(Christiansen, 1968). The Battleford till has a lower carbonate content than the underlying Floral Formation.

2.3.2 Edmonton Site - Area Geology

2.3.2.1 Surficial Geology

Edmonton is located in the physiographic region known as the Eastern Alberta Plain, an area of generally low relief. The topography of the area is largely controlled by elements of the preglacial landscape (Kathol and McPherson, 1975). Present day uplands coincide with preglacial highs and low areas generally follow preglacial valleys. The near surface sediments were deposited primarily in glacial and post-glacial times. They are composed mainly of colluvium and lacustrine sand, silt and clay deposits.

2.3.2.2 Stratigraphic Units

At present the stratigraphic section for the Edmonton area is that proposed by Westgate (1969):

1. Upper Cretaceous bedrock
2. Saskatchewan Sands and Gravels
3. Lower till
   Tofield sands
5. Upper till
6. Lacustrine deposits

The bedrock consists of the Belly River, Bearpaw, and Horseshoe Canyon Formations of Upper Cretaceous age. These formations consist mainly of interbedded and intertongueing sandstones, siltstones, shale and coal.

The Saskatchewan Sands and Gravels consist of late Tertiary and younger gravels and sands deposited by streams flowing northeastward or eastward from the Rocky Mountains. They are composed mainly of quartzitic rock fragments with minor amounts of chert, arkose, petrified wood, coal, clay and ironstone.

The possibility of two distinct till sheets existing in the Edmonton area has been debated. The upper and lower tills are both heterogeneous and similar in composition. They are hard silty clayey sands composed of approximately 40 to 45% sand, 25 to 35% silt and 20 to 30% clay sizes (May and Thomson, 1978). A significant proportion of the clay sizes is montmorillorite. Bayrock and Berg (1966) noted colour differences between the tills but did not acknowledge the presence of separate till sheets. Ramsden and Westgate (1971) indicate that the lower till was laid down by ice moving approximately west of north whereas the upper till was derived from ice moving from east of north. Kathol, et. al. (1975) observed:

> "differently colored tills discontinuously separated by lenses of sand and gravel but, in spite of data from numerous outcrop sections and a detailed drilling program, could not establish the presence of two tills with any degree of certainty or confirm the continuity of the Tofield Sand".

May and Thomson (1978) questioned the concept of two distinct glacial advances due to the presence of water laid fluvial deposits within the till. Recently Shaw (1982) postulated that the main body of the Edmonton Till was formed by a basal melt out process.

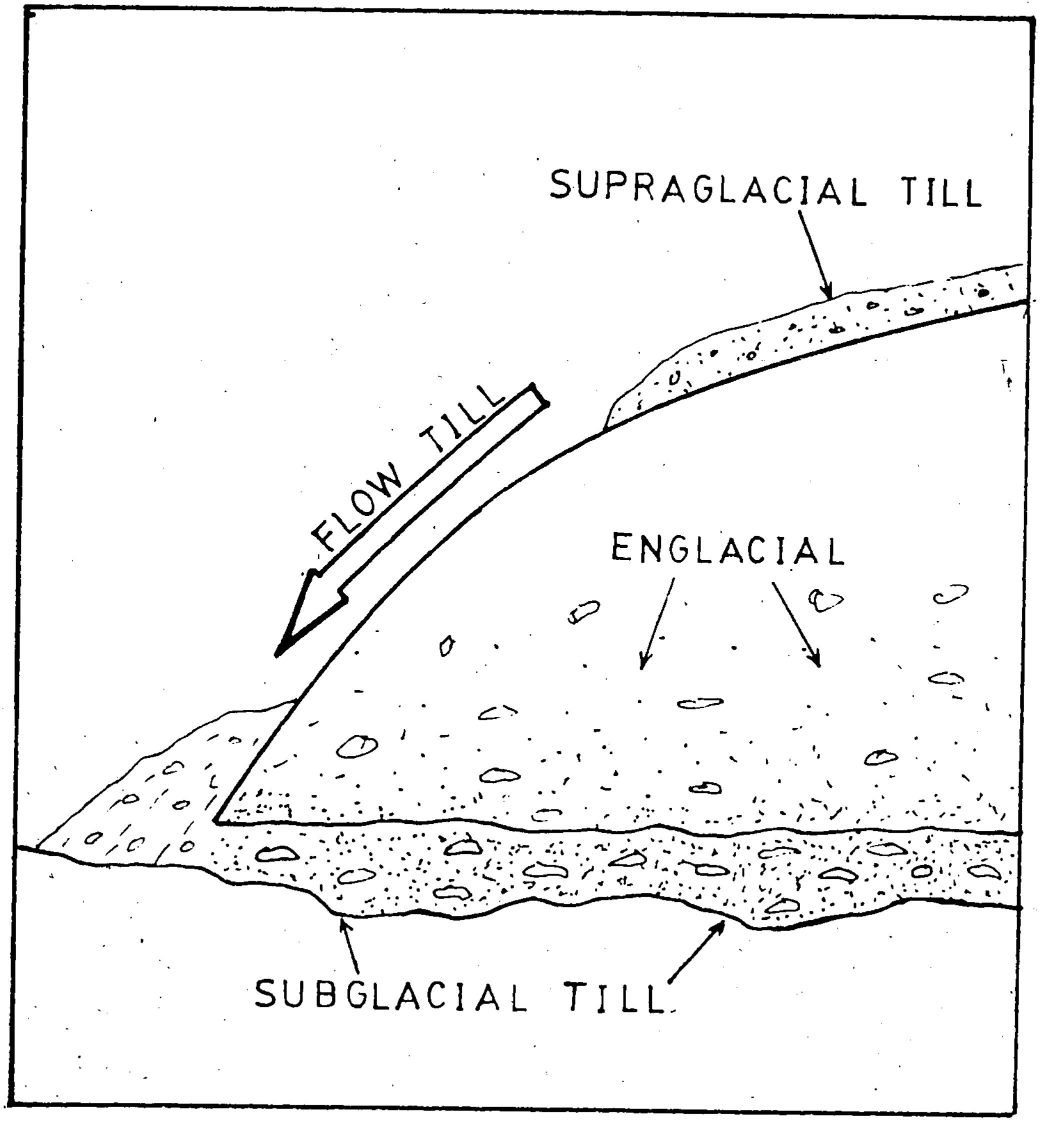

Figure 2.1 Acquisition, transportation and deposition of tills by a glacier

3. SOIL FABRIC AND ENGINEERING BEHAVIOR

3.1 Introduction

Fabric can be defined as the distribution, shape and sizes of particles and voids in a sediment. Structure has been defined as the property of soil which provides its integrity. Depositional processes control the primary fabric which is subsequently modified by secondary processes such as consolidation, shear, and weathering. The same sediment may exhibit different fabrics even though the density and void ratio are the same (Gillott, 1980). When external forces are applied to a soil fabric, mechanical and interparticle forces control its behavior. Interparticle forces are especially important in fine grained sediments.

3.1.1 Total Fabric of Glacial Tills

Recent studies of the total fabric of tills (McGown and Radman, 1975; Derbyshire et. al., 1976) have shown that various units of bedding, stress relief and shear features exist in many lodgement and melt out tills. The orientation and extent of these features would be expected to modify the total and effective shear strength, deformation and consolidation characteristics of these soils. Their occurrence is directly influenced by deposition and post-depositional processes, to which the tills have been subjected. In order to evaluate fully the effects of soil fabric on the engineering properties of a glacial till, it is necessary to analyze the total fabric of a specific till landform. The total till fabric includes a wide range of features of both primary and secondary origin including folds, thrusts, fissures (the macrofabric), disposition of clasts (the mesofabric) and organization of the matrix (the microfabric).

While the macrofabric features probably have the greatest influence on the in-situ engineering properties, microfabric features can also produce significant effects. Marsland (1982) suggests microfabric studies can be used in the following ways:

1. As an indicator of the degree of sampling disturbance.
2. To provide a better overall assessment of the strength along discontinuities.
3. To reveal the presence or otherwise of microfissures and their nature.
4. As a qualitative indication of the frictional properties of the soil.
5. As an indication of their stability under shear.
6. To provide important evidence of the possible environments at the time of the deposition.

The microfabric investigation undertaken for this thesis used these considerations as a guideline.

3.1.2 Soil Structure Models

Casagrande (1932) used the soil structure model for clayey marine soils which consisted of silt grains, clay particles and flocculated colloidal particles. The relative degree of consolidation depended on the location of the clay. The clay present in the large voids between the silt particles is relatively lightly consolidated when compared to the flocs compressed in the small gaps between the silt particles. Casagrande referred to the highly compressed spots as bond clay and noted that they may develop enough strength to make the soil as a whole display considerable strength.

Collins and McGown (1974) suggest a model of soil structure in which aggregations are linked by connections consisting of short or long chains of particles joined in a stepped face-to-face or edge-to-edge fashion. Intrafabric micropores occur within the aggregations and interfabric macropores occur between the aggregations. Aggregations are defined as particle assemblages which were observed to act effectively as individual units within the microfabric. Their size, shape and internal organization are variable.

A soil model proposed by Yong and Sheeran (1973) consists of a structural framework of soil peds comprised of clusters and domains of fine particles. Macropores exist as interfabric unit pore spaces whereas micropores refer to pore spaces established within fabric units by individual particle arrangements. The application of external stress to the soil fabric will cause movement along ped interfaces in addition to deformation of the individual peds. The sand grains and their associated bond clay form a structural framework within a till.

Sergeyev, et. al. (1980) proposed two microstructure models, skeletal and matrix, which are often present in silty clayey soils. A matrix micro structure soil is distinguished by the presence of a continuous, unorientated clay matrix which contains irregularly arranged inclusions of large silt and sand grains. These soils usually contain not less than 15 to 30% clay minerals with a comparatively low silt content.

A skeletal micro structure constitutes a more open, uniform, porous skeleton composed primarily of fine silt grains. The clay material often accumulates on the surfaces of grains as a continuous layer or at the contacts of silt particles, forming a kind of 'bridge' bonding the grains to one another.

A glacial till initially deposited in a slurry mode can be represented by the model shown in Figure 3.1. The sand and large silt grains form a matrix microstructure with the finer silts and clays acting as the matrix. The relative degree of consolidation is represented by the distance between the finer silts and clays. The locally high normal stresses associated with the clasts produces highly compressed zones of bond material. The clasts and adjacent bond material form a unit of soil fabric termed soil peds.

During deformation under low confining pressures these peds would act as individual particles and dilation would probably occur. Under higher confining stress however, the individual peds would not be allowed to dilate and shear would occur through the clayey silt matrix. Therefore, under low normal stress the peds interact with each other, and produce a frictional resistance higher than would normally be expected from a clayey silt.

The volume change or pore pressure response of a relatively rapidly deforming till would be largely controlled by fluid flow through the macro pores. Flow through these micro pores would be relatively minor due to their low conductivity and the existence of strong interparticle forces. The contribution of micro pores would only be significant in sustained loading such as in creep or secondary consolidation.

Under applied stresses the framework of sand and large silt grains would form simple arches, an arrangment mechanically very efficient in resisting load. Pusch (1973) concludes that the many small local regions of open, flocculated clay particle groups of the soft clay type that exist in boulder clays are due to this arching action of soil peds.

3.2 Soil Composition and Engineering Properties

3.2.1 Contribution of the Matrix

Tills may be represented as a mixture of fine and coarse particles occuring in different proportions. Fines are defined as material less than 0.076 mm in diameter. McGown and Derbyshire (1977) classified tills into three groups. (a) Granular tills have less than 15 per cent fines and their engineering behavior is largely controlled by the coarse fraction. (b) Well graded tills have between 15 and 45 percent fines. In such mixtures the coarse and fine components interact and each contributes to the engineering behavior. (c) Tills with a fine fraction greater than 50 percent are classified as matrix dominated tills. The matrix material of these tills controls the engineering behavior by preventing direct interparticle contact between the granular particles. Since there is a tendency for the clay particles to coat the granular particles, the fine particles may exert a significant influence on the soil properties even though they are present in relatively small amounts.

The intact Nipawin and Edmonton tills have fine fractions of 55 and 50 percent respectively and according to the above classifications their engineering behavior would be strongly controlled by the matrix material. If the fine and coarse fractions from both tills are separated, the soil composing the fine fraction has a silt content of approximately 60 to 70% and a clay content of 30 to 40%.

Such a material is termed a clayey silt.

Water is strongly attracted to clay mineral surfaces, whereas nonclay minerals have little affinity for water and do not develop plasticity even when in finely ground form. As a first approximation, it can be assumed that the majority of the water in a soil is associated with the clay phase (Seed et. al., 1964) thus small changes in the overall water content of a till will result in large changes in the water content of the clay phase. Since the strength of a clayey soil is directly proportional to its water content, the strength of matrix dominated tills is highly sensitive even to small variations in overall water content. This has been observed by Marsland (1982).

The fabric of the clayey silt matrix would largely be controlled by the mode of deposition and pore fluid composition, hence, the engineering behavior of a matrix dominated till would also be expected to be dependent on these factors.

3.2.2 Contribution of the Granular Phase

Paduana (1965) tested the effect of granular content on the strength of clay-sand mixtures. He found that in a mixture with 0 to 50% sand content the strength of the soil is primarily controlled by the clay phase. As the sand content increases and the individual grains approach each other the granular phase contributes more to the overall strength of the soil. With a sand content from 70 to 100%

the strength behavior of the soil is dominated by the sand phase. He further states:

> "that the sand grains, by some mechanism, interfere with the consolidation of the clay in soils, even when the clay more than fills the voids of the sand phase. To carry this point to a logical conclusion: in preventing the consolidation of the clay by some mechanism, whatever it may be, the sand grain must by the very same mechanism also contribute to the overall strength of the soil, even when the clay more than fills the voids in sand."

It is likely that the mechanism that Paduana refers to is the arching action of the sand grains (Pusch, 1973). As previously discussed (Section 3.1.2), the arching effect of sand and large silt grains causes the formation of lightly overconsolidated pockets of matrix material within the soil.

3.3 Postdepositional Processes Effecting Engineering Properties

3.3.1 The Effects of Freeze-Thaw

During the freezing process, large negative pore water pressures develop which occasionally cause shrinkage cracks to form. The shrinkage cracks, which are similar to dessication cracks, can lead to an increase in the permeability of the soil mass upon thawing.

As mentioned previously, in a matrix dominated till both the compressibility and the permeability are controlled by the fabric of the clay and silt particles. Freezing and thawing this type of soil results in a reduction in void ratio due to the collapse and rearrangement of the clay domains to a more dispersed structure (Chamberlain, et. al., 1979).

Marsland's (1982) studies on North Sea clays indicate that freeze-thaw caused overlapping ped structures to develop. Associated with these structures were microfissures which provide inherent sources of weakness along which more extensive discontinuities can propagate.

3.3.2 Changes in the Water Content

The lowering or raising of the water table imposes a complex stress history on a soil mass. When the water table is lowered, substantial suction pressures can be created in the partially saturated soil. The suction pressure will cause an increase in effective stress and the soil will consolidate. Boulton (1976) found variations in till void ratios from 0.69 to 0.35 due to the effects of drying only.

Soderman (1970) reported that the lowering of the groundwater table in the geologic past had caused increased consolidation to occur in the upper portions of the St. Clair till deposit. Adams (1970) found a variation in engineering properties related to moisture content changes; while May and Thomson (1977) noted:

> "a distinct trend is evident for higher strengths t
> be associated with lower moisture contents"

in the Edmonton till. They also found a large scatter i moisture content data throughout the deposit indicating the deposit was not drying uniformly. It is possible th differential drying is associated with the "soft" zones the Edmonton till (Thomson, et. al., 1982).

3.4 Development of Shear Zone Structures in Glacial Till

Overconsolidated clays during shear develop a narr failure zone along which the major displacements occur (1972). These failure zones vary in thickness depending the extent of deformation and the type of material being sheared. Morgenstern and Tchalenko (1967a,b) proposed a model explaining the development of shear zones in overconsolidated clay. Three stages were suggested that occur in the development of the failure zone. In the fir stage, en-echelon Riedel slip discontinuities form, incl at half the angle of internal friction to the direction shear. When further movement along the Riedel discontinuities becomes kinetically impossible, a new se discontinuities is formed. They are called thrust slip discontinuities and form approximately parallel to the direction of shear and in the opposite direction to that the shear movement. In the third stage of shear the Ried and thrust slip discontinuities coalesce and form a continuous principal slip surface. Recently it has been suggested that the formation of these discontinuities is the result of the clay failing by tension and not by shear (Vallejo, 1982).

3.5 Scanning Electron Microscope Observations of Tills

McGown (1973) used the SEM to study the matrix of several glacial ablation tills. The soils were multimodal having a predominantly fine fraction acting as a matrix in which the coarse particles were randomly distributed. The clay and fine silt sizes were arranged in a number of structural forms, with many of these structures exhibiting a porous arrangement. e particle arrangements were similar to structural forms postulated by Dudley (1970) and noticed by Barden et. al. (1973) in collapsing soils. Microfabric observations on a subglacial meltout till from Southern Norway revealed an interlocking framework of fine sands and coarse silts clothed with silt and clay particles.

Lodgement tills often present a wide range of particle sizes and are frequently anisotropic, fissured or jointed or all three. Their microfabrics show a moderate to strong parallelism between the clay and silt particles and the surfaces of the entrained clasts.

Marsland et al. (1982) conducted a study of fabric features found in North Sea deposits. The majority of the deposits were composed of low activity clays which were primarly of glacial origin. They observed a range of freeze-thaw and dessication features exemplified by platey ped-like structures and microfissures.

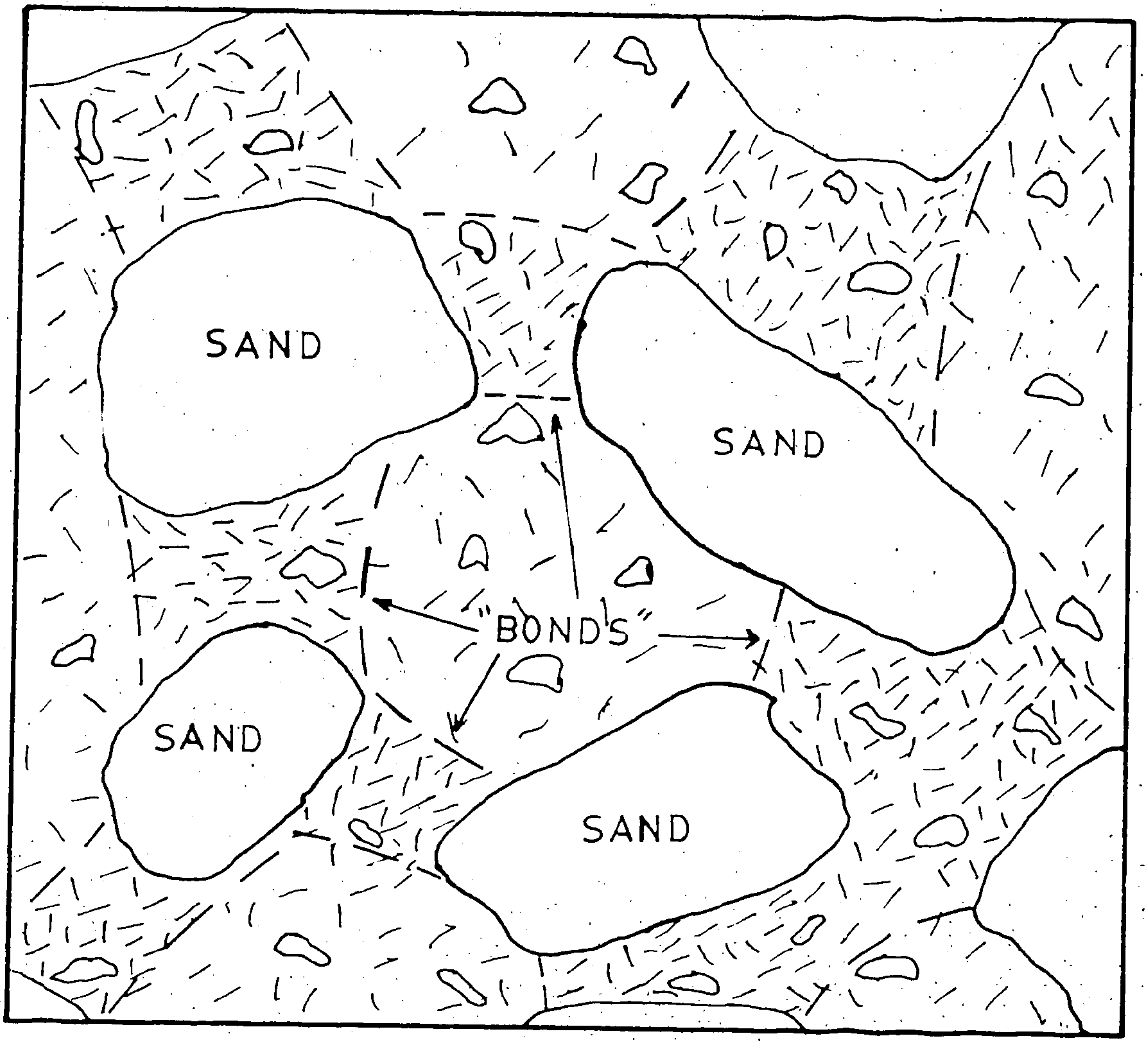

Figure 3.1 Proposed Microstructure of Matrix Dominated Tills

4. SCANNING ELECTRON MICROSCOPE

4.1 Introduction

This chapter presents the observations made during the scanning electron microscope study. Nipawin and Edmonton till samples were first studied in order to obtain a comparison between the intact materials. The intact fabrics were then compared to the fabrics of the reconstituted samples. To understand the effects of fabric upon the engineering behavior of the materials, specimens from sheared samples were observed. Typical micrographs of the various fabrics observed during the study are included at the end of the chapter.

4.2 Description of the Tills Studied

4.2.1 Intact Nipawin and Edmonton Tills

Little difference was observed between the microfabrics of the intact Nipawin and Edmonton tills. The fabrics are relatively open and exhibit macro as well as micro drainage pores. In areas where large sand grains are absent, the microfabric appears to be consistent and open in appearance. In the vicinity of sand grains however, a more closed fabric is found which is orientated around the grain.

Both tills are composed mainly of a clayey silt matrix in which sand particles are embedded. The sand content of the tills appears to be primarily composed of quartz and carbonate grains. The quartz grains are solution pitted and subangular in shape. Occasionally conchoidal fracture surfaces were observed, however they were rare. The carbonate grains, presumably limestone or dolomite, are rounded in shape. The silt matrix constituents are of two types; flat micaceous flakes and subangular silt grains. The clay particles occurred as separate domains or as coatings on the coarse silt and sand particles.

Based on the premise that the pore size distribution of a soil depends on the particle size distribution and structural influences, it appears that the two pore classes may correspond to two levels of structural arrangements. The micro pores may conform with the silt and clay domains whereas the macro pores may be accordant with a larger ped particle fabric.

4.2.2 Reconstituted Samples

Nipawin till materials reconstituted by remoulding or from a slurry are denoted as Nipawin Till #1 and #2 respectively. Nipawin Till #3 samples were reconstituted from a salt water slurry. Both reconstituted Edmonton Till samples were formed from a slurry, however, they differ in preconsolidation pressures. Edmonton Till #1 and #2 samples were preconsolidated to 490 kPa and 800 kPa respectively.

Since the reconstituted tills were formed from the intact Edmonton and Nipawin tills, they are compositionally identical to them. The matrix material of the tills reconstituted from a slurry show an open fabric similar to the intact tills. The bond clay is poorly developed and can only be noticed by the orientation of fabric in the immediate vicinity of the grain.

Nipawin Till Sample #3, consolidated from a salt water slurry, shows an open structure similar to the other slurried samples except that there is a noticeable difference in the clay particle arrangement. The clay particles tend to form flocculated domains rather than coat the silt or sand particles. These domains do not appear to significantly alter the overall fabric and were not expected to influence the engineering behavior during shear significantly.

The remolded reconstituted samples (Nipawin Till #1) exhibited a striking difference in fabric when compared to reconstituted slurry samples (Nipawin Till #2). The remolded samples had a smeared texture and micro pores were not as evident. Macro pores were very well developed probably due to the high pore pressures generated during remoulding. The matrix material of the remoulded samples had a closed fabric.

4.3 Observations of Shear Structures

In order to investigate shear structures in the glacial tills, the scanning electron miscroscope was used to examine representative samples previously sheared to failure during triaxial testing. From Plate 4.4a, it is seen that discrete slip planes are formed in a relatively thin zone. The clay and silt particles seem to be highly disturbed in this zone, whereas the remainder of the sample has a fabric indistinguishable from that of undisturbed specimens. Shear displacements are seen to occur in these discrete zones throughout the sample rather than along a generalized slip surface. The slip planes are highly irregular and exhibit frequent undulations. The clay and silt particles within the slip surface are highly oriented in the direction of shear.

The study of the sheared specimens revealed that the sand grains appeared to have no direct contribution to the shear strength of the soil. The sand grains were found to be scattered throughout the sample and only rarely did sand grains approach each other. Plate 4.4b shows a shear zone located beneath a rather large sand grain. The matrix around the sand grain is virtually undisturbed indicating little or no movement of the sand grain during sample deformation.

4.4 Description of Micrographs

1. Plate 4.1a Intact Nipawin Till

   Photo shows an interlocking framework of sands and coarse silts which are coated with fine silt and clay particles. Note the lack of direct contact between the sand clasts and the highly accordant microfabric in fine silt and clay on cast surface left by sand grain (lower left corner).

2. Plate 4.1b Intact Nipawin Till

Large sand clast within the fine silt and clay matrix. The fabric adjacent to the clast appears to be aligned around the clast.

3. Plate 4.1c Intact Nipawin Till

More diffuse microfabric in area between large clasts. The fabric is more open and is randomly oriented.

4. Plate 4.1d Intact Nipawin Till

Close up view of Plate 4.1c. Fine silt and clay particles are arranged in an open edge to face and edge to edge arrangement. Drainage is controlled by micropores.

5. Plate 4.1e Nipawin Till #1

General view of the fabric shows a closed "smeared" fabric. Silt and clay particles are crudely aligned with the plane of the photo.

6. Plate 4.2a Nipawin Till #1

A large sand clast within the matrix is shown. The fabric is compressed against and around the clast. Note the macropore in the lower right of the photograph.

7. Plate 4.2b Nipawin Till #1

Close up view of Plate 4.2a showing the interface between matrix and the sand clast. The silt and clay particles are "crushed" against the sand grain producing a closed fabric.

8. Plate 4.2c Nipawin Till #2

Close up view of a large well developed macropore.

9. Plate 4.2d Nipawin Till #2

Open nature of the fabric can be seen. Development of "bond" material is poor. Compare the slurry fabric with Plate 4.1c. This fabric more closely represents that of the intact till.

10. Plate 4.2e Nipawin Till #3

Consolidation from a salt water slurry produced a highly flocculated clay fabric. The clay particles tend to aggregate into flocculated groups.

11. Plate 4.3a Intact Edmonton Till

The silt and clay particles in the Edmonton Till are evenly spread throughout the matrix and the general structure was open with the sand size and coarse silt particles being linked and supported by the finer particles.

12. Plate 4.3b Intact Edmonton Till

Close up view of fabric from Plate 4.3a. The clay particles are arranged in an open edge to edge and edge to face arrangement. Compare with the Nipawin Till fabric in Plate 4.1d.

13. Plate 4.3c Intact Edmonton Till

A view of a subangular quartz grain on the intact Edmonton till. Note the solution V-etches and overgrowths on the cleavage surface. The condensed nature of the matrix adjacent to the clast can be seen.

14. Plate 4.3d Edmonton Till #1

This view of the reconstituted slurry sample shows an open fabric similar to the intact Edmonton Till. Some bond clay can be seen on the large silt grain in the centre of the photo.

15. Plate 4.3e Edmonton Till #1

Close up view of the fine silt and clay adjacent to a large sand clast.

16. Plate 4.4a Nipawin Till Sheared

Shear zone observed within intact Nipawin Till. Shear zone is narrow and adjacent fabric is undisturbed. Shear occurred in open clay-fine silt matrix and shows frequent undulation.

17. Plate 4.4b Nipawin Till Sheared

Shear band in intact Nipawin Till. The large sand clast shows no visible sign of movement. Note that shear occurred away from clast.

18. Plate 4.4c Nipawin Till Sheared

Indentations of two sand clasts left in the matrix. The fabric adjacent to the clasts is highly accordant. During shear the sand clasts did not rotate or come into contact.

19. Plate 4.4d Nipawin Till #1

Shear zone in the remoulded material. Some alignment of large silt sizes. It is possible that the shear occurred along a discontinuity produced during the remoulding process.

20. Plate 4.4e Nipawin Till #1

Close up of shear surface from Plate 4.4d. Note the parrellism of the small silt and clay sizes. A medium silt particle can be seen in the background sticking out of the surface.

Plate 4.1a Nipawin Till

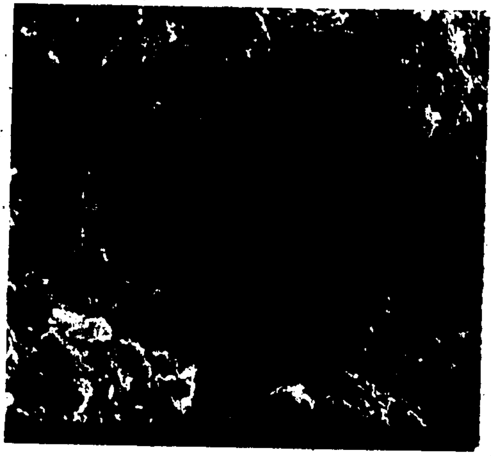

Plate 4.1b Nipawin Till

Plate 4.1c Nipawin Till

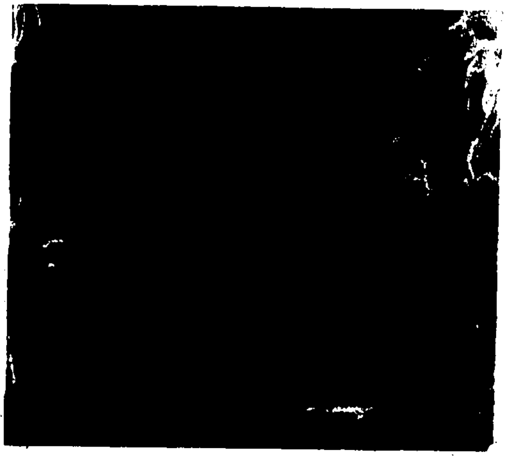

Plate 4.1d Nipawin Till

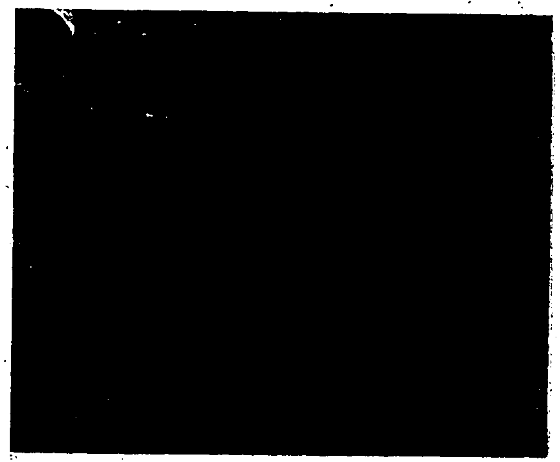

Plate 4.1e Nipawin Till #1
Scale Bar = 10 microns

Plate 4.1 Scanning Electron Micrographs

Plate 4.2a Nipawin Till #1
Scale Bar = 200 microns

Plate 4.2b Nipawin Till #1
Scale Bar = 4 microns

Plate 4.2c Nipawin Till #2

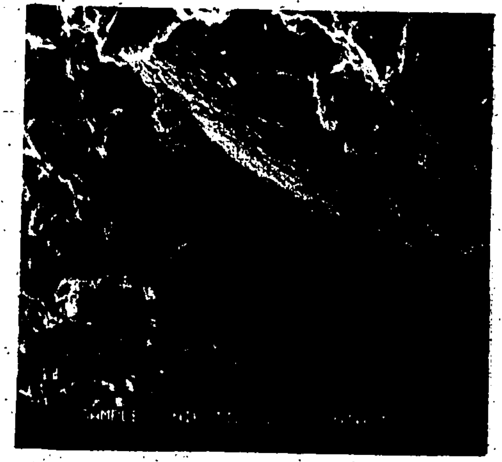

Plate 4.2d Nipawin Till #2

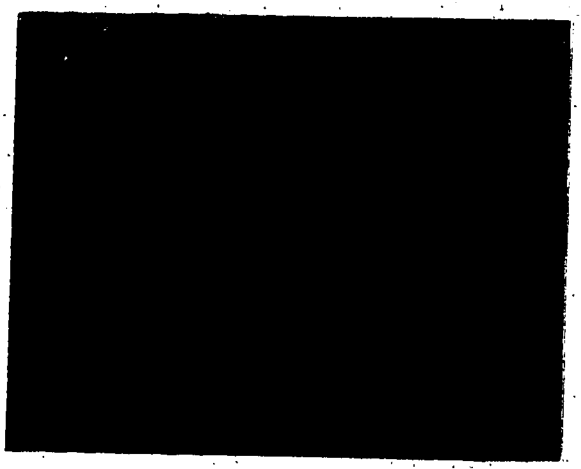

Plate 4.2e Nipawin Till #3

Plate 4.2 Scanning Electron Micrographs

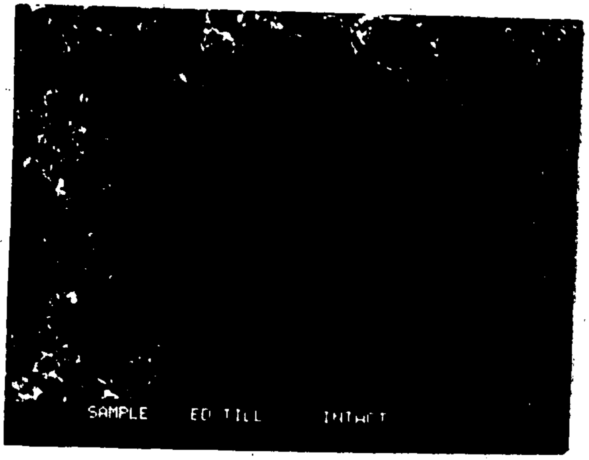

Plate 4.3a Edmonton Till

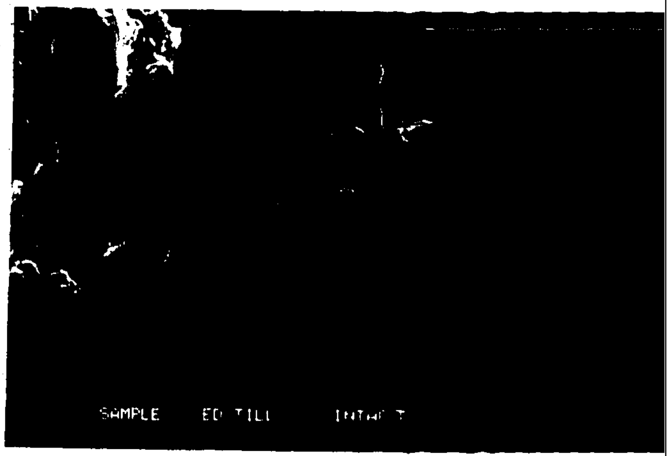

Plate 4.3b Edmonton Ti

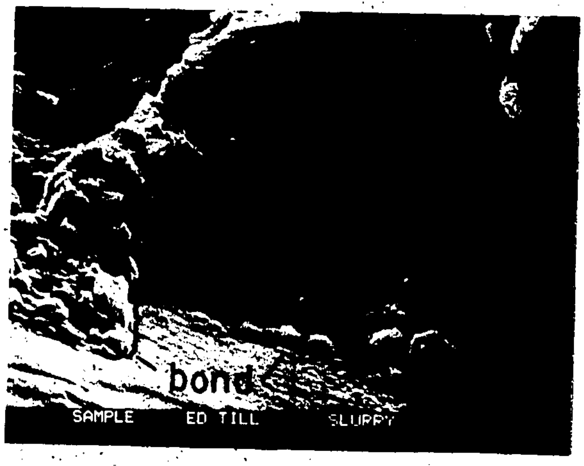

Plate 4.3c Edmonton Till

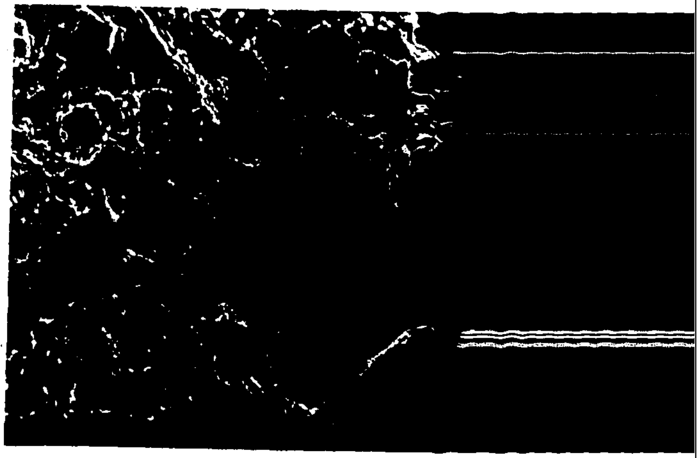

Plate 4.3d Edmonton Til

Plate 4.3e Edmonton Till #1

Plate 4.3 Scanning Electron Micrographs

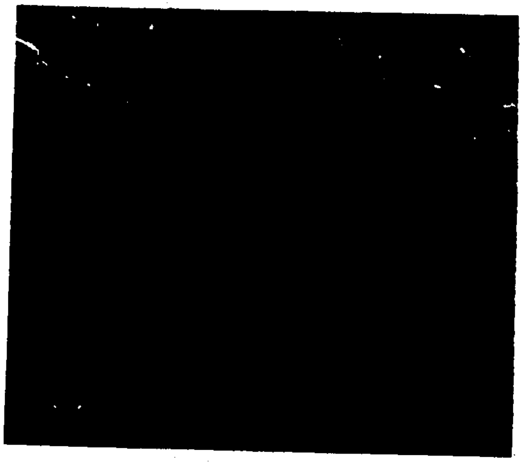

Plate 4.4a Nipawin Till

Plate 4.4b Nipawin Till

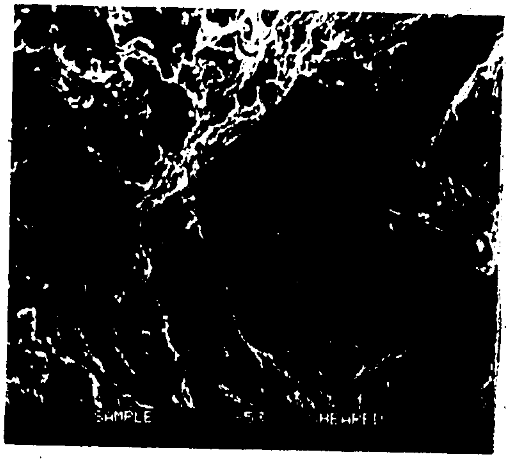

Plate 4.4c Nipawin Till

Plate 4.4d Nipawin Till #1
Scale Bar = 20 microns

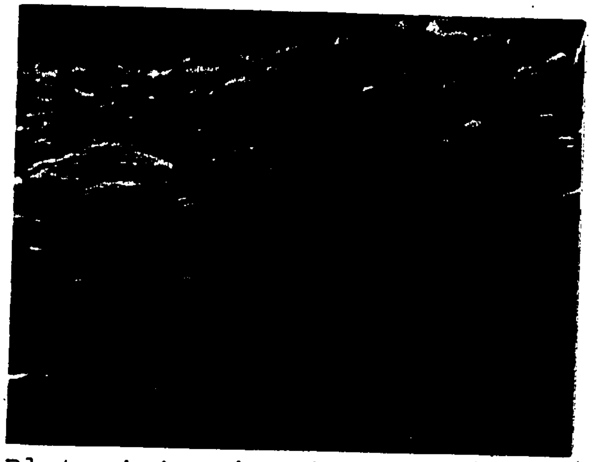

Plate 4.4e Nipawin Till #1
Scale Bar = 4 microns

Plate 4.4 Scanning Electron Micrographs of Sheared Samples

5. LABORATORY TESTING AND RESULTS

5.1 Introduction

This chapter includes a brief description of the sampling and testing programs undertaken for this investigation. A more detailed description of the experimental procedures are given in Appendix A.

During planning of the test program, careful consideration was given to the possible types of experiments, drainage conditions and test instrumentation, which could be used to investigate the influence of soil structure on the strength behavior of tills. The inherent disadvantages of the direct shear test, i.e. unequal stress distribution, predetermined failure plane, and difficult-to-control drainage conditions, encouraged the use of triaxial equipment. The triaxial apparatus has been used extensively in research work and allows control of drainage conditions and the possibility of measuring pore pressures.

The behavior of soils subjected to cyclic loading may differ considerably from their behavior during static loading. There are numerous natural situations in which the durations of cyclic loads are such that little or no drainage of the pore water takes place during the application of the repeated loads.

A study was carried out on the behavior of glacial tills when subjected to cyclic loading. Previous work showed that a soil will stabilize or fail under repeated loadings (Anderson, 1976; Mitchell and King, 1977). These studies showed that the material behavior was related to many factors including mineralogy, overconsolidation ratio, ratio of magnitude of the load applied to failure load, and the number of load cycles. The purpose of the program was to investigate how fabric features effect cyclic loading behavior of glacial tills.

The second part of the chapter presents the results of the laboratory experimental program.

5.2 Sampling

5.2.1 Sampling Locations

Samples from the Nipawin site were obtained from block samples obtained in 1979 (Sego, 1980). Edmonton Till samples were obtained by coring the floor of an excavation at 106 St. and 100 Ave. in Edmonton. Reconstituted samples were obtained from block samples recovered from large diameter odemeters which will be described subsequently.

5.2.2 Sampling Techniques

The dense stony nature of the soils required that sampling be carried out using concrete coring equipment. The coring apparatus used is shown in Plate 5.1. The unit is portable and can be easily moved around a site by two people. Only one person was required during laboratory coring on block samples.

At the Edmonton site, different coring procedures were experimented with in order to optimize core recovery. It was found that larger diameter core barrels achieved greatest core recovery. Typical core barrels used on this project are shown in Plate 5.2. The length of intact sample recovered at the Edmonton site was restricted by horizontal joints which occured at 100 to 125 mm intervals. Therefore core barrels with diameters greater than 100 mm were not used in order that a sample length to diameter ratio of 1 could be achieved.

During the drilling operation, compressed air was used as a drilling fluid to remove cuttings and cool the core barrel. It was found that the efficiency at which cuttings were removed greatly influenced core recovery. Initially an air pressure of 100 to 140 kPa was used however the removal of cuttings proved to be inefficient and core recovery was poor. The [illegible] line was then attached to an onsite compressor capable of 700 kPa. Core recovery improved dramatically and this compressor was used for all subsequent on site sampling.

In the laboratory the core barrels were further modified to enhance recovery. The circulation gaps located along the cutting surface were increased in size by grinding (Plate 5.2). This modification greatly enhanced the removal of cuttings. Laboratory air pressure varied between 400 and

500 kPa. Using the modified core barrels and adequate air pressure it was found that optimum core recovery occured when the coring machine was set on maximum revolutions per minute (750 RPM) and the core barrel was allowed to cut its way through the material using only the self weight of the machine as the downward driving force.

The laboratory coring method produced a intact sample with little sample disturbance. Some slight dessication of the exterior surface of the sample was caused by the air circulation, however, only the outer few millimeters of the sample were affected. When sampling in tills with a moisture content greater than 15% the coring operation tended to smear clayey material on the exterior of the sample. In these instances the sample was trimmed using methods recommended by Bishop and Henkel (1957).

Visual examination of the samples indicated that the coring operations did not disturb the pebbles within the core but cut through them. Moisture contents and densities of the cored samples compared favorably with those of the intact blocks which indicated that sample disturbance was minimal. The sample was trimmed to its desired length using a diamond rock saw. Special sample holders were designed in order to hold the sample firmly and thus minimize possible disturbance during this trimming operation (Plate 5.1).

5.2.3 Reconstituted Samples

To produce different microstructures two methods were adopted for reconstituting samples. Consolidation from a slurry, was the method used to simulate tills deposited at moisture contents above their liquid limits. The second method was to remould a sample at a water content below its liquid limit. Remoulding was accomplished by manually kneading the soil into the oedometer.

Reconstituted soil blocks were produced by consolidation in 15 and 30 cm diameter oedometers which were used throughout the laboratory program (Plates 5.4 and 5.5). Plate 5.3 shows the experimental setup for consolidation of the reconstituted materials. The vertical load was applied by means of a bellofram rolling diaphragm air cylinder. Either a linear voltage differential transducer (LVDT) or a dial gauge was used to measure vertical deformation. In addition to drainage at the top and bottom of the oedometer, the insides of the oedometers were lined with filter paper to further accelerate drainage.

The 30 cm diameter oedometer was lined with teflon to minimize friction along the sidewalls. The glue binding the teflon sheet to the metal wall deteriorated with time and bulging of the material resulted. Also, clasts within the material often scoured the teflon. The irregularities in the teflon surface occassionally impeded the progress of the loading piston as consolidation occurred. Therefore when the 15 cm diameter oedometer was designed, a teflon coating was not used. Instead the body of the oedometer was constructed from chromed hardened stainless steel. This material was hard enough to resist clast scours, provided a smooth surface, and the stainless steel construction inhibited the development of rust. The 15 cm diameter oedometer was preferred because greater consolidation pressures could be used and the shortened drainage path reduced the time for consolidation.

There is little likelihood of the successful matching of reconstituted behavior to that of intact behavior for clay soils. Bishop and Green (1965) compared the undrained shear strength and effective stress paths in undrained compression for intact and reconstituted London clay samples consolidated from a slurry. They found little similarity in behavior between the two types of samples.

Greater success has been found between intact and remolded glacial tills due to their wide grading and low sensitivity. Skempton and Brown (1961) reported that:

> "owning to the presence of occasional large stones it was necessary, in some cases, to form the specimens by packing the material into brass tubes, without change in water content and with zero air voids. As usual with boulder clay no essential difference in strength was found between these specimens and those cut from the undisturbed cores."

Vaughan et. al. (1975) concluded that remolded specimens of boulder clay could be used for the determination of undrained strength. Hight et. al. (1979) found agreement with undrained shear strengths between the intact lower Cromer Till and samples consolidated from slurry.

Considering the well graded nature of the Nipawin and Edmonton tills it is probable that remoulded samples subjected to similar stress histories would exhibit strength properties similar to the intact tills. The complex stress histories of both tills since their initial deposition is not known. A simplified stress history was therefore chosen for the reconstituted samples. Initially all the reconstituted samples were consolidated anisotropically in the large oedometer to a pressure of 500 kPa to simulate ice and overburden load. The consolidated soil blocks were removed and cores were extracted. Before triaxial shearing, the samples were consolidated isotropically to 800 kPa. The final consolidation pressure was chosen because it approximated the preconsolidation pressures found in the intact samples.

5.3 Sample Preparation for Fabric Analysis

The effectiveness of any method for observing soil fabric depends on the reliability of preserving the overall structure during preparation. In scanning electron microscopy, the pore fluid must either be removed or replaced during sample preparation. There are six basic methods employed for removal of the pore fluid:

1. oven drying;
2. air drying;
3. humidity drying;
4. substitution drying;
5. freeze drying; and
6. critical point drying.

Tovey and Wong (1973) concluded that freeze drying and critical point drying cause less sample disturbance than other methods and should normally be used. Critical point drying is considered to be the preferred method for preparing specimens of wet overconsolidated soils. The freeze drying method however has been satisfactorily implemented for a wide range of soils (Gillott, 1969; Sergeyev et. al., 1980).

For this study the freeze drying method was chosen. Small samples in the order of 10 mm in diameter and 10 mm thick were frozen using liquid nitrogen. The samples were then mounted on a -45°C cold plate and a vacuum of 10^{-5} Torr was applied (1 Torr = 1 mm of Hg0°C). Sublimation of the frozen water was allowed to occur over a period of 24 hours. Freeze drying does cause some slight disturbance possibly in the form of heave as the pore water is changed to ice with the accompanying 9.16% expansion.

5.4 Index Tests

Index tests were conducted to determine grain size distribution, density, specific gravity, void ratio, Atterberg limits and moisture content of the soils. For these tests, procedures outlined in ASTM D442-547, D422-63, D423-66, D424-547, and D854-52 were used. Densities were measured from both intact and reconstituted block samples and from all core samples. Moisture contents before and after testing were also determined for all samples.

5.5 Static and Cyclic Triaxial Testing

5.5.1 The Use of Free Ends

In order to equalize the distribution of stresses and pressure throughout the depth of the sample, free ends were employed at the top and bottom platens (Plate 5.7). Rowe and Barden (1964) present a discussion on the importance of free ends in triaxial testing. Free ends were particularly useful during the testing program in that they allow the use of samples having a height to diameter ratio of unity rather than the general ratio of 2 to 1. In addition, a greater percentage of usable cores were available when the jointed natural tills were being tested. Further,

> "free ends perform better with shorter samples, probably because the greased ends receive a greater lateral thrust to conduct them through the initial

> friction peak observed in the shear box tests, and because shorter samples are generally more stable. The requirement that a shear plane must not intersect an end paten is considered to be irrelevant. ...It is therefore suggested that a height to diameter ratios in the range of from 1.0 to 1.5 is suitable for many applications."
> (Rowe and Barden, 1964).

Some researchers have found that the [illegible] of fixed ends with L/D ratio of 2 and 'free' ends with L/D ratio of 1 give approximately the same strength (Olson, and Campbell, 1964; Bishop and Green, 1965). Others have reported that the strength is slightly reduced by using free ends (Rowe and Barden, 1964; Barden and McDermott, 1965; Duncan and Dunlop, 1968).

Marsland (1977) recommends an L/D ratio of 1.5 for stiff highly fissured clays. This ratio gives more freedom for failure to take place along favourably inclined planes of weakness than would be the case for specimens having a L/D ratio of 1. During the laboratory program samples were trimmed to L/D ratios of 1.2 to [illegible]. This allowed recovery of more samples from the available material and also insured that a L/D ratio of greater than 1 would be maintained even after straining the sample.

When free ends are used, there is an associated er in any measured axial deformation due to compression and distortion of the rubber and grease layers. The extraneous deformation is termed the bedding error. Sarsky et. al. (1982) concluded that bedding error is the result of:

> "two major agencies namely, initial lack of fit between the sample surface and the lubricated platen, and penetration of the grease and disc layer by the constituent particles of the specimen. The axial movement due to lack of fit occurs primarily during virgin loading, and it is due to irreversible movement of the surface particles and the viscous grease layer."

As a result of the diamond saw trimming operation, the sample ends were often slightly uneven and an initial lack of fit between the sample surface and lubricated platen existed. The magnitude of this bedding error is small, and it was not expected to adversely effect the strength parameters. The tills tested were composed of clayey silts and it is unlikely that bedding error due to penetration of the rubber and grease layer existed.

5.5.2 Consolidation

In order to insure complete saturation, a cell pressure of 620 kPa was applied to the sample while a back pressure of 586 kPa was applied. The sample remained at this state for a minimum period of 24 hours. A pore pressure response "B" test was carried out on all undrained test samples and isotropic consolidation was not carried out until a B value of at least 0.98 was achieved. This occassionally required an increase in the applied back pressure to improve the degree of saturation of the sample.

The next step was to apply a cell pressure such that for the given back pressure of 586 kPa or greater, the desired effective confining stress would result. When very high effective confining stresses were needed the back pressure would be reduced from 586 kPa accordingly. The reconstituted samples were all initially isotropically consolidated to an effective confining stress of 800 kPa to insure uniformity. They were then allowed to swell to the designated overconsolidation ratio.

During the consolidation process, volume changes were recorded at time intervals to enable the plotting of a consolidation curve. This consolidation curve was used with the procedure recommended by Bishop and Henkel (1957) to calculate the maximum displacement rate that could be used during shearing and still ensure full equalization of excess pore pressure within the sample.

5.5.3 Static Triaxial Testing

5.5.3.1 Static Failure Criteria

In examining the results of test data, it is sometimes difficult to assign a value to the strength of the soil or to decide at what point failure had occurred. Maximum stress difference, $(\sigma_1-\sigma_3)$max, is commonly used in practice since a more conservative result is obtained. For the majority of the undrained tests however, the deviator stress did not level off or decrease but continued to increase graduall Failure, defined as the maximum principal effective stre ratio $(\sigma_1'/\sigma_3')_{max}$, clearly indicated a failure point an therefore was adopted to define failure in this study.

5.5.3.2 Static Triaxial Procedure

During static loading, consolidated undrained and consolidated drained tests were carried out on undisturb and reconstituted samples. During undrained testing pore cell pressure measurements were taken utilizing an electronic pressure transducer. The applied deviator loa were measured by a load cell and an LVDT measured the displacement of the loading ram. The total stress path followed during the static loading tests was to increase with σ_3 remaining constant. The shearing of the samples w conducted using the steps found in Appendix A where detai of the apparatus can also be reviewed.

5.5.4 Cyclic Triaxial Testing

5.5.4.1 Soil Failure Modes During Cyclic Loading

Cyclic loading applied to a soil sample usually caus a volume reduction of the soil structure (compaction or consolidation). During undrained conditions, an increase porewater pressure accompanies deformation of the structu therefore, the effective stress on the samples decreases. the soil structure continues to deform, a condition of zer effective stress could arise.

Failure for a stress controlled test can be defined at a specified magnitude of strain developed in the sample. Failure can be defined when the permanent axial strain reaches a specified value or the cyclic strain reaches the specified value. Cyclic strain is defined as the maximum strain recorded during a loading cycle. For the purposes of the laboratory program, a cyclic strain of 2% was assumed to define failure.

5.5.4.2 Cyclic Triaxial Procedure

To study the effects of repeated loading on the various till fabrics, a cyclic testing program was initiated. The samples were placed in conventional triaxial equipment with rotating bushings. Special equipment was designed for load controlled cyclic loading tests (Plate 5.8).

Directly above the triaxial cell, a Bellofram rolling diaphragm air cylinder was mounted to provide the axial load. An LVDT was connected to the loading ram to measure axial strain. Triaxial samples were prepared identical to static samples using the procedure found in Appendix A.

The cycle interval was chosen to be 25 seconds, a period which would roughly simulate wave or wind loading upon a structure. In addition, this period was necessary to insure near maximum equalization of pore water pressure. When a load cycle starts, the timing device opens the valve and compressed air at a predetermined pressure is allowed access to the bellofram. When the valve is closed the bellofram is exposed to atmospheric pressure and the load quickly dissipates. During the loading interval, the cyclic loading apparatus kept the deviatoric stress stable except for minor fluctuations which occurred when the apparatus was used near its maximum (500 kPa) capacity. The electronic timer had been designed for testing with very rapid cyclic loading periods. The 25 second interval setting was at its upper range and consequently the cycle period was found to fluctuate ±2 to 3 seconds throughout the test program.

The pore water pressure response was measured with an electronic transducer mounted at the base of the cell. For the tills under investigation, with Cv values from 0.5 to 5 m^2/yr., response times of 5 seconds were common. During the cyclic loading it was found that 95 % pore water pressure equalization occured within 10 seconds. The addition of free ends to the loading patterns is felt to greatly increase response time. To insure that porewater pressures could be followed closely in a variety of tills, the cycle period of 25 second duration was adopted.

5.6 Laboratory Results

5.6.1 Unified Soil Classification

The Nipawin Till was found to have a liquid limit of 30.3% and a plastic limit of 14.6%. The Edmonton Till results indicated a plastic limit of 15.8% and a liquid limit of 30.2%. These results agree favourably with previous work (May and Thomson, 1978; Sego, 1980). Using the Unified Soil Classification system, both soils are classified as CL, inorganic clays of low to medium plasticity.

5.6.2 Grain Size Distribution

Figure 5.1 shows the grain size curves for the intact Nipawin and Edmonton Tills respectively. The Nipawin Till is composed of approximately 35 per cent sand, 47 per cent silt, and 18 per cent clay sizes. An X-Ray diffraction test completed on the clay fraction indicates approximately 45 per cent of the clay size fraction is montmorillonite.

The Edmonton Till is composed of approximately 50 per cent sand, 35 per cent silt, and 15 per cent clay sizes. Pawluk and Bayrock (1969) indicate that the clay fraction is composed of approximately 40 to 45 per cent montmorillonite.

5.6.3 Specific Gravity

Specific gravity tests were conducted on both tills. The Edmonton Till had an average specific gravity of 2.65, whereas the Nipawin Till had an average bulk specific gravity of 2.68.

5.6.4 Triaxial Results

5.6.4.1 Strength and Deformation Parameters

The strength and deformation parameters of tills measured in laboratory experiments depend largely upon the following factors:

1. the grain size distribution within the soil and the composition of the various components;
2. the existing in-situ stresses and previous stress history of the soil prior to sampling;
3. the stress path followed during the testing;
4. the degree of disturbance during sampling; and
5. the presence of planes of weakness such as bedding and fissures.

Grading curves of the Nipawin and Edmonton tills are similar and both soils have similar mineralogical compositions. The reconstituted samples have identical grading and composition when compared with the intact material, hence this factor would appear to have little influence on the measured strength and deformation properties.

Little is known about previous and present stresses at either sampling site, therefore the extent to which this factor influences the test results is not known. To eliminate this factor, all the reconstituted tills were subjected to identical stress histories. During testing identical stress paths were used for all the samples.

The same sampling method was used during the entire test program and it was anticipated that the minimal disturbance was uniform throughout the sample preparation. During the sampling operation, if a plane of weakness did exist, the core broke at this point. Unfortunately this led to a testing program in which only the intact and probably the stronger portion of the till unit was tested.

5.6.4.2 Isotropic Consolidation

Four incremental isotropic consolidation tests were conducted during the laboratory program. Results from an isotropic consolidation test on intact Nipawin Till were obtained from a previous study (Sego, 1980). The tests were used to establish the coefficients of consolidation and compressibility characteristics of the various tills. All samples were 2.5 inches in diameter with the exception of the intact Edmonton Till sample which was 4 inches in diameter. The experimental Cv values versus the effective confining stress for which it was obtained are plotted in Figure 5.2. The coefficient of consolidation (Cv) of the tills tested ranges from 0.1 to 3 m^2/year.

The Cv values obtained are typical of values expected for glacial tills. All values decrease with increasing confining pressure. The data is consistent, however, it should be noted that the intact Edmonton till generally had the highest Cv values and the remolded Nipawin till had the lowest. The high Cv values rec ded for the intact Edmonton till may be due to sample size effects. McKinlay, et. al. (1975) have shown that Cv values increase with increasing sample size in lodgement tills.

Figure 5.3 is a plot of the void ratio versus logarithm effective stress for all the tills. The compression indices (Cc) resulting from these experiments are:

1. Nipawin Till = 0.029;
2. Edmonton Till = 0.031;
3. Nipawin Till #1 = 0.043;
4. Nipawin Till #2 = 0.099; and
5. Edmonton Till #1 = 0.098.

A consolidation test was not performed on an Edmonton till #2 sample since it differed from Edmonton till #1 sample only in preconsolidated pressure.

The compression index values are of some interest since they indicate the relative compressibility of the various till micro fabrics. The two tills reconstituted from a slurry have compression indices at least two and one half times greater than of the intact and remolded samples. This suggests that a less compressible structural framework exists in the intact and remolded samples.

5.7 Static Triaxial Results

5.7.1 Behavior During Shear

The deviatoric stress versus strain and porewater pressure versus strain for each sample is plotted in Figures 5.4 to 5.10. Deviatoric stress and volume change versus strain for the drained test are shown in Figure 5.11.

For similar confining pressures, the Nipawin Till samples achieved deviatoric stresses at failure that were approximately double the magnitude of the reconstituted samples. Deviatoric stresses for Nipawin Till #1 samples were slightly higher than those measured for Nipawin Till #2. The deviatoric stresses at failure for the Nipawin Till#3 samples were approximately 200 kPa lower than the Nipawin Till #2 samples at similar confining pressures.

A noticeable difference in porewater pressure response between the intact and reconstituted samples was noticed. The intact samples generally dilated during shear with the exception of two samples which were tested under high effective confining stress. The reconstituted samples commonly contracted during shear, with only the highly overconsolidated samples exhibiting dilatant behavior.

Tables 5.1 to 5.3 show the "A" values at maximum stress ratio for all samples sheared under undrained conditions. The average "A" values for the intact Nipawin Till was -0.25. The reconstituted samples have average "A" values of -0.13 and -0.1 for the remolded and samples prepared from a slurry respectively. Nipawin Till #3 samples had an average "A" value of -0.01. The average "A" values exemplify the greater degree of dilatant behavior for the intact Nipawin Till.

The Edmonton Till samples achieved deviatoric stresses that were approximately double the magnitude of the reconstituted samples for similar confining pressures. Edmonton Till #2 samples had slighly higher deviatoric stresses at failure than Edmonton Till #1. Examining Tables 5.1 to 5.4 , it can be seen that increasing preconsolidation pressure for the Edmonton Till #2 samples caused only a slight reduction in void ratio and water content. Therefore the materials would be expected to act similarly since they were reconstituted with identical depositional and stress histories.

The porewater pressure response was similar in all the samples tested. Under low effective confining stresses the porewater pressure decreases and under effective confining stresses greater than 400 kPa the porewater pressure increases during shearing of the samples. Table 5.1 to 5.3 indicate that all samples had "A" values ranging from -0.1 to -0.3.

An interesting phenomenon in porewater pressure response can be observed for the intact samples sheared under high effective confining stress. The porewater pressure increases during low strains, however at larger strains a decrease in porewater pressure is measured (Figures 5.4 and 5.8).

Considering the conceptual microstructure model suggested for matrix dominated tills (Figure 3.4), an explanation for this behavior can be postulated. During loading at high confining pressures the soil peds would be compressed against each other. At large strains the bond material will become fully compressed and additional straining will result in dilation. The strain at which maximum compression occurs is dependent on the initial development of the soil peds.

In a general way, the porewater pressure responses of the samples under high effective confining pressures correspond to those expected from loose sands. The porewater pressure response under low effective confining pressures is similar to that of dense sands.

5.7.2 Drained Test Results

A series of drained triaxial tests were conducted in order to study the stress-strain and volume change characteristics of the various tills under low confining stresses (Figure 5.11). Sample preparation was the same as for the undrained tests with the exception that volume changes were recorded manually from a calibrated burette. The samples were all subjected to an effective confining pressure of 35 kPa.

Deviatoric stresses and volume change characteristics for the Nipawin Till and Nipawin Till #2 samples were similar. The Nipawin Till #3 sample had a slightly lower maximum deviatoric stress. The Nipawin Till #2 sample had the lowest recorded maximum deviatoric stress, even though it exhibited the greatest dilation. Perhaps the remolding process produces discontinuities which weakened this sample.

The Edmonton Till sample had the highest recorded maximum deviatoric stress. The Edmonton Till #2 sample had a maximum deviatoric stress which was 70 kPa higher than the Edmonton Till #1 sample.

5.7.3 Strength Data

Modified Mohr diagrams are shown in Figures 5.13 to 5.18. The Edmonton Till results (Figure 5.16) are supplemented with data obtained from Medeiros (1979). The diagrams present test results from overconsolidated and normally consolidated samples. The effective strength parameters for the tills will therefore rely on which straight line is fitted to the typically curved failure envelope. The failure envelopes indicate the following effective strength parameters: Nipawin Till - c=30 kPa ϕ=32°, Nipawin Till #1 - c=20 kPa ϕ=35°, Nipawin Till #2 - c=30 kPa ϕ=32°, Edmonton Till - c=35 kPa ϕ=34.5°, Edmonton Till #1 - c=15 kPa ϕ=33.5°, Edmonton Till #2 - c=25 kPa ϕ=31.5°.

The soil strength characteristics can be plotted in different ways to obtain the strength parameters for geotechnical design. It is often useful to present the data on a plot of angle of shearing resistance (ϕ) versus the normal stress on the failure plane. The effective cohesion of the tills varies between 15 and 35 kPa. Cohesions of this magnitude would only significantly affect the strength determination at very low stress levels. For this reason, when calculating the angle of shearing resistance it is assumed that cohesion (c') is zero. This allows the angle of shearing resistance to be calculated from triaxial experimental data as follows:

$$\sin\phi = q/p \qquad (5.1)$$

$$\text{Where: } q = (\sigma_1' - \sigma_3')/2 \qquad (5.2)$$

$$p = (\sigma_1' + \sigma_3')/2 \qquad (5.3)$$

The normal stress on the failure plane is obtained using the following relationship:

$$\sigma n = p - (\sin\phi * q) \qquad (5.4)$$

The results obtained using this analysis are presented graphically in Figure 5.12.

Figure 5.12 indicates that the angle of shearing resistance of the tills tested range primarily from 31° to 37°. It can clearly be seen that the magnitude of the angle of shearing resistance is highly dependent on the normal stress applied. Higher values were measured for normal stresses less than 100 kPa, due to the highly dilatant behavior of the till material under low confining stress. The values decrease with increasing stress until approximately 400 kPa where they generally level off. As mentioned previously, the intact Nipawin Till was dilatant over a wide range of pressures and as a result produced friction angles greater than 40° up to normal stresses of 400 kPa. Above 400 kPa the angle of shearing resistance decreases to between 31° and 37°. The ϕ values for the reconstituted samples rapidly decrease at stresses above 100 kPa.

The reconstituted Nipawin Till samples indicate that remolding or deposition from a slurry appears to have an influence upon the measured angle of shearing resistance. The Nipawin Till #1 samples level off at a ϕ value of 37° whereas the Nipawin Till #2 samples level off at a ϕ value of 34°. The increase of preconsolidation pressure between Edmonton Till #1 and #2 appears to have little effect upon the angle of shearing resistance. Both tills level off at ϕ values of approximately 33°.

5.7.4 Stress Paths

In Figures 5.19 to 5.24, stress paths are plotted for consolidated-undrained experiments the intact and reconstituted samples. In addition, the failure envelope defined at maximum stress ratio is superimposed on the plots. Some scatter in the data will be seen in the early stages of several of the tests but this appears to decrease as failure is approached.

The stress paths clearly show a marked difference in the shape of the stress paths as the effective confining pressure increases. Under low initial confining pressures the porewater pressure decreases and the effective confining stress increases. As the initial confining stress increases contractual behavior in the samples becomes increasingly apparent. The increase in porewater pressure results in a lowering of the effective confining stress as the sample fails.

The effective confining pressure for the overconsolidated samples continues to increase during shear until the stress path reaches the failure envelope. After maximum stress ratio has been attained stress paths follow the failure envelope closely. In the normally consolidated samples, however, the effective confining stress component decreases until the stress path approaches the failure envelope, and then the effective confining stress component suddenly increases. The peak stress ratio is achieved soon after this occurs.

5.7.5 Laboratory Young's Moduli

The Young's Moduli versus deviatoric stress for each sample is shown in Figures 5.25 to 5.30. The Young's Modulus is defined as the tangent modulus at each point along the deviatoric stress versus strain curve. Since the initial

Young's moluli are calculated at low strains it is probable that bedding errors caused by the use of free ends (Section 5.5.[illegible]) would significantly influence the calculated values. For this reason the initial modulus values were not plotted on the figures.

The Nipawin till samples remain stiffer to a higher deviatoric stress than Nipawin Till #1 and #2 samples. The modulus values of Nipawin Till and Nipawin Till #2 samples are highly dependent on the confining stress applied. The modulus values for Nipawin Till #1 samples are similar for all confining stresses.

The data for the Edmonton Till and Edmonton Till #1 and #2 are similar to the Nipawin Till and Nipawin Till [illegible] The increased consolidation stress applied to the Edmonton Till #2 samples results in higher overall modulus values when compared with the Edmonton Till #1 results.

The tangent modulus versus deviatoric stress plots are useful in delineating the till's microstructure. The stiffness of a till with an open matrix structure is dependent on confining stress whereas the stiffness of a closed matrix structure is not as sensitive to changes in confining stress. As expected, the stiffness of a till increases with increased consolidation pressure.

5.8 Cyclic Triaxial Results

5.8.1 Behavior Under Cyclic Loading

All samples were consolidated to an effective confining pressure of 800 kPa and subsequently subjected to a maximum deviator stress of 50 or 80% of the maximum failure stress obtained during static tests of similar material.

In order to appreciate fully the significance of the results, it is useful to study the effective stress, strain and porewater pressure responses during the cyclic load applicati[illegible]gures 5.31 to 5.33 show the effective stress paths for [illegible]amples tested. To present the test results clearly, the data was manually reduced at selected p-q readings taken during the load intervals. In regions where significant changes in behavior occurred, many data points were chosen but when the behavior stabilized, data points were chosen at larger time intervals. Figures 5.34 to 5.36 show porewater pressure versus number of load cycles. Figures 5.3[illegible] to 5.39 show cyclic strain response versus number of load cycles. The data presented corresponds to the cycle intervals chosen for the effective stress plots contained in Figures 5.31 to 5.33.

A comparison of the stress paths followed in all the experiments provides an explanation for their behavior during repeated loading. The most obvious difference noted was their position relative to the strength envelope. Previous work showed that when the stress level is below some critical value, nonfailure equilibrium will be attained. Above the critical value failure occurs and each loading cycle leads to further non-recoverable strain (Anderson, 1976; Mitchell and King, 1977).

5.8.1.1 Normally Consolidated Samples

The normally consolidated samples showed the greatest stress path migration during repeated loading. The migration direction is indicated by an arrow on the Figures. The migration was accompanied by a rapid increase in cyclic strain and porewater pressure as illustrated in Figures 5.34 to 5.39.

Strain and porewater pressure of the intact Nipawin Till samples increased rapidly upon repeated loading. The sample failed after a few cycles and continued to strain until approximately 120 cycles. At this point the strain leveled off at 7%. The porewater pressure continued to increase until approximately 200 cycles. The stress path again appears to migrate towards the strength envelope and then to come to equilibrium close to it. After the initial deformation, the sample stabilized and no further straining was recorded. It is interesting to note the rapid increase in strain that occured during the 287th cycle (Figure 5.37). The slip was associated with a slight increase in porewater pressure.

Nipawin Till #1 samples were subject to cyclic loads of approximately 50% and 80% of static failure stress. At 80%

of maximum failure stress the porewater pressure increased dramatically with the first few loading cycles. Consequently, this caused a large migration of the effective stress path towards the failure envelope. Surprisingly, the sample only experienced approximately 1 % strain during this period. The sample failed (strain > 2%) at 54 cycles. At approximately 200 cycles the strain and porewater pressure leveled off. This was accompanied by a general decrease in stress path migration until a state of equilibrium was attained. In Figure 5.32 it appears that this stress path had migrated to or very near the strength failure enveloped.

At 50% of static failure stress, the stress paths showed a slight migration initially, however, equilibrium was reached at approximately 200 cycles. It is interesting to note that after an initial deformation of less than 1% the strain leveled off even though the porewater pressure increased gradually and leveled off at approximately 200 cycles. The stress applied to this sample was apparently below the critical value and failure did not occur (Figure 5.32).

The Nipawin Till #2 samples showed a rapid deterioration under repeated loading. The stress path quickly migrated to the strength envelope (Figure 5.33) accompanied by porewater pressure increases and large strains (Figures 5.36 and 5.39). The sample failed after a few cycles and deformation continued until the test was terminated (Figure 5.39). Obviously cyclic loading caused a general collapse of the soil structure in these samples.

5.8.1.2 Overconsolidated Samples

At an overconsolidation ratio of 4, the stress paths of the Nipawin Till #1 samples migrated rapidly to the strength envelope. At 80% of failure stress the samples failed at approximately 20 cycles. Initially their pore pressures increased slightly but soon leveled off.

At 50% of failure stress an interesting observation was made. The stress path migrated to the strength envelope and reached equilibrium at or close to it. The cyclic strain quickly rose to 1% and remained stable until approximately 280 cycles. At this point the porewater pressure decreased rapidly. The sudden decrease in porewater pressure was accompanied by only a slight fluctuation in the strain response. An error in the porewater pressuremeasuring system is the probable cause for this phenomenon.

At an overconsolidation ratio of 10 the stress paths quickly reached equilibrium. At 80% of failure stress the porewater pressure increased slightly then began to decrease. The strain increased and levelled off at approximately 1.9%. The sample stress at 50% of failure stress showed a slight dilation and equilibriated at approximately 1.8% strain.

Nipawin Till #2 samples with overconsolidation ratios of 4 and 10 surprisingly showed little effect of repeated loading. Both samples showed little pore pressure response and strains of 1.85 and 1.7% for overconsolidation ratios of 10 and 4 respectively.

The intact Nipawin Till sample with an overconsolidation ratio of 4 was unfortunately subjected to an erroneous deviatoric stress. The applied stress was greater than 100% of the failure stress and the sample quickly strained to failure. It is interesting to note however the lack of response in porewater pressure and the resulting relatively small migration of the stress path toward the failure envelope.

The sample with an overconsolidation ratio of 10 again showed little response to the cyclic loads. The sample strained immediately to approximately 5.6% and then stabilized. It appears that this sample also was stressed beyond the required 80% of failure stress.

5.8.2 Behavior after Cyclic Loading

Anderson (1976) found that cyclic loading accompanied by drainage will influence the stiffness of a soil and its resistance to subsequent cyclic loading. For the long term condition, cyclic displacements may decrease for a normally consolidated clay. For overconsolidated clays however, the long term effect may be increased cyclic displacements and reduced stability.

To study the long term effects of cyclic loading on tills a static triaxial program was initiated. When a sample had reached nonfailure equilibrium during undrained cyclic loading, the drainage valves were opened and the porewater pressures were allowed to dissipate. Drainage was allowed for a minimum 24 hour period and longer if required. Subsequently the sample was sheared under undrained conditions. The deviatoric stress versus strain and porewater pressure versus strain for each sample is shown in Figures 5.40 to 5.42. The p-q plot of the data at failure is illustrated in Figure 5.43.

A comparison of the magnitudes of deviatoric stresses at similar confining pressures indicates that cyclic lo[illegible] has little effect on the magnitude of deviatoric stress[illegible] failure. The plots show however, that the initial stiffness of the cyclic loaded samples is greater than observed in the static loaded samples. This result was anticipated since the cyclic loading had already strained the sample to some extent.

The deviatoric stress strain curves indicate an interesting behavior. When sheared under high confining stresses the samples exhibited a quasi elasto-plastic behavior. Once a certain deviatoric stress level is reached, the sample undergoes plastic deformation with little change in deviatoric stress.

The data suggests that cyclic loading had little effect upon the strength of the Nipawin Till #1 or Nipawin Till #2 samples. Nipawin Till #1 samples showed no change in effective angle of shearing resistance and a slight reduction in cohesion. Nipawin Till #2 sample showed a slight reduction in angle of shearing resistance from 32° to 31.5° after cyclic loading (Figure 5.43). This agrees with the work of Anderson (1976), who found that for overconsolidated Drammen clay samples, c' and ϕ' were not significantly influenced by undrained cyclic loading.

5.9 Discussion of Results

From the grain size curves and Atterberg Limits, it appears that the Edmonton and Nipawin Tills are similar in composition. The Nipawin Till is a clayey sandy silt. The greater sand content of the Edmonton till classes it as a clayey silty sand. The materials can both be classified as matrix dominated tills and the matrix material should dominate the engineering behavior.

The scanning electron micrographs showed that the matrix in the intact and reconstituted slurry samples had an open fabric that would be expected to collapse upon loading. The remoulded samples had a closed fabric that would be expected to dilate. During the triaxial testing program, the intact samples showed the greatest dilation and little difference in porewater pressure response was noticed between the remoulded and slurried samples.

The triaxial test results however support the theory of a structural ped framework within matrix dominated tills (Section 3.1.2). The soil peds act as individual particles that tend to dilate under low confining pressures. The range of pressures under which dilation occurs is dependent on the integrity of the individual soil peds. When comparing intact with remoulded materials it was noted that they exhibited dilatant behavior over a wide range of confining pressures. The effect of the soil peds can be readily seen when examining the porewater pressure responses in Figures 5.4 to 5.10 and the angle of shearing resistance values in Figure 5.12. At normal stresses less than 100 kPa all the samples dilate and angles of shearing resistance of 43° to 50° are recorded.

At higher confining pressures, dilation is surpressed and the strength of the till is increasingly controlled by the clayey silt matrix material. Scanning Electron Microscope observations indicated that the fabric of the Nipawin Till and Nipawin Till #2 samples were very similar. At high confining pressures both materials exhibit angles of shearing resistance of approximately 34°. The Nipawin Till #1 samples, with a more closed fabric, attained angles of shearing resistance of 37°. In addition, the cyclic loading results show that at high confining pressures the Nipawin and Nipawin Till #2 samples quickly strained to failure accompanied by large increases in porewater pressure. The Nipawin Till #1 sample leveled off at 3% strain.

The triaxial test results indicate that the behavior of a glacial till is largely dependent upon the confining pressures to which it is subjected. Under low confining pressures, the soil behavior is largely influenced by the integrity of the structural ped framework. Under high confining stress, the fabric of the matrix material plays an increasingly important role.

When comparing the intact with the reconstituted tills another important difference was observed. The consolidation data indicate that the structure of the intact samples is considerably less compressible than the structure of the reconstituted materials. The Young's modulus values and visual observation of the deviatoric stress strain curves also indicate that the intact samples are stiffer than the reconstituted samples. Relating these observations to the soil model proposed in Chapter 3, the results would indicate that the structural ped framework is developed to a higher degree in the intact samples. This results in a stiffer framework within the soil that is less compressible.

The degree to which soil peds are developed is largely dependent on the amount of consolidation that has occurred. Since drainage in the bond material is controlled by micro pores, secondary consolidation in the matrix material plays an important role in the development of soil peds.

The reconstituted samples were isotropically consolidated in the triaxial cell for a minimum period of 24 hours. Even though excess porewater pressure could not be monitored on the pressure transducer it is likely that secondary consolidation was not completed. The structural peds framework would therefore not develop to the same degree as the intact materials.

Bjerrum and Lo (1963) showed that, even for clays that did not show appreciable secondary consolidation effects, their shear characteristics were dependent on the age of the sample and with time the clay became more brittle. They stated that the effect of time on soil samples could be explained by the growing of cohesion bonds at the contact points between particles. These bonds lead to a greater resistance against shear deformation, but were gradually destroyed by increasing strain. The reduction of water content during secondary consolidation leads to a more stable configuration of the structure therefore developing increased strength and a reserve of resistance against further compression (Whitman, 1960 and Bjerrum, 1967).

Table 5.1

Summary of sample data

Sample Number	Sample Type	Total Unit Weight (g/cc)	Final Measured M/C (%)	Void Ratio	"A" Value Failure
S-1	Nipawin	2.26	12.7	0.32	-
S-2	Nipawin	2.45	11.7	0.31	-0.2
S-3	Nipawin	2.48	13.2	0.31	-0.22
S-4	Edmonton	2.18	13.5	0.38	-0.18
S-5	Edmonton	2.14	13.8	0.41	-0.32
S-6	Nipawin	2.25	12.3	0.32	-0.28
S-7	Edmonton	2.16	13.2	0.39	-0.11
S-8	Nipawin	2.13	12.5	0.40	-0.25
S-9	Nipawin	2.21	12.2	0.35	-0.14
S-10	Nipawin	2.28	12.8	0.31	-0.30
S-11	Edmonton	2.19	13.2	0.37	-0.22
S-13	Nipawin #1	2.14	14.3	0.42	+0.06
S-14	Nipawin #1	2.11	13.2	0.42	-0.15
S-16	Nipawin #1	2.18	15.6	0.40	-0.16
S-17	Nipawin	2.25	13.8	0.34	-0.22
S-18	Nipawin #1	2.16	13.7	0.39	+0.03

Table 5.1 Summary of Sample Data

Table 5.2

Summary of Sample Data

Sample Number	Sample Type,	Total Unit Weight (g/cc)	Final Measured M/C (%)	Void Ratio	"A" Value Failure
S-21	Nipawin #1	2.12	14.8	0.44	+0.56
S-22	Nipawin #1	2.16	14.6	0.40	–
S-23	Nipawin #1	2.15	15.0	0.42	0.00
S-24	Nipawin #1	2.14	14.8	0.42	–
S-25	Edmonton #1	2.14	15.2	0.43	0.00
S-26	Nipawin #1	2.16	15.2	0.41	–
S-27	Edmonton #1	2.14	15.7	0.43	-0.06
S-28	Nipawin #1	2.13	15.5	0.44	–
S-29	Edmonton #2	2.11	14.3	0.44	+0.20
S-30	Edmonton #1	2.15	16.0	0.43	-0.22
S-31	Edmonton #2	2.12	14.7	0.43	-0.08
S-32	Edmonton #2	2.13	15.1	0.43	-0.25
S-33	Nipawin #1	2.08	14.7	0.46	-0.21
S-35	Nipawin #1	2.21	14.7	0.37	-0.31
S-36	Nipawin #1	2.01	14.5	0.43	-0.13
S-37	Nipawin #2	2.16	14.7	0.41	-0.22
S-38	Nipawin #2	2.16	14.0	0.42	-0.23
S-39	Nipawin #2	2.18	15.5	0.40	-0.15

Table 5.2 Summary of Sample D[illegible]

Table 5.3

Summary of Sample Data

Sample Number	Sample Type	Total Unit Weight (g/cc)	Final Measured M/C (%)	Void Ratio	"A" Value Failure
S-40	Nipawin #2	2.15	15.5	0.42	–
S-41	Nipawin #2	2.13	15.2	0.43	–
S-42	Nipawin #2	2.18	15.8	0.41	-0.17
S-43	Nipawin #1	2.10	14.2	0.37	-0.11
S-44	Nipawin #2	2.21	15.5	0.39	-0.19
S-45	Nipawin #1	2.26	16.3	0.37	drained
S-47	Nipawin	2.26	16.4	0.36	drained
S-48	Edmonton #1	2.18	16.8	0.42	drained
S-49	Edmonton #2	2.18	16.8	0.42	drained
S-51	Nipawin	2.22	13.2	0.35	–
S-52	Edmonton	2.19	15.0	0.39	drained
S-53	Nipawin	2.23	14.5	0.36	–
S-54	Nipawin #3	2.11	14.8	0.45	-0.21
S-55	Nipawin #2	2.16	14.9	0.4[illegible]	drained
S-56	Nipawin #3	2.10	15.0	0.45	–
S-57	Nipawin #3	2.09	14.8	0.45	drained

Table 5.3 Summary of Sample Data

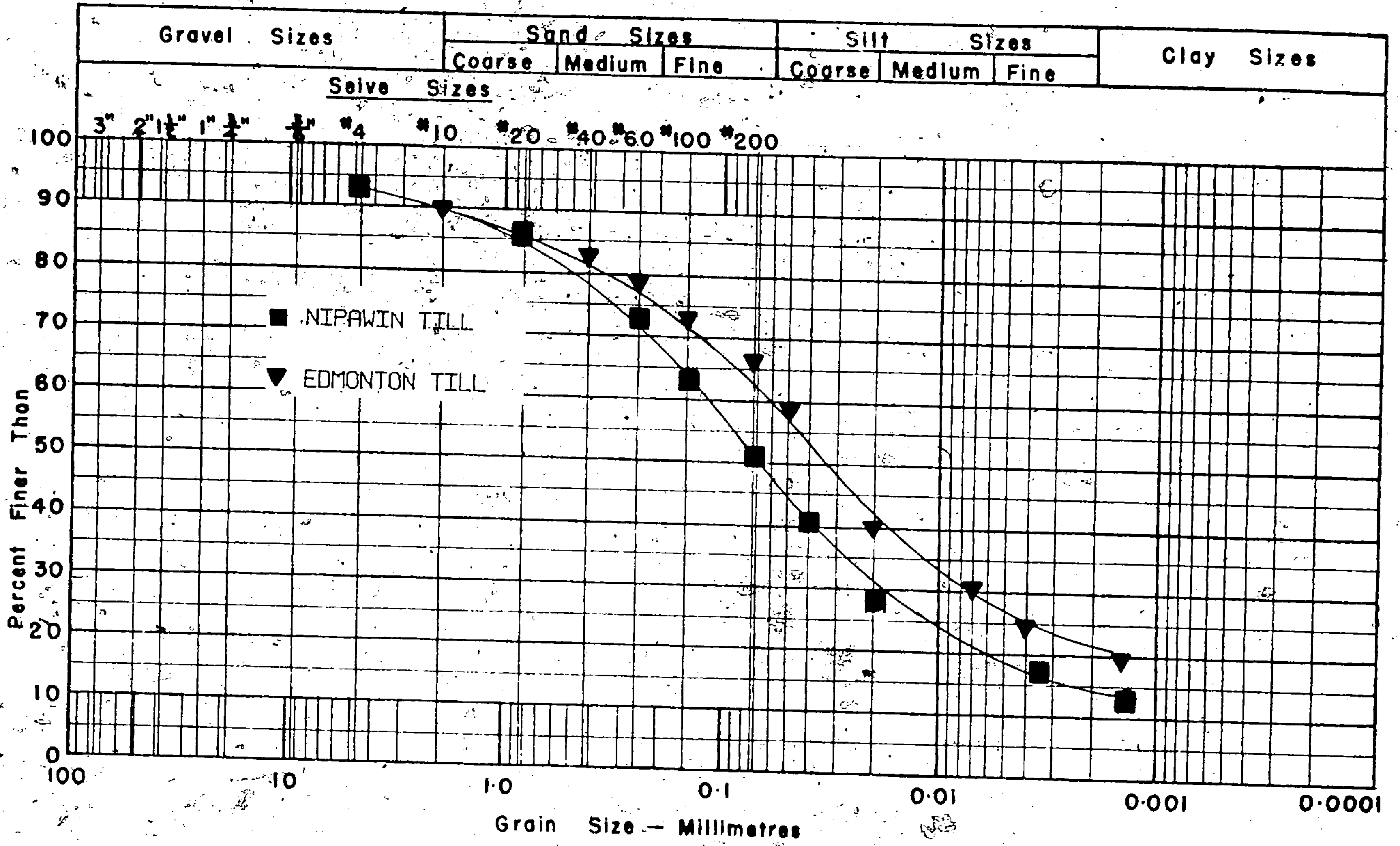

Figure 5.1 Grain Size Distributions

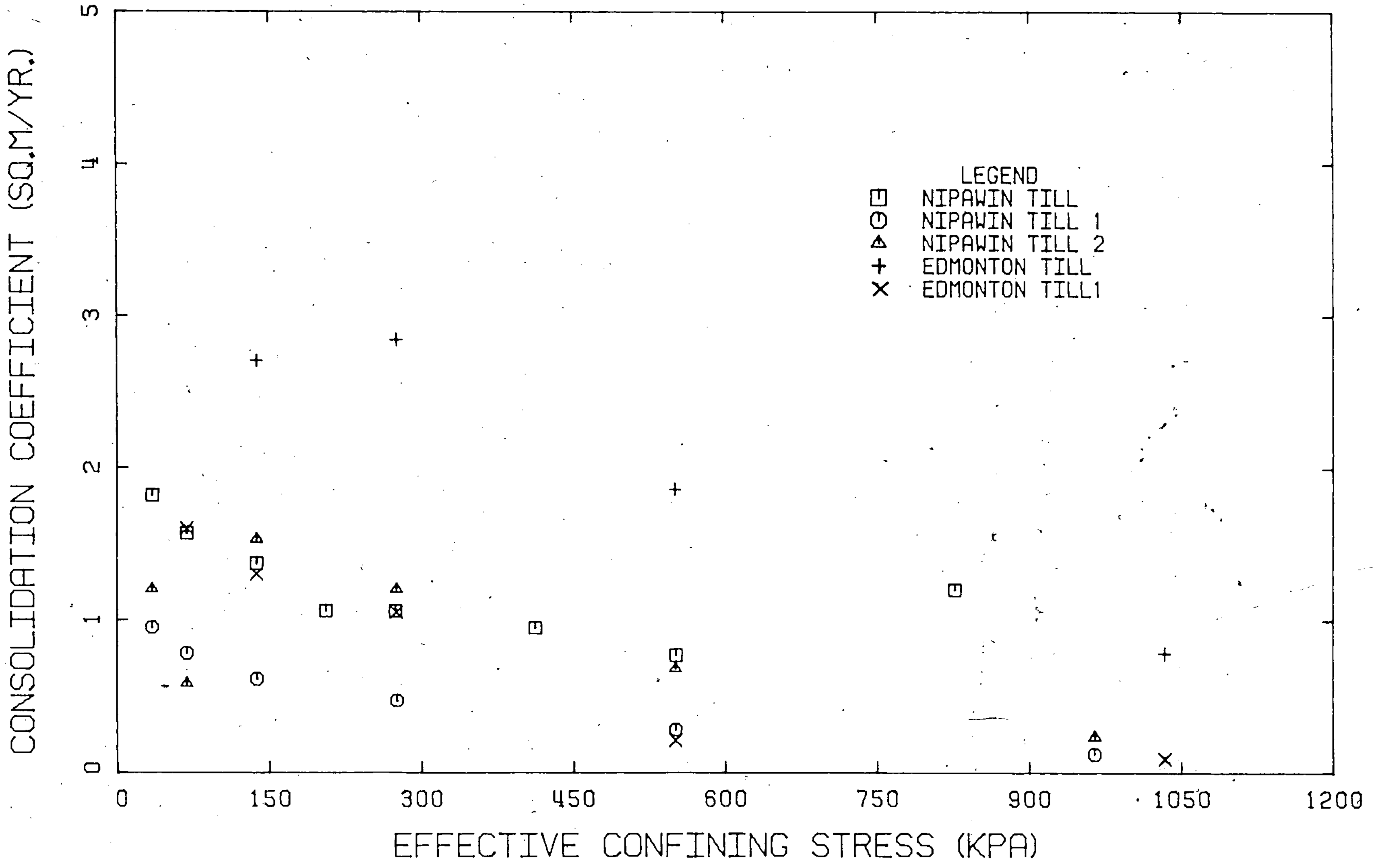

Figure 5.2 Consolidation Coefficient versus Confining Pressure

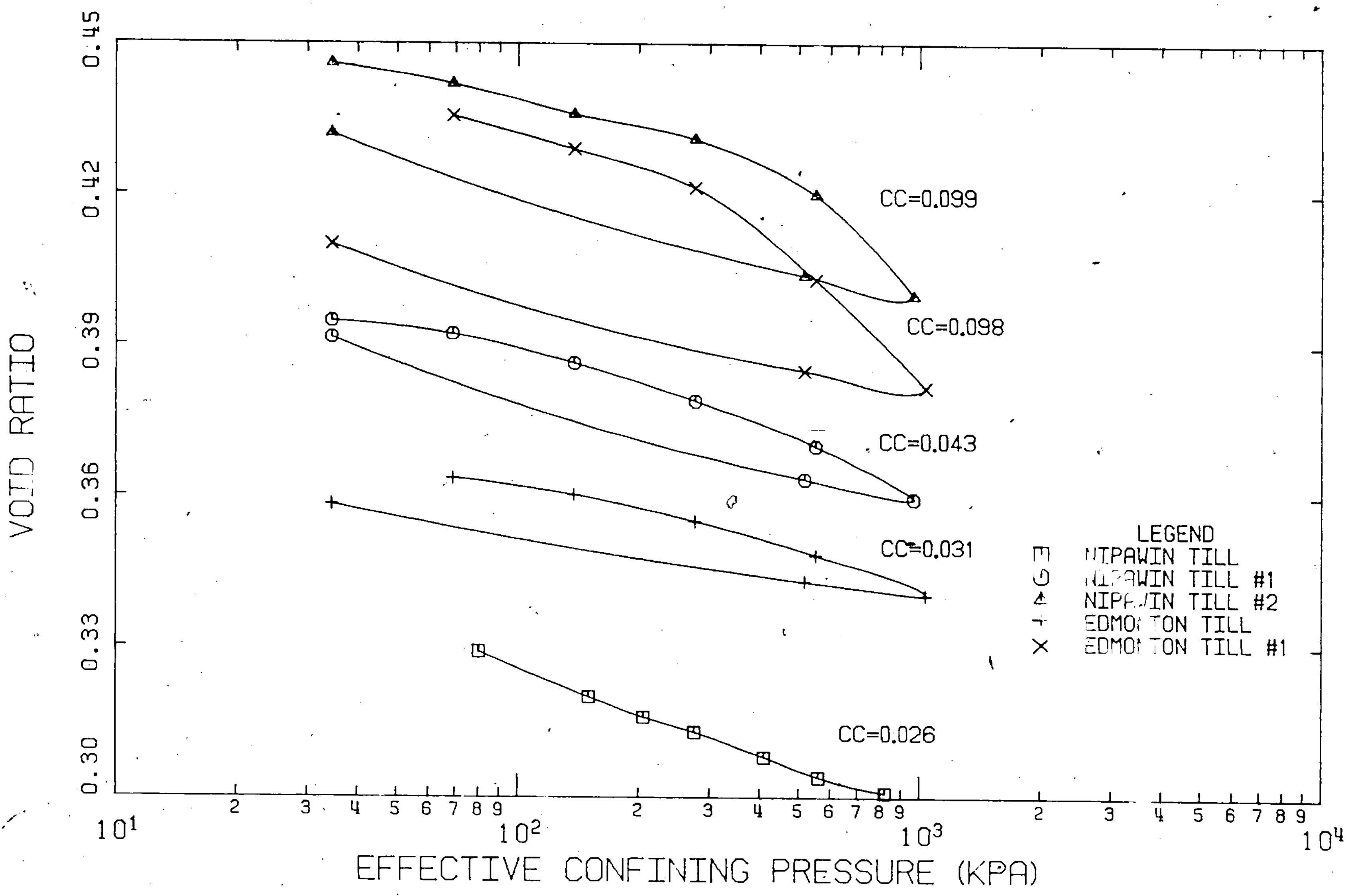

Figure 5.3 Void Ratio versus Logarithm Effective Stress

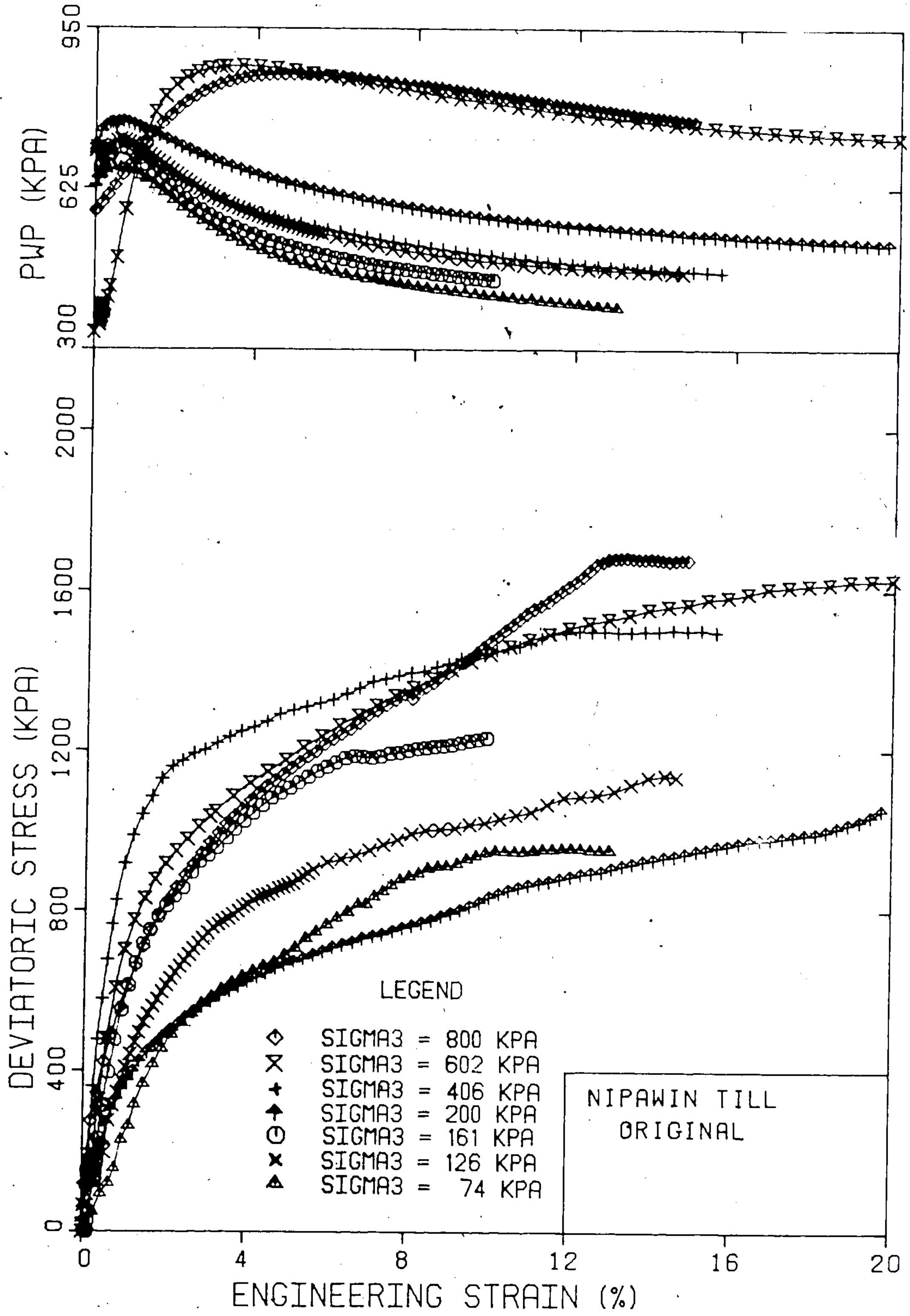

Figure 5.4 Stress-strain curves for Nipawin Till

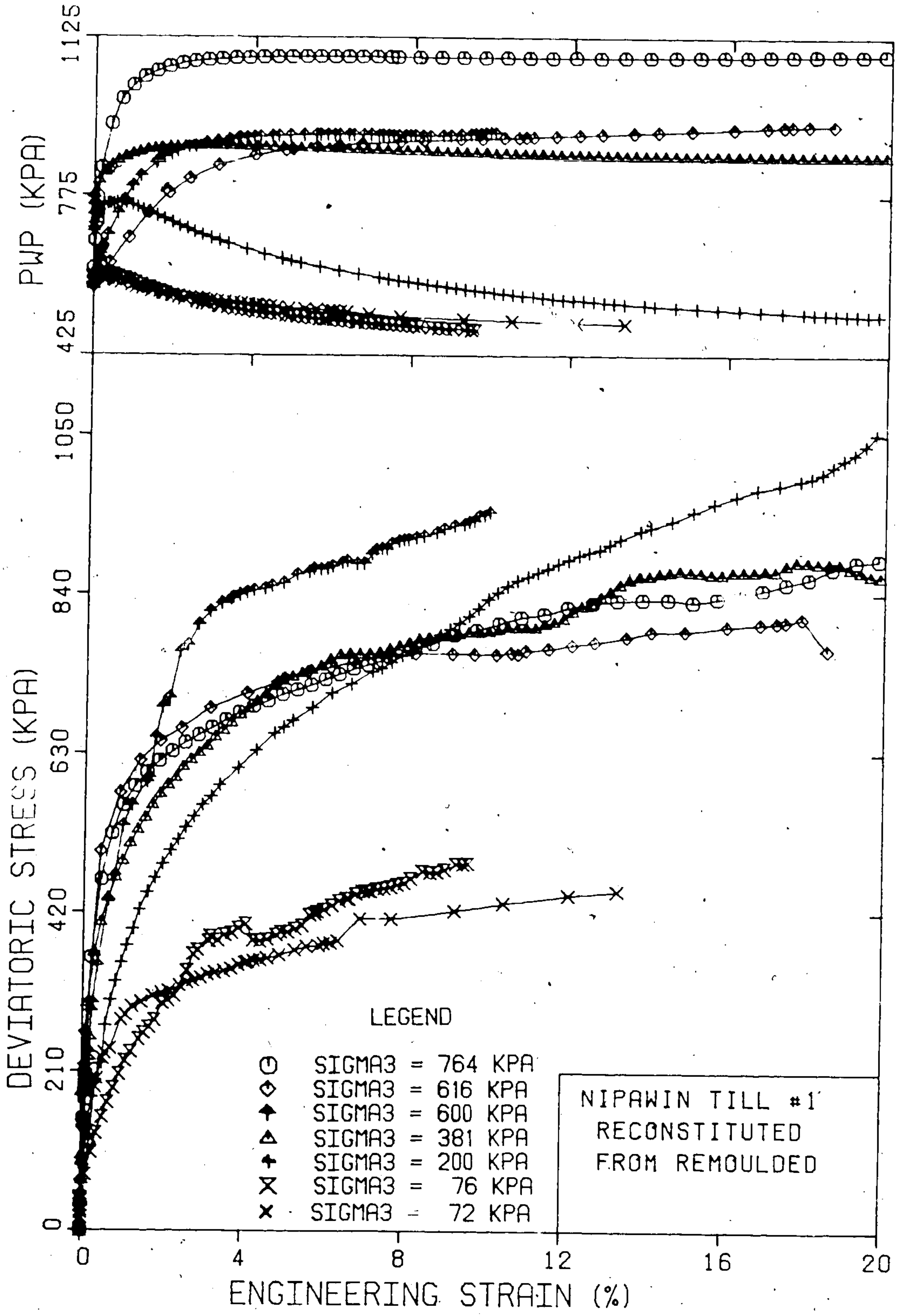

Figure 5.5 Stress-strain curves for Nipawin Till #1

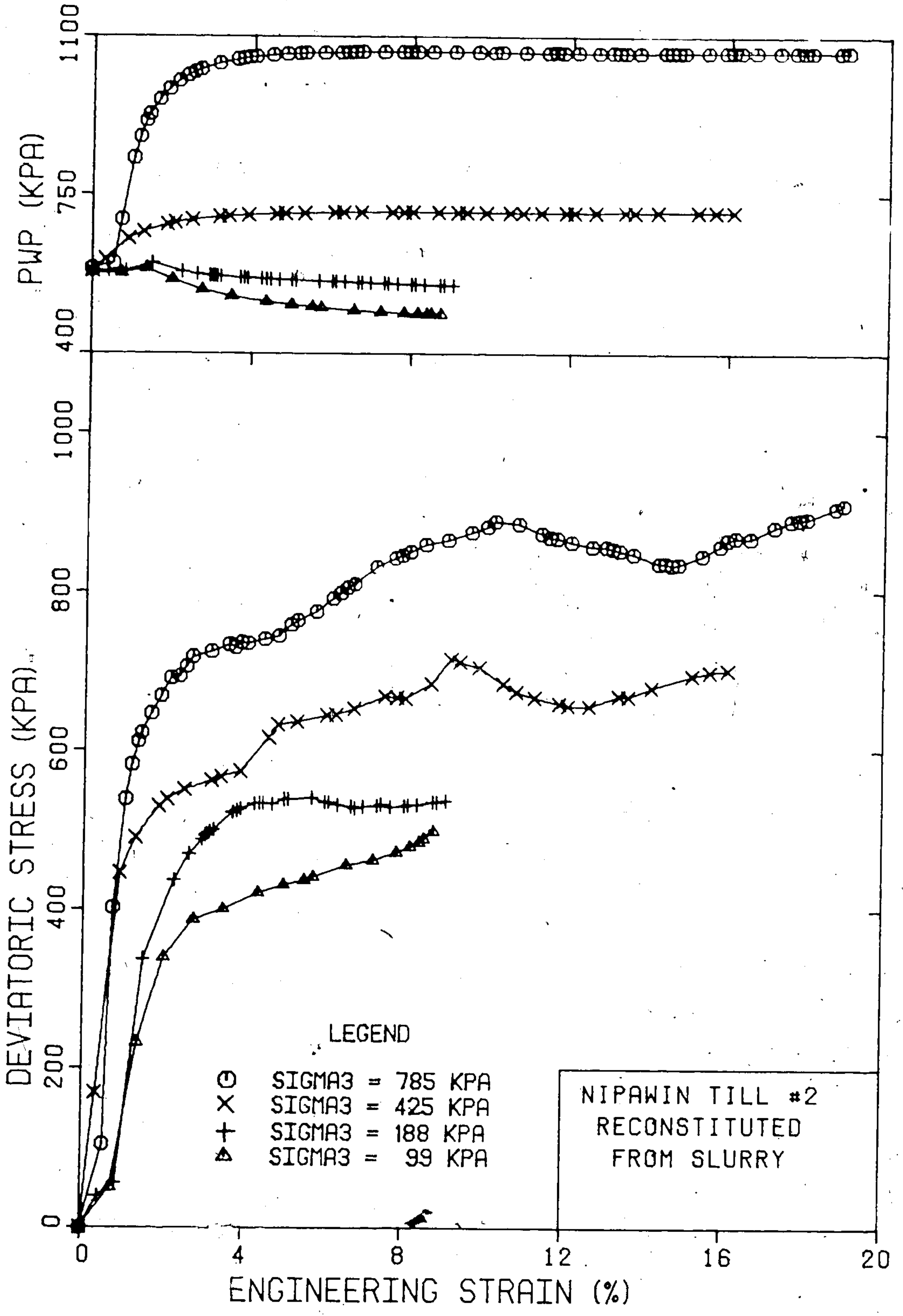

Figure 5.6 Stress-strain curves for Nipawin Till #2

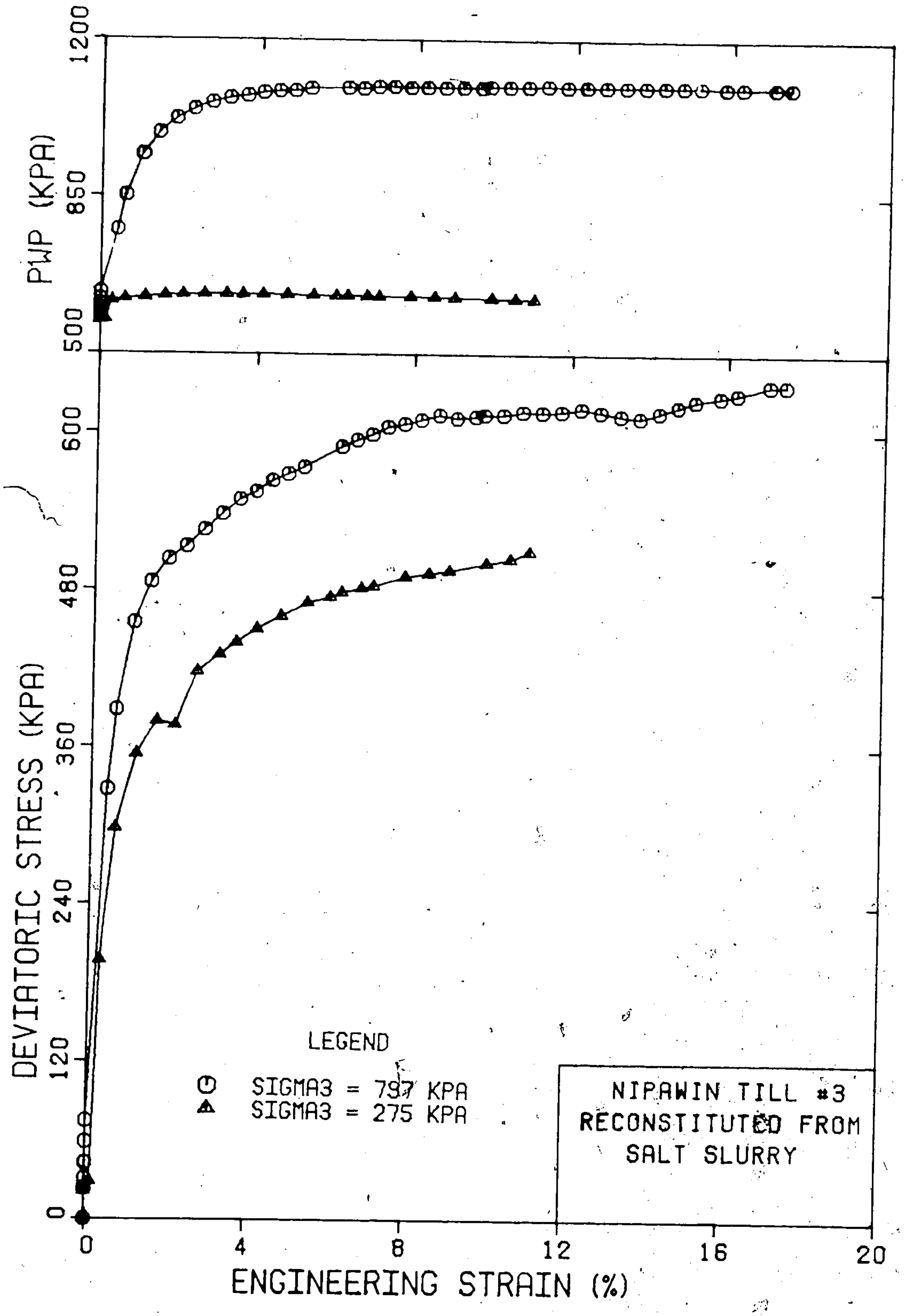

Figure 5.7 Stress-strain curves for Nipawin Till #3

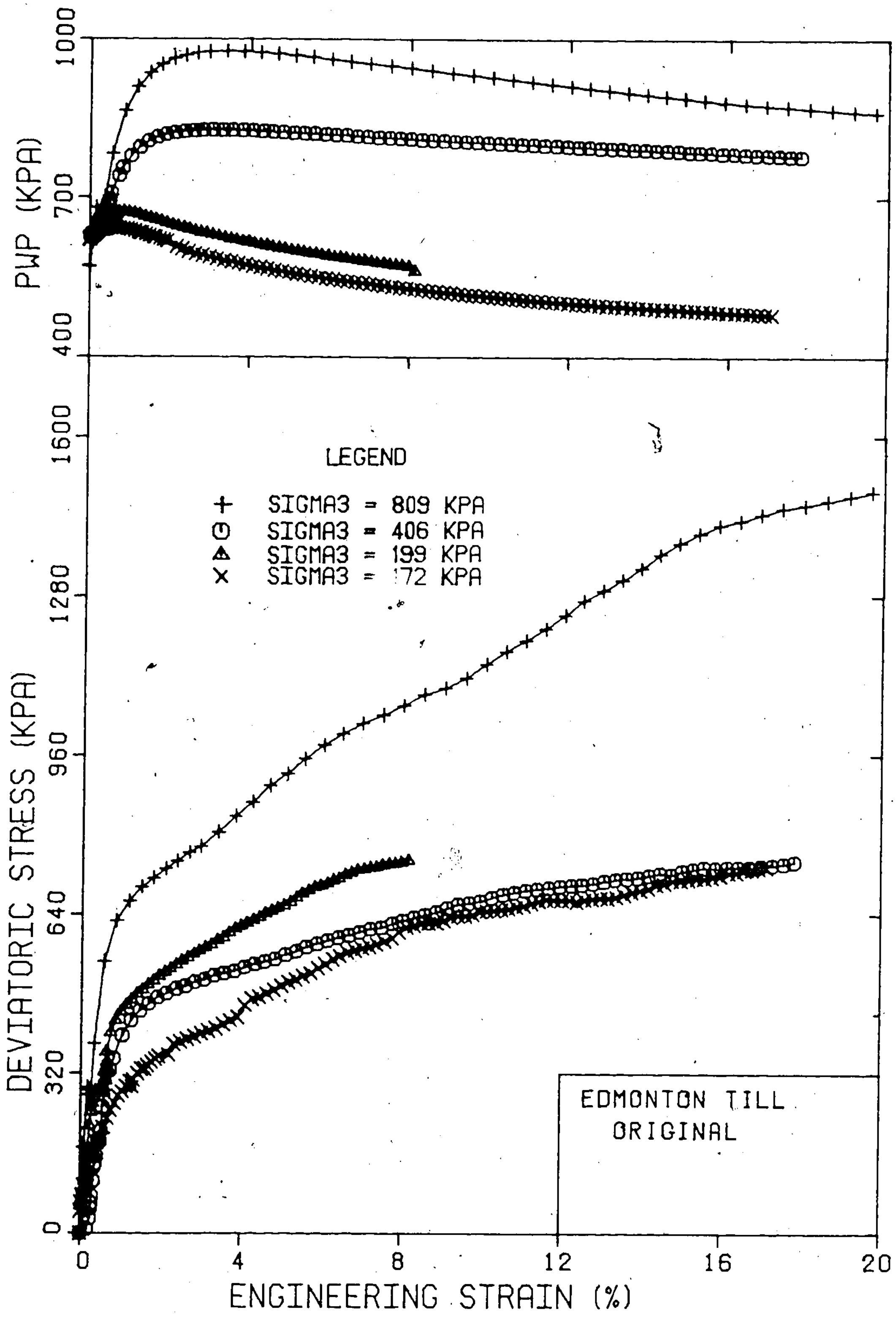

Figure 5.8 Stress-strain curves for Edmonton Till

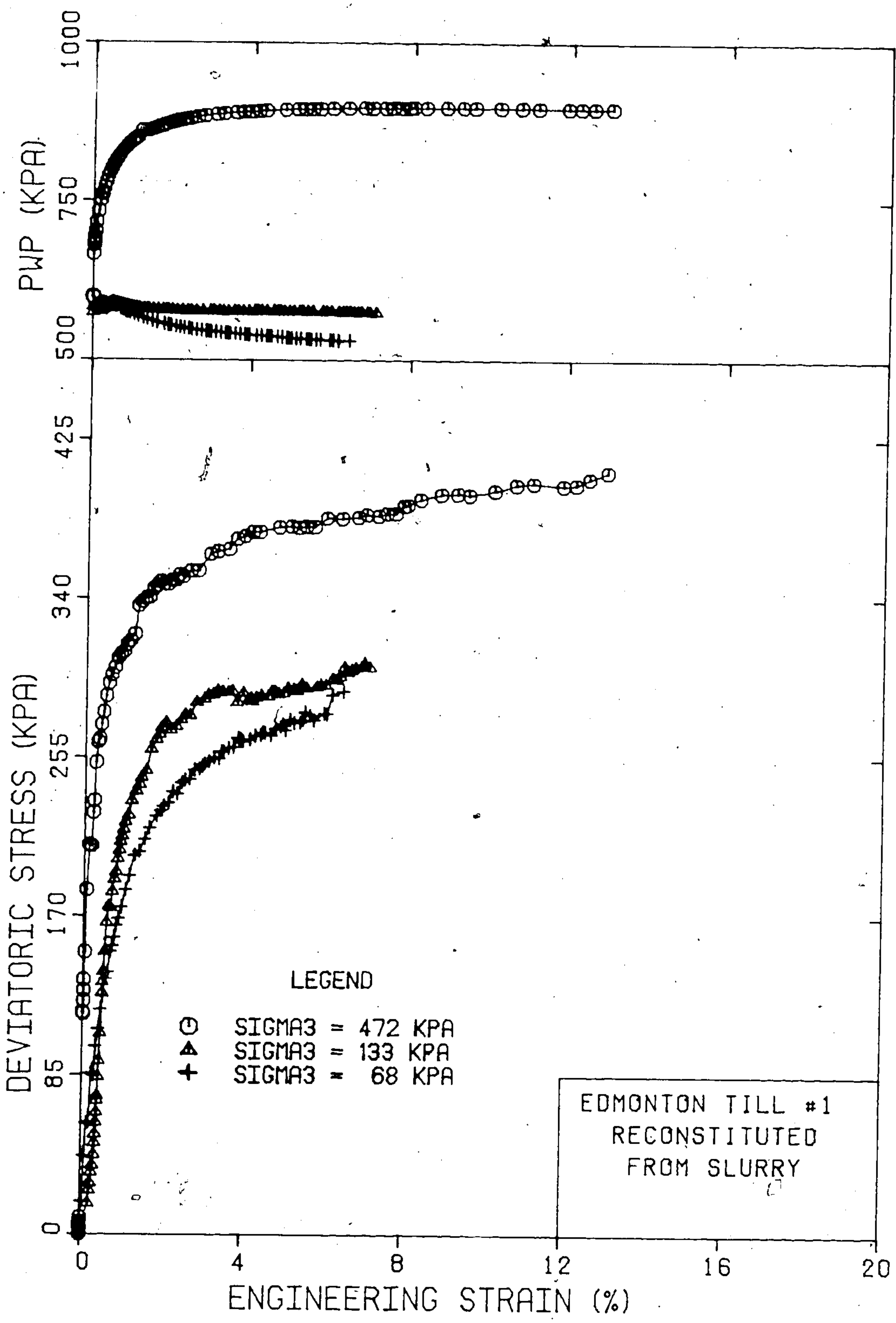

Figure 5.9 Stress-strain curves for Edmonton Till #1

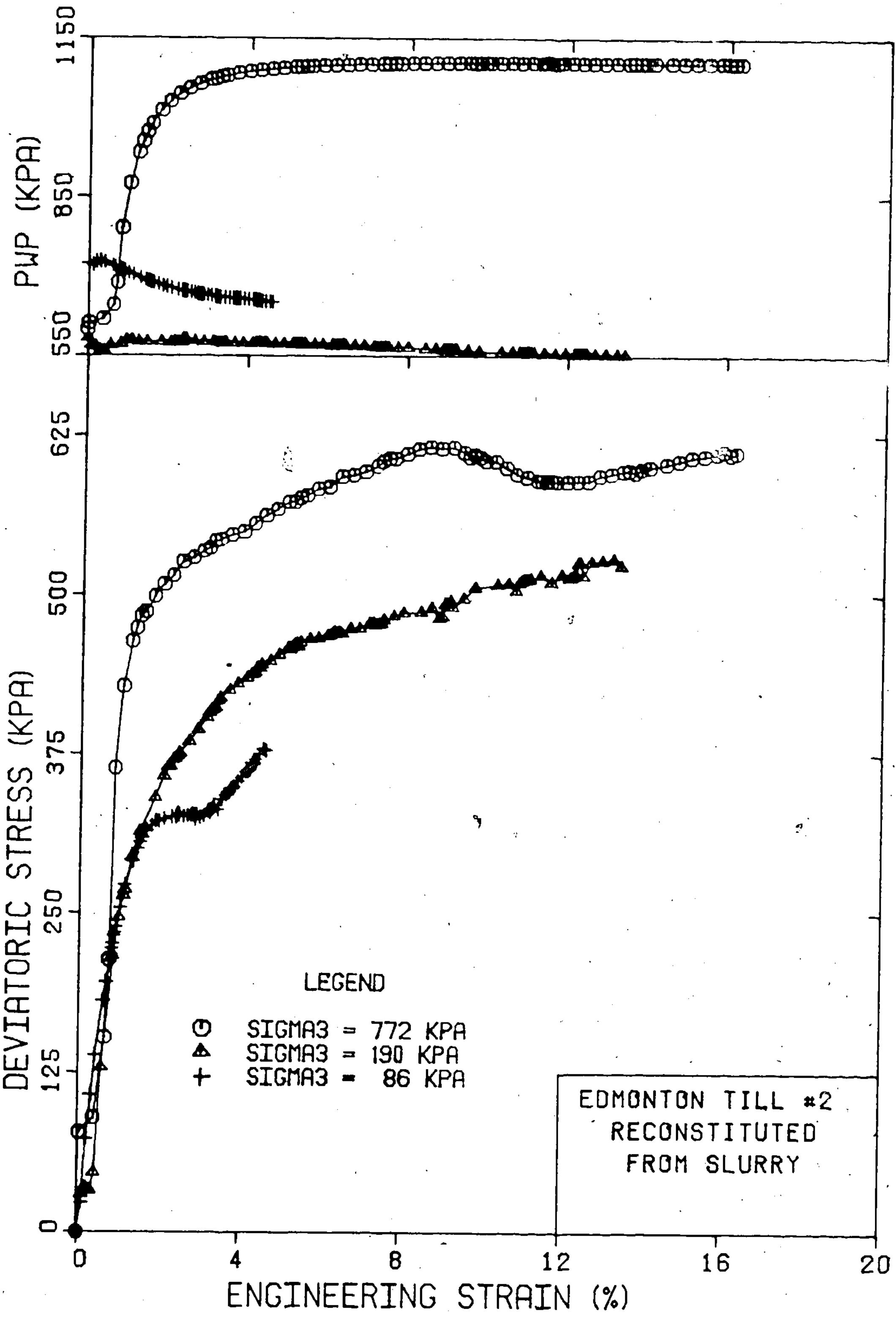

Figure 5.10 Stress-strain curves for Edmonton Till #2

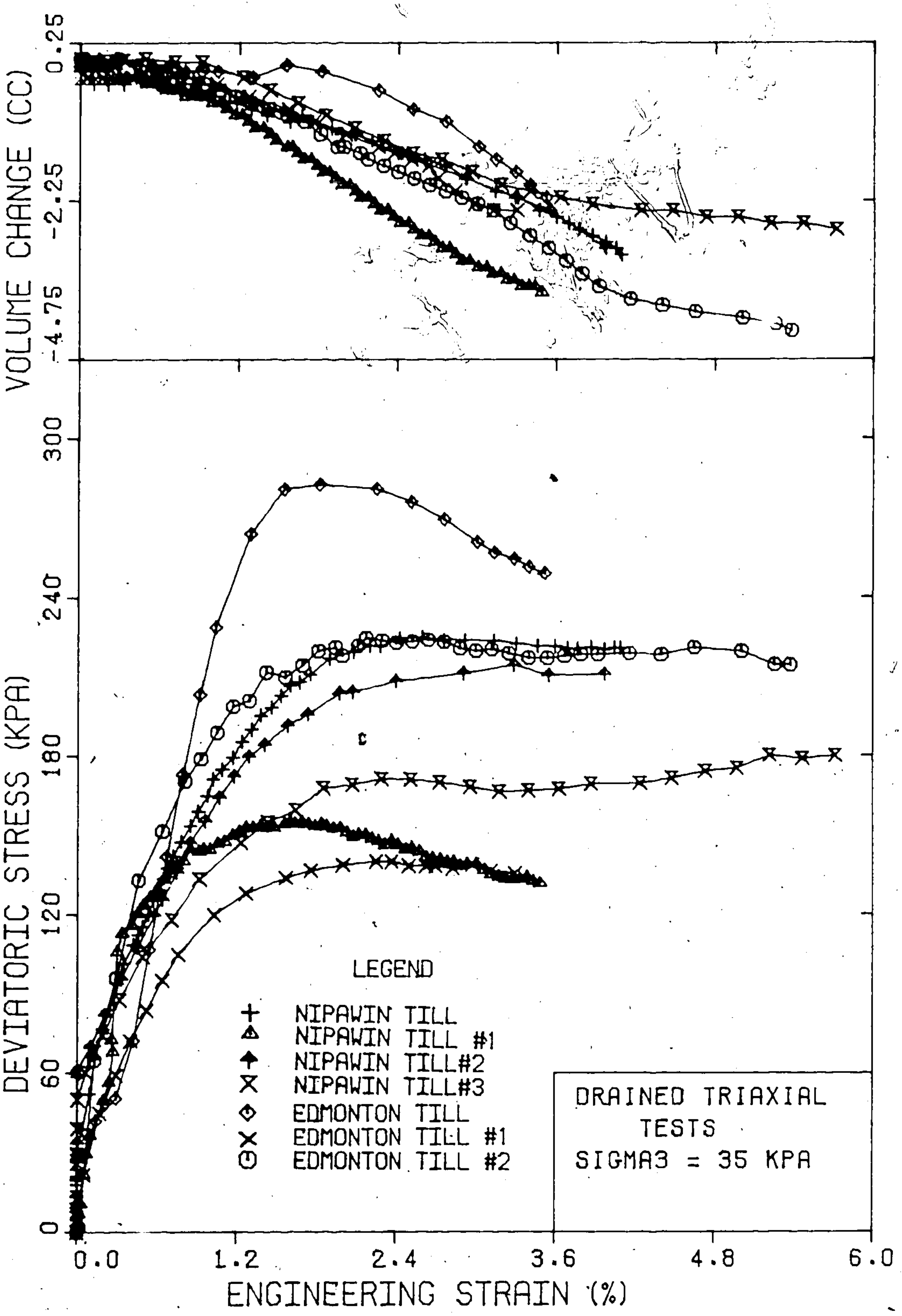

Figure 5.11 Stress-strain curves for Drained Triaxial Tests .

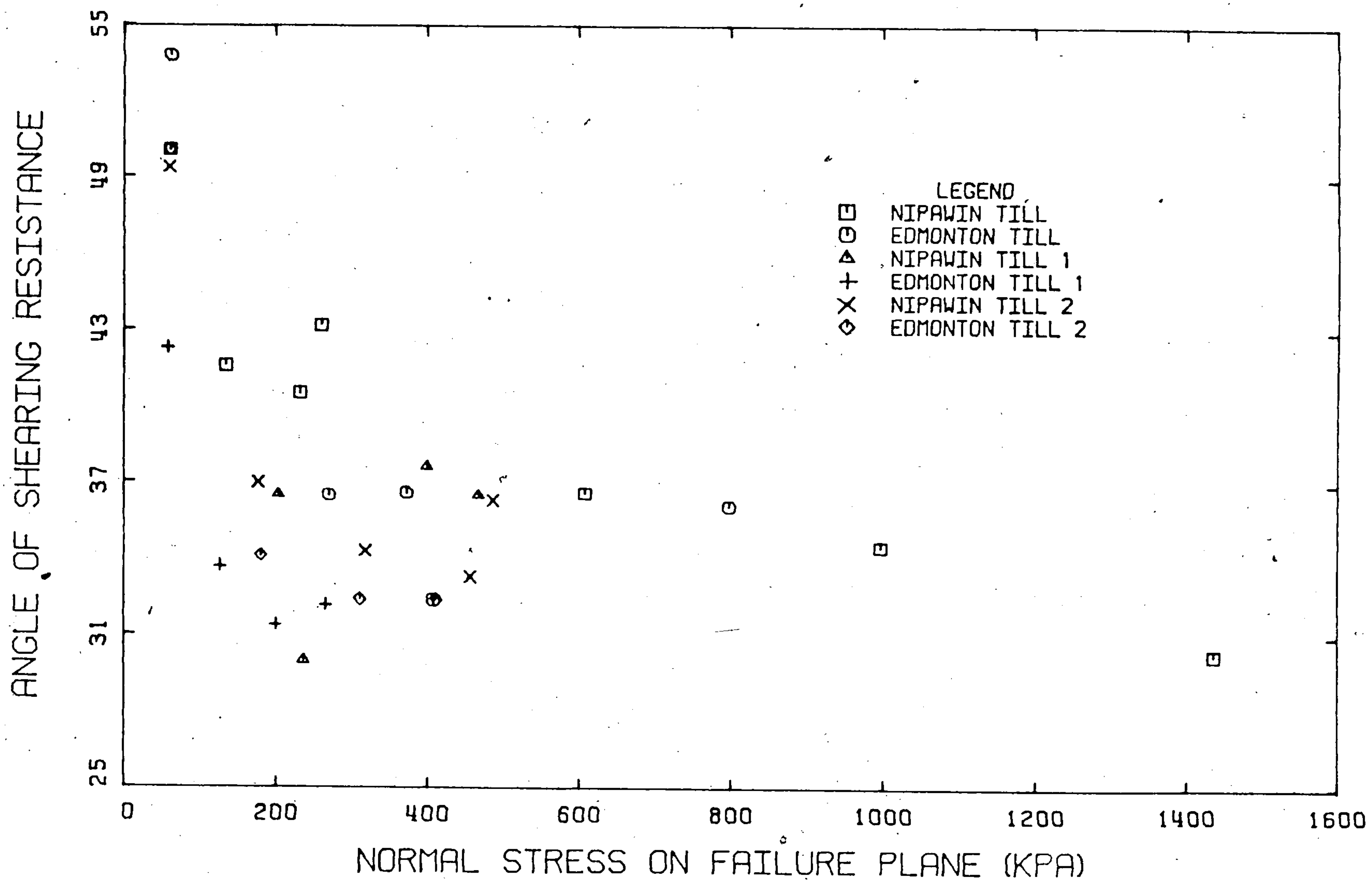

Figure 5.12 Angle of Shearing Resistance versus Normal Stress

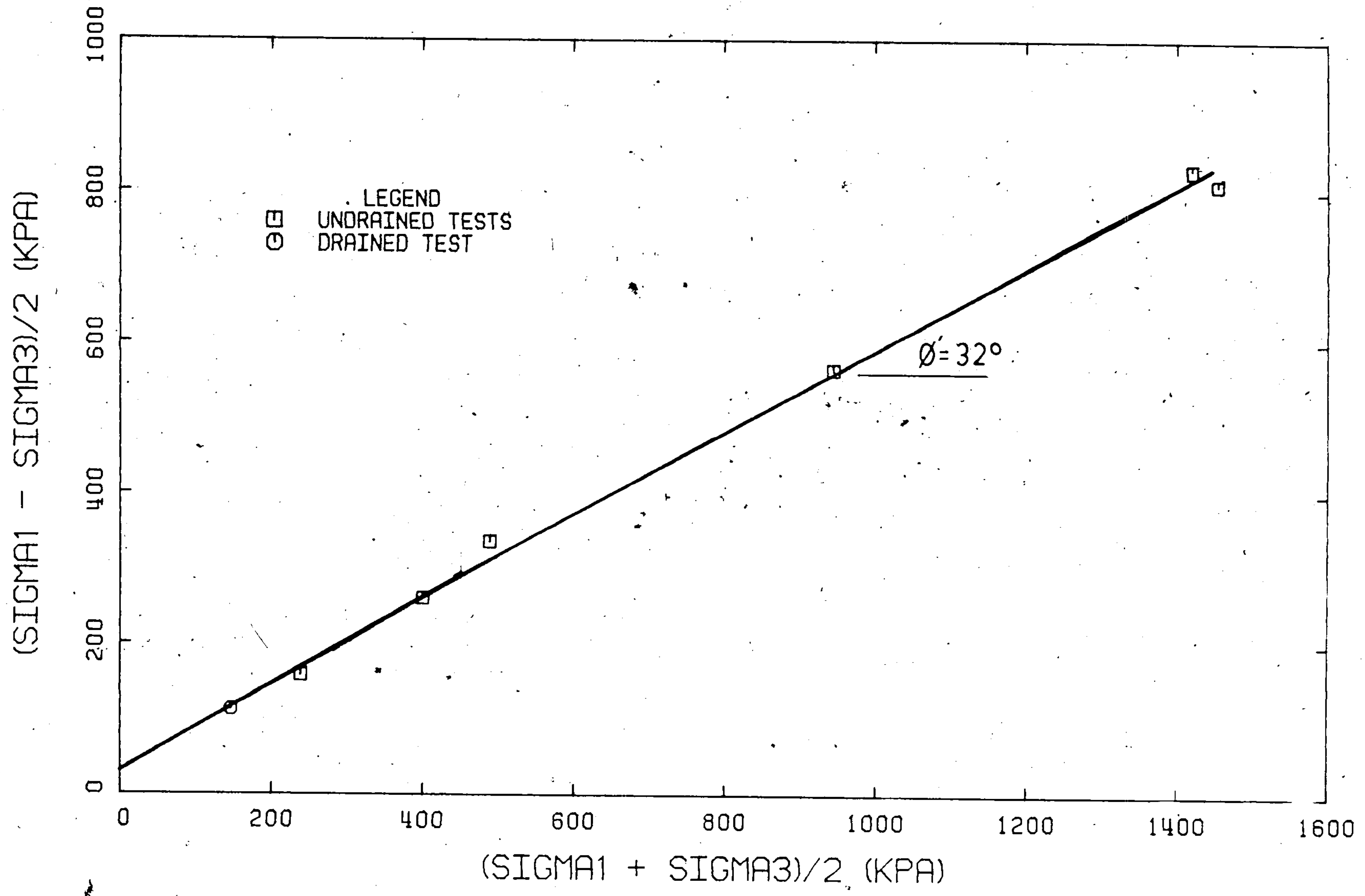

Figure 5.13 Effective Stress at Failure for Nipawin Till

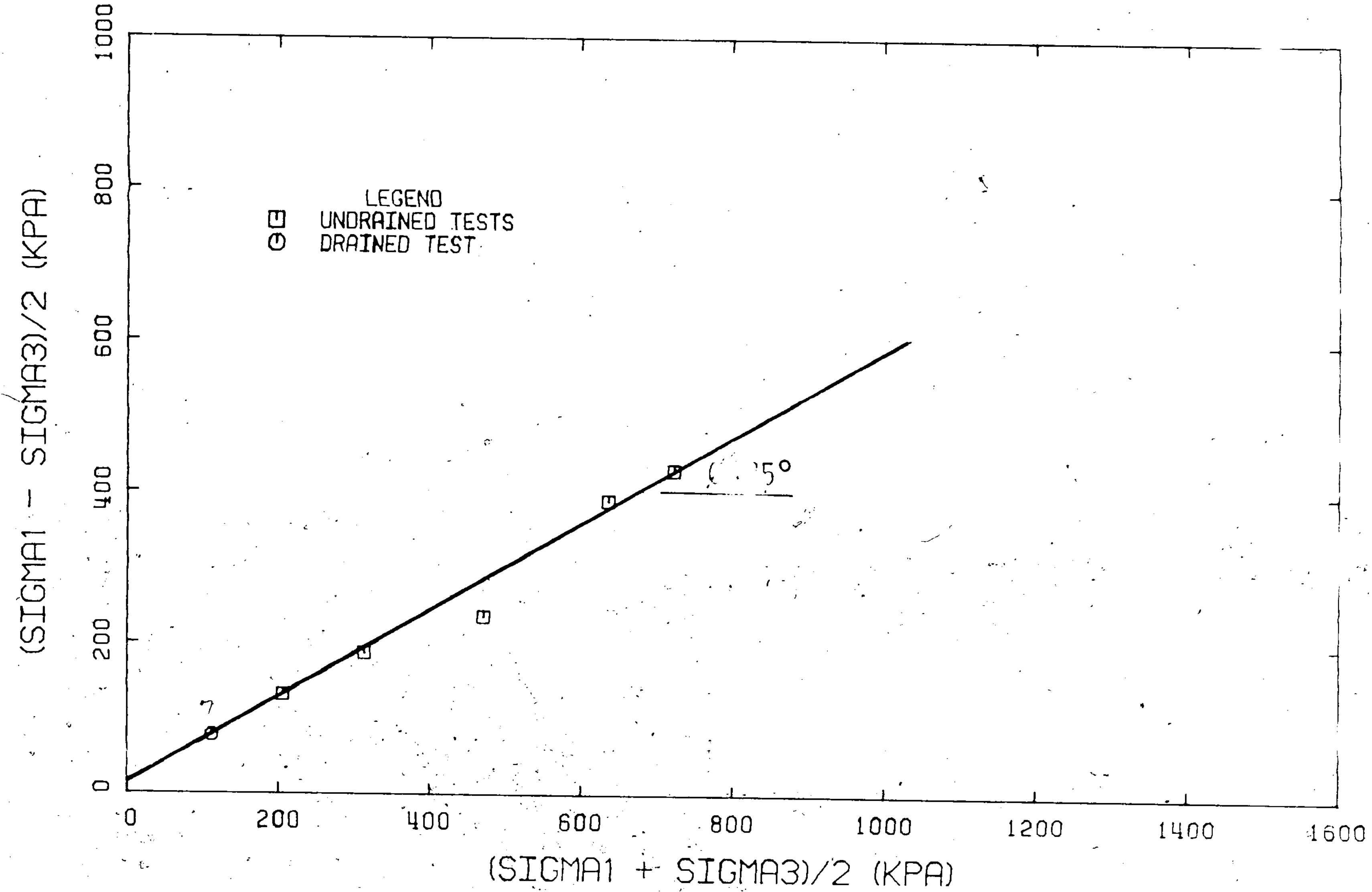

Figure 5.14 Effective Stress at Failure for Nipawin Till #1

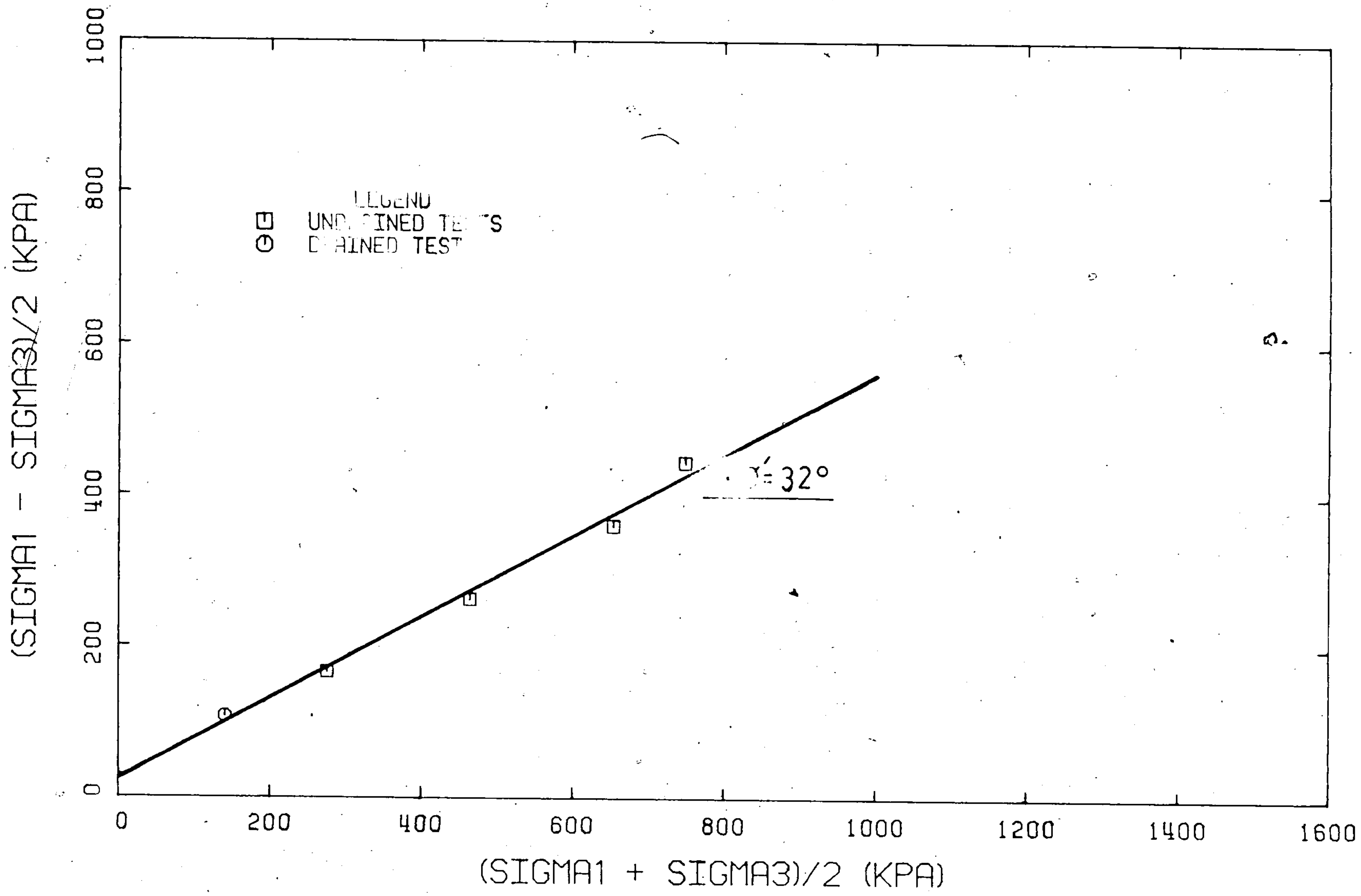

Figure 5.15 Effective Stress at Failure for Nipawin Till #2

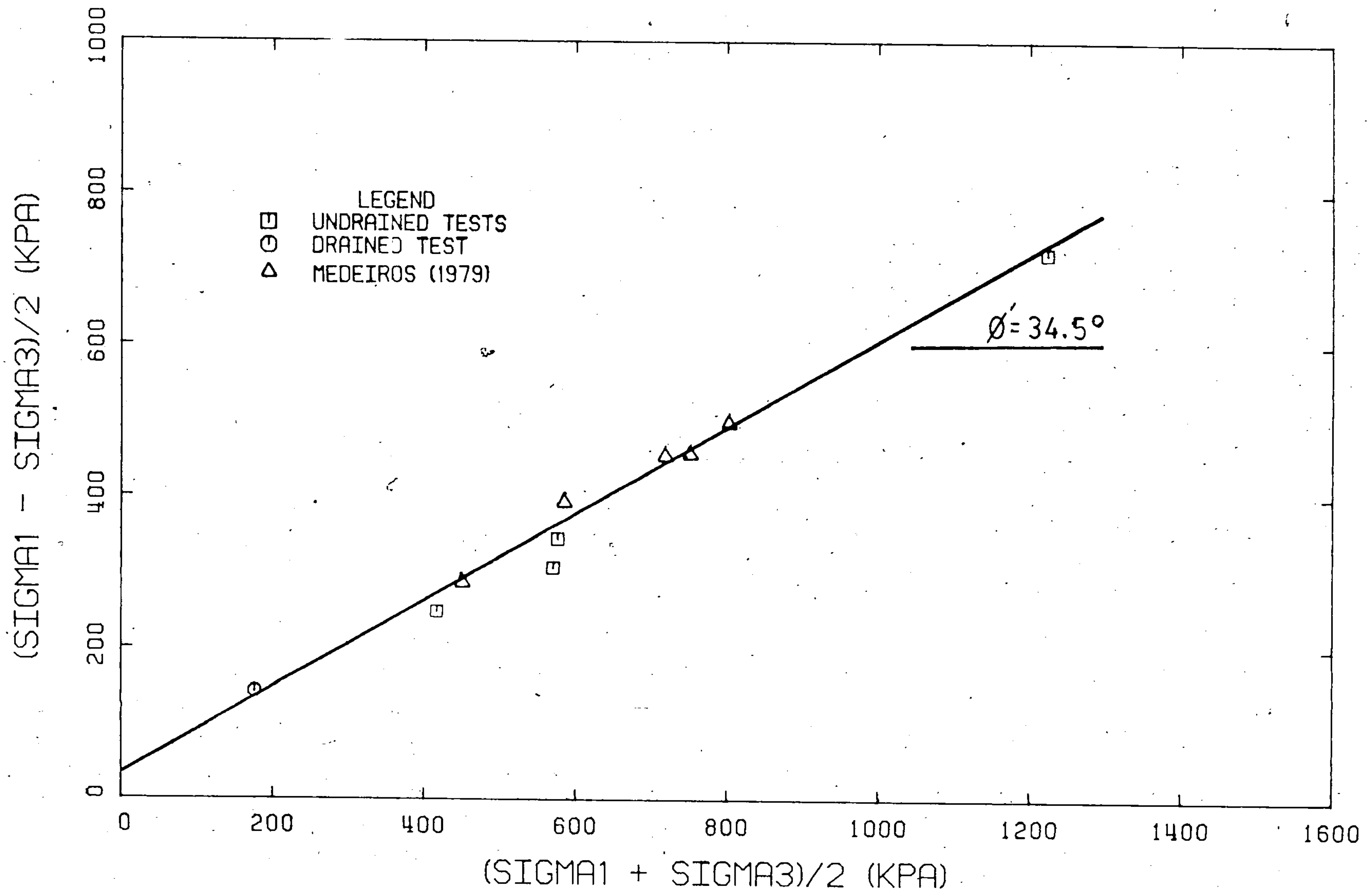

Figure 5.16 Effective Stress at Failure for Edmonton Till

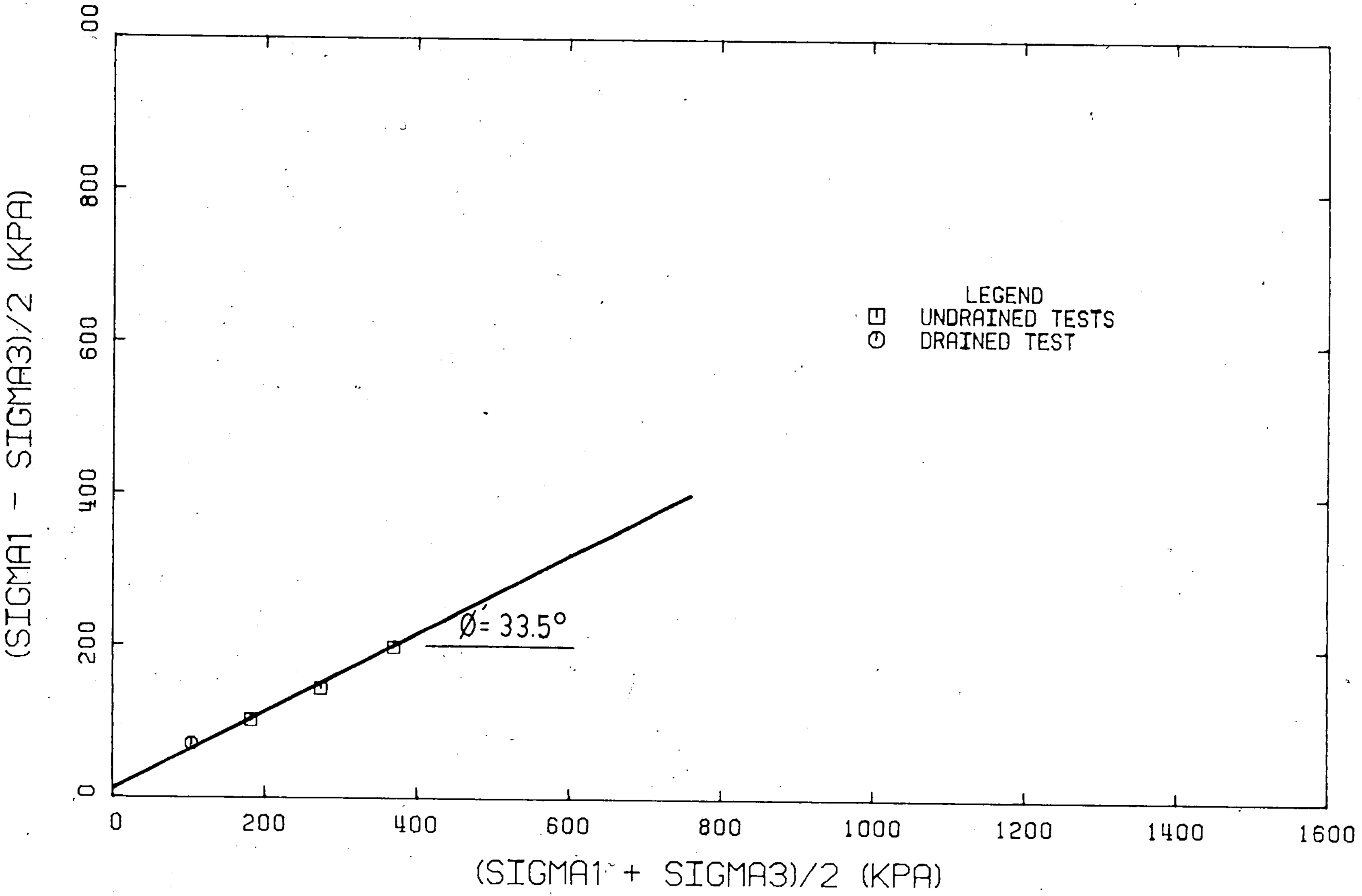

Figure 5.17 Effective Stress at Failure for Edmonton Till #1

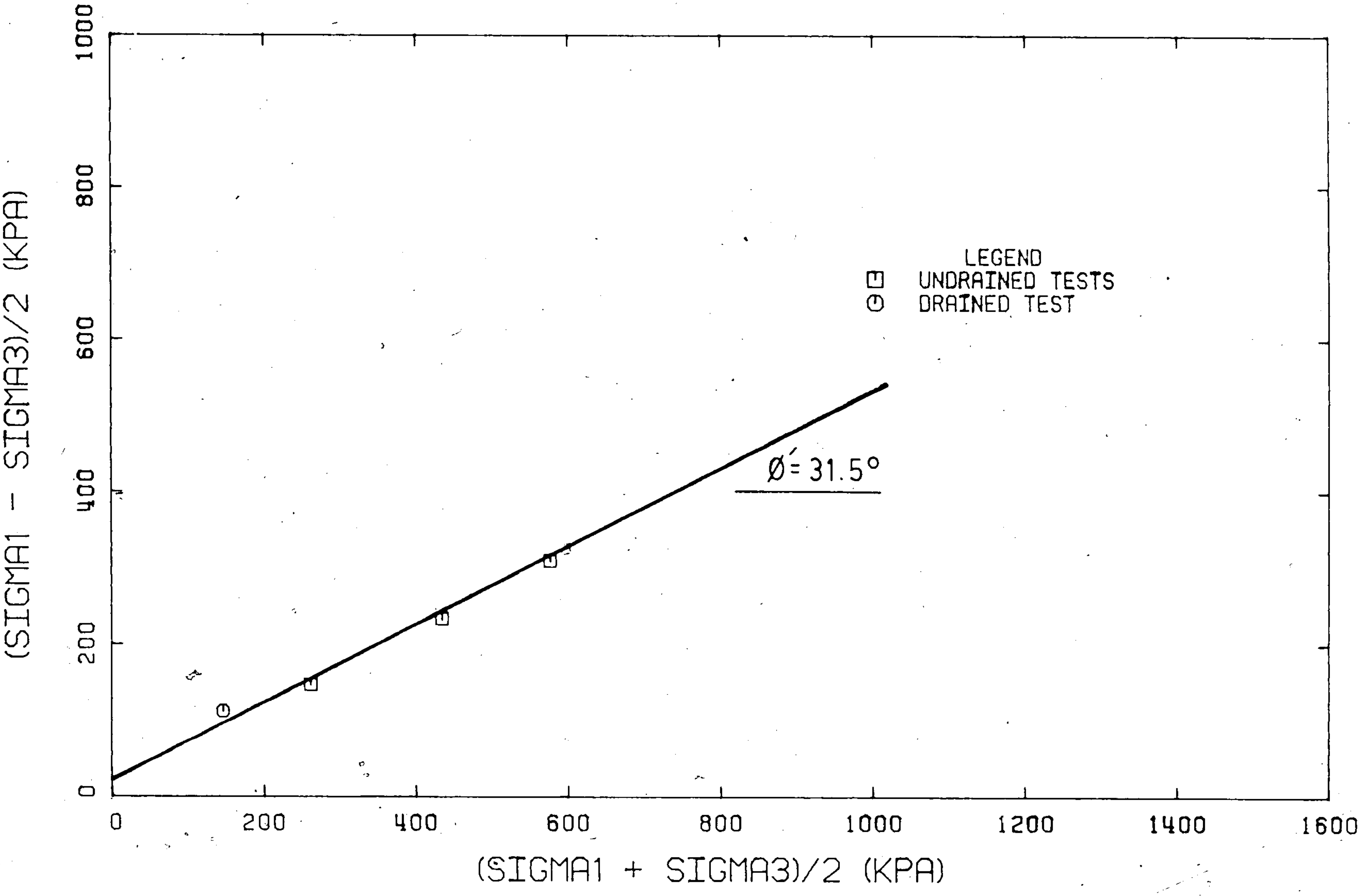

Figure 5.18 Effective Stress at Failure for Edmonton Till #2

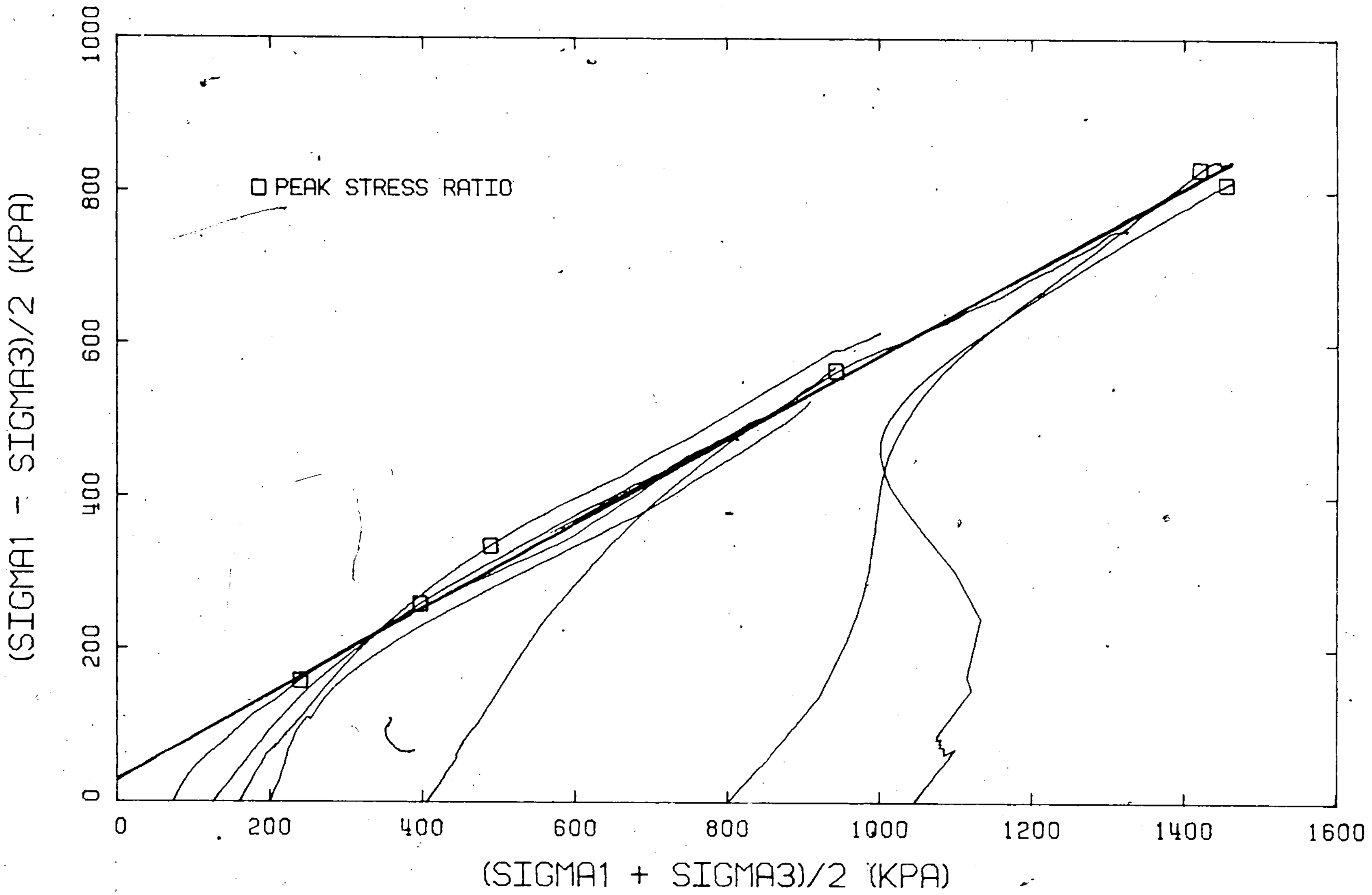

Figure 5.19 Effective Stress Plot for Nipawin Till

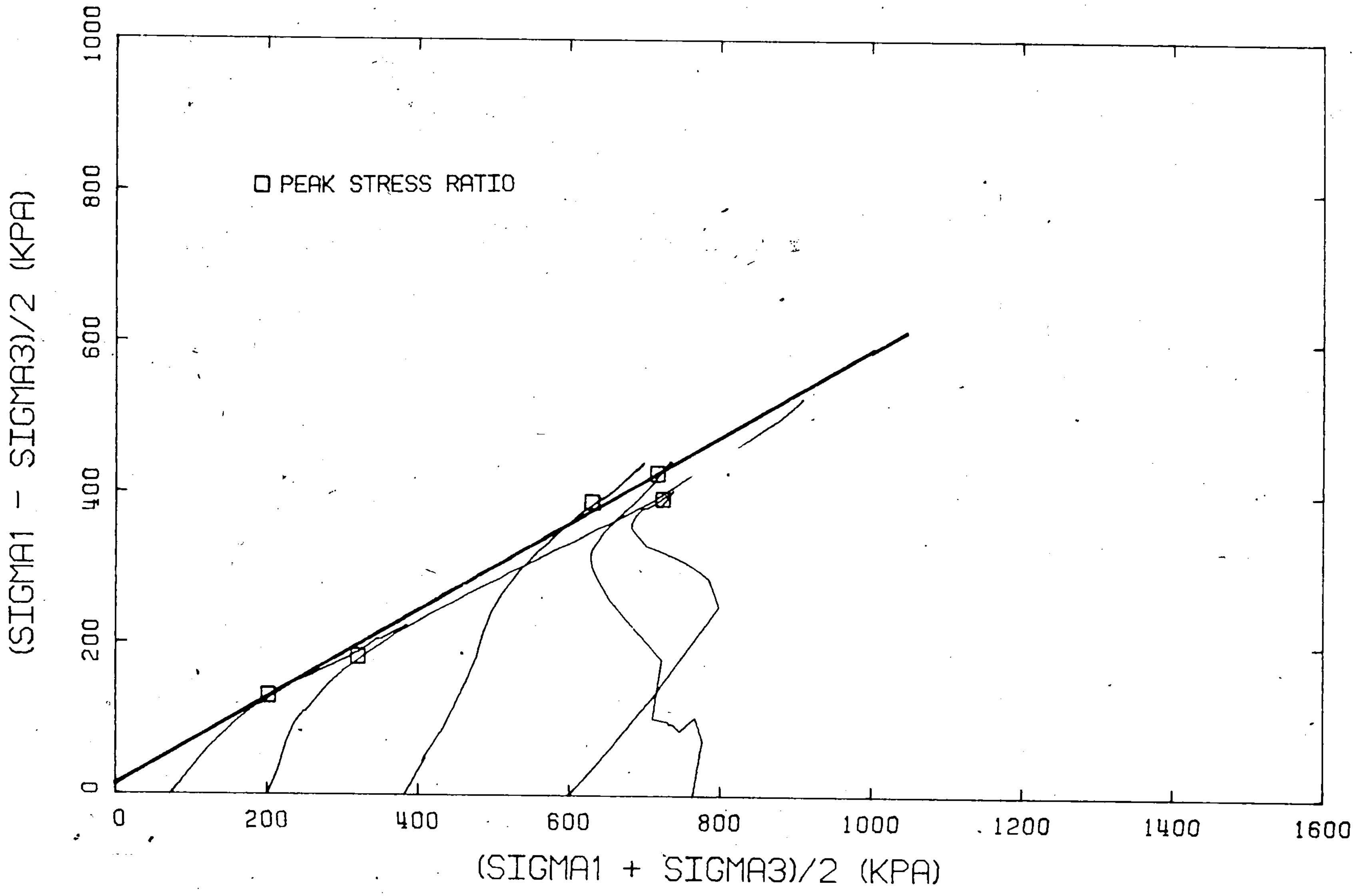

Figure 5.20 Effective Stress Plot for Nipawin Till #1

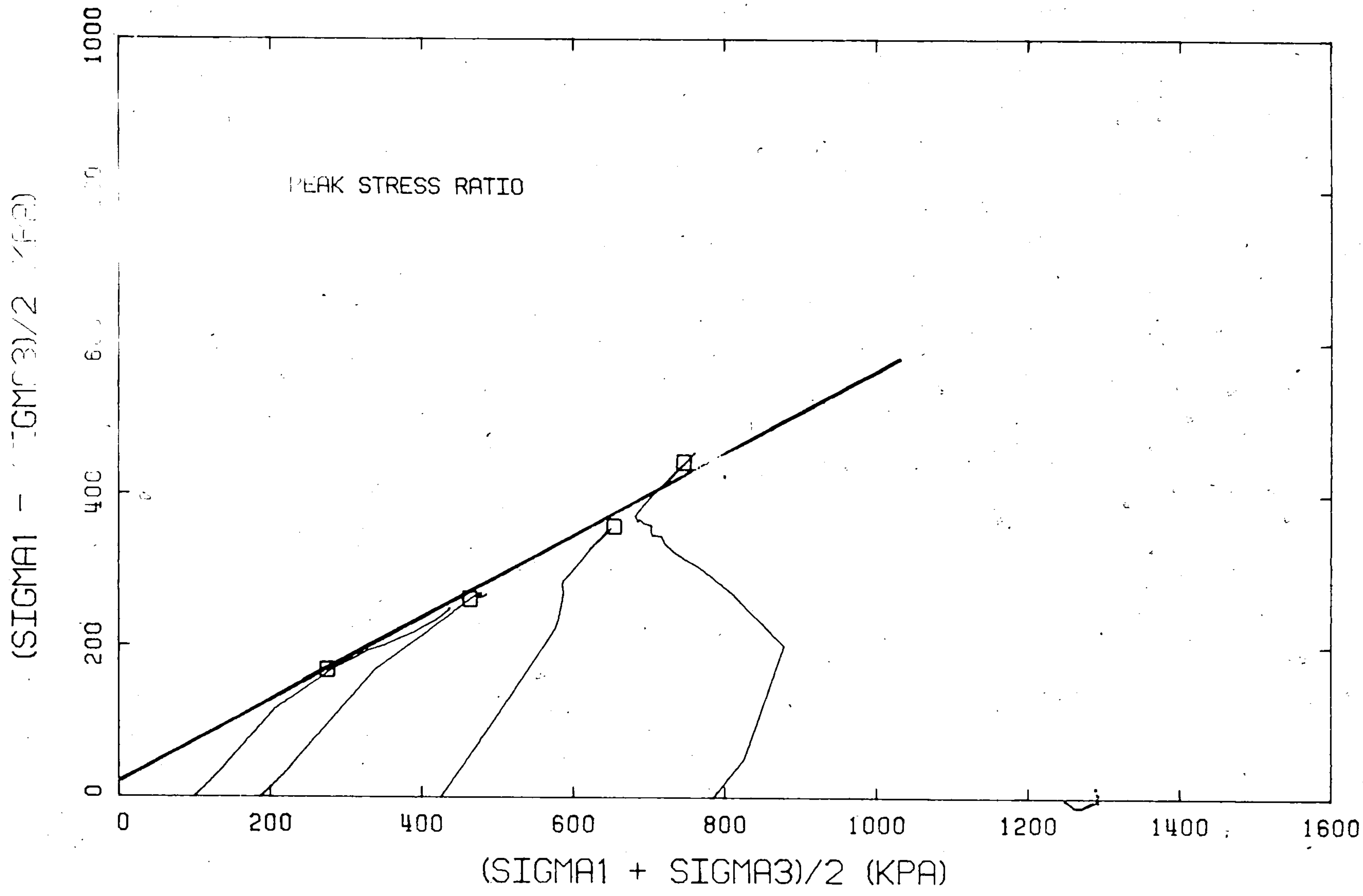

Figure 5.21 Effective Stress Plot for Nipawin Till #2

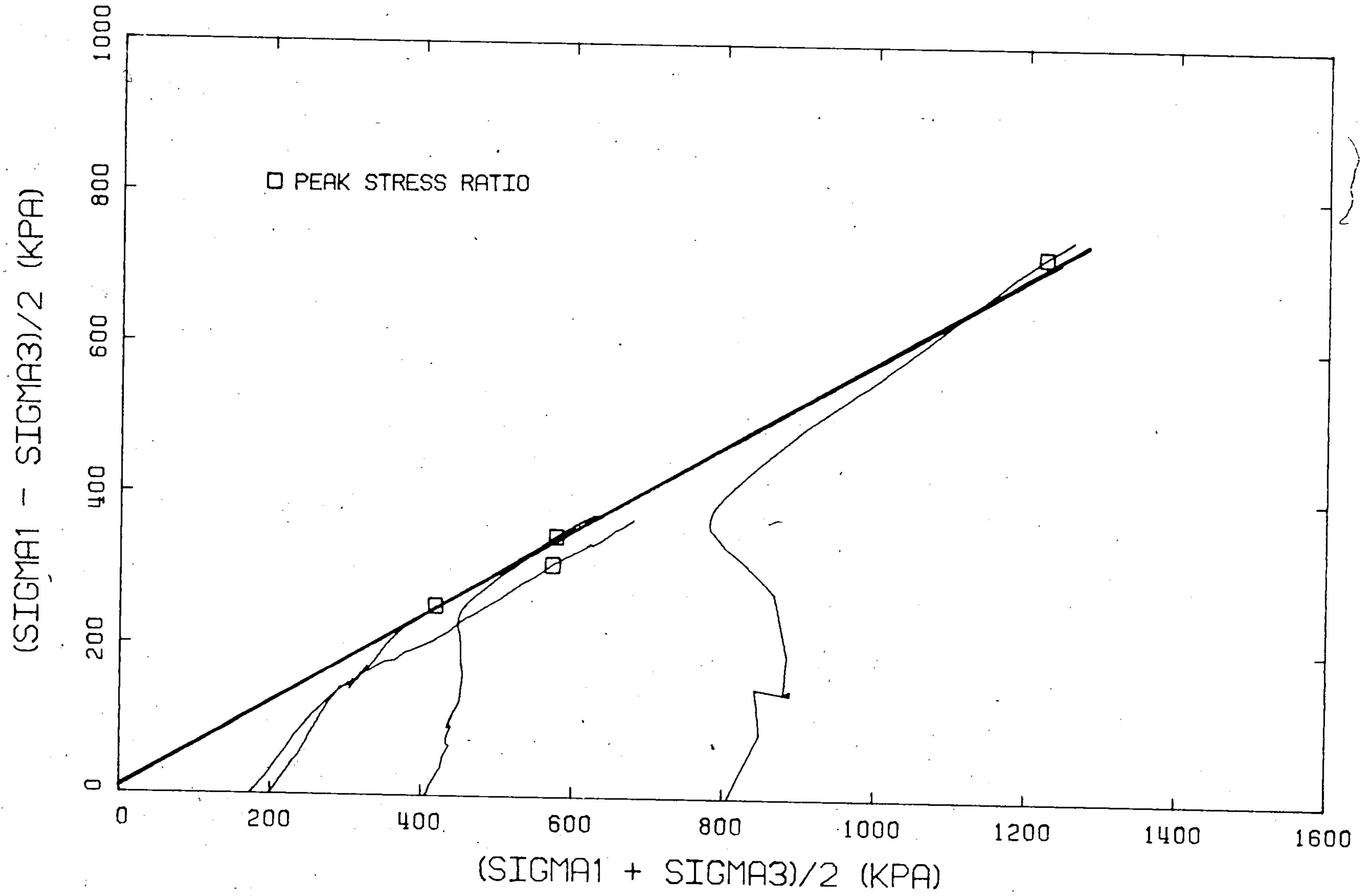

Figure 5.22 Effective Stress Plot for Edmonton Till

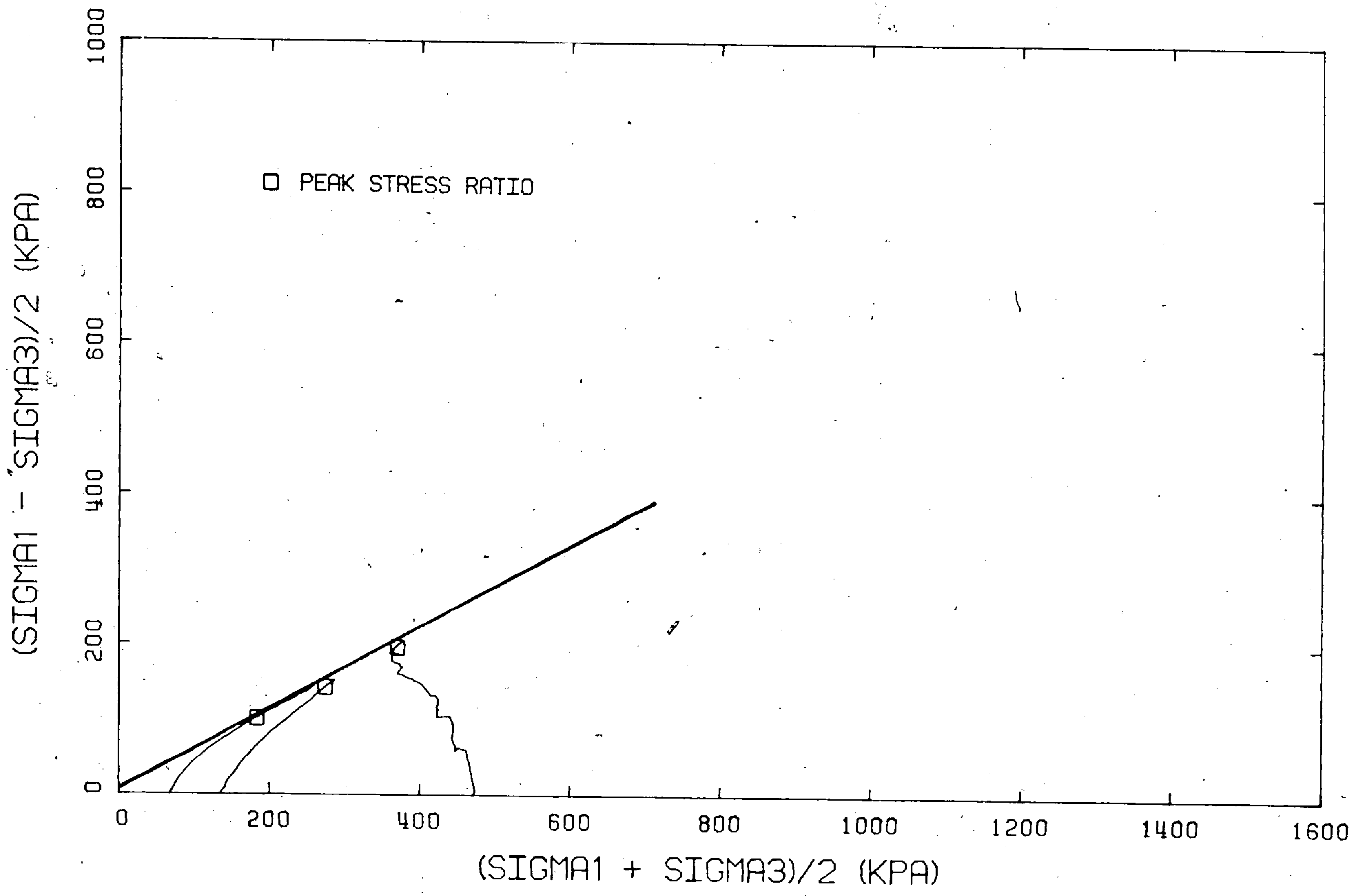

Figure 5.23 Effective Stress Plot for Edmonton Till

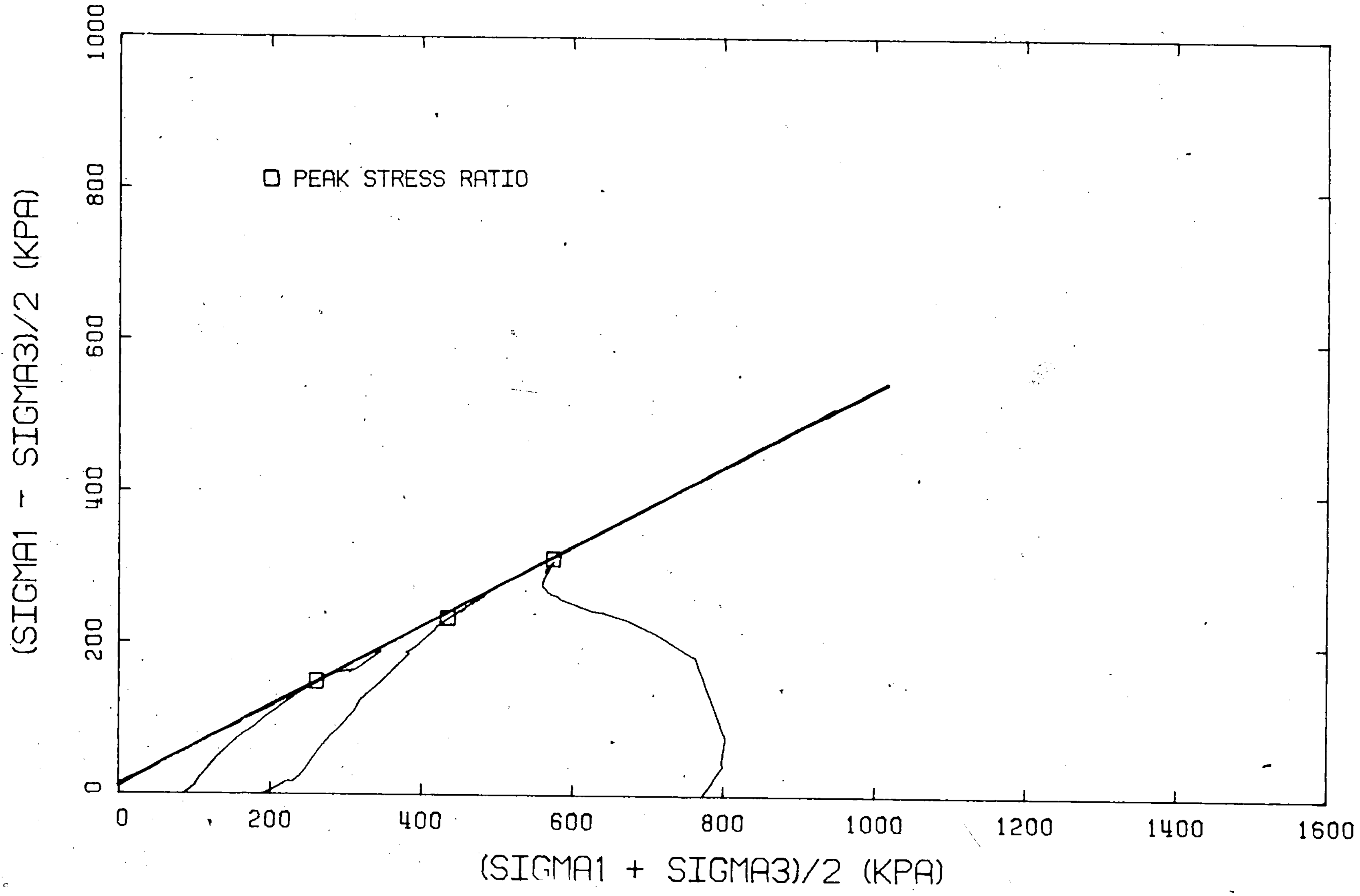

Figure 5.24 Effective Stress Plot for Edmonton Till #2

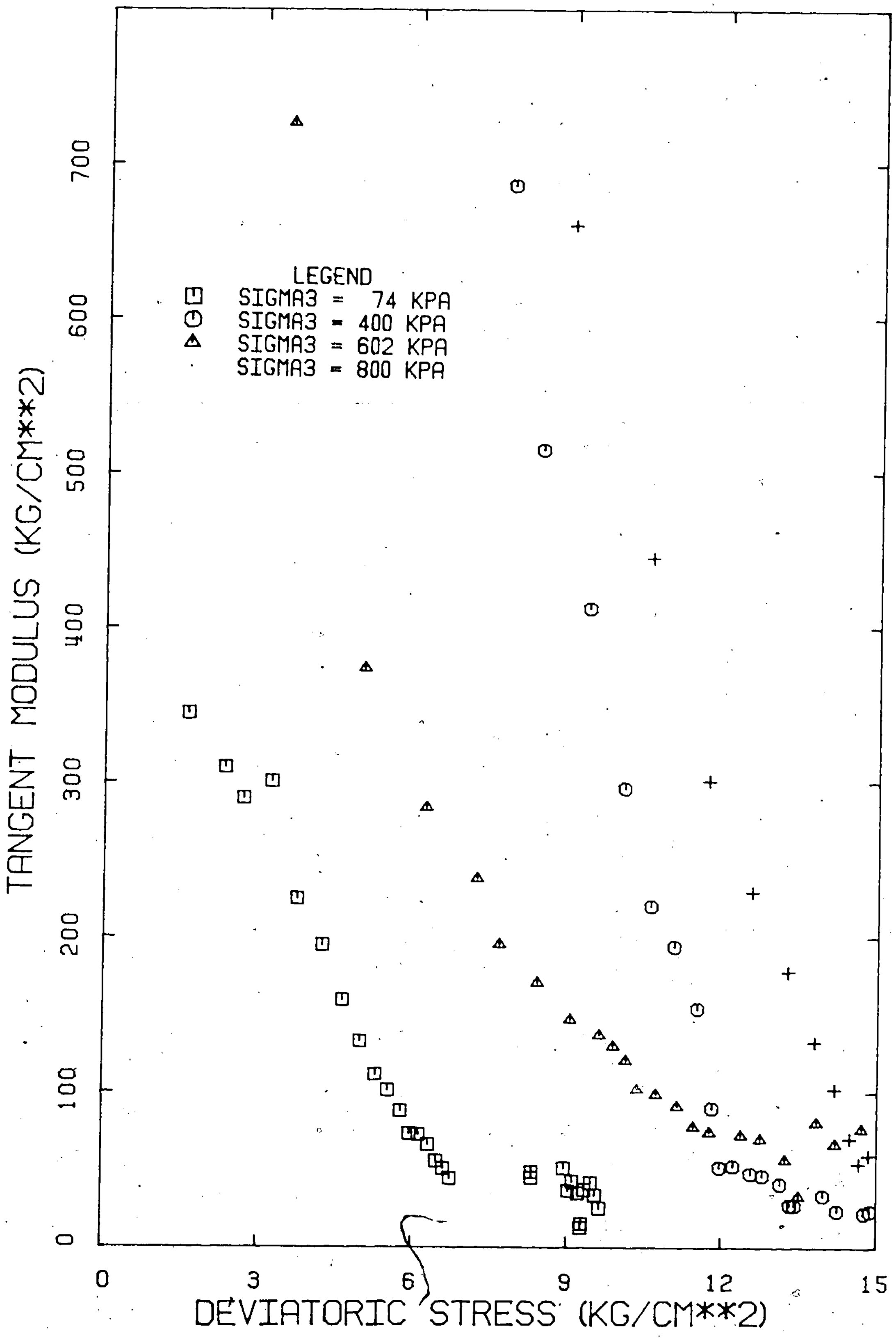

Figure 5.25 Deviatoric Stress versus Tangent Modulus for Nipawin Till

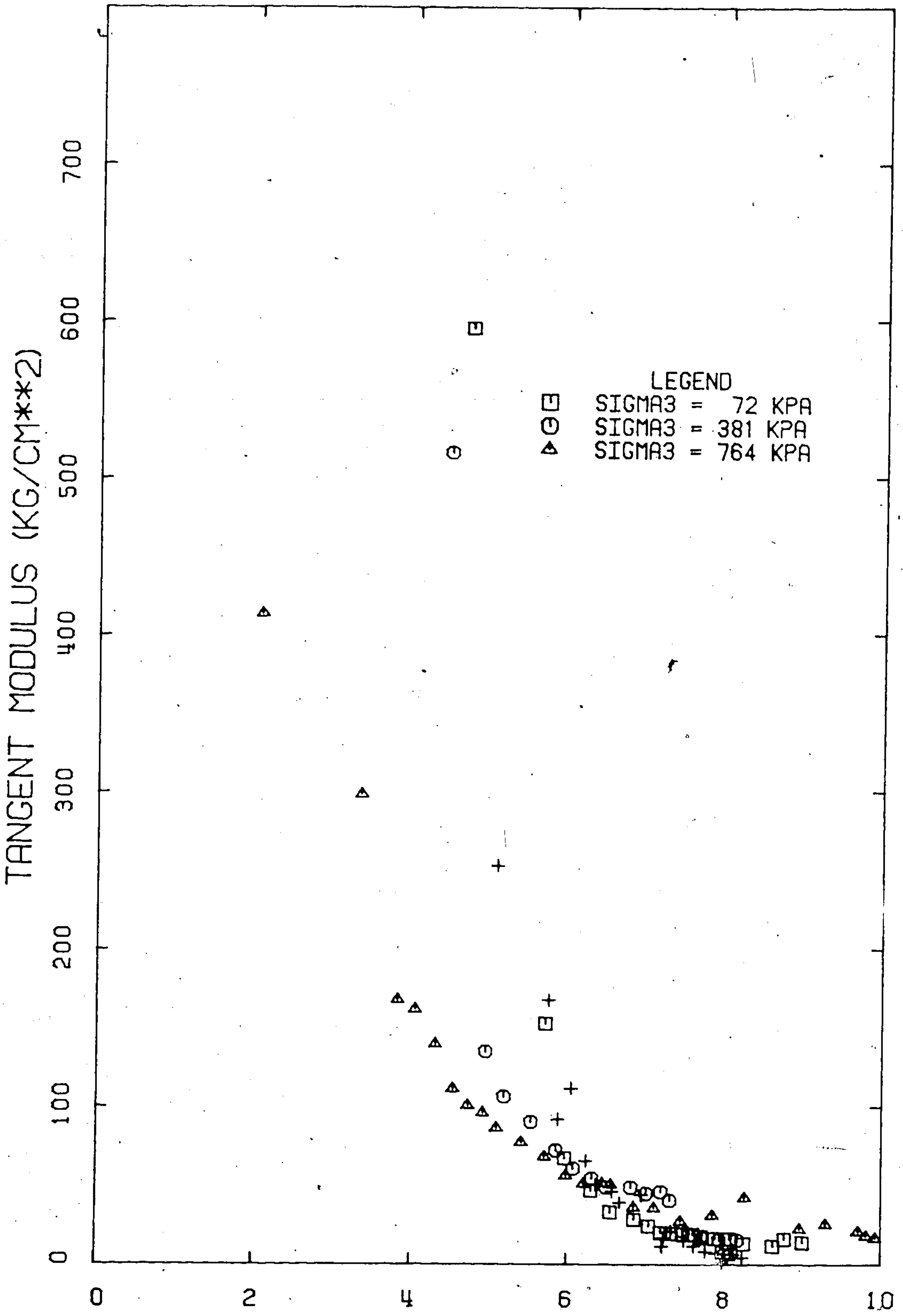

Figure 5.26 Deviatoric Stress versus Tangent Modulus for Nipawin Till #1

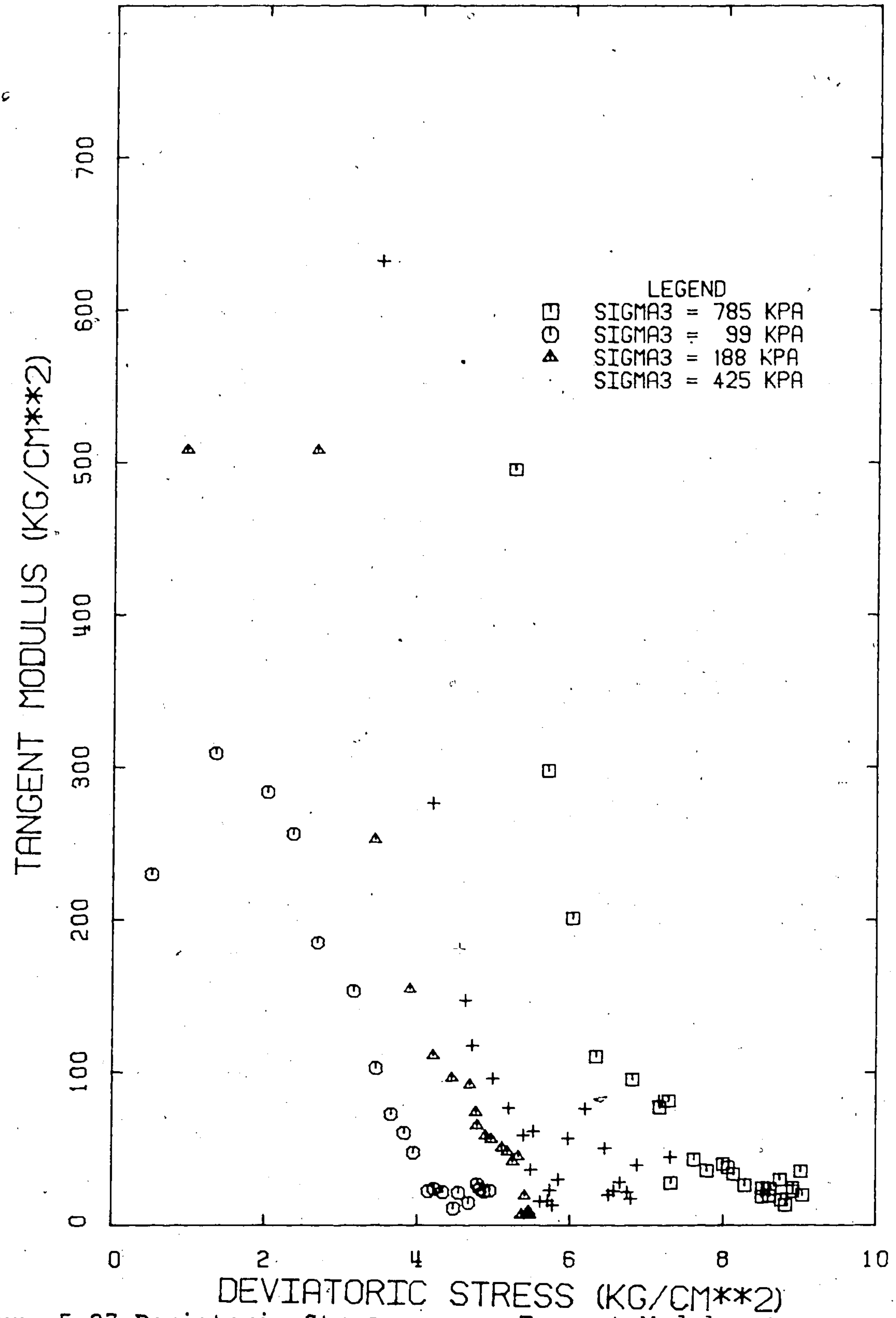

Figure 5.27 Deviatoric Stress versus Tangent Modulus for Nipawin Till #2

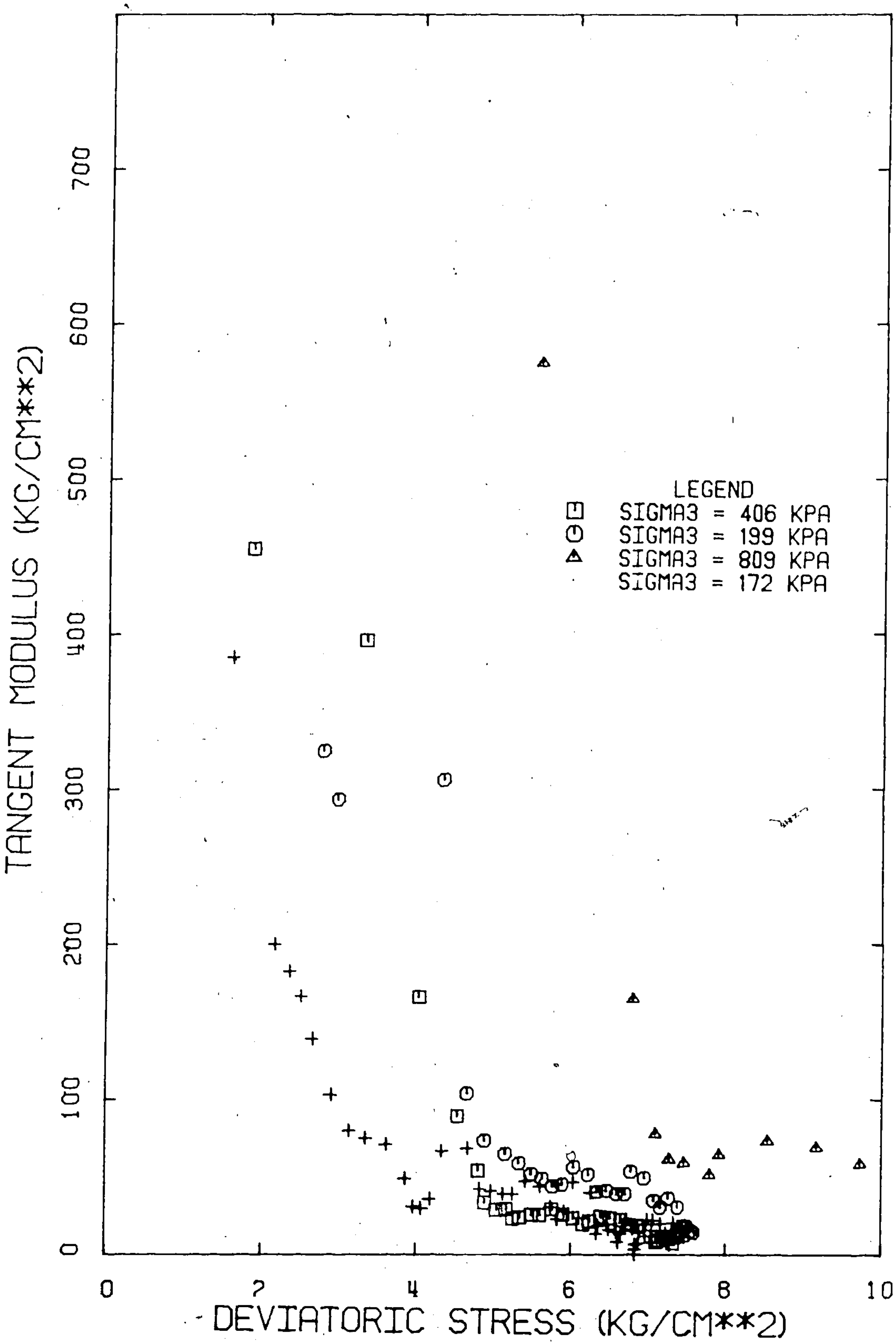

Figure 5.28 Deviatoric Stress versus Tangent Modulus for Edmonton Till

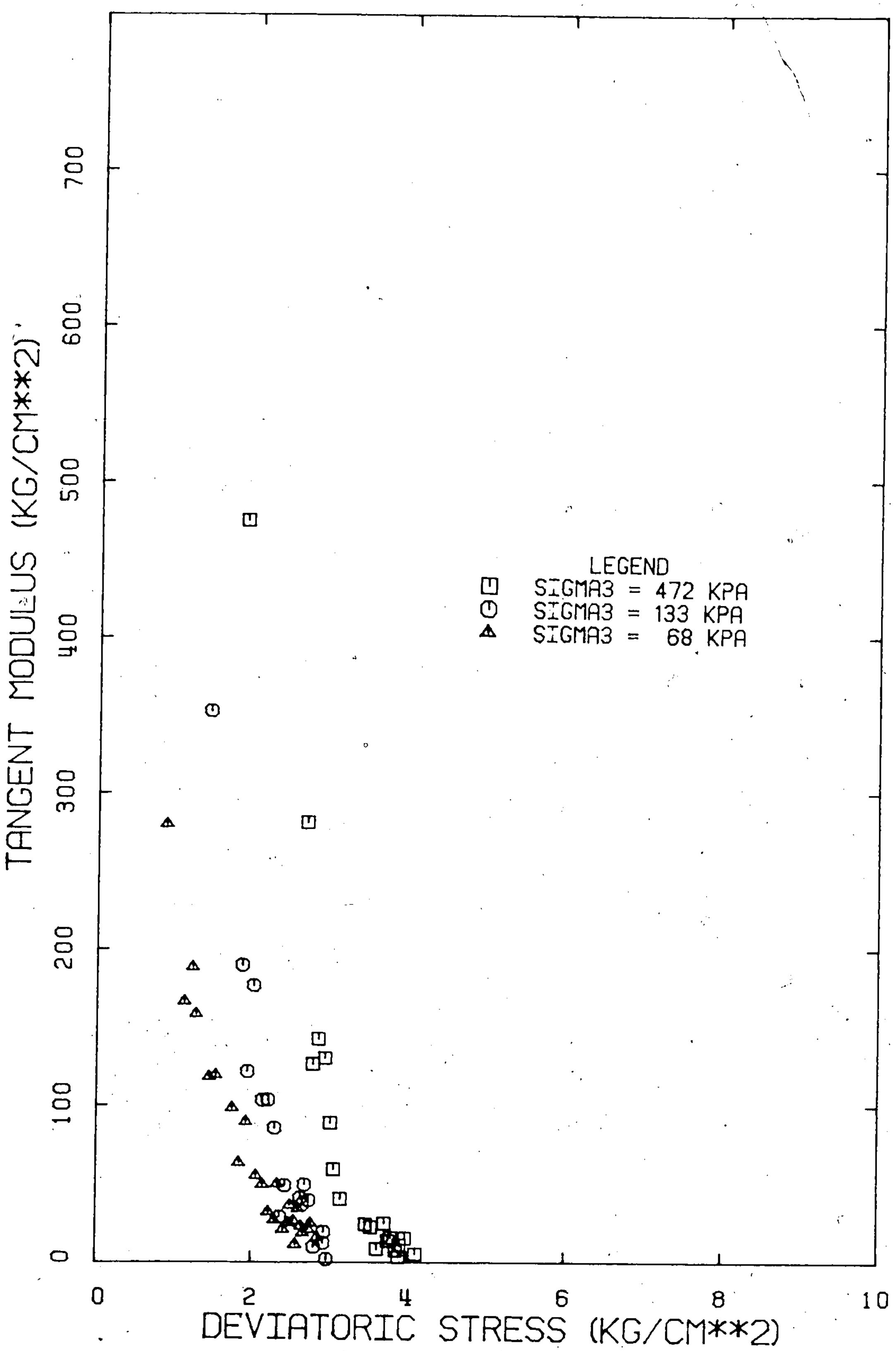

Figure 5.29 Deviatoric Stress versus Tangent Modulus for Edmonton Till #1

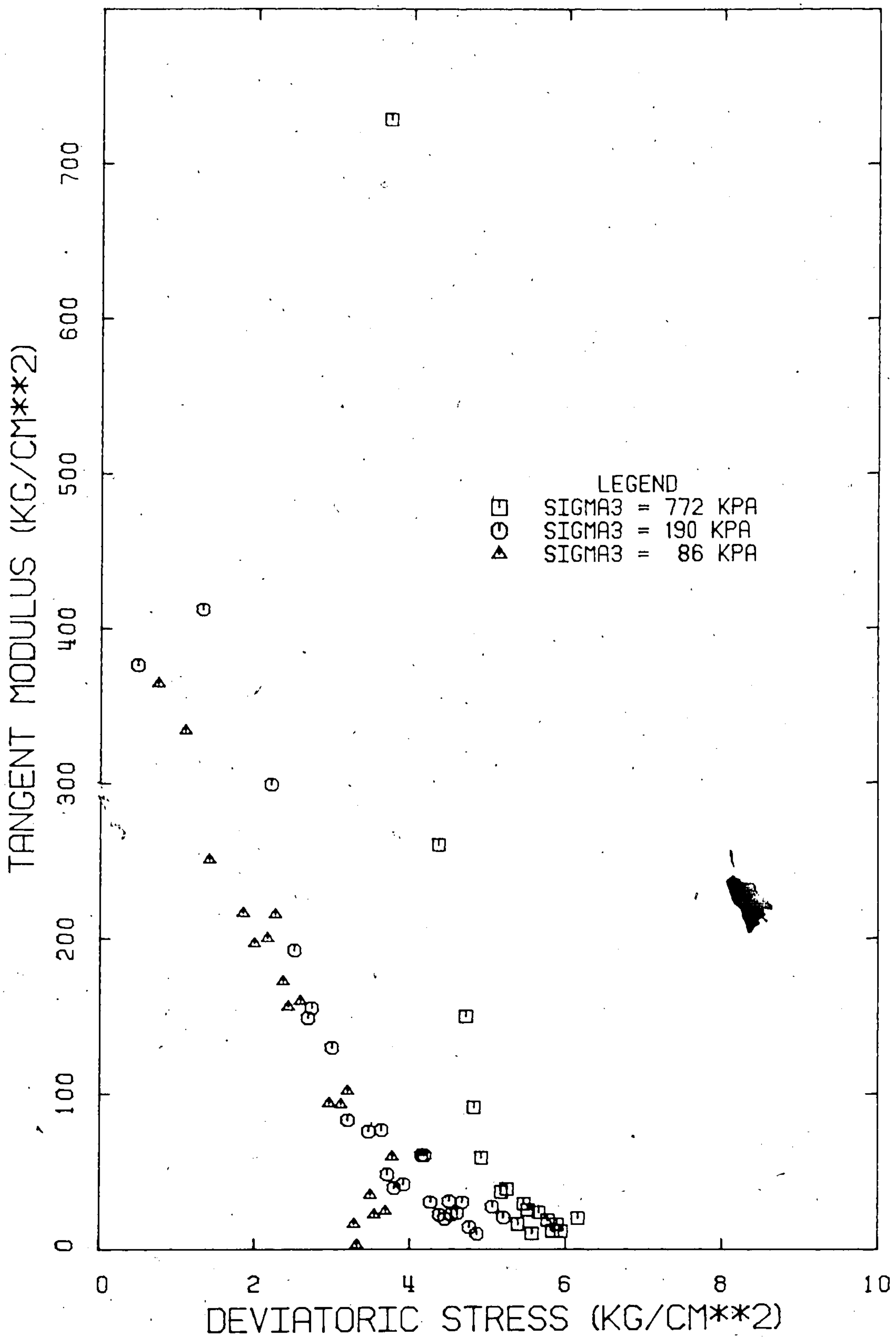

Figure 5.30 Deviatoric Stress versus Tangent Modulus for Edmonton Till #2

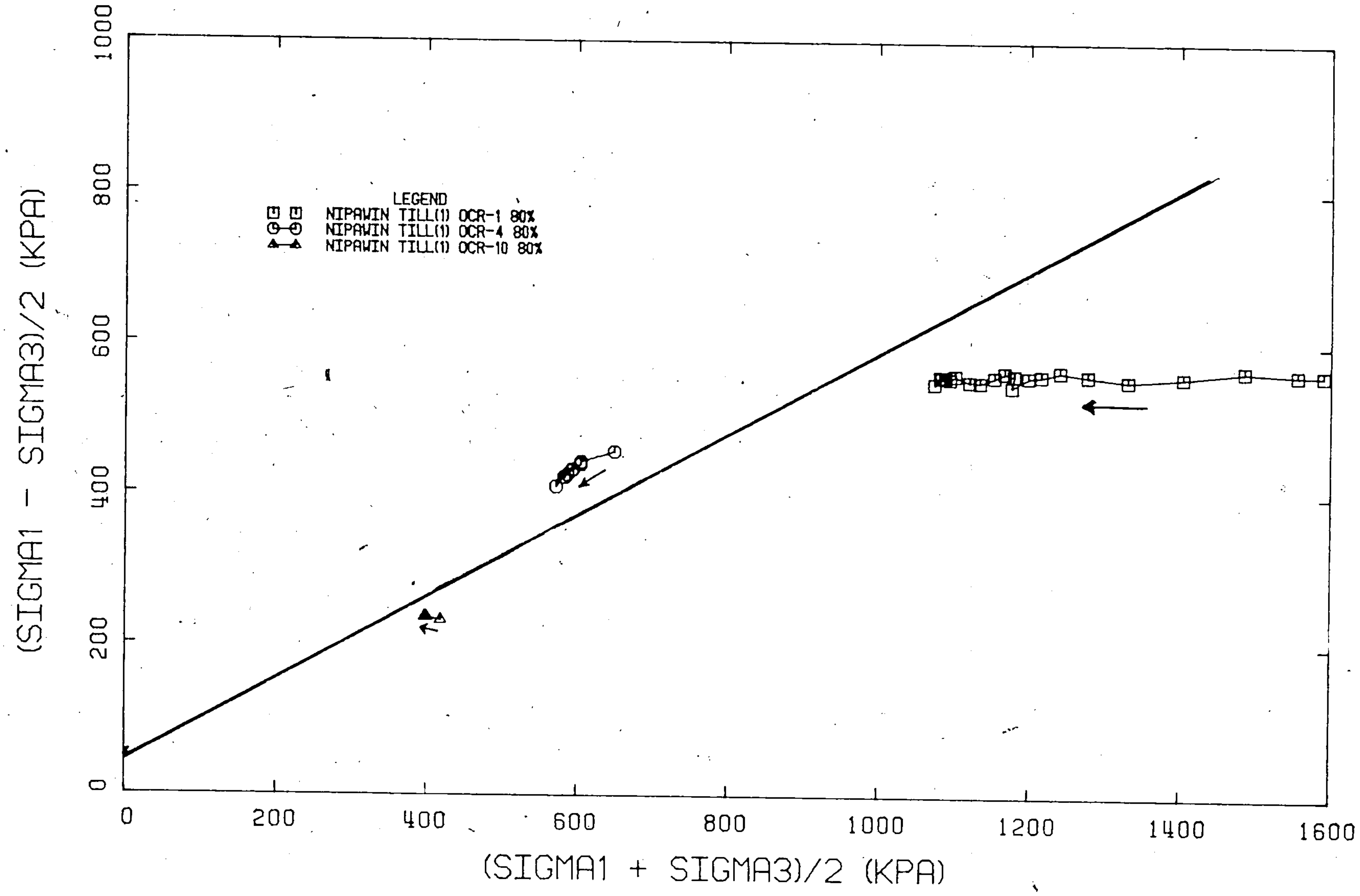

Figure 5.31 Cyclic Effective Stress Plots for Nipawin Till

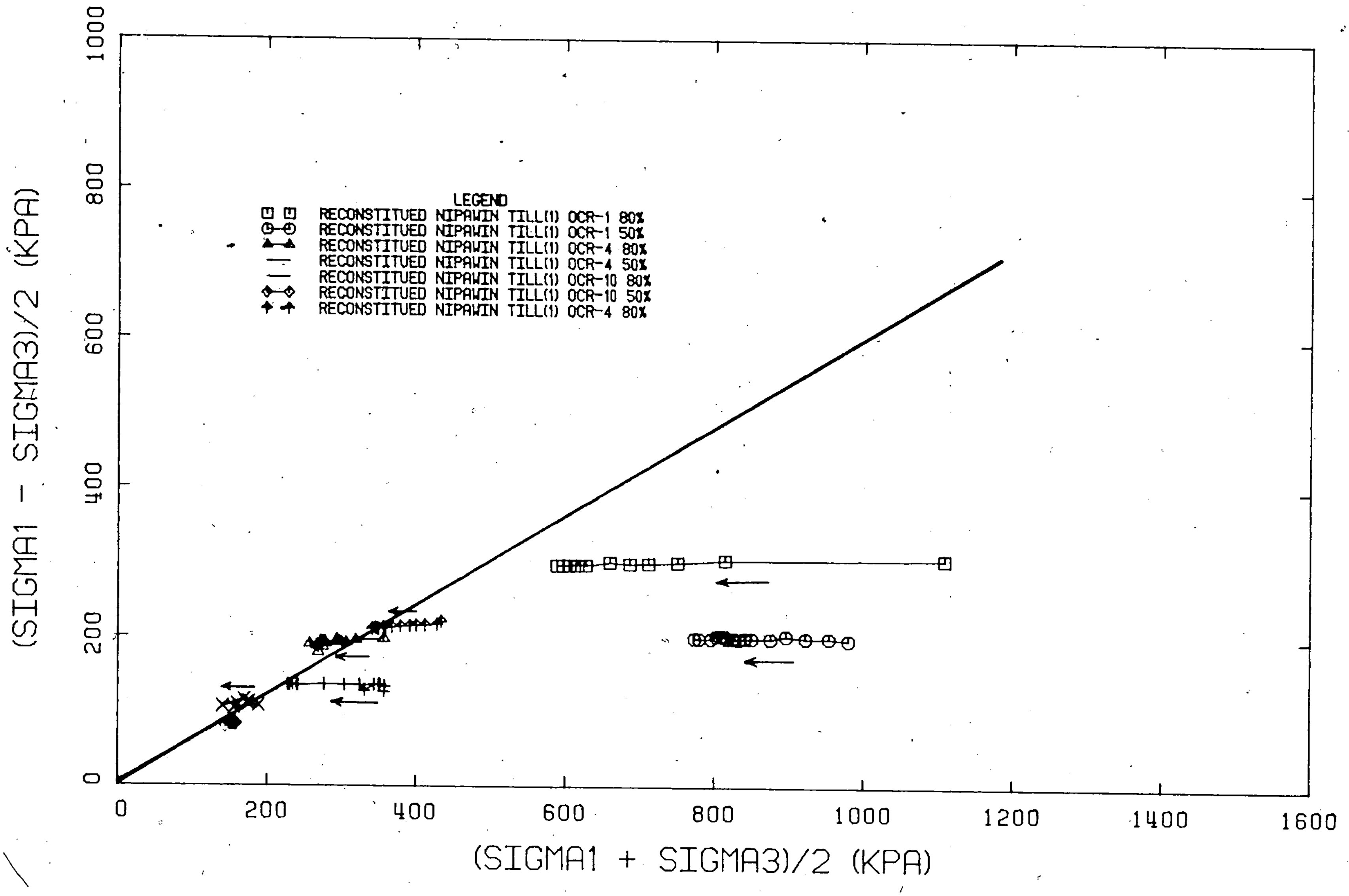

Figure 5.32 Cyclic Effective Stress Plots for Nipawin Till #1

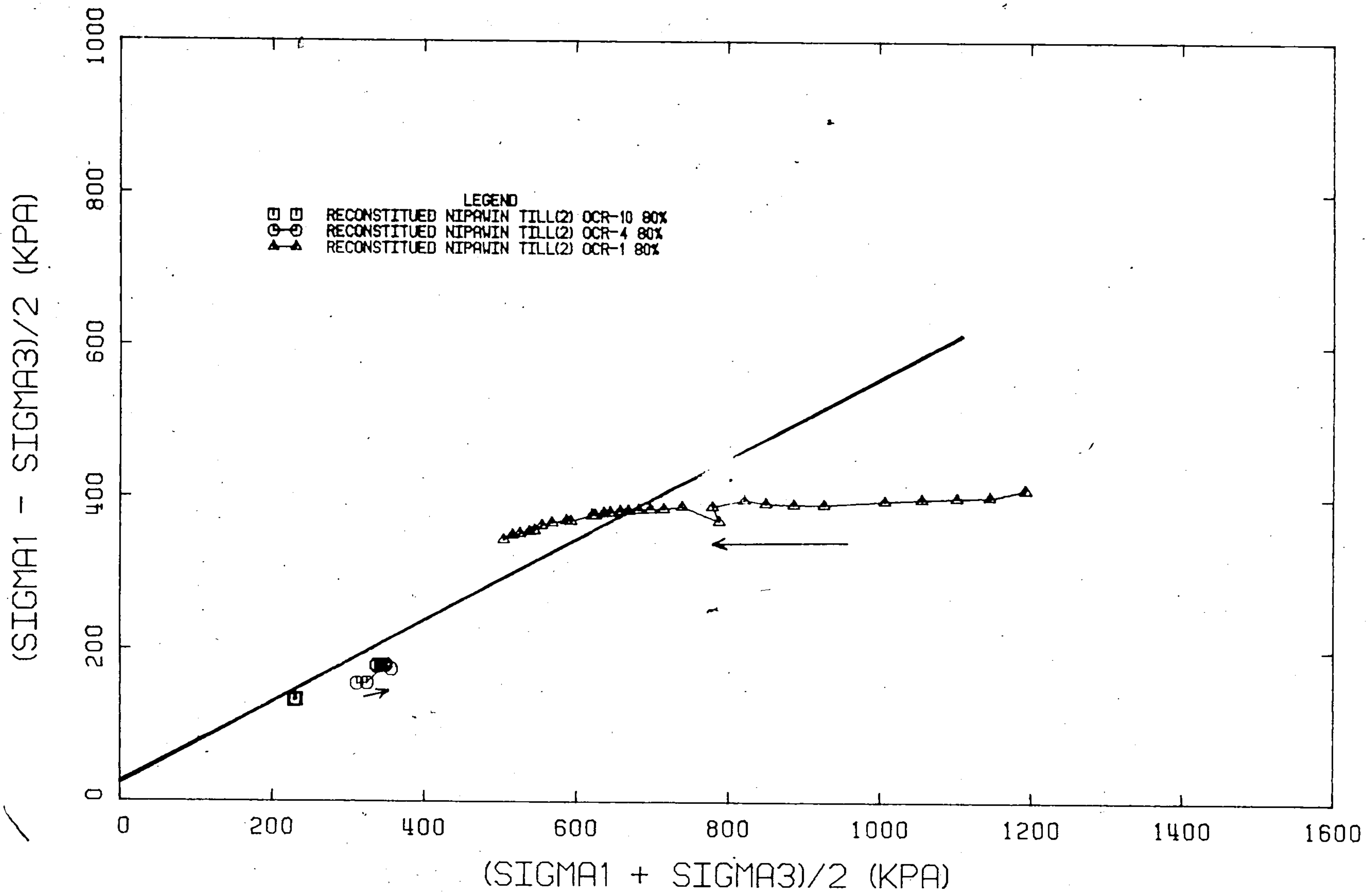

Figure 5.33 Cyclic Effective Stress Plots for Nipawin Till #2

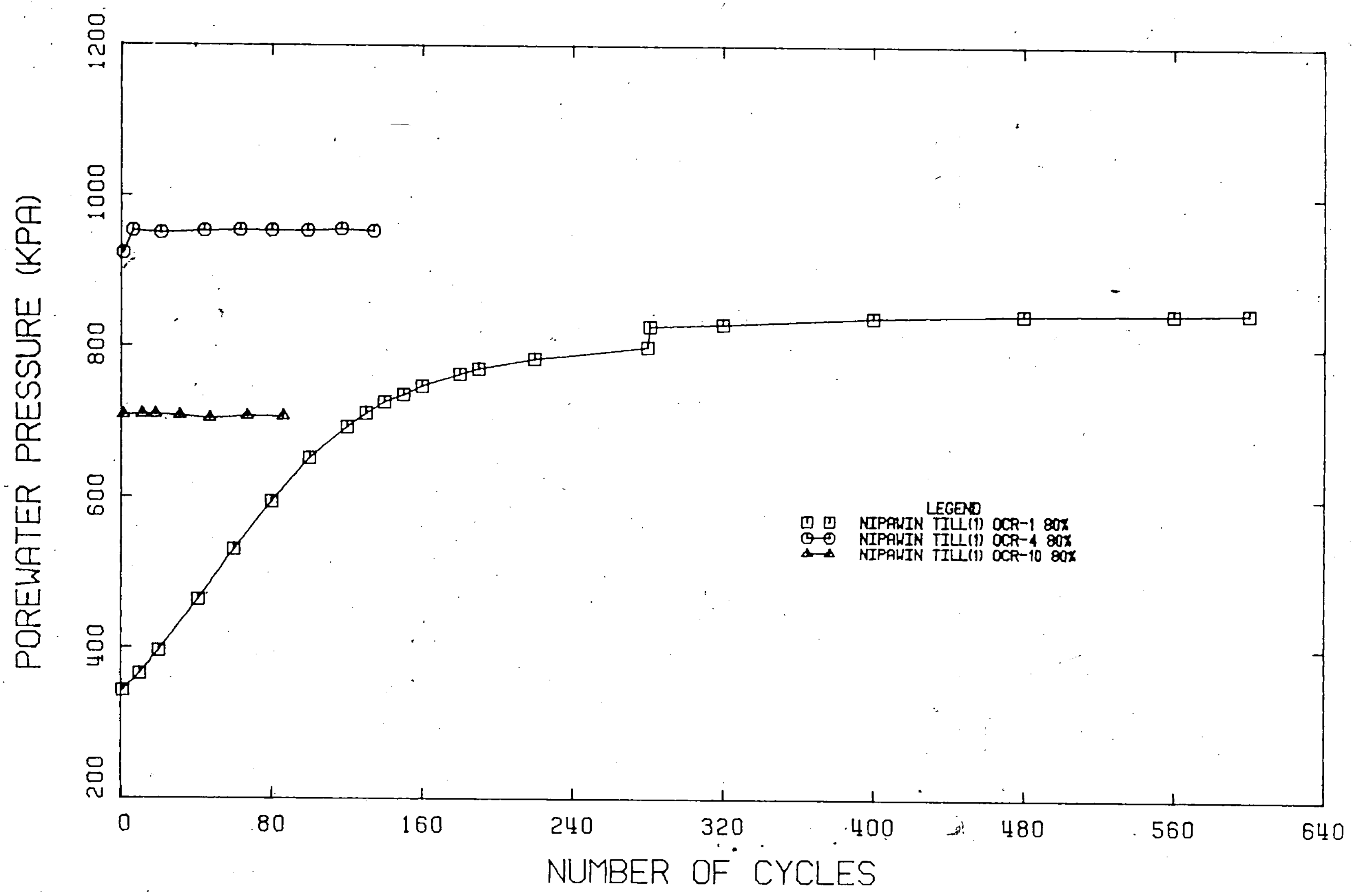

Figure 5.34 Cyclic Pore Pressure Response for Nipawin Till

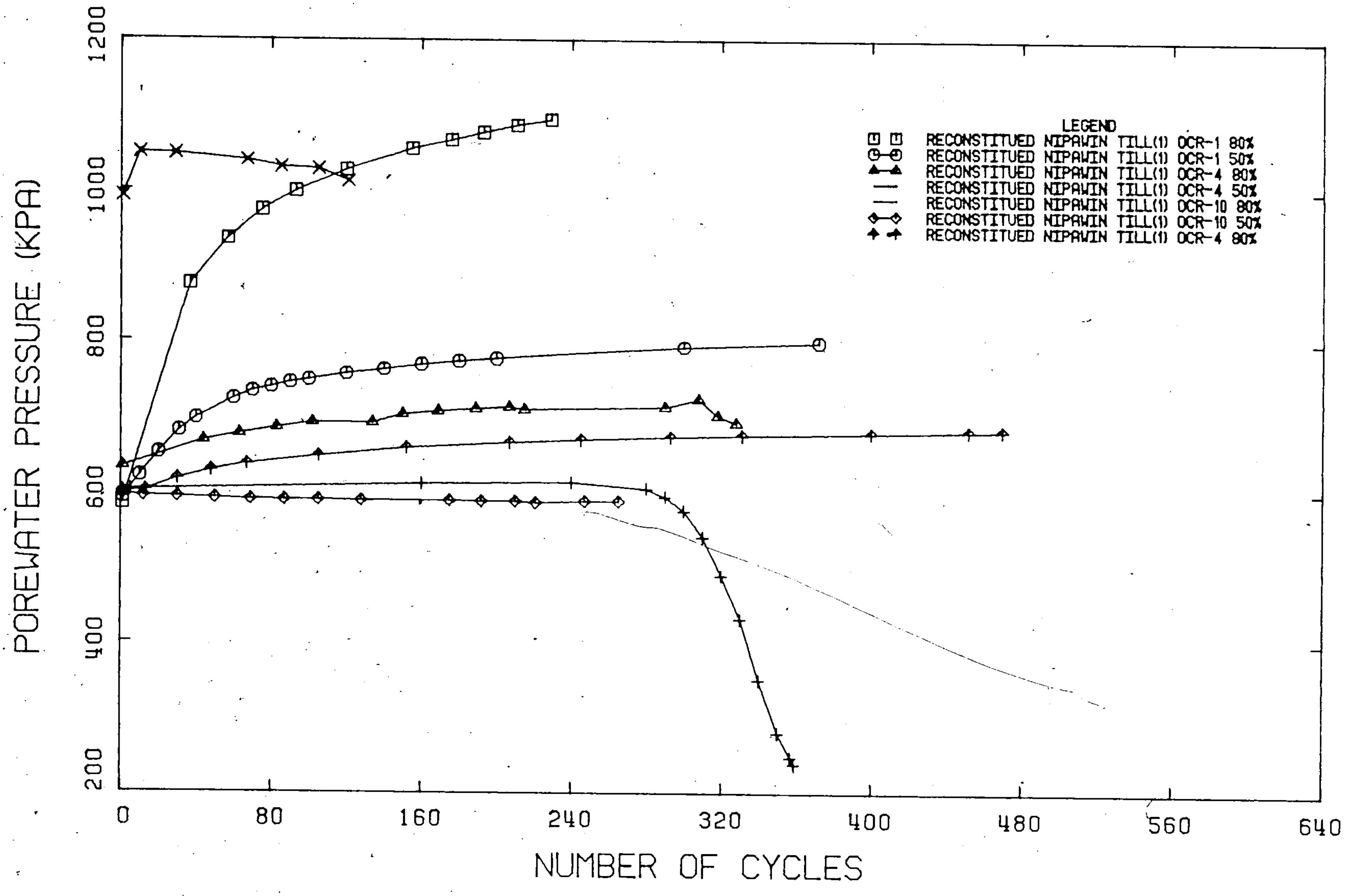

Figure 5.35 Cyclic Pore Pressure Response for Nipawin Till #1

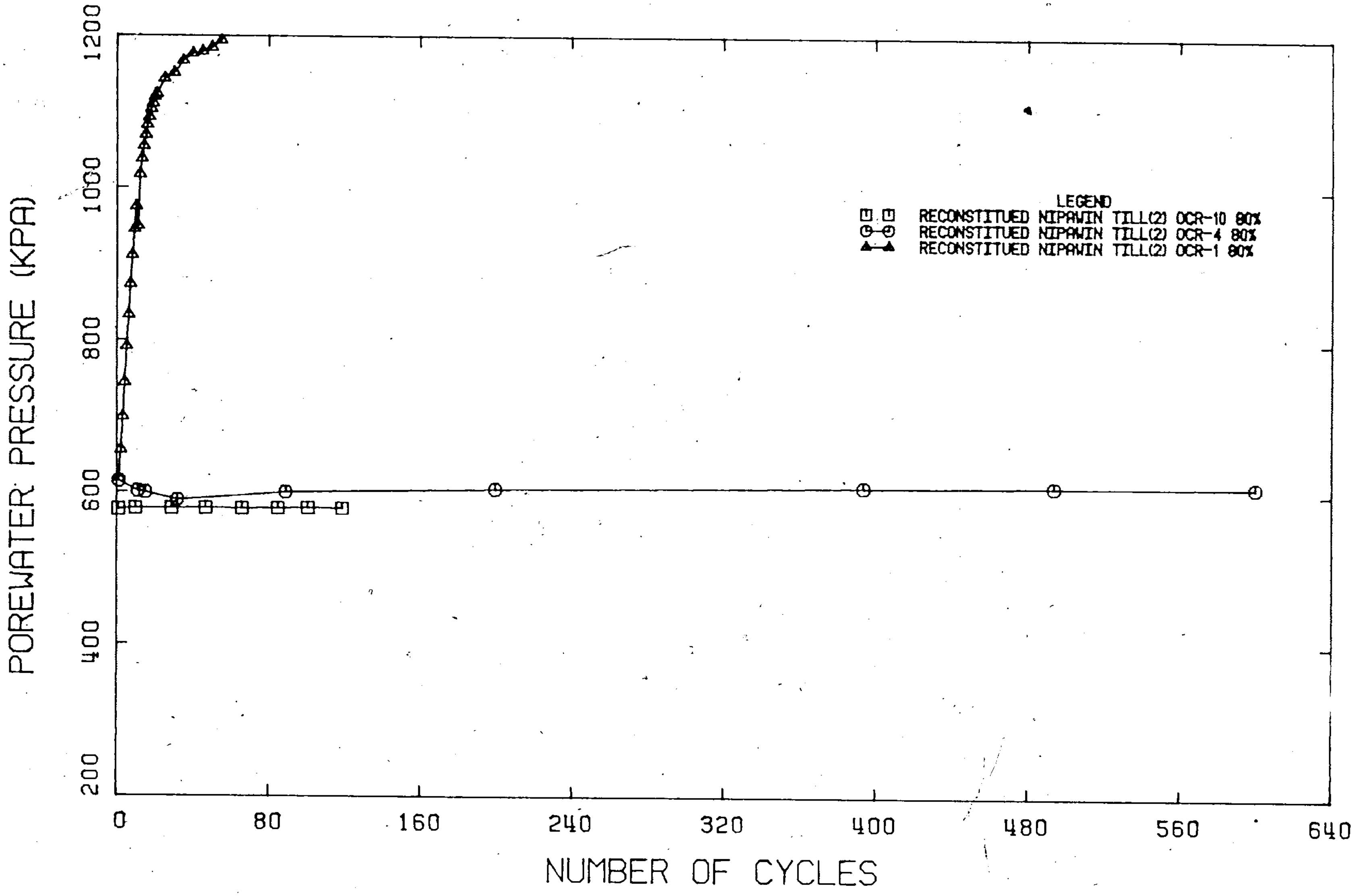

Figure 5.36 Cyclic Pore Pressure Response for Nipawin Till #2

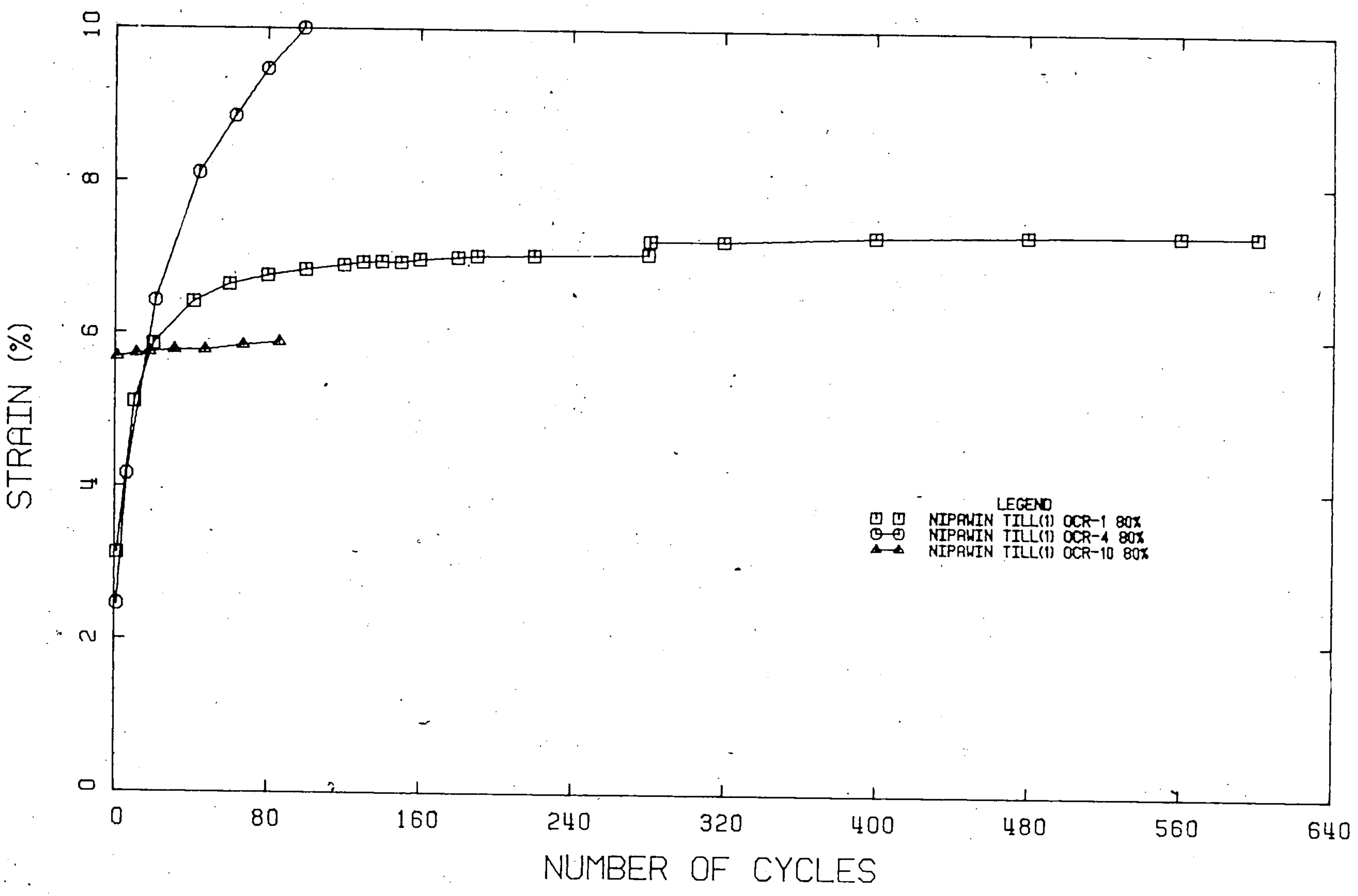

Figure 5.37 Cyclic Strain Response for Nipawin Till

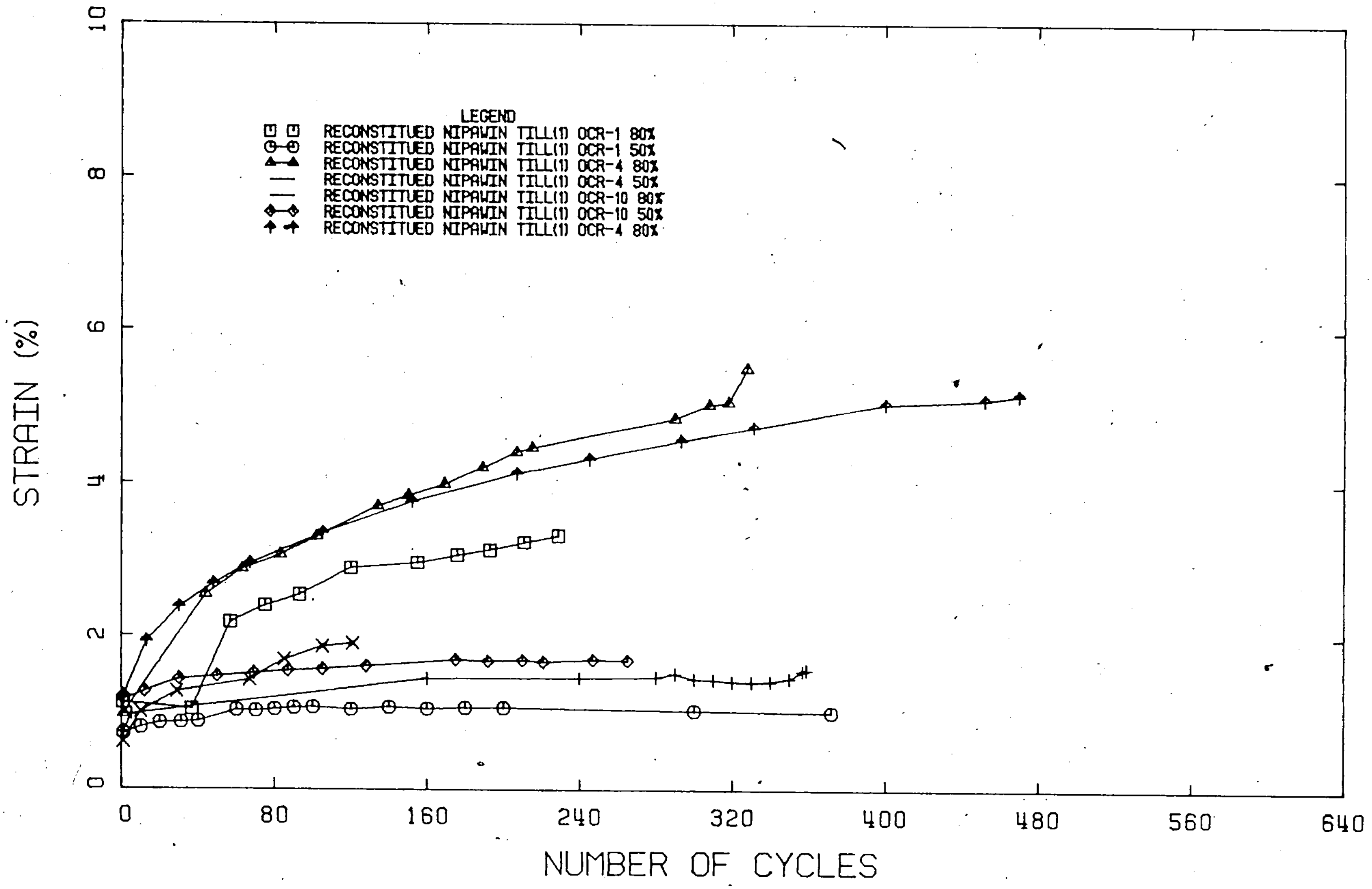

Figure 5.38 Cyclic Strain Response for Nipawin Till #1

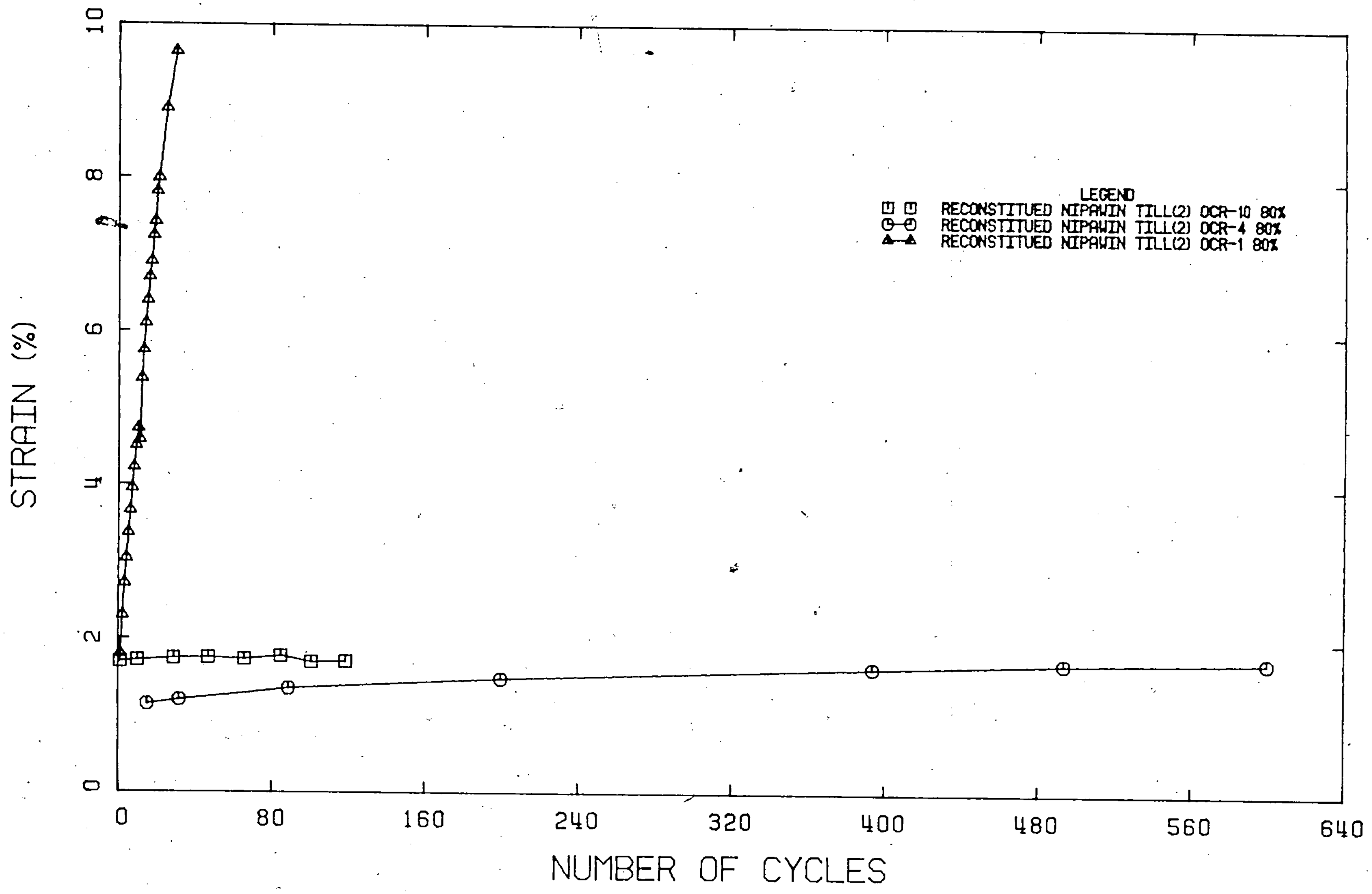

Figure 5.39 Cyclic Strain Response for Nipawin Till #2

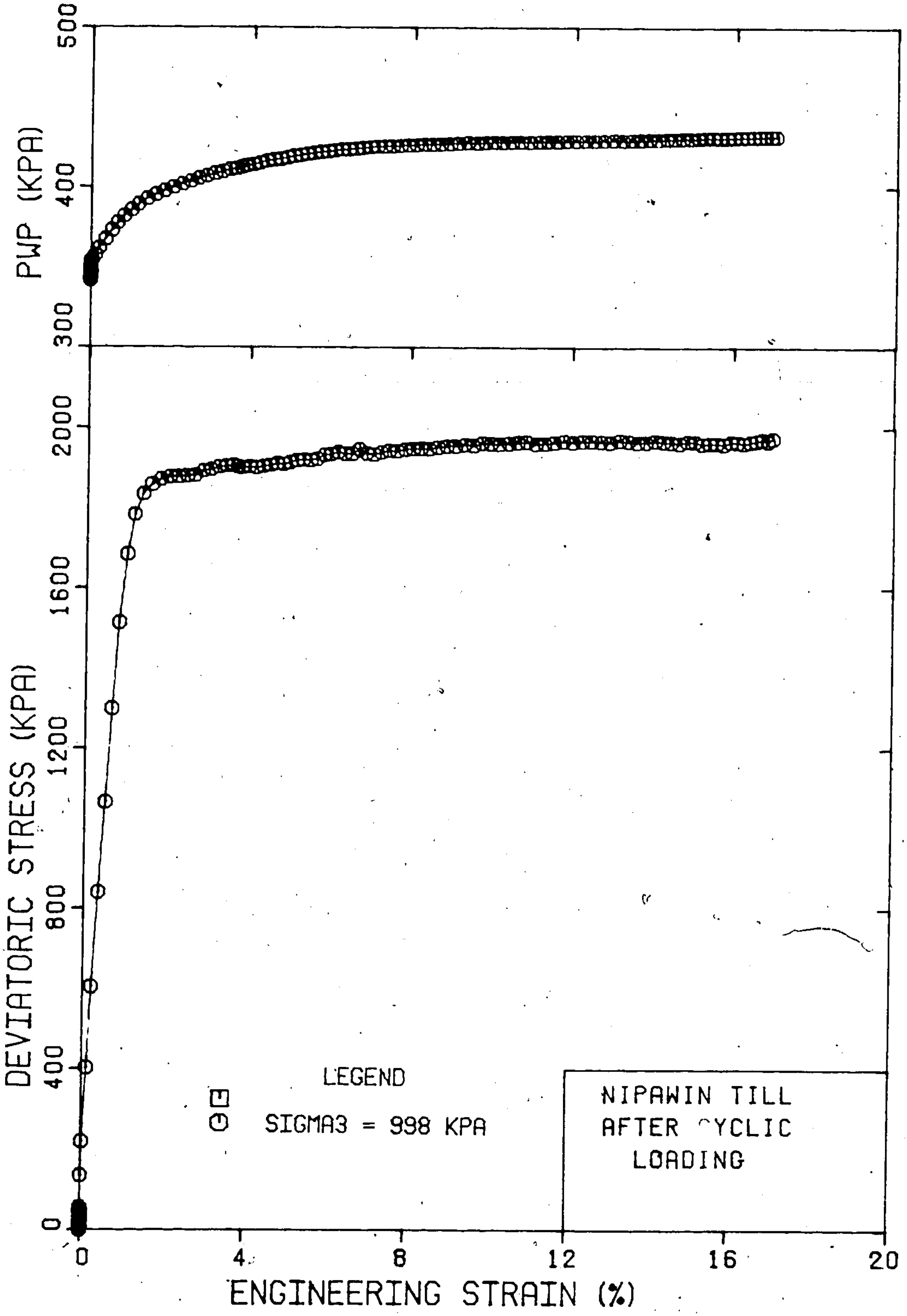

Figure 5.40 Stress-strain curves for Nipawin Till after cyclic loading

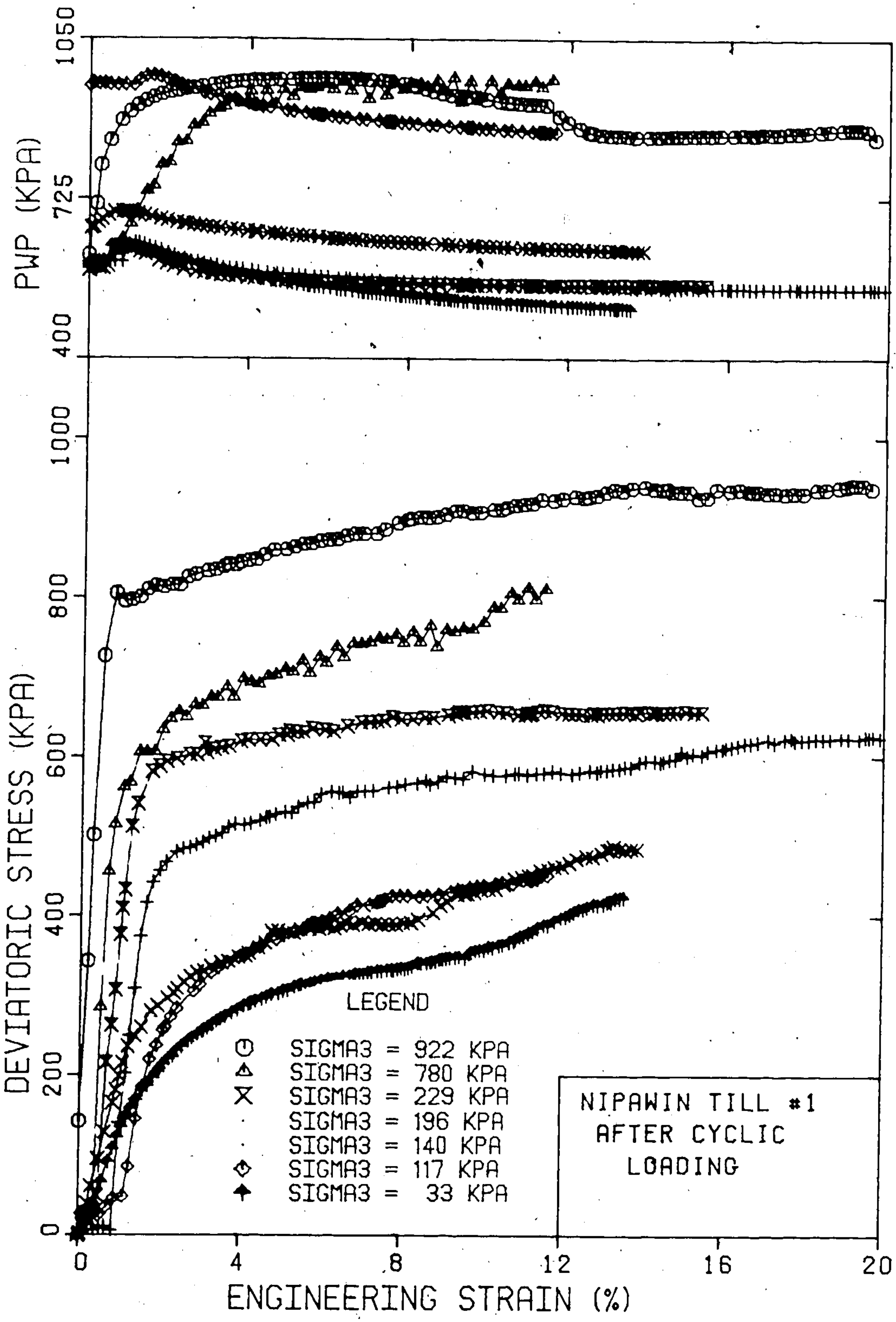

Figure 5.41 Stress-strain curves for Nipawin Till #1 after cyclic loading

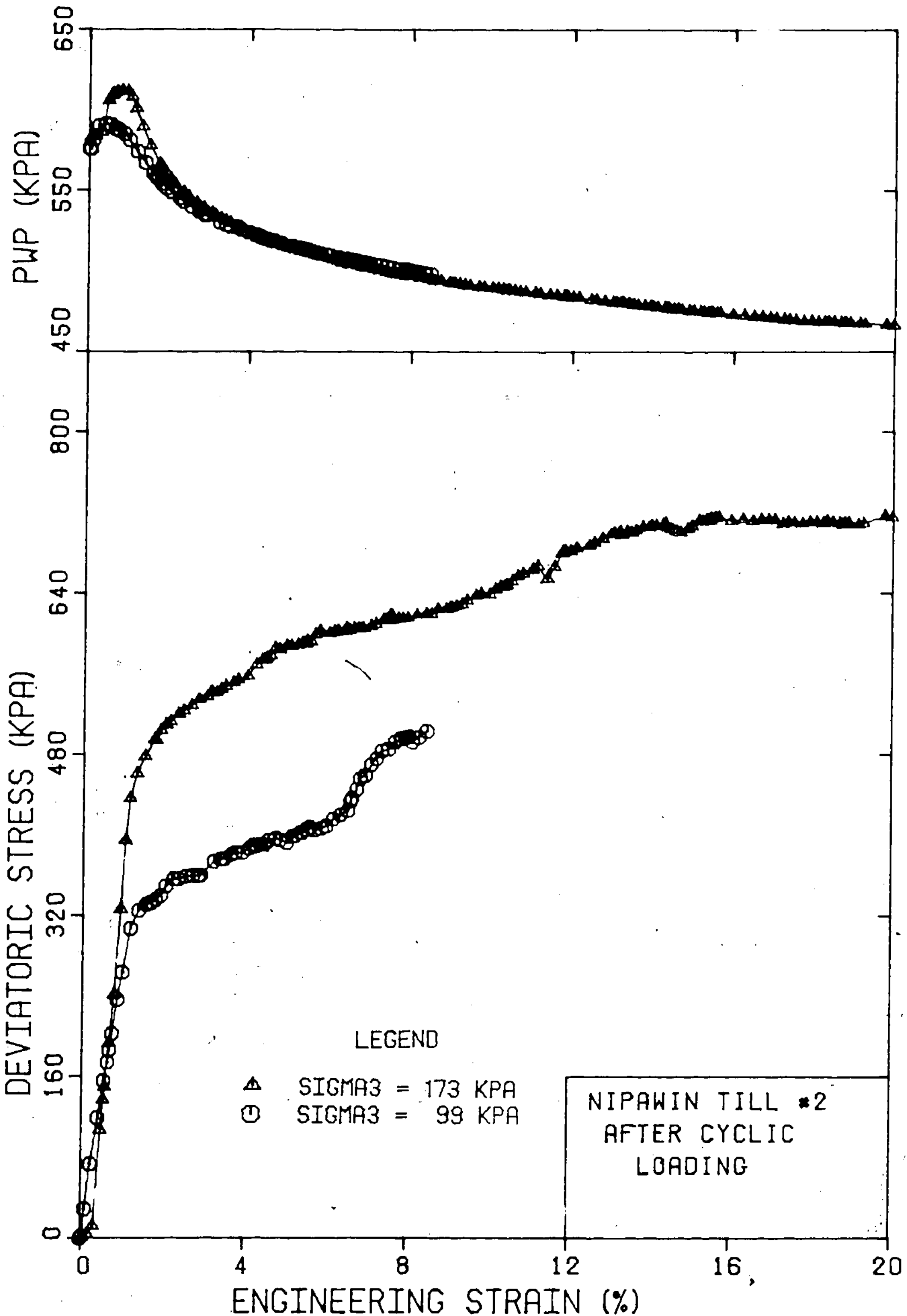

Figure 5.42 Stress-strain curves for Nipawin Till #2 after cyclic loading

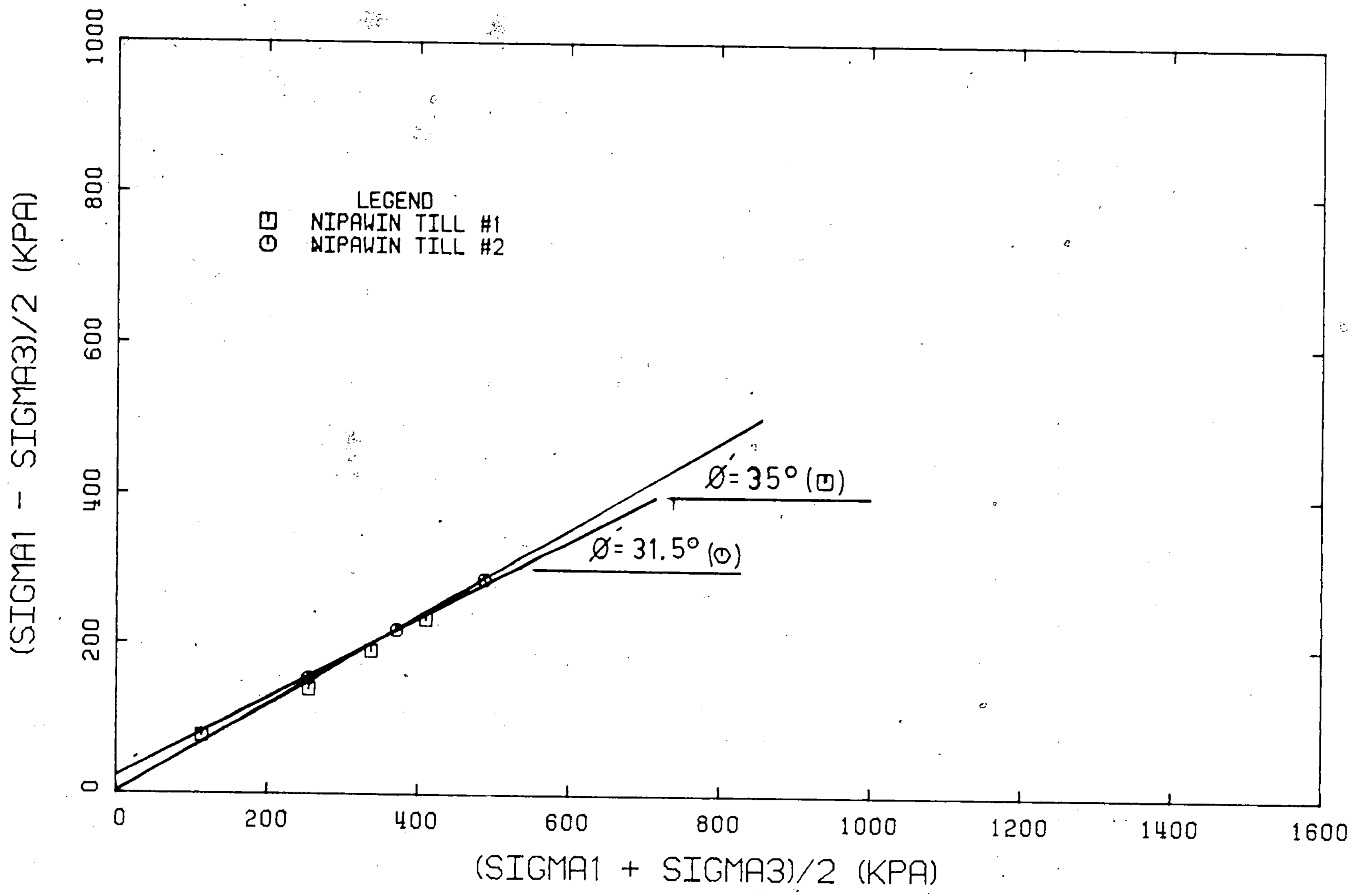

Figure 5.43 Effective stress at failure after cyclic loading

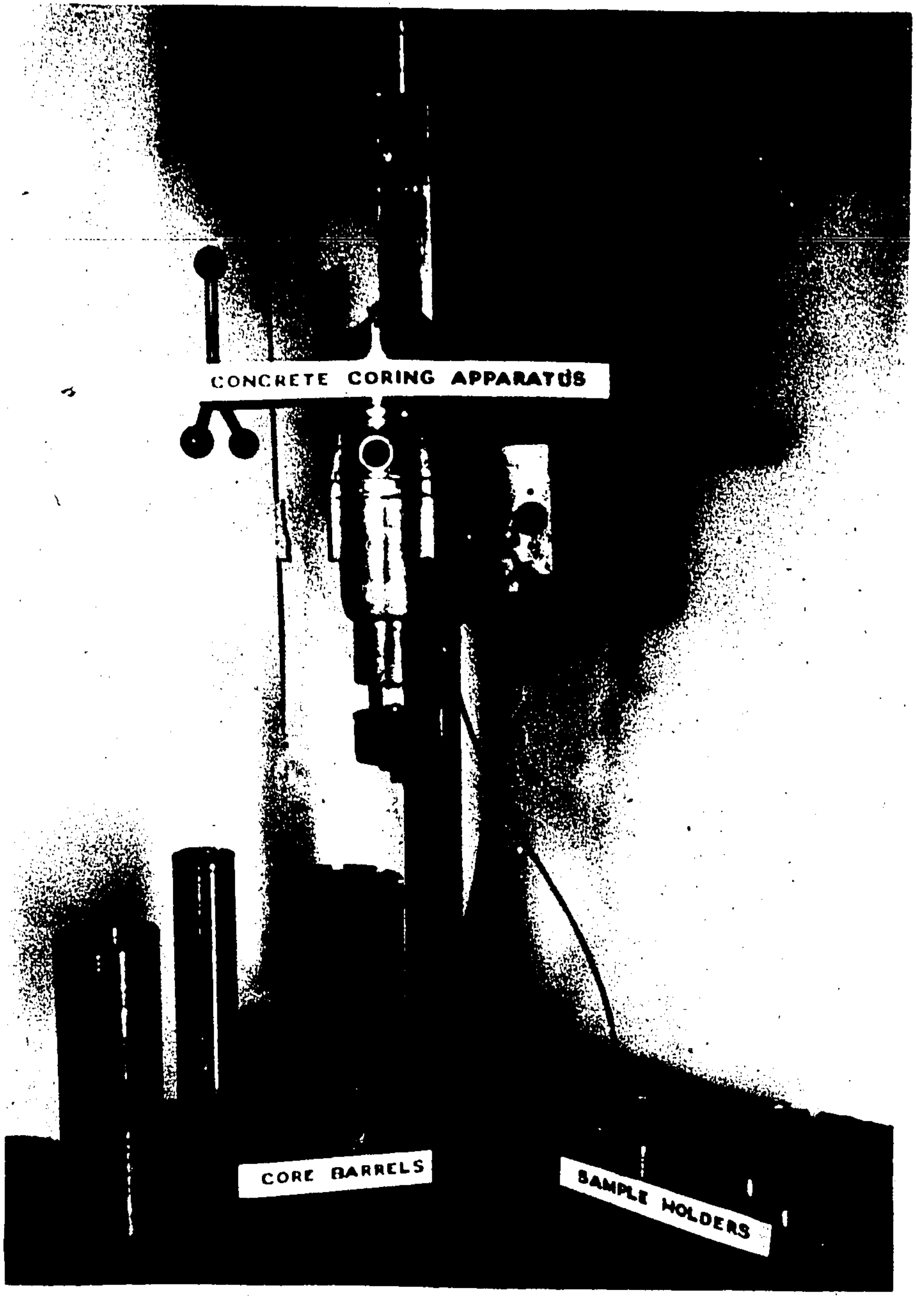

Plate 5.1 Concrete Coring Equipment

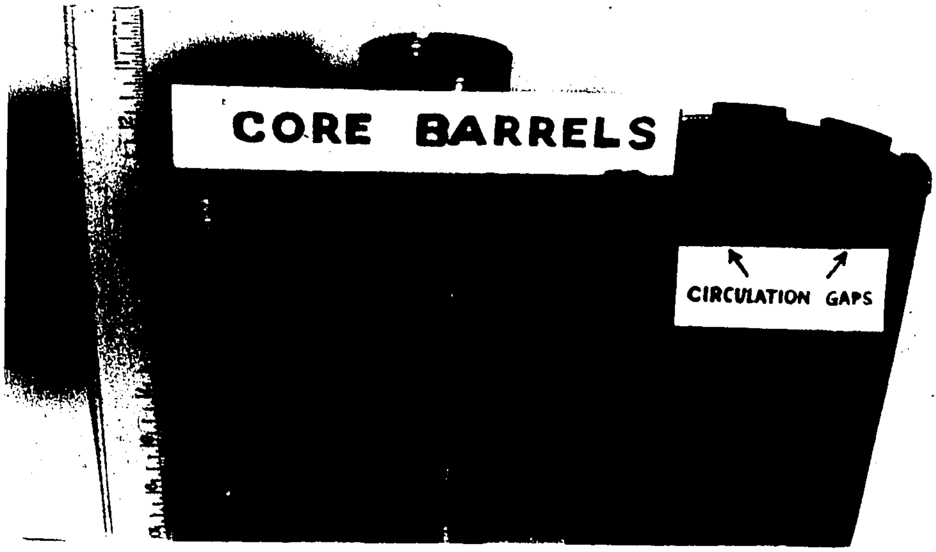

Plate 5.2 Typical Core Barrels

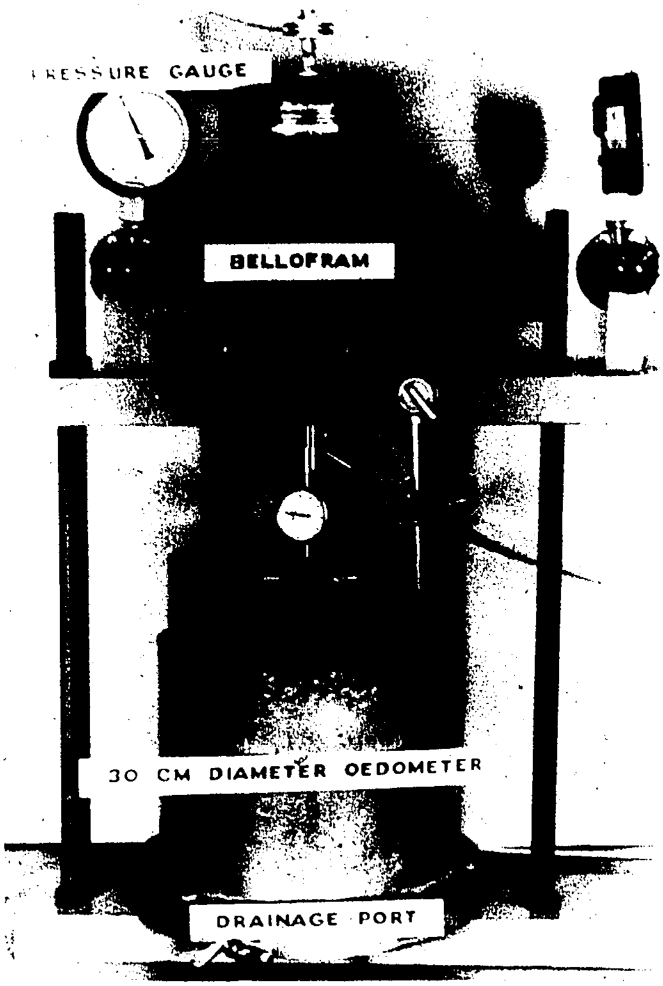

Plate 5.3 Experimental Setup for Large Oedometer

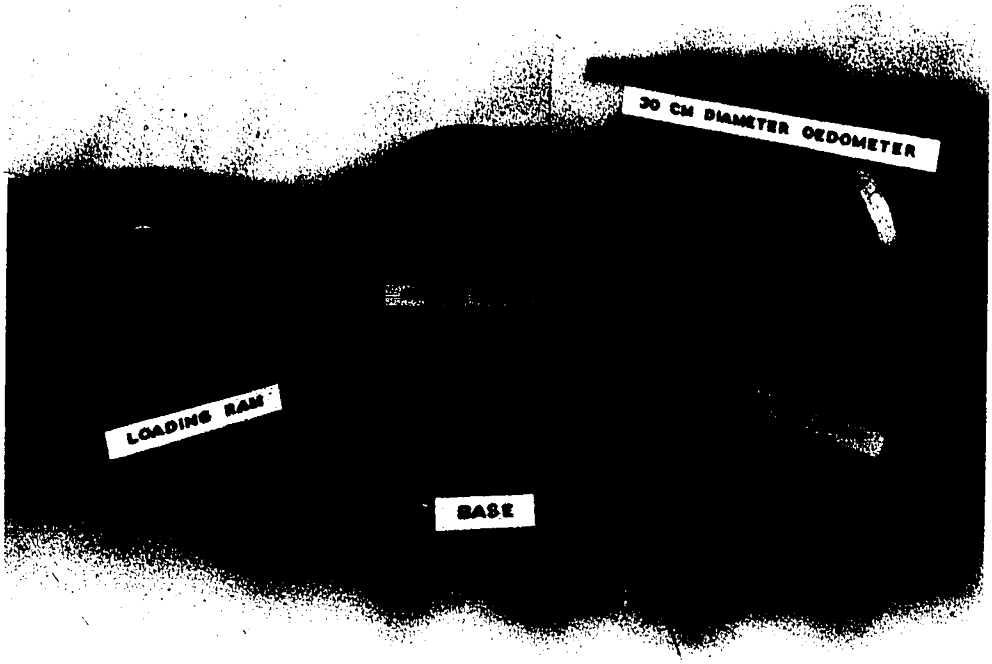

Plate 5.4 Details of 30 cm Oedometer

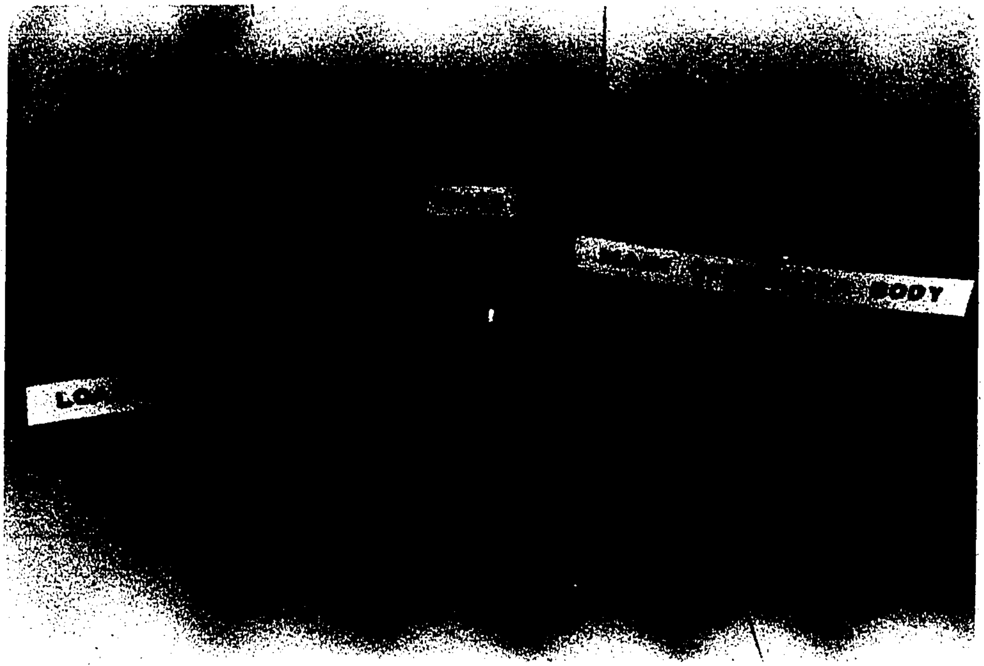

Plate 5.5 Details of 15 cm Oedometer

Plate 5.6 Bottom Platen of Triaxial Cell with Free Ends

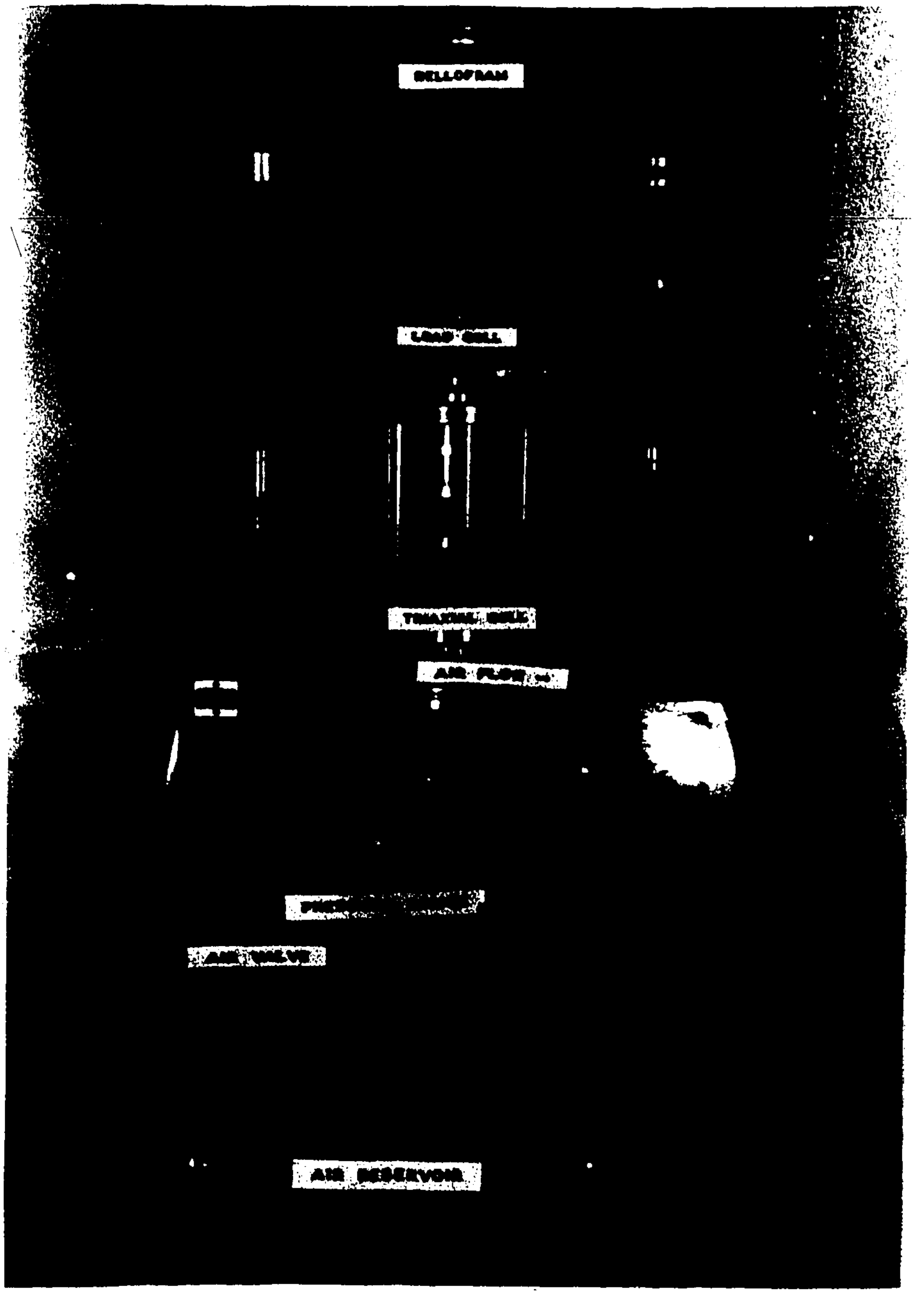

Plate 5.7 Cyclic Loading Apparatus

6. DISCUSSION AND CONCLUSIONS

6.1 Introduction

The effect of fabric upon the engineering behavior of glacial tills has been examined. Scanning electron microscope and laboratory studies were conducted on intact and reconstituted materials to evaluate this influence.

The research has resulted in a better understanding of types of fabrics found in glacial tills and their influence upon engineering properties. The following sections summarize the major findings.

6.2 Scanning Electron Microscope Work

The intact fabrics of the Nipawin and Edmonton Tills are clearly best represented by the samples reconstituted from a slurry. Consolidation from a slurry yields a material which displays a very open structure not unlike the collapse structures found in glacial ablation tills.

Observations of the bond material surrounding the sand clasts and coarse silts of both the intact and reconstituted tills indicated that the bond material is more compact in the intact samples.

It is probable that the reconstituted samples were not held at the required preconsolidation pressure for a sufficient period of time to allow a similar aging of the bond material to develop. Considering the clayey silt nature of the bonding material it would seem likely that secondary consolidation is required before the bonds become fully developed as observed in the intact material.

6.3 Laboratory Program

6.3.1 The Use of Free Ends

During the laboratory testing program the following observations were made:

1. Visual inspection of the samples indicated that the ends expanded and "barrelling" was reduced.
2. Uniform results were obtained and the samples appeared to be more stable.
3. Strain distribution and pore water pressure response were satisfactory even during cyclic loading

These observations indicate that the stress distribution within the samples was more uniform with free ends when compared to fixed end testing.

During the testing program, intact Nipawin Till strength results were compared with results obtained during a previous study using conventional fixed ends (Sego, 1980). No difference was found in peak strength values, however the highly overconsolidated samples exhibited a less brittle behavior when free ends were used.

It is widely recognized that when testing brittle dense samples of clay or sand, free end tests result in a general plastic failure instead of failure along a single slip zone imposed by the boundary condition of the conventional test.

The formation of multiple slip zones increases the likelihood of measuring a representative pore pressure and strength for the material.

6.3.2 Influence of Fabric on Engineering Properties

The scanning electron microscope observations and grain size distributions have shown that the Middle Nipawin and Upper Edmonton tills are matrix dominated. The engineering behavior of these materials is, depending on the confining stress, controlled by the clayey silt matrix or the soil ped framework.

The sand and large silt grains contribute to the strength of the material by acting as nuclei in the formation of a structural framework composed of highly consolidated soil peds. During consolidation the arching effect of these soil peds results in the formation of pockets of less compressed matrix material.

When a load is applied under low confining pressures, the highly consolidated peds act as individual particles and dilation occurs. The degree of dilation that occurs is dependent on the degree to which the soil peds have developed during both primary and secondary consolidation. At high confining stresses, the peds are not allowed to dilate and the strength of the soil is controlled by the clayey silt matrix.

The contribution of the soil ped framework to the strength of the tills tested is observed in Figure 5.12.

Under low confining stresses the ped framework dilates and angles of shearing resistance greater than 40° were measured. At greater confining stresses the strength is increasingly controlled by the matrix material, and angles of shearing resistance decrease to between 33° and 35°. Considering that the clayey silt is composed of angular silt particles and that rock flour is a component of the clay fraction, these angles are considered reasonable.

6.3.3 Behavior under Cyclic Loading

The characteristic behavior observed was a general tendency for pore pressure increase which caused a migration of the stress path towards the failure envelope. Curiously when the samples reached the failure envelope, increasing cyclic strains usually did not occur, instead the sample reached a state of equilibrium with little change in porewater pressure or in strain. Steady state behavior has been observed when the applied deviatoric stresses were below some critical value. Due to the limited number of tests performed, the critical value was not established.

The structural difference between Nipawin Till #1 and #2 samples is indicated by the test results. The differences can best be seen with normally consolidated materials since the material behavior is largely controlled by the matrix material. Large cyclic strains were recorded for normally consolidated intact Nipawin Till and Nipawin Till #2 samples. Normally consolidated Nipawin Till #1 samples however leveled off at considerably lower strains. This behavior is supported by the fabric observations reported in Section 4.2. Both the intact Nipawin Till and Nipawin Till #2 samples have an open fabric whereas Nipawin Till #1 has a closed, less compressed fabric.

6.4 Conclusions

The microstructure of matrix dominated tills consists of two components; the matrix and soil peds. Under low confining stress, the soil peds dilate and the matrix has little influence upon the engineering behavior of the material. At higher confining pressures the dilation of the soil peds is supressed and the matrix increasingly influences the behavior. Secondary consolidation is an important factor in the development of soil peds within the soil.

When examining glacial tills with a Scanning Electron Microscope the two fabric units should be examined in order to judge the strength of the till over a wide range of stress conditions. The soil ped framework can best be observed in a mosaic of detailed micrographs covering a large portion of the sample. The matrix fabric can be studied using individual micrographs. Micrographs can also give an indication of depositional processes which may be useful when correlating till deposits.

The data shows that the confining stress subjected to a till is an important consideration during design. Due to the ped framework, the angle of frictional resistance is highly dependent on the effective confining pressure or the normal stress on the failure plane. When heavier loads are anticipated, the decrease in strength should be considered.

Reconstituted samples are useful in studying the engineering behavior of different soil microfabrics and can be used to indicate the strength parameters of intact materials. They should not be used as a substitute for intact materials since depositional features such as fissures, shears, and micro faults are not present.

6.5 Recommendations for Further Study

A total fabric study of a glacial deposit should be undertaken. The importance of various micro and macro features throughout a deposit should be noted. Comparisons between laboratory, in-situ, and electron microscope data would provide a basis for interpreting the results of standard strength tests in terms of the large scale behavior relevant to foundation design.

The effect of post-depositional processes, such as freeze-thaw and dessication, on the fabric of a soil should also be studied. Marsland (1982) has suggested that micro shears are the result of freeze-thaw processes. Micro discontinuities may provide inherent sources of weakness along which larger discontinuities can propagate. These processes should be studied to establish their influence on the strength and deformation behavior.

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APPENDIX A

LABORATORY PROCEDURES

A.1 Shear Testing

The shearing of the sample was conducted using the following steps:

1. The loading ram was allowed to come in contact with the load cell attached to the reaction frame. This allowed zero deviatoric load to be recorded on the data acquisition recorder.
2. The movable loading platen of the testing machine was now adjusted to bring the sample in contact with the loading ram.
3. A small seating load was applied to ensure contact. The LVDT (linear variable differential transformer) was adjusted to its zero position for the experiment.
4. The displacement rate determined from the consolidation test for this sample was set on the machine.
5. The zero parameter pressure, LVDT, and load cell readings were recorded.
6. The machine was started, thus applying a constant rate of deformation to the sample.
7. At appropriate time intervals the load cell, LVDT and porewater pressure of the sample were recorded.
8. The loading of the sample was continued until the sample had undergone a large strain, i.e., greater than 8% or the more usual case until the resistance of the sample had begun to decrease.
9. At this point the loading was terminated.
10. The sample was disconnected from the pressure systems

and the volume change device.

11. The cell was dismantled, and the sample removed. The failed shape and dimensions were determined.
12. The total specimen was now used to evaluate the final moisture content of the sample.

A.2 Experimental Errors and Their Influence on Test Results

A.2.1 The Effect of Piston Friction

A small force is required to overcome the friction between the loading piston and a rubber Quad-ring seal at the top of the triaxial cell. The friction was so minute that the loading ram would slide through the ring seal under self weight.

A.2.2 The Effect of Filter Paper Drainage Strips

No direct experimental evidence is available on corrections to measured strengths for the effect of continuous slotted filter paper strips covering the entire specimen. However, Bishop and Henkel (1962) investigated the effect of slotted Whatman's No. 54 filter paper strips covering half the surface of the specimen, Olson and Kiefer (1963) made similar investigations using Whatman's No. 1 and No. 50 filter paper. Since continuous slotted filter paper strips were used in this investigation, the results of Bishop and Henkel were utilized. Bishop and Henkel concluded that the measured compressive strengths were about 0.1

kg/cm^2 higher than the actual, for total axial strains larger than 2%. For this investigation the correction for continuous strips would be insignificantly small and therefore were not applied.

A.2.3 Effect of Rubber Membranes

Henkel and Gilbert (1952) investigated the effect of the rubber membrane on the measured strength of triaxial specimens of 1 1/2 inch diameter. Using the theory of elasticity, they developed a method for calculating the correction; the method assumed that the membrane was capable of taking compression and that the sample deformed uniformly. The correction, σr, to the measured strength was given by:

$$\sigma r = \sim dM\epsilon_1/Acorr \qquad (A.1)$$

where:

d = initial diameter of the specimen

M = compression modulus of the rubber membrane, per unit width

ϵ_1 = axial strain

A corr = corrected area of the specimen at axial strain ϵ_1.

Subsequently, Duncan (1965) derived an expression for the correction, which took into account changes in area of the membrane. In either case, the corrections were quite small in this investigation.

A.3 Equations for Cross-sectional Area of Specimen

The cross-sectional area of the specimen at the end of consolidation was computed from the equation:

$$Ac = (Vc)L / Hc = (Vt - \Delta V)L / Ht-\Delta H \qquad (A.2)$$

where A, V, and H refer to the area, volume and height of the specimen and the subscripts specify either conditions after trimming (t) or at the end of consolidation (c). The change in volume (ΔV) and change in height (ΔH) were measured during isotropic consolidation in the triaxial cell.

In some tests, where the measurement of ΔH proved to be inaccurate, the cross-sectional area was calculated in a different manner. Equation (A.2) can also be written as:

$$Ac = Vt(1-\Delta V/Vt) / Ht\ T\ (1-\Delta H/Ht) \qquad (A.3)$$

therefore

$$Ac = At(1-Ev)/(1-E_1) \qquad (A.4)$$

where Ev and E_1 are the volumetric strain and axial strain at the end of consolidation. For an isotropic soil under a hydrostatic stress system, E_1 is equal to Ev/3. Substitution in equation (A.4) gives

$$Ac = At(1-3E_1)/(1-E_1) \qquad (A.5)$$

Although the relationship used here between the axial strain and volumetric strain is a useful concept, Lambe (1964) has shown that the relationship is only an approximation since the stress history, soil structure, and other factors can affect significantly any theoretical relationship between axial strain and volumetric strain.

The cross-sectional area of a specimen at a given strain was computed on the basis that the specimen deformed uniformly with no change in volume (Bishop and Henkel, 1962):