30854

National Library of Canada | Bibliothèque nationale du Canada | CANADIAN THESES ON MICROFICHE | THÈSES CANADIENNES SUR MICROFICHE

NAME OF AUTHOR/*NOM DE L'AUTEUR* Robin Wesley Tweedie

TITLE OF THESIS/*TITRE DE LA THÈSE* An Investigation of the Edgerton Landslide, Wainwright, Alberta.

UNIVERSITY/*UNIVERSITÉ* University of Alberta.

DEGREE FOR WHICH THESIS WAS PRESENTED/ *GRADE POUR LEQUEL CETTE THÈSE FUT PRÉSENTÉE* M Sc

YEAR THIS DEGREE CONFERRED/*ANNÉE D'OBTENTION DE CE GRADE* Fall 1976

NAME OF SUPERVISOR/*NOM DU DIRECTEUR DE THÈSE* Dr S Thomson

DATED/*DATÉ* Nov 15, 1976 SIGNED/*SIGNÉ* Robin Tweedie

PERMANENT ADDRESS/*RÉSIDENCE FIXE* #315 3868 Shelbourne Street Victoria, B.C.

NL-91 (3-74)

THE UNIVERSITY OF ALBERTA

AN INVESTIGATION OF THE EDGERTON LANDSLIDE, WAINWRIGHT, ALBERTA

by

© ROBIN WESLEY TWEEDIE

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

AND RESEARCH IN PARTIAL FULFILMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

FALL, 1976

THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled. "An Investigation of the Edgerton Landslide, Wainwright, Alberta" submitted by Robin Wesley Tweedie in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering.

S. Thomson

S. Thomson,

Supervisor

Z. Eisenstein

N. W. Rutter

DATE ...Nov. 1........ 1976

TO MY MOTHER

ABSTRACT

The Edgerton Landslide, which occurred in 1974 in the glacially modified valley of the Battle River, is located approximately 30 miles northeast of Wainwright, Alberta.

Geologically, the site consists of Upper Cretaceous clayshales overlain by a thin veneer of till. Both sides of the valley, in the vicinity of the landslides are characterised by extensive landsliding, which has flattened the valley to the present day slopes of approximately 11°.

A site investigation was carried out to establish the soil profile, and in addition slope indicators and pneumatic piezometers were installed to monitor ground movements and pore pressures in the soil mass. Measured pore pressures were low, however an increase in pore pressure corresponding to a rise in water table of about 10 feet was recorded in early spring 1976, coinciding with spring runoff. The horizontal portion of the failure plane was located by means of the slope indicators, and consisted of soft bentonitic clayshale. Index tests on this material indicated a clay fraction of 76%, of which 75% was composed of montmorillonite, as reflected in the liquid limit of 143%. Direct shear tests on the failure plane material gave a residual angle of shearing resistance of 8°.

Numerical stability analyses, performed on the incipient South Slide, indicated that the slide was

mobilising residual strength parameters along the horizontal portion of the failure plane, and peak angles of shearing resistance but essentially zero cohesion in the steeply dipping backscarp.

It is proposed that the recent slides are the continuation of a retrogressive series of slides which has been responsible for the ancient landslide topography at the site. It is postulated that the site has been only marginally stable for many years and that catastrophic failure resulted from a very gradual loss of cohesion in the fissured clayshales of the backscarp due to the process of softening, and was finally triggered by a rise in pore pressure induced in the slope following the abnormally high spring thaw in 1974.

ACKNOWLEDGEMENTS

The investigation reported in this thesis was carried out at the Department of Civil Engineering, University of Alberta, under the direct supervision of Professor S. Thomson. The author wishes to express his sincere gratitude to Professor S. Thomson for suggesting this topic for research, and for his continual guidance and support during all stages of this study.

The author also wishes to thank Professor N. R. Morgenstern for his invaluable comments and discussion of the project.

The author is very grateful to the many graduate collegues who assisted with the field programme, and especially to Messrs. I. Bruce, D. Hill and M. Milligan who assisted during the site investigation.

The author thanks Messrs. O. Wood, A. Muir and their staff for their invaluable assistance during the field work and the laboratory testing programme.

Financial assistance received from the University of Alberta is gratefully acknowledged .

The author also thanks the Natural Research Council of Canada and the Transportation and Surface Water Engineering Division, Alberta Research Council for providing financial assistance during all phases of research.

Gratitude is extended to Professor J. Dudas, Department of Soil Science, and Mr. L. Medeiros for performing the mineralogical and physico-chemical tests.

Access to the study area by Messrs. G. Koroluk and J. Chesterman is gratefully acknowledged.

Finally, the author wishes to acknowledge the excellent job of data entry by Lorraine Iwaskow at Computing Services.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER I

INTRODUCTION

1.1 Strength Characteristics of Slopes in Overconsolidated Clays and Clayshales

In his classical paper on the stability of slopes in natural materials Terzaghi (1936) distinguished between soft and stiff clays, and delineated between these on the basis of the Liquidity Index (L.I.), suggesting that most stiff clays have L.I. less than 0.5. It has also been observed that most clays in engineering practice with low L.I. are overconsolidated and fissured (Morgenstern, 1967). It is this group of clays which is most troublesome to the practicing engineer, for although peak effective strength parameters are considered to represent the shear strength of a stiff fissured clay, it has been recognised for some time that the average shear strength mobilised in slides in this material is well below peak strength. The engineer must, therefore, consider many factors before applying suitable strength parameters when designing long-term slopes in stiff fissured clays.

The first explanation for this apparent decrease in strength, was offered by Terzaghi in 1936. He suggested that in stiff fissured clays, the removal of lateral support resulting from excavation, could cause some opening of the fissures. The strength of stiff clays is sufficient to keep a fissure open, even at great depths. Ingress of ground or rain water initiates softening along the sides of the open joints which leads to a reduction in average strength, allowing more deformation to occur. More fissures then open up, and the process continues indefinitely, or until slope failure occurs. This theory was supported by Cassels (1948) and also Skempton (1948) who stated

> "unless slip intervenes, the end product of such a softening process must be a clay reduced essentially to its normally consolidated condition."

Since this early account, the stability of slopes in stiff fissured clays has received considerable attention in the literature. Particular attention was paid to the wide discrepancies between measured laboratory strengths of such soils, and the strengths at which they fail, as computed from field evidence.

In general most early studies were necessarily limited to 0 = 0 analyses, and a more fundamental approach became possible only after the effective stress method of analysis was introduced in 1952 as discussed by Bishop (1955). Henkel (1955) and Bishop and Henkel (1953) analysed slope failures using the effective stress analysis and Gould (1960), in his work on highly overconsolidated clays of California found that the strength actually mobilised in slides, or mass creep, was inversely proportional to the amount of displacement which had occurred previously in the shear zone, and also stated that this reduction in strength with displacement was permanent. However, many case histories in the early 1960's could not be explained simply by the effective stress analysis.

In 1964 the breakthrough came with the Fourth Rankine Lecture, when the phenomenon of residual strength was formally stated and defined (Skempton, 1964). Previous failures mobilising exceptionally low values of c' and O' could then be explained. At this time, parameters for a number of case histories were expressed in term of an 'R' factor, defining a simultaneous decay in both strength parameters. In the years following this, there was a tendency to overemphasize the residual approach to the design of slopes, an overemphasis since knowledge of the actual path to the residual strength of a soil in-situ (the progressive failure mechanism) was still confused and largely hypothetical. Indeed, the behavior of soils at stages well prior to residual, where it might be argued that most slip occurs, was little investigated, possibly on the assumption that the parameters along the slip plane fell to residual immediately on failure.

Bjerrum (1966) postulated a mechanism for progressive failure, as being a result of the large content of "recoverable strain energy" in overconsolidated plastic clays. The conditions for this mechanism were as follows;

1. Unstable stress-strain curve.
2. Local stress concentrations at the foot of the slope.
3. Large movement due to release of locked in strain energy.

Yudbhir (1969) also concluded that the release of horizontal stresses in these soils is a dominant factor in explaining their instability. That is, Yudbhir considered the K effect to be a dominant factor in progressive failure.

The approaches of Bjerrum and Yudbhir, however, leave several questions unanswered, the main one being the amount of strain developed under this mechanism and the amount of strain energy required to produce residual conditions. Bjer (1966) cited several cases where stability analysis sugg ed that strength parameters being mobilised were $c' = 0$, $0' = 0$ ' and explained these cases in terms of his progressive failure mechanism. It has been recognised that these slides were all second time failures and hence cannot be considered definitive examples of this mechanism.

In this regard, Peck (1967) pointed out that

> "possibly the most important factor affecting our ability to predict whether or not a slide will start is whether we are in an old slide area".

This consideration has been supported by a number of case histories in which slope stability was clearly defined by old shear zones (Skempton, 1966; Esu, 1967; Leussink and Kirchenbauer, 1967; De Beer, 1969).

Bishop (1967) also proposed a mechanism based mainly on overstressing, and the formation of a zone of plastic equilibrium at one point in the slope. This zone of failure would then propagate along the potential slip surface, while within this zone the shearing resistance would drop from its peak to its residual state. He suggested that the drop in strength depends on the difference between peak and residual strength, which he described by the

$$\text{" Brittleness Index "} \quad I = \frac{S - S}{S}$$

Where S and S are the peak and residual strengths respectively.

Bishop and Lovenbury (1969) studied the longterm creep characteristics of overconsolidated London clay and normally consolidated Pisa clay under drained conditions. They showed that long term loading does not lead to substantial strength reductions, which suggests that there is no path to residual which by-passes the peak.

One of the most detailed contributions was by James (1970 and 1971) who presented an investigation of some 90 case histories most of which were in overconsolidated English clays. The majority of slides had failed with c' = 0 and 0' = 0 ' consistent with the mechanism of softening as proposed by Terzaghi (1936). Many of the slides which had failed at reduced c' and 0' were old slides, which had been reactivated. James also suggested that very large movements (in the order of several feet) were a pre-requisite for strength reductions in a slope from peak to residual. Bishop et al (1971) reiterated this view while describing investigations on post peak behavior of clays in a new ring shear apparatus. From the results obtained they concluded that residual strength conditions in clay can be reached only after very large strains. The initial loss in strength was explained by a destruction of cohesion due to dilatancy and the breaking of cementation bonds.

Such large strains would necessitate a pre-shearing of the clay and forming of principal shear planes, as described by Skempton and Petley (1967). This would be accomplished by such processes as landsliding, solifluction, tectonic movements, glacial ice movements, and also differential swelling of highly swelling clays constrained between non-swelling materials. Matheson (1972) also described a process of shearing of soils due to bedding plane slip, associated with anticlinal rebound in reponse to valley formation.

James' observations prompted Skempton (1970) to discuss again first time slides in overconsolidated clays. He described the post peak changes in strength as comprising two successive stages,

" a) Dilatancy and the opening of fissures leading to increases in water content and culminating in a drop in strength to the fully softened value, at which stage there is a softened shear zone with numerous discontinuous shears.

b) Development of principal shears of appreciable length, some of which eventually link together and form a continuous shear, when the residual strength is reached along the entire slip surface."

Skempton concluded that the strength of first time slides in London Clay cuttings tends towards, and does not fall significantly below, the fully softened value. He did not however preclude the possibility that strength reduction prior to a first time slide is more substantial in some materials as proposed by Bjerrum (1966), although James (1970) suggested that this could be obtained only where deformations were very much localised, for example, along one thin layer, or at the interface between two different layers.

1.2 A Review of Some Relevant Case Histories

The Western Plains of Canada is an area of generally low relief, consisting predominently of highly overconsolidated shales, sandstones and coal beds of late Cretaceous and early Tertiary ages. Landsliding in this area is ubiquitous, especially along the river valleys in the area and is typified by wedge shaped failures usually controlled by bedding features. Landsliding in the Upper Cretaceous bedrock of Alberta has been an area of active research of the Department of Civil Engineering, University of Alberta for a number of years, (Brooker, 1958; Hardy et al., 1962; Painter, 1965; Rennie, 1966; Hardy, 1967; Sinclair and Brooker, 1967; Hayley, 1968; Pennell, 1969; Thomson, 1970; Eigenbrod and Morgenstern, 1971). These slides have all failed mobilising strengths well below measured peak strengths. Hardy et al. (1962) showed that use of peak strength parameters led to factors of safety well in excess of unity when the slope was failing. Five of the larger slides were re-analysed by Pennell (1969) in view of the development of the concept of residual stress and of progressive failure, and he concluded that residual strength parameters must be employed to obtain a factor of safety of unity.

Two important case histories will be briefly presented, to illustrate typical landslides in Alberta.

Devon Slide (Eigenbrod and Morgenstern, 1971)

The Devon Slide, located about 12 miles west of Edmonton, occurred following a highway cut into the valley of the North Saskatchewan River in 1965 (Figure 1.1). The bedrock of the area is the Edmonton Formation, consisting mainly of Upper Cretaceous mudstone, claystone, siltstone and sandstone. Some coal seams and bentonite layers were also present. A stratigraphic section of the Devon slide is shown in Figure 1.2. A comprehensive field programme revealed that slip had occurred along a sub-horizontal bentonitic clay layer underlain by coal, while the back scarp was inclined at approximately 30-35° and passed through softened mudstone. A summary of index and strength properties of the soils are given in Table I.1.

Stability analysis using the general non-circular limit equilibrium analysis as described by Morgenstern and Price (1965) revealed that for a factor of safety close to unity, residual strength parameters, i.e. $c' = 0$ $\phi' = 8°$ in the bentonitic clay layer, and in the back scarp, the peak angle of friction, but with reduced cohesion, was being mobilised at failure.

It was observed that although the slide was a first time occurrence, it was actually a small part of the pre-existing large slip block which had most likely slid during valley formation. The back scarp was thought to have been a first time movement, however the presence of both softened and unsoftened material in the back scarp indicates that there was some reduction in cohesion, which is an important mechanism in this failure. Cracks in the slope prior to excavation also indicate that the slope was only marginally stable and that some reduction in shearing resistance may have been taking place.

Lesueur Slide

The Lesueur slide first failed in 1963, on the outside of a bend on the North Saskatchewan River, about 4 miles northeast of Edmonton. This slide also occurred in the bedrock of the Edmonton formation. A stratigraphic section of the slide is shown in Figure 1.3. Strength parameters for the slide materials are shown in Table I.2.

Thomson and Morgenstern (1974) suggested that the stability of the slope had been reduced by the annual erosion of the toe, by the river and that a minor rise in the groundwater table might have been the final trigger for failure. Analyses in terms of effective stresses were carried out using the method of Morgenstern and Price (1965), from which it was proposed that the slide was mobilising peak strength parameters along the backscarp, and strength parameters on average less than peak along the horizontal shear plane at failure. It was felt that strength decreased along the horizontal shear plane towards the river bank, consistent with the theory of valley rebound discussed by Matheson (1972).

Since the original slide, continuous but slow movements have increased the scarp height from the original 22 feet to 35 feet over a period of eight years. Subsequent stability analysis in 1971 (Thomson, 1971) indicated that during the interim period the cohesion everywhere had tended towards zero, and the angle of shearing resistance along the horizontal part of the slip surface had tended towards the residual angle, due to continued displacements.

1.3 Purpose of the Research

The two landslides, which have been collectively termed the Edgerton Landslide for the purpose of this thesis, are situated approximately 30 miles northeast of Wainwright in Central Alberta, longitude 11° 29' 20" W, latitude 53° 01' 05" N on the gently sloping west valley wall of the meandering Battle River (Figure 1.1).

Bedrock of the area is Upper Cretaceous in age, consisting of highly overconsolidated, interbedded clay shales and sandstones. This is overlain by a thin veneer of glacial deposits. Catastrophic failure of the North Slide occurred in late August 1974, and in September 1974, a preliminary reconnaissance was conducted by the Department of Civil Engineering, University of Alberta (Thomson, 1974).

The scarp was about 600 feet long and between 40 and 50 feet in height. Upper Cretaceous bedrock was clearly visible on the scarp face. Of particular interest was the fact that the toe of the North Slide cropped out slightly less than half way up the valley wall. This was significant in that it suggested bedrock bedding plane control along the lower slide surface, and also that the Battle River was too far away to act as a triggering mechanism for this slide.

Thus a case was presented of a natural slope, where the mechanism of failure was not immediately obvious. The slope angle of 11° did however suggest that strengths considerably less than peak were being mobilised along a large portion of the failure plane.

Extending approximately 500 feet southwards from the North Slide, a scarp 2 feet high had formed at the time of catastropic failure of the North Slide. This slope is referred to as the South Slide, and it was felt that this also would fail catastrophically in the ensuing year or so.

The Edgerton Slide thus offered an interesting and informative case history to study. The small movements of the South Slide had barely affected the slide profile, and hence the prefailure profile was available as is seldom the case, and it was felt that the slide could actually be monitored during failure.

The purpose of the investigation was, therefore,

1. To determine the boundaries of the sliding area.

2. To establish the overall subsurface profile of both slides and to locate the failure plane(s) especially in the incipient South Slide.
3. To obtain, and test, both undisturbed and remoulded samples of the representative materials, and determine the strength parameters of these materials.
4. To propose a mechanism of failure, and to verify this by means of numerical stability analyses.
5. To enhance predictive techniques for the stability of slopes in clayshales.

1.4 Organization of the Thesis

Chapter II of the thesis presents a general description of the landslide area. Included are the geologic history, geology, climate and precipitation records and also a preliminary description of the landslides.

Chapter III documents the site investigation of the study area, consistency of a topographical survey, subsurface exploration and instrumentation. Results and interpretation of the stratigraphical, piezometric and tiltmeter data are also presented.

Chapter IV outlines the laboratory testing programme which was undertaken.

Chapter V applies the results of the previous chapters and describes the stability analysis performed using these results. A mechanism of failure is also proposed, and discussed in view of the stability analysis.

Chapter VI contains the conclusions and recommendations of this thesis.

TABLE I.1

SUMMARY OF INDEX AND STRENGTH PROPERTIES OF THE DEVON SLIDE

(FROM EIGENBROD AND MORGENSTERN, 1972)

Material	w_N	w_p	w_L	I_p	I_L	%<2μ	c'	ϕ'	c' residual	ϕ' residual
	(%)	(%)	(%)	(%)		(%)	psf	degrees	psf	degrees
Bentonitic Clay	48	39.8	100	60.2	.136	47			0	8
Weathered Back Slope Clayey	31.8	20.1	84.8	64.7	.181	52	190	33		
Weathered Back Slope Sandy Silty	26.6	20.6	44.9	24.3	.247	32				
Unweathered Back Slope Sandy Siltstone	20.9	20.5	43.0	22.7	0	22	3720	19		

TABLE I.2

SUMMARY OF GEOTECHNICAL PROPERTIES OF THE LESUEUR SLIDE

(FROM THOMSON, 1971)

Material	W_L	I_P	% S	% Silt	% C
Bentonite	200-430	150-380	0-15	5	80-95
Clay Shale	100	75	20	35	45
Bentonite Shale	200	170	15	30	55

X-Ray Diffraction

Material	% Montmorillonite	% Illite	% Kaolinite & Chlorite
Clay Shale	80-100	10-0	10-0
Bentonite	100		

Material	Peak		Residual	γ Total
	C'	φ'	φ'	
Clay Shale	1300	24	17	114
Bentonite Shale	865	14	10	112
Bentonite	800	14	8	106
Coal	0	32	32	87

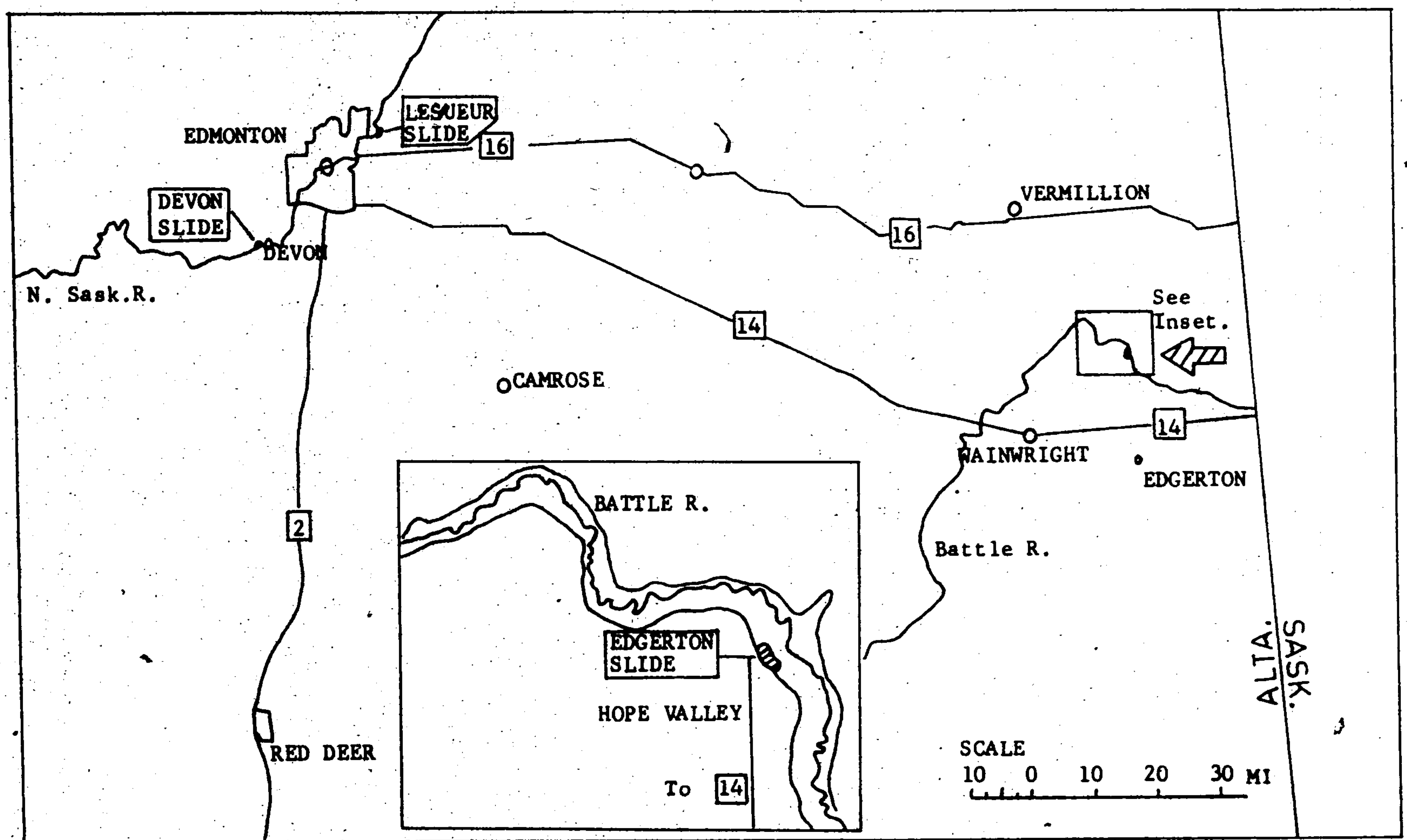


FIGURE 1.1 GENERAL MAP

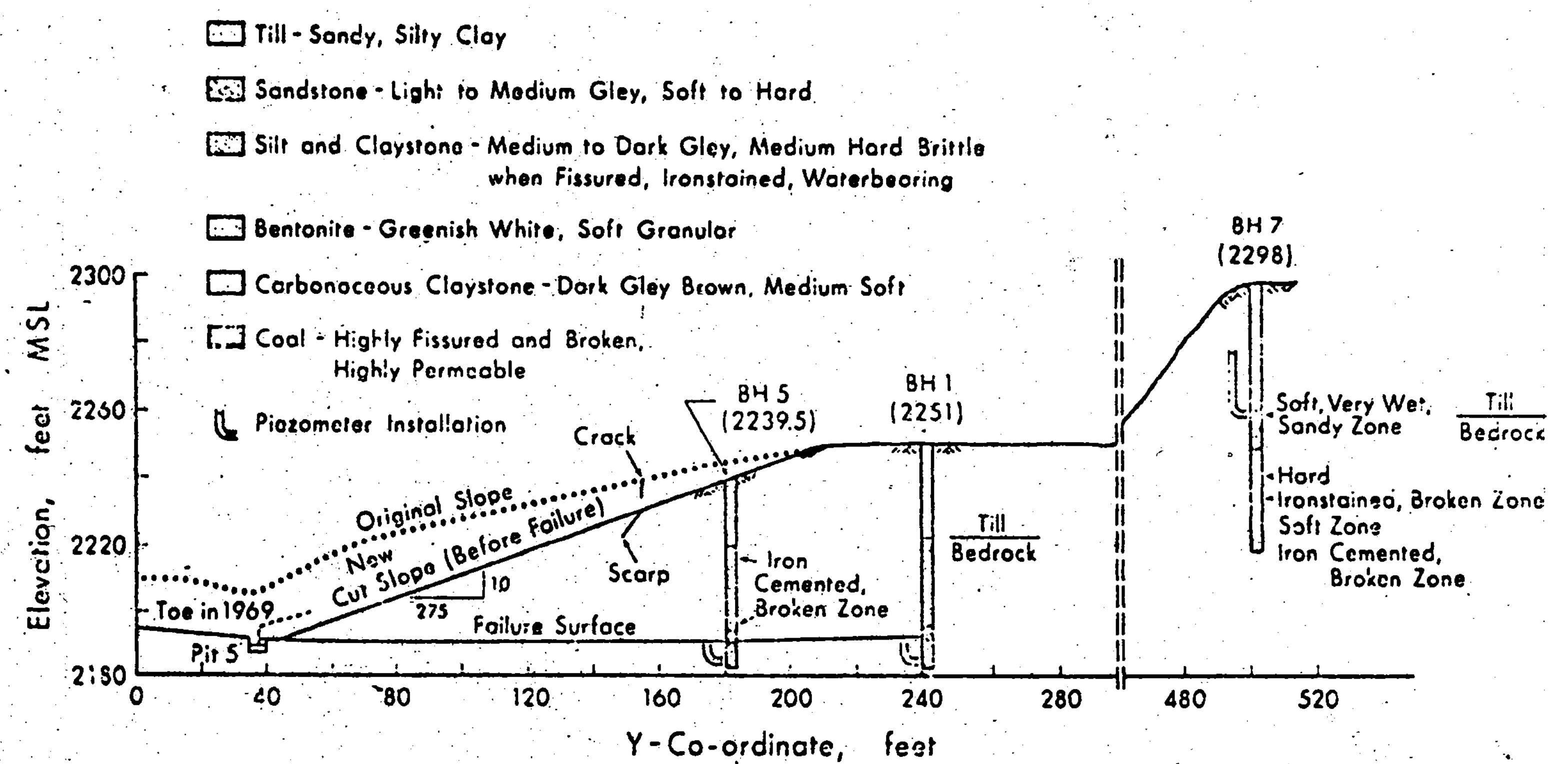


FIGURE 1.2 STRATIGRAPHIC SECTION OF THE DEVON SLIDE
(AFTER EIGENBROD AND MORGENSTERN, 1971)

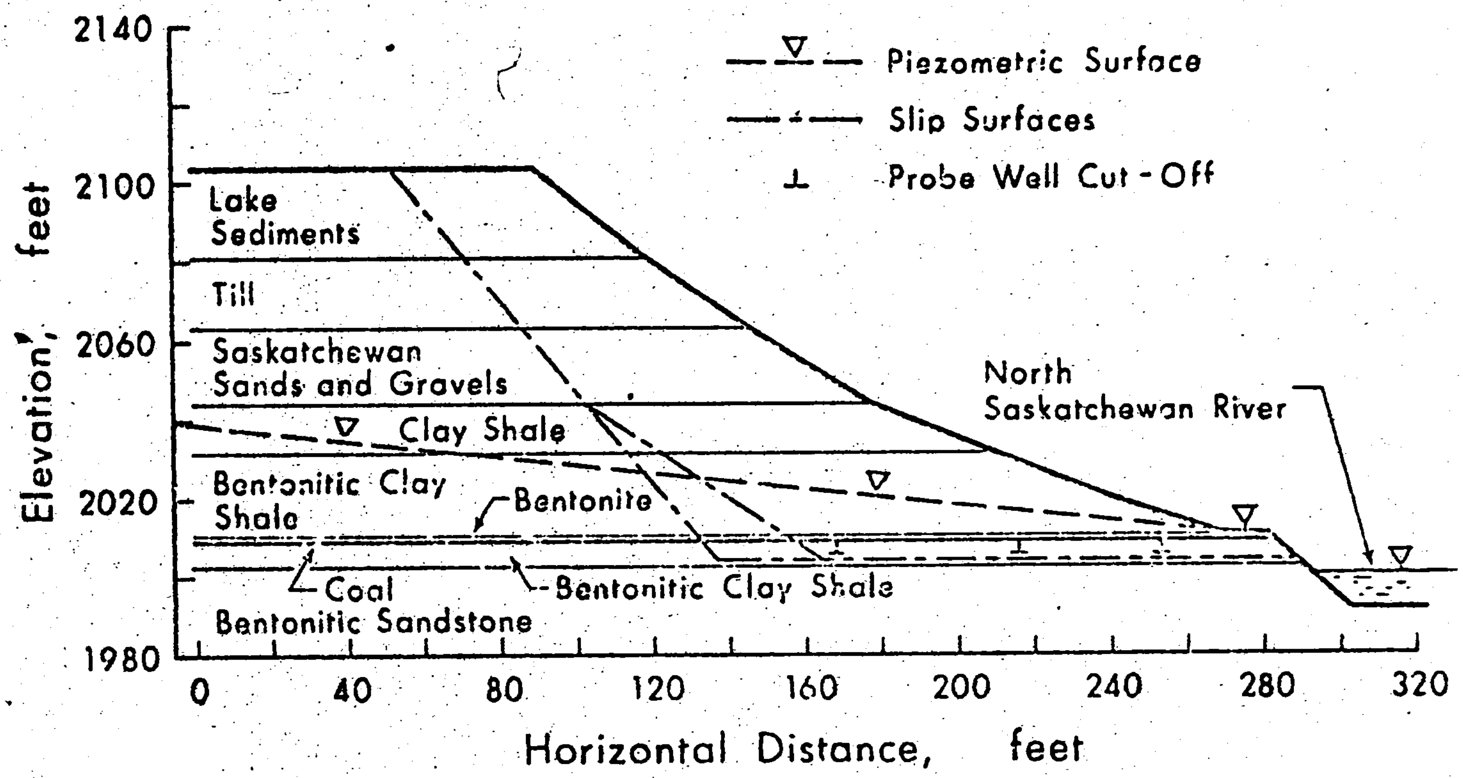


FIGURE 1.3 STRATIGRAPHIC SECTION OF THE LESUEUR SLIDE

(AFTER THOMSON, 1971)

CHAPTER II

DESCRIPTION OF THE SITE

2.1 Geologic History

Late Cretaceous - Tertiary bedrock formations of Central Alberta, form the upper part of a thick succession of clastic rocks which were deposited in a subsiding basin flanking highlands situated to the south and west of the present Rocky Mountains. These "highlands" provided the detritus which was transported by eastward flowing streams to the basin, over a depositional surface which sloped gently to the south and east. Volcanic ash was also transported from these highlands and was deposited widely in the Upper Cretaceous sediments, especially in the Bearpaw and Edmonton Formations of Southern and Central Alberta (Williams and Burke, 1964).

Stratigraphically, this portion of the basin is characterised by shale, siltstone and sandstone strata, which were laid down in or near the shallow epieric seas which covered the basin at this time. Vertical variation from marine shale at the base of the Upper Cretaceous, to deltaic and continental sandstone at the top is common and attests to the alternating transgressive and regressive nature of the epieric seas, caused by the continuously variable interplay of subsidence rate and sediment supply.

Thus each gross lithologic unit in the basin records an episode in a series of tectonic uplifts in the source region and concomitant downwarps in the marginal basin, spread over a period of more than 100 million years (Williams and Burke, 1964).

Sedimentation continued without interruption through the uppermost Cretaceous into Paleocene times resulting in Tertiary rocks that are almost wholly continental. During late Mesozoic and early Tertiary (Paleocene) times the Columbian and Pacific orogeny, resulting in the formation of the Rocky Mountains and Foothills, transformed the Alberta basin from an area of subsidence and deposition to uplift and erosion. Rutherford (1928) estimated that approximately 2000 feet of strata have been removed from the study area during Tertiary times, which has left only small scattered patches of Miocene and Pliocene fluvial deposits in isolated uplands. This prolonged period of erosion is largely responsible for the broad topographic outlines of the Alberta Plains as observed today.

Locally, however, the Late Tertiary landscape has been modified by Pleistocene glaciation. This area was glaciated by the Keewatin ice sheet which left a veneer of surficial deposits from a few inches to several hundred feet thick. The net effect of glaciation on the topography is variable. Glacial meltwaters have cut steep sided stream trenches into the underlying bedrock in places, and elsewhere preglacial valleys and other local depressions have been filled in by glacial deposits, resulting in a reduction in local relief.

It is believed that the present Battle River is a misfit stream, occupying a channel, part of which was carved by glacial meltwaters. However some of the sharp bends in the Battle River valley are related to the underlying geological structure, suggesting that these are parts of the preglacial channel, which was occupied by the preglacial Red Deer River. (Wyatt et al, 1944)

Subsequent to the retreat of the Pleistocene glaciers about 10,000 years ago, renewed erosion by larger streams and rivers has taken place, resulting in deeply incised valleys through the glacial deposits and bedrock. This process has been accompanied by much landslide activity which has gradually flattened these slopes towards their stable angles.

2.2 Geology

2.2.1 Surficial Geology

In the study area, glacial deposits of till have been deposited to an average depth of 15 feet. The till can be observed in the scarp face, and is brown, highly oxidised, columnar jointed, and composed largely of local bedrock. The average till composition is 50 per cent sand, 30 per cent silt, and 20 per cent clay sizes, the clay fraction containing up to 50 per cent montmorillonite. The gravel content of the till ranges between 2 and 3 per cent, and is derived from the Canadian Shield, and also from local post

Cretaceous - pre glacial gravels which overlie Cretaceous bedrock in scattered portions of the Alberta Plains. The gravel portion of the till contains from 30 to 50 per cent quartzite pebbles (Bayrock 1967).

2.2.2 Bedrock Geology

The bedrock of the study area, mapped by Warren and Hume (1939) consists of interbedded sandstone, siltstone and shale, and thin coal seams, of late Cretaceous age. The rocks are poorly indurated and bentonitic, and have a regional dip of a few feet per mile to the southwest.

The general area is underlain by three formations which are, from youngest to oldest, the Bearpaw, Belly River and Lea Park. The Bearpaw Formation, which is of marine origin, consisting mainly of shales has been completely eroded at the location of the site, thus the landslide movements have occurred through the Belly River Formation. The Belly River Formation consists of an interfingering succession of marine shales and deltaic sands, and can be subdivided into a number of members, as listed in Table II.1. Figure 2.1 shows the approximate ontcropping of these members along the river valley, from which it would appear that the Pale and Variegated Beds (Oldman), the Birch Lake, and the Grizzly Bear crop out on the valley wall. The contact between the Birch Lake, and the Grizzly Bear also appears to be approximately halfway between the river and the local plain level.

The bedrock has been weathered to depths of approximately 40 feet and can be observed on the scarp face of the North Slide, as well as in all the drill holes. Ice thrust features, as typified by intense brecciation and slight warping of the bedding planes, were observed over the full height of the scarp face.

2.3 Aerial Considerations

Complete aerial photo coverage is available for the area under consideration from the Department of Lands and Forests in Alberta. The relevant photo numbers for this stretch of the river are 4390. 5301. 195 to 4390. 5301. 200. In addition an aerial reconnaissance was undertaken by the Department of Civil Engineering, University of Alberta in October 1974. The following points are of note from the air photo interpretation and from the aerial reconnaissance observations:

1. Groundwater discharge areas were found along both sides of the river valley, all occurring at approximately the same elevation. These springs usually occur half way between the river and the local plain level, and may indicate the presence of a perched water table, held perhaps by a low pemeability shale bed or bentonite seam. These areas are commonly accompanied by headward erosion and minor slumping immediately above the discharge points, which has caused the development of a slight terrace. Inactive discharge areas were also observed at the time of this reconnaissance, which

combined to form this terrace.

2. The "slump" topography along both sides of the river valley is indicative of ancient landsliding, subsequent to valley formation. Old grabens can be observed along the valley walls, in the vicinity of the slide area.

3. Toe erosion was noticed at many points along the river valley, and especially at the location of the landslide, where the river has encrouched upon the west valley wall. The river meanders along this section, and in fact an oxbow lake occurs close to the bottom of the slide area.

4. There is little doubt that the lower part of the landslides under investigation are a reactivation of a very old landslide. The presence of an old graben feature about 90 feet below the headscarp of the South Slide supports this view. (see Figure 2.4(a))

5. The slope of the valley wall at this section is approximately 11°, which is slightly steeper than neighbouring slopes upstream, especially in the uppermost part of the slope. There is however a more steeply dipping slope immediately downstream of the landslide. This slope is bounded on either side by large gullys, which offer good drainage to the slope, and thus it could be assumed that the water table is comparatively low at this site. It should be noted that the prairie rises gently towards the valley at this section, which is characteristic of valley rebound (Matheson, 1972). In contrast the landslide area is bowl-shaped and therefore rain and snow melt tend to collect in this area, and thus the water table may be comparatively

higher at this site.

6. There is a slight depression in the plain about 500 feet west of the landslide area, in which several ponds have formed. These ponds appear to have no stream outlets, and it is felt that these ponds may drain indirectly into the slide area.

7. The prairie land in this area is good pasture land (Wyatt, et al, 1944) and has been cropped at this location. This aides the infiltration of rainwater and snowmelt into the slope, especially at peak runoff times such as during spring thaw.

2.4 Description of the Landslides

2.4.1 General

The two landslides are located adjacent to one another on the west wall of the Battle River valley, approximately 30 miles Northeast of Wainwright, Alberta. The scarps of both slides follow roughly a north-south direction, and hence the slides have been conviently termed the 'North Slide' and the 'South Side' (Figure 2.2). Both slides are densely forested in local areas, which partially obscures the boundaries and some of the features of the slides.

At both slide sites, pre-slide topography exhibited very old grabens suggesting that the recent slides are the reactivation of old landslides The valley slopes at an average angle of 11° at this site, although the uppermost part of the slope is considerably steeper, and probably represents the remnants of the scarp of a very old slide. In general, erosion has subdued the old landslide topography to a considerable degree, suggesting a great age for these old slides.

The first signs of failure of the recent slides were observed in the early spring of 1974, following spring runoff, when cracking was noticed in the ground along the present scarp area and a depressional feature, approximately 2 to 3 feet deep developed. Catastropic failure of the North Slide followed on August 28th, 1974, when the scarp increased in height to between 40 and 50 feet. Movements were also observed at the location of the South Slide, although a scarp of only 1.5 feet in height was formed at that time. However, it was felt that this slope would also fail catastrophically in the ensuing year or so.

The slides are classed as complex (Skempton and Hutchinson, 1969) and are typically wedge shaped, comprising a sub-horizontal failure plane, and a steeply dipping backscarp, which suggests that failure has been controlled by bedding. This is most clearly seen in the North Slide and is also indicated by a distinct graben feature, well over 150 feet wide.

2.4.2 The North Slide

The North Slide is about 700 feet along the scarp and 900 feet horizontally from the scarp face to the toe outcrop. The scarp is between 40 and 50 feet in height, and was inclined at approximately 70° to the horizontal at failure. The toe is approximately 180 feet below prairie level, and about 100 feet above the river level. This is significant in that it suggests that the river has not been instrumental in initiating the slide failure. The graben is clearly visible in the slide profile (Figure 2.3).

The cracking pattern throughout the slide, and particularly in the lower portion of the slide mass and in the toe area, is extemely complex. The valley wall was by no means a planar feature prior to failure but rather hummocky. As a result of this, large tension and compression cracks developed during the slide movements. On the north side of the slide mass, tension cracks roughly parallel to the direction of movement indicate a splaying out of the toe area.

Yellow-brown to grey till, exhibiting columnar jointing is visible to a depth of 15 feet in the scarp face. This is underlain by the Upper Cretaceous brecciated, brown weathered silty clayshale, which continues to the bottom of the visible scarp. Selenite crystals in sheets up to 6 mm. thick are common over the entire scarp face, and particularly in the joints and fissures of the bedrock.

In the general vicinity of the toe area there are several distinct spring discharge areas. These are clearly outlined by vegetation (grasses), depression or seepage erosion bowls and the presence of gently water washed sediments. These fine grained sediments, being a pale buff colour, resemble deposits from eroded sandstone.

2.4.3 The South Slide

Coinciding with failure of the North Slide, a scarp some 500 feet long developed immediately south of the North Slide. The scarp was between 1.5 to 2 feet in height when measured in the fall of 1974. Subsequent slide movements are described in the following chapter. The toe of the South Slide had not cropped out, although it was felt that a toe would develop with further movement. The boundaries of the slide were also undefined, however it seemed likely that some overlapping of the North and South Slides had occurred. The slide profiles, which are essentially the same as the pre-slide profiles are shown in Figure 2.4. The old graben is clearly outlined, and attests to the existence of very old landslide movements at this site.

The topography of the South Slide, as with the North Slide, was rather hummocky, however the slide movements have not been sufficient to cause serious visible cracking within the slide mass at this time.

Seepage discharge areas were also found in the South Slide corresponding to those found in the North Slide.

2.5 Climate

According to Koppen's classification (Canada Dept. Mines Tech. Service, 1957) the climate of the area is sub humid continental. Based on 19 years of continuous records the average annual rainfall is 12.14 inches and the average annual snowfall is 35.1 inches, giving an average total precipitation of 15.65 inches. The mean annual number of days with precipitation is 71. Rainfall records were obtained for Paradise Valley, latitude 53° 07' N, longditude. 110° 21' W, elevation 2200 feet A.S.L. from 1961 to present time and are listed in Appendix A. Paradise Valley is located approximately 6 miles east of the landslide area.

TABLE II.1

MEMBERS OF THE BELLY RIVER FORMATION

Member	Lithology	Writer	Depositional Environment
Brosseau	"The upper part of the formation consists of flaky sandstones and clayey sandstones... The lower part of the formation consists of brown sandy shales, thin-bedded sandstone and then seams of coal."	Allan (1918)	continental and marine deltaic
Shandro	"...dark grey marine shales containing calcereous and arenaceous concretions." "... also contains carbonaceous shales and brownish-grey silty shale with carbonaceous specks."	Allan (1918) Shaw and Harding (1949)	marine neritic
Victoria	"fine- to medium-grained grey sandstone and brownish-grey, carbonaceous, silty shale with local thin coal seams."	Shaw and Harding (1949)	continental and marine deltaic
Vanesti	"... grey shale, silty shale, clayey shale and fine sand."	Shaw and Harding (1949)	marine neritic
Ribstone Creek	"greenish yellow, massive, soft sandstone at top; green and carbonaceous shale and coal, light grey sandstone at base."	Slipper (1918)	continental and marine deltaic
Grizzly Bear	"..dark blue-grey, marine shale, containing ironstone and sandstone nodules with some beds of yellow incoherent sandstone."	Slipper (1918)	marine neritic
Lower Birch Lake	"massive, cross-bedded sandstone, buff-coloured, containing lenses of harder sandstone."	Slipper (1918)	continental and marine deltaic
Mulga	"grey shale with silt lenses and some carbonaceous material"	Nauss (1945)	marine neritic
Upper Birch Lake	"grey to slightly yellow, medium grained sand which weathers to a light rusty colour."	Nauss (1945)	continental and marine neritic
Oldman	similar to that described for the undivided Belly River Formation	Shaw and Harding (1949)	continental peneplain

(after Shaw and Harding)

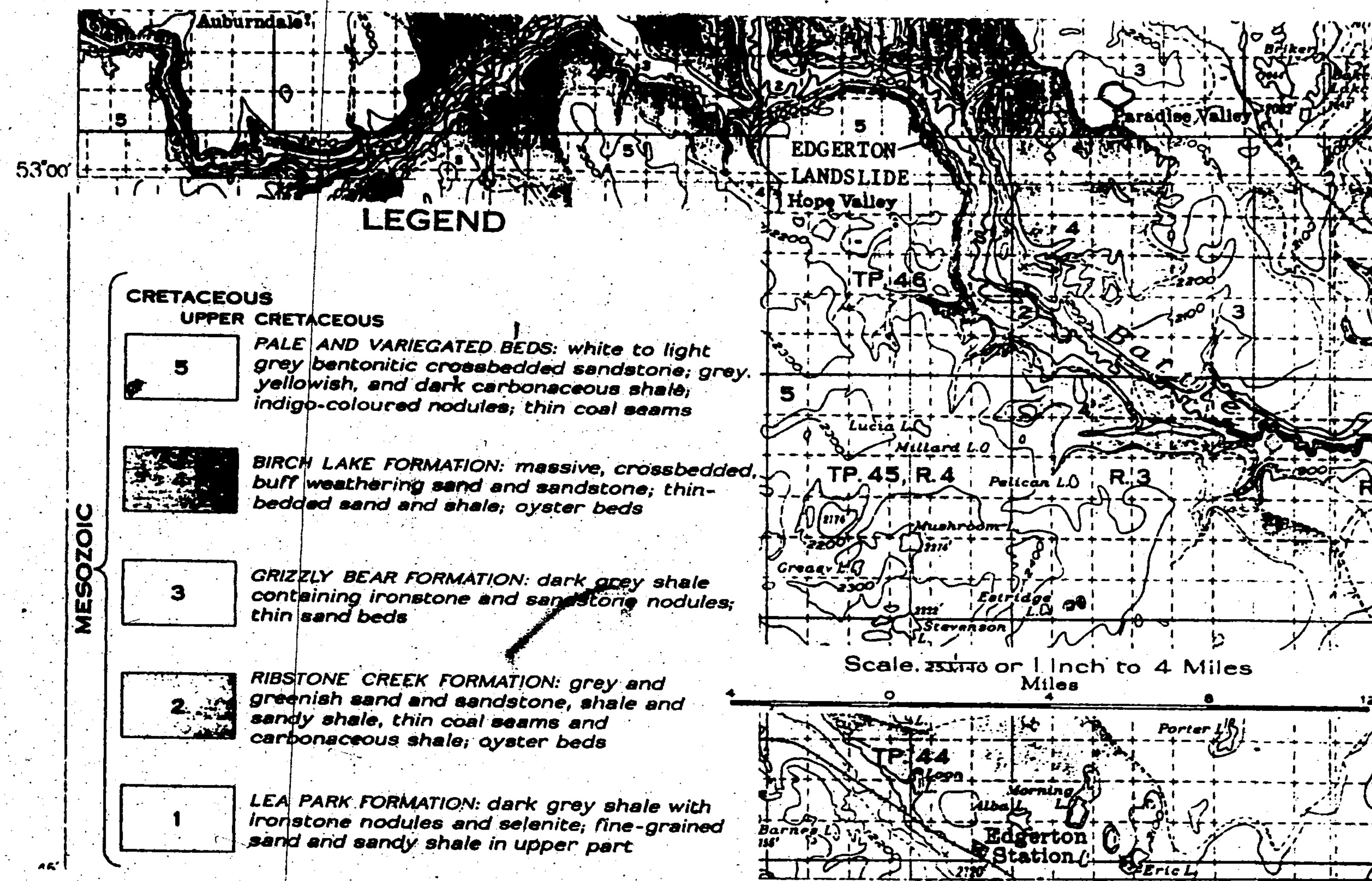


FIGURE 2.1 BEDROCK GEOLOGY OF THE STUDY AREA

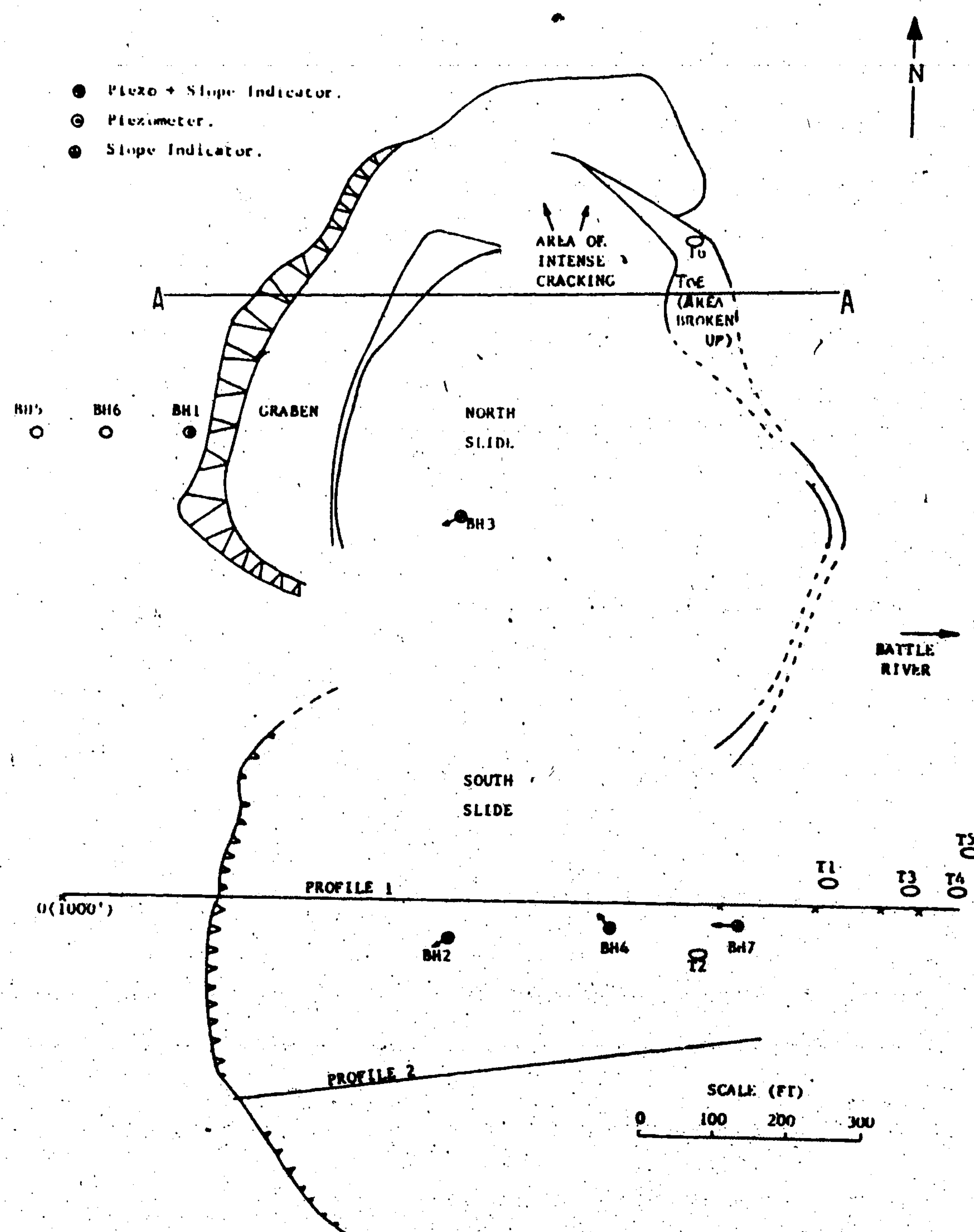


FIGURE 2.2 GENERAL PLAN OF THE LANDSLIDES

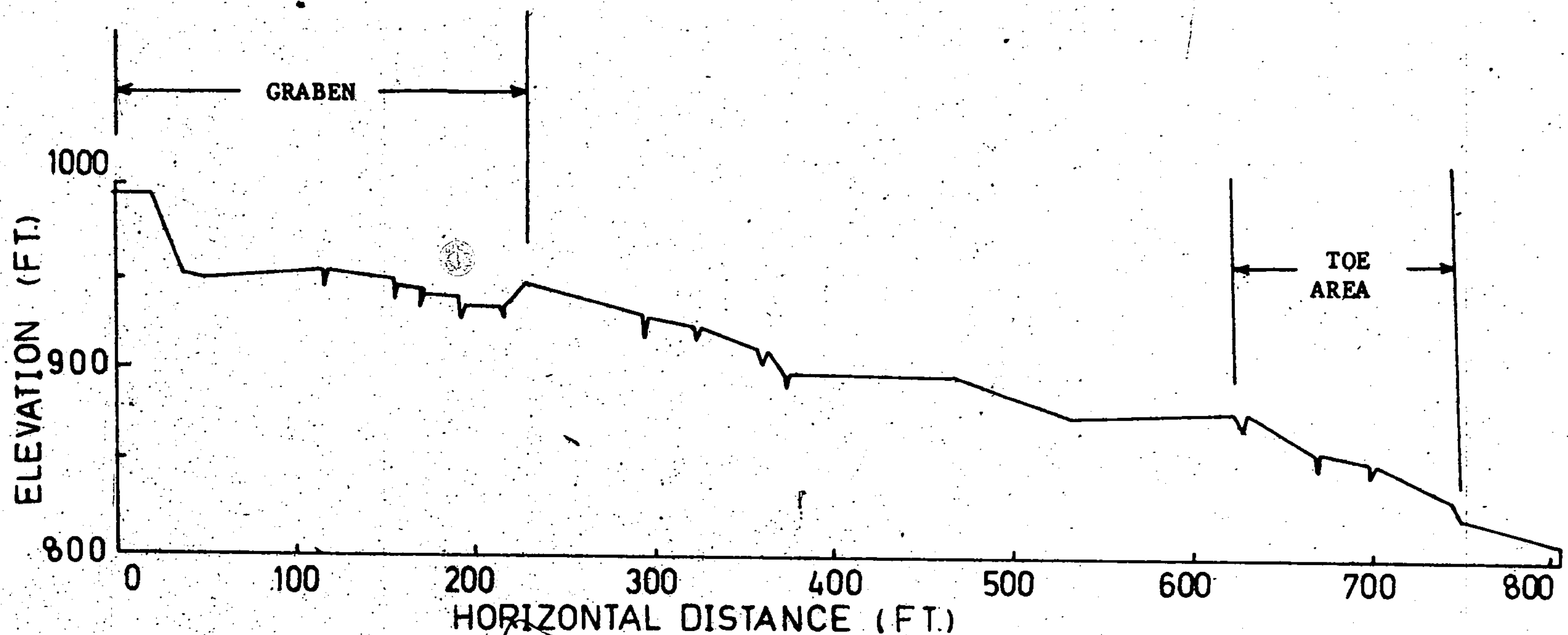


FIGURE 2.3 GENERAL PROFILE OF THE NORTH SLIDE

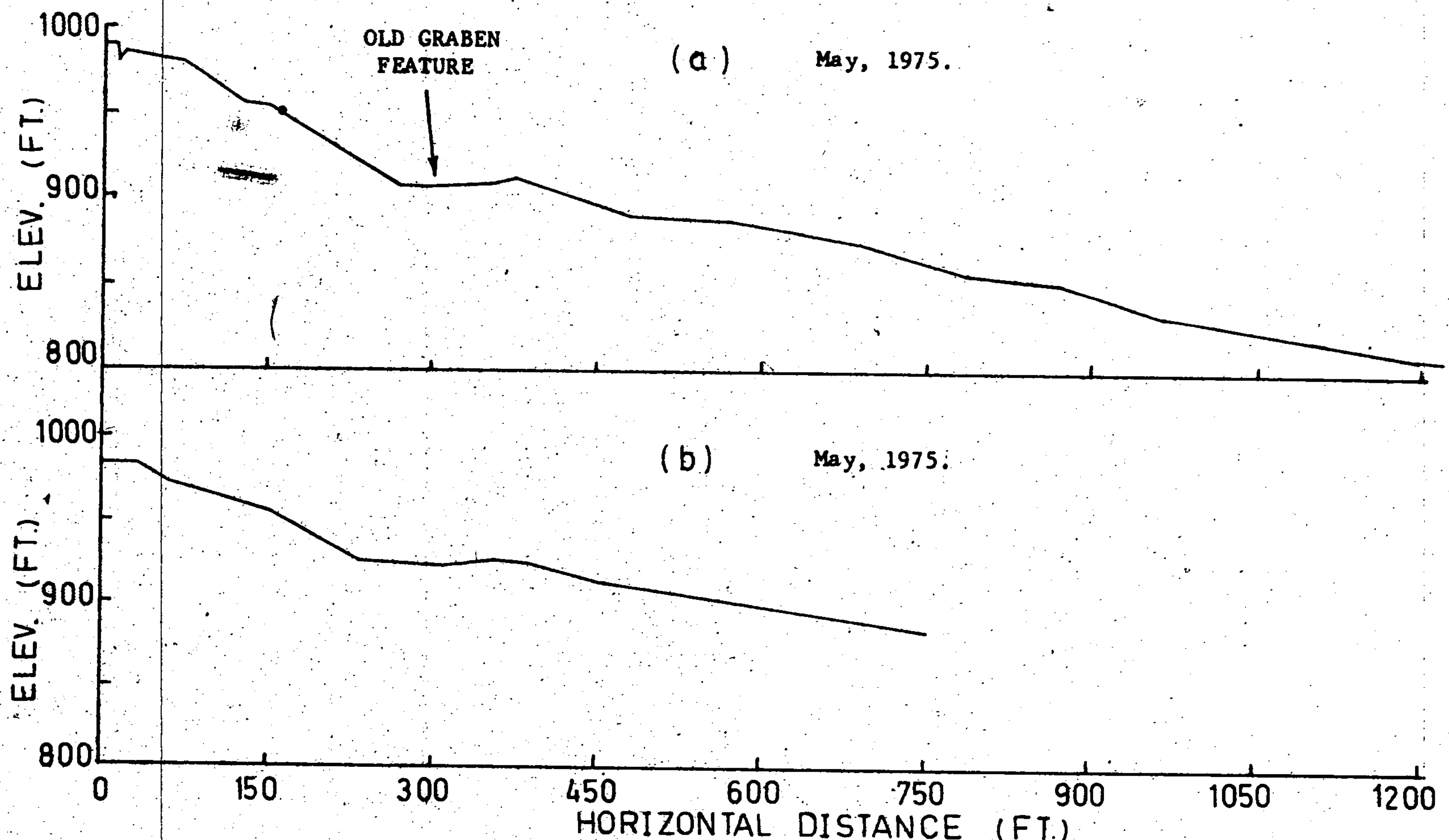


FIGURE 2.4 GENERAL PROFILES OF THE SOUTH SLIDE

(a) PROFILE 1 (b) PROFILE 2

CHAPTER III

SITE INVESTIGATION

3.1 General Survey

A topographical survey of the site was made in early Spring 1975, prior to subsurface exploration, essentially to obtain an accurate map of the slide area. Thick forest vegetation prohibited determining the north and south boundaries of the North Slide in detail, however several points were defined and the boundaries were interpolated between these points, guided by field evidence.

The upper part of the South Slide was thickly forested and hence a full survey was not possible. It was, however, possible to distinguish the scarp area of this slide along the major portion of its length, before it faded out in a gully on the southern flank of the slide.

Two survey lines were also cleared down the South Slide, one at the mid point of the scarp and the second at the south end of the scarp. The locations of these profiles are shown in the general plan of the landslides (Figure 2.2) and the profiles are shown in Figure 2.4.

Surface stakes were also located at convenient intervals down the line of profile 1 and several stakes were placed at the area of the expected toe. These stakes were surveyed fully again in May 1976, and will be surveyed at intervals in the future to follow movements of the soil mass.

The following points are of note;

1. The north scarp had retrogressed up to about 10 feet, since the time of initial failure in August 1974. This was due to fracturing along the vertical columnar jointing in the till layer, accompanied by a sloughing away of the underlying brecciated clayshale bedrock. It is felt that this process will continue until the scarp reaches its natural angle, which has been observed in older slides along the valley wall. Apart from these scarp movements, the North Slide seemed to be stable, and no appreciable movements had occurred since the previous year.

2. Of particular interest, it was noted that the scarp of the South Slide had increased in height from the original 1.5 feet, as measured in August 1974, to a height of 3 feet when visited in early May 1975. From that time it had dropped an additional 2 feet by the time the survey was conducted in June, 1975.

No apparent outcropping of the toe was visible however, whereas the toe of the North Slide was well established being about 180 feet vertically below the top of the north scarp. At this stage, the scarp of the South Slide was poorly defined on the southern flank, where it faded out upon entering a gully. It was also poorly defined at the north end of the scarp, where it disappeared into thick vegetation before possibly merging with the southern boundary of the North Slide.

3. The scarp angle of the South Slide could not be reliably measured, being only 5 feet in height. The recorded scarp angle of the North Slide was however measured at 70° in August, 1974. (Thomson, 1974).

4. Since the time of the survey, the scarp was observed to drop an additional 1 foot by December, 1975 although no emergence of the toe was noticed prior to the onset of winter in 1975. It was also noted in December, 1975, that the south scarp had extended across the gully at the south end of the slide, and was visible to the south of the gully, where it had an average height of 6 inches. However the cracks faded out at this point and could not be traced down the slope. When the slope was re-visited in early April 1976, coinciding with spring thaw the scarp was observed to have dropped an additional 1 foot and measured about 7 feet in height. Snow melt was also observed pouring down the cracks along the top of the south scarp at this time, and one week later, after the lower parts of the slope had thawed, large pools of water had formed around the toe of the South Slide.

In May, 1976, the surface stakes on the South Slide were resurveyed, in addition to the section of the scarp south of the gully. This is included in the general plan of the landslide (Figure 2.2). The average scarp height at the middle of the scarp was recorded at 7.5 feet, however still no cropping out of the toe was observed.

3.2 Subsurface Exploration

3.2.1 Drilling Techniques

Continuous cores of the bedrock were obtained to provide information on the geology of the site. These samples were extruded from the tubes as scon as possible, then waxed on site and transported back to the University, where they were stored in a moisture controlled atmosphere. From there, selected core samples were tested in the laboratory to determine the engineering properties of the rock.

A two week drilling programme was undertaken in July 1975. The Mayhew 3000 rotary drilling rig, using wet drilling techniques, was employed in conjunction with the University of Alberta Pitcher Sampler (Terzaghi and Peck, 1967). Water was used as drilling fluid, except where a bentonite slurry was required to maintain circulation within the hole.

Past experiences have found that conventional drilling, using double walled core barrels, generally yielded good results in the harder Cretaceous rocks, but gave poor recovery in weaker softer stratum. Experience of the Prairie Farm Rehabilitation Administration showed that seams of soft bentonite were often washed out. A typical experience of this was recorded by Wilson and Hancock (1959) at Oahe Dam in the Pierre Shale. Somewhat better results have, however, been recorded using the University of Alberta Pitcher Sampler.

The Pitcher Sampler has been employed successfully in the softer rocks of the Edmonton Formation (Pennell, 1969), in the Smoky River Group in Northern Alberta (Hayley, 1968) and in the Belly River Formation, east of Edmonton (Locker, 1969). Typical core recoveries of 95% have been achieved.

3.2.2 Boreholes

Originally, four boreholes were provided for, which are shown on the site plan as BH1, BH2, BH3 and BH4 (Figure 2.2). BH1 and BH3 are located on the North Slide, and BH2 and BH4 on the South Slide.

Borehole BH1 was drilled to a depth of 155 feet, and sampled continuously from 30 to 155 feet. From preliminary calculations (Thomson, 1974) it was expected that this would be well below the expected slip surface. This hole was drilled primarily to establish the general lithology of the slide area. A moisture content profile was also established for this hole.

Boreholes BH2, BH3 and BH4 were then drilled to depths of 81, 83 and 63 feet respectively. Grab samples were taken in each case, down to the area of interest, and the holes were logged on the basis of the materials flushed to the surface. The holes were however continuously sampled over the area of interest, which was expected to encompass the failure zone in each hole.

While drilling BH1, a very hard, well indurated sandstone layer some 8 inches thick was encountered at a depth of 130 feet. Consequently two further boreholes, BH5 and BH6 were drilled, 300 and 150 feet respectively west of BH1. The purpose of these boreholes was to investigate the theory of valley rebound as proposed by Matheson (1972), in which case this sandstone stratum would be used as a marker bed to trace any upward flexing of the beds, as they approached the axis of the valley.

In addition BH7 was drilled near the toe of the South Slide at the end of October, 1975, by means of a Mobile Auger drill rig, to a depth of 45 feet. This hole was established purely for instrumentation purposes, to locate the position of the failure plane in this area. It was however logged using grab samples, at 2 feet intervals, and a moisture content profile was also obtained.

3.2.3 Toe Trenches

Six toe trenches were excavated by a Gradall ditch digger in mid-October 1975. Trenches T1 to T5 were excavated at the suspected toe of the South Slide, and T6 at the toe of the North Slide. The depths of these trenches ranged from 10 feet to 14 feet, and their locations are shown in Figure 2.2. Bag samples were taken from each trench for identification purposes. A large block sample was taken from trench T5 and eight 3 inch by 5 inch block samples were taken from T6, along a contact between two materials. These samples were again waxed at the site and returned to the University moisture room.

3.3 Instrumentation

Four slope indicator casings, and three piezometers were installed at the site. One piezometer and one slope indicator were installed in the North Slide in BH1 and BH3 respectively (Figure 2.2) . This slope indicator was included for monitoring future creep movements in the North Slide. In the South Slide, a slope indicator was installed in BH7, and in both BH2 and BH4, a piezometer in conjunction with a slope indicator was installed. Aluminium alloy casing, 3.38 inch maximum O.D., was used in the case of BH3, BH4 and BH7, and 2.75 inch O.D. plastic casing was used in BH2. In each case, the tubing was installed in a clean borehole, and the annular space remaining in the borehole was backfilled with sand. Two pairs of mutually perpendicular grooves are provided in the casing, for the purpose of monitoring the inclination of the pipe. These two directions are nominally referred to as North-South and East-West, but the actual orientation of the grooves was determined with a Brunton compass after installation. The orientation of the "North" direction for each slope indicator is shown on the site plan (Figure 2.2). The initial inclination of the tubing is determined with a "sensor" at 2 feet intervals of depth, and by obtaining successive readings at periodic time intervals, the changes in inclination resulting from ground movements can be determined. The sensor used was the SINCO Digitilt, model 50306. A complete description of the slope indicator device is given by Wilson (1962).

Terra Tec pneumatic piezometers, model P1020, were used in each case, in conjunction with the control console, model C1000. The general principle of operation of the pneumatic piezometer, is the application of air pressure through a closed loop system, to balance the pore pressure acting against the sensing element. Model P1020 uses a single bellow to minimize time lag. High sensitivity to pore pressure change is recorded by minute movements of the bellow, thus requiring practically no change in volume of water, and therfore being suitable for low permeability clays. The piezometer was enclosed in a well point system, and embedded in saturated sand at the specified depth. A bentonite plug was placed on top of the sand to act as a seal, and the remainder of the hole was filled with sand. Typical installations are shown in Figure 3.1.

3.4 Results and Interpretation

3.4.1 Stratigraphic Profile

Borehole logs for BH1 to BH7 are given in Appendix B. Moisture content profiles were obtained for BH1 and BH7 and accompany the respective borehole logs. Atterberg limits were also performed on the profile soils of BH1 and also on selected soil samples from BH2 and BH4, and are also included with the borehole logs.

As mentioned previously, BH1, located at the top of the North Slide was sampled continuously to a depth of 155 feet. The uppermost materials, consisting of till underlain by silty clayshales, were weathered to a depth of approximately 35 feet. The soil was brecciated and crumbled upon extrusion from the Shelby tubes. Soil samples taken from a depth of about 30 feet exhibited highly weathered joints inclined at about 15° to the vertical, with lenses of selenite up to 3/8 inches thick between the joints, similar to those observed on the scarp face of the North Slide. Jointing was also present in the underlying unweathered material, and especially in the clayshales around 80 feet deep, which were used for direct shear tests. Planar joint surfaces were again inclined at approximately 10-20° to the vertical, and although these surfaces were not oxidised, some softening was observed along the joint surfaces.

At a depth of 130 feet, a hard indurated sandstone stratum, some 8 inches thick was encountered. This was overlain by soft, dark grey clayshale, which was rich in montmorillonite. White fossil specks were present in this material, and several white fossil layers, averaging 1/8 inch in thickness, were also found a few feet above this material. These layers may in fact correspond approximately to the junction between the Birch Lake and the Grizzly Bear members of the Belly River formation, as shown in Figure 2.2. The sandstone stratum was underlain by dark grey clayshales interbedded, with lighter grey sandstone layers, carbonaceous seams and bentonite layers.

Figure 3.2 shows the stratigraphic profile for BH5, BH6, BH1 and BH3, as located in Figure 2.2. Good correlation exists between BH5, BH6 and BH1 however the correlation with BH3 is not as apparent. It seems that BH3 has been displaced downwards due to previous landsliding. It should be noted that each soil type contains large amounts of clay sizes, and that clayshale seams are prominent in the sequence.

Weathering was observed in each borehole and seems to parallel the ground surface at a depth of about 35 feet. It should be noted that there appears to be an upward flexing of the sandstone stratum at approximate elevation 855 feet (Figure 3.2), as it approaches the valley axis, observed from BH5, BH6 and BH1, which may indicate evidence of valley rebound. It should also be noted however, that the sandstone stratum was not detected in BH3. Figure 3.3 shows the stratigraphic profile for BH2, BH4 and BH7 on the South Slide. Weathering again parallels the ground surface down to a depth of some 35 feet. Fossil layers were found in BH2 at a depth of 66 feet, which corresponds in elevation with those layers found in BH1. Small fossil specks were also found in BH4, at a depth of about 46 feet, (Figure 3.3) that is a downward displacement of some 10 feet from BH2. This feature may reflect displacements in the soil mass due to previous landsliding.

The fossil layer is underlain by dark grey clayshales, sandstone layers, occasional carbonaceous layers up to 1 inch thick, and also thin bentonite layers. This again corresponds with that found in BH1.

In general, however, it is difficult to trace unambiguously, any soil horizons throughout the soil mass, due partly to the weathering process, but mainly due to the retrogressive sliding, which has been responsible for the slide topography at this site.

3.4.2 Piezometric Study

Piezometers P1, P2 and P4 were installed in boreholes BH1, BH2 and BH4 respectively. Pore pressures were monitored throughout the fall of 1975 until early December, and read again in April 1976. The piezometric levels are shown in Figure 3.4. Average readings of 1.2 and 1.1. kg/cm² were recorded by P2 and P4 respectively during fall, 1975. No readings were taken during winter, however, when the slide was revisited in late March, 1976, thaw had started at the top of the slide, and streams of water were observed flowing into the cracks along the scarp of the South Slide, and similarly along the North Slide. The lower part of the slides were still frozen, and protected by a snow cover about 2 feet deep, such that seepage of water from the slopes was prohibited, and thus pore pressures would be expected to have been at their highest at this time. Unfortunately readings could not be obtained, due to the problem of the console freezing.

When the slide was revisited in early April 1976, the slope had thawed and large pools of water had collected near the toe of the slide. A reading of 1.38 kg/cm^2 was obtained from BH4 corresponding to an increase in water level of almost 10 feet. Unfortunately P2 had become inoperative, and hence no pore pressure reading was obtained from BH2. Three weeks later the piezometer reading had dropped to a value of 1.23 kg/cm^2 in BH4.

From these results, it is surmised that the pore pressures reached their maximum at some time during spring runoff, when the lower part of the slope was still frozen, and as this part thawed, the pore pressures were able to dissipate. Thus, it is most likely that the critical pore pressure was greater than that recorded during April, 1976.

3.4.3 Slope Indicator Study

The original slope indicators, S2, S3 and S4, located in BH2, BH3 and BH4 respectively, were initially read in early August 1975, approximately one month after installation. Following this, the slope indicators were monitored at frequent intervals during the fall, 1975, until early December, 1975, and then again in early April 1976. Results were processed by computer and the plots are shown in Figures 3.5 to 3.8.

Unfortunately, ground movements during July and August were such that the casing in BH2 had deformed excessively in the vicinity of the failure plane, hence it was not possible to lower the inclinometer to the bottom of the hole. The instrument stuck at a depth of 60 feet below the top of the pipe, hence this depth has been regarded as the upper limit of the failure plane. It can be seen from Figure 3.5 that no other shearing planes existed above the point. S4 was successfully monitored during September and October 1975, however it also had blocked off at a depth of 44 feet by early November 1975, after which time, no further readings were obtained. The results of S3 (Figure 3.6) indicate no substantial creep movements up to date, and thus it was not possible to define the depth of the failure plane. It is expected that this slope indicator will be monitored in future to follow creep movements.

Slope indicator S7, was installed in BH7 near the toe of the South Slide in late October 1975, to locate the failure plane at this point. It also was monitored until early December 1975, and the failure plane was located at a depth of 26 feet below the top of the pipe (Figure 3.8).

When this indicator was read in early April, 1976, deformations during Spring thaw had caused the casing to block off at a depth of 22 feet.

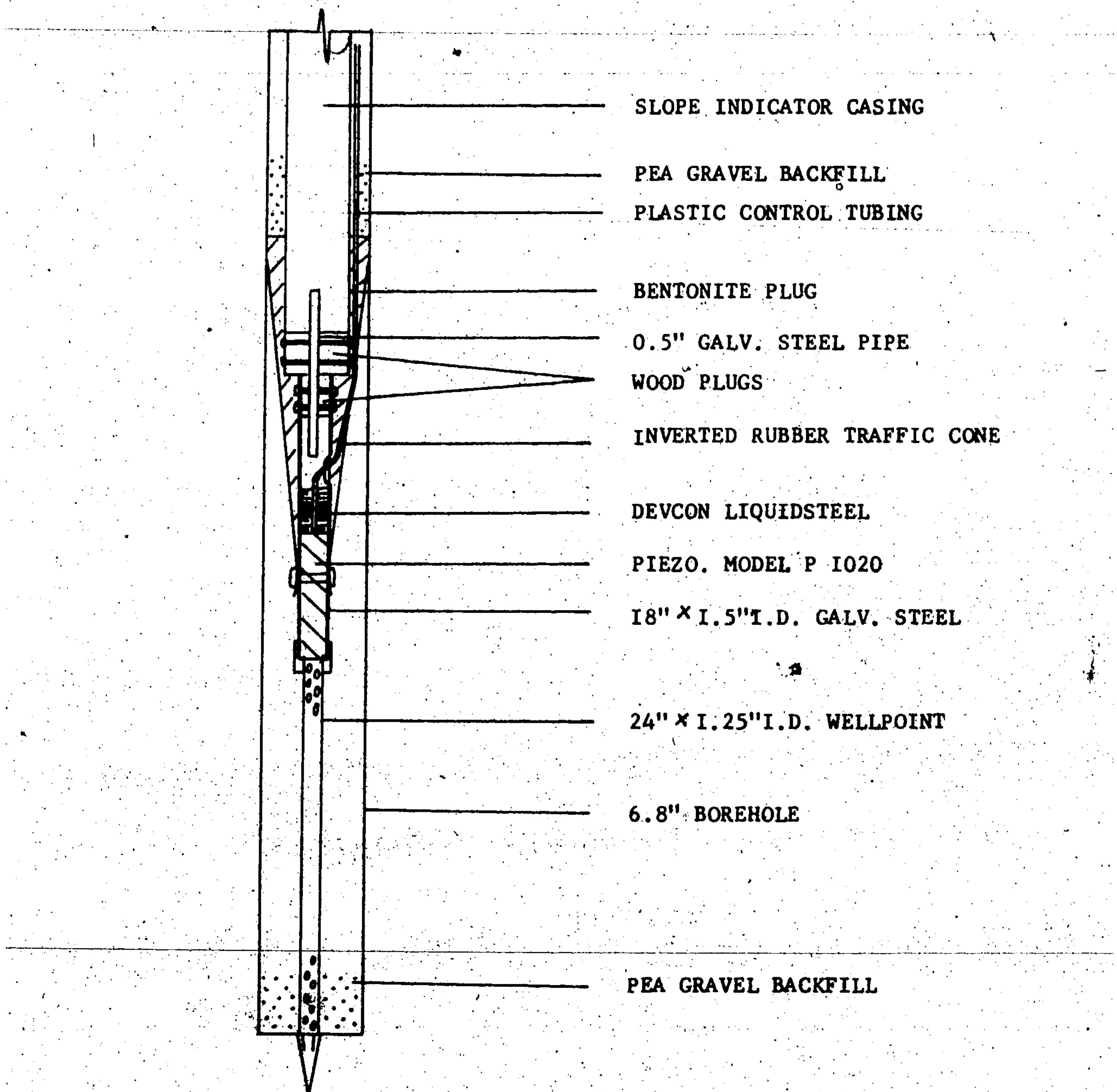


FIGURE 3.1 TYPICAL INSTALLATION OF PIEZOMETERS AND SLOPE INDICATORS

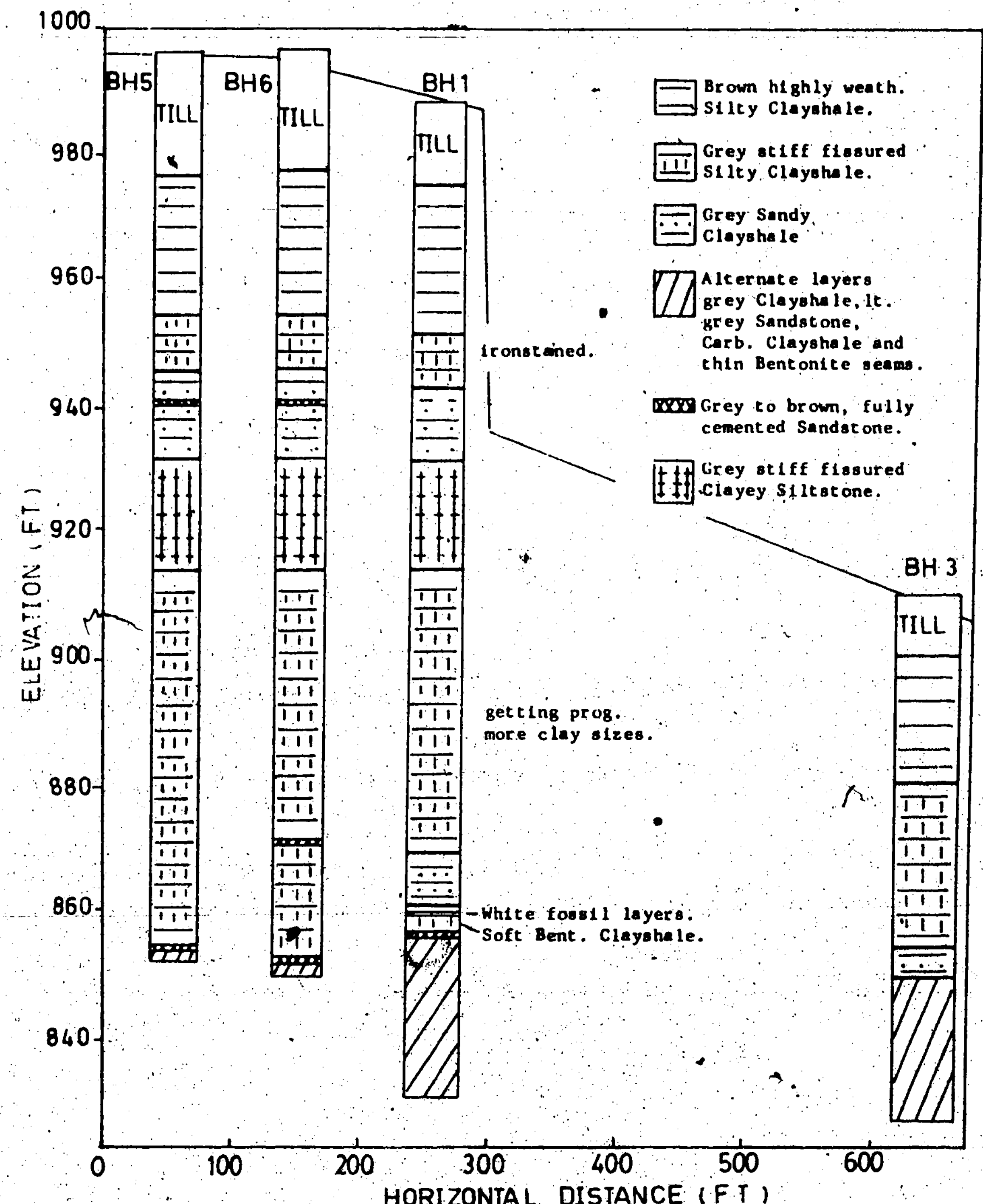


FIGURE 3.2 STRATIGRAPHIC PROFILE OF THE NORTH SLIDE

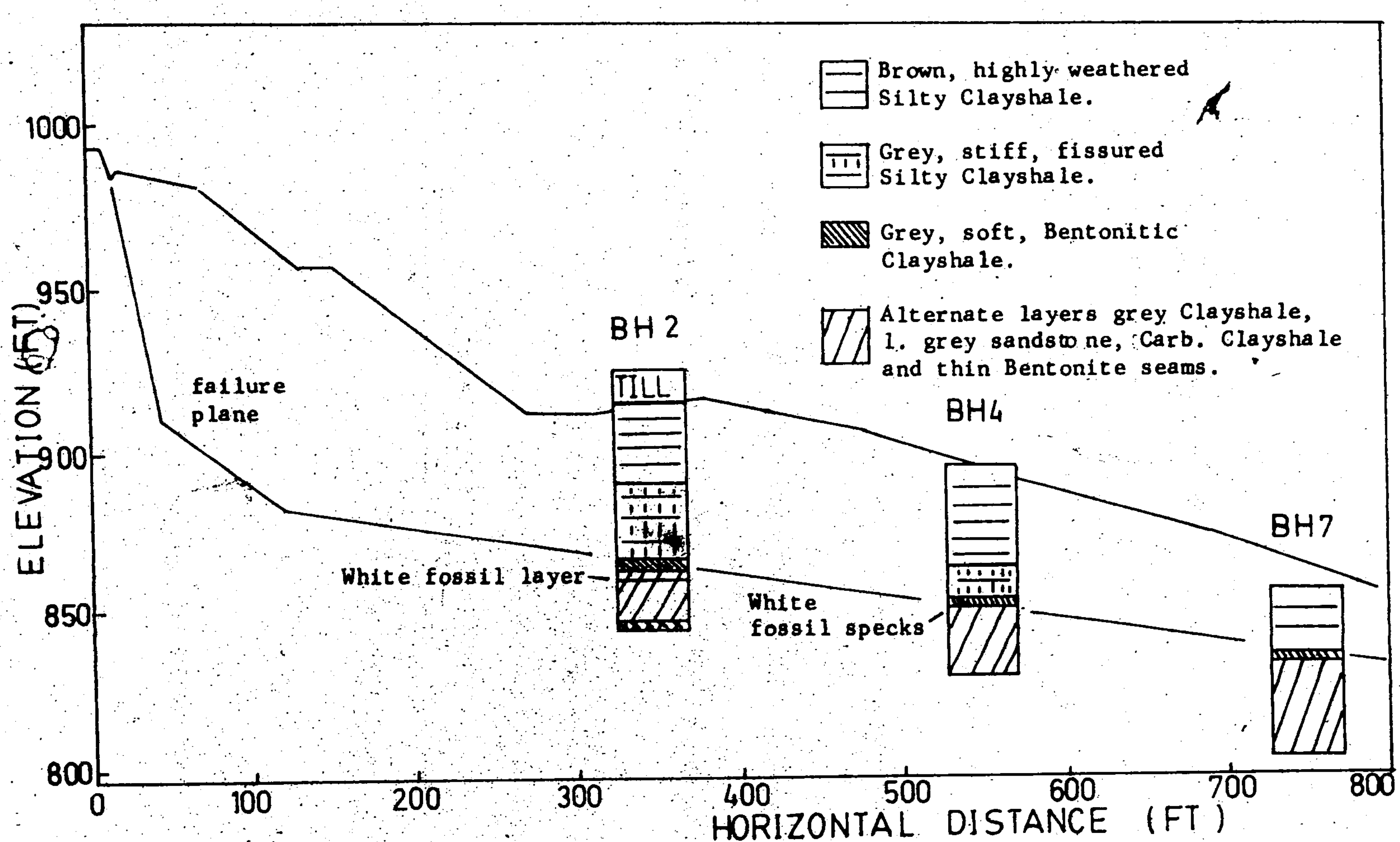


FIGURE 3.3 STRATIGRAPHIC PROFILE OF THE SOUTH SLIDE

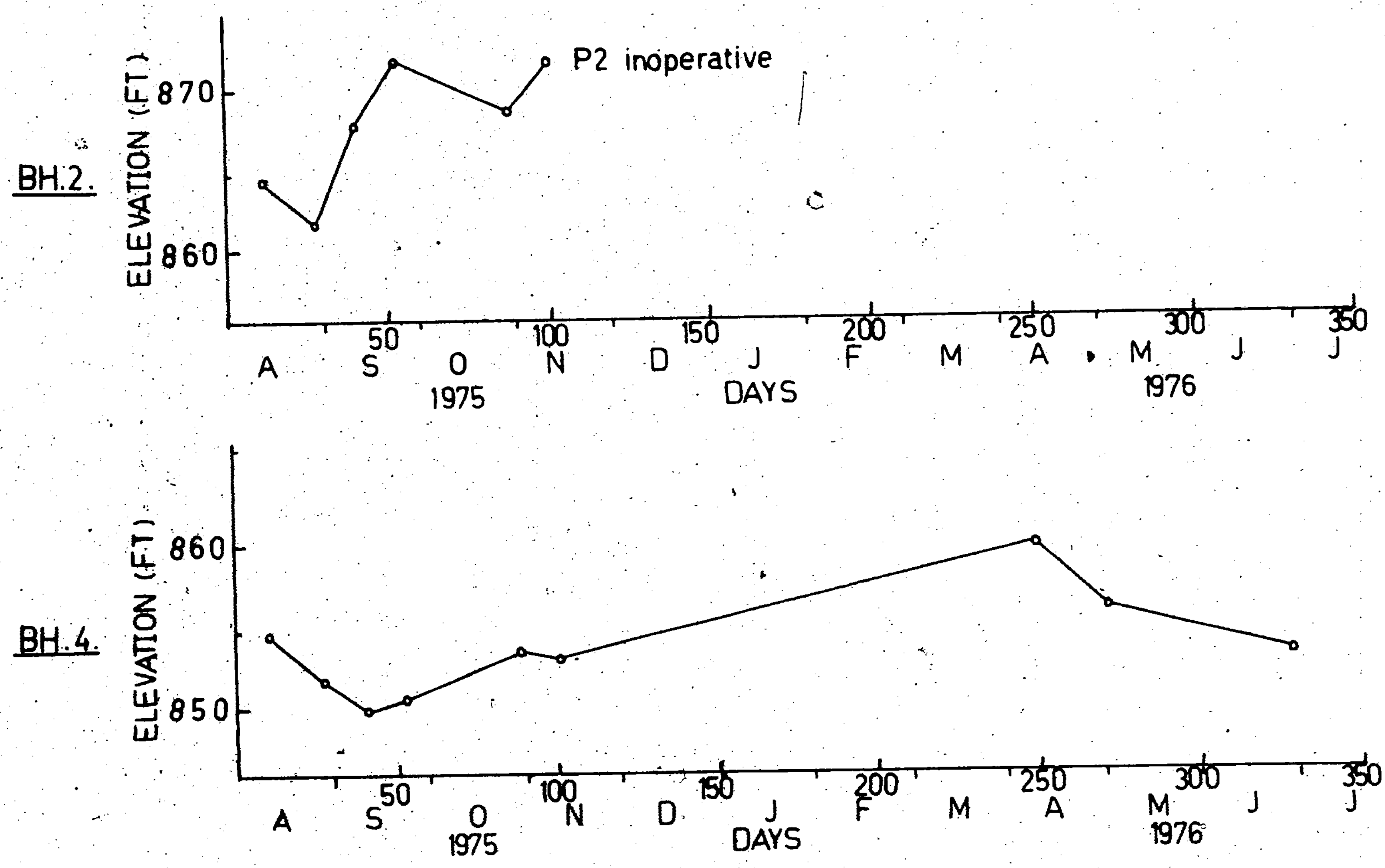


FIGURE 3.4 PIEZOMETRIC READINGS

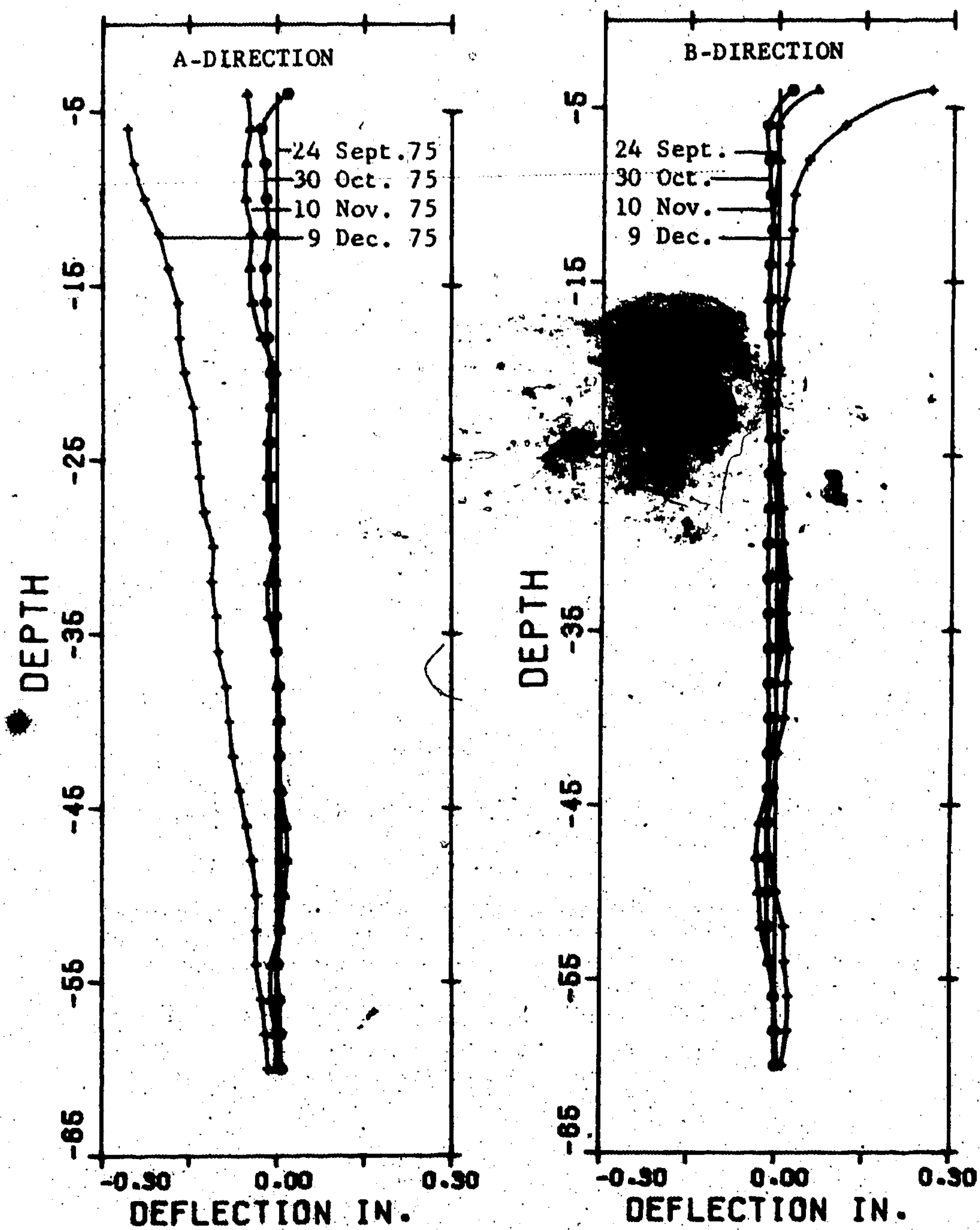


FIGURE 3.5 SLOPE INDICATOR READINGS FROM BH2

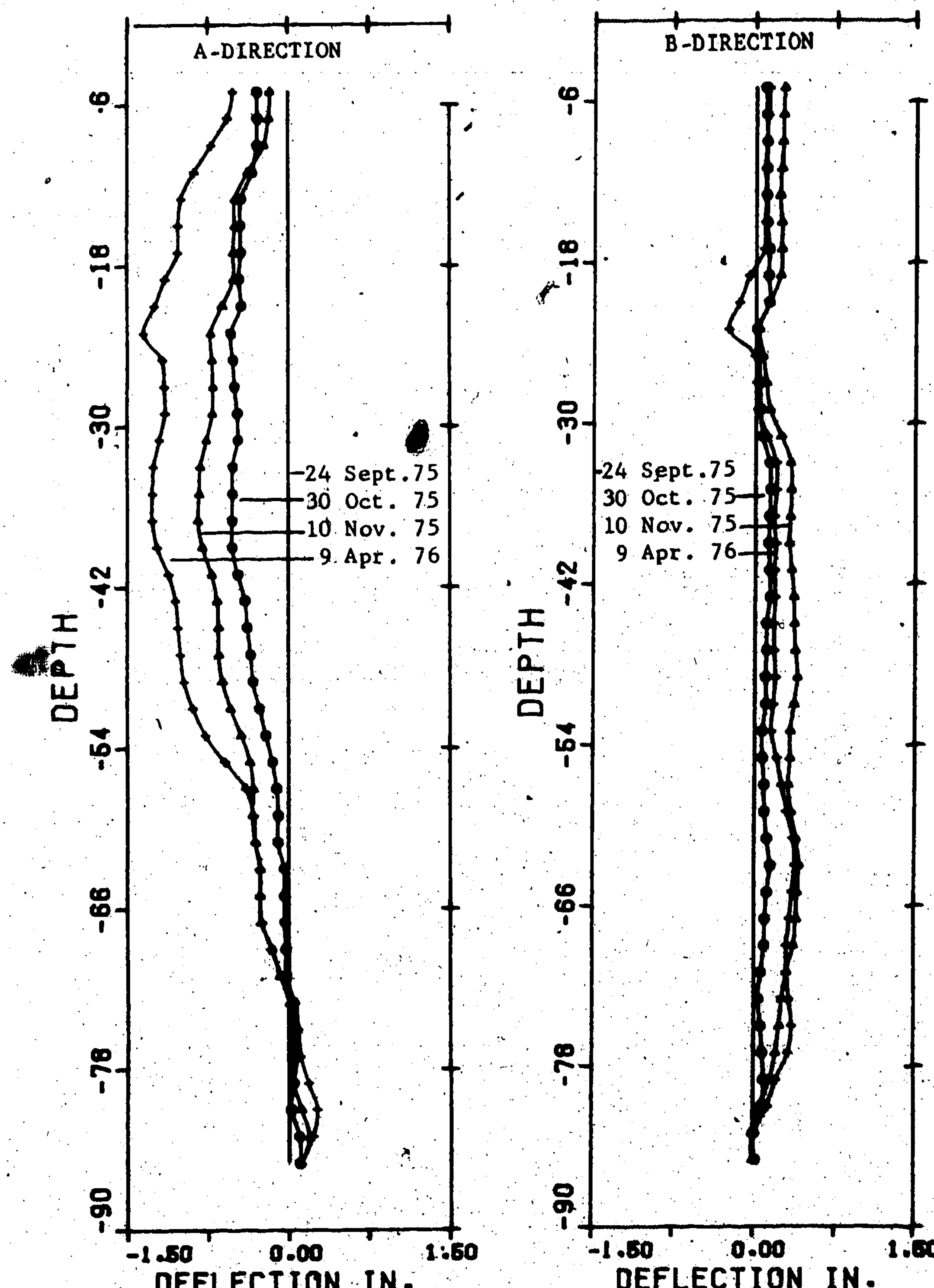


FIGURE 3.6 SLOPE INDICATOR READINGS FROM BH3

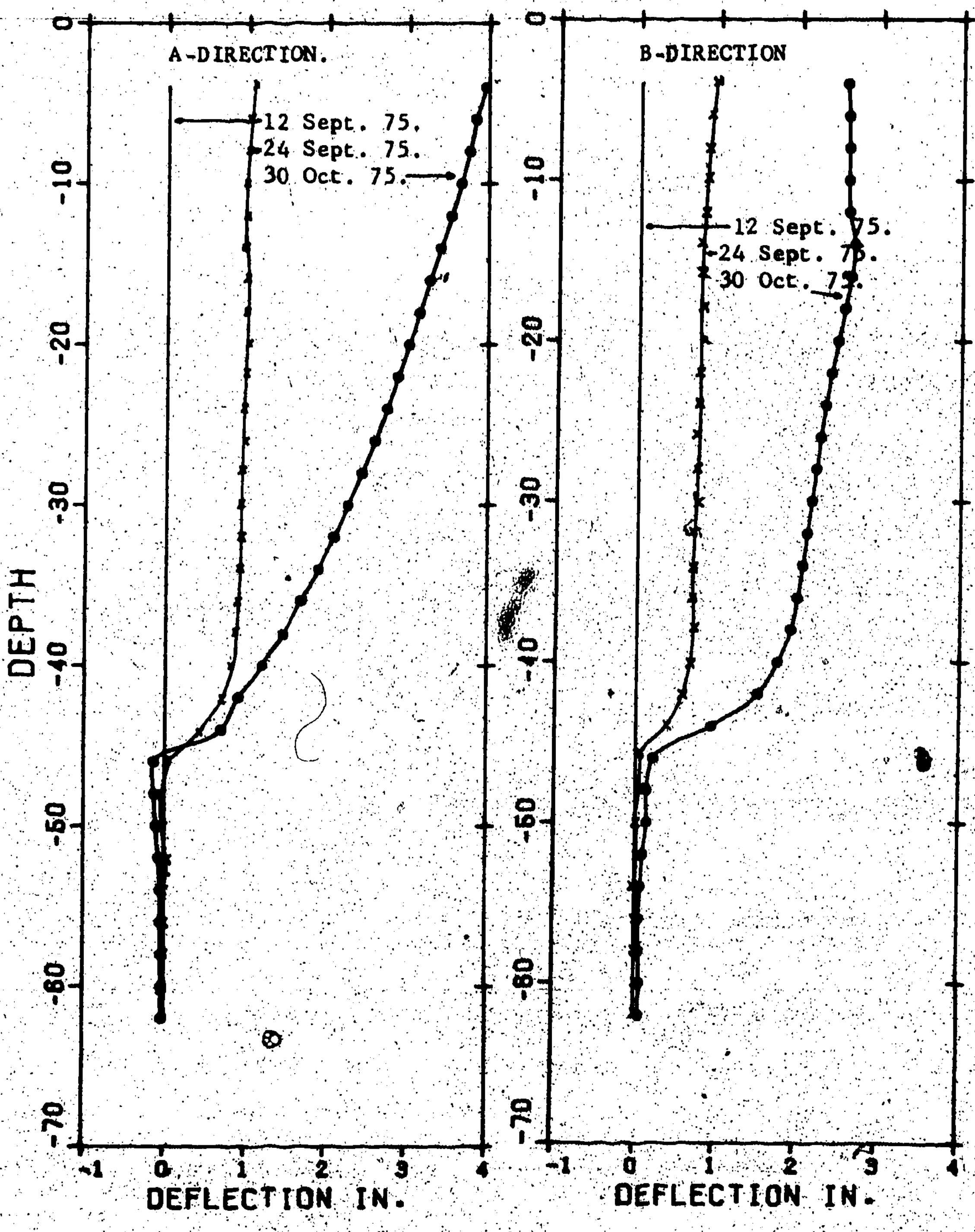


FIGURE 3.7 SLOPE INDICATOR READINGS FROM BH4

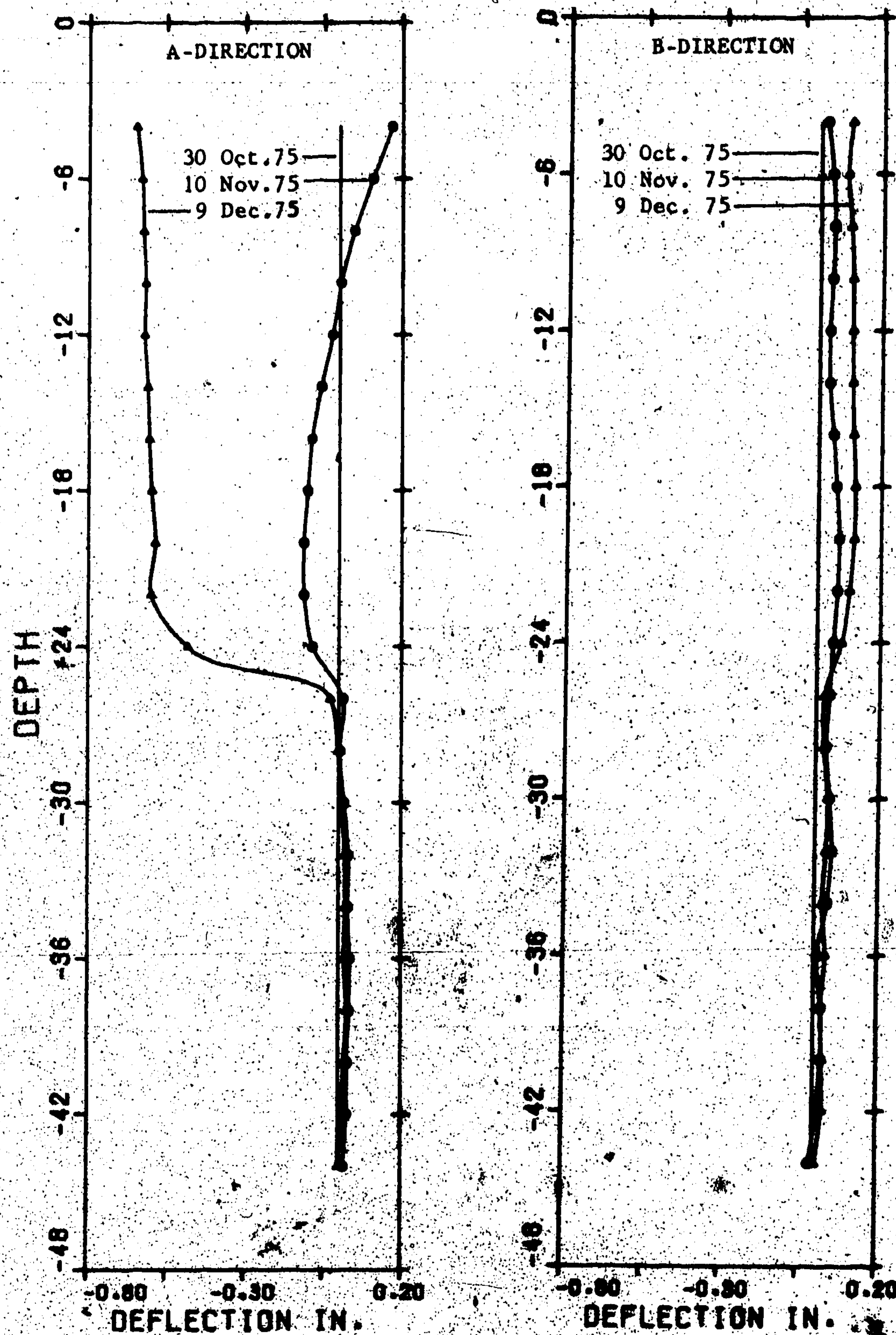


FIGURE 3.8 SLOPE INDICATOR READINGS FROM BH7

CHAPTER IV

LABORATORY TESTING PROGRAMME

4.1 General

A routine laboratory testing programme was conducted to determine peak and residual strength parameters of representative soils for the stability analysis. In addition, index tests were performed on the soils described in the previous chapter. Limited mineralogical and physico-chemical tests were also carried out on a sample obtained from the failure plane. The results and interpretation of the testing programme, along with brief descriptions of testing procedures are described in this chapter.

4.2 Index Tests

Index tests were performed on the profile soils of BH1 and on selected samples from BH2 and BH4. The results of these tests are summarized in Table IV.1. Moisture content profiles were obtained for BH1 and BH7 and are included in the borehole logs in Appendix B. In addition, grain size distribution analyses were performed on selected profile soils from BH1, BH2 and BH4. Results of these tests are also summarized in Table IV.1.

The index tests were performed by the Highways Testing

Laboratory, Department of Transport, in accordance with the procedures detailed in ASTM (1958).

4.3 Mineralogical and Physico-Chemical Analyses

Mineralogical and physico-chemical analyses were performed on the bentonitic clayshale of the failure plane taken from BH4. Both sets of tests were undertaken by the Department of Soil Science, University of Alberta. The mineralogical composition of the clay size fraction of the sample was obtained by x-ray diffraction techniques and is considered accurate only within ± 10 percent. Physico-chemical analyses were performed to determine the cation exchange capacity, the water soluble cations and the exchangable cations. The results are given in Table IV.2.

4.4 Shear Strength Study

4.4.1 Sample Preparation and Testing Procedure

Shear strength parameters were required for representative soil samples of the slide material, and in particular, a measure of the residual strength parameters of the failure plane material, for use in stability analyses. Large deformations are required to reduce the strength of undisturbed samples to the residual value (James, 1970), and for this reason a reversal direct shear box, supplied by Wykeham Farrance Co. of England was chosen for the tests. Advantages and disadvantages of the direct shear test are outlined by Chattopadhyay (1972).

Three soil types were tested, viz;

1 Undisturbed, stiff, fissured, silty clayshale samples were taken from the core of BH1 between 80 and 90 feet. This soil formed a large portion of the backscarp.

2 Undisturbed, weathered, silty clayshale samples were taken from a large block sample obtained from Pit 5. This material formed part of the backscarp, as well as part of the failure plane, nearest the toe.

3 Remoulded bentonitic clayshale samples were taken from the core of BH4 at the location of the failure plane.

Both undisturbed materials were carefully carved from core samples and block sample respectively to fit the shear box. All samples were two inches square and one inch thick. Samples were first stressed to their peak strength at rates determined by the Gibson-Henkel (1954) formula, to ensure fully drained conditions during shear. Upon reaching peak strength, the samples were then sheared to residual strength, taken when the shear strength remained constant over two or more successive compression or tension cycles. Since Kenney (1967) demonstrated that residual stress is primarily dependent on mineral composition, and independent of stress history, original structure, strain rate and other factors which dominate the path dependent properties of a soil, reasonable rates of displacement (0.00098 and 0.00144 inches per minute) were chosen for all tests. These rates are in keeping generally with the recommendations of Cullen and Donald (1971), who suggested a speed of 0.001 inches per minute for fissured overconsolidated clays

Intact samples of soft bentonitic clayshale from the failure plane were not obtained as this material had been completely disturbed during drilling. The material was however saved and later remoulded and prepared for multi-stage reversal direct shear tests. Normal stresses for the three stages were in the order of 50, 20 and 10 p.s.i. It has been demonstrated by several workers (e.g. Petley, 1966 and Tchalenko, 1967) that shearing along a precut plane is the most suitable method for determining the residual shear strength of a clay. This helps achieve alignment of the particles along the plane, and furthermore requires smaller amounts of horizontal displacement (and hence less numbers of reversal cycles) to attain the residual state. This lessens the amount of extrusion during the test, which is usually a problem during the later stages of conventional reversal shear tests. For these reasons, it was found preferable to use precut planes to determine the residual strength in these tests.

The samples were first mixed to form a uniform slurry, at a moisture content slightly greater than the liquid limit, thus ensuring full saturation. The slurry was then remoulded into a deep shear box, specially adapted for consolidating samples. The samples were then loaded in stages up to a normal stress (100 p.s.i.), twice the value of normal stress required for the first stage of shearing.

Overconsolidating the sample in this way helps prevent significant volume change and extrusion during shear. Consolidation readings were noted during the last stage of loading ~~to calculate the coefficient~~ of consolidation. The sample was then unloaded to the required normal stress for the first stage of the test, and allowed to equilibrate.

As with previous tests, the two halves of the box were separated by turning the screws one half to three quarters of a turn, and then the screws were removed. A plane was cut through the middle of the sample using a piano wire.

The sample was first sheared for 6 cycles at a rate of 0.072 inches per minute. The reason for this was to achieve the residual structure quickly. Following this the sample was sheared at a rate of 0.00096 inches per minute for as many cycles as was required to ensure that the sample was in fact at its residual value.

At this point the test was stopped and the sample unloaded to the second normal stress level, for the second stage of the shear test. The sample was allowed to equilibrate at this stress, and then sheared using the same techniques as described above, and similarly for the third stage of the shear test.

4.4.2 Direct Shear Test Results

Shear strength versus displacement curves for the stiff fissured clayshale samples (A1 to A5) are presented in

Appendix C in Figures C1 to C5. The curves are characterised by large drops from the peak to residual strength, which is usually reached after 2 to 2.5 inches (5 to 6 cms) displacement.

A phenomenon which occurred in each test was a different shear stress-displacement curve for the tension cycle and the compression cycle. The plots had higher shear strengths in the tension cycle, this being the direction of initial shear in each case, than in the compression cycle. Although the exact reason for this variation is not known, several factor may contribute to the difference in strengths. 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. This idea was supported by examination of the failure plane of these samples, which all revealed non-planar, hummocky surfaces, most likely caused by hard inclusions in the samples. Bending in the yoke which transmits the load to the load cell may also result in different shear stress readings in the two directions. It is also possible that the zero reading of the load cell may change during the test period. However, as the exact reason for the two displacement curves is uncertain, the practice adopted in these tests was to take the average of the tension and compression cycles.

No area corrections were applied for obtaining the normal effective stress on the samples. This was the procedure adopted by Kenney (1967), who stated that area correction affects both the shear stress and the normal effective stress and hence / ' remains constant. This practice was also adopted by Cullen and Donald (1971). The Mohr envelope for this series of test is shown in Figure 4.1.

Shear strength versus displacement curves for weathered silty clayshale samples (B1 to B6) are shown in Appendix C in Figures C6 to C11. Figure 4.2 shows the Mohr envelope for this series of tests.

Two series (C and D) of multi-stage reversal tests were carried out on the remoulded clayshale. Results of these tests are presented in Appendix C. Series C tests are shown in Figures C12 to C14 and series D tests in Figures C15 to C17. The Mohr envelopes for both series of tests are shown in Figure 4.3, and two different residual angles of shearing resistance of 5 and 8 degrees respectively were obtained.

A summary of the shear strength results is given in Table IV.3.

4.5 Interpretation of Results

4.5.1 Index Tests

The index tests and limited mineralogical data performed on the profile soils gave results, similar to those obtained from the Upper Cretaceous shales of the

Edmonton Formation, outlined by Thomson and Morgenstern (1974). All profile soils have relatively high proportions of clay sizes, with smaller proportions of silts sizes and almost no sand sizes present in the clayshales. The bentonitic clayshale, which makes up the major portion of the failure plane has a clay size portion of 76%, of which, approximately 75% is composed of montmorillonite. This value is reflected in the liquid limit of 143%. The liquid limits of the clayshales also indicate a large proportion of montmorillonite present in the clay fraction.

All soils exhibit placticity, with plastic limits ranging between 17% and 30%. In most cases the natural moisture content coincides approximately with the plastic limit, or in some cases, is slightly less than this limit, hence making the liquidity index negative.

4.5.2 Direct Shear Tests

The Mohr envelope for the peak stresses of the unweathered silty clayshales (Figure 4.1) indicates c' = 3340 p.s.f. and Øp' = 41°. It should be noted that several different interpretations of the peak strength envelope are possible due to the variation of the peak strength results. Clearly, it would require several more points in order to define the peak strength envelope with complete confidence. The interpretation shown in Figure 4.1 is however the most reasonable for the results obtained. These results, although appearing quite high, are comparable to the results tabulated by Locker (1969) on siltstone and clayey siltstone rocks of Central Alberta. He recorded angles of shearing resistance of 40° for siltstones and 35° for clayey siltstones, and also comparatively high values of cohesion, which would indicate the presence of cementation. The high values recorded in this test are related to the mineralogical composition of the clay fraction and also to the proportion of silt sizes present (Table IV.1). Unfortunately no mineralogical data is available for this material, however the residual angle of 19° suggests that a much lower proportion of the clay sizes is composed of montmorillonite than the value of 75% obtained from the remoulded clayshale samples of the failure plane, despite a liquid limit of about 104%.

As mentioned previously, the slip surfaces observed in these samples were non-planar and rather hummocky. Few slickensides were observed on the slip surface. This type of surface was also observed in tests on undisturbed materials by Cullen and Donald (1971). Pennell (1969) tested several clayshales and found in most cases that the undisturbed residual angle was between 3° and 5° higher than the residual angle measured with precut plane samples. He concluded however, in the light of his evidence, that the undisturbed residual angle of shearing resistance more closely represented the actual field evidence, whereas Kenney (1967), Petley (1966) and Tchalenko (1967) showed that shearing along a precut plane is the most suitable method for determining the residual shear strength of a clay, as obtained from tests on actual slip surfaces. The true residual angle of shearing resistance may therefore lie between 16° - 19° for this material.

Results of the tests performed on the weathered silty clayshales (Figure 4.2) gave peak strength parameters of, C' = 1430 p.s.f. and Ø' = 23°, and Ø' = 10°. The proportion of clay in the samples was 66% and the liquid limit was about 80%. The variability of the natural material is seen from the peak strength envelope (Figure 4.2). This variability is probably attributed to the amount of cohesion in each sample, due to the brecciated nature of the material. Slickensides were observed along the slip surface similar to those observed along some planes in-situ in pit 5.

The Mohr envelopes for the remoulded bentonitic clayshales (Figure 4.3) indicate residual angles of shearing resistance of 5° and 8° for series C and series D tests respectively. This material has a liquid limit of 143% and clay fraction of 74%. The mineralogical analysis indicates that approximately 75% of the clay sizes are montmorillonite, which accounts for 55% of the total sample. It is felt that an angle of 8° obtained from series D tests is more representative than the 5° obtained from series C tests. Series C tests were performed using a stiffer load cell and the readings obtained were beyond the lower limit of the load cell's capability, whereas a more sensitive load cell was used for series D tests. A residual angle of 8° is therefore taken as being applicable and, in addition, is in good agreement with previous work carried out on the bentonitic clayshales of the Edmonton Formation (Locker, 1969; Thomson, 1970 and 1971; Eigenbrod and Morgenstern, 1972), although residual angles as low as 2.6-3.6° have been recorded for the Bearpaw shales of the South Saskatchewan Dam by P.F.R.A. (1970).

TABLE IV.1

SUMMARY OF CLASSIFICATION TESTS

MATERIAL	W	Wp	WL	Ip	% Sand	% Silt	% Clay
WEATHERED SILTY CLAYSHALE	24	21-28	64-68	40-43	7	46	47
UNWEATHERED SANDY CLAYSHALE	23	17.5	41.5	24	30	**	
UNWEATHERED SILTY CLAYSHALE	20-24	23-27	84-104	57-80	4	46	50
BENTONITIC CLAYSHALE*	22-45	26-30	126-149	100-123	1	25	76

* SAMPLE TAKEN FROM FAILURE PLANE
** NO HYDROMETER ANALYSIS

TABLE IV.2

(a) MINERALOGICAL COMPOSITION OF CLAY FRACTION OF BENTONITIC CLAYSHALE

% MONTMORILLONITE	% MICA	% KAOLINITE
75	15	10

Results considered accurate within ± 10%

(b) PHYSICO-CHEMICAL PROPERTIES OF BENTONITIC CLAYSHALE

	Ca^{++} Me/100g	Mg^{++} Me/100g	Na^{+} Me/100g	K^{+} Me/100g
EXCHANGEABLE CATIONS	18.96	16.24	6.08	1.20
WATER SOLUBLE CATIONS	.187	.498	5.55	.173

CATION EXCHANGE CAPACITY = 42.8 Me/100g.

TABLE IV.3

SUMMARY OF SHEAR STRENGTH RESULTS

MATERIAL	SHEAR STRENGTH PARAMETERS			
	PEAK C' (p.s.f.)	PEAK ϕ' (DEGREES)	RESIDUAL C' (p.s.f.)	RESIDUAL ϕ' (DEGREES)
UNWEATHERED SILTY CLAYSHALE	3340	41	0	16-19
WEATHERED SILTY CLAYSHALE	1430	23	0	10
REMOULDED BENTONITIC CLAYSHALE	–	–	0	8

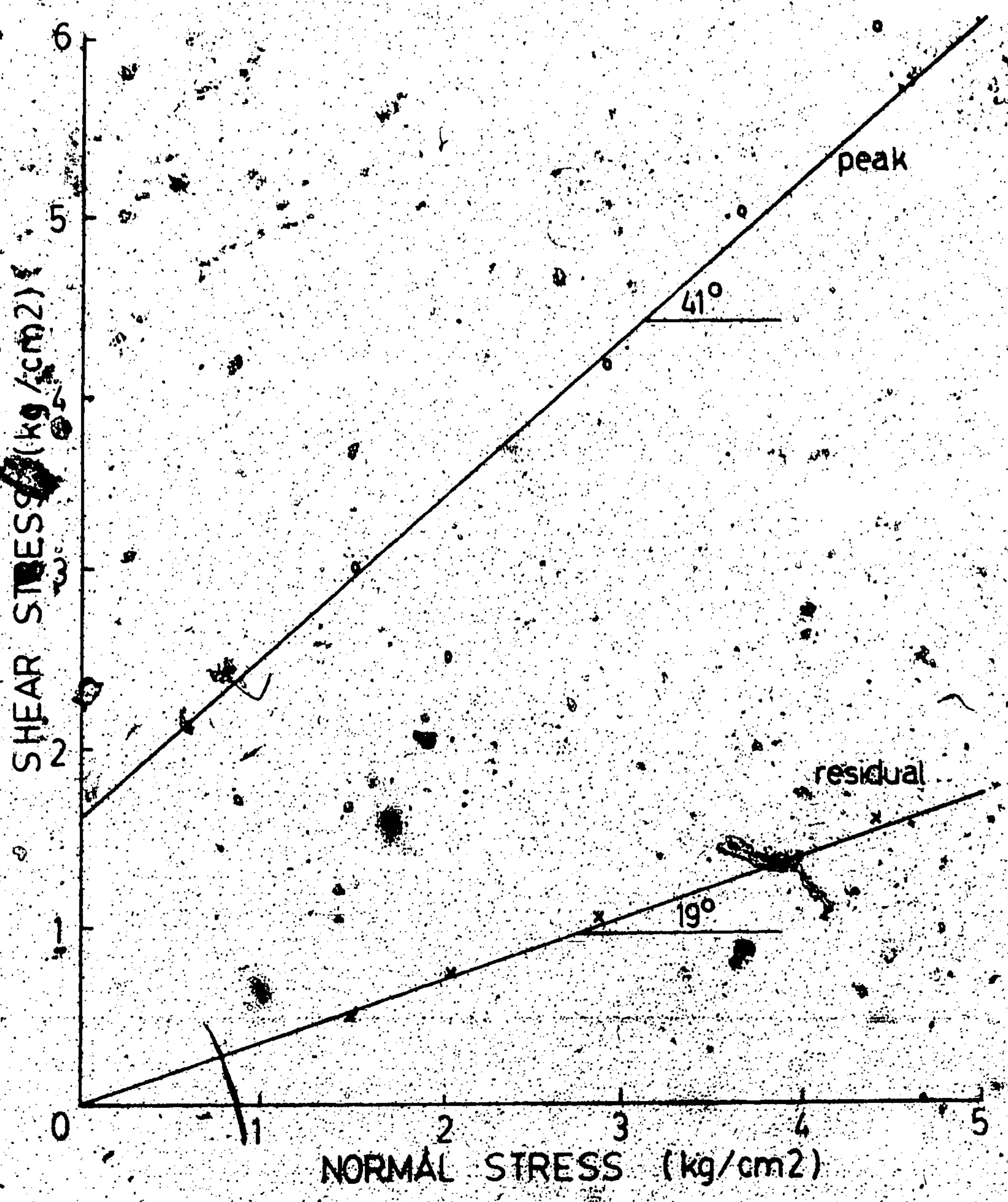


FIGURE 4.1 MOHR ENVELOPE FOR UNWEATHERED SILTY CLAYSHALE

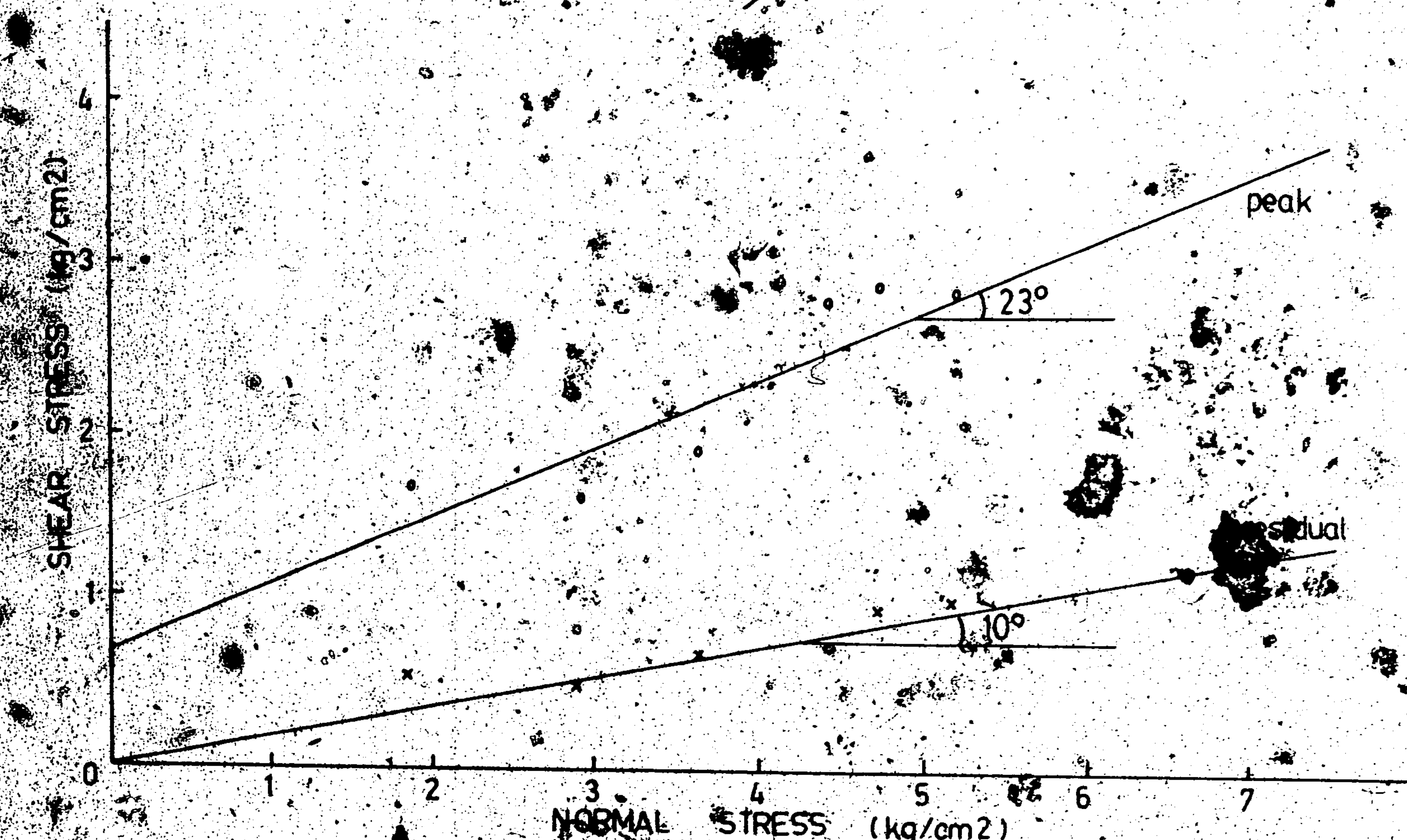


FIGURE 4.2 MOHR ENVELOPE FOR [illegible] SILTY CLAYSHALE

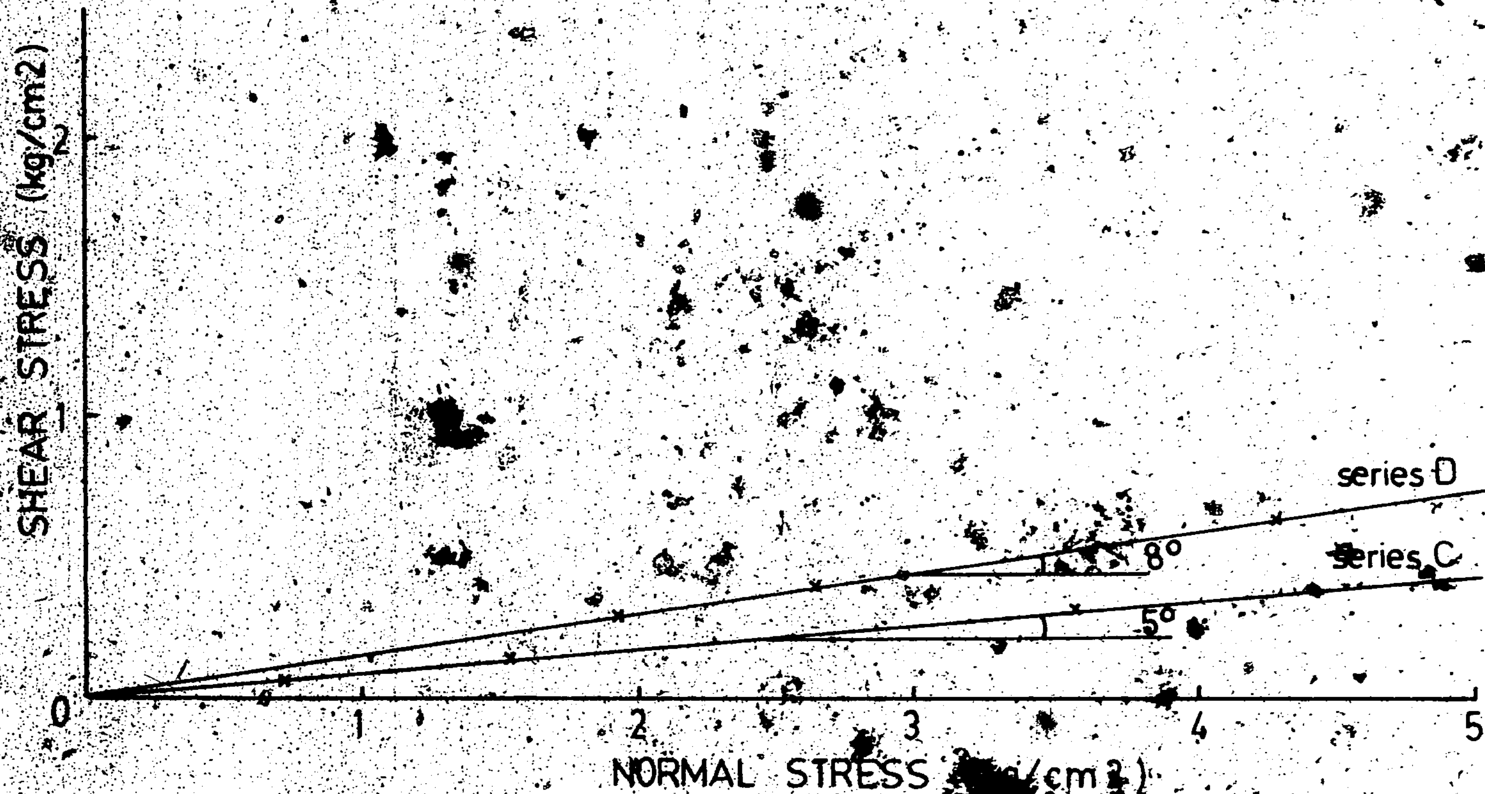


FIGURE 4.3 MOHR ENVELOPE FOR REMOULDED BENTONITIC CLAYSHALE

CHAPTER V

STABILITY ANALYSIS AND DISCUSSION

5.1 Proposed Mechanism of Failure

There is little doubt, from topographical considerations, that this site is an area of very old landsliding, and that the recent landslides are the continuation of a retrogressive series of block failures which has resulted in the present day gentle slopes.

This retrogressive mechanism may suggest a stepped failure plane, with sub-horizontal failure surfaces almost parallel to the regional bedding. It is not possible to ascertain from the borings alone the exact locations of such blocks, as the failure surface has only been located at three points along its entire length. The dimensions of these blocks can only be surmised from the ground topography, and in particular, the breaks in the ground surface, which may indicate successive phases of sliding. An interpretation of the stepped failure plane, which is analysed in section 5.2, is shown in Figure 5.1. In this figure the failure plane is seen to dip at 2.5° between successive steps.

A second and possibly more feasible failure plane is the semi-flat surface, sloping at 4.5° to the horizontal, as indicated by the slope indicators S2, S4 and S7. This failure surface is shown in Figure 5.2. The actual failure plane may in fact lie somewhere between these two limiting cases, being the result of a gradual flattening of the 'steps' during the several phases of sliding, which have preceded the recent landslides. This would lead to a semi-planar failure surface, or in the limit, to the flat failure surface shown in Figure 5.2. The actual failure surface can be more accurately described only with the installation of several more slope indicators down the slide profile.

Previous landslide movements suggest that the angle of shearing resistance is at residual along the major part of the failure plane from the toe inwards. Only the backscarp, which has not been subjected to previous movements, and that section of the failure plane closest to the backscarp can be expected to mobilise angles of shearing resistance at, or close to their peak values.

For the same reason it is proposed that zero cohesion is being mobilised along the horizontal portion of the failure plane. But perhaps the important factor towards explaining the mechanism of failure, is ascertaining the cohesion acting along the backscarp of the slide mass.

Earlier site investigation evidence has shown that the bedrock is higly weathered and brecciated down to about 35 feet. Below this depth joints and fissures are still prevalent and were [illegible] in core samples at depths of 90 feet. These joints were usually flat surfaces, and in some cases showed signs of softening between the joints. The opening of joints at this depth may have been the result of lateral stress relief caused by previous landslide activity, a mechanism postulated by Terzaghi in 1936. The weathered and brecciated material, accounting for the top 35 feet, has been responsible for the easy ingress of ground and rainwater and it is conceivable that this water has been able to percolate to greater depths by means of these joints and fissures, and hence initiate the process of softening between the joints. It is therefore felt that the cohesion has been almost completely lost along the backscarp slide plane.

This gradual loss of cohesion, coupled with the rise in pore pressure, due to the high spring runoff, during the spring of 1974, are therefore proposed as being the cause of failure at this time. Several factors may have helped the build up of an unfavorable piezometric level. Snow drifts up to 20 feet in height were observed to accumulate along the fence line at the top of the scarp area, during the winter prior to the failure of the North Slide. This snow melt would be expected to drain directly into the existing cracks along the top of the scarp.

Similarly, the fields along the top of the landslide are inclined towards the scarp area and snowmelt was observed draining in channels down the fields and into these cracks in the scarp area.

The piezometric data in Figure 3.4 indicates that there is a critical piezometric level reached during the period of thaw, when spring runoff is percolating into the cracks in the scarp area but discharge from the toe of the slide is being prevented by the frozen ground. This situation was observed in March 1976. At this stage the snow along the top of the slope had completely thawed and water was seen "flowing" down cracks in the ground along the scarp area, and yet snow cover still protected the lower part of the slopes and maintained the frozen state. One week later the snow cover had melted and large pools of water were observed below the toe of the slide.

The fact that the South Slide has not yet failed catastrophically, but is creeping constantly, may indicate that the sli[illegible] is dependent upon reaching a critical piezometric [illegible] which has not yet been reached.

5.2 Stability Analysis

5.2.1 General

The purpose of a stability analysis is to establish a rational explanation for the development of these landslide movements. Thus the Morgenstern and Price stability analysis of a general slip surface was used to analyse the South Slide and to ascertain the strength parameters being mobilised by the slide materials at failure.

Briefly, the Morgenstern and Price stability analysis is based on the concept of limit equilibrium, but has the advantage over most other such analyses in that all the equations of static equilibrium are satisfied. The surface of the potential slide mass, the piezometric surface and the proposed failure surface are all specified, as a series of straight lines. The strength parameters being mobilised by the failure surface are also input. The sliding mass is then analysed as a series of slices, whose equilibrium is considered in sequence and whose summation, satisfying the boundary equations, gives the overall stability. Several different failure surfaces, and combinations of strength parameters can be analysed quickly using the computer programme. A full discussion of the method is presented by Morgenstern and Price (1965, 1967).

The surface profile was specified, as surveyed in May, 1975, which was essentially the same as the pre-failure profile. Unfortunately, the piezometric [illegible] in the slide mass at failure was not known, however, it was possible to deduce the approximate elevation of the piezometric head from piezometric data recorded during the fall 1975 and spring 1976.

Two different modes of failure were analysed, viz;

1. The straight failure plane as obtained from the slope indicator readings, dipping at an angle of 4.5° to the horizontal, as shown in Figure 5.2.
2. The stepped failure plane indicating a block type failure mechanism, as described earlier and shown in Figure 5.1. One and 2.5° angles of dip of the failure plane between successive steps were analysed separately.

Three basic soil types were specified as shown in both

Figures 5.1 and 5.2. Soil type 1 consisted of till and accounted for only the top 13 feet of material. Soil type 2 consisted of weathered silty clayshale, which paralleled the ground surface at a depth of between 35 and 45 feet, and soil type 3, was the unweathered silty clayshale underlying the weathered material. A summary of the soil properties and strength parameters used in the stability analysis is given in Table V.1, and an explanation of the choice of strength parameters for the stability analysis is given in the following section.

5.2.2 Choice of Strength Parameters

Table V.1 summarises the laboratory results obtained for the major components of the backscarp and the failure plane materials, and also indicates the strength parameters used as a basis for the stability analysis. In addition to the strength parameters shown in Table V.1, several other combinations were also tested, in particular the effect of an increasing cohesion intercept with the unweathered silty clayshales of the backscarp. A peak angle of shearing resistance of 24° and zero cohesion were the assumed properties of the till, in the absence of laboratory data on this material. A lack of cohesion was apparent from the presence of columnar joints in the till.

A notable anomaly is the peak angle of 41° for the unweathered silty clayshales of the backscarp. On reviewing the laboratory testing programme, it is felt that the strength parameters obtained from shear tests on the intact blocks of clayshale are not representative of the strength parameters being mobilised along that portion of the backscarp in-situ, which is proposed as being a series of interconnected joints. Zero cohesion would be available in the joints, however it would also be suspected that softening and weathering processes may have taken place along the sides of these joints, resulting in a disaggregation of the shale particles, as well as a general disruption of the soil structure, and consequently a decrease in the angle of shearing resistance, which, in the limit may fall to the value of 23°, as obtained from test on the weathered materials. The effect of reducing the angle of shearing resistance of this material is therefore investigated.

The major portion of the lower part of the failure surface is thought to have been part of very old landslides, and hence residual strength parameters of 8° in the unweathered bentonitic clayshale and 10° in the weathered material is applied to this section. The effect of a small cohesion intercept with this material is also investigated.

One section of the failure plane, closest to the backscarp (Figure 5.2) is however thought to have been a first time movement. It would therefore be suspected that strength parameters greater than residual are being mobilised along this section of the failure plane. No peak strength results are available for this material, however a peak angle of shearing resistance of 14° may be assumed, as was recorded for s[illegible]tonitic shales of the Edmonton Formation (Thomson 1970, 1971).

5.2.3 Stability Analysis Results

Flat Failure Surface

Results of the Morgenstern and Price stabiliy analysis preformed on the slide profile shown in Figure 5.2 are summarised in Table V.2. It should be noted that the factors of safety shown in Tables V.2. and V.3. are given to three decimal places only to compare the effects of increasing cohesion and piezometric pressure. These results are however considered accurate only to one decimal place, considering the practicalities of the stability analyses. The following points are of note:

1. A factor of safety of 0.893 was obtained for trial surface 1 which specifies residual strength parameters for both the backscarp and the failure plane. Similarly, assuming residual strength parameters along the failure plane, and zero cohesion in the backscarp but with peak angles of friction gives a factor of safety of 0.912 assuming Ø'=23°, for the unweathered material of the backscarp (trial surface 2), and a factor of safety of 0.941 if Ø'=41° is assumed. (trial surface 3).
2. Trial surfaces 4, 5 and 6 use the strength parameters tabulated in Table V.1. The residual angle of 8° was used along the pre-failed portion of the failure plane. However the angle of shearing resistance increases to 10° and then

14° in the unfailed sections of the failure plane, sections 5 and 4 respectively in Figure 5.2. Using these strength parameters, a factor of safety of 1.056 is obtained (trail surface 4). This factor increases to 1.059 if an angle of shearing resistance of 10° is specified in the weathered material closest to the toe, in section 8 (trial surface 5). This value increases to 1.090 if Ø' for the unweathered backscarp material (section 3) is increased from 23° to 41° (trial surface 6).

3. The effect of an increasing cohesion in the unweathered backscarp material is investigated in trial surfaces 7, 8 and 9. Cohesion intercepts of 1000, 2000 and the peak value of 3000 p.s.f. are applied to the clayshales and factors of safety of 1.091, 1.112 and 1.154 are obtained respectively. This demonstrates that, the factor of safety is relatively insensitive to cohesion in the backscarp, but indicates that a very low value of C', much less than that measured was acting.

4. In comparison the effect of a small cohesion intercept of 100 p.s.f. in the failure plane material is shown in trial surfaces 10 and 11. Adding 100 p.s.f. to the clayshale in section 4 only (Figure 5.2) increases the factor of safety from 1.059 to 1.070, however if 100 p.s.f. is applied along the entire length of the failure surface, the factor of safety increases to 1.225. This shows that the factor of safety is highly sensitive to any cohesion on the lower portion of the failure surface.

5. The effect of decreasing the piezometric level is

demonstrated in trial surface 12. In this analysis, the piezometric level recorded in the fall of 1975 was inserted, corresponding to a drop in piezometric level of about 8 to 10 feet, and an associated increase in factor of safety from 1.059 to 1.172.

The preceding points indicate that the factor of safety is most sensitive to the strength parameters along the essentially horizontal portion of the failure plane, and also the piezometric level, and is least sensitive to both cohesion and angle of shearing resistance of the backscarp materials.

It may be concluded from these considerations, that the most feasible solutions giving a factor of safety close to unity are trial surfaces 4 and 5. This indicates that peak angles of shearing resistance but almost zero cohesion are being mobilised in the backscarp, with residual angles of shearing resistance and zero cohesion acting in the prefailed section of the failure surface, and an angle of shear[illegible]ance on average between peak and residual in the [illegible]ction of the horizontal part of the failure surface.

Stepped Failure Surface

Two stepped failure plane models were analysed. In the first model, the failure plane dipped at 1° to the horizontal between successive steps, hence necessitating steps up to 15 feet in height, and in the second model, a slope of 2.5° was specified, resulting in steps of between five and eight feet in height. (Figure 5.1). The results of the Morgenstern and Price stability analysis on these models are summarised in Table V.3.

It can be seen immediately that the factors of safety for these models, especially the 1° stepped failure plane, are much higher than the corresponding factors of safety for the flat failure plane (Table V.2). The 1° failure plane model gives a unreasonably high factor of safety of 1.259 assuming Ø'=23° in the unweathered backscarp material and 1.409 assuming Ø'=41°, and in both cases assuming a residual angle of shearing resistance along the horizontal portion of the failure plane of 7°. The corresponding factors of safety for the 2.5° failure plane are 1.119 and 1.245.

Indeed, to obtain a factor of safety close to unity requires assuming a residual angle of shearing resistance of 6° along most of the failure plane, with a 10° angle along the first block, and with peak angles in the backscarp but zero cohesion. This analysis gives a factor of safety of 1.060 (trial surface 2d).

It is therefore concluded from a consideration of the results of both types of failure planes, that if the soil strength parameters specified in Table V.1 are the parameters being mobilised by the sliding material, then to obtain a factor of safety close to unity requires that the failure plane must approach, in the limit, the case of an essentially linear failure plane with an average dip of

4.5°.

5.3 Discussion

5.3.1 Stability Analysis

The following points can be concluded from the results of the stability analysis presented in the previous section;

1. In order to obtain a factor of safety close to unity, residual angles of shearing resistance, as obtained from laboratory tests, and zero cohesion were being mobilised along the major portion of the essentially horizontal part of the failure plane, when slip occurred.
2. Peak angles of shearing resistance were being mobilised by the materials comprising the backscarp, however there had been almost a complete loss of cohesion in the backscarp material prior to failure.
3. In order to reduce the factor of safety to unity, a piezometric level slightly in excess of that level recorded in early spring, 1976, was required. This would indicate that the piezometric pressure acted as a final triggering mechanism for the landslide.
4. It would appear from the analysis, that a stepped failure plane, the results of which are summarised in Table V.3 is an inappropriate model.

The results summarised in Table V.2 indicate that the factor of safety is most sensitive to the strength parameters being mobilised along the flat lying portion of the failure plane and also to the position of the piezometric head. The factor of safety is least sensitive to the strength parameters of the backscarp.

The strength parameters used in the stability analysis to obtain a factor of safety of unity, are specified in Table V.1. Residual angles of shearing resistance of 8° and 10° were applied respectively to the unweathered and weathered materials, making up that portion of the failure plane, which had formed part of the very old landslide failure plane, that is, sections 6, 7 and 8 in Figure 5.2. The remaining sections of the horizontal portion of the failure plane, i.e. sections 4 and 5, which are proposed as being a first time failure were, on average, mobilising angles of shearing resistance between peak and residual values. Unfortunately no peak strength parameters were obtained for this material, and hence a peak angle of 14° was assumed, as recorded in similar shales of the Edmonton Formation (Thomson, 1970, 1971). An angle of 14° was therefore applied in section 4 (Figure 5.2), and stability analysis indicated a reduced value of 10° was acting along the failure plane in section 5. This increase in ϕ' along the length of the failure plane, is similar to the approach taken for the Lesueur Landslide (Thomson, 1971). It is suggested that the decrease in ϕ' from the peak value to a value of 10°, in the unfailed section, may have been due to previous creep movements, or possibly as a direct result of valley rebound (Matheson, 1972).

Although the factor of safety is relatively insensitive to the strength parameters being mobilised along the backscarp, the most reasonable solution requires that peak angles of shearing resistance are being mobilised, with zero cohesion in the weathered bedrock, and at most, a small cohesion in the unweathered bedrock. The peak cohesion intercept, obtained from tests on intact samples of the unweathered clayshale, was 3340 p.s.f., and thus, even if a small cohesion was being mobilised, it still infers that a large decrease in cohesion has occurred prior to failure. Results of this series of tests also indicated a peak angle of shearing resistance of 41°. It is, however, suggested that weathering along the joint surfaces in the fissured bedrock may have reduced this value considerably. Joints were observed in the core samples recovered, at depths of 80 feet. It would therefore be expected that, in the vicinity of the backscarp, ingress of water along these joints would have caused considerable softening and weathering. Laguros et al (1974), investigating the field weathering of some Oklahoma shales, observed that such weathering produced disaggregation of the shale particles, i.e. an increase in the amount of clay particles in each case, and in addition, field weathering produced degradation of the clay particles. They also proposed that weathering over a long period of time could conceivably alter the clay mineral types. This increase in clay particles, coupled with the general disruptions in the soil structure, resulting from the weathering processes, is therefore responsible for the reduction in $\emptyset'$ along the joints of the backscarp. The angle of shearing resistance obtained for the weathered clayshales was 23°, and hence, if this value is assumed as a lower limit for ϕ' along the joint surfaces of the unweathered material, a factor of safety of 1.059 is obtained, in comparison with 1.090, when $\phi' = 41°$ is specified in the stability analysis. The angle of shearing resistance for this material may in fact lie somewhere between the two limits, resulting in a factor of safety also between the limits. Hence it must be concluded that the laboratory tests performed on the intact blocks of the unweathered clayshale, which yielded results of $C' = 3340$ p.s.f. and $\phi' = 41°$ are not representative of the strength parameters mobilised along the joint surfaces.

Table V.3, summarizing the results of the stability analyses assuming a stepped failure plane (Figure 5.1) shows that in all cases the factors of safety obtained were much higher than the corresponding values assuming a flat failure plane. It must therefore be concluded that although the idea of a retrogressive failure mechanism appears valid, the idea of a stepped failure plane with sub-horizontal sections roughly parallel to local bedding, is inappropriate. If such 'steps' originally existed, then it must be interpreted that these have been rounded off and essentially flattened to a linear feature with successive block movements.

5.3.2 Mechanism of Failure

It is to be concluded from the previous discussion, that the recent slides are the continuation of the long term slide movements. The landslide topography, subdued by excessive erosion, indicates that previous landslides must have taken place at long intervals of time probably in the order of several centuries

It is postulated, that several stages of landslide activity have preceded the most recent slides, the result of which is a pre-sheared failure plane mobilising the residual angle of shearing resistance. The most recent landslide movements, have been a result of a very gradual decrease in the strength of the backscarp materials. This has made the slide mass only critically stable, and undoubtedly this has been triggered by the high piezometric level following the spring runoff, in 1974.

As a result of the previous landslides, the state of stress in the adjoining bedrock has been altered, due to a removal of the lateral support. This has resulted in a lateral expansion, and an opening of the joint and fissure systems in the bedrock, to a considerable depth. The strength of the bedrock is sufficient to hold these joints open, and hence has allowed an ingress of ground and rainwater, which has initiated the process of softening and weathering along the joint surfaces. This process continues until the clay along the joints is reduced to the "fully softened state" as described by Skempton (1970). The softening and swelling of the material in the joints consequently leads to the opening of more joints and the softening process continues. Hence there has been a slow deterioration in the strength of the backscarp material, manifested by a translation of the Mohr envelope downwards until it ultimately passes through the origin, that is, a loss of cohesion but a retention of the peak angle of shearing resistance. This process is described in detail by Skempton (loc. cit.).

It must not be construed that the cohesion had vanished entirely throughout the backscarp, but the presentation of the stability analyses (Table V.2) showed that when a cohesion intercept of 3000 p.s.f. was applied, a factor of safety well in excess of unity was obtained. However low values of cohesion along this part of the backscarp, had only a small effect on the factor of safety. Therefore, although a small cohesion value may have existed, this still implies that a large decrease from the test value of 3340 p.s.f. has had to occur prior to failure.

Indeed this case is very similar to that of the Devon Slide (Eigenbrod and Morgenstern, 1972). This slide also occurred in an area of very old landslides, mobilising residual angles along the base of the slide, with peak angles along the backscarp, but essentially zero cohesion. Again several centuries had elapsed since the time of the original failures, as attested to by the subdued topography. This slide was only critically stable prior to failure, as was indicated by the presence of cracks along the scarp area. The slope may have remained critically stable for many more years, until triggered by some natural force, such as high pore pressure as in the case of the Edgerton Slide, however the slide was triggered by a cut made into the slope for a highway embankment. It would therefore appear that many natural slopes in areas of old landslides may exist at very low factors of safety for a long time, until a fortuitous combination of factors initiates movement.

A progressive decrease in cohesion, as described above, is also proposed as the reason for failure in many of the case histories of first time slides in overconsolidated fissured London Clays, analysed and discussed by James (1970 and 1971).

It is worth noting the time aspect for this softening process to occur. In both the Edgerton and the Devon Slides, it would appear that several centuries were required to effect this softening process, whereas in most of the cases of embankment cuts made in the London Clays, failure occurred within 50 years. These embankments were up to 50 feet in height. In the case of the Quesnel Slide, Edmonton, failure occurred in 1974, about six years after the original embankment cut was made. The cut, which was about 25 feet in depth was through the lightly overconsolidated glacial lacustrine clay of Edmonton. Again cohesion had completely disappeared throughout the failure plane (Thomson, 1976). It would therefore be concluded that the time for this softening process depends on such factors as, the mass of soil, the nature and stress history of the soil, and also the availability of the softening agents.

Undoubtedly the triggering mechanism of the Edgerton Slide has been the increase in piezometric level which occurred during spring, 1974, when, the initial failure occurred causing a slight depression along the scarp area. This is not to say that at this time it was the highest piezometric level ever. The piezometric level was a high if not higher in previous years, however the necessary requirement is for the strength of the backscarp to have decreased to such an extent, that the factor of safety was low enough, such that a recurrence of a critical piezometric level was sufficient to lower the safety factor to a value near unity and hence initiate slide movements.

These small initial movements, accompanied by a subsequent drop in the piezometric level, may have been responsible for the temporary stabilising of the slides. It is however postulated that the continuous creep movements which have followed in both the North Slide and South Slide, have further reduced the strength parameters along those sections of the failure plane not yet at residual, and consequently also lowering the factor of safety of both slides. Such creep movements have been detected in the South Slide, which has not yet failed catastrophically, during the fall, 1975, and spring, 1976. In the case of the North Slide however, creep movements during the spring and summer of 1974, were enough to lower the safety factor sufficiently, such that catastrophic failure occurred in late August, 1974.

Catastrophic failure has not yet occurred in the South Slide, and in fact, to date no emergence of the toe of the slide has been observed. Projections of the lower part of the failure surface (as deduced from slope indicator data) indicates that the toe is in the order of 1200 feet horizontally from the scarp of the South Slide. The increasing scarp height, coupled with the movements detected in the slope indicators are clear evidence that initial failure has occurred

The slope indicator in BH2 became inoperative in August 1975, that in BH4 in November 1975 and that in BH7 in April 1976. Despite the fact that little movement is required to block off the slope indicator tube, nevertheless it is further evidence that failure is progressing from the scarp towards the toe area. This case history demonstrates that one cannot assume that failure will progress from the toe upslope. The direction in which failure will propagate in any particular landslide will depend upon site specific detail and also upon the triggering mechanism.

In seeking a tentative explanation of the fact that the toe has not yet emerged, whereas a drop of 7.5 feet has been recorded at the scarp it is suggested that the movements so far may have been absorbed within the slide mass. This would be effected initially by a closure of the joints and fissures within the soil mass, and possibly a very slight bulging of the soil in the lower part of the slide mass. In addition the geometry of the South Slide is more favourable, from a stability point of view, than that of the North Slide. First the sliding mass is about 200 feet longer than the mass of the North Slide, which most likely offers more resistance to movement. Secondly, the South Slide is slightly flatter than the North Slide, having an average slope of less than 10°, as compared with that of 11° recorded for the North Slide.

It must be concluded that scarp movements will continue in the South Slide until such time as the factor of safety is lowered sufficiently to cause catastrophic failure, or alternatively, triggered by sufficiently high piezometric pressures within the next year or so. No appreciable creep movements have been detected in the North Slide since catastrophic failure occurred, and it is to be concluded that this slide is now quasi-stable.

Finally it may be asked why some areas nearby, especially those having steeper slopes did not fail at the same time as this slope. To answer this question would require a careful study of the groundwater pattern of the entire area in detail. It is suggested that these areas may have more favourable drainage systems and consequently comparatively lower piezometric levels than in the slide areas. As mentioned previously, a system of ponds, sloughs and marshy areas exist to the west of the slide area, and it is suspected that these areas all drain indirectly into the slide zone. Secondly a local study of the topography along the valley wall will reveal the effects that old landslides have had on the present-day stability in these areas.

TABLE V.1

SUMMARY OF SOIL PROPERTIES USED IN STABILITY ANALYSIS

	MATERIAL	WET DENSITY p.c.f.	SATURATED DENSITY SAT p.c.f.	MEASURED STRENGTH PARAMETERS				STRENGTH PARAMETERS USED IN STABILITY ANALYSIS	
				PEAK		RESIDUAL			
				C' p.s.f.	φ' DEGREES	C' p.s.f.	φ' DEGREES	C' p.s.f.	φ' DEGREES
BACKSCARP	TILL	120	130	–	–	–	–	0	24
	WEATHERED SILTY CLAYSHALE	112	120	1470	23	0	10	0	23
	UNWEATHERED SILTY CLAYSHALE	126	126	3340	41	0	19	0	41 (23)
FAIL. PL.	BENTONITIC CLAYSHALE	140	140	–	–	0	8	0	8
	WEATHERED CLAYSHALE	112	120	1470	23	0	10	0	10

TABLE V.2

SUMMARY OF STABILITY ANALYSES ON FLAT FAILURE PLANE

TRIAL SPC. NO.	BACKSCARP STRENGTH PARAMETERS								FAIL. PLANE STRENGTH PARAMETERS								F	REMARKS
	1* TILL		2 WEATH. SILTY CLAYSHALE		3 UNWEATH SILTY CLAYSHALE		4 CLAYSHALE		5 BENT. CLAYSHALE		6 BENT CLAYSHALE		7 WEATH CLAYSHALE		8 WEATH CLAYSHALE			
	C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'		
1	0	18	0	18	0	18	0	8	0	8	0	8	0	8	0	8	.893	Residual
2	0	24	0	23	0	23	0	8	0	8	0	8	0	8	0	8	.912	Resid. Fail. Pl.
3	0	24	0	23	0	41	0	8	0	8	0	8	0	8	0	8	.941	Peak Backscarp
4	0	24	0	23	0	23	0	14	0	10	0	8	0	8	0	8	1.056	Parameters
5	0	24	0	23	0	23	0	14	0	10	0	8	0	8	0	10	1.059	from Table V.1
6	0	24	0	23	0	41	0	14	0	10	0	8	0	8	0	10	1.090	
7	0	24	0	23	1000	23	0	14	0	10	0	8	0	8	0	10	1.091	INCREASING
						41											1.112	COHESION
8	0	24	0	23	2000	23	0	14	0	10	0	8	0	8	0	10	1.112	IN BACKSCARP
						41											1.135	MATERIAL
9	0	24	0	23	3000	23	0	14	0	10	0	8	0	8	0	10	1.154	
						41											1.159	
10	0	24	0	23	0	23	100	14	0	10	0	8	0	8	0	10	1.071	COHESION
						41											1.101	ALONG
11	0	24	0	23	0	23	100	14	100	10	100	8	100	8	100	10	1.225	FAILURE PLANE
						41											1.260	
12	0	24	0	23	0	23	0	14	0	10	0	8	0	8	0	10	1.172	DECREASE IN
						41											1.206	PIEZO.- LEVEL

* Numbers refer to sections shown in Figure 5.2

TABLE V.3

SUMMARY OF STABILITY ANALYSES ON STEPPED FAILURE PLANE

TRIAL SPC NO	ANGLE OF DIP	BACKSCARP						FAILURE PLANE										P	REMARK
		TILL		WEATH. CLAYSHALE		UNWEATH. CLAYSHALE		BLOCK 1*		BLOCK 2		BLOCK 3		BLOCK 4		BLOCK 5			
		C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'	C'	ϕ'		
1	1	0	24	0	23	0	41	0	14	0	8	0	8	0	8	0	8	1.693	STATICALLY UNACCEPTABLE
2 a	1	0	24	0	23	0	23	0	10	0	7	0	7	0	7	0	7	1.259	$\phi'=23°$ IN BACKSCARP
b	2.5									0	7	0	7	0	7	0	7	1.119	
c	2.5									0	8	0	8	0	8	0	8	1.180	
d	2.5									0	6	0	6	0	6	0	6	1.060	
3 a	1	0	24	0	23	0	23	0	10	0	7	0	7	0	7	0	10	1.274	
b	2.5									0	7	0	7	0	7	0	10	1.130	
c	2.5									0	8	0	8	0	8	0	10	1.188	
d	2.5									0	6	0	6	0	6	0	10	1.074	
4 a	1	0	24	0	23	0	41	0	10	0	7	0	7	0	7	0	7	1.409	$\phi'=41°$ IN BACKSCARP
b	2.5									0	7	0	7	0	7	0	7	1.245	
c	2.5									0	8	0	8	0	8	0	8	1.309	
d	2.5									0	6	0	6	0	6	0	6	1.186	
5 a	1	0	24	0	23	0	41	0	10	0	7	0	7	0	7	0	10	1.423	
b	2.5									0	7	0	7	0	7	0	10	1.256	
c	2.5									0	8	0	8	0	8	0	10	1.317	
d	2.5									0	6	0	6	0	6	0	10	1.197	

* Numbers refer to blocks in Figure 5.1

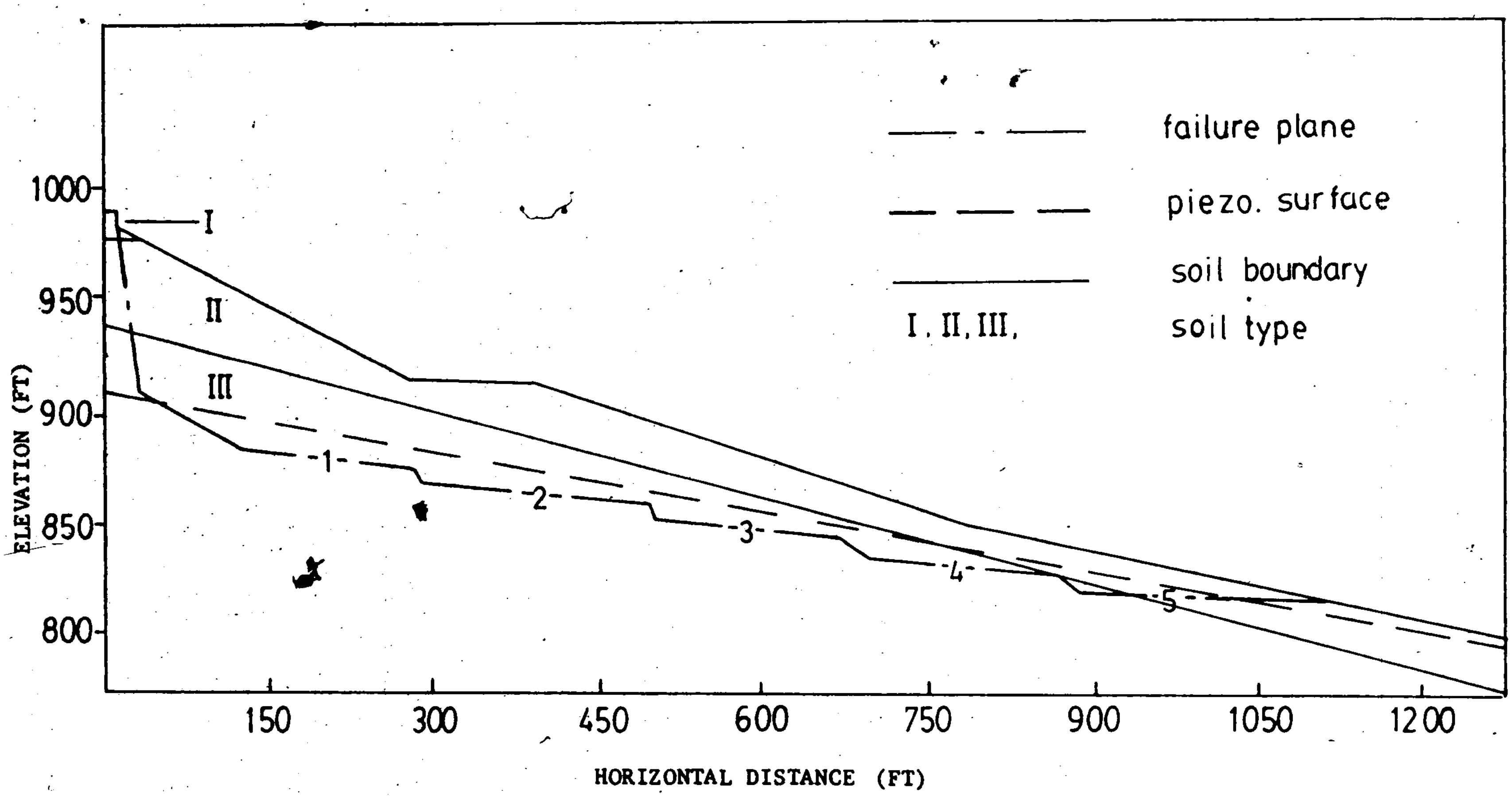


FIGURE 5.1 EDGERTON SLIDE
- USING STEPPED FAILURE PLANE

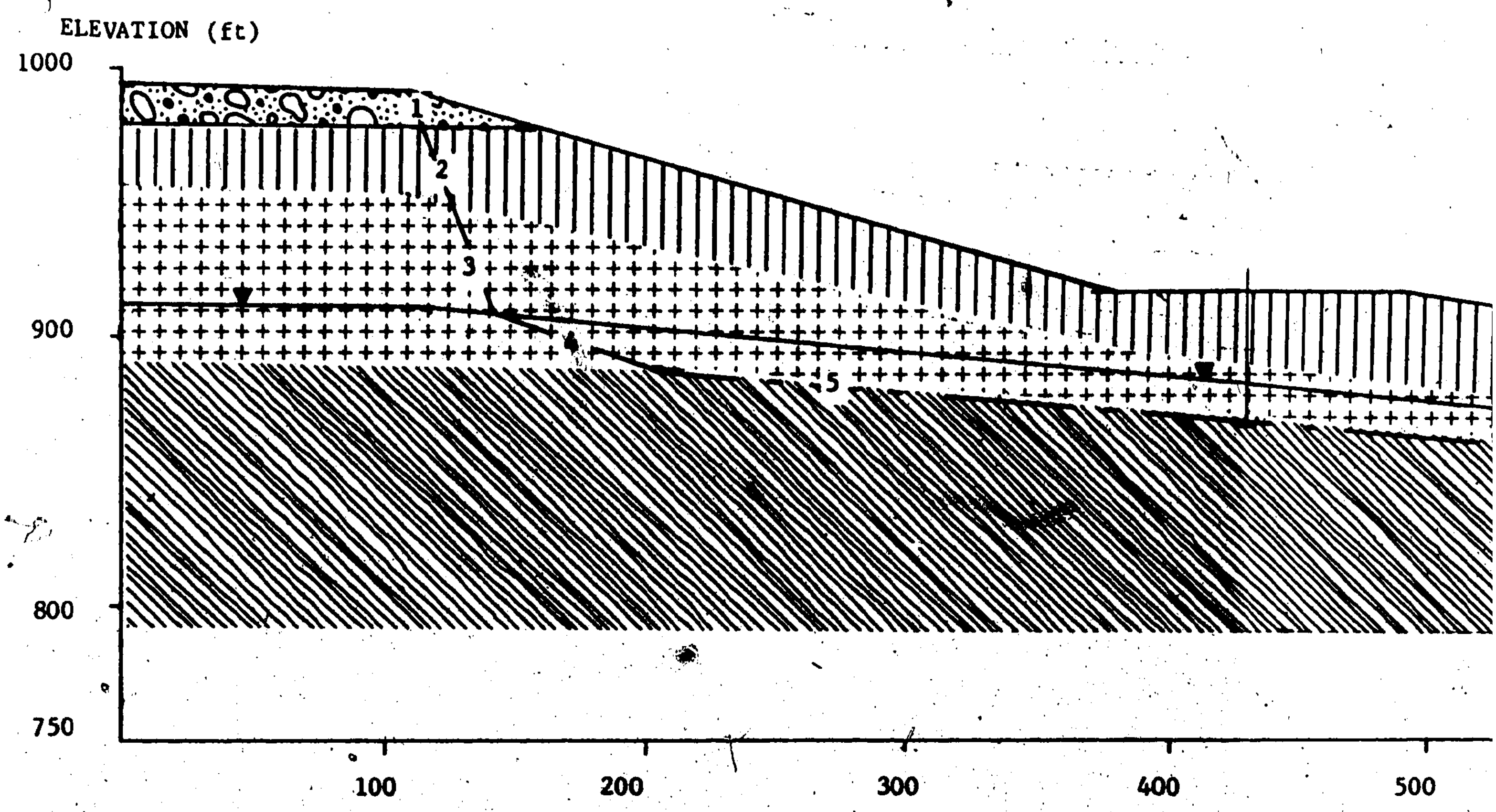
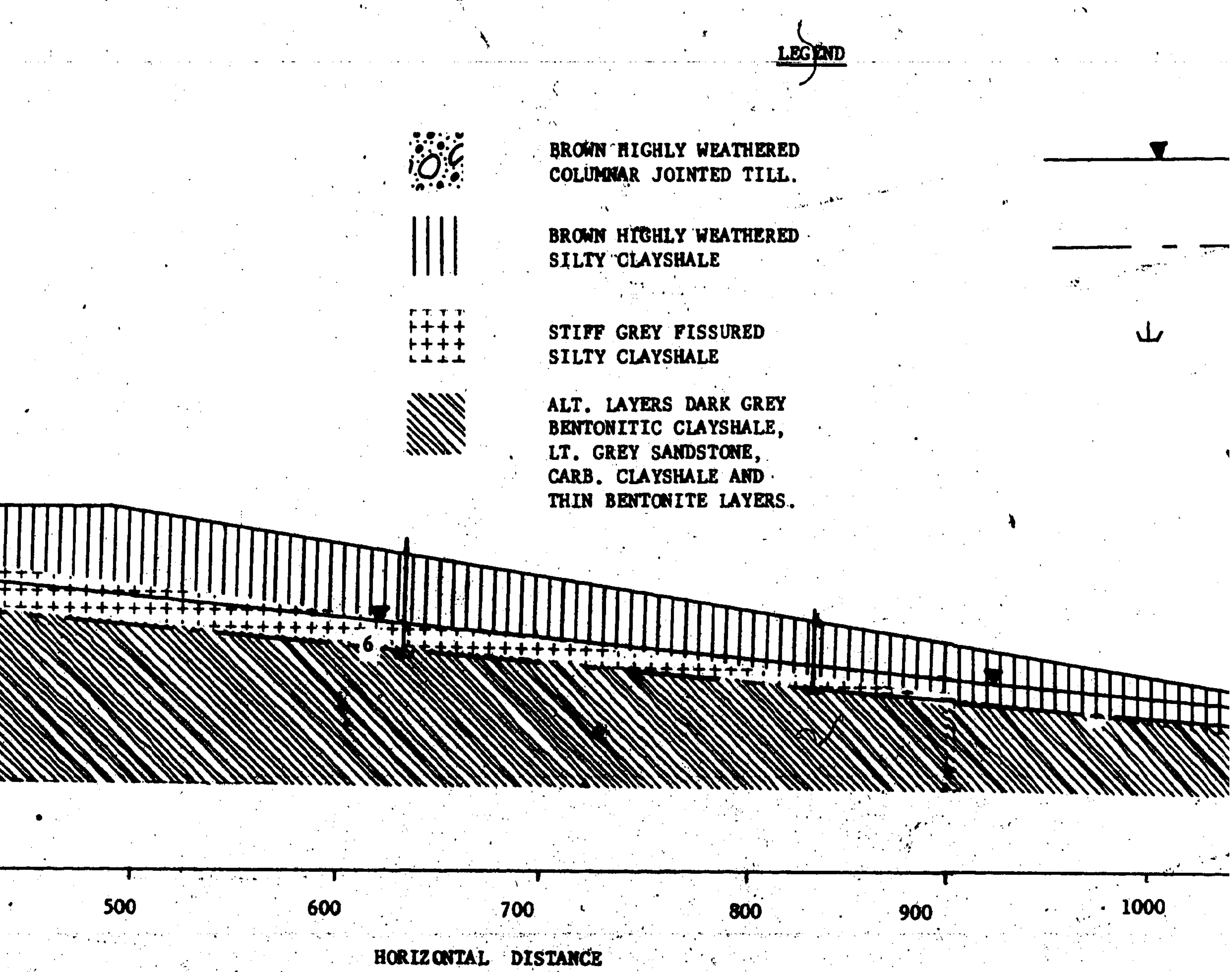
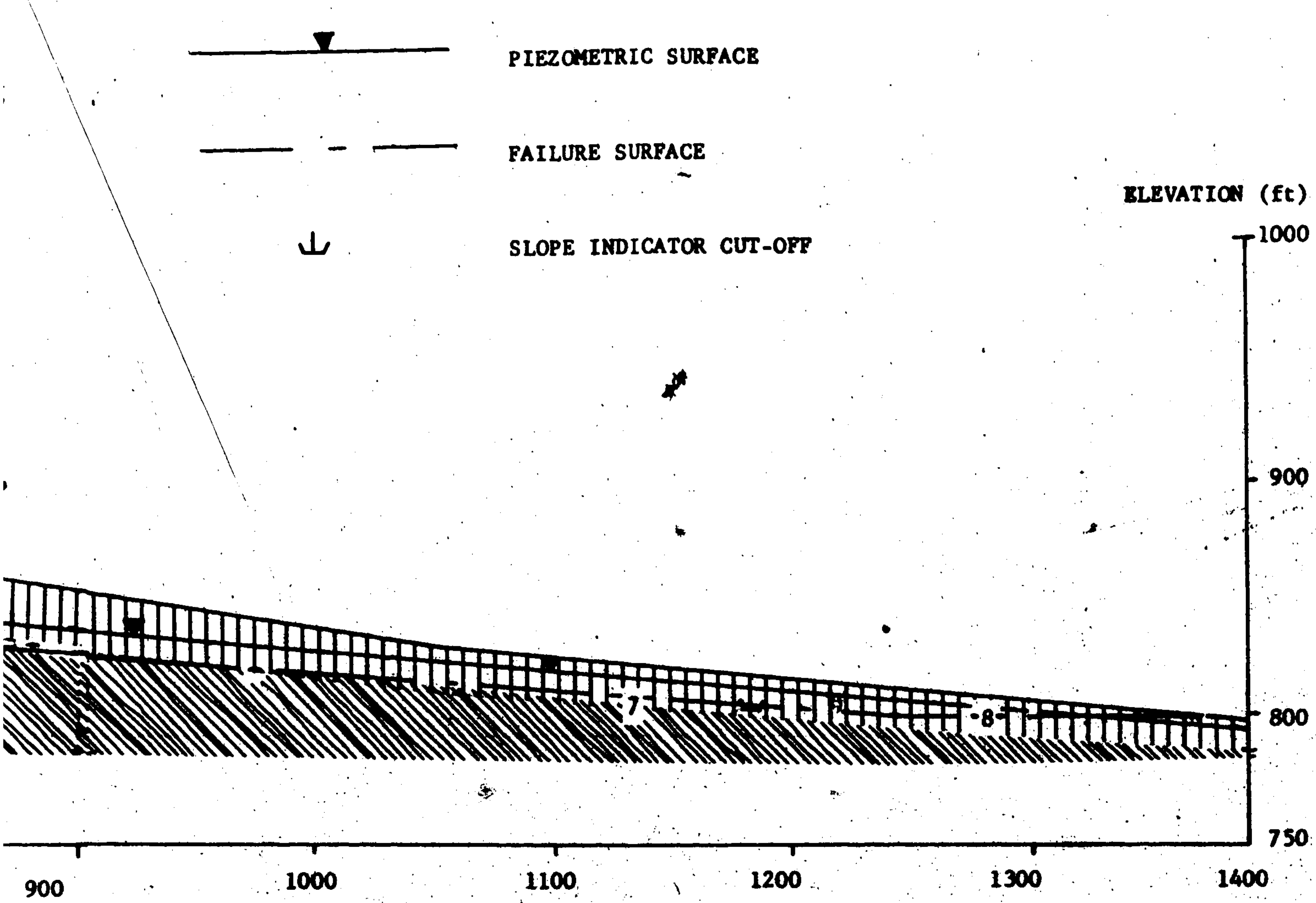


FIGURE 5.1 EDGERT



DGERTON SLIDE USING FLAT FAILU



LAT FAILURE SURFACE

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The object of this research was to investigate the recent landslide movements which occurred on the valley wall of the Battle River, northeast of Wainwright, Alberta, to propose a mechanism of failure and to support this by means of numerical stability analyses. The following conclusions are drawn in the light of the results and discussion presented in the previous chapters.

1. The recent slides are the continuation of a retrogressive series of slides which have resulted in the present day slopes. It is postulated that several phases of sliding have preceded the recent slides, however the landslide topography has been subdued by erosion which suggests a time span in the order of several centuries between successive phases of movement. The fact that the toe of the North Slide breaks out almost half way up the valley wall suggests that the failure planes have been controlled by bedding, with failure taking place along weak bentonitic clayshale layers.

2. Stability analyses performed on the incipient South Slide has shown that the use of a 'stepped' failure plane is inappropriate. The slope indicator data, coupled with the results of the stability analysis indicate that the soil

mass is sliding along an essentially planar failure surface, inclined at about 4.5° to the horizontal. It is estimated that the backscarp of the slide is inclined at about 70° to the horizontal, based on measurements obtained from the North Slide, and the portion of the south scarp visible, which at the time of writing was in the order of 7.5 feet.

3. Results of stability analyses, performed on the South Slide, suggest that residual strength parameters are being mobilised along the major portion of the horizontal part of the failure plane, with peak angles of shearing resistance, but essentially zero cohesion in the backscarp materials. Angles of shearing resistance on average between peak and residual are being mobilised along that part of the horizontal portion of the failure plane nearest the backscarp, which is suggested as being a 'first time' movement. The drop in O' from the peak value may be a result of previous creep movements, or alternatively, some other shearing process, such as valley rebound (Matheson, 1972).

4. The reason for failure is proposed to have been a result of a very gradual loss in the cohesion along the backscarp, due to the process of softening in the joints and fissures (Terzaghi, 1936; Skempton, 1970). The soil mass may have been critically stable for several years, but failure was eventually triggered by the increase in pore pressures induced in the slope, following the [illegible]ing cutoff in 1974. The North Slide, having failed catastrophically in 197[illegible] is now quasi-stable and no visible creep movements have been

observed. The South Slide is still creeping steadily, however failure has not yet progressed to the toe of the slide. It is expected that this slide will also fail catastrophically at some time in the near future.

5. It follows from the discussion that there is a time element involved for the softening process to take place. In the case of this landslide, and similarly for the Devon Slide, the subdued landslide topography indicates that a long period of time is required to effect the softening in the backscarp. In comparison many slides in embankment cuts in the London clay analysed by James (1970) occurred within 50 years. Obviously the time for this softening process depends on such factors as, the mass of soil, the nature and stress history of the soil, the availability of the softening agents, and also whether the slopes are natural or man made. It would be expected that embankment cuts would expedite the softening process.

6. This case demonstrates the importance of the piezometric pressure in explaining landslide activity especially in natural slopes. Although the piezometric levels recorded here were not abnormally high as also noted in the Devon Slide, any increase or decrease in the piezometric level has a pronounced effect on the stability of the slide. In the case of the Edgerton Landslide it is postulated that the pore pressure per se was not the prime reason for failure, rather it was the triggering mechanism, as noted previously. However it is of ultimate importance to record the piezometric pressures accurately, in order to

carry out realistic analyses.

7. In the light of this case history it is suggested that many slopes exist in meta-stable equilibrium for long periods of time similar to this slope, until the fortuitous combination of factors initiates movement. This concept has many implications for highway or similar construction associated with valley walls. Several other cases of similar slides have been reported, although in most of these cases, outside agencies have been responsible for triggering the failure. In the case of the Peace River Bridge collapse (Hardy, 1966) the bridge construction, shift in the river channel, and finally the excessive precipitation were responsible for reactivating failure. In the Devon Slide (loc.cit.) the meta-stable equilibrium was disrupted by minor charges to the slope for highway purposes.

8. Finally, it is suggested that the retrogressive failure mechanism described in this case history is a dominant process of valley widening in the Upper Cretaceous rocks of Western Canada. The Edgerton Landslide is typical of a large number of slides occurring in the Upper Cretaceous bedrock of Western Canada. The failure surfaces are generally wedge shaped consisting of a steeply dipping backscarp abruptly curving into an essentially horizontal lower portion. Illustrations of typical failure surfaces are shown in Figures 1.3 and 5.2. The lower portion of the slide surface is stratigraphically contolled and occurs roughly parallel to bedding planes in the bedrock. The rate and degree of softening of the soils in the backscarp area are unknown,

but in the limit the strength of these soils is reduced to the fully softened state. This state is represented by the peak angle of shearing resistance and zero cohesion. Therefore these parameters should be used for the steeply dipping portion of the failure surface.

In areas where geomorphic evidence indicates that old landsliding has occurred it is most probable that the new landslides will reinitiate movement along the old slip surfaces. Therefore it is recommended that residual strength parameters are appropriate along the lower flat lying part of the failure surface.

9. It has been recognosed, as mentioned previously, that pore pressures exert a pronounced influence on the stability of these slopes. Therefore any remedial measures which will reduce or inhibit the rise in piezometric surface will be of considerable benefit. This case history has indicated that there is an increase in pore pressure in spring as a result of spring runoff. Therefore positive surface drainage in the scarp area and for some distance back from the crest of the valley wall should be installed to inhibit the infiltration of surface water. This may also influence the rate of softening of the materials in the backscarp.

3.2 Recommendations

In view of the fact that the toe of the South Slide has not yet cropped out, it is suggested that slide movements be observed occasionally in the future. Accurate profiles were obtained for the South Slide in May, 1975 and 1976, and these should be compared with successive surveys in the future to trace the movement of these stakes. It is suggested that piezometric readings also be taken throughout the summer and again next spring following spring thaw.

It is also recommended that the North Slide be monitored in the future to follow any creep movements, although no appreciable movements have been detected up to this time.

An integral part of the mode of failure of this slide is the softening process. However the mechanism and time rates of the softening process are imperfectly known. It has been suggested that this phenomena is a function of such factors as, soil type, stress history, fracture permeability, soil mass and the availability of rainwater. It is recommended that a further study of the softening process and its effects on the strength parameters of the soils of Western Canada be undertaken.

LIST OF REFERENCES

LIST OF REFERENCES

American Society for Testing Materials, 1958. "Procedures for Testing Materials.

Bayrock, L. A., 1967. "Surficial Geology of the Wainwright Area (East Half), Alberta." Research Council of Alberta, Report 67-4, Edmonton, Alta. 10pp.

Bishop, A. W., 1955. "The Use of the Slip Circle in the Stability Analysis of Slopes." Geotechnique, 5, pp. 7-17.

Bishop, A. W., 1967. "Progressive Failure with Special Reference to the Mechanism Causing it." Proc. Geotechnical Conf., Oslo, 2, pp. 142-150.

Bishop, A. W. and Henkel, D. J., 1953. "Pore Pressure Changes during Shear in Two Undisturbed Clays." Proc. 3rd Int. Conf. Soil Mech. and Found. Eng., Zuerich, 1, pp. 94-99.

Bishop, A. W. and Lovenbury, H. T., 1969. "Creep Characteristics of Two Undisturbed Clays." Proc. 7th Int. Conf. Soil Mech. and Found. Eng., Mexico, 1, pp. 29-37.

Bishop, A. W., Green, G. E., Garga, V. K., Andresen, A. and Brown, J. D., 1971. "A New Ring Shear Apparatus." Geotechnique, 21, pp. 273-328.

Bjerrum, L., 1967. "Progressive Failure in Slopes of Overconsolidated Plastic Clays and Clayshales." (3rd Terzaghi Lecture) J. ASCE, 93, SM5, pp. 3-49. And: NGI Publ., 77, pp. 1-29.

Brooker, E. W., 1958. "Special Problems in Stability of Slopes." Unpub. M.Sc. Thesis, Univ. of Alberta. 126pp.

Cassel, F. L., 1948. "Slips in Fissured Clay." Proc. 2nd Int. Conf. Soil Mech. and Found. Eng., Rotterdam, 2, pp. 46-49.

Chattopadhyay, P.K., 1972. "Residual Shear Strength of Some Pure Clay Minerals." Unpub. Ph.D. Thesis, Dept. of Civil Eng., Univ. of Alberta. 340pp

Cullen, R. M. and Donald, I. B., 1971. "Residual Strength Determination in Direct Shear." Proc. 1st Australian - New Zealand Conf. on Geomech., Melbourne, Australia, pp. 1-11.

De Beer, E., 1969. "Experimental Data Concerning Clay Slopes." Proc. 7th Int. Conf. Soil Mech. and Found. Eng., Mexico, U 2, pp. 517-525.

Eigenbrod, K. D. and Morgenstern, N. R., 1971. "A Slide in Cretaceous Bedrock, Devon, Alberta." In Geotechnical Practice for Stability in Open Pit Mining, Ed. by C. O. Brawner and V. Milligan, AIME, pp. 223-238.

Esu, F., 1967. "Influence of Weathering on Behaviour of Stiff Clays; Structure of Clays; with Special Reference to Experience with Italian Clays." Proc. Geotech. Conf., Oslo, 2, pp. 154-158.

Gould, J., 1960. "A Study of Shear Failure in Certain Tertiary Marine Sediments." Research Conf. on Shear Strength of Cohesive Soils, ASCE, University of Colorado, Boulder, Colo., pp. 615-641.

Hardy, R. M., 1966. "Failure of the Peace River Highway Bridge." Proc. 4th Am. Eng. Geol. and Soils. Eng. Symp., Univ. of Idaho, Moscow, Idaho, pp. 49-59.

Hardy, R. M., Brooker, E. W. and Curtis, W. E., 1962. "Landslides in Overconsolidated Clays." The Engineering Journal, 45, 6, pp. 81-89.

Hayley, D. W., 1968. "Progressive Failure of a Clayshale Slope in Northern Alberta." Unpub. M.Sc. Thesis, Dept. of Civil Eng., Univ. of Alberta. 60pp.

Henkel, D. J., 1957. "Investigation of Two Long-term Failures in London Clay Slopes at Wood Green and Northolt." Proc. 4th Int. Conf. Soil Mech. and Found. Eng., London, 2, pp. 315-320.

James, P. M., 1970. "Time Affects and Progressive Failure in Clay Slopes." Ph.D. Thesis, Univ. of London. 210pp.

James, P. M., 1971. "The Role of Progressive Failure in Clay Slopes." Proc. 1st Australia - New Zealand Conf. on Geomech., Melbourne, 1, pp. 344-348.

Kenney, T. C., 1967. "The Influence of Mineral Composition on the Residual Strength of Natural Soils." Proc. Geotech. Conf., Oslo, 1, pp. 123-129. And: 1968, Oslo, NGI. Publ., 76.

Laguros, J. G., Kumar, S. and Annamalai, M., 1974. "A Comparative Study of Simulated and Natural Weathering of Some Oklahoma Shales." Clays and Clay Minerals, 22, pp. 111-115.

Leussink, H. and Mueller-Kirchenbauer, H., 1967. "Determination of the Shear Strength Behavior of Sliding Planes Caused by Geological Features." Proc. Geo. Conf., Oslo, 1, pp. 131-137.

Locker, J. G., 1969. "The Petrographic and Engineering Properties of Fine Grained Sedimentary Rocks of Central Alberta." Unpub. Ph.D. Thesis, Dept. of Civil Eng., Univ. of Alberta, 285pp.

Matheson, D. S., 1972. "Geotechnical Implications of Valley Rebound." Unpub. Ph.D. Thesis, Dept. of Civ. Eng., Univ. of Alberta, 424 pp.

Morgenstern, N. R., 1967. "Shear Strength of Stiff Clay." Proc. Geotech. Conf., Oslo, General Report, Session 2, pp. 59-71.

Morgenstern, N. R. and Price V. E., 1965. "The Analysis of the Stability of General Slip Surfaces." Geotechnique, 15, pp. 79-93.

Morgenstern, N. R. and Price, V. E., 1967. A Numerical Method for Solving the Equations of Stability of General Slip Surfaces." The Computer Journal, 9, 4, pp. 388-393.

Painter, W. T., 1965. "An Investigation of the Lesueur Landslide, Edmonton, Alberta. Unpub. M.Sc. Thesis, Dept. of Civil Eng., Univ. of Alberta, 101 pp.

Patton, F., 1966. "Multiple Modes of Shear Failure in Rock and Related Materials." Unpub. Ph.D. Thesis, Univ. of Illinois, Urbana.

Peck, R. B., 1967. "Stability of Natural Slopes." J. ASCE, 93, SM4 pp. 403-418.

Pennell, D. G., 1969. "Resisdual Strength Analysis of Five Landslides." Ph.D. Thesis, Dept. of Civil Eng., Univ. of Alberta, 166 pp.

Petley, D. J., 1966. "The Shear Strength of Soils at Large Strains." Unpub. Ph.D. Thesis, London Univ.

P. F. R. A., 1970. "Interim Study of Residual Direct Shear Test Results on Bearpaw Shales. Soil Mech. and Matls. Testing Div., Saskatoon, Saskatchewan.

Rennie, R. J., 1966. "Residual Strength of a Clay Till Applied to Little Smoky River Landslide." Unpub. M.Sc. Thesis, Dept. of Civil Eng., Univ. of Alberta, 162 pp.

Rutherford, R. L., 1928. "Geology of the Area between North Saskatchewan and McLeod Rivers, Alberta." Scientific Industrial Research Council, Report No. 19.

Sinclair, S. R. and Brooker, E. W., 1967. "The Shear Strength of Edmonton Shale." Proc. Geotech. Conference, Oslo, 1, pp. 295-299.

Skempton, A. W., 1948. The Rate of Softening of Stiff Fissured Clays, with Special Reference to London Clay." Proc. 2nd Int. Conf. Soil Mech. and Found. Eng., 2, pp. 50-53.

Skempton, A. W., 1964. "Long Term Stability of Clay Slopes." Geotechnique, 14, 2, pp. 77-102.

Skempton, A. W., 1966. "Some Observations on Tectonic Shear Zones." Proc. 1st Int. Conf. Rock Mech., Lisbon, 1, pp. 329-335.

Skempton, A. W., 1970. "First Time Slides in Overconsolidated Clays." Geotechnique, 20, pp. 320-324.

Skempton, A. W., and Hutchinson, J. 1969. "Stability of Natural Slopes and Embankment Foundations." State of the Art Volume, 7th Int. Conf. Soil Mech. and Found. Eng., Mexico, pp. 291-340.

Skempton, A. W. and Petley, D. J., 1967. "The Strength along Structural Discontinuities in Stiff Clays." Proc. Geotech. Conf., Oslo, 2, pp. 29-46.

Tchalenko, J. S., 1967. "The Influence of Shear and Consolidation on the Microscopic Structure of Some Clays." Unpub. Ph.D. Thesis, Univ. of London.

Terzaghi, K., 1936. "Stability of Slopes in Natural Clays." Proc. 1st Int. Conf. Soil Mech. and Found. Eng., Cambridge, Mass., 1, pp. 161-165.

Terzaghi, K. and Peck, R. B., 1967. "Soil Mechanics in Engineering Practice." Wiley, New York, 729 pp.

Thomson, S., 1970. "Riverbank Stability Study at the University of Alberta, Edmonton. Can. Geo. J., 7, 2, pp. 157-168.

Thomson, S., 1971. "Analysis of a Failed Slope." Canadian Geotech. Journ., 8, 4, pp. 596-599.

Thomson, S., 1974. "A Reconnaissance Report on the Edgerton Landslide." Unpub. Internal Report, Dept. of Civil Eng., Univ. of Alberta.

Thomson, S., 1976. Personal Communications.

Thomson, S. and Morgenstern, N. R., 1974. "Landslide Characteristics in Argillaceous Bedrock, Alberta, Canada." Internal Report, Dept. of Civil Eng., Univ. of Alberta.

Warren, P. S., and Hume, G. S. 1939. "Ribstone Creek, Alberta." Geol. Surv. Can. Map 501 A.

Williams, G. D. and Burke, C. F., 1964. "Upper Cretaceous." In Geological History of Western Canada. Ed. R. G. McCrossan and R. P. Glaister. Alberta Soc. of Pet. Geol., Calgary, pp. 169-189.

Wilson, S. D., 1962. "The Use of Slope Measuring Devices to Determine Movement in Earth Masses." ASTM STP No. 322, Field Testing of Soils.

Wilson, S. D. and Hancock, C. W., 1959. "Horizontal Displacement of Clay Foundations." Proc. 1st Pan-American Conf. Soil Mech., Mexico City, 1, pp. 41-57.

Wyatt, F. A., Newton, J. D., Bowser, W. E. and Odynsky W., 1944. "Soil Survey of the Wainwright and Vermillion Sheets." Univ of Alberta, College of Agriculture Bull. 42, 127 pp.

Yudhbir, 1969. "Engineering Behavior of Heavily Overconsolidated Clays and Clayshales with Special Reference to Long-term Stability." Ph.D. Thesis, Connell Univ., Ithaca, N. York.

APPENDIX A

RAINFALL RECORDS FOR PARADISE
VALLEY

PRECIPITATION RECORDS.

PARADISE VALLEY 53°07'. 110°21' 2200 FT A.S.L.

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1961							1.94	0.56	0.54	0.99	1.23	0.48
1962	0.97	1.35	1.22	0.80	1.39	3.91	4.13	0.62	0.83	0.59	1.37	0.88
1963	1.04	1.12	1.19	1.21	1.68	4.36	2.96	2.31	0.50	0.69	0.70	0.59
1964	1.40	0.59	0.46	1.07	2.51	1.02	1.75	2.04	3.81	0.90	1.02	0.68
1965	1.25	1.17	0.47	3.06	6.17	1.74	3.10	0.73	0.04	0.76	0.50	0.47
1966	1.32	0.37	0.49	0.57	0.46	0.71	3.36	2.63	0.46	0.08	1.36	0.63
1967	0.84	0.71	0.88	0.83	0.97	2.09	1.38	1.78	0.07	1.15	1.33	1.21
1968	0.89	0.53	0.82	1.38	0.83	2.20	3.97	3.88	3.91	1.09	0.20	1.10
1969	1.63	0.55	0.18	1.24	1.40	0.53	4.82	1.10	2.62	0.69	0.60	1.17
1970	0.49	0.53	1.46	0.22	1.07	6.26	4.98	0.37	0.51	1.94	1.13	0.94
1971	1.52	0.39	0.95	0.33	1.11	3.21	4.92	2.32	0.42	0.58	0.87	1.47
1972	0.80	0.93	1.37	1.18	0.69	1.30	2.31	1.91	1.96	0.32	1.00	0.68
1973	0.54	0.75	0.91	1.66	0.64	4.64	3.28	4.66	1.07	0.94	1.76	0.79
1974	1.68	0.48	1.02	0.94	1.32	4.64	2.94	1.03	1.02	0.21	0.01	0.59
1975	0.68	0.56	1.67	2.12	1.46	3.81	2.14	2.97				
1976												

APPENDIX B

BOREHOLE LOGS

SUMMARY OF SAMPLING & LABORATORY TESTS

RESEARCH COUNCIL OF ALBERTA
EDMONTON — ALBERTA

PROJECT: EDGERTON LANDSLIDE (BOREHOLE I)

B. Tweedie

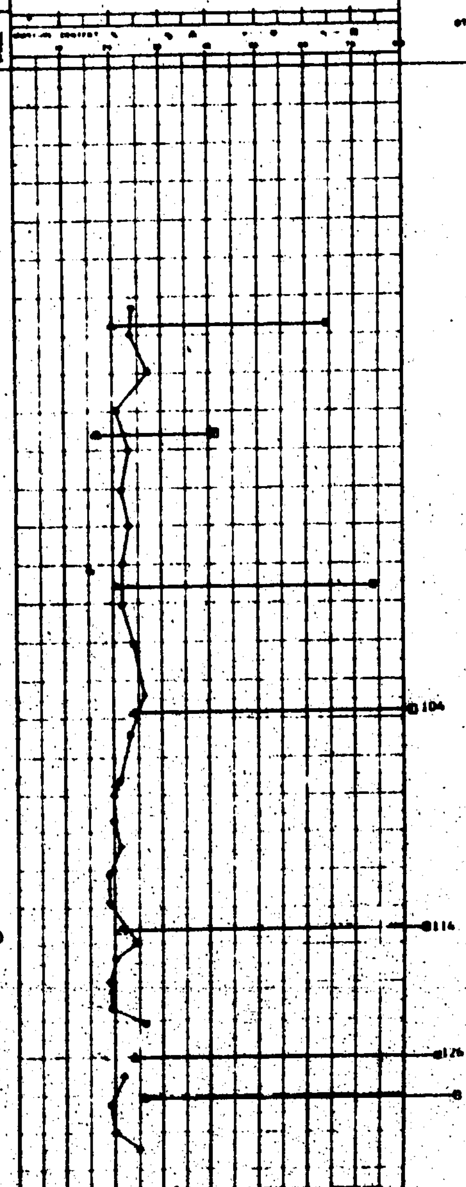
PLATE

SOIL PROFILE — SOIL DESCRIPTION

GROUND ELEV. (989.0R above datum)

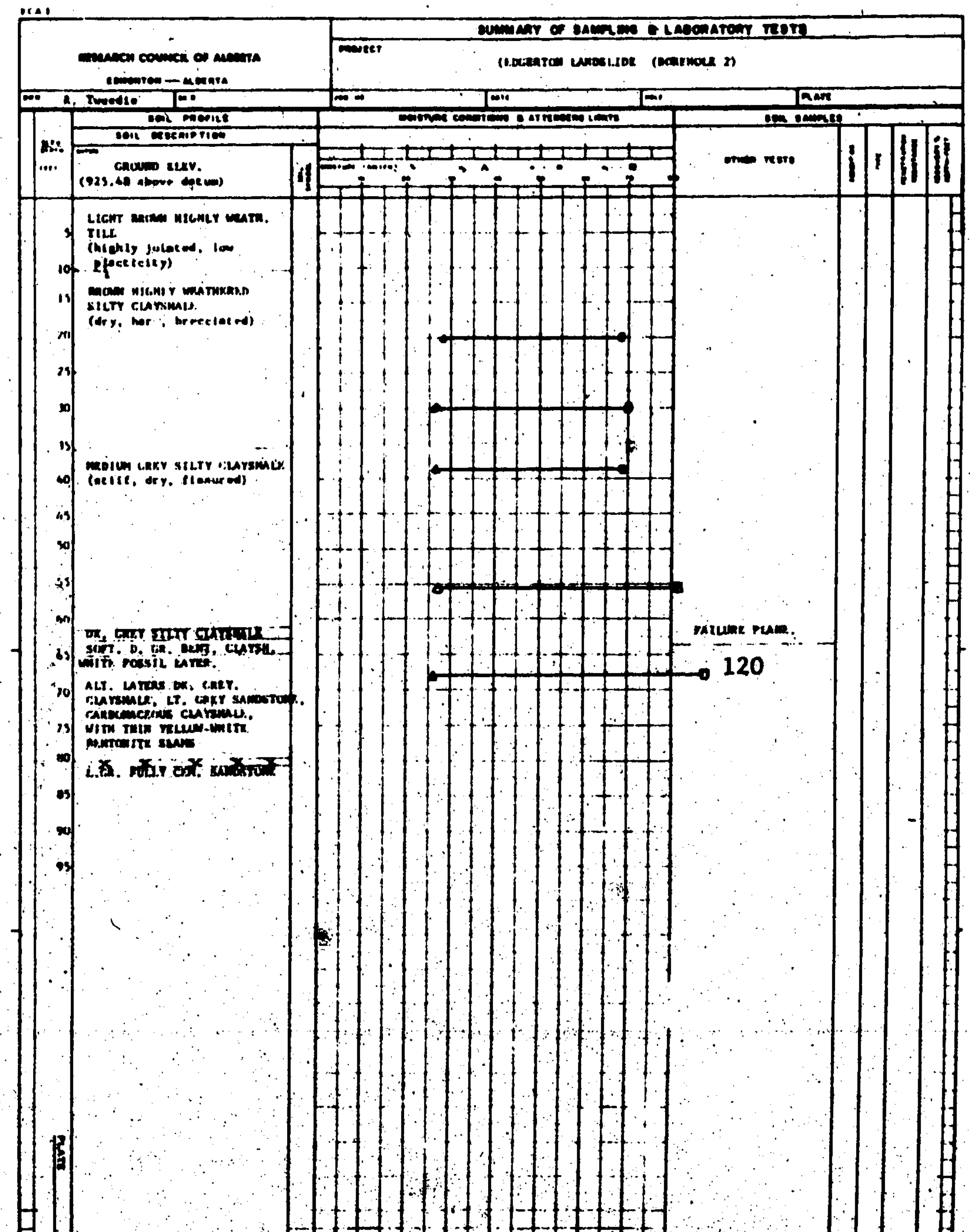
DEPTH (feet)	SOIL DESCRIPTION
5, 10	BROWN HIGHLY WEATHERED TILL (columnar jointed)
15, 20, 25, 30, 35	BROWN HIGHLY WEATHERED SILTY CLAYSHALE (dry, hard, brecciated, with selenite seams up to 6mm thick)
40, 45	DARK GREY FISSURED SILTY CLAYSHALE (ironstoned)
50, 55	DARK GREY FISSURED SANDY CLAYSHALE (poorly cemented)
60, 65, 70	DARK GREY MED. GRAINED FISSURED SILTY CLAYSHALE (dry, hard, soil gets finer grained with depth)
75, 80, 85, 90, 95, 100, 105, 110	DARK GREY FISSURED SILTY CLAYSHALE (very dry, very stiff, Soil gets stiffer and finer grained with depth) (some clay lenses)
115	DARK GREY CLAYSHALE WITH SANDY SEAMS (v. dry v. stiff)
120	DARK GREY POORLY CEMENTED SANDY CLAYSHALE (dry, stiff)
125	DK. GREY CLAYSHALE (yellow-white fossil layers)
130	SOFT BENT. CLAYSHALE LAYER
	POORLY CEM. LT. GR. SANDSTONE
	DARK GREY CLAYSHALE WITH SANDY SEAMS, CARBONACEOUS CLAYSHALE AND THIN YELLOW BENTONITE LAYERS

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES

OTHER TESTS


SUMMARY OF SAMPLING & LABORATORY TESTS
RESEARCH COUNCIL OF ALBERTA
EDMONTON — ALBERTA
PROJECT
(EDGERTON LANDSLIDE (BOREHOLE 2)
R. Tweedie
PLATE
SOIL PROFILE
SOIL DESCRIPTION
GROUND ELEV.
(925.48 above datum)
MOISTURE CONDITIONS & ATTERBERG LIMITS
SOIL SAMPLES
OTHER TESTS
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
LIGHT BROWN HIGHLY WEATH. TILL
(highly jointed, low plasticity)
BROWN HIGHLY WEATHERED SILTY CLAYSHALE
(dry, har , brecciated)
MEDIUM GREY SILTY CLAYSHALE
(stiff, dry, fissured)
DK. GREY SILTY CLAYSHALE
SOFT. D. GR. BENT. CLAYSH.
WHITE FOSSIL LAYER.
ALT. LAYERS DK. GREY. CLAYSHALE, LT. GREY SANDSTONE, CARBONACEOUS CLAYSHALE, WITH THIN YELLOW-WHITE BENTONITE SEAMS
L.GR. FULLY CEM. SANDSTONE
FAILURE PLANE
120
PLATE

RESEARCH COUNCIL OF ALBERTA
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: EDGERTON LANDSLIDE (BOREHOLE 3)

K. Tweedie

SOIL PROFILE — SOIL DESCRIPTION | MOISTURE CONDITIONS & ATTERBERG LIMITS | SOIL SAMPLES — OTHER TESTS

GROUND ELEV. (911.90 above datum)

Depth (ft.)	Soil Description
5	LIGHT BROWN HIGHLY WEATHERED TILL (highly jointed)
10	
15	LIGHT BROWN HIGHLY WEATH. SILTY CLAYSHALE (hard, brecciated)
20	
25	
30	DK. GREY CEM. CLAYSHALE
35	LIGHT GREY SILTY CLAYSHALE (stiff, fissured)
40	(clay promp present)
45	(getting softer)
50	
55	
60	MED. GR. SANDY CLAYSHALE (poorly cemented)
65	ALT. LAYERS DARK GREY CLAYSHALE, LT.GR. SANDSTONE, GR. CARB. CLAYSHALE LAYERS WITH THIN YELLOW-WHITE BENTONITE SEAMS
70	
75	
80	
85	HARD, CEMENTED SANDSTONE
90	

PLATE

RESEARCH COUNCIL OF ALBERTA
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: EDGERTON LANDSLIDE (BOREHOLE 4)

R. Tweedie

PLATE

SOIL PROFILE — SOIL DESCRIPTION | MOISTURE CONDITIONS & ATTERBERG LIMITS | SOIL SAMPLES — OTHER TESTS

GROUND ELEV. (801.26 above datum)

Depth	Soil description
	LIGHT BROWN HIGHLY WEATHERED TILL (dry, stiff, brecciated)
5	
10	
	LIGHT BROWN HIGHLY WEATHERED SILTY CLAYSHALE
15	(dry, stiff, highly brecciated)
20	
25	
30	HIGHLY WEATHERED SILTY CLAYSHALE, WITH SOME GREY NUGGETS
35	LIGHT GREY SILTY CLAYSHALE (stiff, highly brecciated)
40	SOFT MED. GREY BENTONITIC CLAYSHALE STIFF CLAYSH. AND SILTSTONE WITH YELLOW-WH. BENTONITE.
45	LAYER SOFT GREY CLAYSHALE WITH WHITE FOSSIL SPECKS
50	ALT. LAYERS DARK GREY FISSURED CLAYSHALE, LT. GREY SANDSTONE, DARK CARB. CLAYSHALE, WITH THIN YELLOW BENTONITE SEAMS
55	(v. stiff, v. dry)
60	
65	

143

FAILURE PLANE

102

116

PLATE

RESEARCH COUNCIL OF ALBERTA

EDMONTON - ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: EDGERTON LANDSLIDE (BOREHOLE 5)

R./ Tweedie | PLATE

SOIL PROFILE — SOIL DESCRIPTION | MOISTURE CONDITIONS & ATTERBERG LIMITS | SOIL SAMPLES — OTHER TESTS

GROUND ELEV. (997.7(above datum)

Depth	Soil description
5	
10	BROWN HIGHLY WEATHERED TILL
15	(columnar jointed)
20	
25	BROWN HIGHLY WEATHERED SILTY CLAYSHALE (dry, hard, brecciated.)
30	
35	
40	
45	DARK GREY FISSURED SILTY CLAYSHALE (ironstained)
50	WELL CEM. SANDSTONE LAYER
55	
60	DARK GREY FISSURED SANDY CLAYSHALE
65	
70	DARK GREY FISSURED MED. GRAINED SILTY CLAYSHALE
75	(dry, hard, soil gets finer grained with depth)
80	
85	DARK GREY FISSURED SILTY CLAYSHALE (very dry, very stiff)
90	WELL CEM'D. GREY SANDSTONE
95	
100	(soil gets stiffer and finer grained with depth).
105	
110	
115	(No sand particles visible)
120	
125	
130	DARK GREY CLAYSHALE WITH LT. GREY FINE SANDY SEAMS (2 l.g. soft clayeh. zones) (yellow-white fossil layer)
	FULLY CEM. SANDSTONE LAYER
	DARK GREY CLAYSHALE.

PLATE

R.C.A.S

SUMMARY OF SAMPLING & LABORATORY TESTS

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

PROJECT: EDGERTON LANDSLIDE (BOREHOLE 4)

BY: R. Tweedie | LOC: | JOB NO: | DATE: | HOLE: | PLATE:

SOIL PROFILE — MOISTURE CONDITIONS & ATTERBERG LIMITS — SOIL SAMPLED (OTHER TESTS)

DEPTH FEET	SOIL DESCRIPTION
	GROUND ELEV. (997.25 above datum)
5	
10	BROWN HIGHLY WEATHERED TILL (columnar jointed)
15	
20	
25	BROWN HIGHLY WEATHERED SILTY CLAYSHALE (dry, hard, brecciated)
30	
35	
40	
45	DARK GREY FISSURED SILTY CLAYSHALE (ironstained)
50	DARK GREY FISSURED SANDY CLAYSHALE
55	
60	
65	WELL CEM. SANDSTONE LAYER X X X X X X X
70	DARK GREY FISSURED MED. GRAINED SILTY CLAY SHALE (dry, hard, soil gets finer grained with depth)
75	
80	
85	
90	DARK GREY FISSURED SILTY CLAYSHALE (very dry, very stiff)
95	
100	(soil gets stiffer and finer grained with depth)
105	
110	
115	
120	MED. GREY POORLY CEMENTED SANDY CLAYSHALE (dry, stiff)
125	WELL CEM. SANDSTONE
130	ALT. LAYERS DARK GREY CLAYSHALE, SANDSTONE, CARB. CLAYSHALE (THIN YELLOW-WHITE BENT. MATERIAL)
	FULLY CEM. L. BR. SANDSTONE X X X X X
	DARK GREY CLAYSHALE

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: EDGERTON LANDSLIDE (BOREHOLE 7)

PLATE

SOIL PROFILE

SOIL DESCRIPTION

DEPTH	SOIL DESCRIPTION	OTHER TESTS
	GROUND ELEV. (856.59 above datum)	
5	LIGHT BROWN HIGHLY WEATHERED TILL (coal chips, pebbles, brecciated, low plasticity)	
10	LIGHT BROWN WEATHERED SILTY CLAYSHALE (med. plasticity)	
15	(traces of grey silty clayshale	
20	SOFT LIGHT GREY SILTY CLAYSHALE (Brown weathered silty clayshale nuggets present)	FAILURE PLANE.
25	LIGHT GREY SILTY CLAYSHALE (stiff, low plasticity	
30	GREY SILTY CLAYSHALE WITH SAND LENSES PRESENT (stiff, dry) (lenses of dry lt. yellow bentonite)	
35	(traces of yellow-grey)	
40		
45		

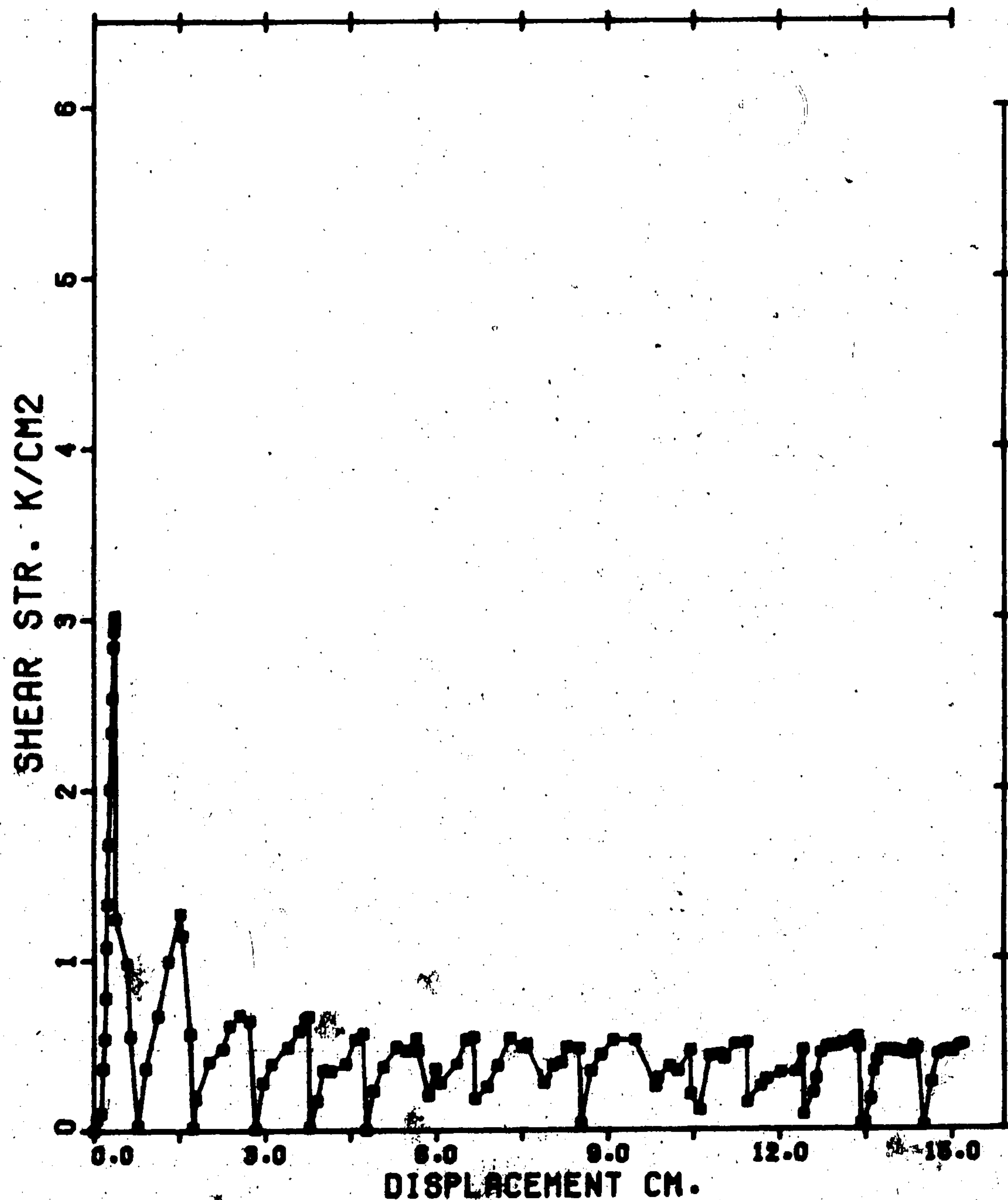
MOISTURE CONDITIONS & ATTERBERG LIMITS

SOIL SAMPLES

PLATE

APPENDIX C

DIRECT SHEAR TEST RESULTS



SAMPLE A1 UNWEATHERED SILTY CLAYSHALE

FIGURE C1 DIRECT SHEAR TEST ON UNWEATHERED SILTY CLAYSHALE

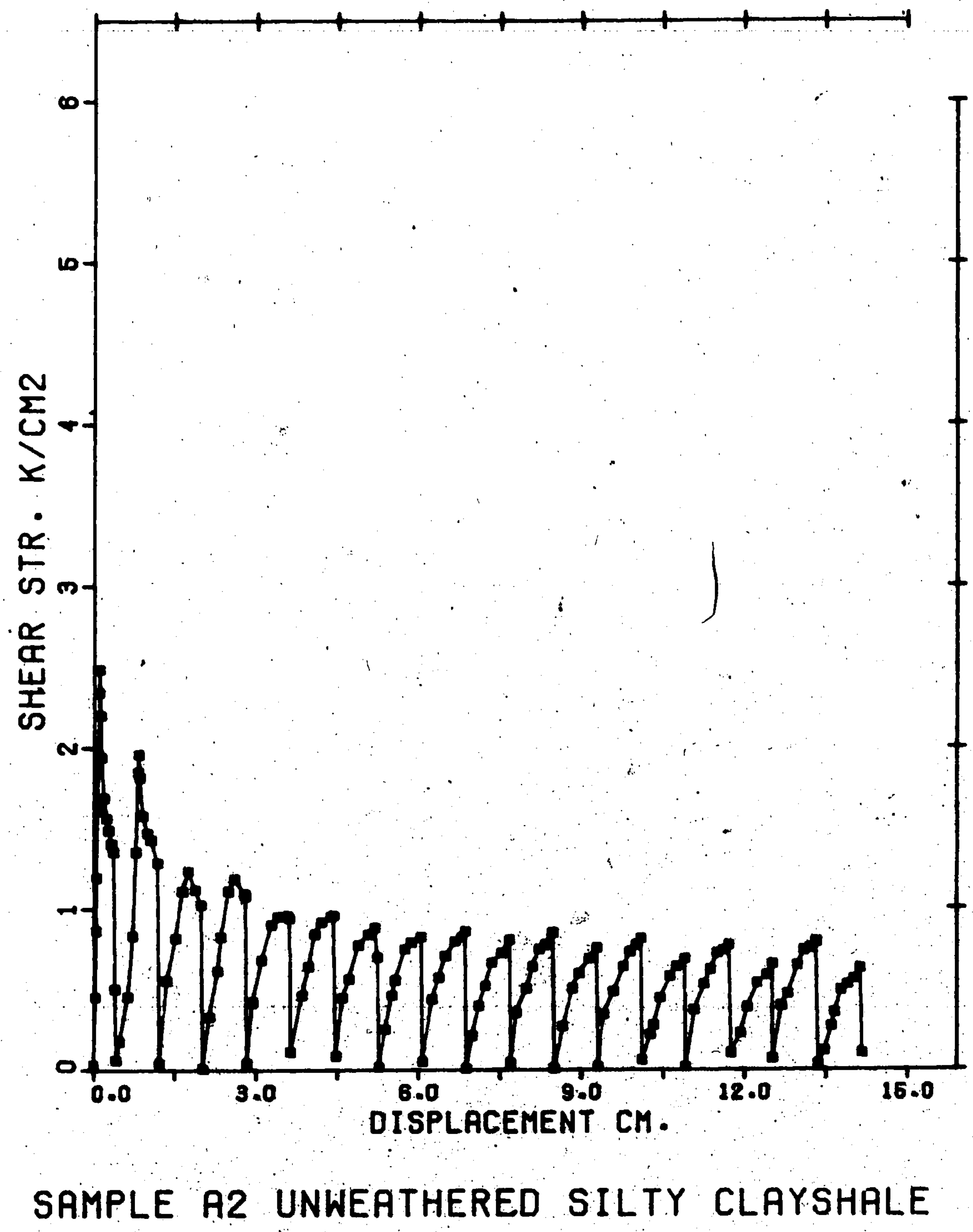


FIGURE C2 DIRECT SHEAR TEST ON UNWEATHERED SILTY CLAYSHALE

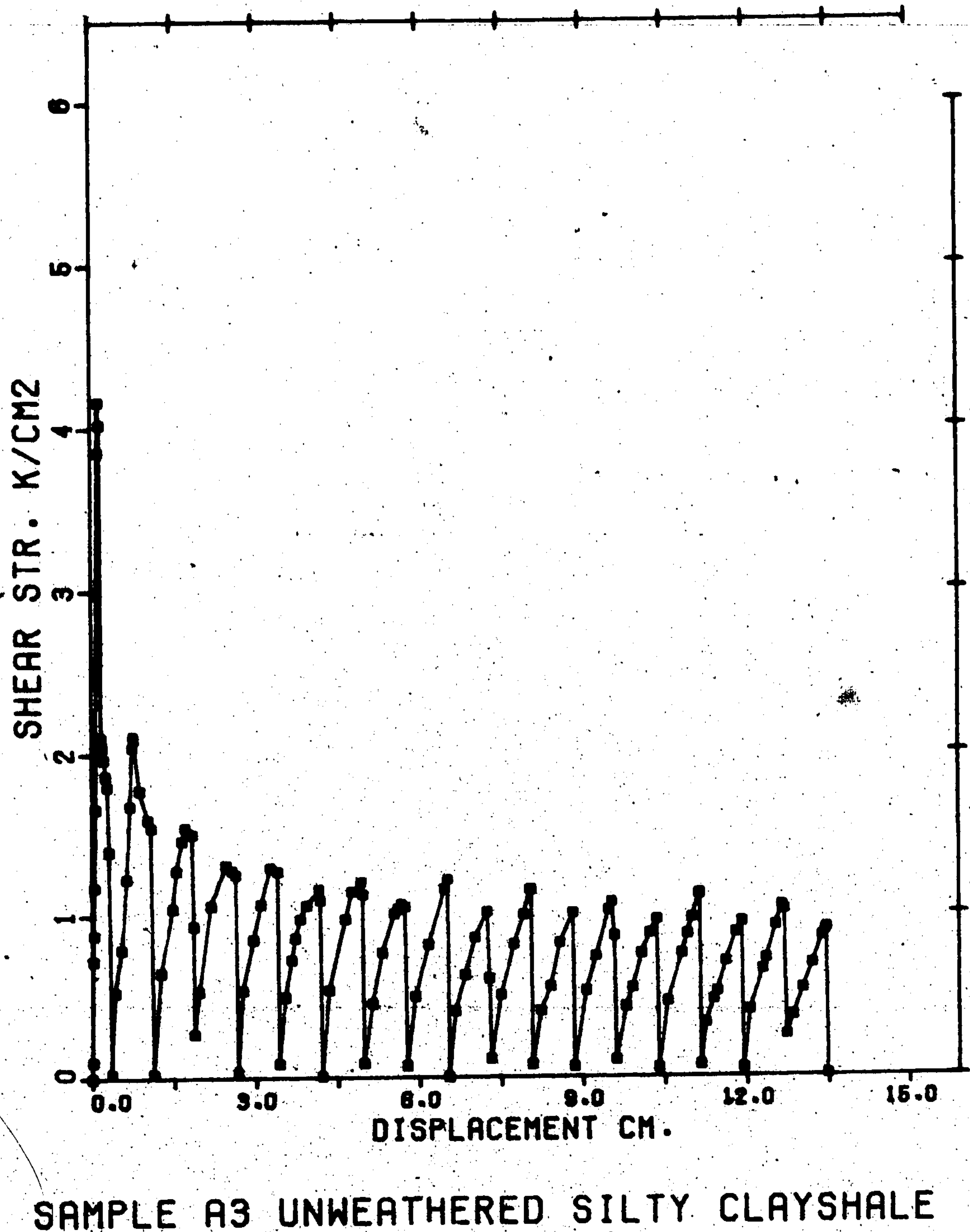


FIGURE C3 DIRECT SHEAR TEST ON UNWEATHERED SILTY CLAYSHALE

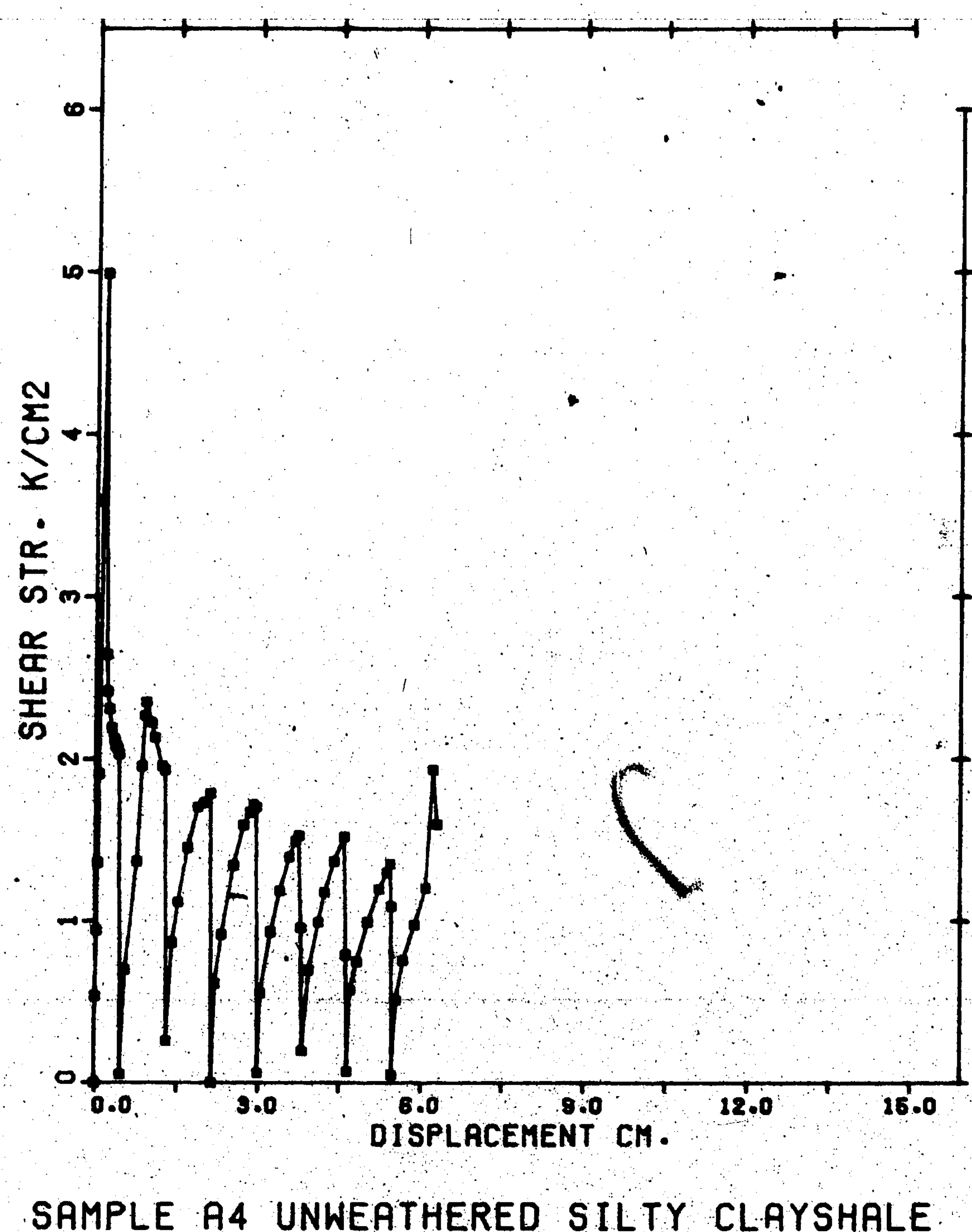
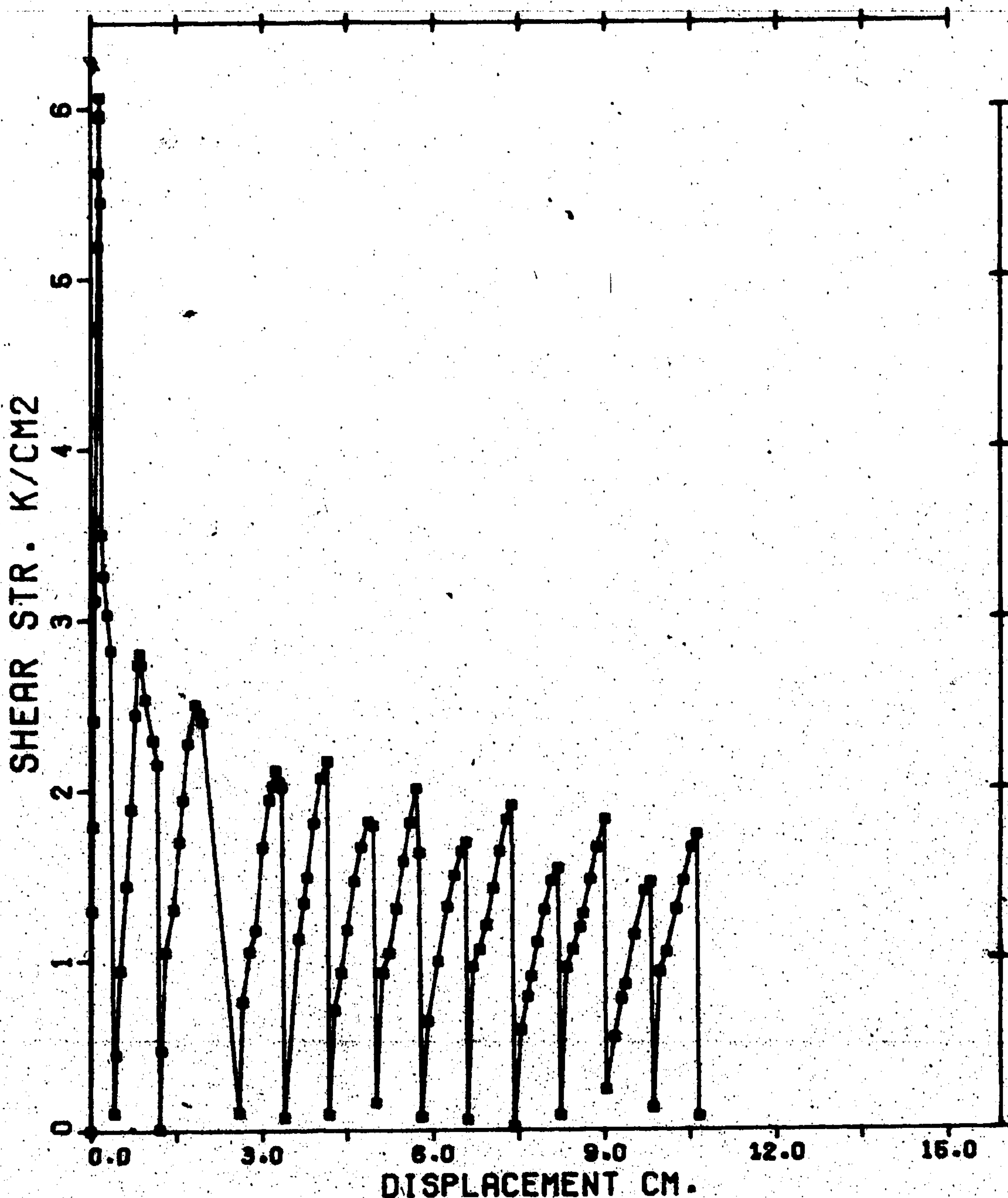


FIGURE C4 DIRECT SHEAR TEST ON UNWEATHERED SILTY CLAYSHALE



SAMPLE A5 UNWEATHERED SILTY CLAYSHALE

FIGURE C5 DIRECT SHEAR TEST ON UNWEATHERED SILTY CLAYSHALE

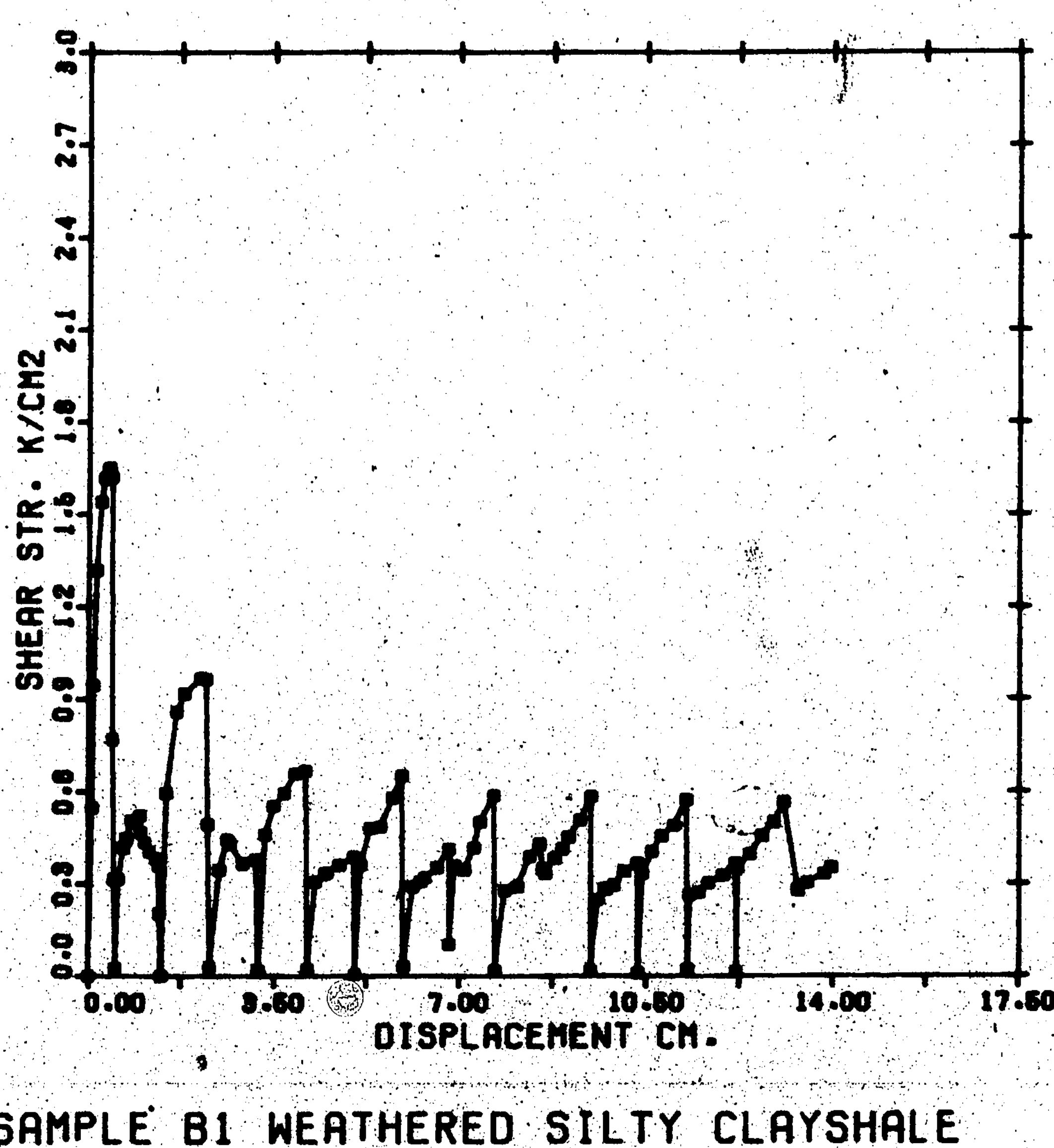


FIGURE C6 DIRECT SHEAR TEST ON WEATHERED SILTY CLAYSHALE

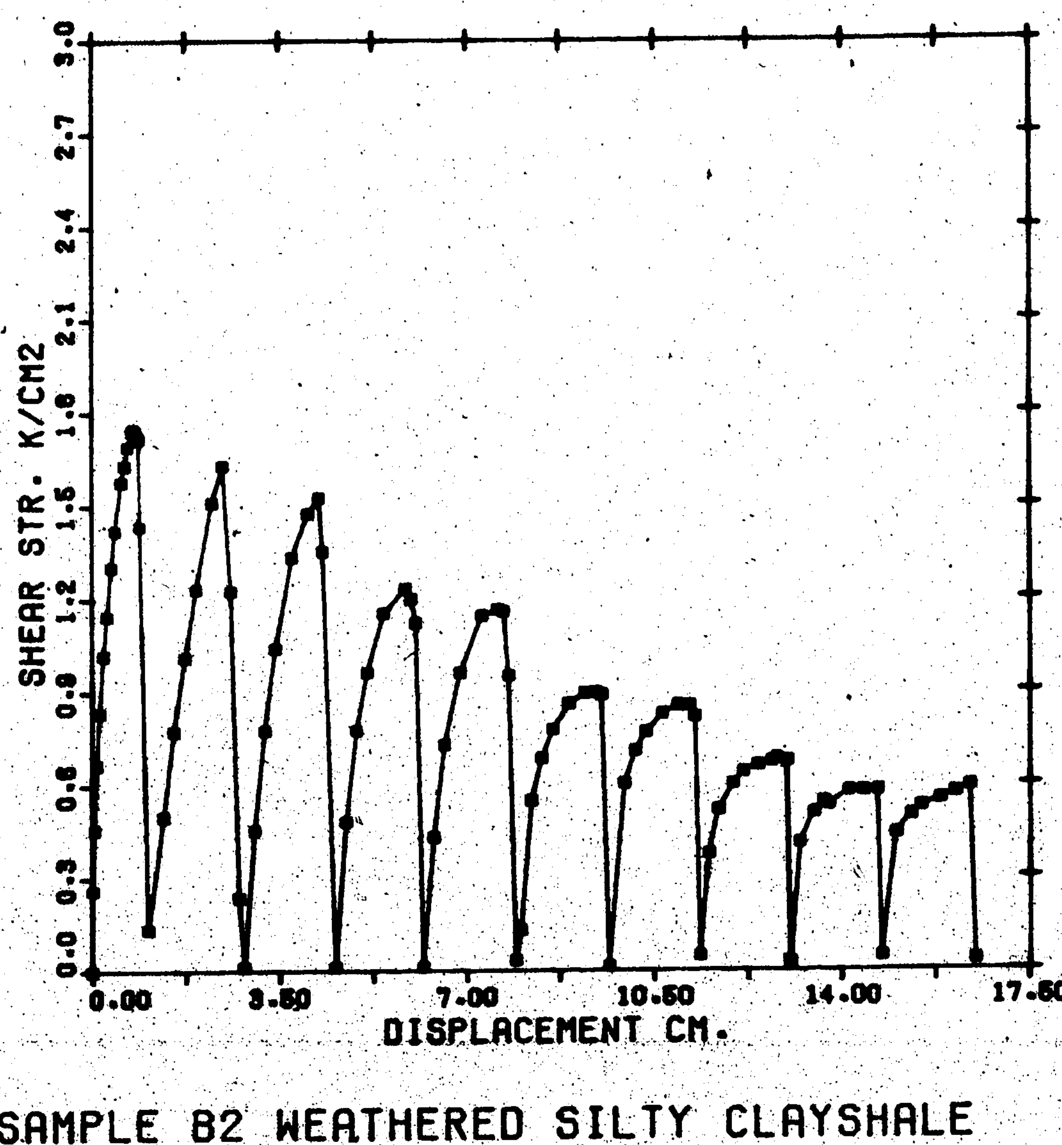


FIGURE C7 DIRECT SHEAR TEST ON WEATHERED SILTY CLAYSHALE

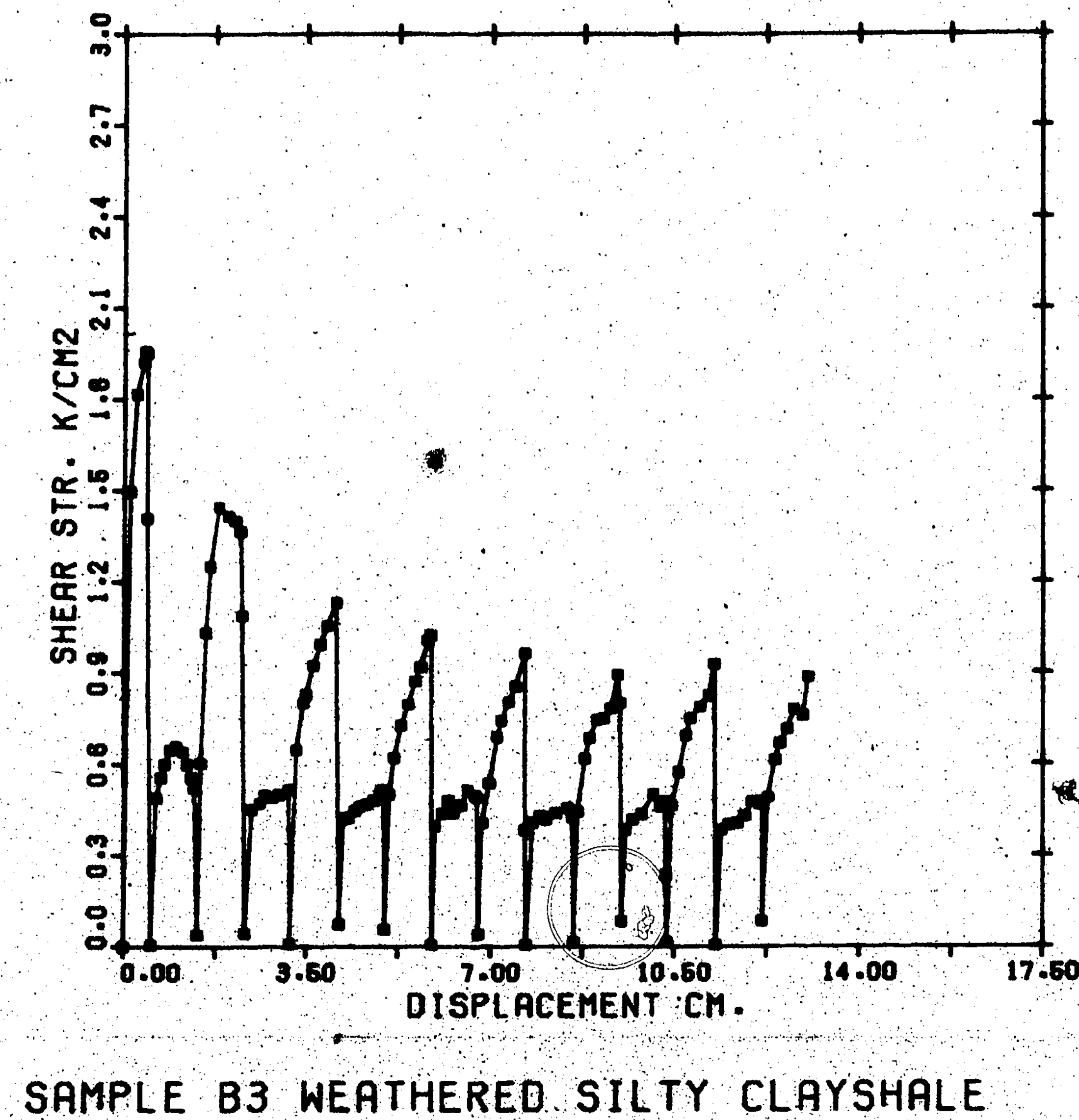


FIGURE C8 DIRECT SHEAR TEST ON WEATHERED SILTY CLAYSHALE

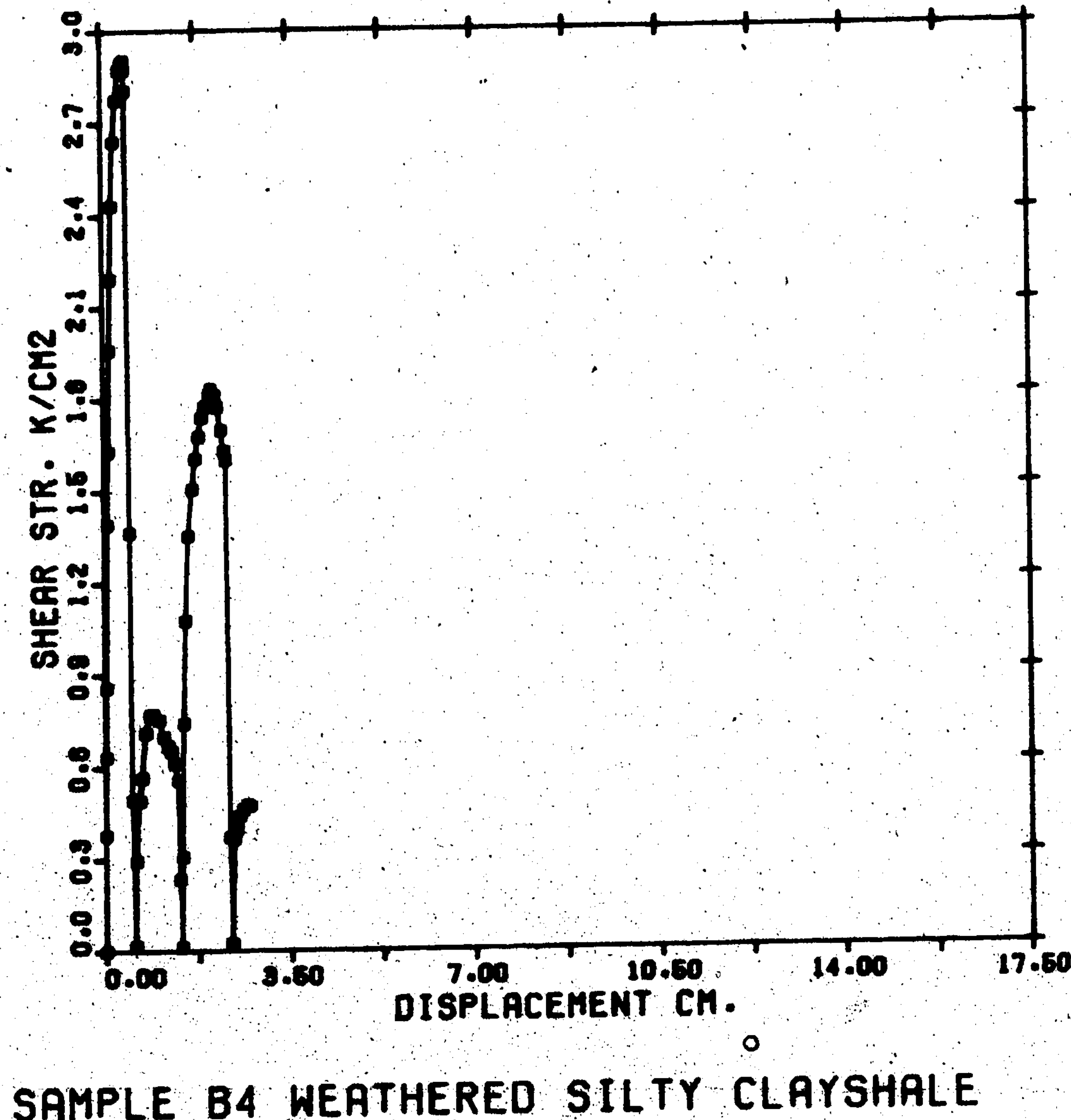


FIGURE C9 DIRECT SHEAR TEST ON WEATHERED SILTY CLAYSHALE

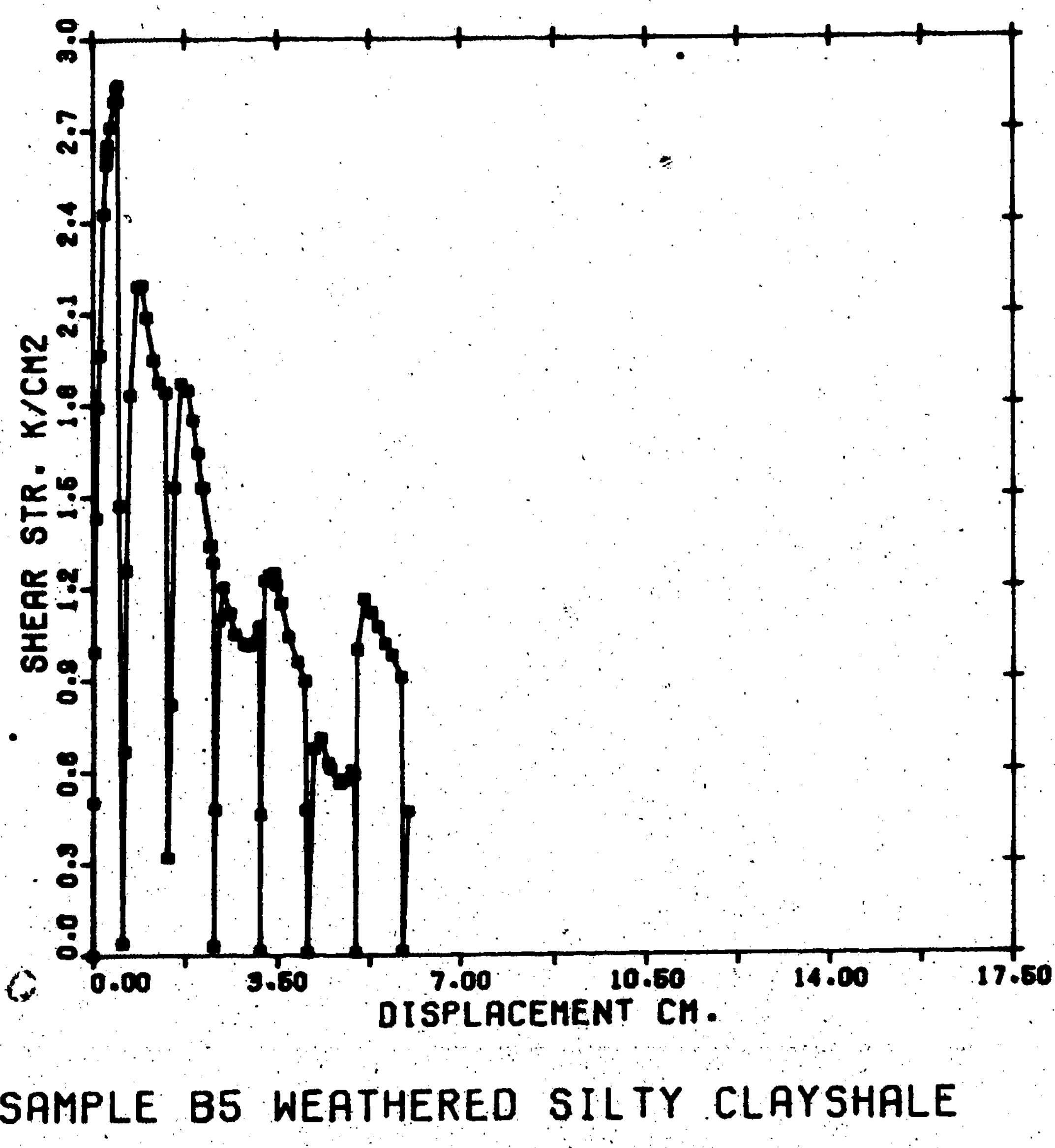


FIGURE C10 DIRECT SHEAR TEST ON WEATHERED SILTY CLAYSHALE

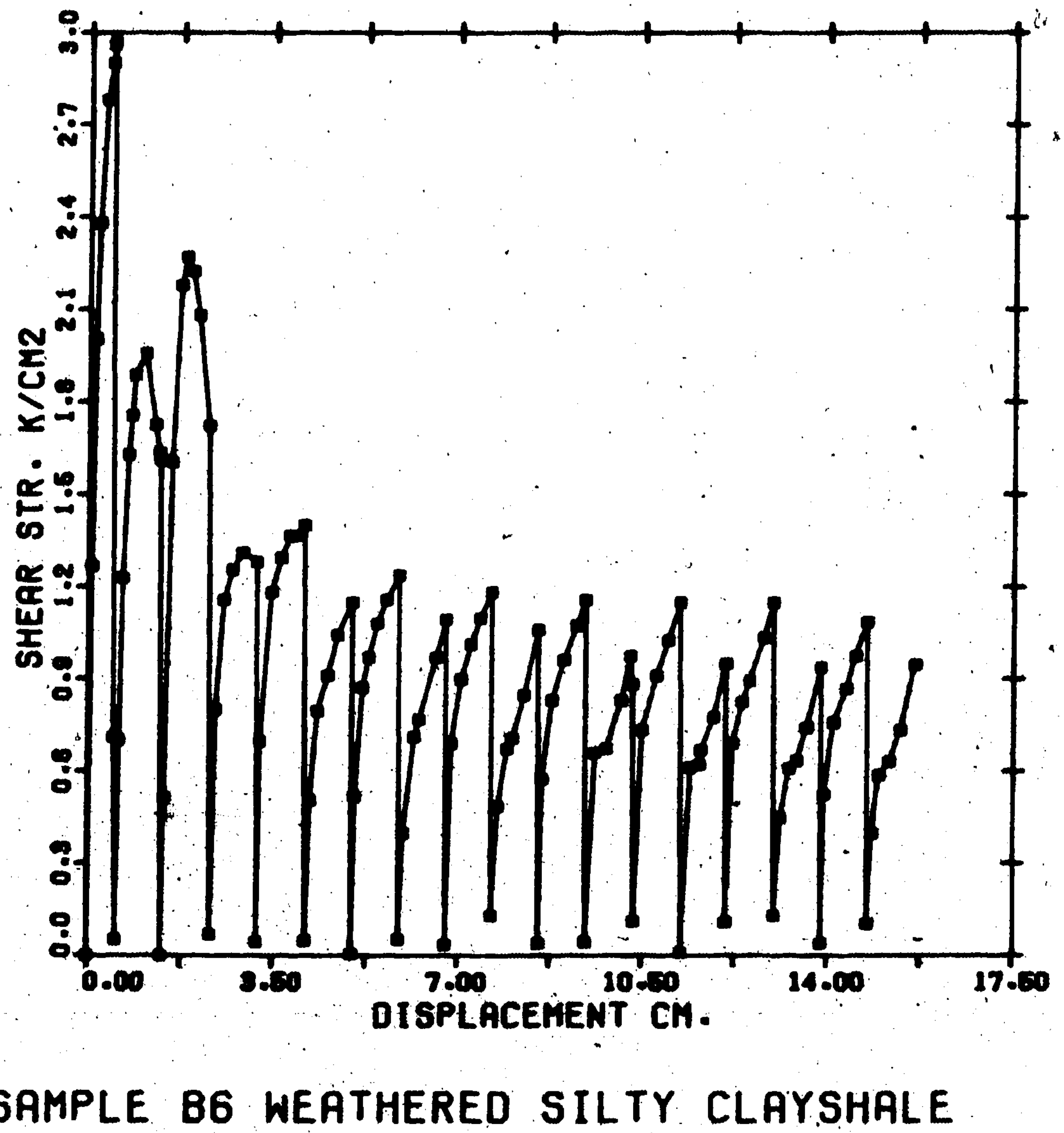


FIGURE C11 DIRECT SHEAR TEST ON WEATHERED SILTY CLAYSHALE

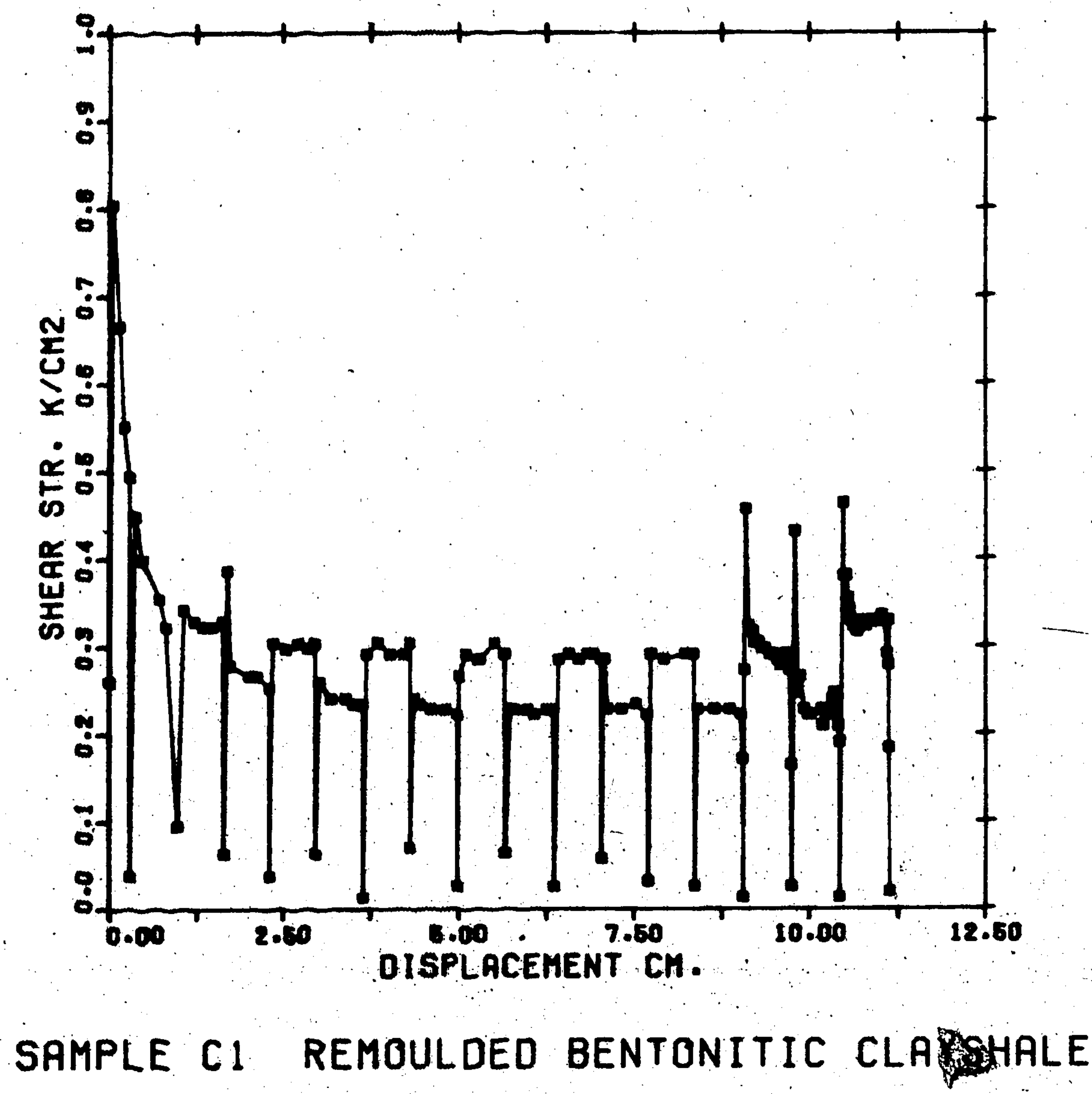


FIGURE C12 DIRECT SHEAR TEST ON REMOULDED BENTONITIC CLAYSHALE

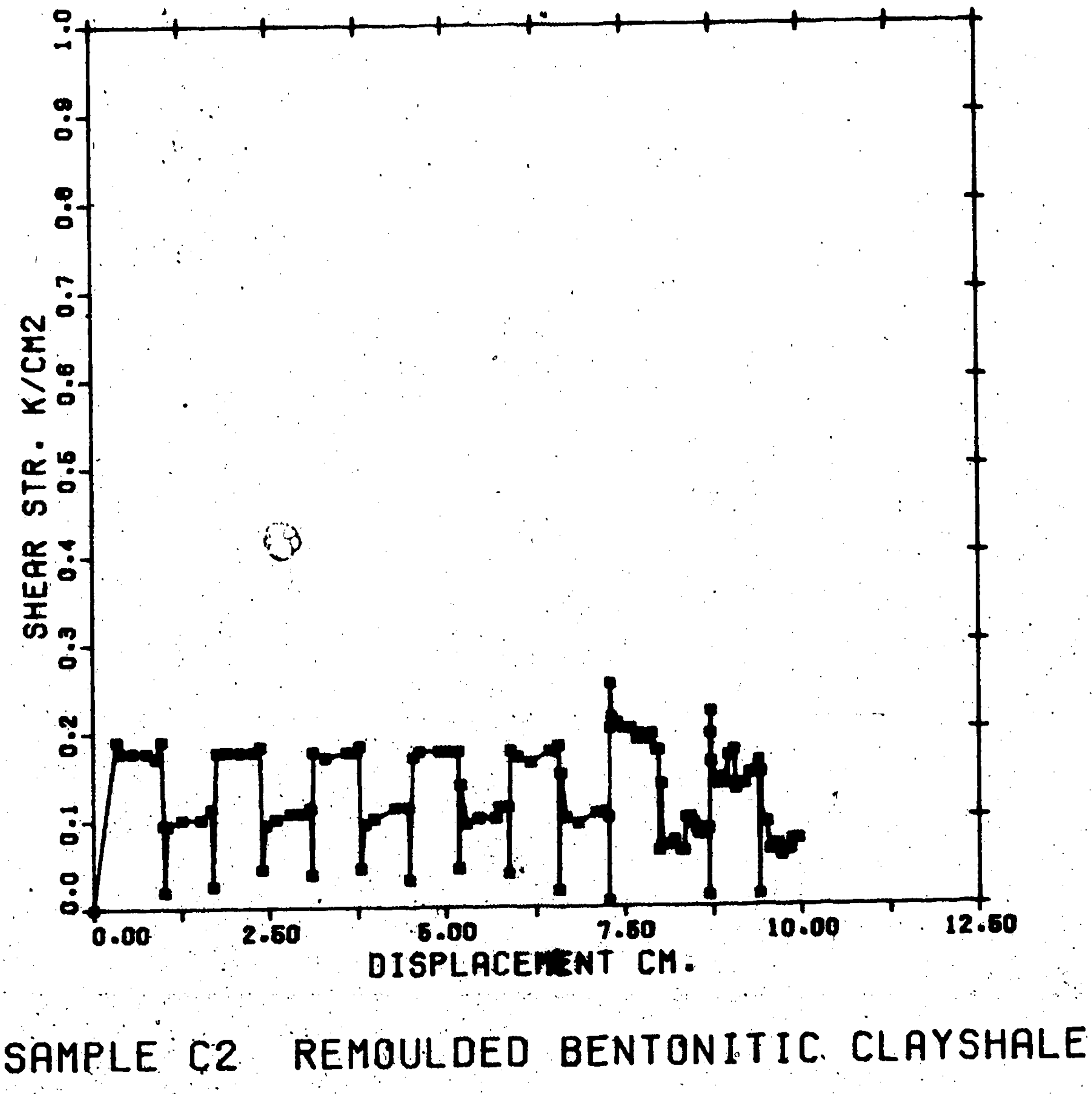


FIGURE C13 DIRECT SHEAR TEST ON REMOULDED BENTONITIC CLAYSHALE

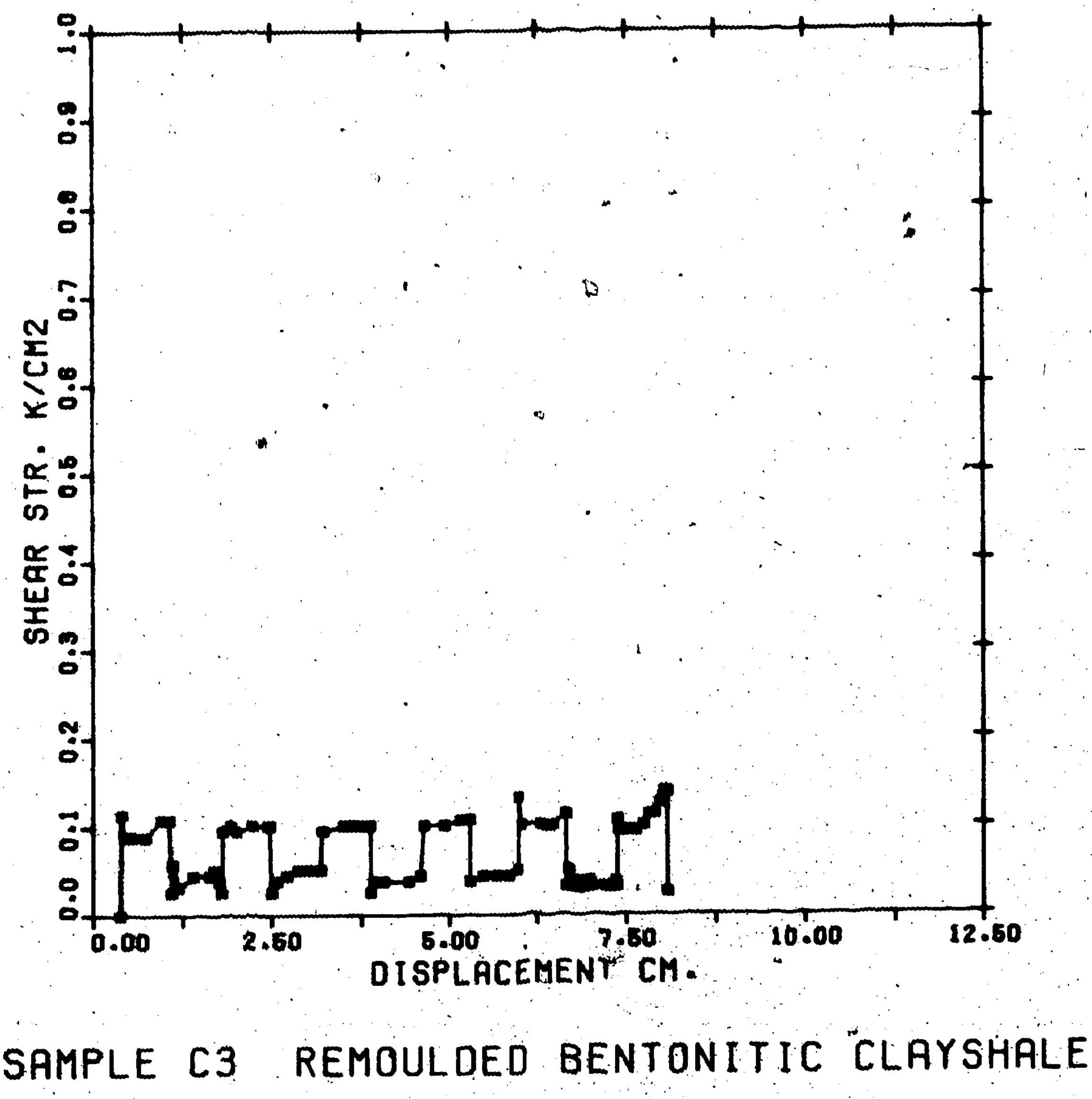


FIGURE C14 DIRECT SHEAR TEST ON REMOULDED BENTONITIC CLAYSHALE.

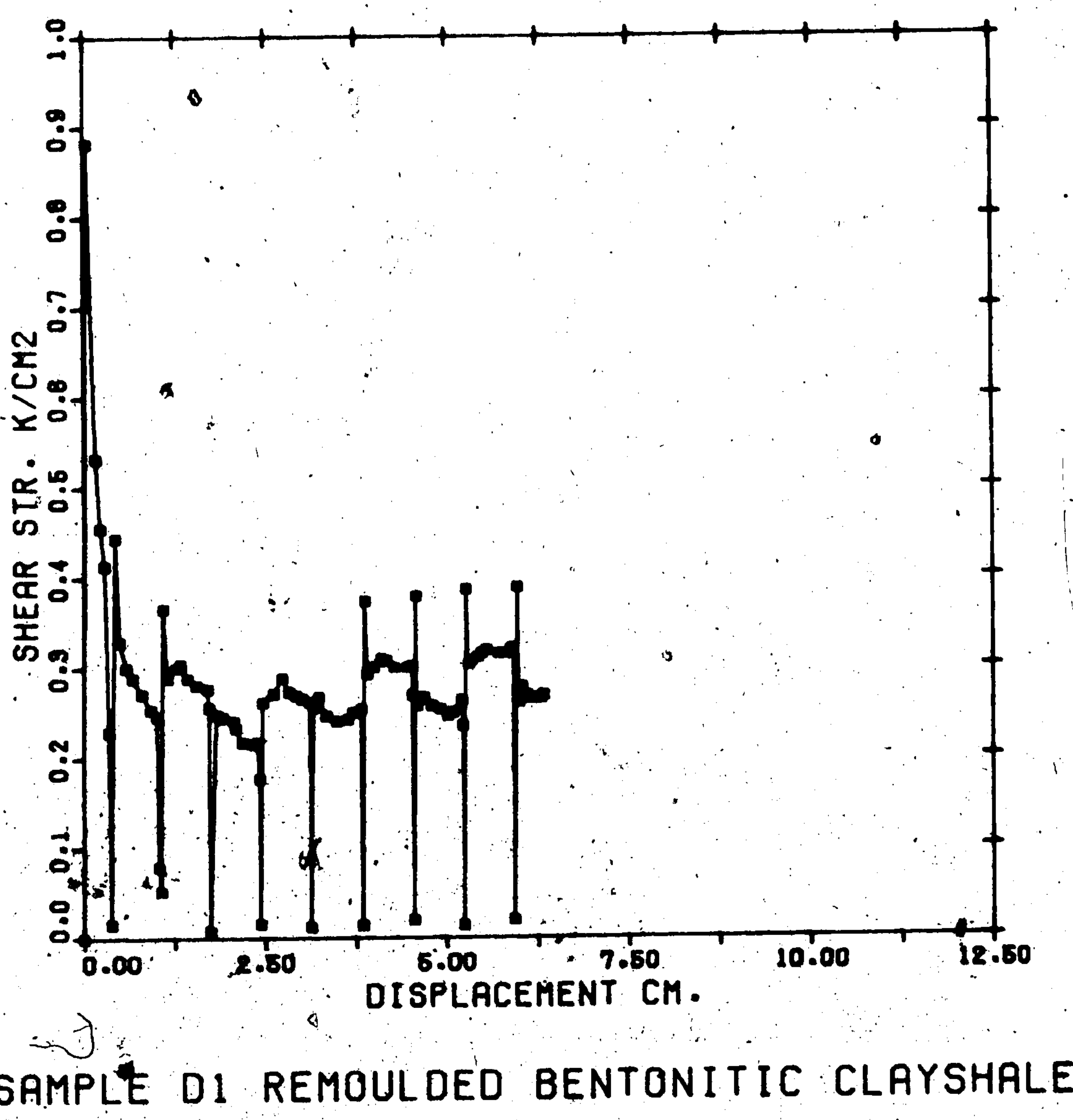
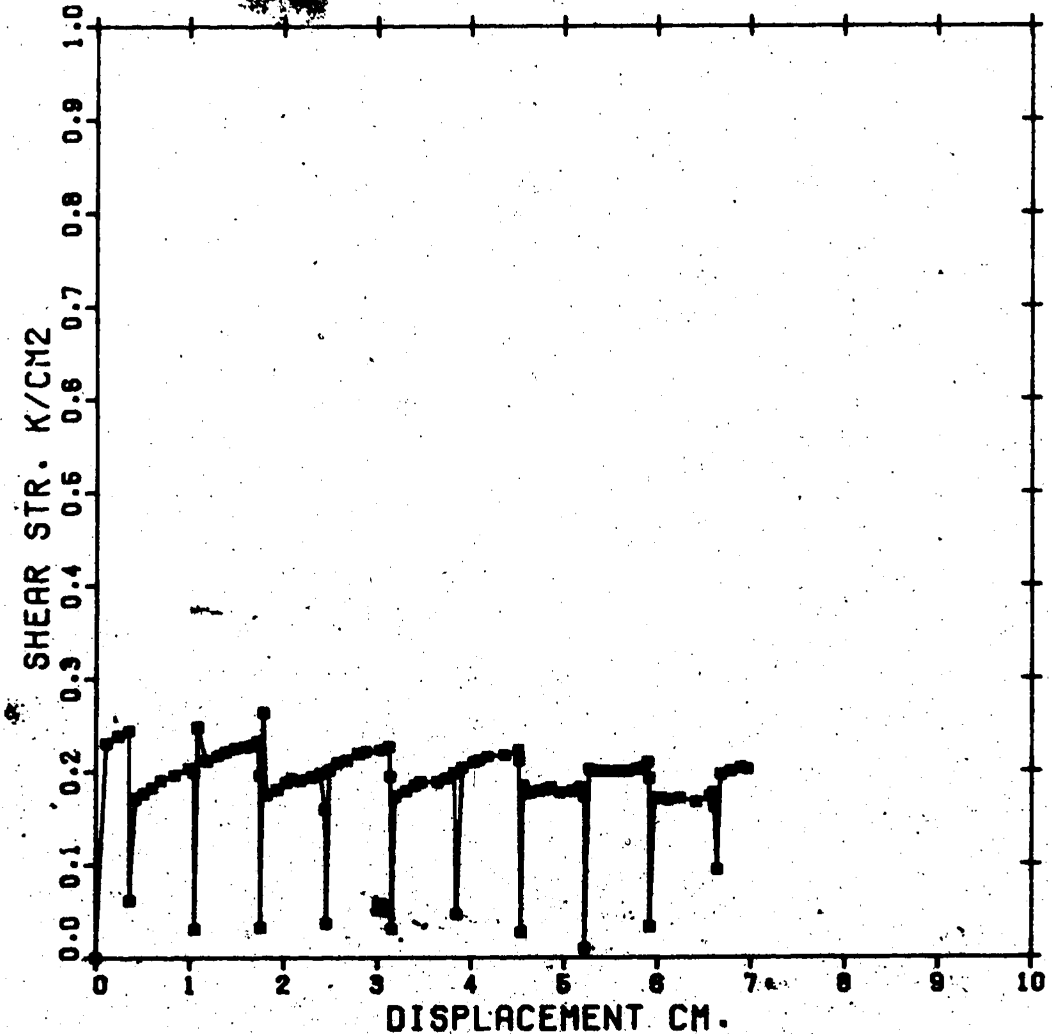
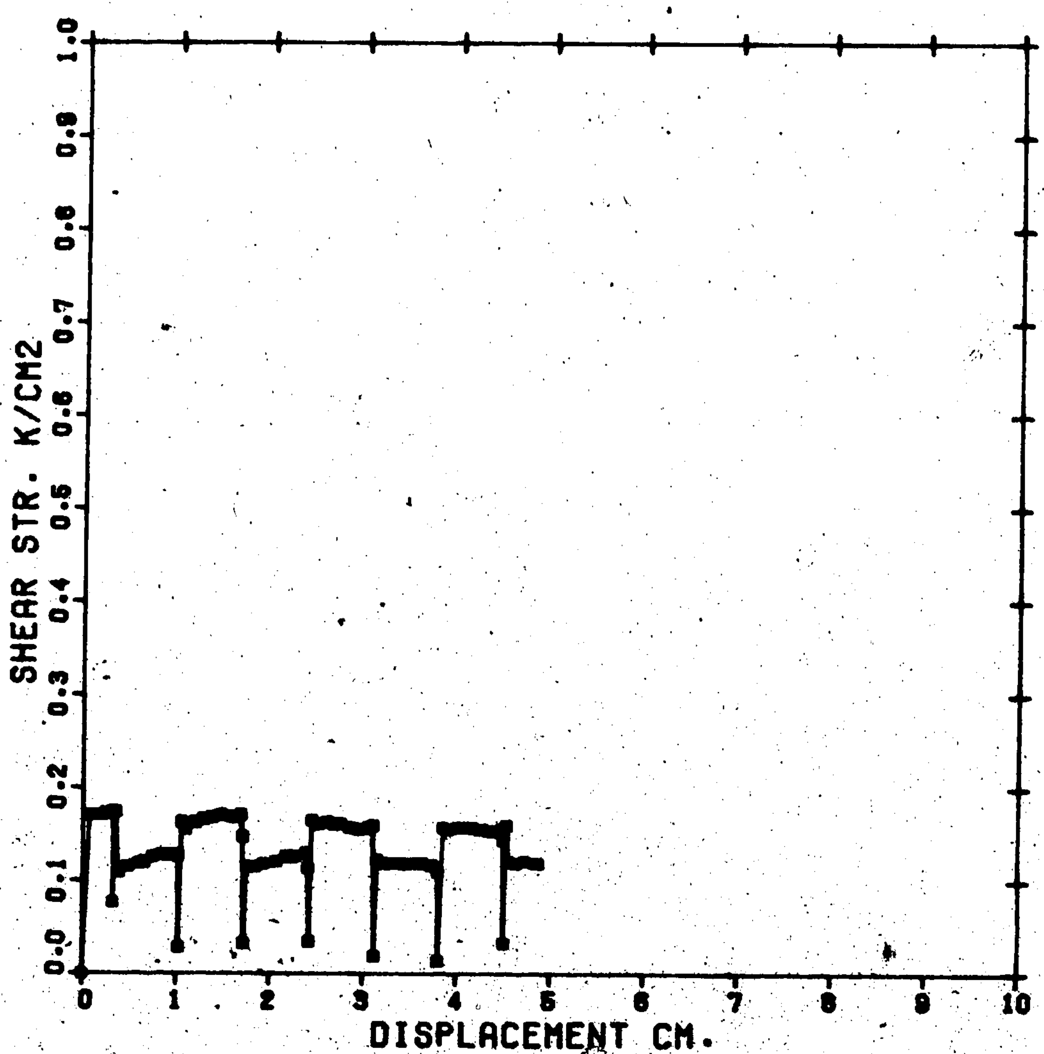


FIGURE C15 DIRECT SHEAR TEST ON REMOULDED BENTONITIC CLAYSHALE



SAMPLE D2 REMOULDED BENTONITIC CLAYSHALE

FIGURE C16 DIRECT SHEAR TEST ON REMOULDED BENTONITIC CLAYSHALE



SAMPLE D3 REMOULDED BENTONITIC CLAYSHALE

FIGURE C17 DIRECT SHEAR TEST ON REMOULDED BENTONITIC CLAYSHALE