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Landslides

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THE UNIVERSITY OF ALBERTA

RESIDUAL STRENGTH ANALYSIS OF FIVE LANDSLIDES

by

(C) DOUGLAS GORDON PENNELL

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR

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UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance a thesis entitled "RESIDUAL STRENGTH ANALYSIS OF FIVE LANDSLIDES", submitted by DOUGLAS GORDON PENNELL in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

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ABSTRACT

The purpose of the research was to re-evaluate five landslides which occurred in Alberta and British Columbia. The soil types studies included bedrock Cretaceous clay shales and Pleistocene deposited preconsolidated clays.

A laboratory testing program involved the determination of the residual strength parameters by reversing direct shear for undisturbed, remolded and pre-cut samples. The results indicated that the residual cohesion and the residual angle of internal friction were higher for the undisturbed samples than for the remolded or pre-cut samples. The higher undisturbed parameters were primarily attributed to the failure plane irregularities which developed in the undisturbed case.

The irregularities which developed during the shearing of an undisturbed sample were postulated to change the mode of failure from sliding across irregularities to shearing off irregularities. This phenomenon resulted in a decrease in the angle of internal friction at high normal stresses.

It was found that the residual angle of internal friction for natural stiff clays may be related with equal reliability to clay fraction, percent montmorillonite of total sample, activity or liquid limit. In addition, an increased density resulted in an increased residual angle of friction.

The stability analyses of this study were performed by the method of Morgenstern and Price (1965) employing a computer. The analyses of the slides indicated that residual strength parameters

must be utilized in order to obtain a factor of safety of one for limiting equilibrium. Peak strength parameters gave factors of safety much above one.

Wedge-shaped slides in bentonitic soils occurred with the peak strength developed along the scarp and the residual strength along the horizontal failure plane. The ultimate slope angle approached the residual angle of internal friction but was dependent upon the piezometric level and overburden above the slide material.

Slope reduction after failure may not be entirely related to a strength decrease. Caution must be exercised when analyzing post-slide slopes in which re-adjustments of the slope have occurred because of a change in the mode of failure.

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The Soil Science Department of the University of Alberta for the physico-chemical tests.

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GLOSSARY OF TERMS AND SYMBOLS

Symbols

W_L	Liquid limit
W_P	Plastic limit
I_P	Plasticity index
A	Activity
W_N	Natural moisture content
e_n	In situ void ratio
I	Liquidity index
γ_t	Total wet density
G_S	Specific gravity
ϕ'_{CU}	Peak effective angle of friction from consolidated undrained triaxial tests
C'_{CU}	Peak effective cohesion from consolidated undrained triaxial tests
ϕ'_P	Peak effective angle of internal friction
C'_P	Peak effective cohesion
ϕ'_r	Effective residual angle of internal friction
C'_r	Effective residual cohesion

Terms

ASTM	American Society for Testing Materials
LVDT	Linear Variable Differential Transducer

GLOSSARY OF TERMS AND SYMBOLS (continued)

Pre-till	Soils deposited below glacial till and subjected to glacier overburden weights.
Post-till	Normally consolidated soils deposited above glacial till.
Preconsolidated	Description of soils which were subjected to weights greater than the present overburden. It is used synonymously with overconsolidated.

CHAPTER I

INTRODUCTION

1.1 PURPOSE OF THE RESEARCH

The research which comprises this thesis is based on landslides which have occurred in the Edmonton area, in other areas of Northern Alberta and in North-eastern British Columbia. The troublesome slide materials include highly consolidated clays and clay shales. Previous analyses by Hardy (1957), Brooker (1958), Hardy *et al*. (1962) and Hardy (1963) using peak effective stresses and field pore pressures yielded factors of safety much in excess of unity. Analyses based on the conventional peak effective strengths were related to piezometric heads and indicated that heads much above ground surface were required to initiate failure. Various other mechanisms or phenomena which have been used to explain the above anomaly include the swelling pressure hypothesis by Hardy *et al*. (1962) and the "zero-cohesion" analysis by Henkel and Skempton (1955). The swelling pressure concept has undergone considerable criticism and in many slide materials would be found inapplicable as high swelling pressures are not indicated. The "zero-cohesion" analysis has generally been found to indicate factors of safety greater than 1.0 because the peak angle of friction is still retained.

This re-evaluation of the numerous documented landslides in the Alberta area was initiated by the most promising contributions by Skempton (1964) and Bjerrum (1966) on residual strength and progressive failure. Skempton introduced the residual factor, in order to quantitatively express the shear strength developed in a slide. Haefeli (1950) also postulated the application of residual strength to landslides and introduced an expression called the residual ratio, similar to Skempton's residual factor. Hvorslev (1960) advocated the applicability of residual strength to soils subjected to large strains. These concepts will be related to the soils and landslides investigated.

Sinclair et al. (1966) have considered some of the slides with regard to residual strength.

1.2 SCOPE OF THE STUDY

The scope of this thesis involved collecting and re-evaluating all the data which was available on the landslides. The geology of each area was examined and related to the slide development. The previous data was augmented by additional field piezometer installations and undisturbed samples were obtained for purposes of obtaining the peak and residual strengths. An extensive laboratory testing program was undertaken with emphasis on obtaining the residual strength from undisturbed, remolded and pre-cut specimens.

In order to perform numerous rapid stability calculations, the method of Morgenstern and Price (1965) was used, which treats surfaces of any shape and satisfies the statically indeterminate condition which exists in most limit analysis. A computer program written by Morgenstern and Price was converted to the Fortran IV programming language of the University of Alberta IBM 360 computer.

All soils were classified by standard geotechnical tests including liquid limit, plastic limit, grain size analysis and specific gravity. Consolidation tests were also performed to determine swelling characteristics, permeability and preconsolidation pressures. Physico-chemical properties were indicated by cation exchange capacity and salt content. Thin sections were prepared for geological studies to disclose microstructure and macrostructure features of the materials before and after shear.

A summary of the specific objects of this thesis includes:

(a) Evaluation of the residual factor and applicability of residual strength to several landslides.

(b) Comparison of the residual strength parameters obtained from undisturbed, remolded and pre-cut specimens.

(c) Generally, re-evaluation of slope stability of stiff clays in the study area.

1.3 ORGANIZATION OF THE THESIS

Chapter II of this thesis is a general summary of the latest concepts in the shear strength and analysis of stiff clays.

Chapter III includes a summary of the laboratory data obtained in this study along with other documentated studies in the area.

Chapter IV is a discussion of the laboratory results. A complete chapter is devoted to the discussion because a combined rather than separate treatment of the results is indicated for the soils.

Chapter V and VI include the detailed description of the slide areas. Chapter V comprises the slides which exist along the Peace River and Chapter VI is a description of those along the North Saskatchewan River in the Edmonton Formation bedrock. These two chapters include a detailed review of all the laboratory tests, field investigations and stability analyses performed previous to the present study. A description of the applicable geology for each slide is given. Two chapters are employed because the two areas require separate general geology descriptions. Five slide areas are investigated in detail with regard to applying residual strength concepts. Chapter V includes slides at Taylor, British Columbia, Dunvegan, Alberta and Peace River, Alberta. Slides of Chapter VI occurred along the North Saskatchewan River and are referred to as the Grierson Hill slide in the City of Edmonton and the Lesueur slide, 3 miles east of Edmonton. Two other slide areas referred to in this thesis are the slopes adjacent to the University of Alberta campus and the Little Smoky slide, located on the Little Smoky River, 80 miles south of the Town of Peace River. The location of the slides is shown in Figure I-1. Both Chapters V and VI incorporate the field investigation carried out in this study to supplement previous work and the computer stability analyses.

Chapter VII is a combined discussion of slope stability of the study area.

Chapter VIII outlines the conclusions of the study and recommendations are suggested for future research.

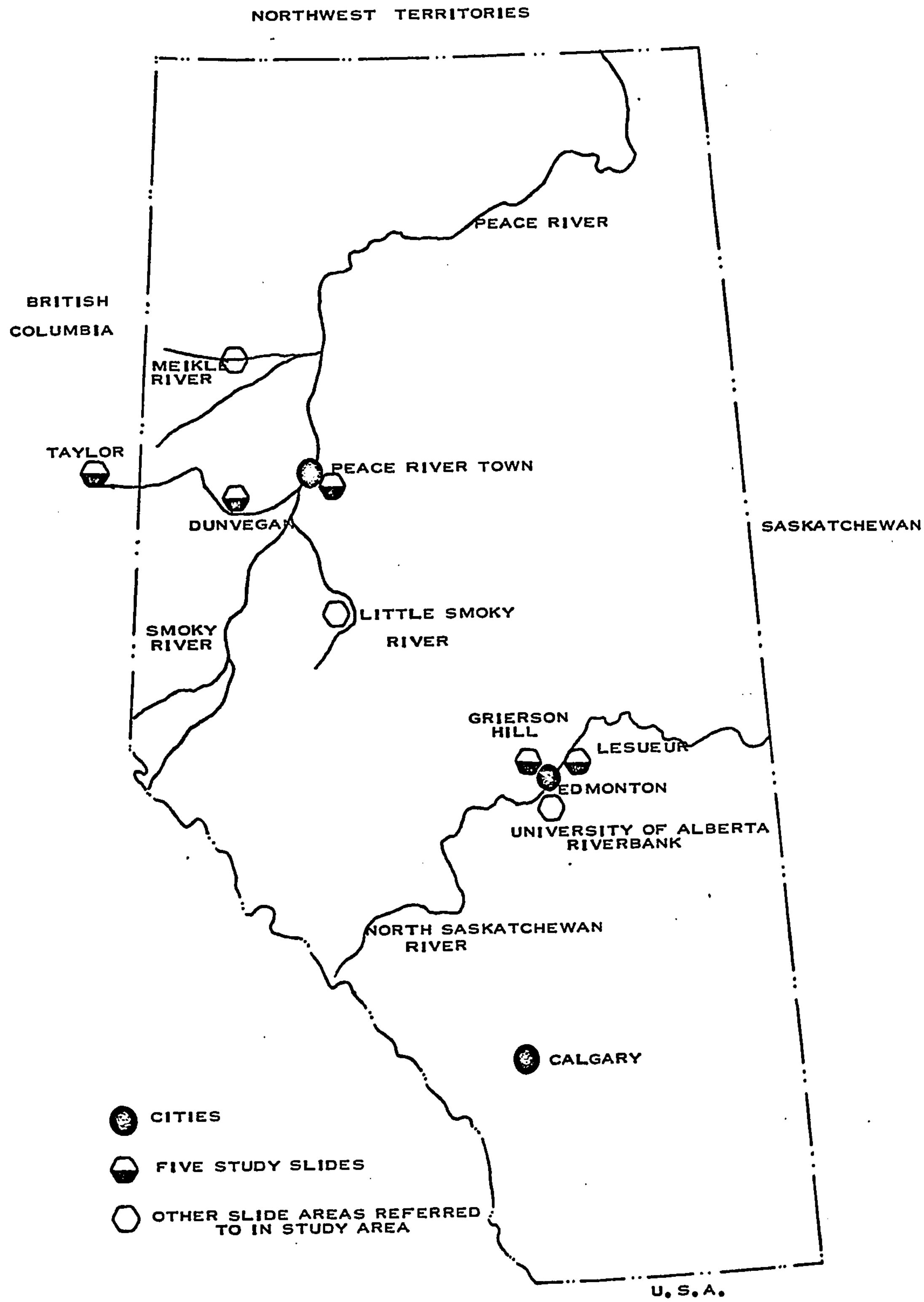


FIGURE I-1 LOCATION OF SLIDES IN STUDY AREA

CHAPTER II

LATEST DEVELOPMENTS IN SHEAR STRENGTH AND ANALYSIS OF STIFF CLAYS

2.1 NATURE AND FORMATION OF SEDIMENTARY DEPOSITS

(a) Introduction

The soils which were studied most extensively in this thesis were formed by sedimentary processes. Consequently, a resumé of the fundamental processes which produce a stiff clay is considered to be required. Generally, the shear strength of natural stiff clays reflects in many aspects the geologic history of the particular deposit. Soil mechanics literature contains few examples relating the geotechnical properties of a stiff clay to its geological history. Only a general approach to this very important phase of stiff clays will be attempted. It is felt that a closer association between geotechnical and geological phenomena is required in order to evolve a clearer comprehension of the shear strength of stiff clays.

(b) Classification of Argillaceous Sediments

The main interest in this study are the argillaceous sediments produced by settling in water. Geological classifications of sediments are based on environment of deposition or mineralogical content (Pettijohn, 1957). Neither of these classification systems have been suitable because the engineering properties of the soils have not been included.

In soil mechanics, argillaceous soils are commonly subdivided into broad groups including soft clays and stiff clays. Stiff clays occur in both the fissured and intact state, although the fissured is most common. The boundary of each group is not definite, but each group exhibits fundamentally different engineering properties. The groups generally indicate the degree of consolidation the clay has undergone.

Terzaghi (1936) arbitarily selected a liquidity index of 0.5 as the division between stiff and soft clays. Normally loaded clays may possess a liquidity index as low as 0.5 but most clays of liquidity indices of 0.5 or less are highly overconsolidated.

The differentiation between a stiff clay and a shale in the engineering usage is nebulous. A shale is normally considered to possess fissility. The term "clay shale" is often used to describe shales which possess fissility but behave similarly to stiff clays when subjected to weathering and when water is made available. Geological classifications of shales are based on cementation and consolidation, grain size, chemical and mineralogical composition and fissility. A classification by Mead (1936) divides shales into two broad groups including:

(a) Compaction or "soil-like" shales, which have been consolidated by the weight of the overlying sediments and lack intergranular cement.

(b) Cemented or "rock-like" shales, in which the cementing is due to the physical bonding agents or the recrystallization of its clay minerals.

The compaction shales disintegrate rapidly under cyclic wetting and drying and behave like overconsolidated stiff clays. The compaction shales would commonly be referred to as clay shales. Sometimes, geologists differentiate between shales which possess fissility and high induration and claystones which do not exhibit fissility. Terms such as "claystone" and "siltstone" are used to indicate high percentages of clay and silt, respectively.

(c) Depositional and Diagenetic Processes

Chemical and physical changes begin to take place in many sediments immediately after deposition. These changes will ultimately be reflected in the geotechnical properties of the material. In the majority of cases, these early changes start to operate at the temperature of deposition and without any significant rise of pressure as a result of loading. Early change occurs as water is expelled without any significant rise of pressure. As the weight of the overlying sediment increases, the particles of the buried deposits are brought into closer contact and the unstable constituents are broken down and the pore water is expelled as the void ratio decreases. These processes lead to the primary lithification (Pettijohn, 1957) of the deposits.

Changes that occur within sedimentary accumulations are known collectively as diagenesis and ultimately result in rock lithification. The term diagenesis usually refers to all the physical and chemical modifications that take place either after deposition or while deposition is still in progress. The various diagenetic changes include the following main processes as indicated by Pettijohn (1957):

(i) Mechanical compaction - geologically, infers a consolidation process in which increased density results as mechanically trapped pore water is expelled.

(ii) Cementation - cementing of a particle to another by such a compound as calcite.

(iii) Recrystallization- recombination to form new compounds or changes in the texture.

(iv) Chemical changes - including such processes as dehydration and oxidation and reduction.

The processes of diagenesis tend toward the lithification of the rocks involved. In the elucidation of the engineering properties of a clay the past diagenetic processes which acted on the clay will be of paramount importance. The effects of weathering will be dependent on whether the clay is of a compacted or cemented nature.

A rock of particular interest and unique character is bentonite. Bentonite is a product of the devitrification of pyroclastic rocks such as pumice. Unweathered material is light green or pale greenish-yellow in colour. It swells to many times its original volume before breaking down to a soapy paste. Bentonite consists essentially of montmorillonite or closely allied clay minerals giving a very large total surface area. Many slides in Western Canada have been attributed to materials with high bentonite contents.

2.2 SHEAR STRENGTH OF STIFF CLAYS

(a) Effect of Structure on Strength

The shear strength of a laboratory sample is only indicative of the field strength if the material and structure are similar for both cases. A description of a clays' structure, includes both its macrostructure and microstructure. Macrostructure features are those which may be observed by the naked eye as suggested by Morgenstern and Tchalenko (1967), and commonly are fissures and slickensides. Most stiff clays possess a macrostructure and consequently, an investigation of a stiff clay usually infers a stiff fissured clay. Often high laboratory strength results may be attributed to a sample size smaller than the macrostructure field fissure spacing, that is, the sample may consist of intact soil which does not exist on a large scale in the field. Various causes of fissuring in clays are postulated to include desiccation, tectonic movements, overstressing and synaeresis.

Some microstructure features of a clay can be investigated with a petrographic microscope. The components of microstructure have been termed by Morgenstern and Tchalenko (1967) as original fabric and shear-induced fabric. Original fabric is related to depositional history and may be random or preferred. If the soil has been subjected to post-depositional shear strains a shear-induced fabric is often present.

The importance of macrostructure or microstructure fissures as weakening agents is well recognized. Although methods for describing fissure intensity are not well defined, the scatter of laboratory test results should perhaps be related to the variations in the structure of the individual test samples. Normally, the scatter of laboratory results is only related to changes in the composition.

(b) Peak and Residual Strength

The peak strength of a clay is the most commonly obtained strength from laboratory direct shear or triaxial tests. The historical success of applying peak strength parameters to slides in stiff clays has been unsatisfactory. Generally, the peak strength has been shown to be much greater than the strength obtained from analysis of landslides.

The peak strength obtained in the laboratory may be under either drained or undrained conditions. Under undrained conditions the strength of a preloaded clay may be either smaller or larger than the drained strength, depending upon the value of the overconsolidation ratio. If the overconsolidation ratio lies in the range between 1.0 and about 4.0 to 8.0 (Terzaghi and Peck, 1967) the volume of the clay tends to decrease during shear and the undrained strength is less than the drained strength. For values of overconsolidation ratios greater than about 4.0 to 8.0 the volume tends to increase, the pore pressure decreases, and the undrained strength exceeds the drained value. For high overconsolidation ratios, negative pore pressures develop which tend to draw the water into the soil and reduce the strength. Consequently, the application of the undrained strength to overconsolidated clays leads to results on the unsafe side. The discrepancy between undrained laboratory results and field results increases as the liquidity index of the soil decreases (Peck and Lowe, 1960).

The peak effective strength represents the shear strength of a stiff clay better than the undrained strength based upon the total stresses. The determination of the peak strength of a fissured undisturbed clay may be misleading because it is well known that fissured clays lose strength with time. The peak strength normally is most meaningful when obtained from an intact sample.

Even though the peak effective strength parameters are considered to represent the shear strength of a stiff clay, it has been found that the average shear strength developed in a slide is much below the peak.

The recent concept of residual strength by Skempton (1964) appears to be the basis for the field behaviour of stiff fissured clays. The residual strength or ultimate strength is that strength which a soil possesses at large strains. Skempton (1964) found that the average shear strength developed along a failure surface may vary from the peak to the residual strength. He introduced the residual factor to be a convenient quantitative expression which indicates the amount the strength has fallen from the peak to the residual. The residual factor is defined by the expression:

$$R = \frac{S_f - \bar{S}}{S_f - S_r}$$

S_f = peak strength

S_r = residual strength

$\bar{S}$ = average shear strength acting on the failure plane

A residual factor of 1.0 indicates that the average shear strength acting on the failure surface is the residual strength.

The magnitude of the residual strength of a clay is necessary for rational design when dealing with material subject to previous shear movements.

The residual strength is a parameter of fundamental significance, being independent of stress history and factors which dominate the path dependent properties of soil. It is believed that a unique structure probably occurs at the residual state (Kenney,1967). Kenney has shown that the residual strength of some clay shales is dependent on mineral composition and system chemistry.

(c) Effect of Time on Strength

It has been shown by Skempton (1964) that the decrease in shear strength of London clay and some other preconsolidated soils is dependent upon time. Fissuring, weathering, creep and progressive failure all become important factors in the discussion of time effects.

Terzaghi (1936) observed that the presence of a fissured structure provides a mechanism for progressive failure by softening and swelling. A fissured structure is believed to develop in soils undergoing unloading in which there is a variation of vertical and horizontal pressures. Progressive failure appears to be a feature of slopes in stiff clays and is probably restricted to unloading conditions. Both Cassel (1948) and Skempton (1948) also postulated that strength decrease is due to softening in fissures.

Skempton (1964) has analyzed slides in which various residual factors have been indicated for London clay. He suggests a rate of increase of the residual factor with time (Morgenstern, 1967) based on slides over a 50 year period. The increase of the residual factor with time will depend, of course, on the material and its environment. The rate at which the residual factor increases is of great importance when cuts are made in stiff clays. Designing a slope on the basis of the residual strength may prove prohibitive and therefore, time-related strength data would be very important. If hundreds of years are required for the attainment of the residual strength, the design for long term conditions based on residual strength would not be warranted. Consequently, the designer of slopes is interested in a time versus residual factor for the soil involved. This relationship can only be obtained from well documented landslides of both natural and artifical slopes.

Bjerrum (1966) has postulated a mechanism of progressive failure in stiff clays which does not depend upon fissuring. His hypothesis relates the various factors of unloading, stored strain energy, weathering, residual strength and progressive failure. He substantiates qualitatively his concepts by citing numerous landslides. Bjerrum explains the initiation of a progressive failure by relief of high lateral stresses which produce stress concentrations at the toe of the slope causing the migration of a failure surface adjacent to the base of the slope. The following conditions in a preconsolidated clay or shale slope must be present in order to facilitate a progressive failure as postulated by Bjerrum:

(i) A discontinuity must occur such as a highway cut or a stream-formed valley.

(ii) The shear stresses which develop in the slope must be greater than the peak strength so that the residual condition can be achieved.

(iii) The soil must possess sufficient stored strain energy so that upon release the strain will be large enough to stress the clay past failure towards the residual strength. The available strain energy depends upon both diagenetic and weathering processes.

(iv) The soil must exhibit a large drop in strength from the peak to the residual strength.

Bjerrum considers weathering as an agent which liberates stored strain energy and destroys diagenetic bonds, thus allowing large strains to occur. He assumes that the progressive failure advances in a direction parallel to the slope near the bottom of the weathered zone. This assumption is based upon observations of several slides, although many slides have been found to occur on horizontal failure planes controlled by geological bedding.

The relationship of creep and progressive failure is of significance in a clay shale slope. Bjerrum (1966) suggests that "creep is a phenomenon related to the slow volume expansion accompanying disintegration of heavily overconsolidation clays and shales containing large amounts of stored strain energy". The most important contributing factor to creep is the continuous increase of lateral expansion and reduction in shear strength. During the process of toe unloading, a failure plane is developed along which occur creep movements. As deterioration of the clay shale occurs, a decrease in strength continues until creep movements expand into a catastropic slide. Consequently, creep movements can normally be found to precede a slide.

In summary, the working hypothesis of progressive failure in stiff clays by Bjerrum has set the stage for research in such problems as effects of diagenesis on shear strength and the rate of destruction of diagenetic bonds.

(d) Summary of Section 2.2

The treatment of shear strength of stiff overconsolidated soils is presently focused upon residual strength and progressive failure. The ultimate solution of slope stability in overconsolidated clays will be partially solved when laboratory test results may be used to forecast slope safety. Of course, the present dilemma of forecasting rates of progressive failure, weathering, stress release and strength reduction will require solution before a complete rational approach is achieved. The solution of the problem will presently depend upon our opportunities to observe and analyze field case histories of both natural and artifical slopes.

2.3 METHODS OF STABILITY ANALYSIS

The approaches to slope stability include the total stress or $\phi = 0$ analysis and the effective stress analysis. The total stress analysis for stiff clays has been shown to give erroneous results when considering long-term cases. Therefore, the shear resistance developed on a potential failure surface is a function of the normal effective compressive stress acting on the surface. The determination of the actual normal stresses which may exist is an indeterminate problem. A number of methods of calculation are in common use, each based on somewhat different simplifying assumptions.

Because the failure surfaces for the slides investigated in this study were of non-circular shape, the circular-arc methods of analysis including the standard method of slices, the "modified" Swedish method (U.S.C.E., 1960) and Bishop's (1955) method were not applicable. Consequently, a method of analysis was selected which treated non-circular slip surfaces.

The wedge method of analysis (U.S.C.E., 1960) has been shown by Sinclair *et al*.(1966) to represent the shape of some failure surfaces in the study area. This method requires that the direction of the active and passive forces be assumed. Assumptions are based on the shape of the surface and the location of the active and passive blocks. The method becomes laborious when numerous trial surfaces are investigated and when several soil types must be considered gross approximations must be made.

Other methods of analysis have been developed by Janbu (1954) and Kenney (1956). These procedures are all based on method of slices, limiting equilibrium and general slip surfaces. None of these methods satisfy all equations of equilibrium and are therefore in error by an unknown amount (Morgenstern and Price, 1965).

Because the stresses in a slope are not known, it is necessary to utilize limit analyses and also to invoke an assumption in order to render the problem statically determinate. Morgenstern and Price (1965) developed a general slip surface analysis which satisfies all the conditions of equilibrium and is statically determinate. In order to satisfy the statically indeterminate condition, an assumption may be made relative to the distribution of the normal pressure, the position of the line of thrust or a third assumption which is used by Morgenstern and Price. They assume a relation between the thrust force E' and the vertical shear force X. It is assumed that

$$X = \lambda f(x) E'.$$

If f(x) is specified, the problem becomes determinate and λ, an overall force ratio scaling constant, and the factor of safety may be found from a solution of two governing differential equations which satisfy the appropriate boundary conditions. The function f(x) can take any prescribed form; however, soil limitations usually dictate a prescribed range for f(x). In most cases the factor of safety will be found to be relatively insensitive to f(x). Complete details of the analysis is given by Morgenstern and Price (1965) and the numerical techniques for solving the governing differential equations with the use of a computer are given by Morgenstern and Price (1967).

It was decided to utilize the method of Morgenstern and Price in this thesis because of the availability of a computer program, rapid analysis of many trials, and accuracy of the method. Some of the stability analyses performed by computer have been compared to the wedge method in Chapters V and VI.

CHAPTER III

PRESENTATION OF LABORATORY RESULTS

3.1 GENERAL

Standard geotechnical tests as well as miscellaneous tests have been performed on all slide materials. Although an insufficient number of soils were studied in order to derive conclusions for all parameters and phenomena, it is hoped that future slides and slide materials can be compared to those of this thesis. Consequently, all test results are included for the purpose of completeness and future reference.

Included in the summary tables are results for soils of the University of Alberta riverbank studied by the Department of Civil Engineering (1968) and for the Little Smoky landslide reported by Hayley (1968). These results are included because they relate to the general discussion of landslides in the study area.

The soils investigated are those which have been shown to be the primary slide materials. The four major soils tested include the Taylor clay shale, Dunvegan pre-till clay, Peace River pre-till clay, and Lesueur bentonitic clay shale. Two other soils, Peace River post-till clay and Lesueur brown clay shale have been included with the purpose of comparing the soil profile materials. Strength tests were performed on the Lesueur brown shale to disclose variations in the strength of the Edmonton Formation. Post-till Peace River clay was included to indicate variations between pre-till and post-till clays. "Post-till" is used to describe the soil existing above glacial till deposits and which has not been subjected to glacier weights.

A weathered sample of Shaftesbury clay shale from the Heart River valley at Peace River was obtained to compare its composition to the Shaftesbury at Taylor, B.C.

3.2 CLASSIFICATION OF SOILS

Classification tests were performed in accordance with the procedures detailed by A.S.T.M. (1964). All soils were air-dried previous to being crushed by a mechanical grinder. Results of all the classification tests were summarized in Table III-1. Included in Table III-1 are the results of liquid limit, plastic limit, grain size analysis, specific gravity, natural moisture content and bulk density tests. The natural moisture contents were obtained from the shear strength sample trimmings. The bulk density was calculated by using the sample shear box dimensions and initial wet weight. Other parameters summarized in Table III-1 include the plasticity index, activity, liquidity index and natural void ratio.

The mineralogical composition of the clay fraction of the cohesive soils was determined using X-ray diffraction methods. The results given in Table III-2 were obtained by the Geology Division of the Research Council of Alberta.

Analyses were performed to determine the cation exchange capacity and exchangeable cations. These results are summarized in Table III-3. The cation exchange capacity was determined by the author in the laboratory of the Soil Science Department of the University of Alberta. The percentage of exchangeable cations was determined by the Soil Science Department.

3.3 LABORATORY SHEAR STRENGTH RESULTS AND TEST PROCEDURE

(a) Direct Shear Test Procedure

The direct shear apparatus was employed to obtain peak and residual effective strength envelopes. In order to achieve the large displacements required to reduce the peak strength to the residual value, a reversing technique was employed.

A rectangular (3.25" by 2.00") direct double shear box described by Rennie (1966) was utilized in obtaining results for the Peace River soils. All remaining materials were tested in a standard automatic reversing (2" diameter) single direct shear box supplied by the Wykeham Farrance Co. of England.

The shear box was allowed to displace .12 inches in both the forward and the reverse directions, thus, providing a total displacement of .24 inches per cycle.

The shear load was measured by a calibrated load cell and both the vertical and the horizontal displacements by linear variable differential transducers (LVDT). Readings were recorded automatically on paper tape by an electronic recorder. The complete operation of this recording system is given in detail by Rennie (1966).

Generally, the residual condition was assumed to have been reached when the shear load dropped to a minimum and remained constant.

The displacement rate used in obtaining the peak effective stresses was .110 inches per day. Using this rate, it required approximately 24 hours to attain the maximum displacement of.12 inches.

In most cases, the displacement rate of 2.76 inches per day (2 hours per cycle) was employed after the peak stress had been attained.

(b) Direct Shear Strength Results

The laboratory strength program involved testing 45 individual samples at a total of 70 accumulated normal loads.

The direct shear results are summarized in Table III-4 for all peak and residual parameters. The actual test results on Mohr envelopes are illustrated in Figures III-1 and III-2.

For some soils the three residual envelopes are shown including the undisturbed, remolded and pre-cut conditions.

Where linear Mohr envelopes were indicated, the method of least squares was used to fit the data. All peak and undisturbed residual test samples were consolidated to 100 per cent theoretical consolidation based on logarithm of time.

The remolded samples were prepared by crushing air dried soil and then sieving them through a No. 40 mesh sieve. A slurry sample was consolidated to a pressure of 4 tons per sq. ft. before being subjected to the various normal loads of the direct shear test.

Pre-cut specimens were prepared from undisturbed samples. They were consolidated to the desired normal load before and after cutting in order to reduce settlement of the pre-cut surface below the shear box edges.

3.4 THIN SECTION STUDY

Thirteen samples were impregnated with Carbowax 6000 before preparation of geological thin sections. Carbowax is used to reduce shrinkage of the sample as it replaces the water.

The thin sections were prepared by the Geology Division of the Research Council of Alberta. A summary of the microscopic and macroscopic features found in these thin sections is given in Table III-5.

The macrostructure features viewed by eye included primarily the bedding and structural features from previous stressing.

In the microstructure study, structure, texture, composition and orientation were investigated. Structure included the extent of laminations, homogeneity and the degree of particle "break-down" in the remolded specimens. Texture involved the shape and alignment of coarse silt and sand particles, as well as a microscopic description of the particles. Composition included a microscopic identification of coarse silt and sand particles as well as zones of montmorillonite. An estimate was made of the percentage of coarse silt and sand particles of the total section by a "point count". The major minerals were identified and noted. The orientation of the minerals within the sample was determined by crossed nicols. A qualitative percentage is given which indicates the percentage of the sample which possessed significant orientation. The orientation is described as patchy when areas of the sample exhibit orientation and others do not.

The microscopic examination of the failure plane was focused upon its irregularity or straightness and smooth or fragmented walls.

3.5 CONSOLIDATION RESULTS

Consolidation tests were performed on Dunvegan pre-till clay, Peace River pre-till and post-till clay and Lesueur bentonitic shale. The principal purpose of the tests was to establish the approximate permeability of the soils. The results are listed in Table III-6 and include preconsolidation load, coefficient of permeability, compressive index, constant volume swelling pressure and theoretical 100% consolidation.

TABLE III-1

PROPERTIES OF SLIDE MATERIALS***

Soil Type	W_L	W_P	I_P	A	W_N	e_n	I	γ_t	% Sand	% Silt	% Clay	G_S
Taylor Clay Shale	37	20	17	0.5	14	0.59	-.35	128	10	56	34	2.67
Dunvegan Pre-till Clay	47	14	33	0.7	25	0.75	.33	124	1	54	45	2.75
Peace River Pre-till Clay	73	27	46	0.7	32	0.93	.11	118	4	31	65	2.75
Peace River Post-till Clay	60	22	38	0.7	27	0.85	.13	119	3	40	57	2.77
Shaftesbury Shale Peace River	51	35	16	0.4	--	--	--	--	7	53	40	2.68
Lesueur Bentonitic Clay Shale	227	51	176	2.6	36	1.02	-0.9	112	5	27	68	2.67
Lesueur Brown Clay Shale	83	45	38	0.9	40	1.09	-.13	110	1	59	40	2.58
Riverbank Clay Shale*	60	25	35	1.2	29	0.70	.11	115	30	40	30	--
Riverbank Bentonitic Clay Shale*	125	40	85	1.9	40	1.00	0	112	17	38	45	--
Riverbank Bentonite*	210	60	150	2.3	60	1.50	0	103	5	3	92	--
Little Smoky Clay Shale**	48	23	25	0.6	21	0.41	-.08	139	5	55	40	2.71

*Results from Department of Civil Engineering Report (1968)
** After Hayley (1968)
*** Results obtained from single representative specimen

TABLE III-2

MINERALOGICAL COMPOSITION OF CLAY FRACTION***

Soil Type	% Montmorillonite	% Illite	% Kaolinite and/or Chlorite
Taylor Clay Shale	2	73	25
Dunvegan Pre-till Clay	2	66	33
Peace River Pre-till Clay	17	58	25
Peace River Post-till Clay	22	58	20
Shaftesbury Shale Peace River	6	77	17
Lesueur Bentonitic Clay Shale	80	20	--
Lesueur Brown Clay Shale	94	4	2
Riverbank Clay Shale*	90	5	--
Riverbank Bentonitic Clay Shale*	90	5	--
Riverbank Bentonite*	100	--	--
Little Smoky Clay Shale**	20	60	20

*After Department of Civil Engineering (1968)
**After Hayley (1968)
*** Smaller than 2 microns

TABLE III-3

PHYSICO-CHEMICAL PROPERTIES OF SOILS

Soil Type	Cation Exchange Capacity	Total Cations	Na^{+}	Ca^{++}	Mg^{++}	K^{+}
Taylor Clay Shale	14.2	33	0/0	29/87	1/2	4/11
Dunvegan Pre-till Clay	13.6	48	12/25	30/63	.6/1	5/11
Peace River Pre-till Clay	23.2	45	2/5	36/80	1/2	6/13
Peace River Post-till Clay	18.0	41	1/2	29/72	.4/1	10/25
Shaftesbury Shale Peace River	22.0	25	0/0	23/90	1/3	2/7
Lesueur Bentonitic Clay Shale	50.1	69	35/50	30/43	1/2	4/5
Lesueur Brown Clay Shale	44.3	57	31/54	23/41	1/1	2/4
Riverbank Clay Shale*	41.0	51	3/5	38/75	1/2	9/17
Riverbank Bentonitic Clay Shale*	69.0	90	4/5	74/82	12/13	1/1
Riverbank Bentonite*	71.0	114	1/1	70/62	1/1	42/36
Little Smoky Clay Shale**	--	--	--	--	--	--

*After Department of Civil Engineering (1968)
**After Hayley (1968)

Note: All results in milliequivalents per 100 gms. air dried soil except percentage of cation follows thus/62

TABLE III-4

SUMMARY OF SHEAR STRENGTH RESULTS

Soil Type	ϕ'_P	C'_P	ϕ'_{UR}	C'_{UR}	ϕ'_{RR}	C'_{RR}	ϕ'_{PR}	C'_{PR}
Bentonitic Clay Shale Lesueur	22.5 19°*	1.0 1.5*	10°	5.2	7.5	0	8.5	1.4
Brown Shale Lesueur	29.0 9*	1.0 21.0	7.5	7.1	--	--	--	--
Taylor Clay Shale	37* 31*	0* 10.4*	16.1	1.6	15.2	0	--	--
Dunvegan Pre-till Clay	21.5 21.5*	7.0 3.0*	13.4	0	11.0	2.4	9.0	2.1
Peace River Pre-till Clay	12.7	10.4	9.6	4	11.0	0	8.0	0
Peace River Post-till Clay	--	--	12.0	1	--	--	--	--
Riverbank* Bentonitic Shale	25.0	0	8.5	4.5	--	--	--	--
Riverbank* Clay Shale	28.0	7.0	12.0	2.6	--	--	--	--
Riverbank* Bentonite	14.0	6.0	8.0	4.5	--	--	--	--
Little Smoky* Clay Shale	32.0	0	14.0	4.0	19.5	3.0	14.0	0

*From previous work

ϕ'_P - effective peak angle of friction

C'_P - effective peak cohesion

ϕ'_{UR} - effective undisturbed residual angle of friction

C'_{UR} - effective undisturbed cohesion

ϕ'_{RR} - effective remolded angle of friction

C'_{RR} - effective remolded cohesion

ϕ'_{PR} - effective pre-cut angle of friction

C'_{PR} - effective pre-cut cohesion

Note: Cohesion in psi

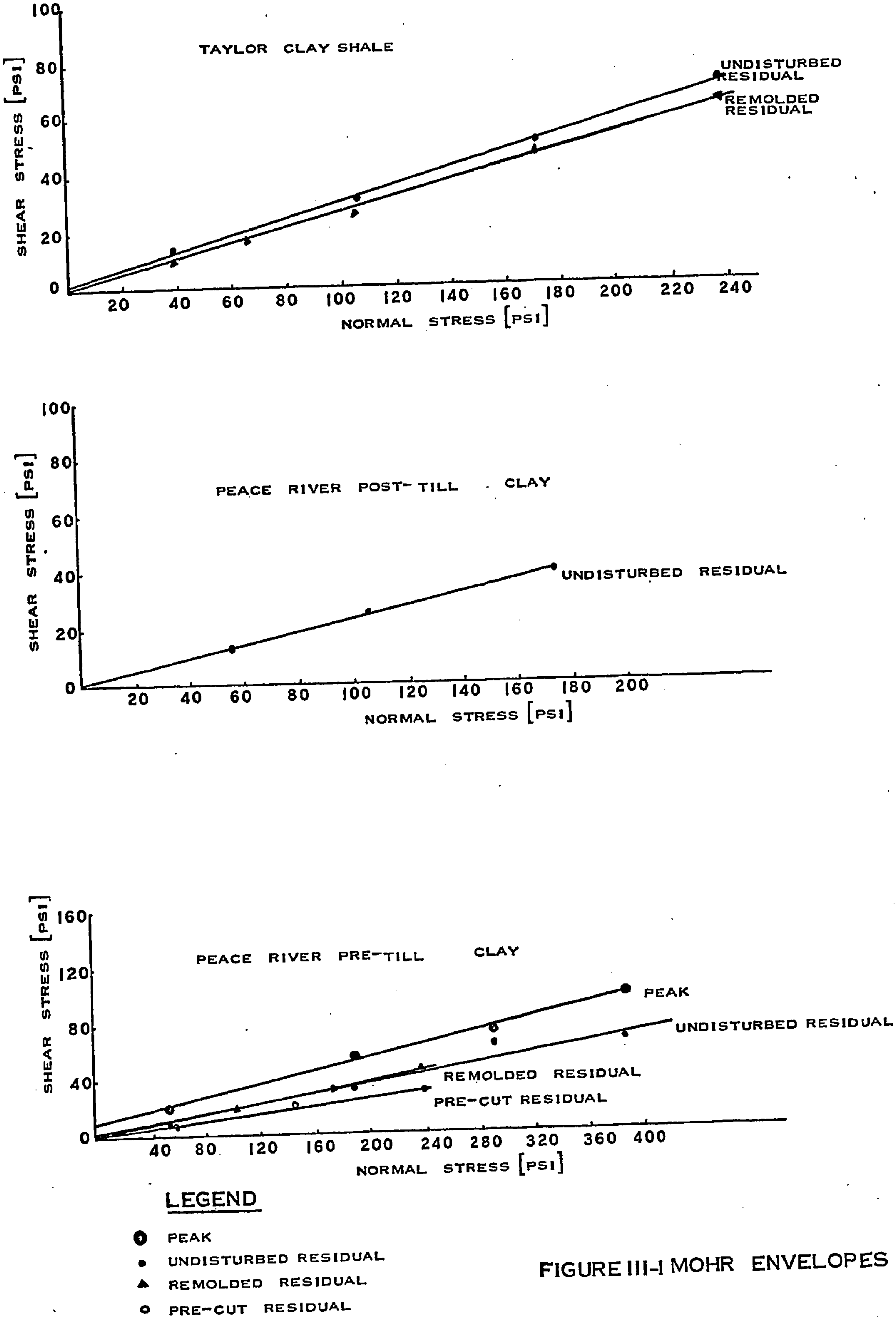
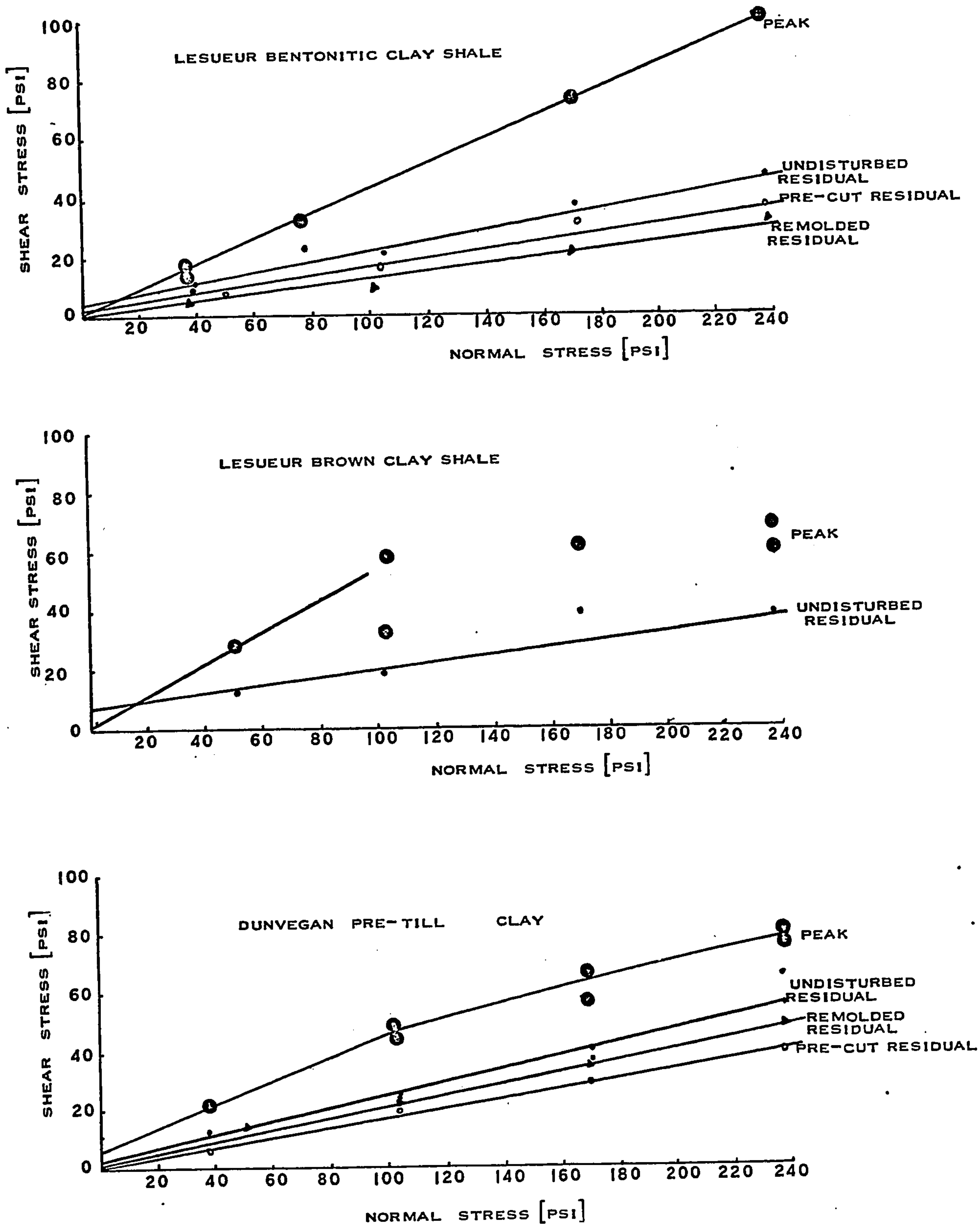


FIGURE III-1 MOHR ENVELOPES



LEGEND

- PEAK
- UNDISTURBED RESIDUAL
- REMOLDED RESIDUAL
- PRE-CUT RESIDUAL

FIGURE III-2 MOHR ENVELOPES

TABLE III-5

SUMMARY OF THIN SECTION STUDY

Soil Type	Macrostructure	Microstructure: Structure	Texture	Composition	Orientation	Failure Plane
Lesueur Bent. Clay Shale Undisturbed	Bedding present	Nil	Coarse silt in clay matrix. Clay lenses with very low silt content	30% coarse silt and sand. Quartz*, volcanic debri*, mica, organic, pyrite	<10%	Fragmented, full of shear lenses. Excellent remolded material. 80% of F.P. through clay
Lesueur Bent. Clay Shale Remolded	Nil	Pushed structure Homogeneous	Coarse silt in clay matrix. Alignment of coarse silt particles	15% coarse silt and sand. Quartz* Chert montmorillonite feldspar, mica	60% to 70%	Poorly defined. No additional orientational in F.P.
Lesueur Brown Shale Undisturbed	Bedding present Coal growth	Nil	Scattered silt altered volcanic material (Shards) in clay matrix. large amount of organic parallel to bedding	<5% silt organic and montmorillonite	10 to 20%	Fragmented. Sample is highly fractured. F.P. difficult to find
Taylor Shale Undistrubed	Bedding present Stained	Laminations due to high and low organic content fine grained material associated with organic	Fine to coarse grained material in clay matrix	<10% coarse silt. Quartz, organic*, stained micaceous	10 to 20% patchy Higher in stained area	Completely fractured
Taylor Shale Remolded	Nil	Pellets of original material	Fine to coarse grained material in clay matrix	<10% coarse silt Quartz*, organic, chert	Original material 90% remolded 20 to 30%	Smooth, slightly irregular
Peace River Preglacial Clay Undisturbed	Bedding present Contorted	Failure along slip planes Contorted	Fine grained clay Light gritty patches, coarse silt in clay matrix	Fine grained clay. Light gritty patches 30% coarse silt, Quartz, mica	90% in clay None in gritty patches	Smooth, irregular
Peace River Postglacial Clay Undisturbed	Bedding present Contorted	Failure along slip planes	Homogeneous fine grained	Very fine grained mica* organic, pyrite	Patchy 70%	Two distinct F.P. but movement along many planes of weakness
Dunvegan Preglacial Clay Undisturbed	Bedding present	Nil	Irregular subparallel lenses of fine silt in clay matrix, high organic content. Also with low organic content	Organic* finely shredded mica, few quartz grains. Carbonate fragments	40 to 50%	Smooth, slightly irregular
Dunvegan Preglacial Clay Remolded	Nil	Nil	Homogeneous, fine grained silt in clay matrix	Finely shredded mica* quartz, organic*	50 to 60%	Smooth, straight
Dunvegan Preglacial Clay Pre-cut	Bedding present	Laminations due to variations in silt content	Fine to very fine silt in clay matrix	Little coarse silt, miceceous shreads, quartz	Patchy, 10 to 50%	Smooth, partially destroyed by thin sectioning

*Material of highest percentage

TABLE III-6

SUMMARY OF CONSOLIDATION RESULTS

Soil Type	Load Increment (TSF)	P_C	k	C_C	P_S	100% Consolidation (MIN)
Dunvegan Pre-till Clay	2-4	11	1.67×10^{-7}	.22	.25	8
Peace River Pre-till Clay	2-4	2.7	1.66×10^{-8}	.30	.40	145
Peace River Post-till Clay	1-3	12	2.12×10^{-8}	.365	1.0	10
Lesueur Bentonitic Clay Shale	2-4	9.4	1.58×10^{-9}	.68	2.0	1000
Little Smoky Clay Shale*	-	>48	3×10^{-8}	-	1.4	--

*After Hayley (1968)

P_C = preconsolidation load tons/ft^2
k = coefficient of permeability cm/sec
C_C = compressive index
P_S = swelling pressure tons/ft^2

CHAPTER IV

DISCUSSION OF LABORATORY RESULTS

4.1 DESCRIPTION OF MATERIALS

The materials investigated most thoroughly in this study were subjected to slaking and wet-dry cycles. All of the soils except the Shaftesbury clay shale from Taylor and Peace River reverted to a disintegrated clay mass after one day of immersion. The Shaftesbury clay shale samples were subsequently destroyed by two wet-dry cycles.

The Dunvegan and Peace River clays do not possess fissility, consequently, they are referred to as preconsolidated clays of pre-till origin. The Taylor clay shale and Lesueur bentonitic shale in their intact state are highly indurated and possess some fissility. The term "clay shale" is placed on such materials which revert quickly to clay when subjected to wet-dry cycles. Philbrick (1950) uses the term clay shale for any shale which reverts to a clay in less than five wet-dry cycles.

The rapid deterioration of an intact soil subjected to slaking conditions is a good indication that the soil possesses weak diagenetic bonds. According to Bjerrum's classification (1966) of weak, strong, and permanent bonds, the weak bonded soils will release their strain energy quickly upon weathering. Consequently, the rate of destruction of the bonds indicates the rate of release of available strain energy. Brooker (1967) has suggested that the more strain energy a soil stores, the greater will be its degree of disintegration. At the present time a satisfactory quantitative measure of strain energy is not available. The stress history of the soils in this study are unknown with regard to stress release, consequently, the amount of strain energy available upon weathering is indefinite.

The soils of this study are classified as weak bonded soils and according to Bjerrum's hypothesis are susceptible to progressive failure.

Generally, the soils possess such plasticity characteristics as to fall slightly above and parallel to the "A" line on Casagrande's plasticity chart (Figure IV-1). The only two soils to fall below the "A" line were the Lesueur brown clay shale and the Shaftesbury clay shale from Peace River. The Lesueur brown shale contained a high content of coal and organic fibres which is rereflected in its low specific gravity of 2.58. Kenney's (1967) results on clay shales are plotted along with those of this study and all fall parallel to the "A" line. Normally, this is interpreted as soils with common geological origins but this, of course, is not true because the soils are from numerous localities.

The liquidity indices for the clay shales are less than zero but for the pre-till clays they are between .1 and .3. These liquidity indices are all below .5 and therefore, satisfy

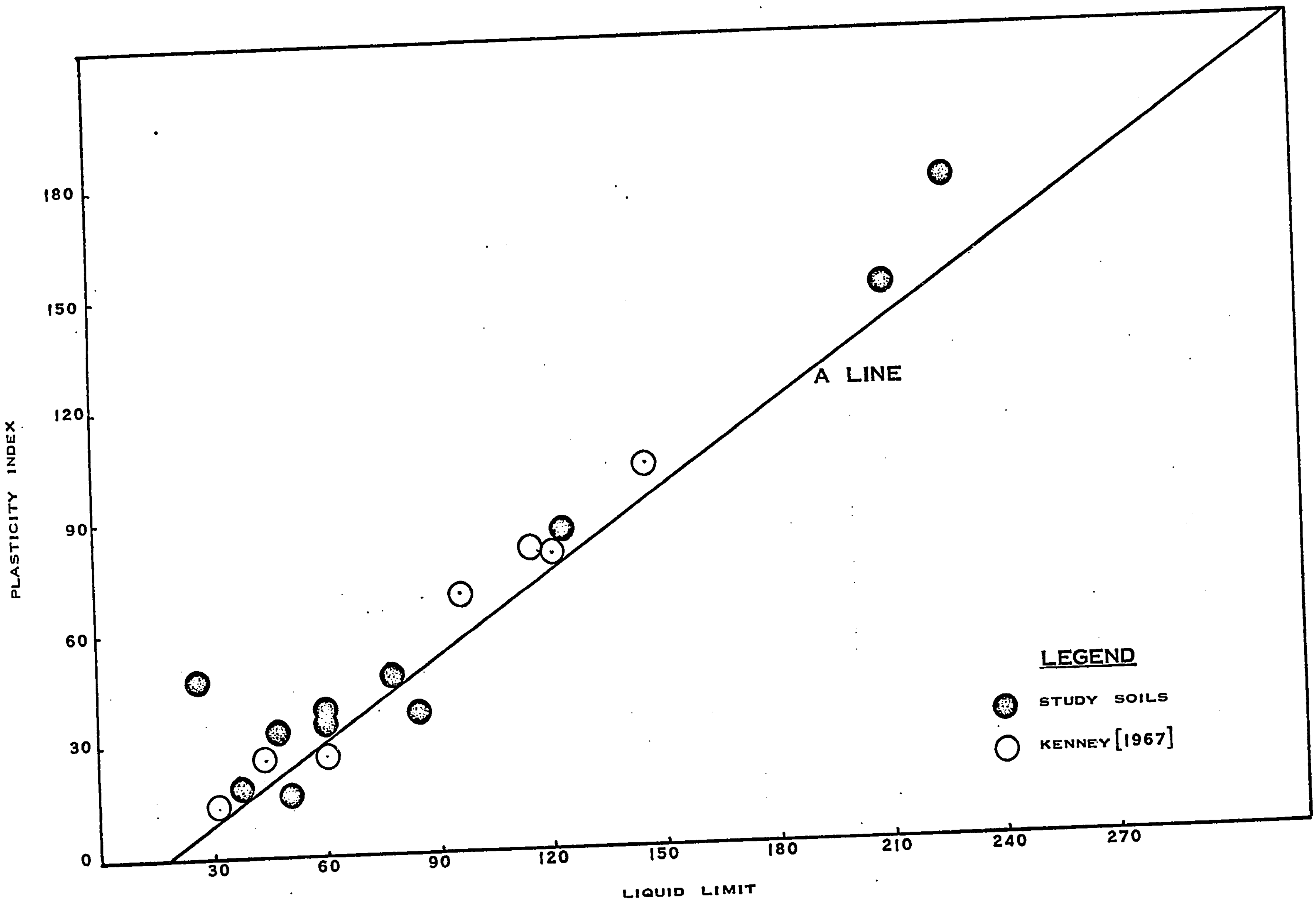


FIGURE IV-I PLASTICITY CHART

Terzaghi's (1936) broad classification for stiff clays.

The samples of this study were obtained from slides which are geographically widespread. The materials from the north sites including Shaftesbury clay shale (Taylor and Peace River), pre-till clay (Dunvegan and Peace River) and Puskwaskau clay shale (Little Smoky) contain a maximum of 20 per cent montmorillonite based on the total clay content but are high in illite content. The Shaftesbury shale which makes up the bedrock in the Peace River area was obtained in a weathered state from an outcrop for comparison with the Taylor Shaftesbury. The soils found in the Edmonton Formation contain over 80 per cent montmorillonite based on the total clay fraction. The difference in clay minerals may be indicative of different sources. The montmorillonite is an altered product of volcanic debris whereas the illite is primarily a product of the weathering of micaceous rocks. It is also possible that the variation in clay minerals is the result of a difference in volcanic activity in the study area.

The high montmorillonite content of the Edmonton Formation is reflected in the high cation exchange capacity as compared to the low exchange capacity of the illite materials. All soils contain a high content of calcium cations except for the Lesueur bentonitic clay shale and brown shale which contain 50% sodium. At the present time it is not possible to relate physico-chemical characteristics of the clay shales to large scale slides.

It is of interest to note that the bentonitic materials obtained from the University of Alberta riverbank are high in calcium content whereas those of the Lesueur bentonitic materials are high in sodium.

The thin sections of the intact soils indicated that the clay particles are only slightly oriented and would generally not be considered to be a significant factor in the stability of the slide areas. Although little orientation is present, the soils do exhibit bedding. Horizontally bedded materials which are subject to progressive failure tend to develop the wedge type of failure common to the slides of Western Canada (Scott and Brooker, 1968).

4.2 SHEAR STRENGTH RESULTS

(a) Comparison of Results with Previous Strength Parameters

The Lesueur bentonitic clay shale was found by the direct shear test to have a peak angle of friction of 22.5 degrees and a cohesion of 1 psi. The samples were obtained from the same depth as those by Painter (1965) but he refers to the material as bentonite which contains 92 per cent clay sizes. The bentonitic clay shale of this investigation with 68 per cent clay sizes was only obtained 30 feet from the test hole drilled by Painter. Assuming that both results are correct, this is an indication of the significant variations which exist in the Edmonton Formation.

Results from tests on a bentonitic clay shale sample from the University of Alberta riverbank (Sinclair and Brooker, 1967) indicate a peak angle of 25 degrees and zero cohesion by both triaxial and direct shear methods.

Tests by Painter (1965) on Lesueur bentonitic clay shale resulted in a peak angle of shearing resistance of 17 degrees and cohesion of 4 psi.

Consequently, the peak angle of shearing resistance for the Edmonton Formation bentonitic shale appears to average 22 degrees.

The peak parameters obtained for Dunvegan clay were $\phi = 21.5°$ and $C = 3$ psi. These results are very similar to those obtained from triaxial results reported by Hardy et al.(1962) in which $\phi = 21.5°$ and $C = 7$ psi.

Previous results were not available for the Peace River clay and the peak parameters for Taylor clay shale were not determined in this investigation.

From the results of this study, it appears that the direct shear test parameters are consistent with those obtained from triaxial results. Therefore, the use of the direct shear apparatus for peak strengths is justified.

(b) Comparison of the Methods of Determining Residual Strength

The principal object of the laboratory investigation was to determine the residual strength parameters applicable for stability analysis.

A definite trend was indicated from the undisturbed, remolded and pre-cut procedures. Generally, the undisturbed residual parameters were slightly higher than the remolded parameters but significantly higher than the pre-cut results as summarized in Table III-4.

The residual angles of shearing resistance from undisturbed and pre-cut samples for Lesueur bentonitic clay shale were 10^{o} and 8.5^{o}, respectively, for Dunvegan pre-till clay 13.4^{o} and 9.0^{o}, respectively and for Peace River pre-till clay 9.6^{o} and 8^{o}, respectively.

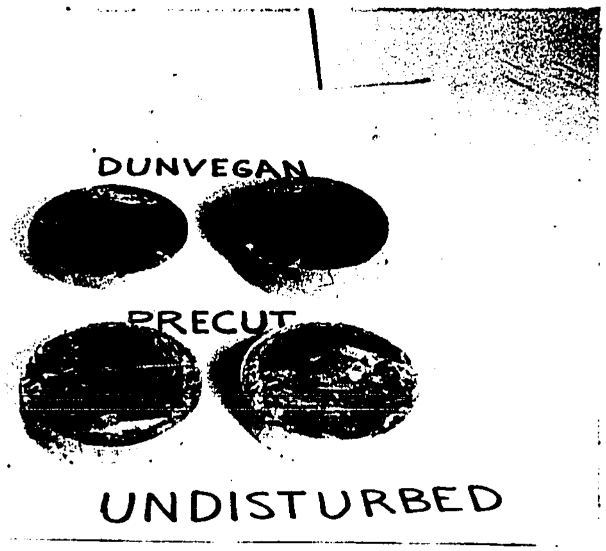
From Table III-4 it is also noted that all undisturbed residual results except for Dunvegan clay exhibited cohesion intercepts. The undisturbed cohesion parameters were generally larger than those for the remolded and pre-cut specimens. The average cohesion exhibited by all the soils in Table III-4 for the undisturbed, remolded and pre-cut results were 3.5, 1.1 and 0.9 psi, respectively. The best interpretation of the Mohr envelopes indicates the presence of residual cohesion although the magnitude of the cohesion may be slightly in error because the envelopes were not defined in the low normal stress range. It is postulated that residual cohesion is a form of frictional resistance developed when irregularities are present and it is the shearing resistance at zero normal load.

It has definitely been shown that the residual strength parameters obtained from undisturbed samples are higher than those obtained from the remolded or pre-cut specimens.

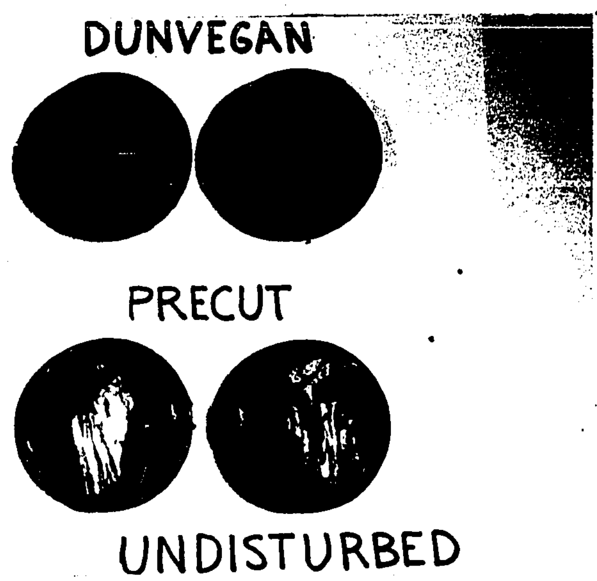
The higher results from undisturbed specimens are the result of macroscopic and microscopic irregularities along the failure plane. Irregularity is used as a measure of the departure from a straight line.

Plate IV-1 (a) and (b) show two views of the failure surfaces of undisturbed and pre-cut samples of Dunvegan clay. The irregularities of the undisturbed sample are approximately one-quarter inch in height which corresponds to hollows of one-quarter inch. Plate IV-2 shows the relative roughness of the failure planes for Dunvegan clay thin sections for the undisturbed pre-cut and remolded cases, respectively. The undisturbed sample contains a great number of irregularities, whereas, the pre-cut samples are smooth.

Plate IV-3 which is an enlarged photograph of a thin section of Peace River pre-till clay shows the irregular shape of the two failure planes produced during direct double shear. This photograph also shows on a macrostructure scale, the shear induced structure which was produced by previous slide activity. Numerous slickensides are visible in the block sample from which test specimens were carved. The shear structure exhibited by the photograph should not be confused with those shear induced structures described by Morgenstern and Tchalenko (1967) and which are produced in the direct shear apparatus. The Peace River clay contains significant variations in composition. The photomicrographs of Plate IV-4 show two typical areas of different composition. Plate IV-4 (a) shows a coarse gritty silt area in which individual silt particles can be seen and Plate IV-4 (b) indicates the predominantly fine grained nature of the clay areas. The microscopic variation in composition may have significant effect upon the actual residual

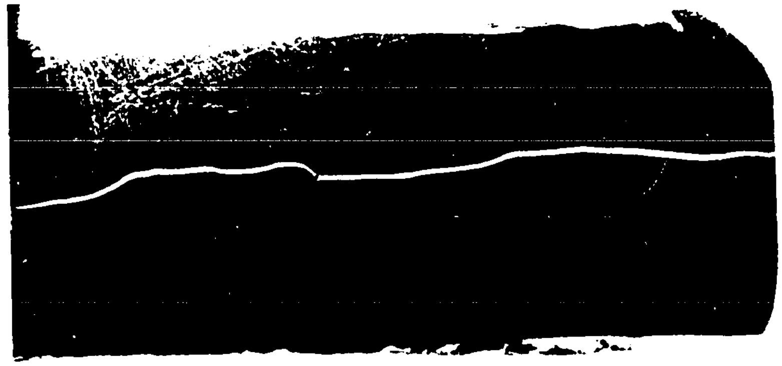


[A] ANGLE VIEW

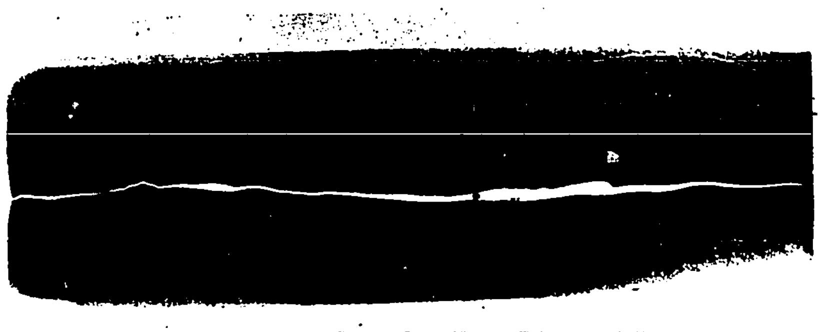


[B] PLAN VIEW

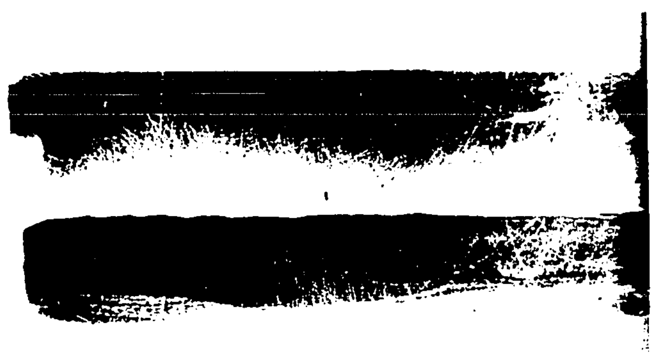
PLATE IV–I IRREGULARITIES OF DUNVEGAN PRE–GLACIAL CLAY



[A] UNDISTURBED FAILURE PLANE



[B] REMOLDED . FAILURE PLANE



[C] PRE-CUT FAILURE PLANE [PARTIALLY DESTROYED]

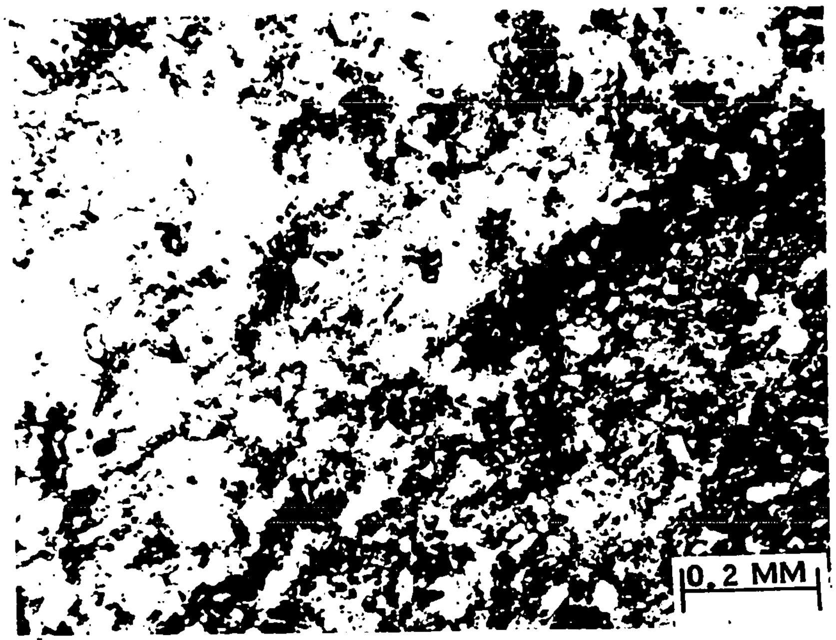
[ENLARGED 2 TIMES]

PLATE IV-2 THIN SECTION PHOTOGRAPHS OF DUNVEGAN CLAY FAILURE PLANES

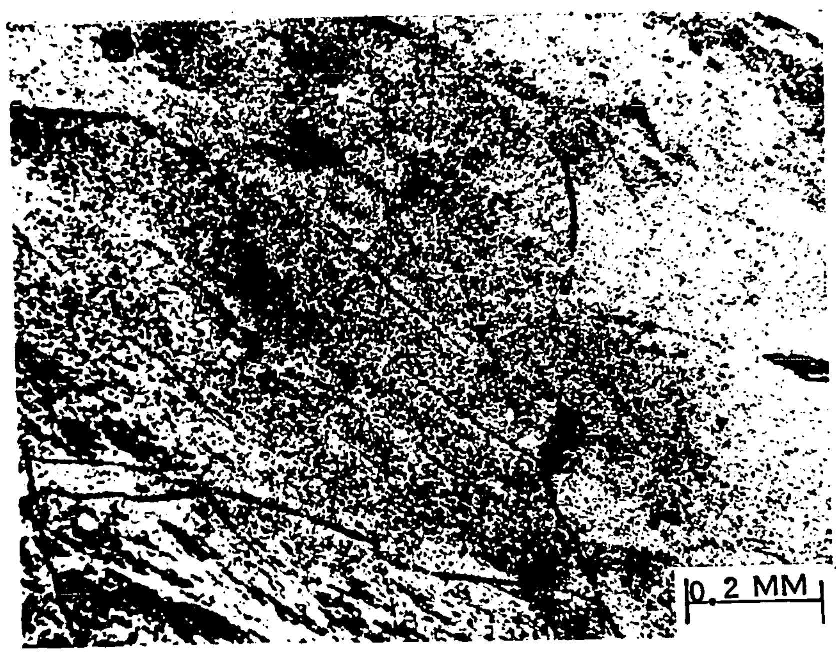
→ FAILURE PLANE LOCATION

(ENLARGED 3.5 TIMES)

PLATE IV 3 PEACE RIVER PRE GLACIAL CLAY

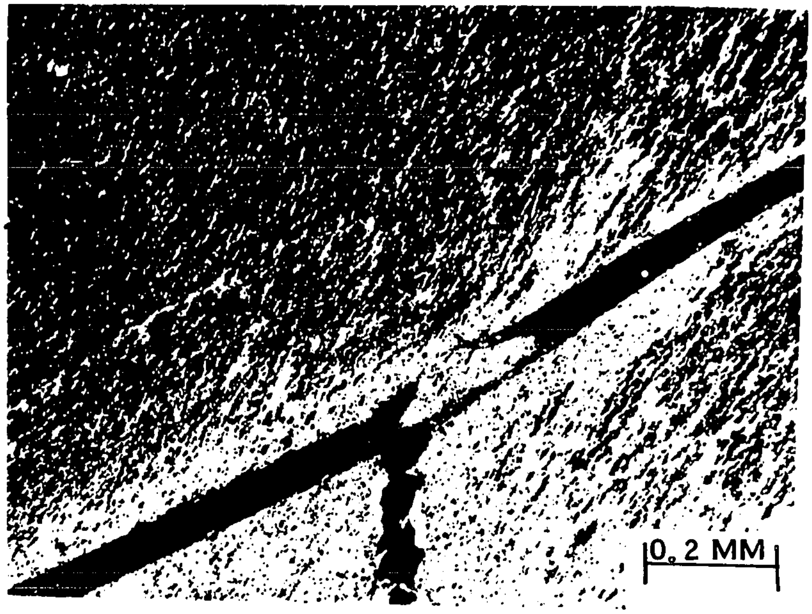


[A] PEACE RIVER PRE-GLACIAL CLAY PHOTOMICROGRAPH OF SILTY AREAS [PLAIN LIGHT]



[B] PEACE RIVER PRE-GLACIAL CLAY PHOTOMICROGRAPH OF CLAY AREAS [CROSSED NICOLS]

PLATE IV-4 PHOTOMICROGRAPHS OF PEACE RIVER PRE-GLACIAL CLAY SHOWING COMPOSITION VARIATION



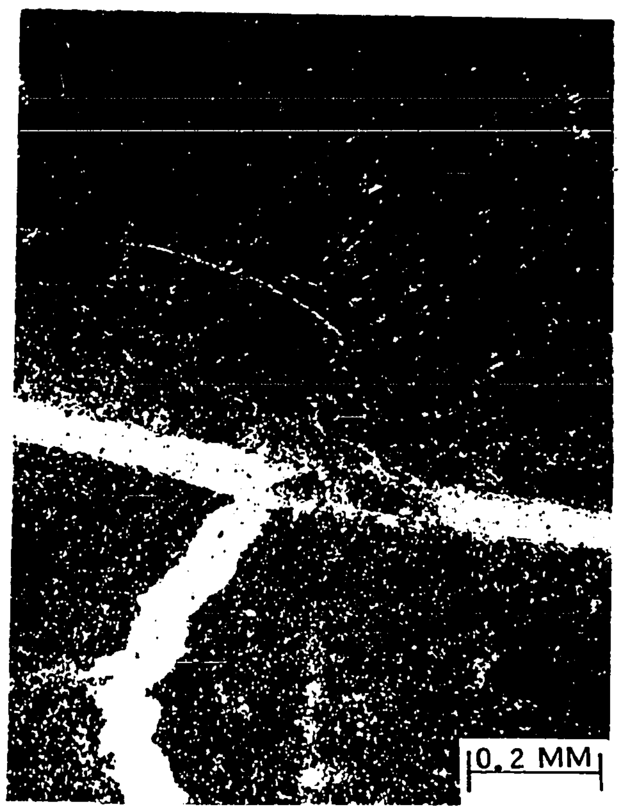
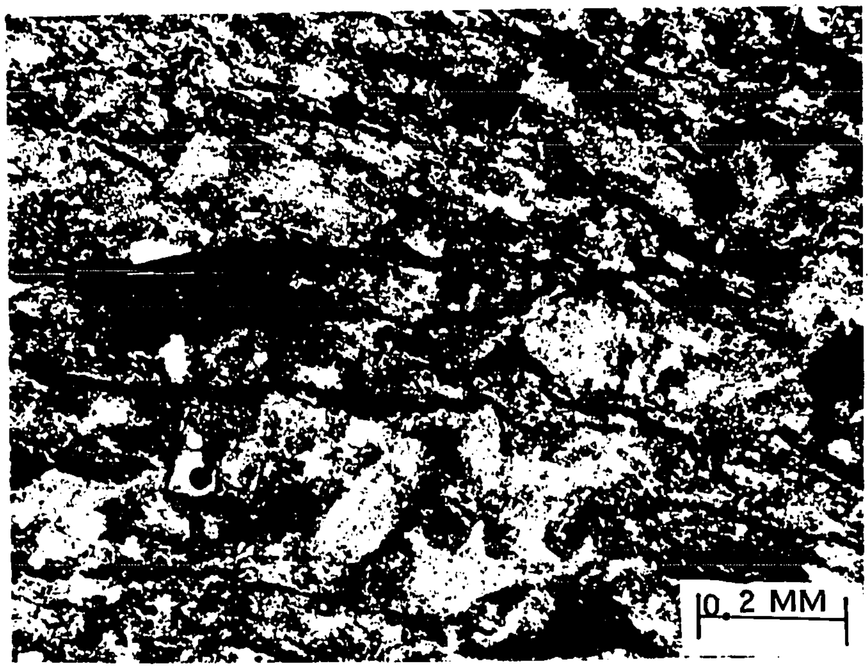
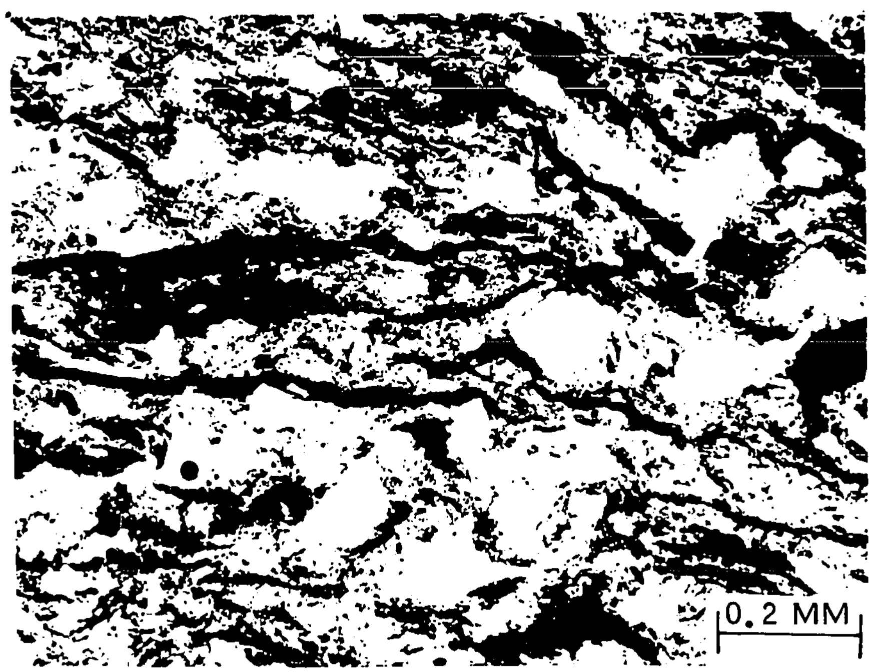


PLATE IV–5 HIGHLY ORIENTATED CLAY ZONES OF PEACE RIVER PRE–GLACIAL CLAY [CROSSED NICOLS]



[A] SHARDS [PLAIN LIGHT]



[B] SHARDS [CROSSED NICOLS]

PLATE IV–6 SHARDS IN LESUEUR BROWN SHALE strength developed. Caution is necessary when suggesting a laboratory residual strength from small samples for field failure planes as long as 2000 feet. The Peace River pre-till clay was the only soil of this study which exhibited high orientation. Plate IV-5 (a) and (b) show the orientation by crossed nicols. The areas of greatest orientation exist along the field induced failure surfaces. These photomicrographs also show the smooth walls of the failure surface but because only a small portion of the sample was viewed, the irregular nature of the failure surface is not demonstrated.

Generally, the undisturbed samples possess greater irregularity than the remolded and the remolded slightly more irregular than the pre-cut sample. An anomaly to the above is the 7.5^{o} obtained for the remolded Lesueur bentonitic clay shale as compared to 10.0^{o} and 8.5^{o} for the undisturbed and pre-cut samples, respectively. The lower remolded angle of friction is a result of the breakdown of the soil aggregates of the undisturbed soil. In the undisturbed state, soil aggregates of volcanic debris (shards) increase the resistance along the failure plane. Shards were found in the Lesueur bentonitic clay shale but were not well defined. An excellent example of volcanic debris was found in the Lesueur brown shale and is demonstrated by the photomicrographs in Plate IV-6. The large flakes are examples of shards which convert to montmorillonite particles when remolded. Fundamentally, the remolded soil does not represent the in situ condition in this case. It is therefore justified to use the undisturbed parameters in stability analyses.

The Mohr envelopes obtained for the Lesueur brown clay shale are indefinite because of the extreme scatter which was experienced. This scatter is attributed to the high organic content and shard structure.

The results of the tests performed in this study indicated that the residual strength parameters depend upon whether the samples are undisturbed, remolded or pre-cut. A general statement of the relative strength magnitude of the undisturbed, remolded and pre-cut specimens cannot be made for all soils but rather each soil must be evaluated individually. The Peace River pre-till clay had a residual remolded angle of friction slightly above the undisturbed. This phenomenon may be explained by the variation in composition of the material but definite evidence was not found. All other soils exhibited remolded strengths less than the undisturbed strength.

Skempton (1964) stated that "test results at present show almost invariably that residual cohesion is very small, and probably not significantly different from zero". Consequently, it has been standard procedure by most investigators to assume zero cohesion in the residual case. It will be shown later in Chapter VI that a numerically small cohesion (1 to 2 psi) is significant in a stability analysis and produces significant changes in the factor of safety.

Kenney (1967) in his study of the influence of mineralogy on residual strength initially tested undisturbed samples and found the slip surface to be irregular and striated. He stated that the irregularities increased the apparent strengths. In order to eliminate this influence he used a pre-cut sample so that results were reproducible. He also found that samples prepared from a slurry gave exactly the same results as the pre-cut samples. It is believed that Kenney would not be justified in disregarding the higher strengths of the undisturbed samples when relating to field conditions.

It appears that the action of failure surface irregularities in clay shales and preconsolidated clays is either disregarded or not completely understood.

Most clay shales and preconsolidated clays are anisotropic and heterogeneous in their internal strength characteristics because of bedding and the presence of soil aggregates. Even if a soil macroscopically has isotropic strength characteristics, it will probably be composed of different minerals. These minerals will vary in their hardness, internal strength, cleavage, and coefficients of friction. All of these inequalities within a soil tend to produce an irregular shearing surface. In a multi-mineral composed soil, the failure surface is likely to follow the path of least resistance. For example, it will pass around quartz grains and follow cleavage directions of mica and feldspar. On a macroscopic scale irregularities may develop as a result of structural inconsistencies such as soil aggregates which are harder than the ambient material. These soil aggregates have been found to be present in the bentonitic soils of this study (Plate IV-6).

Many investigators in the field of rock mechanics have reported the presence of failure surface irregularities. Their general conclusion was that irregularities contribute to strength. Goldstein et al. (1966) report that the angle of inclination of the irregularities are related to the shear strength of the material. They postulated that the shear strength along a developed irregular failure plane was given by:

$$\tau = \sigma \tan (\alpha + \phi)$$

σ = normal stress

α = angle of irregularity to horizontal

ϕ = residual angle of shearing resistance of a horizontal surface

Goldstein performed tests on fissured rock surfaces and concluded that the residual angle of shearing resistance of undisturbed rock samples was similar to that along fissures.

DeBeer (1967) found that for Boom clay, the undisturbed residual angle of internal friction, at a strain rate of .4 mm. per hour, was 24^{o} 20' and the pre-cut angle was 19^{o} 20'; this is a decrease of 5^{o}. The peak angle of internal friction was also 24^{o} 20' as determined in the torsion ring apparatus. He did not suggest reasons for the drop but testing technique may be inherent in the results. It is also of interest that the undisturbed residual is similar to the peak angle of internal friction.

It is concluded that the undisturbed residual strength is higher than the pre-cut residual strength because of the presence of irregularities. It is the author's opinion that the undisturbed laboratory strength is the most representative of the field conditions. This is substantiated by the visual inspection of the natural slickensides which existed in the Peace River pre-till clay. The irregularities along the direct shear undisturbed failure surface were similar in magnitude to those of the slickensides, approximately one-quarter inch in height. Although sample size effects are normally significant when comparing laboratory and field strength results, this may not be true when the strength variation is only dependent upon the irregularities of a developed shear surface. Therefore, if the irregularities in the laboratory undisturbed test are similar to those in the field the resistance to shear should be similar in both cases. The maximum size of laboratory irregularities is no doubt restricted but larger field irregularities may form.

The limited thin section study indicated that remolded material was present in the failure surface of undisturbed specimens which had been sheared to the residual strength. The orientation of the remolded soil was similar to that of the ambient material. Therefore, it has been shown that remolded soil occurs along the failure plane but the shear strength properties are not the same as for samples prepared from completely remolded soil. This means that remolding along the failure plane does not justify the use of the remolded strength.

(c) Factors Affecting Residual Strength

A correlation presented by Skempton (1964) showed that residual strength decreased with clay fraction and it has been generally held that the amount of clay in a natural sample was the major factor governing the magnitude of the residual strength.

Kenney (1967) explored the influence of mineralogy on residual strength and concluded that it was the controlling factor.

The results of Skempton and Kenney are plotted along with those of this study in Figure IV-2. The plot is that of per cent clay fraction versus undisturbed residual angle of friction. Considering all materials, a poor relationship exists, but Skempton's points along with those of this study appear to establish a trend such that clay fraction could be used to estimate the residual angle. The Lesueur brown shale probably deviates from the trend as a result of its high organic content.

Figures IV-3, IV-4, IV-5, and IV-6 include residual strength results of Kenney and those summarized in Table III-4 plotted against per cent montmorillonite of the clay fraction, per cent montmorillonite of the total sample, activity, and liquid limit, respectively. A general decrease in residual strength was noted for a decrease in the variables except for the activity which increases. The plot of liquid limit versus residual angle indicates that soils with liquid limits greater than 75 will possess residual angles of shearing resistance less than 10 degrees. The plot of per cent montmorillonite of the total sample versus residual angle indicates that materials containing more than 20 per cent montmorillonite will exhibit residual angles of 10 degrees or less. The activity plot indicates that stiff clays and shales will possess residual angles of 10 degrees or less if the activity is greater than .7. The plot of per cent montmorillonite based on clay fraction

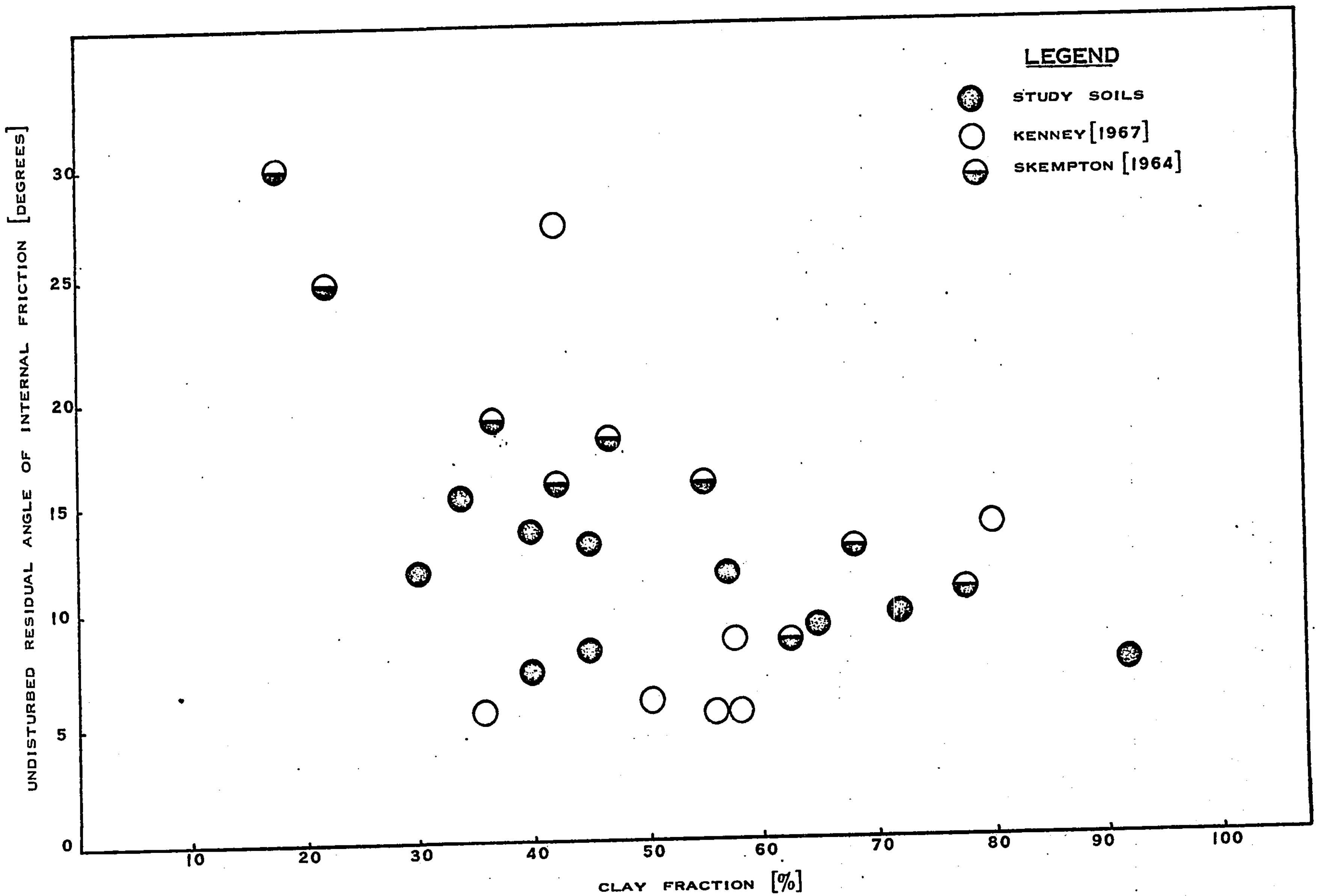


FIGURE IV–2 RESIDUAL ANGLE OF INTERNAL FRICTION VERSUS PER CENT CLAY FRACTION

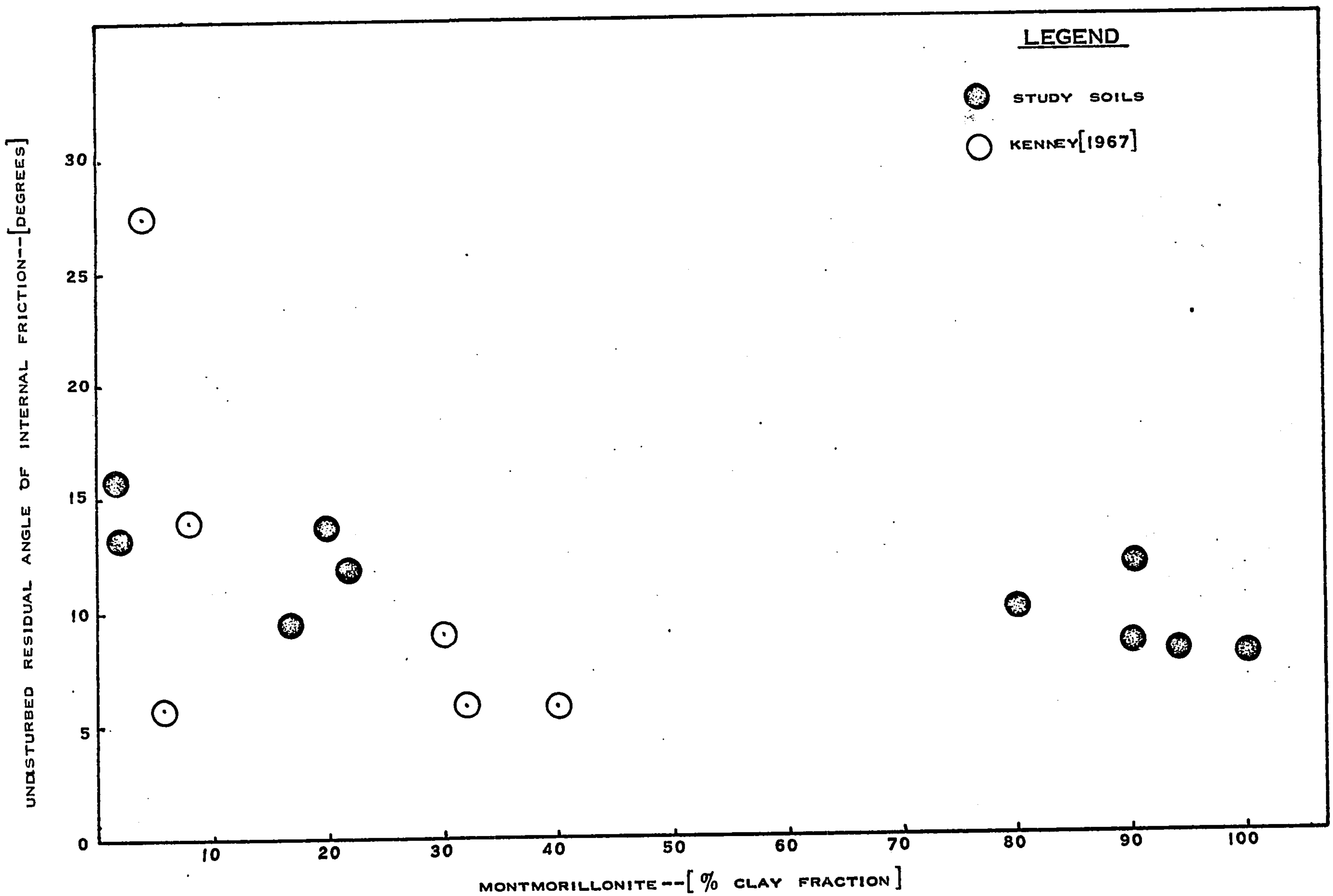


FIGURE IV-3 RESIDUAL ANGLE OF INTERNAL FRICTION VERSUS PER CENT MONTMORILLONITE

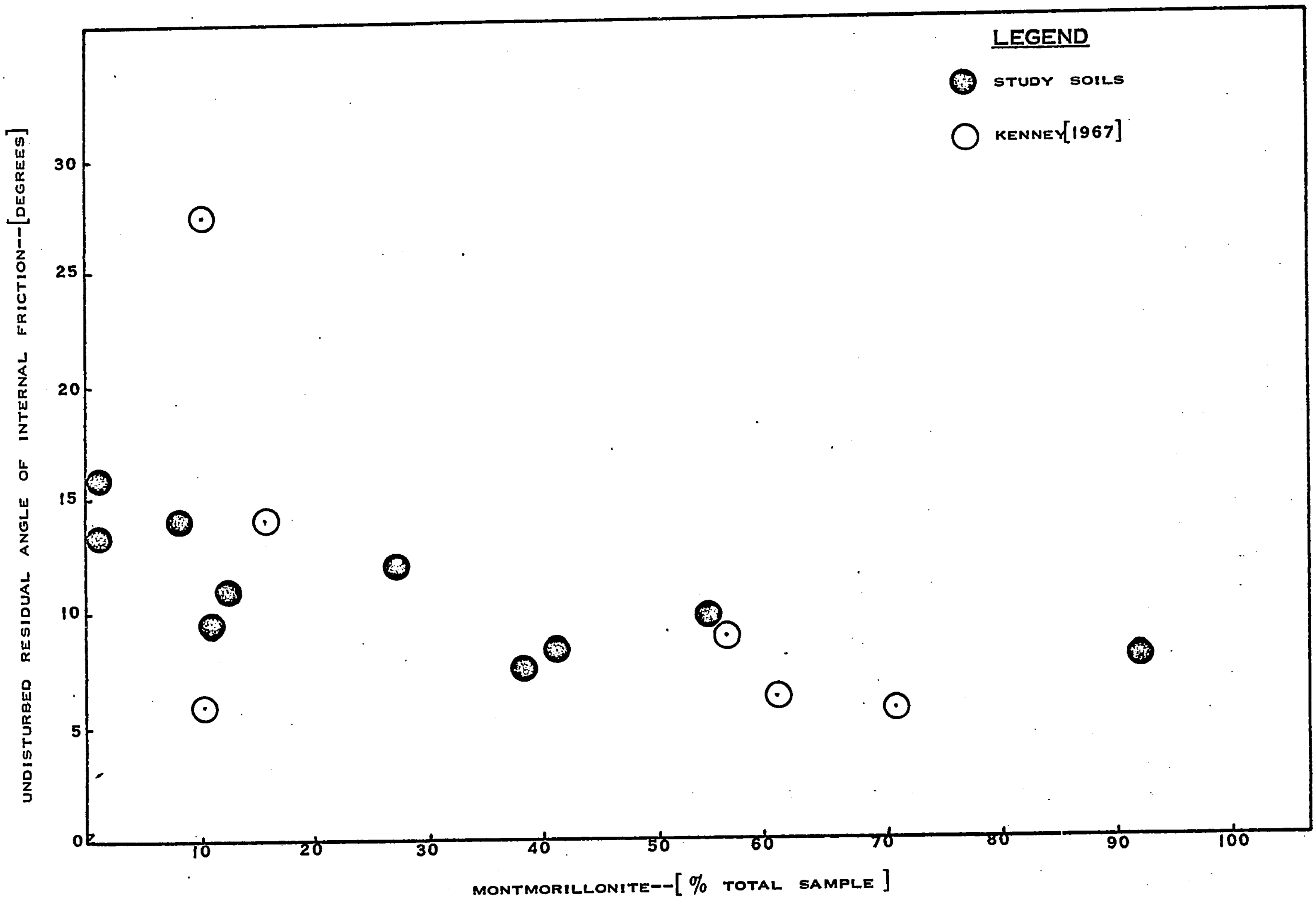


FIGURE IV–4 RESIDUAL ANGLE OF INTERNAL FRICTION VERSUS PER CENT MONTMORILLONITE

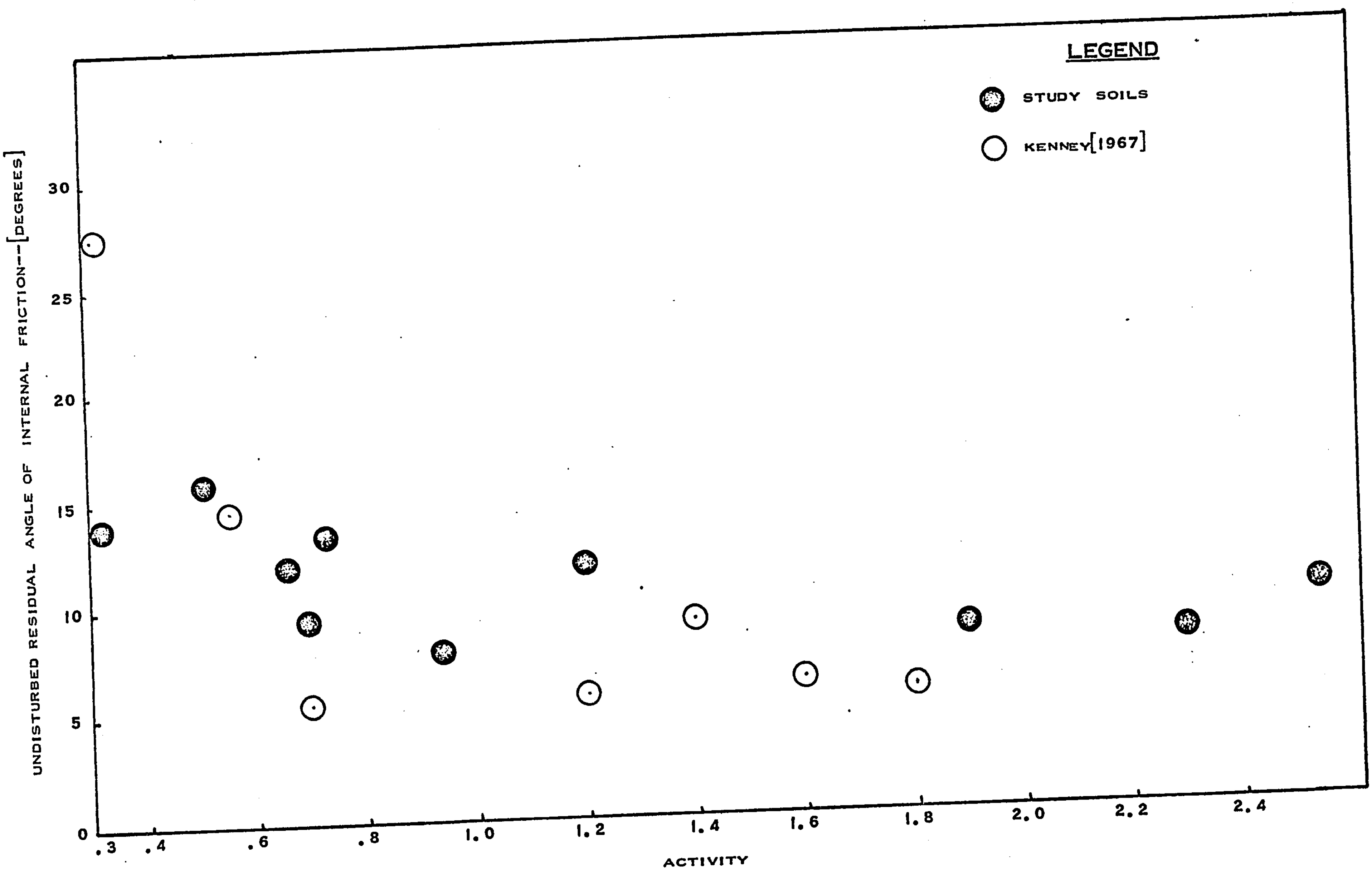


FIGURE IV-5 RESIDUAL ANGLE OF INTERNAL FRICTION VERSUS ACTIVITY

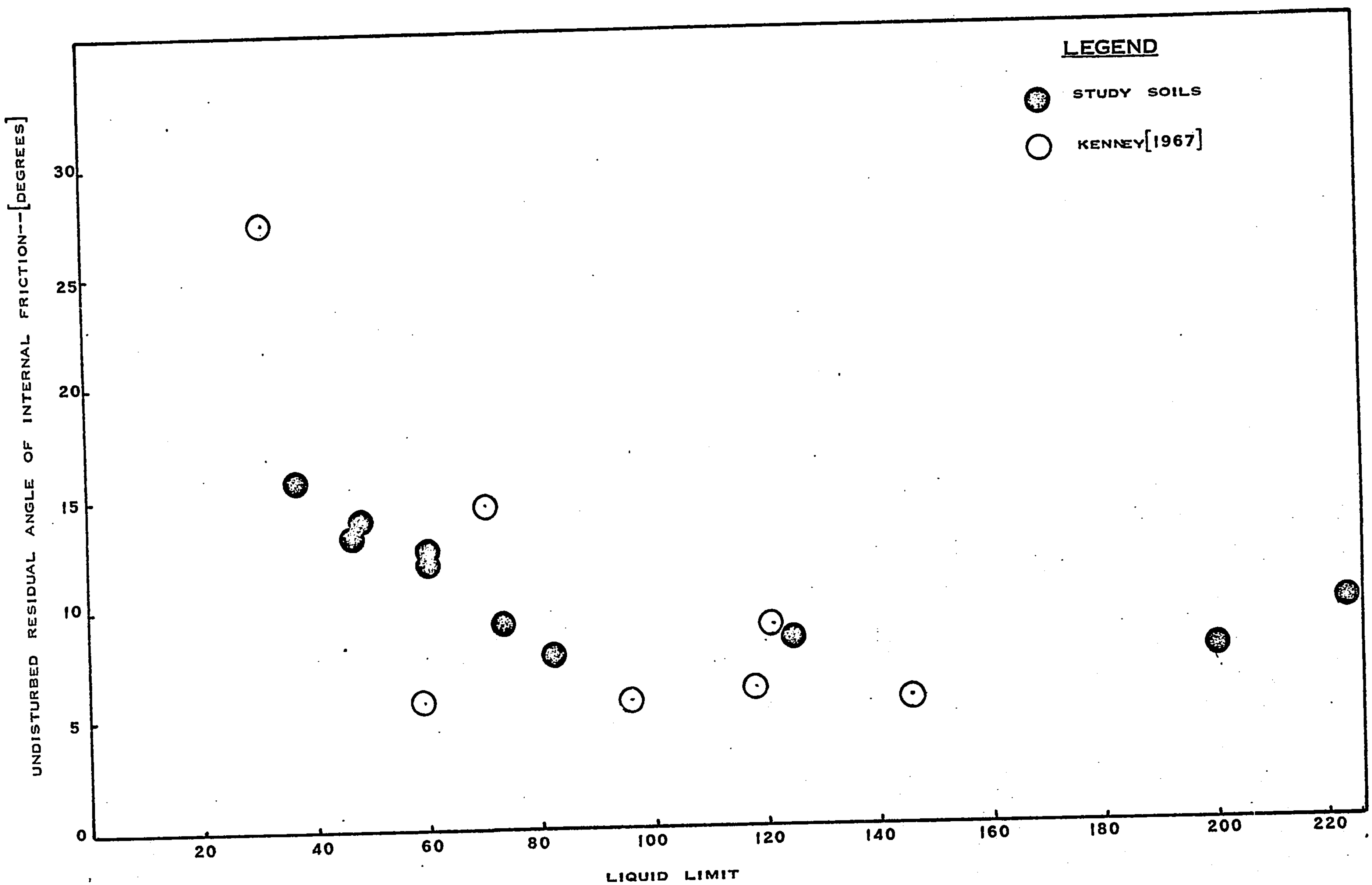


FIGURE IV–6 RESIDUAL ANGLE OF INTERNAL FRICTION VERSUS LIQUID LIMIT

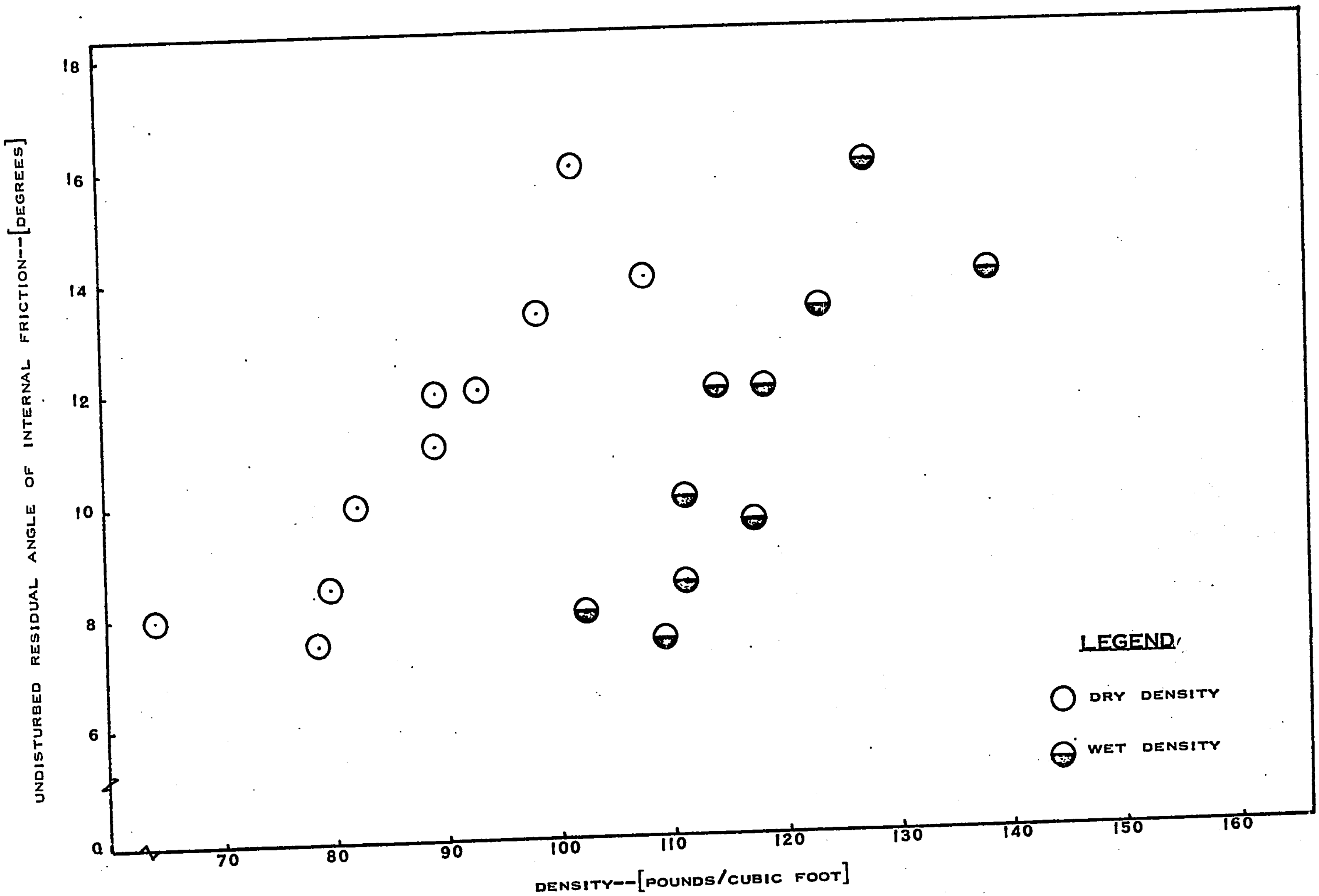


FIGURE IV-7 RESIDUAL ANGLE OF INTERNAL FRICTION VERSUS DENSITY

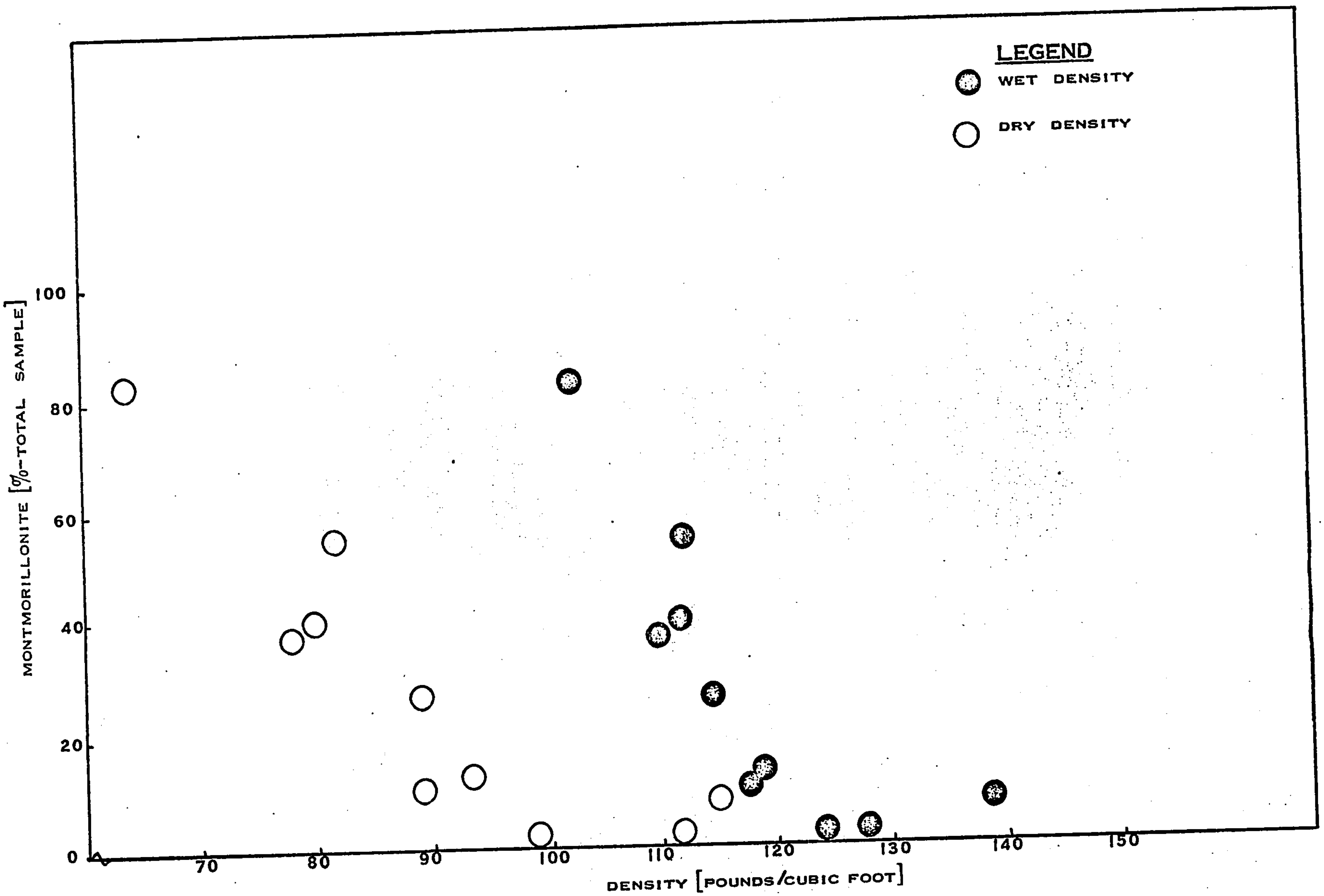


FIGURE IV-8 PER CENT MONTMORILLONITE VERSUS DENSITY represents the least reliable indicator for the residual strength angle.

Although Kenney (1967) has shown in his study of natural soils and minerals that mineralogical composition is the primary controlling factor the prediction of residual strength angles can be made with as great a reliability using the liquid limit or the activity of the material. The natural materials contain an assortment of minerals whose combined properties are reflected in the liquid limit, whereas the per cent montmorillonite does not consider the effects of other minerals.

Of interest is Figure IV-7 in which both original dry density and total density are plotted versus the undisturbed residual angle of friction. In both cases a well established relationship of increasing residual strength with increasing density is indicated. Figure IV-8 which is a plot of density versus per cent montmorillonite indicates that as density increases the montmorillonite content decreases. Therefore, the effect of density on residual strength can perhaps be related to mineralogical content.

(d) Effect of Normal Stress on Internal Friction

The curvature of the Mohr envelope or decrease in internal friction angle with increasing normal stress is commonly encountered in porous materials including concrete, rock and soil. The Dunvegan pre-till clay exhibited this phenomenon. The peak angle of internal friction was found to be 21.5^{o} for normal stresses below 100 psi. At 237 psi the friction angle decreased to 13 degrees, which is comparable to the residual angle of friction. Because this study has indicated that the residual strength is influenced by the magnitude of irregularities along the failure plane, and that normal stress affects shear strength, a review of available literature was made to ascertain the possible relationship between irregularities, normal stress and shear strength.

Terzaghi (1962) reviewed tests performed on marble by Ros and Eichinger in which a drop from 40^{o} at a normal stress of 100 kg. per sq. cm. to 25^{o} at 1000 kg. per sq. cm. was obtained. He postulated that the decrease resulted because an increase of the normal stress was associated with an increase in the number of failures across grains. This reduced the resistance against sliding because of the interlock reduction between grains.

Ripley and Lee (1961) found from sliding tests on rock surfaces that higher angles of internal friction were obtained due to the sliding block having to slide up rough projections. The increase in angle of internal friction due to "riding up" on the rough projections amounted to from 10^{o} to 18^{o}. With regard to curved envelopes the "riding up" is probably reduced at high stresses and shearing through particles or groups of particles occurs with subsequent reduction in the angle of internal friction.

Henkel _et al_. (1964) in testing granulite for Monar Dam also found that it was necessary to shear through small projections. In all cases the onset of relative displacement across the joint was associated with an upward movement of the shear box indicating a "riding up" on discontinuities.

Deklotz et al. (1966) investigated the shear strength of an anisotropic schistose gneiss. Of significance was the variation in the angle of internal friction from 17^{o} to 23^{o} at a normal stress of 24,000 psi for various orientations. Curved envelopes were noted to be affected by the anistropic nature of the material. This again perhaps indicated the effect of structure on the Mohr envelope.

Terzaghi (1945) outlined failure components of quasi-isotropic materials including concrete, porcelain and rock. He concluded that failure can be resolved into one or more components including:

(1) Failure by tension in the bond between grains.

(2) Failure by tension across grains.

(3) Failure by shear in the bond between grains.

(4) Failure by shear across grains.

At high pressures the shear occurs almost exclusively within the grains. Consequently, there is definite evidence that as the normal stress increases a change occurs in the mode of failure. Terzaghi also comments on uneven failure surfaces. The humps with the steepest slopes will fail by shear across the hump, and the resistance against such a failure is independent of the normal stress. But the majority of the humps will be overriden. Because displacement is along slopes of gentle humps the direction of movement deviates from the horizontal direction of the direct shear test. With increasing pressure an increasing percentage of the total area of shear cuts across grains. Therefore, the rate of increase of the shearing resistance decreases with increasing normal pressure and a curved envelope results.

Richart et al. (1928) found for concrete and from tests on marble by Karman' that sliding failure at the lower loads was dependent upon the normal stress and was largely between crystals while at very high pressures it depended upon shearing stress only and was largely failure within the crystals.

Lane and Heck (1964) suggest that the angle of internal friction may be a maximum at low pressures and reduces as the points of rock crystals are sheared off. This is again interpreted to produce a broken line type of Mohr's envelope.

Most of the above references which postulate change in mode of failure or structural considerations were based on natural materials in which individual variables could not be isolated. Recent work by Patton (1966) included the detailed investigation of the effect of geometry of irregularities on the shear strength of rocks. Patton investigated several natural slopes and concluded that there are two possible modes of failure including sliding over or shearing off of irregularities. In order to investigate the various parameters of importance he precast plaster of paris samples and subjected them to direct shear. The main conclusions of Patton's investigations which are pertinent to this discussion include:

(1) The inclination of the primary portion of the failure envelopes for specimens with inclined teeth was similar to (ϕ_r + i). Where, ϕ_r = residual angle along a horizontal surface and i = angle of inclination of inclined teeth with the horizontal.

Failure of the steeper teeth occurred at lower normal stresses because the cross sectional area of the intact material is less than for flatter teeth. The secondary portion corresponded to ϕ_r being independent of the peak envelope.

(2) Doubling the number of teeth doubled the vertical distance between the secondary portion of the maximum strength envelope and the residual envelope.

(3) Increasing the strength increased the secondary portion of the failure envelope relative to the residual. On high strength specimens it was found that specimens with four teeth often gave failure envelopes that were only slightly greater than the envelopes for the specimens with two teeth. This was interpreted as evidence of progressive failure.

(4) Specimens with an irregular surface would be expected to have different failure envelopes in each direction.

It appears that the phenomenon which is reported frequently in rock mechanics and concrete literature is the change in mode of failure criteria such that failure changes from between particles to intraparticle failure at high stresses.

Most of the literature pertains to materials other than clays or even shales with regard to curvilinear envelopes. It is of interest that Skempton (1961) refers to curved envelopes for rock, concrete and sand but it appears that examples of curved envelopes for clay were absent.

Curved envelopes on London Clay are presented by Bishop *et al*. (1965) on undisturbed and remolded specimens. The change in angle of internal friction for undisturbed samples was from 30^{o} to 10^{o} and for the remolded from 16^{o} to 10^{o}. The envelopes are curved up to stresses of 350 psi and the estimated preconsolidation load is 600 psi. The final internal friction angle was near that of the residual angle of friction determined by Skempton (1964). Bishop states that in the higher pressure range the peak of the stress-strain curve is not associated with marked dilatancy as in cohesionless soils; though at lower stresses dilatancy occurs. This probably indicates a greater tendency at high stresses for particle aggregates to break down during shear.

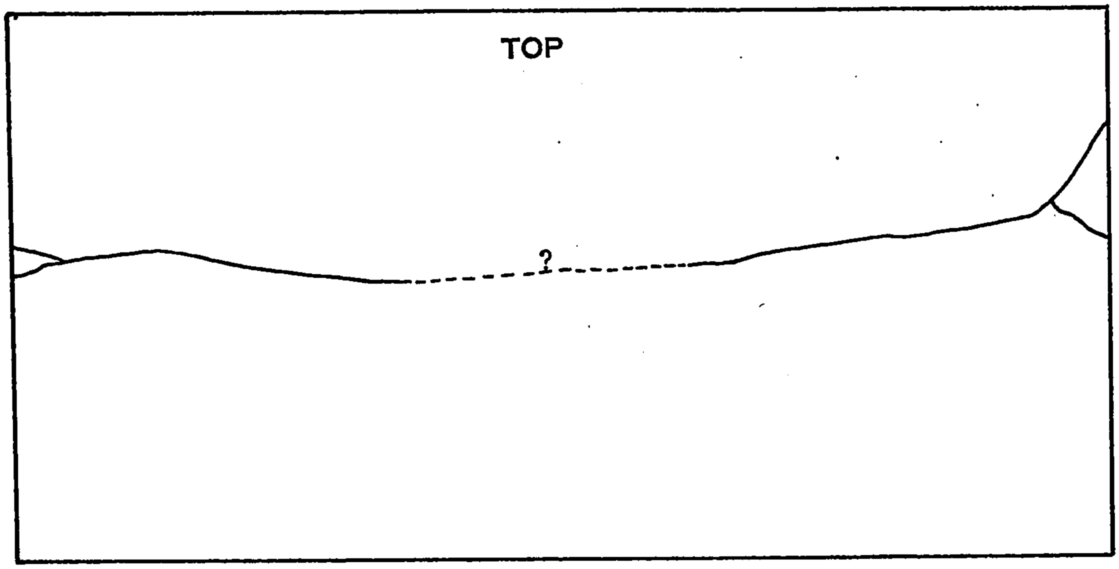
Tests on Cretaceous clay shale by Sinclair and Brooker (1967) indicate a curved envelope with a peak shear angle at high stresses equal to the residual shear angle. The stress-strain curves do not appear to indicate a specific change from a brittle to more ductile failure. Positive pore pressures were measured in all cases with increasing confining pressure. Dilatancy effects were not indicated at low stresses but, of course, the actual conditions existing along the failure surface were unknown. The envelopes are curved up to approximately 150 psi and at higher stresses the residual angle is duplicated. The preconsolidation load of these materials has been estimated to be in excess of 800 psi.

Patton (1966) in his application of change in mode of failure to rocks noted that the curvilinear envelope was typical for such brittle materials as rock and concrete. He concluded that because examples were not found for curved envelopes on soft shales and clay that his concept did not apply. His conclusion of applicability of the change in mode of failure would likely be changed if the above two references by Bishop et al. (1965) and Sinclair and Brooker (1967) had been available to him.

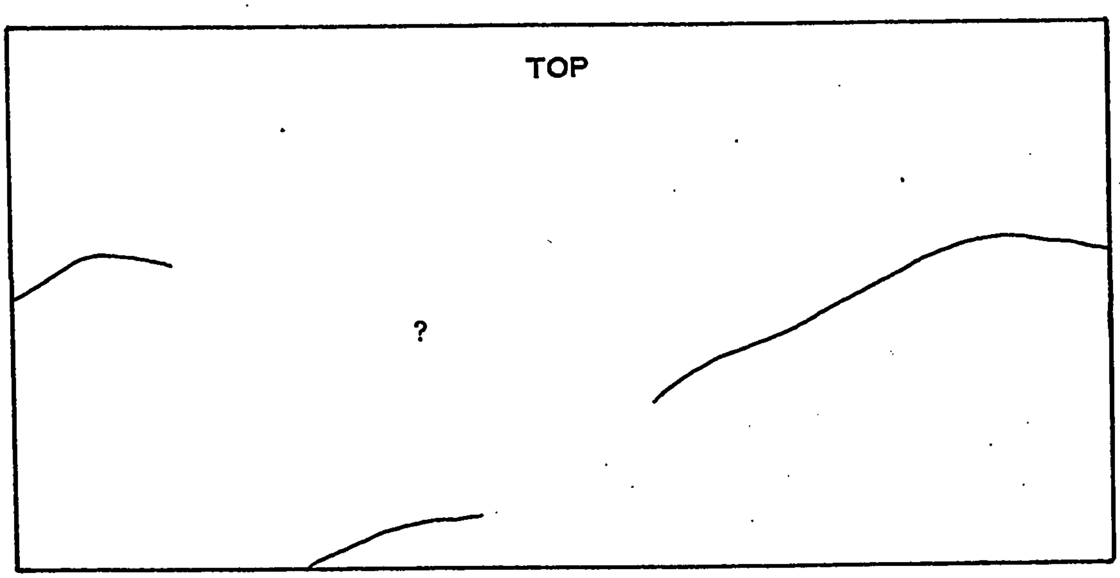
There is definite evidence that a change in mode of failure is at least partially responsible for the decrease in angle of internal friction as the normal stress increases. Many dense materials such as preconsolidated clays and clay shales undergo initial expansion upon application of shear stress. This dilatant effect has been postulated by Bishop (1966) as an explanation for the curvilinear envelopes. It is likely that more than one phenomenon is involved.

In order to determine the irregularities which are developed, two thin sections were made of Dunvegan clay sheared to the peak stress at normal stresses of 37 psi and 237 psi. Because the failure surfaces were difficult to define sketches were obtained from a microscopic study of the thin sections. These sketches are shown in Figure IV-9. Generally, the failure plane at 237 psi was better defined and was less irregular than the one at 37 psi. The failure plane at 37 psi was only defined near the edges of the sample and appeared to be widespread along paths of least resistance.

This investigation and the literature review indicate that irregularities result in a possible change in mode of failure dependent upon the normal stress level. This subject will prove worthy



[A] FAILURE PLANE FOR NORMAL STRESS OF 237 PSI
[ENLARGED 2.6 TIMES]



[B] FAILURE PLANE FOR NORMAL STRESS AT 37 PSI
[ENLARGED 2.6 TIMES]

FIGURE IV–9 FAILURE PLANES FOR DUNVEGAN CLAY AT TWO NORMAL STRESSES of future research.

Because the higher residual strength of undisturbed samples as compared to pre-cut samples is attributed to the presence of irregularities, a decrease in residual angle of friction would be expected. That is, as the normal stress increases the magnitude of the irregularities decreases and the residual envelope should become curvilinear. This phenomenon was not observed in this study, but Kenney(1967) reports that for layer lattice soils a definite decrease in the residual angle of shearing resistance was noted. He did not provide an explanation but it may have far reaching implications. Shouldice (1963) reports evidence of sliding along a bentonitic seam inclined at 2° to the horizontal and beneath 1100 feet of overburden. A decrease in angle of internal friction with increasing normal stress would account for this action.

4.3 DIRECT SHEAR TESTING TECHNIQUE

The direct shear reversing technique (Skempton, 1964) was utilized for determination of the residual and peak strength parameters. The reversing technique does not simulate the conditions in an actual slide where displacement is in one direction. Nevertheless, the test allows the peak shear strength to drop to a residual value. Worthy of research would be a comparison between the reversing direct shear test and a ring shear test where displacement is in one direction and area does not change.

In this study a displacement of .12 inches was allowed in the forward and reverse direction. This relatively small displacement of the 2 inch diameter shear box produced only a small change in area. If large displacements are allowed to occur, the area correction becomes large and the magnitude of the stresses is questionable. The average area of 4.66 sq. in. was arbitarily employed in calculating both the normal and shear stresses. This area is the average of the initial area and that when the shear box has undergone .12 inches displacement.

One disadvantage of using a small displacement arises when determining the peak parameters. The soils investigated in this study do not exhibit a brittle failure and consequently the peak stress becomes difficult to define as the shear stress increases slightly up to maximum displacement.

A phenomenon which occurred for several residual tests was a different shear stress-displacement curve for the forward and reverse directions. Although the reason for this is unknown, various explanations may be postulated. Different failure plane irregularities may develop in the two directions. According to Patton (1966) because irregularities have different inclinations in two directions the strength will depend upon the shear direction. Bending in the yoke, which transmits the load to the load cell may result in different shear stress readings in the two directions. Also, the zero readings of the load cell changes during the test period. In some cases a compressive or tensile reading remains when the load is removed. This may be due to static electricity or calibration changes in the load cell. It should be noted that the calibration of the load cell was checked several times during the 9 month testing program and no changes were disclosed. Therefore, because the exact reason for the two displacement curves is unknown the average was utilized. Perhaps the average is justified on the basis that the two directions gave the greatest difference when high normal loads were applied. If this difference is a result of greater bending stresses in the loading yoke a machine error is apparent.

The direction of shear which gave the highest residual strength was variable and did not depend upon the initial direction of shear displacement.

Tests performed on Ottawa sand at various normal loads resulted in higher readings in the tension direction. At normal loads of 104 psi and 237 psi the tension direction was higher than the compression by 5 per cent and 8 per cent, respectively. Apparent cohesion was not found with the Ottawa sand. Therefore, machine error is disregarded as a cause of the residual cohesion found for the intact clay shale samples.

During the entire testing program vertical LVDT readings were obtained which indicated without exception, that downward vertical movement was present. This movement was greater than that during primary consolidation and is attributed to soil loss during shear. The vertical LVDT placed at the centre of the loading yoke generally disclosed a slight upward movement as the shear box approached its maximum movement in each direction. This is a result of soil accumulation near the edges of the box. Although relative vertical movements occurred along each cycle, the loading head was lower after each cycle. The vertical displacements were merely caused by the test procedure and were not indicative of the properties of the soil.

4.4 EFFECT OF STRAIN RATE ON STRENGTH

Both Skempton (1964) and Kenney (1967) have stated that strain rate has little effect upon the residual strength of both natural soils and minerals using the reverse shear box procedure. DeBeer (1967), contrary to Skempton and Kenney, observed a tendency for tests at higher rates of deformation to give lower residual strengths and attributed this to the reforming of bonds at lower rates of deformation. DeBeer used a torsion ring apparatus and found for Boom clay, a stiff fissured clay, that the residual angle of friction was 24° 20' at a strain rate of .4 mm. per hour (2.4 in. per day) and 19° 20' at a strain rate of 2.1 mm. per hour (12 in. per day). These discrepancies may be a question of testing technique. Research comparing alternative methods is of immediate interest.

Horn and Deere (1962) report that increases in strain rate for mica from .7 to 6.0 in. per min. resulted in a 12 per cent increase in the frictional resistance under oven-dried/ air-equilibrated conditions and 28 per cent under saturated conditions. For massive minerals no effect of strain rate on strength was noted. Their study indicated that the mineralogical content is significant when considering strain rate effects.

Most of the residual strength tests in this study were performed at a strain rate of 2.76 in. per day (2 hours per cycle).

Figure IV-10 shows that for Dunvegan clay at a normal stress of 104 psi an increase in strength of 8 per cent was obtained when the strain rate was increased from 1.38 in. per day (4 hours per cycle) to 34.56 in. per day (10 minutes per cycle). The higher strain rate and shear strength is consistent with the normal phenomenon for porous materials but is inconsistent with the results found by DeBeer. Consequently, an increase in strain rate of 25 times resulted in a 8 per cent higher residual strength. Another test performed at a strain rate of .11 in. per day did not indicate a significant decrease in shear strength from the 1.38 in. per day rate. In order to reach the residual strength some 40 cycles were often required. Commonly an approach was used in which several cycles were obtained at rapid rates as high as 69.1 in per day (5 minutes per cycle) and then the slower rate of 2.76 in. per day (2 hours per cycle) was used to measure the lower residual strength.

4.5 CONSOLIDATION TEST RESULTS

Consolidation tests were principally performed on the slide materials to determine the range of coefficients of permeability in order to estimate peak strain rates. The time for 100 per cent consolidation varied from 8 minutes for the Dunvegan clay to 1000 minutes for the bentonitic clay shale at pressure ranges of 2 to 4 tons per sq. ft. The coefficients of permeability for the Dunvegan clay, Peace River pre-till clay and Lesueur bentonitic clay shale were 1.67×10^{-7}, 1.66×10^{-8}, and 1.58×10^{-9} cm/sec., respectively.

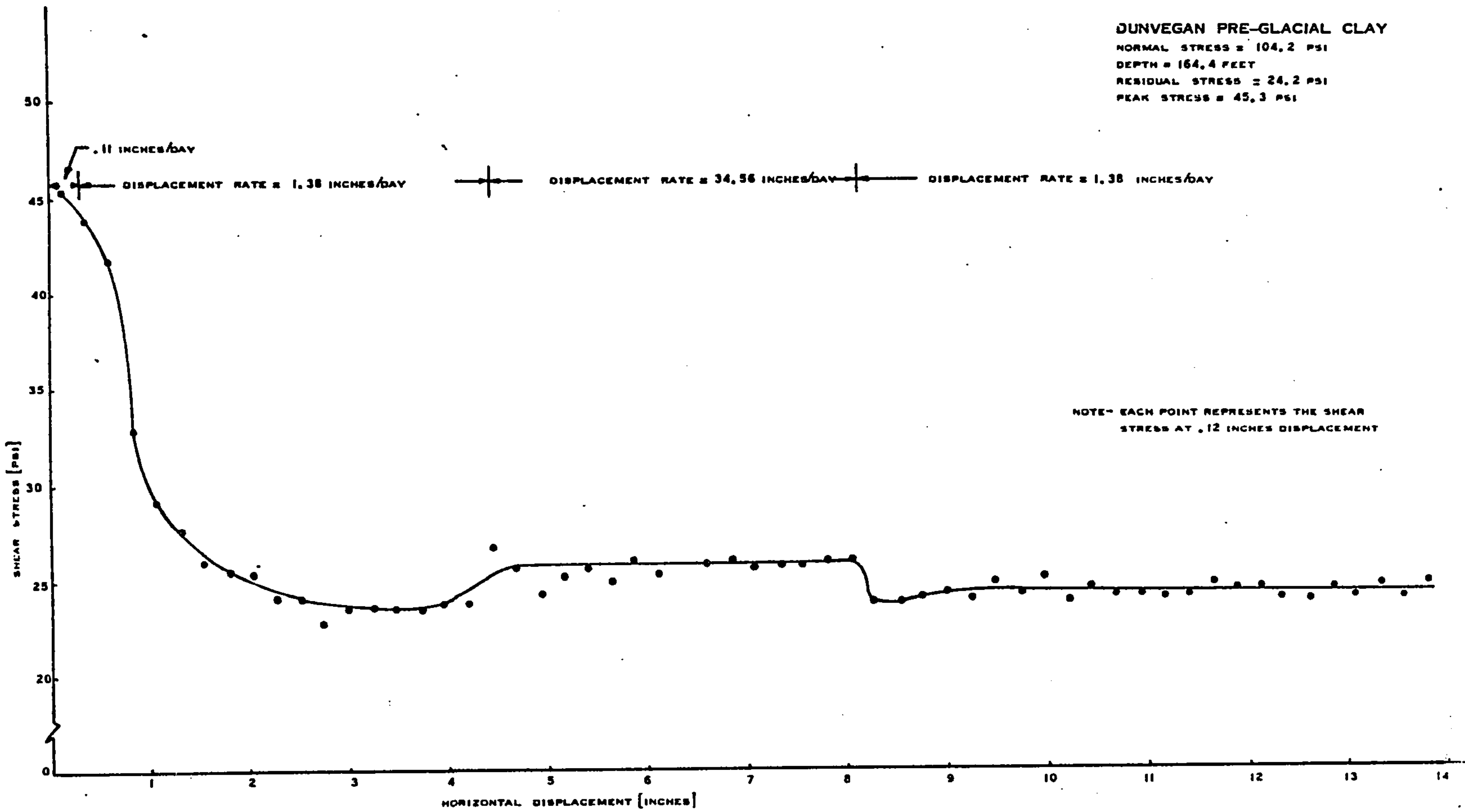


FIGURE IV-10 SHEAR STRESS VERSUS HORIZONTAL DISPLACEMENT

A practical rate of strain consistent with the time required for 100 per cent consolidation was selected. A rate of .111 inches per day was used to shear all soils to the peak strength. This rate is also similar to the range .039 to .195 inches per day reported by Sinclair and Brooker (1967) on similar soils.

The bentonitic soils with low permeability necessitate the use of a piezometer possessing a rapid response time in order to obtain meaningful readings.

The preconsolidation pressures indicated by the consolidation tests are questionable. Pressures from 2 to 12 tons per sq. ft. were indicated but the curvature of the e log p curves was difficult to interpret.

CHAPTER V

DESCRIPTION AND ANALYSIS OF SLIDES IN THE PEACE RIVER AREA

5.1 BEDROCK GEOLOGY OF PEACE RIVER AREA

The Peace River district is underlain by a series of marine and non-marine sandstones and shales of Cretaceous age.

Surficial deposits consisting of till, sands, gravels and lacustrine silts and clays cover most of the bedrock with thicknesses ranging from a few to several hundred feet. Generally, the surficial deposits thin towards upland areas and are thickest in the valleys (Jones, 1966).

Most of the Cretaceous strata are quite soft and tend to slump quite readily, the disturbed and slumped material commonly obscuring true bedrock crop outs; however, many exposures of bedrock are present along the river valleys of the area and along the upland margins.

The Cretaceous strata of most interest in the area are those cropping out near river levels including: the Dunvegan Formation, Shaftesbury Formation and Peace River Formation. The Shaftesbury and Peace River Formations belong to the much larger Fort St. John Group. The Fort St. John shale was deposited as clay and silty clay during Lower Cretaceous times in an alluvial plain built out into a shallow northern sea which spread southward from the Arctic

Ocean to Southern Alberta. At a later date, during Upper Cretaceous times this northern sea spread and joined with a northward extension of the Gulf of Mexico to form a great interior sea, dividing the continent. Sediments continued to be deposited until Upper Cretaceous times when the marginal alluvial plains rose above sea level. Several thousand feet of sediments accumulated above the clay and silty clay of the Fort St. John Group compacting them into an extremely dense material of low water content. Cementation did not occur so that the effects of compaction resulting from the intense overburden loading are not permanent. The shale disintegrates rapidly into a clay when the overburden loading is reduced and the shale has access to water.

The lowermost Cretaceous strata exposed along the Peace River are the sandstones and shales of the Peace River Formation which crop out along the Peace River near the water level from just South of Peace River town northward. This unit is recognizable in the subsurface from electric logs and extends eastward from the Alberta-British Columbia boundary to Lesser Slave Lake. The upper member of the Peace River Formation is composed mainly of sandstone. (Alberta Society of Petroleum Geologists, 1964)

The Shaftesbury Formation is composed of a series of marine shale and silty shale beds that overlie the Peace River Formation and underlie the Dunvegan Formation. Rocks of the Shaftesbury Formation crop out mainly along the Smoky and Peace Rivers. The Shaftesbury Formation is divided into two members, the lower member comprised mainly of thinly bedded fissile shales with at least three fish-scale zones in the upper part and the upper member consisting of silty and sandy shales relatively free of fish scales. The boundary between the Upper and Lower Cretaceous bedrock is commonly taken as the base of the fish-scale sand bed. Many geologists regard the top of the Shaftesbury Formation as the division between Upper and Lower Cretaceous. The uppermost part of the formation becomes silty to sandy with minute carbonaceous fragments, and is transitional into the overlying non-marine Dunvegan sandstone.

The Dunvegan Formation consists mainly of fresh-water sandstones and shales cropping out prominently on the north side of the Peace River at Dunvegan Crossing. The sandstones and shales of the Dunvegan Formation crop out along both sides of the Peace River from the Alberta-British Columbia border downstream to beyond Dunvegan Crossing and also on the Smoky River as far upstream as Watino. The strata consist of an alternating series of sandstones and shales with all gradations between these two types of strata. Some of the beds are over fifty feet thick. Sandstones are more prevalent in the middle part of the formation while shales predominate in the upper and lower parts. The sandstones are usually fine-grained, crossbedded and massive, light grey to buff in color, and varied hardness. The harder phases are usually confined to the thicker massive sandstones and are dark green on fresh surfaces. During the erosion of these sandstones along the valley crop outs, the harder phases often weather out as large spherical masses up to ten feet in diameter, a feature common to Upper Cretaceous sandstones in many parts of the foothills of Alberta. The large blocks are not concretions but appear to be centres which have undergone greater cementation. The shale phases vary from siltstones to thin bedded dark grey shales.

The Dunvegan sandstones in Alberta represent the outer margins of a Cretaceous delta built out as a broad apron skirting an upland centering on the southern Mackenzie mountains and the Cassiar mountains of Northern British Columbia.

5.2 RECENT ADVANCES IN FIELD INVESTIGATION OF SLIDES

The case histories cited in this thesis demonstrate previous unsuccessful elucidation of slide failures in clay shales. The unrelenting utilization of peak strengths has been justified by erroneously high piezometer levels. The inability to recognize perched water tables has been a major contributor to the acceptance of high piezometric levels. The use of piezometers with long response times has proven unsatisfactory in many highly impermeable soils, particularly in the Western Canadian landslides (Brooker and Lindberg, 1965). The use of the pressure transducer as the major component of a piezometer has provided a short response time for use in clay shales. The transducer piezometer as developed jointly by the University of Alberta and the Geological Survey of Canada (Scott and Brooker, 1968) has proven to provide reliable data on piezometric levels. The obtainability of reliable field piezometric data now allows the employment of effective stress concepts with greater accuracy.

Another major adversity has been the inability to obtain and retain undisturbed samples of clay shales and preconsolidated clays. The use of conventional drive and push sampling procedures in clay shales had been unsatisfactory due to the hardness of the soils. The standard rock coring methods have also been more or less unsuccessful because of extensive disturbance or loss of samples. The employment of a Pitcher sampler developed by the Geological Survey of Canada has provided excellent samples for laboratory study. Fundamentally, the sampler comprises a cutting bit attached to an outer rotating barrel which follows a variable distance behind a sample tube. The sample tube is advanced by application of hydraulic pressure up to 600 psi. Construction and operation of the Pitcher sampler is given by the Department of Civil Engineering, University of Alberta (1968).

The above two innovations have increased the accuracy of collecting field data. In this study both the transducer piezometer and the Pitcher sampler were utilized.

5.3 TAYLOR B.C. SLIDE

(a) Detailed Geology of Taylor Flat B.C.

Taylor Flat is a semi-circular high level gravel terrace about 200 feet above the Peace River (Figure V-1). It was formed by erosion by the Peace River subsequent to the last ice age. The uplands about 2 miles from the north anchorage of the Alaska Highway Bridge extend several hundred feet above Taylor Flat. At the time the bridge was built in 1942 the main stream of the Peace River was flowing along the south shore but by 1949 it had shifted so it was attacking the north bank. The slope from the south bank of the river to the uplands is extremely flat and irregular. The north bank of the river at the location of the bridge site is generally steep with slopes of about 1 vertical to 1.5 horizontal with the high level terrace at about elevation 1510 and the river at about elevation 1320. However, a small low level terrace about 30 feet above the river interrupts this slope at Taylor Flat (Mathews, 1963).

The cropping out of the shale exposed along the north bank of the river belongs to the Shaftesbury Formation of Lower Cretaceous age (McLearn and Kindle, 1950). This shale extends from several thousand feet below the river to about 100 feet above it and is overlain by up to 100 feet of sand and gravel deposited by the river. In the upland areas the shale is overlain by the Dunvegan Formation of Upper Cretaceous age, which is in turn capped by glacial materials.

Prior to Pleistocene time a river occupied the approximate channel of the present Peace River and the old river valley was wide with fairly gentle slopes to the upland surface. During Pleistocene time this old valley was filled with glacial drift and glacial lake deposits. Following the disappearance of the ice, the present river began to excavate the glacial materials and today has succeeded over parts of its course in establishing itself more or less in its old channel.

(b) Soil and Bridge Movements Prior to Landslide October, 1957

The data previous to the present study was obtained from reports by Hardy (1963) and (1966), Moran et al.(1958) and the Master of Science thesis by Brooker (1958).

The bridge at Taylor Flat was completed in 1943 as part of the Alaska Highway System.

In 1949 the north pier was underpinned, having suffered severe scour by river erosion. In 1952 it was suspected that the bridge had lost some of its camber and there was evidence that the north anchor had moved horizontally by about 3 inches. In September, 1957 there was visual evidence of further movement of the anchor block and a check survey was run in early October. This indicated a shift southward of 1.08 feet and a definite shift westward of 0.25 feet of the top of the north cable bent from 1952 to 1957. During the same period the north anchor block showed a tilting with a possible horizontal movement southward of 1.6 feet. From September, 1957 to October 8, 1957, a total settlement of 18 inches occurred in the approach fill to the north anchorage. The north span of the bridge collapsed at 12:40 P.M., October 16, 1957.

Surface run-off from about two square miles north of the bridge collected in the road ditches and discharged over the bank on either side of the anchor block and ran freely into holes at the corners, this indicated the possible existence of a void. The total precipitation during 1957 was about 80% above the average for the 35 years for which records were available and about 70% of this occurred during the summer months in 1957. This would have the effect of further aggravating the stability conditions.

The landslide extended approximately 450 feet east of the centerline of the bridge and 630 feet west. The maximum distance from the landward edge of the slide to the waters edge of the slide was approximately 450 feet.

(c) Previous Field Investigations and Subsoil Stratigraphy

A zone of pulverized or fractured clayey shale was found at about elevation 1290. The water contents in this zone were generally about 25 per cent and were substantially higher than the water contents of the immediately overlying and underlying shale. It was assumed that this zone represented the base of sliding of the landslide which occurred in October, 1957, although only a few reliable observations were made (Hardy, 1966).

Of particular significance is the fact that previous to the erection of the bridge in 1943 a spring existed just east of the location of the anchor block. Zones of seepage had been mapped 200 feet east of the north anchorage at elevation 1440 and 500 feet west at elevation 1400. Consequently, strong evidence appeared to exist that a ground water table was near the surface at the north anchor block. This water was probably seepage from the perched water table in the gravel on the high terrace.

The soils from the top of the bank on the high terrace, elevation 1510 to about elevation 1420, consist of gravel and sand with traces of clay. Beneath approximately elevation 1420 to depths far beyond river bottom, the bedrock consists of shales and

and soils derived from weathered shale. In some instances the upper part of the shale was weathered into a soft clay. The low terrace about 30 feet above the river, at about elevation 1350, generally consists of a variable thickness of surface gravel and sand overlying shale and soil derived from the weathering shale. The shale below approximately elevation 1270 to 1280, about 50 to 80 feet below the low terrace, is relatively unweathered. It is dark grey to black, horizontally bedded, hard and has natural water contents of 3 to 9 per cent. Certain weathered zones have reverted to a soft clay and natural water contents are as high as 20 to 30 percent.

(d) Previous Laboratory Tests

The results of Atterberg liquid and plastic limit tests indicated that the shale and clay derived from weathering of the shale were inorganic, silty and low plastic. The liquid limits varied from 26 to 48 per cent and plasticity indices from 8 to 24. The results of mechanical analysis indicated that the percentage of clay sizes was low and that the shale consisted of 40 to 70 per cent silt sizes with some sand.

Slaking tests performed on the shale produced extremely rapid decomposition which indicated that the shale was formed by compaction without cementation under the weight of overlying sediments. Some shale samples exhibited more weathering than others.

Consolidation tests indicated that continued weathering affected the structure of the shale and the indicated preconsolidation load was progressively reduced. Compressive indices of .20 were found implying that the shale was only slightly compressible.

The shear strength of the clayey shale was determined by unconfined compression and triaxial compression shear tests. The triaxial compression tests included consolidated-quick tests, quick tests, and slow tests. Pore pressures were measured in two of the consolidated quick tests to give effective stress results along with the slow tests. It was found that the shear strength of samples having natural water contents less than 10 per cent was so high that such materials were unlikely to have participated in the landslide of 1957 (Moran et al. 1958). The compressive strength of samples having water contents greater than 10 per cent was variable, ranging from values as low as .5 tons per sq. foot to a maximum of 5.6 tons per sq. foot. The degree of weathering and fissuring of individual samples may have produced the strength data scatter.

The peak angle of internal friction was concluded to be 37 degrees with zero cohesion (Moran et al. 1958). The degree of saturation for the many samples varied from 74 to 95 per cent and moisture content varied from 9.8 to 33 per cent.

Three triaxial tests performed by Brooker (1958) indicated an effective peak angle of internal friction of 30.6 degrees and cohesion of 0.75 tons per square foot.

(e) Previous Stability Analyses

A Phi-circle analysis on the section along the bridge center line was performed by Brooker (1958) and the water level was assumed to fill a nearly vertical crack behind the anchor block. The analysis indicated that an internal angle of friction of 15.6 degrees was required for a factor of safety of 1.0. A sliding wedge analysis was concluded to best fit the actual physical conditions in the field. From a wedge analysis and from the assumption that the ground water level was at the level of the base of the anchor block and as indicated by test holes in the lower slide area, a friction angle of 11.1 degrees was found to be necessary for a factor of safety of 1.0. If the ground water level was assumed at the top of the anchor block a friction angle of 18.2^{o} was required for a factor of safety of 1.0. If it was assumed that the angle of internal friction was 30.6 degrees and water table at the base of the anchor, an excess hydrostatic head of 76.4 feet above static ground water level was required for a factor of safety of 1.0. If the ground water level was assumed at the top of the anchor, an excess head of 41.5 feet was required for instability.

Because the geometry of the slide would not allow the high excess hydrostatic pressures to exist, it must be concluded that the actual angle of internal friction which was mobilized was much less than 30.6 degrees.

Two limiting types of stability analyses were performed by Moran _et al_. (1958). They included the $\phi = 0$ analysis where the shear strength was assumed independent of the stress on the failure plane and the C = 0 analysis in which the soils were assumed to be frictional materials and any cohesion had been destroyed during straining. The $\phi = 0$ analysis indicated that the average shear strength, or cohesion of the shale at the time of failure probably was about 1.7 kips per square feet for the most likely ground water conditions existing at the time of failure; example, elevation 1338 at the anchorage and elevation 1320 at the river's edge. This corresponds to a compressive strength of 3.4 kips per square foot. The C = 0 analysis was performed assuming a slow or effective angle of internal friction of 37 degrees. The results indicated that the water pressure on a failure plane at elevation 1290 at the time of the landslide corresponded to an average head at elevation 1362 and 1381, for assumed ground water levels at the anchorage of 1423 and 1338, respectively. Of course, both average water table assumptions were above the ground surface.

The results of the $\phi = 0$ and C = 0 analyses indicated that the laboratory strength data available was incompatible with field behavior and gave a factor of safety greater than one for the slide.

A wedge analysis performed by Sinclair *et al*.(1966), similar to that reported by Brooker (1958), produced a factor of safety of 1.0 using an assumed residual angle of friction of 22 degrees and placing the piezometric water level at the bottom of the north edge of the anchor block.

(f) Study Field Investigation of Taylor Slide

Water levels measured during the summer of 1967 indicated levels generally consistent with the bottom of the gravel layer. These levels were similar to those reported at the time of the slide in 1957.

In order to recover undisturbed samples of the clay shale, it was necessary to penetrate 30 feet of gravel and sand. It was found impossible to place casing by conventional means using a Failing 1500 rotary drill. Consequently, the cost of obtaining samples proved to be prohibitive. It was fortunate that a fresh cut had been made in the clay shale slope. Although the drilling rig could not be advanced to the area, it was possible to obtain a partially weathered sample of clay shale. This block sample was utilized to obtain residual strength envelopes for the "undisturbed" and remolded cases.

(g) Computer Analysis of Taylor Slide

The pertinent data essential for analysis of the slide is summarized in the following paragraphs:

(i) The failure plane was located horizontal at elevation 1290 in the Shaftesbury clay shale. Evidence for the failure plane was based on a zone of pulverized shale.

(ii) The piezometric head was based on water levels in test holes drilled following the movement in 1957 and was found to be located at the bottom of the lower terrace gravel layer. The river level existed at 1320 and placing the water table at the base of the gravel at the north edge of the anchor resulted in a piezometric level near elevation 1340. It is probably justified to employ a piezometric head slightly above that indicated by post-slide water levels because of the volume of runoff which entered the slide area prior to the slide.

(iii) The undsiturbed residual angle of shearing resistance was found to be 16.1 degrees with residual cohesion of 1.6 psi and the remolded residual angle was 15.2 degrees with zero cohesion.

The peak angle of shearing resistance was previously found to vary between 30 and 37 degrees. It will be shown that peak parameters were not applicable.

(iv) The selection of a scarp angle was found to be arbitary because of the limited information relative to its location. Evidence was given that a vertical crack appeared behind the anchor block immediately prior to the landslide.

Various scarp surfaces were analyzed to disclose the sensitivity of the analysis to the scarp angle.

Table V-1 summarizes the factors of safety obtained from the computer analyses for the most significant surfaces.

The results of the computer analysis indicated that a residual angle of friction of 16^{o} on the scarp and horizontal plane resulted in a factor of safety of 1.012 when a scarp angle of 65 degrees changing to 45 degrees was assumed as well as a water table 20 feet above the 1957 level at the anchor block (Table V-1). If the failure plane possessed a slope from elevation 1300 at the anchor block to 1280 at the toe a factor of safety of .988 was obtained. Consequently, only slight changes in the factor of safety resulted when a sloping failure plane was utilized.

When the residual cohesion of 200 pounds per square foot for the undisturbed specimens was applied, a factor of safety of 1.158 resulted. With a drop in the failure surface of 20 feet, a void for 80 feet of the scarp length and developed cohesion, a factor of safety of 1.033 was found. Therefore, it has been shown that slight variations in field interpretation can justify the use of the residual cohesion. It is likely that some cohesion was developed but perhaps less than 200 pounds per square foot.

A residual factor of 1.0 is concluded to be applicable for this slide at the time of failure.

If the peak angle of internal friction was employed, a factor of safety of 3.265 was indicated for the limiting equilibrium state.

The analysis shows that the factor of safety is insensitive to a change in f(x), the factor relating the X and E forces.

Recorded movements of the bridge over a 20 year period indicated that creep was occurring in the slope before the 1957 failure. This creep was taking place under the residual stress. The change in ground profile subsequent to the slide was only slight. The relatively small slope change from pre-slide to post-slide conditions is indicative of a slide exhibiting a residual factor of 1.0. In other words, a strength decrease does not occur as the large movements of the slide take place and only a slight change in slope is necessary to maintain equilibrium.

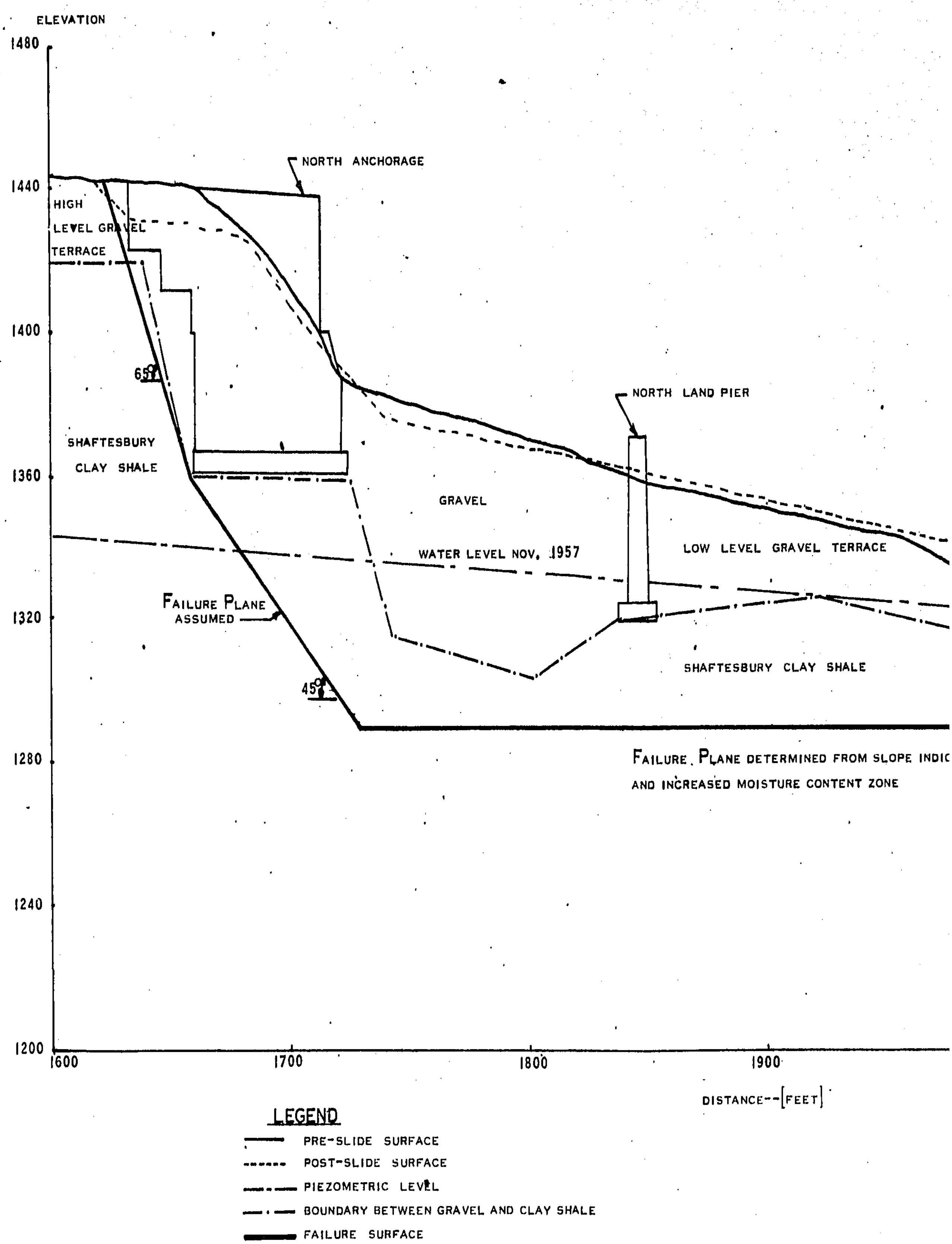
The fundamental internal cause of the slide was the decrease in strength of the clay shale from the peak to the residual. Two external causes for the slide were the river toe erosion and the above average precipitation. It is postulated by the author that the immediate cause of the slide was an increase in pore pressure which transformed a creep movement into a disastrous slide failure.

TABLE V-1

RESULTS OF TAYLOR SLIDE ANALYSES

Surface Number	Factor of Safety	ϕ' Failure Plane	Scarp Slope	ϕ' Scarp	$f_{(x)}$ Scarp	$f_{(x)}$ Toe	Remarks
1	1.703	16°	65° to 45°	30°	1	1	1957 water table
2	3.265	ϕ_p	"	ϕ_p	1	1	1957 water table
3	1.097	16°	"	16°	1	1	1957 water table
4	1.175	16°	"	16°	1	1	1957 ϕ_p on passive block
5	1.092	16°	55°	16°	1	1	1957 ϕ_p on passive block
6	1.661	16°	75°	16°	1	1	1957 ϕ_p on passive block
7	1.192	16°	55°	16°	1	1	1957 ϕ_p on passive block
8	.697	16°	65° to 45°	16°	1	1	water table at surface
9	1.012	16°	"	16°	1	1	water table 20' above 1957 level
10	1.163	16°	60' vert 45°	60' ϕ=0 16°	1	1	1957 water table
11	1.088	15°	60' vert 45°	60' ϕ=0 15°	1	1	1957 water table
12	1.027	15°	65° to 45°	15°	1	1	1957 water table
13	1.025	15°	"	15°	.5	.5	1957 water table
14	1.025	15°	"	15°	.2	.5	1957 water table
15	1.064	15°	"	15°	3	1	1957 water table
16	non converge	15°	"	15°	.1	.1	1957 water table
17	.988	16°	"	16°	1	1	water table 20' above 1957 level - slope 1300 to 1280
18	1.158	c=200 ϕ=16°	"	c=200 ϕ=16°	1	1	water table 20' above 1957 level - horizontal
19	1.033	"	"	"	1	1	water table 20' above 1957 level - slope 1300 to 1280
5(a)	1.10	Same as 5					Wedge analysis
10(a)	1.15	Same as 10					Wedge analysis

Note: Cohesion in psf



STRATIGRAPHIC PROFILE OF TAYLOR SLIDE ALONG

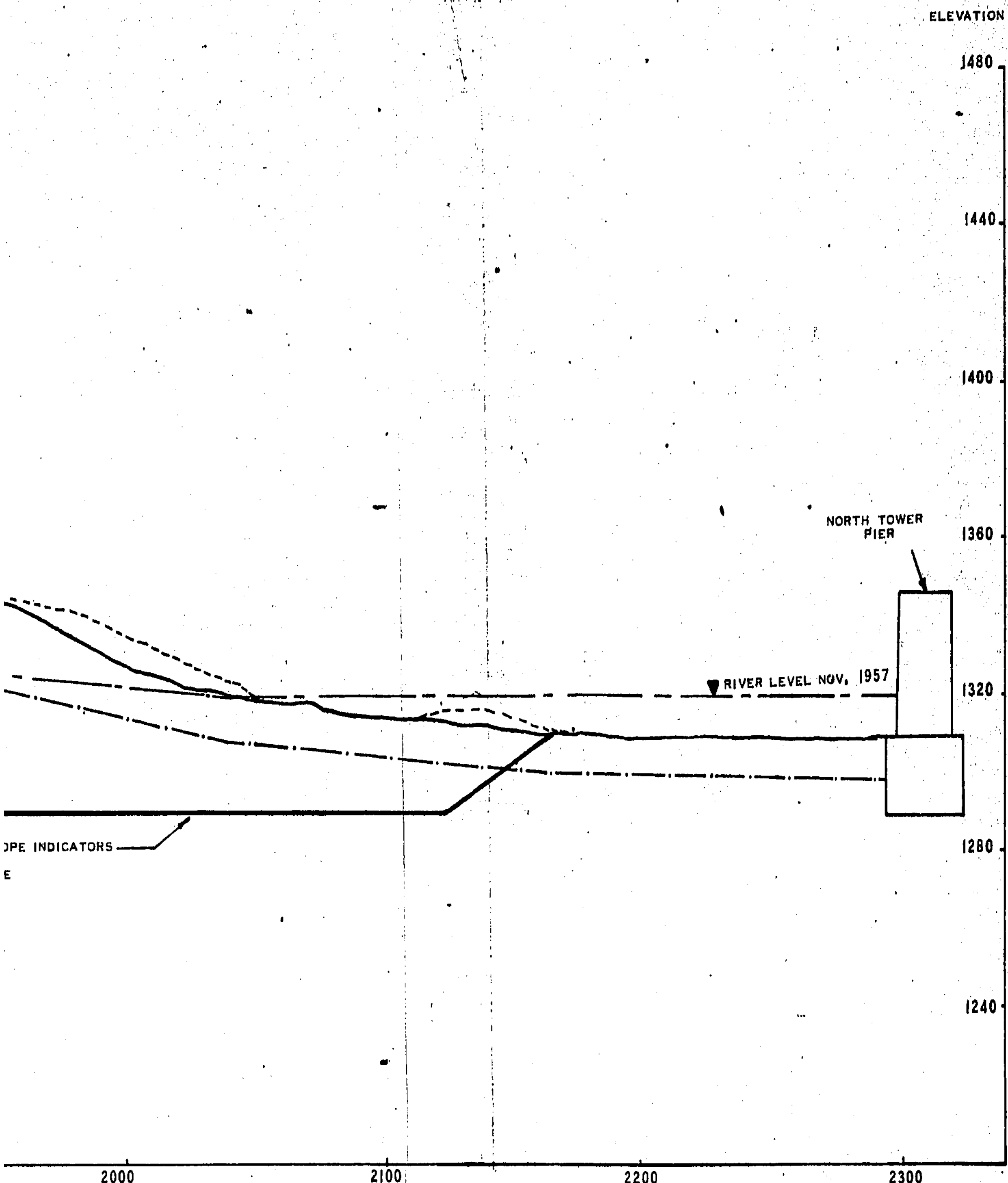


FIGURE V-I

ALONG THE ALASKA HIGHWAY BRIDGE

5.4 DUNVEGAN SLIDE

(a) Geology of Dunvegan Slide Area

The Dunvegan slide occurred along the slopes of a pre-glacial valley in which numerous old landslides are in evidence. The slide material was composed of a dark grey silty pre-till lake clay with sand and silt seams. This deposit was "plastered" on the bedrock walls of the pre-glacial valley. The term "pre-till" in this thesis is used when referring to soils deposited below glacial till and which have been subjected to glacial compaction. In geological terms, the material is classed as "fill" over the underlying Cretaceous bedrock. At this site the bedrock is made up of the Dunvegan Formation. Numerous crop outs of the Dunvegan formation are present at the mouth of the Dunvegan creek along the Peace River.

(b) History and Movement of the Dunvegan Slide

Most of the data obtained on the Dunvegan slide was from the Alberta Research Council files and a publication by Hardy et al. (1962).

The Province of Alberta main Highway No. 2 connecting the City of Grande Prairie and the Town of Peace River in Northwestern Alberta crosses the Peace River at Dunvegan. Between 1958 and 1960, a new suspension bridge was built at the crossing where the valley of the Peace River is about 700 feet deep. In order to improve the grades on the highway out of the valley on the south side a new location was selected along the Dunvegan valley. This is a steep "coulee" type of valley extending back about seven miles from the

Peace River and in the bottom of which flows Dunvegan Creek. It was not possible to locate a satisfactory alignment for a road out of this valley from the new bridge without crossing old slide areas.

At the site of the slide, the road embankment required a fill with a maximum depth of approximately 100 feet. The construction of this embankment was commenced in the fall of 1958 and a maximum depth of fill of 70 feet was placed before freeze-up. Construction was continued about May 1, 1959.

On May 19, 1959, cracks were noticed on the ground surface below the new embankment. These became more extensive during the following four days and on May 23, a rapid movement developed. The top of the slide cut through the top of the new road embankment against the natural ground and initially extended to a toe in a drainage trench about 12 feet deep being constructed some 900 feet to the west (Figure V-2). Within a few hours the instability had extended down the full slope of the valley to the creek bed, a distance of about 1600 feet west of the road embankment.

At the crown of the slide, the road grade dropped vertically about 70 feet. At the north flank of the slide area there was a vertical displacement of some 30 feet and the toe of the slide filled the creek to a depth of about 30 feet. The surface within the slide area was broken by a network of cracks as much as 3 feet wide and extending 20 feet deep. The slide covered an area of about 50 acres and involved a movement of some four to six million cubic yards of soil.

(c) Previous Field Investigation

Previous to construction of the new highway, some 60 test holes were drilled extending to depths of from 50 to 150 feet. These borings showed the soil to be medium to highly plastic dark brown and dark grey inorganic clays interbedded with silt and silty sand seams. Of 30 test holes drilled in the immediate slide area, free ground water was observed in 15. These ground water levels ranged between 12 to 70 feet below the ground surface. There was no consistency found between water levels in adjacent test holes. During observations preceding the slide, no significant artesian pressures were recorded.

Standard penetration tests indicated blow counts per foot between 30 and 150. Generally, the moisture contents were very near the plastic limit (Table V-2).

Following the slide, 60 additional test holes were drilled. These holes were located adjacent to the slide area and on a proposed realignment. These post-slide test holes indicated similar soil types to those of the holes drilled before the slide.

Following the slide, a total of 12 Casagrande type piezometers was installed in and adjacent to the slide area. These were placed at depths varying from 20 to 80 feet below the surface. The majority were installed at a depth where the moisture content and limit profiles showed an increase in liquidity index. Six were placed at shallow depths in silty sand in the lower portion of the slide area. None of those placed in the pre-till clay showed measurable pore pressure. In fact, sensitivity tests on the piezometers in the preg-till clay indicated a permeability of the soil greatly in excess of that for the intact soil; Hardy et al.(1962). The loss of water was assumed to be through fissures in the clay. The six piezometers in the silty sand in the lower portion of the slide area showed some artesian pressure but the magnitudes were not sufficient to affect the stability of the slide area.

A number of the pre-slide test holes were cased with galvanized pipe or plastic tubing. Observations were made for movements in these by probing, commencing on May 1, 1959, and continuing until the day before the final rapid movement on May 19. Several of these gave positive indications of the depth of the shear surface developed in the slide area.

(d) Previous Laboratory Results and Stability Analyses

TABLE V-2

SUMMARY OF DUNVEGAN SOIL TESTS (102 samples)

	Min.	Max.	Average
Liquid Limit	45	80	50
Plastic Limit	20	30	24
Moisture Content	18	30	22
Unconfined Comp. Strength T/ft.^{2}	.87	11.8	4.5
Standard Pen. Test (blows per foot)	30	150	60

Both an infinite slope analysis and a standard slices method were performed on the slide. The best fit arc through the shear surface to the location of the toe in the initial movement had a radius of about 3,000 feet. Using this arc and the method of slices gave an average shearing stress of 1,750 psf on the shear surface. The infinite slope analysis using the average depth to the shear surface over the entire length of the slide, down to the toe at the creek gave an average shearing stress of 1,290 psf. The value of 1,750 psf was considered the more accurate by Hardy et al.(1962), since observations at the time of the failure indicated a certain amount of progressive failure to the extent that the lower portion of the slide developed after the formation of the upper toe.

If the computed average shearing stress is compared with the average shearing strength (Table V-2) from the unconfined compressive strength tests, a factor of safety of 2.6 is indicated. Therefore, it was concluded that the method of analysis using undrained unconfined compression strength data gave unrealistic results.

A set of consolidated undrained triaxial compression tests with pore pressure measurements were performed on samples from a depth of 75 feet. The natural moisture content of the samples was 30%, liquid limit 80% and the plastic limit was 29%. The effective stress parameters were $\phi = 21.5$ degrees and $C = .2$ kg./cm^2 as reported by Hardy et al.(1962). All the pore pressures which were recorded during the triaxial tests were positive.

Using an infinite slope analysis and considering fully mobilized effective stress parameters with zero pore pressure, the factor of safety was found to be 2.81. If the effective stress parameters were considered fully mobilized, then the pore pressure required for a factor of safety of unity was found to be equal to a piezometric level 96 feet above the slide surface or 29 feet above the ground surface. If the cohesion parameter is taken as zero and zero pore pressure is assumed, the factor of safety was found to be 2.50. With the assumption that the cohesion is zero, the required pore pressure to give a factor of safety of unity is equal to a piezometric level 78 feet above the slide surface or 11 feet above the ground.

The average swelling pressure obtained from the laboratory "free swell" test was 1.8 tons/sq. ft. It was postulated that the swelling pressure reduces the effective normal stress acting on the failure plane (Hardy et al. 1962). Therefore, if the swelling pressure of 1.8 tons/sq. ft. is subtracted from the effective normal stress, the factor of safety was found to be 0.97. It is noted however, that constant volume swelling tests gave swelling pressures in the order of 0.6 tons/sq.ft.

Stabilization of the Dunvegan slide was accomplished by extending a toe across the creek at the lower end of the slide with a culvert to carry the flow. The highway grade was also changed so that only a cut section was present above the slide area.

(e) Study Field Investigation of Dunvegan Slide

During the current investigation one test hole, DU1, was drilled to a depth of 160 feet from which undisturbed samples were obtained using the Pitcher sampler. The depth of the hole allowed the samples to be obtained from an elevation coincident with the failure plane near the scarp. The test hole was drilled above the scarp (Figure V-2) because extensive road access work would have been necessary to allow drilling equipment to advance on to the lower slope. The soil profile was basically similar to that reported by Hardy et al.(1962), composed principally of overconsolidated silty clay with varying thicknesses of silt and sand.

After drilling and sampling of the test hole had been completed the 1 1/2 inch casing and well point were grouted into place at the bottom of the drill hole for later installation of a transducer piezometer. Because wet drilling must be utilized with the Pitcher sampler the water level was at the surface upon completion of drilling. The day after drilling the water level had dropped 40 feet below the surface. Two weeks later when the piezometer was installed, the test hole was completely dry. This drop in water level was significant, indicating that the soil is not highly impermeable and that the water table must exist below the bottom of the test hole and consequently, below the failure plane. Piezometer readings taken over the winter of 1967-68 and summer of 1968 did not indicate a piezometric level above the bottom of the hole. Test holes drilled in 1958 which remained intact in the upper portion of the slide indicated water levels 65 feet below the surface.

The results of the piezometer above the scarp and the water levels in the test holes are consistent, the pertinent conclusion being that the water table in the upper portion of the slide is not near the surface but only slightly above the failure surface. None of the 1958-59 test holes in the lower slide area was operative although it would appear that the water table exists nearer the surface in the lower area.

(f) Computer Analysis of Dunvegan Slide

The data utilized in the analysis of this slide are summarized in the following paragraphs:

(i) The slide surface was located by probewells placed prior to the slide. These observations indicated a sliding surface approximately parallel to the surface at a depth of 75 feet in the upper 900 feet of the slide. The upper portion of the slide developed a toe in a drainage ditch which had been excavated to a depth of 12 feet. The remaining lower portion of the 1600 foot slide developed the following day. Probewells gave scanty positive evidence that the slip surface was located at approximately 70 feet below the surface in the lower zone of the slide.

(ii) Casagrande piezometers did not indicate any significant pore pressures except in the lower portion of the slide. Test hole water levels appear to give a good estimate of the water table. Test holes in the upper slide area indicated water levels approximately 60 feet below the surface. In the lower slide near Dunvegan

Creek water levels were recorded very near the surface although inconsistencies existed. The transducer piezometer installed in September 1967 did not indicate a piezometric head for the period readings were taken. Basically, the piezometric line shown in Figure V-2 is related to test hole ground water levels.

(iii) The undisturbed peak angle of shearing resistance was 21.5 degrees. The residual angles of friction for the undisturbed, remolded and pre-cut cases were 13.4, 11, and 9 degrees, respectively. The decrease in angle is attributed to increasing smoothness of the failure plane. This material did not exhibit residual cohesion for the undisturbed situation. The Dunvegan clay is the only soil which did not exhibit undisturbed residual cohesion. This may be explained by sample variation and least squares fitting. Additional samples would perhaps indicate some residual cohesion. The analysis of this section is based on a angle of residual friction of 13.4 degrees without cohesion.

Generally, the slope satisfies the requirements of an infinite slope. Consequently, an infinite slope analysis was compared to the computer results (Table V-3).

The factor of safety was found to be 1.099 for the upper slope using the known field data and the undisturbed residual angle of internal friction of 13.4°. Because the upper slope "toed out" in a 12 foot drainage ditch the factor of safety should be less than surface 1 in which the full passive pressure was assumed and greater than surface 2 (Table V-3) in which passive resistance was neglected. Therefore, the computer analysis indicates a factor of safety of approximately 1.05.

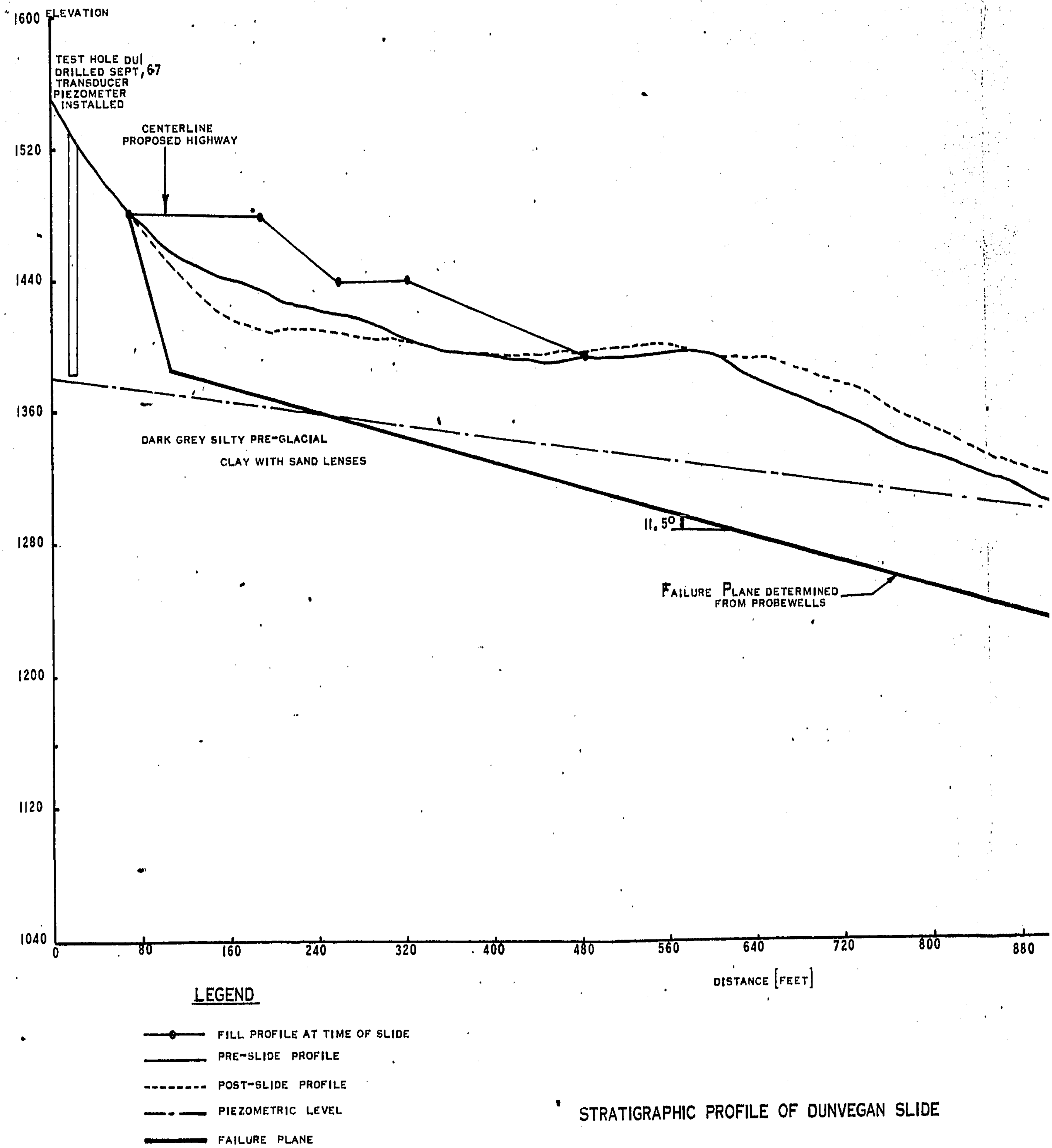
The length to depth ratio of 20 for the slide justifies an infinite slope analysis. In order to perform an infinite slope analysis an average water table for the entire slide was assumed. Assumed average water levels of 50 and 40 feet below the surface, resulted in factors of safety of 1.09 and 1.00 respectively. The factor of safety for the complete slope using the computer gave 1.022.

The slide was initiated by the highway loading on an old landslide area. The quasi-equilibrium slide condition has been shown to exhibit residual strength parameters.

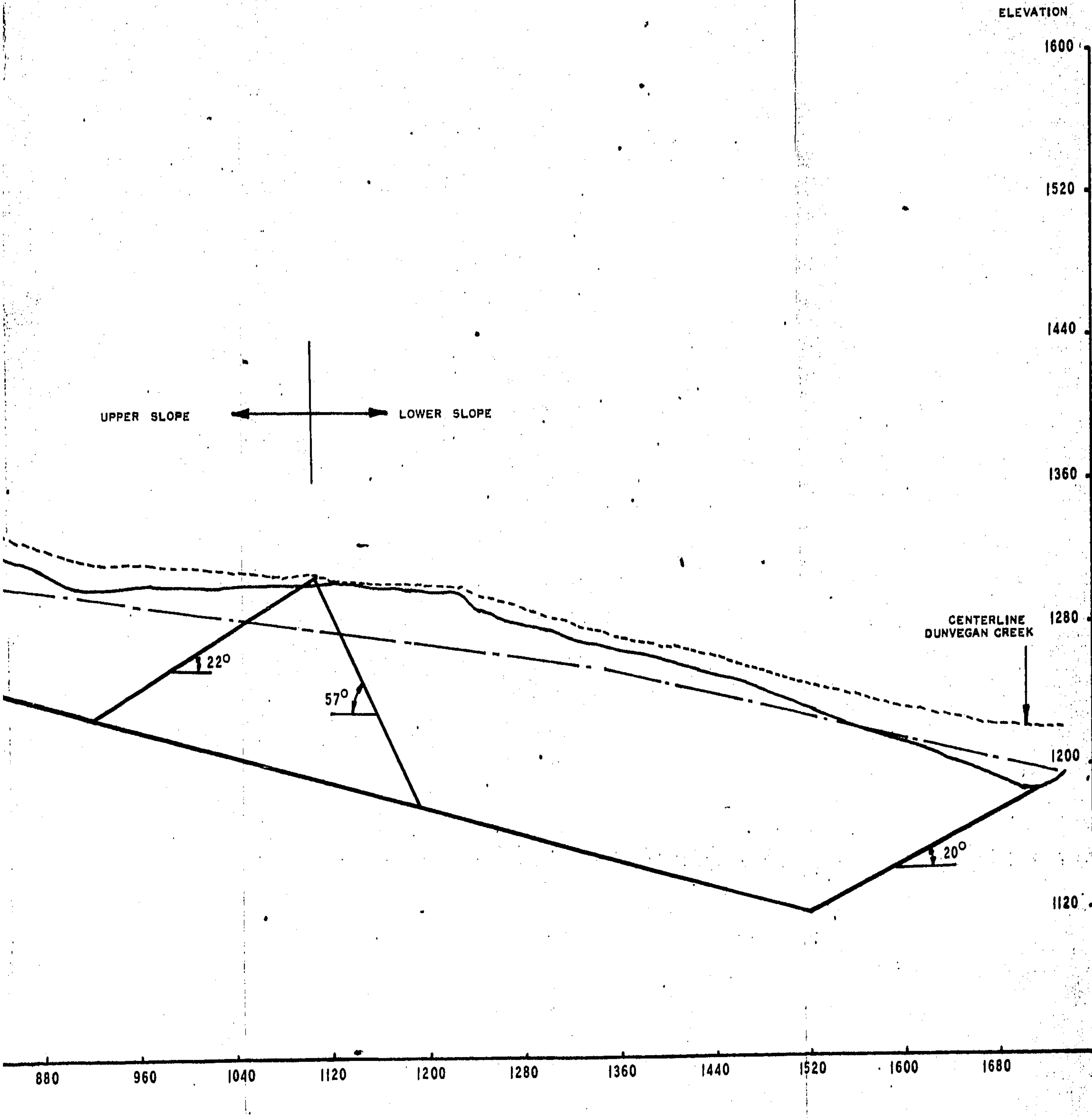
TABLE V-3

RESULTS OF DUNVEGAN SLIDE ANALYSES

Surface Number	Factor of Safety	ϕ' Failure Plane	Slope of Failure Plane	Scarp Slope	ϕ' Scarp	f Scarp	f Toe	Other Data or Remarks
1	1.099	13.4°	11½°	70°	13.4	1	1	Upper slope passive wedge at 22°
2	1.001	13.4°	11½°	70°	13.4	1	1	Upper slope ϕ=0 passive
3	1.207	13.4°	10°	70°	13.4	1	1	Upper slope passive wedge 22°
4	1.100	13.4°	11½°	70°	13.4	3	3	Upper slope passive wedge 22°
5	1.096	13.4°	"	70°	13.4	.5	.5	Upper slope passive wedge 22°
6	.896	11°	"	70°	11°	1	1	Upper slope passive wedge 22°
7	2.698	ϕ_P	"	70°	ϕ_P	1	1	Upper slope passive wedge 22°
8	.926	13.4°	"	57°	13.4	1	1	Lower slope passive at 37°
9	.769	11°	"	57°	11°	1	1	Lower slope passive at 37°
10	1.025	13.4°	16°	57°	13.4	1	1	Lower slope passive at 37°
11	1.022	13.4	11½°	70°	13.4	1	1	Complete slope passive at 37°
12	1.09	13.4	10.0°	--	--	-	-	Infinite slope, water table 50' below surface, 80' deep
13	1.00	13.4	10.0°	--	--	-	-	Infinite slope, water table 40' down, 80' deep
1(A)	1.11	Same as 1						Wedge analysis



STRATIGRAPHIC PROFILE OF DUNVEGAN SLIDE



NOTE- SECTION 1208 [HARDY ET AL, 1962]

FIGURE V-2

5.5 SLIDES AT PEACE RIVER TOWN

(a) Geology of the Heart River Area

The area is underlain by Upper and Lower Cretaceous bedrock materials mantled by thin glacial deposits in the uplands and very thick deposits along the valley walls. The Heart River valley is a pre-glacial valley and now exists as a deeply buried river valley which trends north.

The Lower Cretaceous Peace River sandstone crops out along the Heart River at elevation 1050. Along the proposed 3 mile access of Highway #2 into the Town of Peace River a very thin seam of the Shaftesbury shale crops out above the Peace River sandstone at elevation 1140 adjacent to slide area #1 as shown in Figure V-3. The shale is black and very fissile.

The Heart Valley contains various depths of "buried channel deposits". "Buried channel deposits" are deposits of gravel, sand, silt, and clay that directly overlie Cretaceous bedrock and underlie deposits of definite glacial origin. These deposits may be alluvial or lacustrine. The gravels and sands above the Shaftesbury shale are favoured to be of pre-glacial origin by Jones (1966). He postulates that the buried sand and gravel deposits are probably the northern equivalent of the Saskatchewan sands and gravels described by Rutherford (1937) from central and southern Alberta and for which a pre-glacial age has been suggested. Pre-glacial is defined in the sense by Rutherford that these gravels and sands were deposited before or during glaciation from the northeast and that they lie on

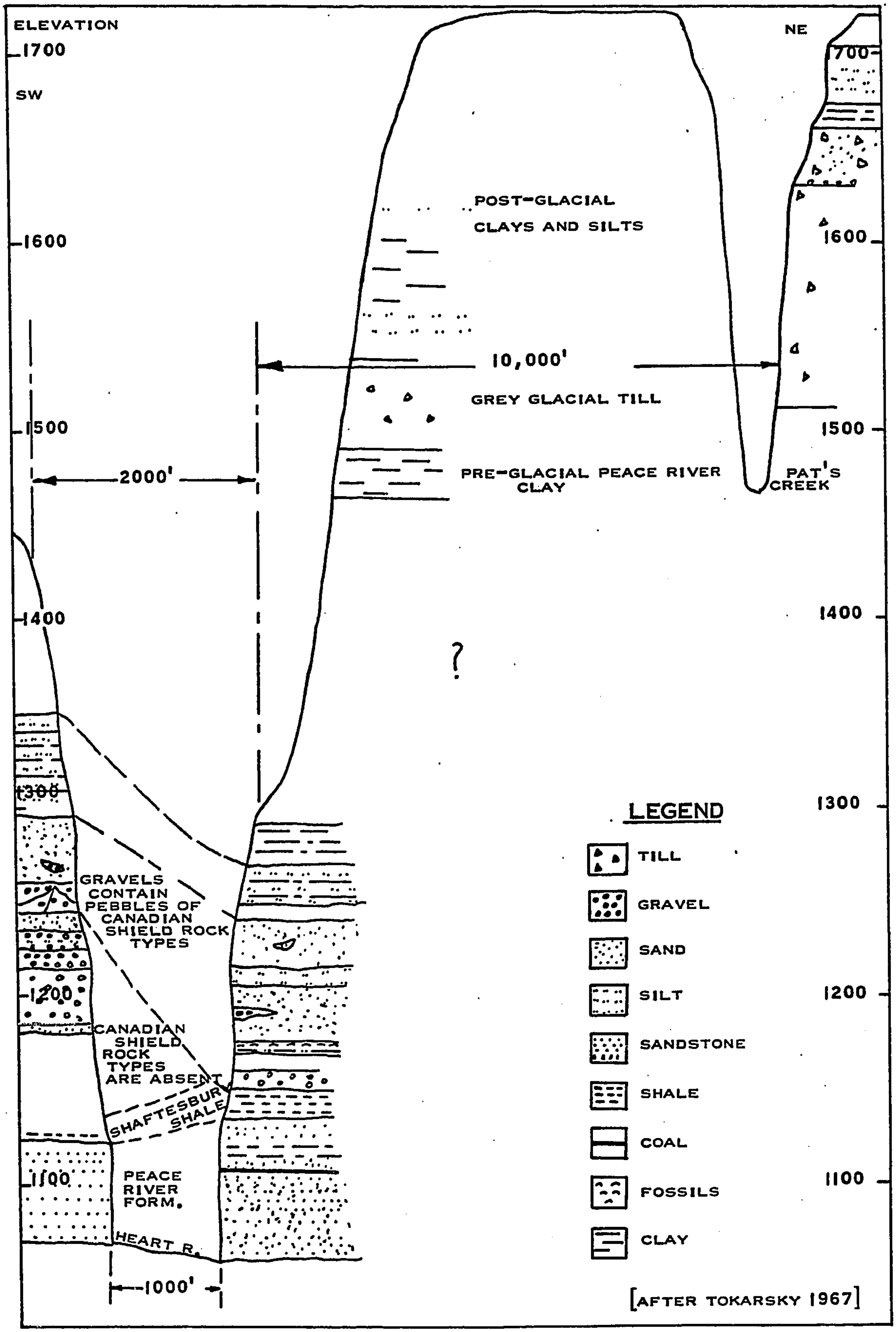


FIGURE V-3 OUTCROP SECTION ADJACENT TO PEACE RIVER SLIDES bedrock. Crop outs of the deeply buried deposits are scarce because of the large-scale slumping along the river valley walls. The slumping along the Heart River is extensive because the valley walls are cut in thick surficial sequences. The base of the gravels is at approximately elevation 1150. These gravels are composed predominantly of well-rounded, mainly quartzite pebbles of Rocky Mountain origin but contain a few gneissic and granitic pebbles typical of the Canadian Shield. The gravel is overlain by a resistant unit of hard-packed silt and fine sand in which layers of gravel are found. (Tokarsky, 1967). This upper gravel consists of pebbles which are predominantly poorly sorted and angular and consist of igneous and highgrade metamorphic rocks of the Canadian Shield. Above the silt 24 feet of a sticky clay is present. The remaining portion of the section is masked by slump debris. Highway excavations have provided exposures which have assisted in partially completing the section mapped by Tokarsky. In slide area #3, a glacial till layer of 45 feet depth was found with its base at elevation 1490 and was found to be underlain by 25 feet of dark grey soft slickensided clay. Below the dark grey clay test holes have indicated the presence of sand and silt. Above the till varying thicknesses of varved clays and silts can be found.

In general, in the"buried valley deposits", sands and gravels containing glacially derived pebbles directly overlie deposits which do not contain glacially derived pebbles. The lower deposits have been dated as 35,000 years old during mid-Wisconsin

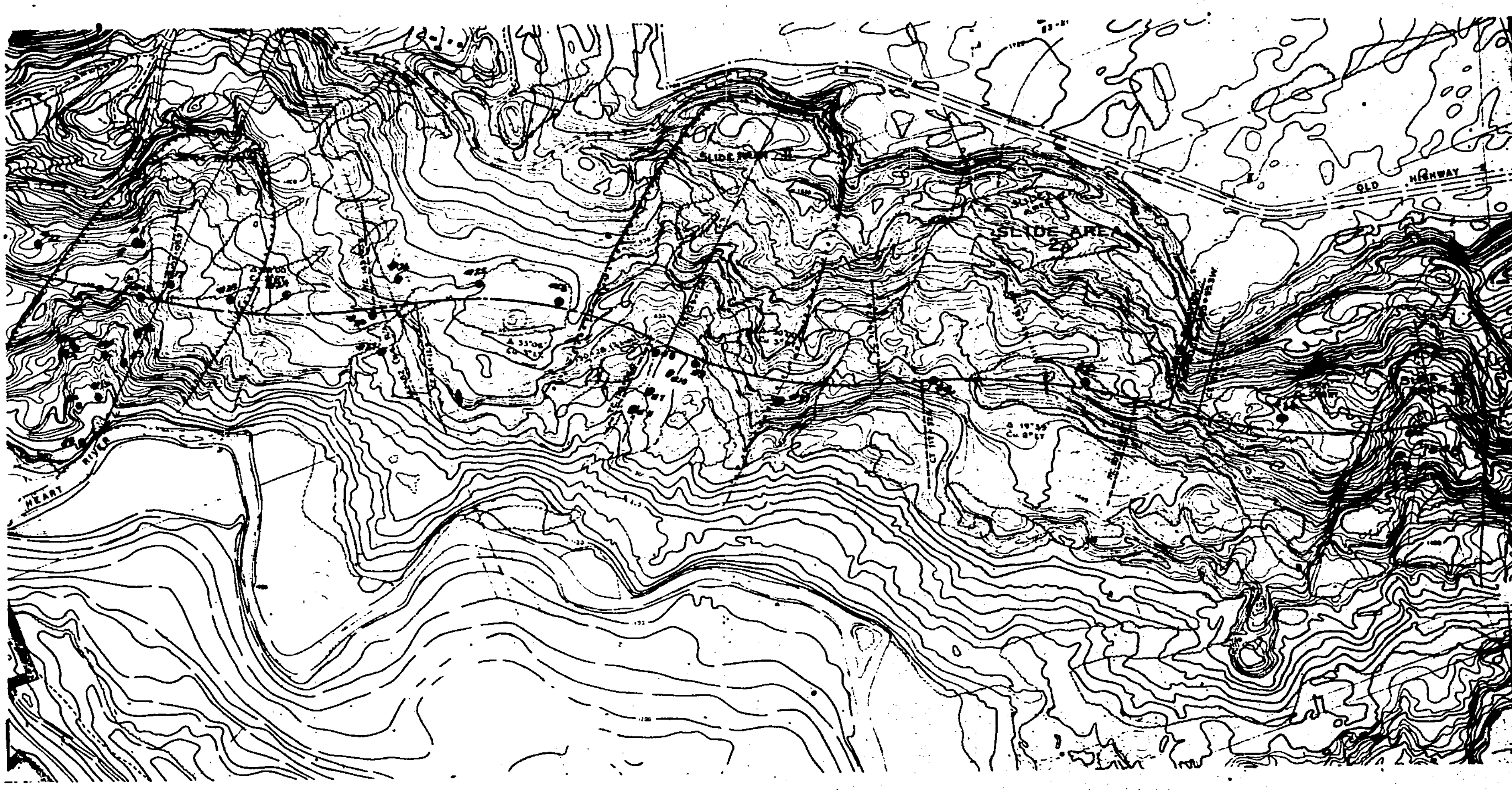
(Tokarsky, 1967).

(b) Previous Investigation of Slide Area

The relocation route of Highway 2 into the Town of Peace River has been selected along the preglacial valley of the Heart River, adjacent to the Peace River. The valley is scarred by numerous landslides which probably occurred during the cutting of the Heart River into the glacial fill materials. The slides are in a quasi-equilibrium state and sliding is easily precipitated. Figure V-4 is a contour plan of the slide area and shows the extreme slump topography.

Numerous test holes were drilled from 1963 to 1966 by the Alberta Department of Highways along the proposed alignment to determine soil types and water conditions. Typical of all the slide areas was a highly plastic silty clay layer varying in elevation and commonly found below a layer of stiff glacial till. Generally, the profile consists of alternating clays, silts and sands and from the outcrop profiles the depth of these materials is approximately 400 feet. The maximum elevation drop from the uplands to the Heart River is 700 feet. The clay found below the glacial till was subjected to extensive laboratory tests and is referred to as pre-till.

Water was present in various degrees in sand layers and would be considered perched relative to the Heart River. Four gullies were isolated as being unable to carry highway fills over 30 feet high. Consequently, much of the surface material was removed and a gravel



SCALE- 1 INCH = 750 FEET

● TEST HOLES DRILLED BY ALBERTA DEPARTMENT OF HIGHWAYS 1963 TO 1965

FIGURE V-4 CONTOUR PLAN OF PEACE RIVER SLIDE AREAS filter was placed below the compacted fill. Slide areas have been referred to as 1, 2, 2A and 3. A failure of the fill occurred in slide area 2A in October, 1967 before the design grade could be reached. Other movements of backslopes and fills have occurred, necessitating the realignment of portions of the highway.

Field standard penetration tests as high as 150 blows/ft. are probably indicative of the sands, gravels and dense till.

The liquid limit was found to be as high as 70 for the pre-till clay but most soils exhibiting plasticity possessed liquid limits of 35.

Laboratory shear strength data or stability analyses have not been reported. Existing data of the Peace River slide area was obtained from the files of the Alberta Research Council, Highways Branch.

(c) Study Field Investigation of Peace River Slides

The field investigation of the area consisted primarily of an examination of the numerous cut and crop out sections. Descriptions of materials are given in 5.4 (a) and (b). In one excavation at slide area #3, a large block sample was recovered of the pre-till clay exhibiting numerous slickensided surfaces. This sample appeared to be part of an old failure zone, consequently it was subjected to extensive residual strength tests.

Ground profiles were also obtained of the major slide areas prior and subsequent to construction of the highway embankments.

(d) Study Analysis of Peace River Slides

The analysis of the Peace River slides was based primarily on observations of the area and laboratory strength results. Instrumentation for location of failure zones and piezometric levels was not installed.

The four isolated slide areas which indicated extensive previous sliding and referred to as 1, 2, 2A and 3, possess average slope angles of 10.3, 11.4, 10.8, and 10.5 degrees, respectively. All areas were undercut to remove wet surface material before placement of a gravel filter and highway fill. Areas 1, 2, and 3 have performed satisfactory with no visible movements. Slide area 2A failed through the fill in October, 1967 after the grade had reached the ground elevation which had been present before undercutting. Subsequent to this failure the highway was re-located to a higher elevation but another movement occurred immediately after equipment had begun undercutting. These slides definitely indicate the quasi-equilibrium state of the area.

Two conditions were noted to exist at the 2A slide which were not present at the other localities including:

(i) The amount of undercut at slide 2A was much less than any of the other areas.

(ii) The most significant factor was the large amount of saturated sand which was uncovered in the 2A area. It was observed that significant flowage of the sand occurred when confining cover was removed. It is worthy to note that the second movement at slide area 2A also occurred immediately after a sand layer was uncovered. This flow phenomenon may be similar to that described by Terzaghi and Peck (1967) in which lateral spreading of a clay occurs when the effective stress on a layer decreases to zero.

It is obvious that localized water and soil conditions have produced varied slide situations. The author maintains that present sliding is occurring in the "buried valley deposits" of the area. The Heart River has undercut below the Shaftesbury Shale and sliding within the bedrock is not present.

The condition common to all the slide areas was the average slope of approximately 10 degrees. It is interesting that the residual angle of friction for the pre-till clay in slide area 3 was 10 degrees. If an infinite slope analysis is assumed to be applicable, then the water table must exist below the failure plane if the residual strength is acting. Another argument which is commonly postulated is that the water table is assumed to exist at the surface and a peak angle of friction is mobilized which results in a slope angle of 10 degrees. This latter explanation is not applicable because the "peak" angle of shearing resistance of the slickensided clay was 12.4 degrees. The strength along failed planes was, no doubt, that of the residual.

The computer stability analyses performed on surfaces near parallel to the surface of both slide 3 and 2A resulted in factors of safety near one (Table V-4). Numerous lengths of slides were investigated and generally the factors of safety were all near one. The water table was assumed below the failure plane, which of course, when analyzed as an infinite slope produces a factor of safety of 1.0 when using a slope angle of 10 degrees and residual angle of shearing resistance of 10 degrees.

Of particular interest is the failure of a 2:1 (26°) cut approximately 100 feet high which was made during the summer of 1967. This failure occurred two months after completion and subsequently has been recut to 2.5:1 (22°). The soil types which were uncovered were similar to those in the slide areas. Because the water conditions and failure surface were not known for this cut, a meaningful stability analysis could not be performed. It is of significance that these cut slopes are much steeper than the failed slide areas which are in an equilibrium state. The shear stress prior to the failure of the cut was likely consistent with a residual factor less than 1. This example is cited, in order to emphasize the need for monitoring slopes of various inclinations and the length of time required for failure. The time versus strength reduction relationship for various materials is of great practical importance.

TABLE V-4

RESULTS OF PEACE RIVER SLIDE ANALYSES

Surface Number	Factor of Safety	Failure Plane Slope	ϕ' Failure Plane	Scarp Slope	ϕ' Scarp	f Scarp	f Toe	Remarks
1 (Slide 2A)	1.020	11°	10°	65°	10°	.5	1	Complete slope, original surface
2 (Slide 2A)	1.000	11°	10°	65°	10°	.5	1	Fill slope
3 (Slide 3)	1.046	11½°	10°	45°	10°	1	1	Original slope
4 (Slide 3)	1.045	11½°	10°	45°	10°	1	1	Fill slope

CHAPTER VI

DESCRIPTION AND ANALYSIS OF SLIDES IN THE EDMONTON AREA

6.1 HISTORICAL GEOLOGY OF THE EDMONTON AREA

After deposition of the Paskapoo Formation at the beginning of Tertiary time, the Plains were subjected to a series of erosion cycles during later Tertiary and early Pleistocene times. The last of these cycles led to establishment of a drainage system, now largely buried by glacial deposits, similar to the existing one. Portions of the present North Saskatchewan River Valley formed a part of the pre-glacial drainage system. It has a valley up to 200 feet deep and a few miles wide, bounded by steep banks. During the development of the valley now occupied by the North Saskatchewan River, the Saskatchewan Sands and Gravels were deposited as valley fill during the Pleistocene Epoch. More than one cycle of deposition and erosion took place as evidenced by the distribution of the sands and gravels at different elevations in Edmonton and surrounding areas. However, deposition of the sands and gravels ceased as they were overriden by the ice sheet from the northwest. The ice advance occurred during classical Wisconsin time, approximately 20,000 years ago, the ice reaching a thickness of over 5000 feet in the Edmonton area (Bayrock and Hughes, 1962). Deglaciation occurred about 10,000 years ago.

Following the retreat of the ice sheet, glacial Lake Edmonton was formed through accumulation of meltwaters, covering the City of Edmonton area. The lake initially drained southeast through the Gwynne Outlet and upon further wasting of ice, lower outlets to the east ultimately drained Lake Edmonton.

After draining of Lake Edmonton, the postglacial North Saskatchewan River began downcutting and draining the Edmonton area developing its present river valley.

6.2 GENERAL GEOLOGY OF THE EDMONTON AREA

The Edmonton area is underlain by a variety of sedimentary deposits ranging from coal to glacial lake sediments. These deposits can be divided into four distinct units which are, in ascending order: (1) bedrock consisting of sandstone, shale and coal of the Edmonton Formation (2) preglacial Saskatchewan Sands and Gravels (3) glacial till (4) glacial Lake Edmonton sediments. Each of the units possess characteristic geologic and engineering properties.

(i) Edmonton Formation

The Edmonton Formation of Upper Cretaceous age consists of interbedded bentonitic shales and sandstones with numerous coal seams. The sediments are poorly consolidated and dip southwestward at 20 feet per mile.

The shales and sandstones are often bentonitic and contain high proportions of the clay mineral montmorillonite. The coal seams are generally only a few inches in thickness but in some areas several feet of coal are encountered. These thicker coal seams were mined at the turn of the century.

(ii) Saskatchewan Sands and Gravels

"Saskatchewan Sands and Gravels" refer to all sands and gravels lying above the Edmonton Formation and beneath the till. The Saskatchewan Sands and Gravels occur sporadically throughout the area. According to Bayrock and Hughes (1962) the origin of the Saskatchewan Sands and Gravels in the Edmonton district is complex and more than one depositional cycle was involved. The sands and gravels are separated into three units on the basis of elevations of occurrence. The lowest unit is found as a pre-glacial valley fill in valleys incised into the underlying bedrock. Fossils found in this unit are of pre-glacial origin. The highest unit of Saskatchewan Sands and Gravels occur as cappings on hills and are overlain by glacial till. Fossils in this unit are of early Pleistocene age. Saskatchewan Sands and Gravels at intermediate elevations form cores of hills on the Lake Edmonton plain.

The lithology of the various units of the Saskatchewan Sands and Gravels is very similar being composed of quartzites and cherts with occasional arkose pebbles, all of which were derived from the Rocky Mountains.

The Saskatchewan Sands and Gravels may be distinguished from overlying glacial gravels and sands by the absence of igneous and metamorphic rocks of the Canadian Shield. They tend to fill irregularities in the underlying bedrock surface and reach thicknesses of over 65 feet in pre-glacial valleys (Bayrock and Berg, 1966). These gravels are often absent or thin where the bedrock surface is high.

(iii) Glacial Till

Till is unsorted, unstratified sediment deposited by a glacier.

In the Edmonton area till is widely present in the form of ground moraine and hummocky dead-ice moraine. The average mechanical composition of till in the Edmonton area is: sand 41%, silt 31% and clay 28% (Bayrock and Hughes, 1962). Although surface tills of the Edmonton area are more or less uniform in grain size and are similar in composition, there are local deviations. All gradations between "average" till, sandy till and gravel have been found.

The clay-size fraction of the till contains a large percentage of montmorillonite derived from the local Cretaceous bedrock.

The till is brown where oxidized and grey where unoxidized; the color change from brown to grey occurs about 20 feet below the surface. Generally, the till is extremely dense and provides high bearing capacity. Lenses of stratified sand and gravel are commonly present in the till, representing minor washing of glacial debris by running water.

In most cases the contact between the till and the Saskatchewan Sands and Gravels is sharp and easily recognized in the field.

(iv) Glacial Lake Edmonton Deposits

Glacial Lake Edmonton sediments range from sand to clay laid down in a large proglacial lake at the close of the Wisconsin glacial period.

Lake Edmonton deposits are classified under three main headings (Bayrock and Hughes, 1962):

(1) Normal deposits, which are the most common type of sediment of the lake, are not modified by later action. They consist of bedded fine sands, silts and clays.

(2) Modified deposits are deposits of "normal" type that were subsequently partly or wholly eroded, or had additional materials deposited over them; or both.

(3) Pitted deltas consist of fine to medium grained sand and silt.

The clays are brown near the surface but may be grey at depths exceeding twenty feet. Lake Edmonton deposits range in thickness from about 100 feet to less than one foot. The deepest deposits occur along the course of the present North Saskatchewan River in the Edmonton district.

The Lake Edmonton deposits are normally consolidated with some preconsolidation of the upper few feet due to desiccation.

6.3 LESUEUR SLIDE

(a) General Description of Lesueur Slide

The undisturbed stratigraphic sequence at this location is typical of the Edmonton area. It is illustrated in Figure VI-1.

The description and analysis of the Lesueur slide was previously made by Painter (1965) in his Master of Science thesis from which much of the following account is taken.

The Lesueur slide is typical of many slides along the North Saskatchewan River Valley which normally are located on the outside banks of meander bends. It was initiated in January, 1963 and major movements took place in August, 1963. The slide undermined the home of Mr. R. Lesueur and necessitated its removal.

The scarp of thirty feet height can be seen to comprise Lake Edmonton silts and sands. On the east flank of the slide about ten feet of vertical movement has occurred and visual inspection discloses that some strata are not in their original stratigraphic sequence because of extensive past sliding. For example, Saskatchewan Sands and Gravels are overlain by the Edmonton Formation. A two feet seam of yellow weathered bentonite is also present on the east flank.

(b) Previous Field Investigation

The slide was believed initiated by river toe erosion over the last two centuries. Carbon dating of wood in old slump blocks indicated an age of 170 years.

Eight test holes were drilled to establish the soil profile and recover undisturbed samples. In addition, seven holes were drilled and cased with three inch diameter aluminum pipe to be used as probe-wells in locating the failure zone. The aluminum pipes were also perforated and used as groundwater level indicators. Probewell readings taken in October, 1965 and April, 1965 gave a positive indication of at least the upper surface of failure. The failure plane was found to exist at approximately elevation 2002 in the highly bentonitic materials of the Edmonton Formation bedrock.

Five Bishop hydraulic-type piezometers were installed in and adjacent to the slide area. These piezometers were placed in the lower portion of the slide and the maximum piezometric level was found to exist at elevation 2022. The placement of piezometers in the stable slope adjacent to the landslide was assumed to simulate conditions prior to the slide. This assumption may not have been correct as a deep ravine exists to the east of the slide. It also seems questionable whether the east stable slope would have had the same pore pressure conditions as existed in the slide before failure. The maximum pore pressures occurred in February, 1965 and were attributed to freezing at the toe of the slope. Pore pressures began to dissipate in April, 1965 but began to increase with spring break-up as the river level increased. During December and January, the piezometric levels indicated were as much as fifteen feet below the tip elevation. This low piezometric level could have been measured because of incomplete deairing of the piezometric lines. Difficulty was encountered in deairing the lines and readings taken after 1965 were considered to be erroneous.

(c) Previous Laboratory Investigation

Numerous unconfined compression tests were performed on the typical profile soils. The Edmonton Formation materials exhibited unconfined compressive strengths as large as 9 kilograms per square centimeter.

Four series of consolidated undrained triaxial tests with pore pressure measurements were performed to obtain peak effective strength envelopes. These tests were performed by Painter and included:

(i) undisturbed dark brown clay shale- $\phi'_{CU} = 9°$ and $C'_{CU} = 1.5$ kg/cm^2

(ii) undisturbed bentonite- $\phi'_{CU} = 4°$ $C'_{CU} = .85$ kg/cm^2

(iii) undisturbed bentonitic clay shale- $\phi'_{CU} = 17°$ $C'_{CU} = .29$ kg/cm^2

(iv) remolded bentonitic clay shale- $\phi'_{CU} = 19°$ $C'_{CU} = .10$ kg/cm^2

The high cohesion of .85 kg/cm^2 and low friction angle of 4^o for the undisturbed bentonite appears incorrect. The Mohr envelope obtained by Painter is shown in Figure VI-2. The circle obtained at a confining pressure of 4.9 kg/cm^2 is too low and an alternate interpretation as shown in Figure VI-2 results in cohesion of .4 kg/cm^2 and angle of internal friction of 12^o.

(d) Previous Stability Analyses

Several stability analysis methods were used to determine the shear stresses which acted in the river bank before failure and the factor of safety of the adjacent stable bank.

For purposes of stability analyses, the peak values of cohesion and angles of shearing resistance of the three principal soil strata were arithmetically averaged, where the shear failure occurred through the entire depth of the Edmonton Formation. The principal soils were assumed to be the dark brown shale, bentonitic shale and bentonite.

The phreatic line was principally based on the maximum piezometer readings at the bottom of the bank and upon groundwater levels at the top of the bank. The phreatic line at the top of the bank, based upon test hole water levels, was high and assumed that a perched water table did not exist. The presence of a perched water table at this site is possible due to water bearing sand and gravel above an impermeable till layer.

The geometry of the landslide resembled that of a sliding block which appeared to have its base within highly bentonitic soil.

The following stability results were obtained from the slide slope by Painter:

(i) Bishop's (1955) method using a circle tangent to the bentonitic clay shale gave a factor of safety of 0.92 and tangent to the bentonite at elevation 2002 gave a factor of safety of 1.13.

(ii) Sliding block analysis, using an average slope of 2:1 and a bank height of 105 feet, gave a factor of safety of 0.97, assuming the base of the sliding block in a layer of bentonite and 0.65 if based in bentonitic clay shale.

(iii) A "zero-cohesion" analysis according to Henkel and Skempton (1955) gave a factor of safety of 0.45, using the bentonite strength parameters.

(iv) Using Bishop's method with the most critical circle and average friction angle for the Edmonton Formation of 10^{o}, it was found that the required cohesion was 0.61 kg/cm^2 for a factor of safety of 1.0.

Painter also obtained the following stability results from analysis of the stable slope adjacent to the slide:

(i) A sliding block, tangent to a bentonite layer at elevation 2011 with an average slope of 2.5:1 and a bank height of 95 feet, gave a factor of safety of 1.37.

(ii) Bishop's analysis with circles tangent to a layer at elevation 2011 gave factors of safety of 1.97 and 1.73.

(iii) A sliding block, assuming a plane tangent to the bentonitic soil at elevation 2002, indicated a factor of safety of 1.00.

The analyses which resulted in several factors of safety near 1.0 were based on a high piezometric level and peak strengths.

Some of the peak strengths have been shown to be low by other investigations (Sinclair and Brooker, 1967).

(e) Study Field Investigation of Lesueur Slide

In order to obtain undisturbed samples of the slide materials, a test hole, LA 9 was drilled above the scarp adjacent to test hole 3 reported by Painter. The Pitcher sampler was successful in recovering undisturbed samples of the Edmonton Formation. The soil profile is shown in Figure VI-1. The test hole was drilled in July, 1967 and the water level remained at 36 feet below the surface throughout the summer. This water table was assumed by Painter to be active at the failure plane, approximately 105 feet below the surface. In November, 1967 the 1½ inch pipe and well point were grouted into place for future installation of the transducer piezometer. Readings of the piezometer throughout the winter and summer of 1968 indicated a maximum head of 32 feet (elevation 2038) above the horizontal failure plane (Figure VI-3). Therefore, it has been conclusively shown that a perched water table exists above the clay till strata.

Water levels which were measured in October, 1967 in the slide area were similar to those recorded by Painter. In the central portion of the slide his results from water levels and piezometer data indicated that the phreatic line was approximately at elevation 2022, whereas water levels in October, 1967 indicated an elevation of 2025. These results are consistent with the level indicated by the transducer piezometer (Figure VI-3).

Ground profiles of the slide area taken in September, 1966 indicated that the slide material was moving, resulting in a flattening of the slope. The slope was reduced from an inclination of 16.7^{o} to 14.1^{o}. Since September, 1966 the slope change has been approximately 0.4^{o} as determined by ground surface profiles.

(f) Study Analysis of Lesueur Slide

The data used in this analysis is summarized in the following paragraphs:

(i) Probewells installed subsequent to the slide gave evidence that a horizontal failure plane existed in a bentonitic clay shale at approximately elevation 2002.

(ii) The University of Alberta piezometer installed above the scarp confirmed that a perched water table existed at the site. Readings of this piezometer indicated a maximum piezometric head exists to elevation 2038 at the scarp.

(iii) The visible scarp slope exists at 50^{o} to the horizontal and must be assumed to be the same angle below ground level. The analysis is sensitive to changes in the scarp angle. Results of surfaces 4 to 7 of Table VI-1 indicate that 50^{o} gives the lowest factor of safety.

(iv) The bentonitic clay shale, the suspected failure material, exhibited an undisturbed angle of residual friction of 10^{o} and a residual cohesion of 700 psf. The peak parameters were 22.5^{o} and 144 psf. The remolded residual angle of shearing resistance and pre-cut angle of friction were 7.5^{o} and 8.5^{o}, respectively. The remolded state has been shown to be inapplicable because of the completely different structure produced by the remolding process.

In Table VI-1 the parameters used in the analysis for the soils above the Edmonton Formation were:

Lacustrine deposits	$\phi'_p = 22^o$	$C'_p = 0$
Glacial Till	$\phi'_p = 26^o$	$C'_p = 500$ psf.
Sand	$\phi'_p = 32^o$	$C'_p = 0$

The various parameters used for the Edmonton Formation are indicated in columns of Table VI-1 for failure plane and scarp parameters.

Two interpretations are possible for the Lesueur slide as shown by the results of the computer analysis.

(1) Consideration of the preslide surface and utilization of average peak strength parameters for the Edmonton Formation on the scarp ($\phi'_p = 22^o$, $C'_p = 700$ psf) resulted in an angle of internal friction of 14^o required for limiting equilibrium along the horizontal failure plane. Because the residual undisturbed angle is 10^o the residual factor of the slope becomes .69. A friction angle of 20^o must be developed if the same strength is to be assumed along the entire length of the Edmonton Formation.

(2) The alternate approach is based on the existence of a residual cohesion. It is common practice to disregard residual cohesion when utilizing residual strength. The discussion in Chapter IV has indicated that residual cohesion exists. If a residual cohesion of 288 psf and a residual angle of internal friction of 10^o is used along the horizontal failure plane and peak parameters along the scarp, a factor of safety of 1.043 is obtained.

The above dilemma arises from our incomplete understanding of the shear strength of stiff clays. Specifically, the rate of decrease of the cohesion in the field is unknown.

The preslide slope angle was 23 degrees (2.4:1) and the postslide slope was 16.7 degrees (3.3:1). This slope change is of particular significance because it demonstrates the decrease in developed shear strength during the slide. The analysis of the post-slide profile along the established slide failure plane resulted in a factor of safety of .885 assuming a residual angle of 10^{o} and zero cohesion on both the scarp and the horizontal failure plane. A residual cohesion of 144 psf would raise the factor of safety to 1.083. The analysis of the slide subsequent to failure indicates that the slope may be considered a single moving block, although there is visual evidence that differential movement is occurring within the slide mass. Hayley (1968) found that a postslide profile of a 2200 feet slide indicated a factor of safety much above 1.0 when utilizing residual strength along a horizontal failure plane. His procedure was to analyze the slide as seven individual blocks in which the downhill blocks moved faster than the uphill blocks, consequently, reducing the passive pressure.

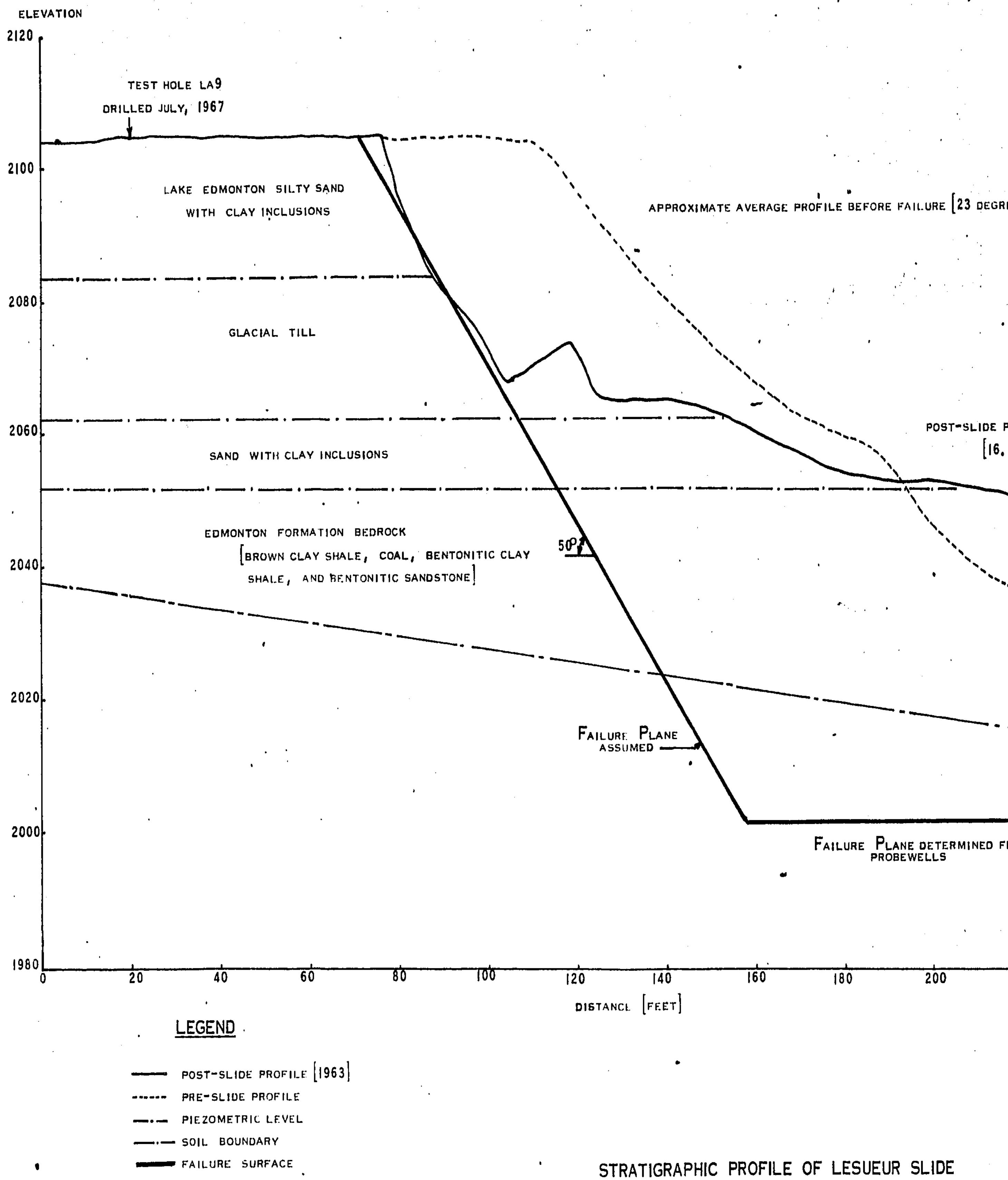
The Lesueur postslide profile measured two years after the major movement in August, 1963 was 16.7 degrees but since then has flattened to 14.1 degrees. It can be presupposed that the ultimate slope angle will approach 8^{o} to 10^{o}, as this is the long-term slope for many failed slopes in the Edmonton Formation. An analysis of the long-term slope employing the conditions used in the 16.7 degree slope will result in a factor of safety greater than one. Either a strength decrease must be associated with the slope flattening or soil creep must be active on the slope.

TABLE VI-1

RESULTS OF LESUEUR SLIDE ANALYSES

Surface Number	Factor of Safety	Failure Plane Elev.	ϕ' Failure Plane	Scarp Slope	ϕ' Scarp	f Scarp	f Toe	Remarks
1	.885	2002	10°	50°	10°	1	1	Post slide profile
2	1.083	2002	c=144psf ϕ=10°	50°	c=144 ϕ=10°	1	1	Post slide profile
3	1.651	2002	c=556psf ϕ=10°	50°	c=556 ϕ=10°	1	1	Post slide profile
4	.941	2002	10°	50°	c=1000 ϕ=22°	1	1	Preslide profile
5	1.104	2002	10°	65°	c=1000 ϕ=22°	1	1	Preslide profile
6	.987	2002	10°	60°	"	1	1	Preslide profile
7	.985	2002	10°	40°	"	1	1	Preslide profile
8	1.008	2002	12°	50°	"	1	1	Preslide profile
9	1.004	2002	14°	50°	c=700 ϕ=22.5°	1	1	Preslide profile
10	1.043	2002	c=288psf ϕ=10°	50°	"	1	1	Preslide profile
11	1.040	2002	20°	50°	20°	1	1	Preslide profile
12	1.003	2002	18°	50°	22.5°	1	1	Preslide profile
13	.545	2002	10°	50°	10°	1	1	Preslide profile
14	.976	2002	c=556psf ϕ=10°	50°	c=556 ϕ=10°	1	1	Preslide profile
4(A)	.83	Same as 4						Wedge analysis
14(a)	1.00	Same as 14						Wedge analysis

Note: Cohesion in psf



STRATIGRAPHIC PROFILE OF LESUEUR SLIDE

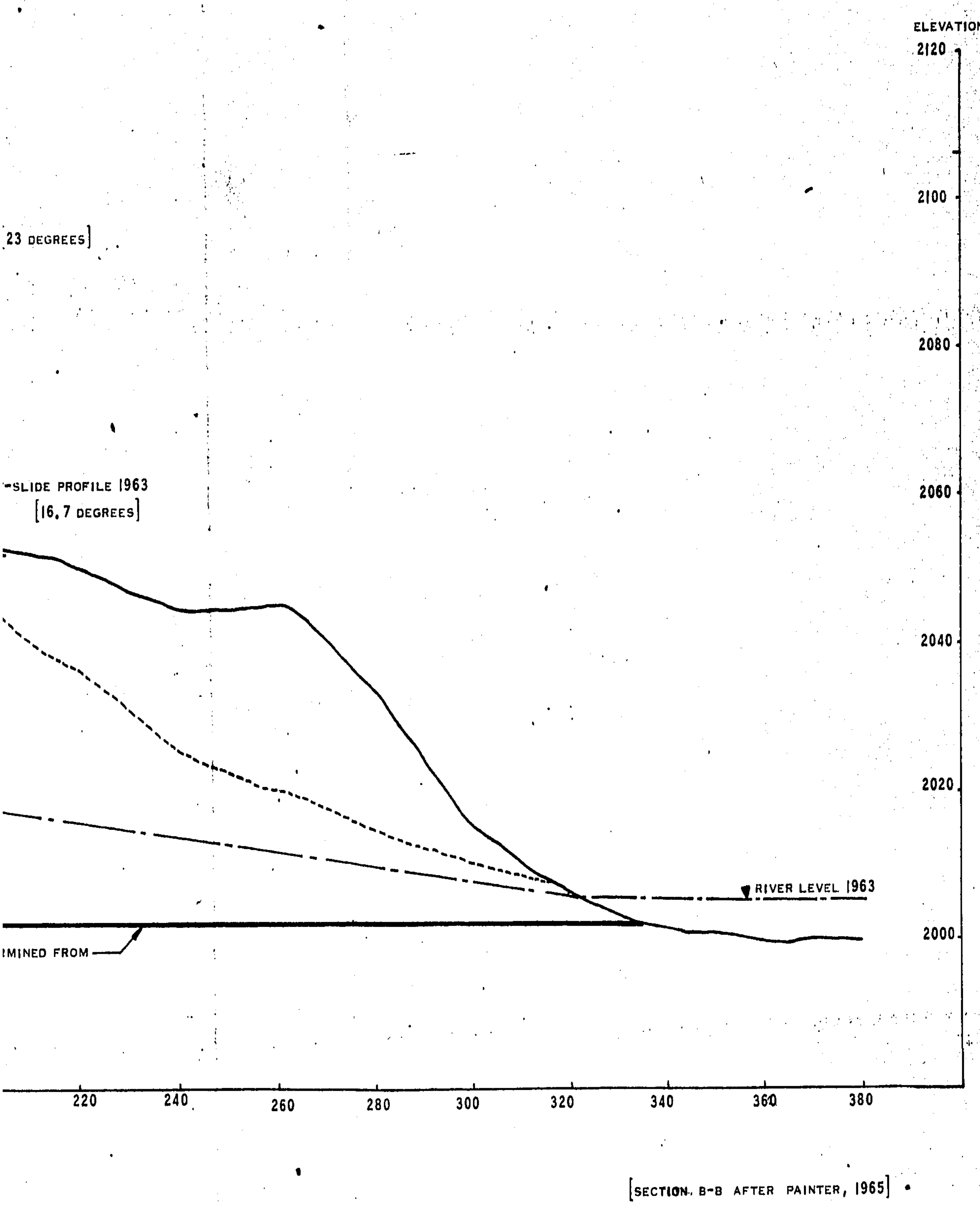

ELEVATION
2120
2100
2080
2060
2040
2020
2000
23 DEGREES]
-SLIDE PROFILE 1963
[16.7 DEGREES]
RIVER LEVEL 1963
IMINED FROM
220
240
260
280
300
320
340
360
380
[SECTION B-B AFTER PAINTER, 1965]

FIGURE VI-I

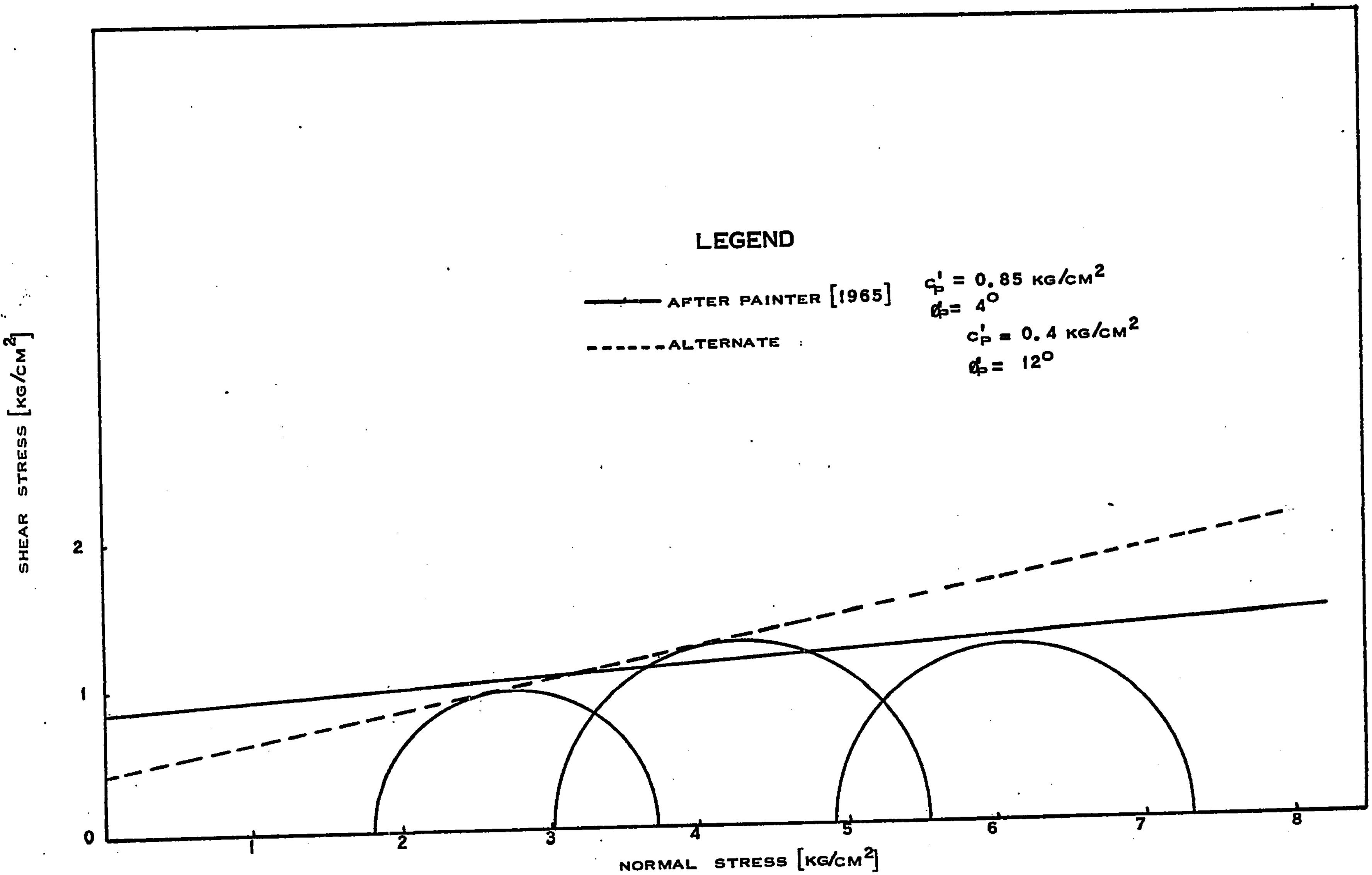


FIGURE VI-2 MOHR DIAGRAM FOR LESUEUR BENTONITE

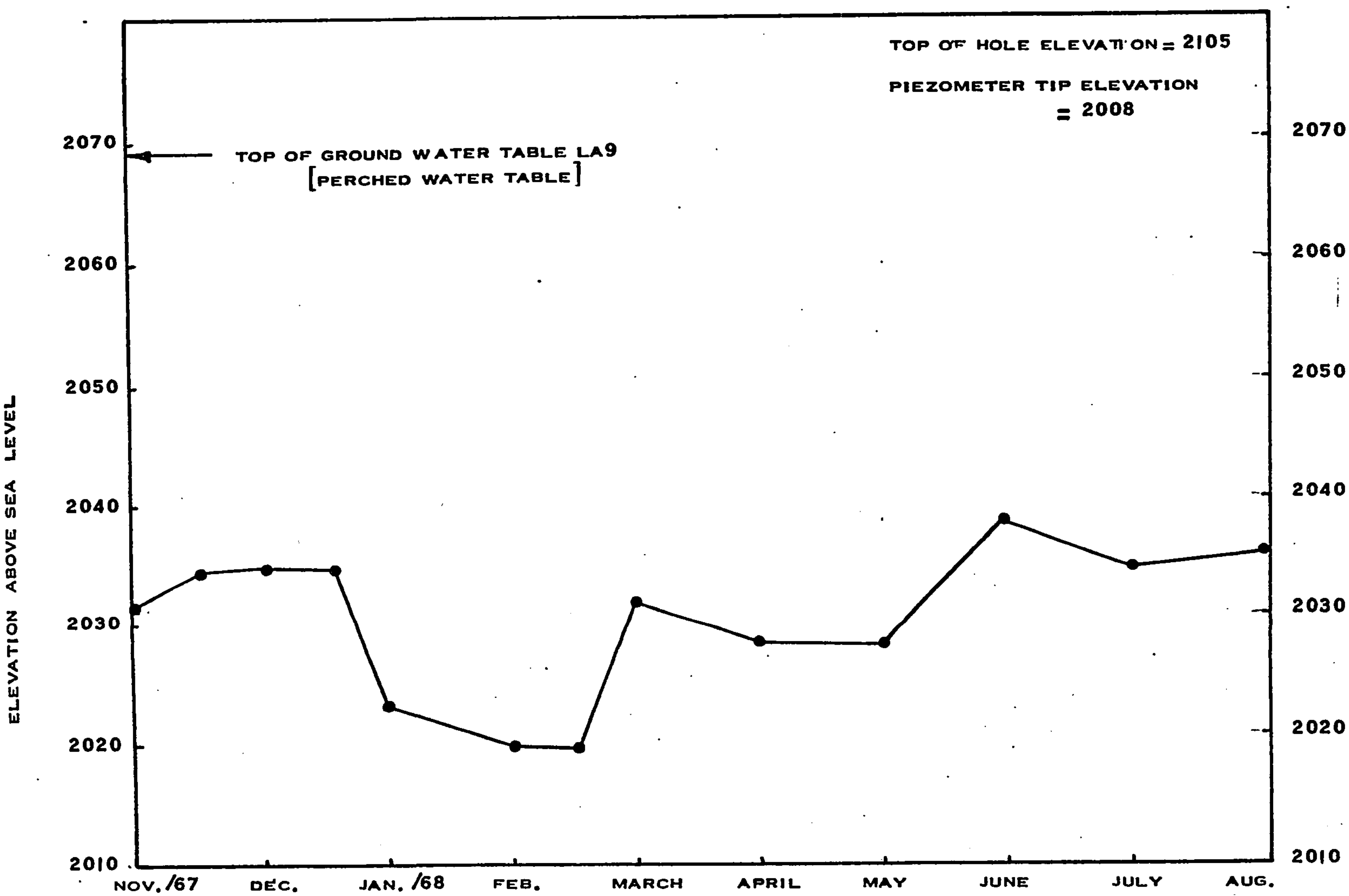


FIGURE VI-3 TRANSDUCER PIEZOMETER READINGS IN LESUEUR TEST HOLE LA9

It is concluded that caution must be employed when analyzing poststlide conditions in which movements are still prevalent. If a slide exhibits residual strength subsequent to a failure, then any later analysis on flatter slopes will be ambiguous when previous pore pressures and failure surfaces are utilized.

6.4 GRIERSON HILL SLIDE

(a) History of Movements and Stabilization of Slide

Recorded movements on Grierson Hill date back to 1887. In that year the average slope of the bank was about 2.5:1 (22^{o}) (Figure VI-4). A major movement in 1905 caused the top of the bank to move back 75 feet after a six year period of high precipitation. Between 1905 and 1915, the top of the bank moved back 50 feet, but from 1915 to 1957, only small movements occurred in the top of the bank in the east area of the slide (Hardy, 1957).

The river erosion has been very extreme in the area, and between 1887 and 1893 the river encroached into the bank about 50 feet. Since 1893 the river has been pushed out by the dumping of fill and is now 400 feet from its 1893 position at the section of maximum movement. From 1911 to 1915, a period of above average precipitation, the toe encroached on the river by about 100 feet. In 1911 the lower portion of the slope was used as a dump to help stabilize the area; this activity has resulted in an accumulation of up to 50 feet of garbage, straw, bricks, fill and clay, etc. The hillside is now on an average slope of 5:1 (11.5^{o}), as compared to the much steeper slope of 2.5:1 of 1887.

Since 1952, considerable effort has been expended in stabilizing the slope so that a paved roadway could be sustained on the hillside. Drainage of the slope was undertaken in 1952 in order to reduce soil movements. Major sources of water appeared to be concentrated in coal seams of the Edmonton Formation. Several test holes were drilled so that lowering of the ground water table could be accomplished by pumping. A drainage gallery of five foot diameter was dug to facilitate gravity drainage of the area. The gallery intersected the workings of a mine which was one of the many mines actively worked at the turn of the century. Only a portion of the old mine was found to be intact and was submerged. The sound condition of timber bracing probably indicated that the mine had been submerged since the slide of 1905 which put the mine out of operation.

In September, 1958 after three inches of rain, soil cracks developed which indicated movement of the whole slide area. Movements and cracking during 1958 indicated that a deep-seated slide existed as well as movements in the surface fill.

In the fall of 1959, during the drilling of the drainage gallery, a definite shear zone was encountered. This shear zone was believed to be the surface along which the original deep-seated slide occurred in 1905 and which cut off the mine entry shaft. Markers on the shear zone indicated a total movement of sixteen inches during 1959 and 1960.

In September, 1960 three tiltmeter installations were placed to locate the failure plane. One tiltmeter hole which penetrated a drainage gallery gave evidence of the location of the failure surface by direct observation from the top of the hole or from the mine adit.

One set of test holes drilled in the east zone of the slide indicated a ground water level at elevation 2170 to 2180. At the time these holes were observed they were blocked off from a coal seam at elevation 2130. Another set of test holes in the same zone, but drilled below elevation 2120, showed free water at elevation 2120 to 2130. Pumping of either set of test holes produced no measurable change in the water level in the other set. It appeared that the upper water level was perched and was not directly connected to the old mine workings.

Settlement gages and horizontal movement hubs indicated movements in the slope were approximately equal from the bottom of the slope to near the top. Horizontal movements were found to be approximately fifteen feet over a period of three years. These movements were attributed to the stage highway fill which was being placed. The large movements indicated at the surface were shown to be unrelated to the smaller movements of the slope indicators placed in the Edmonton Formation shales and coal.

The stabilization of the area by removal of water by the drainage gallery and a storm sewer installed during 1960 in a water-bearing zone above the main scarp reduced the relatively large movements. At the present time only minor movements occur in the pavement.

(b) Soil Profile

The upper portion of the slide consists of glacial till underlain by the Edmonton Formation consisting of clay shale, coal and bentonitic clay shale. Approximately 300 feet of the lower portion of the slide is covered by varied fill. Due to the extensive slide activity in the past, the soil is disturbed in the area of the landslide (Figure VI-4).

(c) Previous Stability Analyses

The failure surface, located by the old shear surface in the mine entrance and tiltmeter movements, was the basis for the stabiliy analyses which were performed. Several circular failure arcs were assumed and average developed shear strengths for various profiles were determined and results indicated stresses from 120 to 930 lbs/ft^2 (Hardy, 1957).

Along the line of tiltmeter installations T3, T2 and T1 there was some evidence of two distinct failure zones. The first was an upper one of the circular arc type which was approximated by T3 and T2. The lower surface indicated by T1 was analyzed as a sliding block.

It was reported by Hardy that several sections along the shoreline had developed shear stresses from 740 to 810 $lbs/ft.^2$. On the upper slopes the computed shearing stresses ranged from 420 to 900 $lbs/ft.^2$ It was considered significant that on the slopes where the shearing stresses were in the low range, the test holes in the area showed comparatively high water levels. In the above analyses, the pore water pressure was assumed zero. The cohesion was assumed to be zero and the strength was therefore represented by the well-known equation: $S = (p-u) \tan\phi$. Two areas in the slide where pore pressures were known had a computed shear resistance equal to the actual shearing stress in the soil when angle ϕ of 20^{o} was used.

The value of 20^{o} for the angle of internal friction was considered reasonable for the types of soil and agreed with published values for the peak angle of internal friction for the highly preconsolidated soils.

Laboratory strength data was not reported for the slide when analyzed by Hardy (1957).

(d) Study Analysis of Grierson Hill Slide

Additional field work was not undertaken at this slide. The shear zone had been indicated to be in a bentonitic clay shale. Consequently, soil data from similar sites in the Edmonton Formation was utilized for the Grierson Hill analysis.

The data used for the analysis of this slide is summarized in the following paragraphs:

(i) Three slope indicators placed the failure zone in a bentonitic shale layer of the Edmonton Formation. Another movement was located in the mine adit and gave a positive location of the scarp. The slope indicators showed that the upper 500 feet of the slide possessed a failure plane which had a three degree inclination.

(ii) The piezometric head was found to exist at elevation 2130 at the scarp. This was based upon water levels from numerous test holes. Pumping of various test holes gave evidence that a perched water table existed at the site above elevation 2130.

Stability analyses were performed using the various known profiles from 1900 to the present. Analysis of the 1900 profile (22^{o}) prior to a major 75 feet regression of the scarp resulted in a factor of safety of .982 (Table VI-2). The peak shear parameters of $C_p' = 700$ psf and

$\phi_p' = 22^o$ were used on the scarp and the residual undisturbed angle of internal friction for Edmonton Formation bentonitic clay shale of 10^o with zero cohesion was applied along the failure plane sloping at 3^o.

By 1915 the bank had regressed approximately 125 feet from its 1900 position. An analysis of the slope subsequent to these major movements resulted in a factor of safety of .972 using the 1915 profile (14.5^o) and strength parameters of $C_p' = 144$ psf and $\phi_r' = 10^o$ for the Edmonton Formation on the scarp and failure surface.

Up until 1915 the river had been in the process of encroaching on the bank. After the major movements, a program was begun in which fill was placed at the toe of the slope near the river and at the toes of steeper portions of the slide towards the escarpment. This placement of fill resulted in an "artifical" flattening of the slope to 11.3^o as it now exists. Major movements at the 1915 scarp did not occur after fill was placed at the toe but two definite scarps have developed on the flat area of the slope.

Analysis of the 1958 profile produces a factor of safety of 1.060 using a residual angle of shearing resistance of 9^o on both the scarp and failure plane. Consequently, the fill material placed on the slope plus flattening as a result of roadway construction has produced conditions so that equilibrium is maintained by an angle of shearing resistance equal to 9^o. In this case the slope has been "over-flattened" as compared to the slope which would have resulted from natural degradation.

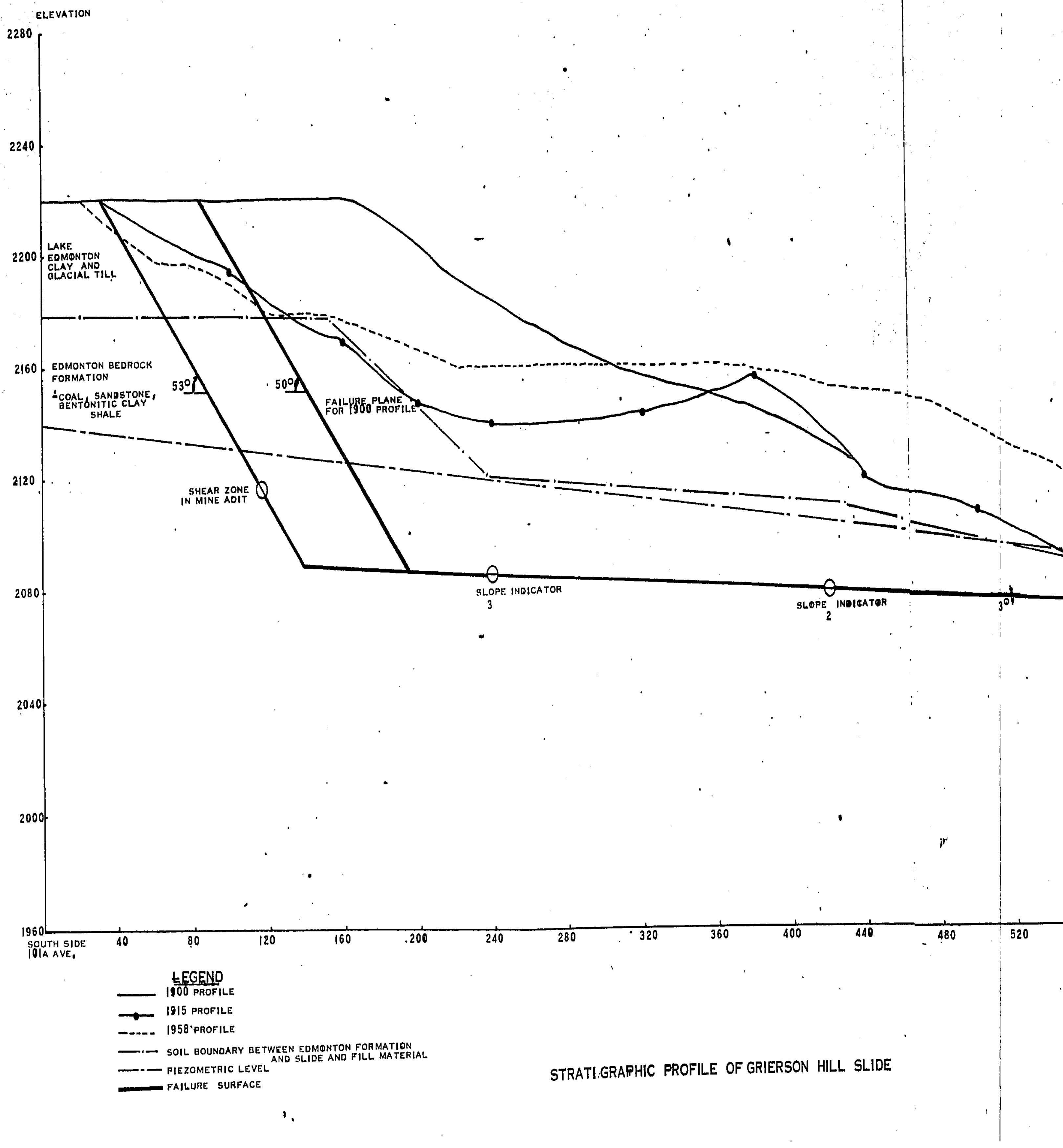
Results of other analyses applying varying strength parameters are included in Table VI-2.

TABLE VI-2

RESULTS OF GRIERSON HILL SLIDE ANALYSES

Surface Number	Factor of Safety	Failure Plane Slope	ϕ' Failure Plane	Scarp Slope	ϕ' Scarp	f Scarp	f Toe	Remarks
1	.982	3°	10°	50°	c=700 ϕ=22	1	1	1900 profile scarp 75'back
2	2.287	3°	22°	50°	"	1	1	1900 profile scarp 75'back
3	1.040	3°	16°	50°	16°	1	1	1900 profile scarp 75'back
4	.972	3°	c=144 ϕ=10°	53°	c=144 ϕ=10°	1	1	1915 profile
5	.996	3°	12°	53°	12°	1	1	1915 profile
6	1.060	3°	9°	53°	9°	1	1	1958 profile
7	1.001	≃T3,T2,T1 5°	10°	53°	10°	1	1	1958 profile

Note: Cohesion in psf



STRATIGRAPHIC PROFILE OF GRIERSON HILL SLIDE

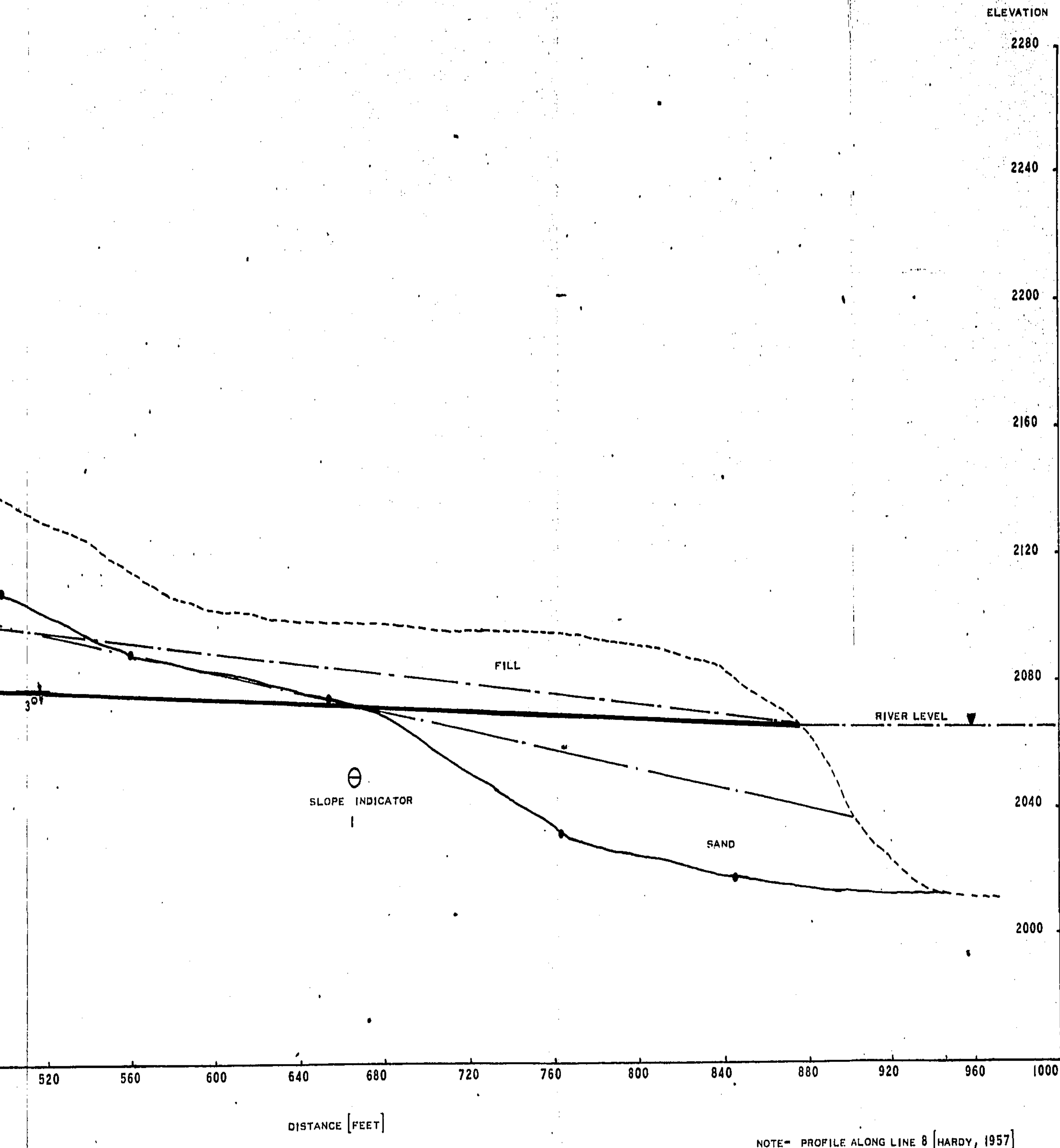

ELEVATION
2280
2240
2200
2160
2120
2080
2040
2000
FILL
RIVER LEVEL
SLOPE INDICATOR
I
SAND
3
520
560
600
640
680
720
760
800
840
880
920
960
1000
DISTANCE [FEET]
NOTE- PROFILE ALONG LINE 8 [HARDY, 1957]

FIGURE VI-4

CHAPTER VII

DISCUSSION OF STABILITY ANALYSES

7.1 SHEAR STRENGTH PARAMETERS

The undisturbed residual strength parameters have been utilized for all stability analyses. The justification for this was given in Chapter IV and was based primarily on the effect of failure plane irregularities. Although remolded soil was present along the failure plane of the undisturbed specimens the strength was higher than for the samples prepared from completely remolded material. Results of all stability analyses indicate that all developed shear strengths were greater than the strengths obtained by utilizing the pre-cut parameters.

This study had generally indicated the presence of residual cohesion in the bentonitic soils for the preslide and immediate post-slide slopes and the individual analysis show the sensitivity of the factor of safety to residual cohesion.

Retaining of residual cohesion is contradictory to most investigators including Skempton (1964) and Henkel and Yudhbir (1966). Generally, cohesion is disregarded because of its low value with the anticipation that it will ultimately decrease to zero and in some cases the misconception that the small cohesion has negligible effect on stability. The results of failure surfaces 4 and 5 for Grierson Hill (Table VI-2) show that when the cohesion equals 144 psf (1 psi) and the angle of internal friction equals 10^{o}, the factor of safety is similar to that obtained using an angle of internal friction of 12^{0} and zero cohesion. In this case one psi cohesion produces the same shear strength as an internal angle of friction equal to two degrees.

Research should be undertaken to ascertain the factors which produce the relative decreases in the cohesion and angle of internal friction as a specimen is sheared to the residual strength. Skempton (1964) reports that an orientation of particles and an increase in water content occur as the strength decreases. The effect of the above two factors and failure plane irregularities on both cohesion and angle of internal friction would be of interest.

The author appreciates the argument that the cohesion may decrease from values obtained in the short-term laboratory test to negligible magnitudes over long geological time but for slides of less than 60 years such as Lesueur and Grierson Hill, the presence of residual cohesion is realistic.

7.2 STABILITY ANALYSES OF SLIDES

Dunvegan and Peace River slides occurred when highway fills disturbed the quasi-equilibrium state of the old landslide areas. The developed shear strength was consistent with the undisturbed residual angle of shearing resistance in both cases. The residual factor becomes 1.0 and confirms reports by Bjerrum (1966), Skempton (1964) and Morgenstern (1967) that quasi-equilibrium slides which have undergone large strains exhibit residual stresses.

The Taylor slide occurred in a clay shale which appears to be relatively homogeneous as compared to the Edmonton Formation Cretaceous strata consisting of sandstone, shale and bentonitic seams. Progressive failure as outlined by Bjerrum (1966) resulted in a decrease to the residual strength along the horizontal failure plane as well as along the inclined scarp. The development of the residual strength along the scarp for the Taylor Shaftesbury clay shale is opposite to that of the Edmonton Formation where the peak was found to be present along the scarp. This difference is believed the result of progressive failure along the highly bentonitic soils which results in instability conditions before progressive failure begins along the scarp. The slight change in the inclination from preslide to postslide conditions is indicative of the small change in shear strength and a residual factor of 1.0.

Both the Grierson Hill and Lesueur slides are classed as slides in which progressive failure has occurred along a bentonitic soil. Failure occurred along the near horizontal surfaces and analyses indicate that the peak strength of the material acted along the scarp. Subsequent to the primary movements, the inclinations decreased and analyses indicate that the residual parameters are applicable. In both cases residual cohesion of 144 psf was applied to obtain a factor of safety equal to one. Additional slope degradation of these two slide areas has resulted since the slides occurred. This flattening may be related to further strength decrease. If strength decrease is not the reason for the continual decrease in slope then such phenomena as creep and secondary movements within the original wedge are present. If a strength decrease is not occurring then a rational analysis using the immediate postslide conditions is not possible. Hayley (1968) from field slope indicators at Little Smoky slide has shown that several shear zones are present within a slide which has undergone progressive failure. His analysis of separate blocks moving along a horizontal failure plane is inconsistent with an overall slope inclination of 8 degrees and a residual angle of shearing resistance of 14 degrees. An alternate interpretation of the slope indicator data would disclose that as a result of the amount of the re-adjustment which has occurred in the slide area plus relative movement between blocks a modified infinite slope situation has resulted. One slope indicator exhibited creep phenomenon with greatest movement at the surface.

An extensive study by Gould (1960) of slides and creep in California Tertiary deposits has disclosed a wide spread existence of "mass creep" occurring at depth under the influence of gravity forces.

The Lesueur and Grierson Hill slopes may be undergoing creep which will not allow a rational design unless the residual cohesion is assumed to disappear as the slope inclination decreases.

The University of Alberta riverbank slope section 6-6 which was reported by the Department of Civil Engineering (1968) has attained an average slope of 8.5^{o}. The reported residual angle of friction for the bentonitic clay shale was 8.5^{o}. This is a quasi-equilibrium slope in the Edmonton Formation and is an example of the long-term slope. An infinite slope was utilized in the analysis of the slide, although it is quite apparent that numerous slump blocks exist and that the failure of the area occurred in a progressive manner.

The University of Alberta riverbank slope section 2-2 which was also reported by the Department of Civil Engineering (1968) exists at an average inclination of 28 degrees and is presently stable. Although the slope is underlain by bentonitic soils its stability can be attributed to the 130 feet of Saskatchewan Sands and Gravels which provide drainage as well as increased strength.

7.3 RESIDUAL FACTOR

The residual factor as defined by Skempton (1964) requires clarification when applied to the soils and landslides of this study. Skempton in his analyses assumed the residual strength to consist of zero cohesion and the minimum angle of internal friction. The minumum strength would be that obtained from pre-cut samples. If the actual residual strength is greater than this, residual factors less than one will be incorrectly indicated. Therefore, a decision must be made as to which test represents the residual parameters in the field. It appears that a general conclusion for all soils is not possible. For example, the lowest residual angle of 7.5° for the Lesueur bentonitic soil was obtained from a remolded sample whereas, the pre-cut condition appears to be the lowest for most others.

Many of the slides investigated by Skempton were shallow infinite slopes in which the strength along the inclined scarp surface was negligible. The depth to length ratio of the Lesueur slide is approximately 1:2.5 which places considerable importance on the strength developed along the scarp and the interpretation of the residual factor. The residual factor is related to the preslide condition and refers to the decrease in shear strength of the soil undergoing progressive failure. The Lesueur analysis which indicated peak parameters along the scarp with $\phi'_r = 14^{\circ}$ and $C'_r = 0$ or $\phi'_r = 10^{\circ}$ and

$C_r' = 288$ psf along the horizontal bentonitic soil for equilibrium should have a residual factor based only upon the strength of the horizontal layer. The problem again arises as to the correct residual strength. By using $C_r' = 0$ and $\phi_r' = 10^o$, a residual factor of .69 was obtained when considering only the horizontal failure plane. If the strength along the scarp is included then a residual factor much less would be apparent. If $C_r' = 144$ psf and $\phi_r' = 10^o$ the residual factor is .87.

An excellent example of the importance of the strength developed along the scarp was given by the Department of Civil Engineering (1968) in analysis of the stable section 2-2 of the University of Alberta riverbank. This 180 feet section consisted of lacustrine sands and clays, till and Saskatchewan Sands and Gravels to a depth of 130 feet. The strengths which are assumed for these materials influence the conclusions derived for the strength of failure surfaces in the bentonitic clay shales. The critical failure surfaces using the wedge method of analysis and an average angle of internal friction of 33^o for the overburden disclosed that stability could not be confirmed using only the residual angle of 8.2^o. It was necessary to employ 4 psi cohesion. The computer results for this slope compare favourably to previous analyses as indicated in Table VII-1. In order to apply the computer analysis to a circular surface the failure surface was approximated by four straight lines. The factor of safety was found to be very similar to Bishop's method as summarized in Table VII-1.

TABLE VII-1

COMPARISON OF ANALYSES FOR SLOPE 2-2 UNIVERSITY OF ALBERTA

Failure Surface	Factor of Safety by Computer	Factor of Safety by other Methods	Strength Parameters
Wedge-Scarp at 40^o	1.372	1.27 (Wedge)	Scarp $\phi = 33^o$ Horz. Failure P. $\phi = 27^o$
Wedge-Scarp at 40^o	1.109	1.08 (Wedge)	Scarp $\phi = 33^o$ Horz. Failure P. $\phi = 8.2^o$ C = 576 psf
Wedge-Scarp at 60^o	1.179	1.10 (Wedge)	"
Circle-260 feet Radius	1.092	1.08 (Bishop)	Peak Parameters

An approach by Conlon which was cited by Peck (1967) indicates that because the normal stresses in a slope are variable, the peak strength for various portions occurs at different strains. Skempton used the average normal stress when evaluating the residual factor. This simplified approach is no doubt justified in shallow slopes but in deeper slides will produce incorrect residual factors. Of course, to utilize Conlon's approach the displacement along the failure surface must be known and the assumption must be made that the slide consists of one block. It is obvious that the peak strength cannot be mobilized simultaneouly along the entire surface of sliding and therefore, the residual factor will always necessarily be greater than zero.

7.4 GEOMORPHIC ASPECTS OF LANDSLIDES

The landslides which occur along the river valleys constitute an important degradation process. The modes of failure and causes of landslides are of interest to the engineer in order to perform realistic stability analyses.

The modes of failure for the slides in this study are of two types including the single block or wedge slide including the Lesueur, Grierson Hill and Taylor slides and the infinite slope type at Peace River and Dunvegan.

Naismith (1964) described a "mature" and "youthful" landslide profile and recognized it as representing different stages in the development of the Meikle River valley in Northern Alberta. The "youthful" slides consisted of glacial till slump blocks which occurred during initial river erosion at the toe of 35^{o} slopes. At the present time " mature" slides occur when the lower slope is at a slope of 12^{o} to 17^{o}. After movement of the lower slope continues for a time, the main scarp in the "mature" slide is oversteepened to 35^{o}. However, failure of the main scarp does not occur by rotational slumps; rather, weathering and frost action produce shallow five foot earthflows.

Naismith's description of the landslides in the Meikle valley indicates the necessity for elucidating the geomorphic history of the area. The analysis in this study, particularly Lesueur and Grierson Hill, demonstrate that if a continual decrease in the inclination of the slope occurs after failure a decrease in strength must be assumed to obtain a factor of safety of 1.0.

The causes of landslides are normally classified as fundamental or immediate (Terzaghi, 1950). The fundamental causes of the Taylor, Lesueur and Grierson Hill slides were the decrease in strength and the continual river downcutting which promoted progressive failure as well as some steepening of the slope. The immediate cause of the Taylor slide was the high precipitation which increased the piezometric head. The re-activitation of the slides at Dunvegan and Peace River resulted from the placement of roadway fills and the cause would be classified as immediate.

7.5 SLOPE STABILITY CHARTS

It is common practice (Lane, 1961) to utilize slope stability charts to ascertain design slopes for large projects. A slope chart consists of a plot of slope heights versus slope inclinations. A slope chart is illustrated by Figure VII-1 in which are included all available documentated slides in the study area. It should be emphasized that a slope chart should include slopes of similar materials and geological origin in order to resolve the safe slope angles. The slope chart in Figure VII-1, which is composed of slopes in soils of varying characteristics, cannot be used in the conventional sense but it provides an over-all view of the slopes in the area.

Two groups of slides appear to exist. One group includes those which have heights from 180 to 320 feet and lengths from 1200 to 2200 feet. These slopes are inclined at an angle near the residual angle of shearing resistance. Included in this group are Dunvegan, Peace River and the University of Alberta section 6-6. When analyzed as infinite slopes, these slopes produce a factor of safety of 1.0 if residual strength is employed. The Little Smoky slide is in this group and has previously been interpreted by the author as behaving similar to an infinite slope. This group appears to have degraded to an infinite slope consistent with the residual angle of internal friction.

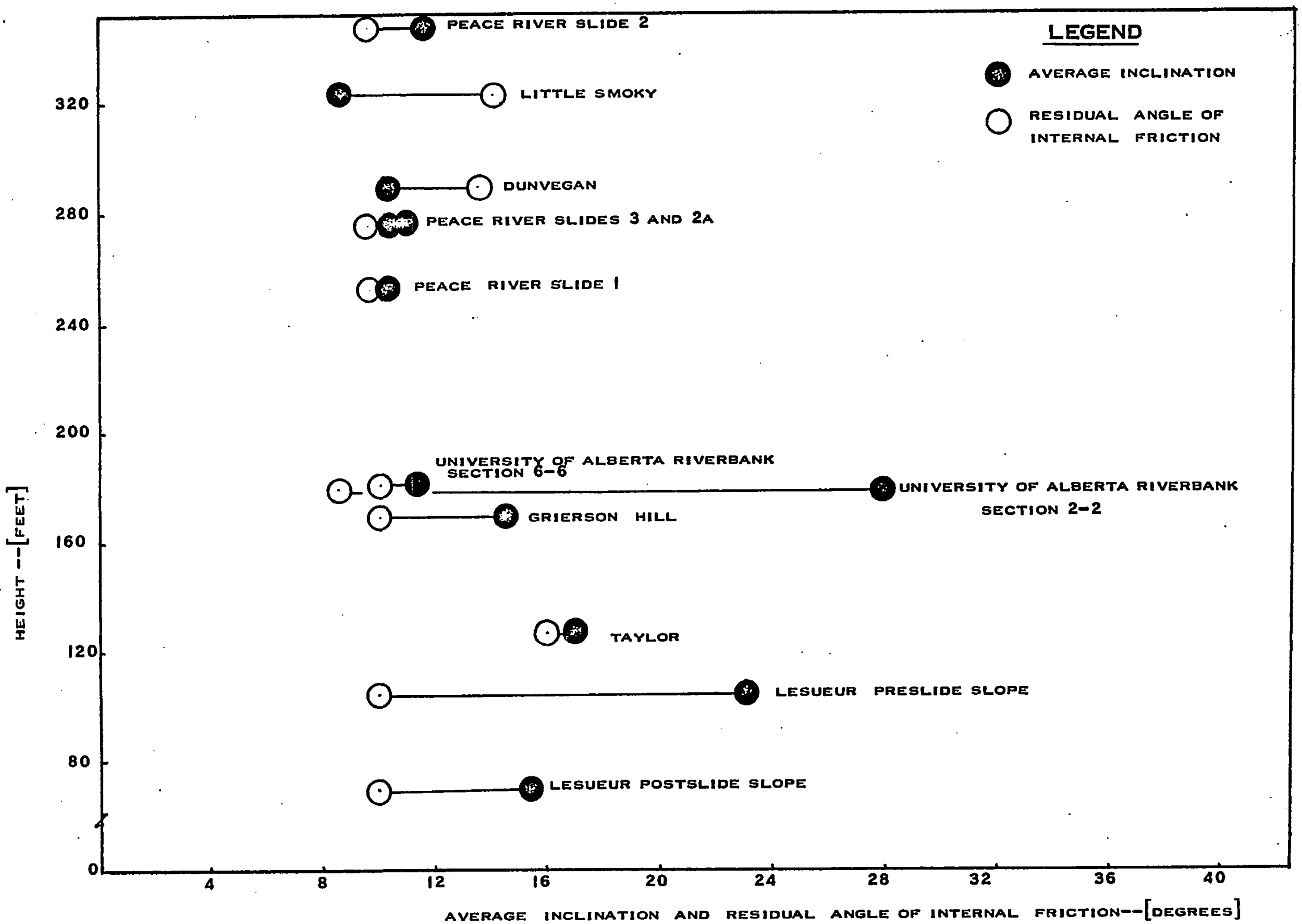


FIGURE VII-I HEIGHT VERSUS AVERAGE INCLINATION AND RESIDUAL ANGLE OF FRICTION

These slide areas would be considered "mature" relative to the second group.

The second group of slides included the typical one block type slide of Lesueur, Grierson Hill and Taylor. These are recent slides and for Lesueur and Grierson Hill, the postslide inclination is greater than the residual angle of shearing resistance. Eventually, additional slides may occur by progressive failure until a long flat slope develops similar to group one.

Figure VII-2 indicates the relationship between average slope and slide length. The length is probably related to the age of the slope or to the rate of river downcutting.

A summary of slope inclinations, heights, lengths and residual angles of shearing resistance are given in Table VII-2.

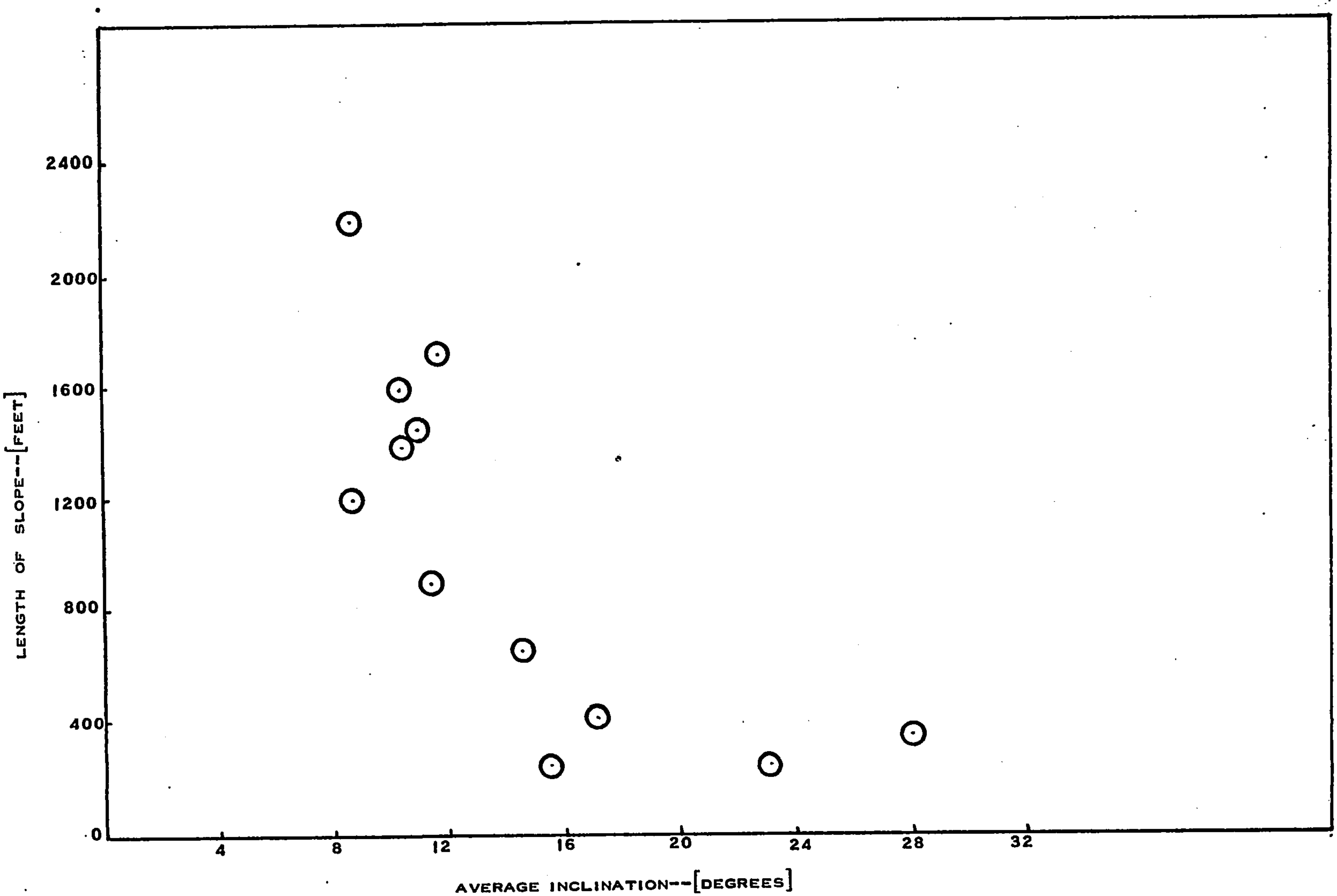


FIGURE VII-2 LENGTH VERSUS INCLINATION OF SLIDE AREAS

TABLE VII-2

SUMMARY OF SLOPE INCLINATIONS

Slide	Average Slope	Maximum Slope	HT.	Length	ϕ'_R
Lesueur(preslide)	23°	35°	105'	250'	10°
(postslide)	16.7°	35°	75'	250'	
Grierson Hill					
(1958)	11.3°	20°	180'	900'	10°
(1915)	14.5°	23°	170'	660'	
U. of A. 2-2	28°	36°	180'	360'	8.5°
U. of A. 6-6	8.5°	20°	180'	1200'	8.5°
Little Smoky	8.5°	20°	325'	2200'	14°
Dunvegan	10.3°	16°	290'	1600'	13.4°
Taylor	17°	28°	128'	420'	16°
P.R. 1	10.3°	20°	255'	1400'	9.6°
P.R. 2	11.4°	16°	350'	1730'	9.6°
P.R. 2A	10.8°	16°	275'	1450'	9.6°
P.R. 3	10.5°	15°	275'	1480'	9.6°

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS FROM THE LABORATORY INVESTIGATION

(a) Slaking and wet-dry cycles may be used to determine qualitatively the relative strength of diagenetic bonds and thereby, indicate whether the soil is susceptible to progressive failure. The test does not indicate the amount of stored energy that will be liberated unless the stress history is known and controlled.

(b) Particle orientation along the failure planes of the undisturbed samples was slight and was not a contributing factor in the shear strength of the materials in the study area. The decrease in strength from the peak to residual strength may be a result of the destruction of the original structure. Other investigators have attributed the decrease in strength from the peak to residual to orientation of clay particles along the failure plane.

(c) The drained peak strength parameters of this study were similar to those obtained in other investigations in which the triaxial test was utilized. Practically, the direct shear test may be used to advantage because of its simplicity.

(d) The residual strength parameters utilized for design purposes should be obtained from undisturbed samples. The results obtained from pre-cut samples are lower than the undisturbed because the irregularities are destroyed. Remolded specimens produce parameters which may be equal to or less than the undisturbed parameters. The deviation of the remolded and undisturbed residual strengths depends upon the remolding effects.

The increase in residual angle of friction for undisturbed specimens is directly related to the inclination of the irregularities. These irregularities were observed to be very flat.

(e) Residual cohesion has been shown to be present for most of the soils examined. The residual cohesion is related to the irregularities along the failure plane. Evidence indicating that cohesion is produced by irregularities is given by the remolded and pre-cut samples which possessed negligible cohesion and irregularity along the failure plane.

(f) The residual angle of shearing resistance for natural soils can be related to per cent clay fraction, liquid limit, montmorillonite content based on per cent total sample, montmorillonite based on per cent of clay fraction and activity. The residual angle decreases curvilinearly with the above factors except for the activity for which it decreases as activity increases.

(g) The residual angle of friction increases with density. This phenomenon is partially explained by the increase in density with decreasing montmorillonite content.

(h) Decrease in peak angle of friction with increasing normal stress for soils is related to a change in mode of failure. The change in mode of failure results when irregularities are sheared across rather than around.

(i) An increase in strain rate of 25 times from 1.38 inches per day to 34.56 inches per day resulted in an eight per cent increase in the residual shear stress for Dunvegan clay.

8.2 CONCLUSIONS FROM THE STABILITY ANALYSES

(a) Residual strength parameters must be employed in the analysis of the clay shales and preconsolidated clays in order to obtain a factor of safety of one for limiting equilibrium. Peak parameters produced factors of safety much greater than one.

(b) Numerically small residual cohesion must not be neglected in stability analyses. Analysis of Grierson Hill indicated that one psi cohesion contributed the same strength as an increase in the internal angle of friction of two degrees.

(c) Dunvegan and Peace River slides are re-initiated quasi-equilibrium movements in pre-till materials. The residual factor for these cases is one.

(d) Taylor slide is a wedge type slide in which the residual factor is one. The slide occurred when the shear strength had been reduced to the residual along the scarp and horizontal failure plane.

(e) The Lesueur and Grierson Hill slides took place in bentonitic soils of the Edmonton Formation. Major movements took place on surfaces in which the peak strength was developed along the scarp and the strength developed along the main failure plane was near the residual. Immediately after the slide, the strength developed along the entire failure plane was found to be composed of one psi cohesion and the residual angle of shearing resistance.

It is postulated that slides which develop in bentonitic soils decrease to the residual strength rapidly and failure occurs when the peak strength is still present along the scarp.

(f) Slides which exhibit a residual factor of 1.0 after failure cannot be analyzed rationally if the inclination of the slope decreases without further strength decrease. The degradation of the slope may be produced by soil creep or by secondary slumping within the slope. In mature slides which occurred shortly after deglaciation solifluction may have been active in the periglacial environment.

(g) The long-term slope inclinations for the slopes in the study area are near the residual angle of shearing resistance. Of course the ultimate slope is dependent upon the ground water table and soil types overlying the slide material.

(h) The piezometric level which exists along the North Saskatchewan River Valley is low relative to the bank height and is related to the river water levels. Caution must be exercised in detecting the presence of perched water tables.

(i) The residual factor requires clarification with regard to the various methods of determining the residual strength parameters and the treatment of the strength developed along the scarp.

(j) The slides in the study area appear to fall into two groups. One group consists of long flat slopes and is indicative of "mature" slides in which many numerous progressive block failures have occurred. The second group consists of "youthful" slides in which river erosion is active and the typical wedge failure is prevalent.

(k) The slides in the study area generally all exhibited a residual factor near one. All occurred in preglacial river valleys. It is postulated that water has been available to the slide materials in these valleys for a long time and a strength decrease has occurred from the peak to the residual strength. Highway fills, pore pressure increases and toe erosion cause the residual strength to be exceeded.

8.3 RECOMMENDATIONS FOR FUTURE RESEARCH

(a) A study should be initiated to compare various laboratory techniques for evaluating the residual strength parameters. Of greatest significance would be a comparison between the reversal direct shear test and a ring shear apparatus. The ring shear would allow large displacements in one direction and area corrections could be eliminated. No doubt, sample preparation for a ring shear test from the clay shales will prove difficult.

The limited reported information with regard to residual strength evaluation discloses variations which appear to be related to test technique.

(b) The effect of irregularities and structure should be studied in detail. Initially, irregularities may be precast by laboratory remolding of natural soils and then the study could be extended to undisturbed soils. Shear structures developed at various normal stresses may add to our understanding of curvilinear Mohr envelopes. This study would necessitate the microscopic study of thin sections.

(c) A detailed investigation of residual cohesion would be of interest. The study should comprise the separation of internal friction angle and cohesion at various strains and the dissipation of cohesion with time.

(d) Future investigations of slide areas should take account of the profile change and slope movements which occur subsequent to landslides.

(e) A study of existing failed and stable slopes is considered to be of immense practical importance. This study would involve determination of slope inclinations, slope heights, slope lengths and the extent of river erosion. For a meaningful study the depth and character of soils above the potential slide material must be ascertained. This information is often derived from crop outs but where slumping is prevalent a rapid drilling procedure would provide stratigraphic data. Adjacent stable and failed slopes should be investigated in the study area to ascertain the reasons for stability of some slopes and failure of others.

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