

Erosion and Stability of Till Cliffs
on the Holderness Coast

This thesis is submitted in fulfilment of the requirements
for the Degree of Doctor of Philosophy
in Geotechnical Engineering

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University of Newcastle upon Tyne
Geotechnical Engineering

July 1990

ABSTRACT

The Holderness coast, located in the county of Humberside, is undergoing severe erosion because the cliffs are composed of soil, predominantly clay tills, and are exposed to strong marine attack. The cliffs are subject to several erosion mechanisms and modes of failure, which vary with location on the coast. A general mapping survey of the coast and detailed field and laboratory investigations at two sites were undertaken, to study them and the physical behaviour of the cliff as it recedes.

The research has shown that the main controls on the mode of failure are the cliff height, state of the beach and presence of minor soil types within the tills. The cliffs stand at a steep angle ($40-50^{\circ}$) on unprotected coast, because pore water pressures are low, as a result of drainage to the underlying Chalk and sand layers. Where the cliff toe is protected, the slope angle decreases to between 20° and 30° , by shallow slips and mudslides. Indirect evidence of the stress relief caused by unloading was given by observation of ground movements adjacent to the cliff. Depressed pore water pressures resulting from undrained unloading were not observed. However stability analyses indicate that it is likely there are small depressions of pore water pressure in the cliff.

Stability analyses have shown that a more realistic factor of safety is obtained if strength parameters are measured by tests which take account of the slip geometry.

Erosion and removal of cliff debris from the toe, particularly when the beach is absent, is necessary for the continued occurrence of deep seated slips and overall recession of the coast.

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DEFINITIONS OF SYMBOLS USED

A	pore pressure parameter
A _f	pore pressure parameter at failure
\bar{B}	pore pressure parameter
c'	effective cohesion
c _s	coefficient of swelling
c _u	undrained shear strength
c _v	coefficient of consolidation
e	void ratio
H	driving head in falling head permeability test
H ₀	equilibrium head in falling head permeability test
k	coefficient of permeability
K ₀	coefficient of earth pressure at rest
m _v	coefficient of volume compressibility
N _k	cone factor
p'	$\frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3}$
q _c	cone resistance
r _u	pore pressure ratio
s	$\frac{\sigma'_a + \sigma'_r}{2}$
s'	$\frac{\sigma'_a + \sigma'_r}{2}$
t	$\frac{\sigma'_a - \sigma'_r}{2}$
t ₉₀	time for 90% consolidation

u	pore water pressure
u_a	pore air pressure
w	water content
γ_w	unit weight of water = 9.8 kN/m^3
σ	total stress
σ_a	total axial stress
σ_r	total radial stress
σ'_v	effective vertical stress
σ'	effective stress
σ_1	major principal stress
σ_3	minor principal stress
τ	shear stress
ϕ'	effective angle of shearing resistance

ACKNOWLEDGEMENTS

The work has been conducted during the tenure of an S.E.R.C. "Total Technology" research studentship. Financial assistance for fieldwork accommodation and labour was provided by Humberside County Council and Hornsea Town Council. Permission to use the Grimston site was granted by Mr. Beadle of Moat Farm, Grimston, and the Rolston site by Humberside County Council. I am grateful to my supervisors, Dr. B.G. Clarke and Dr. M.S. Money, for their assistance and encouragement, and to all the students who helped with the fieldwork. Finally, thanks to my family for their support during the study.

SUMMARY

In this thesis the results and conclusions from field, laboratory and analytical work on the till cliffs of the Holderness coast are given. The fieldwork was in two parts: a general study of the coast, and detailed studies at two cliff sites. In the general survey photographs were used to record geology and modes of failure along a 28km length of coast. At the sites, cliff changes and instrumentation to measure pore water pressure and ground movement were monitored for a two year period. Laboratory and field testing was done to determine soil profiles, index properties, and shear strength and consolidation parameters. The results were then used to analyse groundwater conditions and cliff stability.

The Holderness coast is in the county of Humberside, between the headlands of Flamborough and Spurn. It is being severely eroded because the cliffs are composed of soil, and are often exposed to strong marine attack. The cliffs show several modes of failure, which vary along the coast. The main controls of this variation are the geology, cliff height and beach conditions.

In the general survey the coast was divided into zones, based on the mode of failure and cliff morphology. The relation of the zones to the above-mentioned controls was investigated, and the cycle of recession in each was described.

In areas where the beach is absent, erosion takes place rapidly, and accelerates the cycle of recession. The beach at any place can be removed, at intervals of several months, by natural beach lows, locally termed "ords". These are important in maintaining the high rate of recession of the coast. In areas where the beach is artificially built up, the cycle of recession is changed. Mudslides and shallow slips replace deep seated slips and falls as the chief modes of failure.

On actively eroding coastline slips become more common than falls with increasing cliff height. The slip geometry is strongly influenced by the geology of the cliffs. A layer of stone-free silty clay often caused base failures to occur, instead of toe failures which occur in homogenous till. Sand and red clay layers in the tills were found to encourage falls. They were more easily eroded than till, causing the cliffs to become undercut and collapse.

The study at the two sites showed that recession of the cliffs takes place under predominantly drained conditions. Depressed pore water pressures induced by unloading of the cliff are expected to dissipate quickly in relation to the rate of recession. However as minor unloading occurs frequently, a small depression of pore water pressure in the cliff is possible. Deep seated rotational slips occurred at both sites. Regular surveying of slip movements showed the failure surface to be non-circular, with a linear back portion and curved toe.

The rate of slip movement was influenced by seasonal pore water pressure changes, beach thickness and toe erosion.

Stability analyses showed that the slipped soil masses must be completely eroded, and the cliff steepened, before a new slip occurs. This is consistent with the observed cycle of recession at the sites. The steep angle of 40° to 50° at which the cliffs stand before failure, is caused by low pore water pressures, resulting from drainage to the sand layers and underlying Chalk.

Ground movements behind the cliff top were measured over an 18 month period, and showed that deformation was occurring in response to stress relief. The effect of stress path on the effective stress shear strength parameters was investigated, although the results were inconclusive. The factor of safety obtained in stability analyses of cliff profiles was generally slightly less than 1, unless zero pore water pressure was assumed. This suggests that the pore water pressure on the slip surface is depressed by unloading to some extent. If ϕ' from direct shear tests was used at the toe of the slips, a factor of safety closer to unity was obtained. These tests are more representative than triaxial tests of the mode of shearing around the slip toe.

The research shows that to slow recession of the coast, erosion of the till shore platform and cliff toe must be prevented. This can be best achieved by engineering works that reduce the

wave energy reaching the shore, and provide some erosion protection. Offshore barriers have been proposed to reduce the height of incident waves. However, it is likely that recession would continue, unless works are also built at the shore to prevent natural lowering of the beach, and which do not themselves cause beach depletion.

CHAPTER 1

INTRODUCTION

In this chapter the Holderness coast and nature of the erosion are described. It is then shown how the work undertaken for this thesis relates to possible coast protection schemes. Finally the work is outlined and the organization of the thesis described.

1.1 Location

The Holderness coast is located in the county of Humberside, bounded by the chalk headland of Flamborough to the north, and the spit of Spurn to the south (see Figure 1.1). The Holderness plain extends inland from the coast to the Yorkshire Wolds, and is a low lying undulating area. Land use in the coastal belt is predominantly for agriculture. There are two urban areas at the towns of Hornsea and Withernsea, a number of caravan parks, a North Sea gas terminal and a military test range. The cliffs are from 4m to 30m in height, with a mean height of 15m. They are formed entirely of glacial soils, predominantly tills, and are underlain by Chalk below sea level.

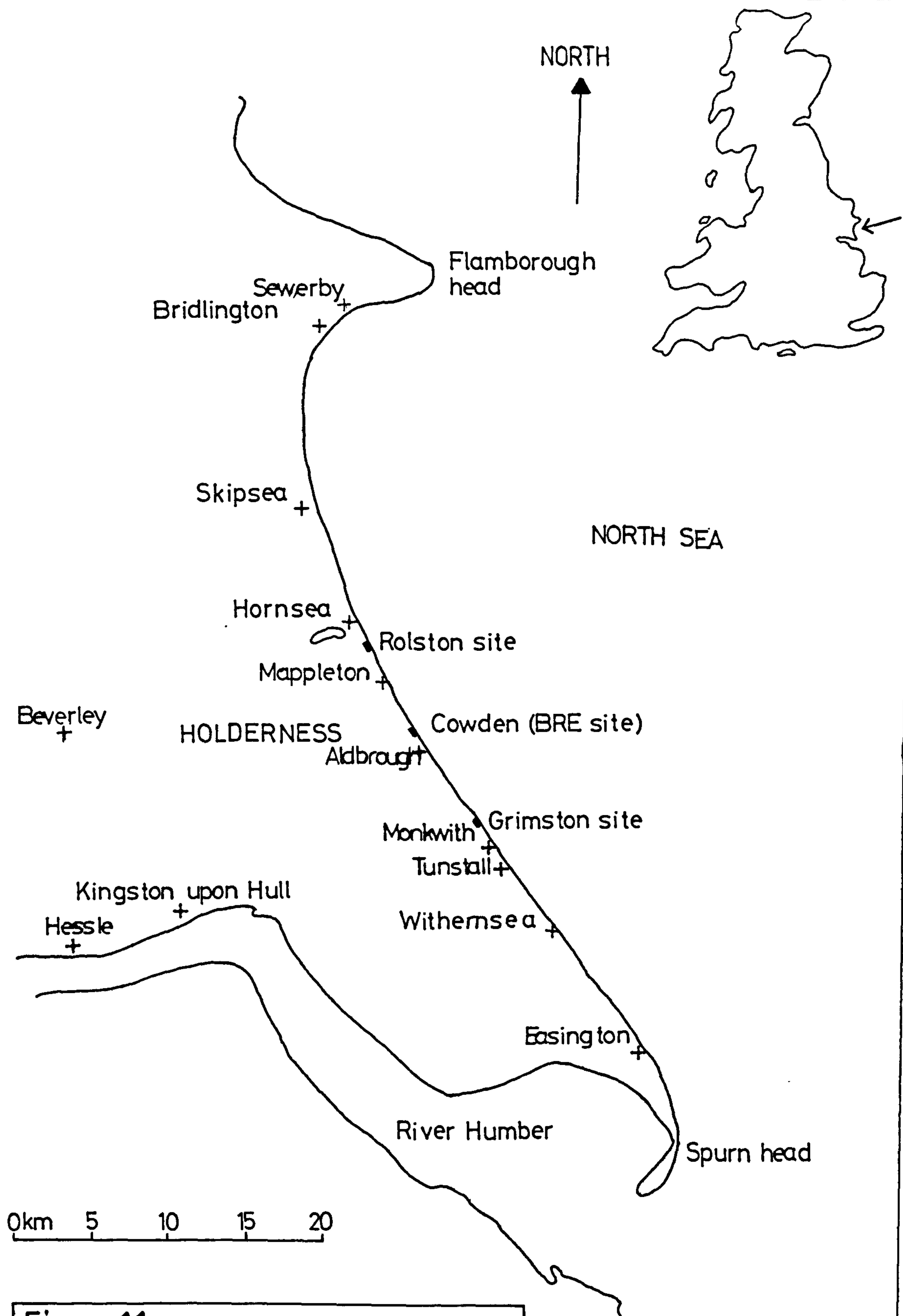


Figure 1.1
Location map of the Holderness coast

1.2 Erosion Problem and Joint Advisory Committee on Coast Protection

The coast is undergoing severe erosion owing to the nature of the cliff materials, and the high energy wave conditions they are subjected to (see Figure 1.2). Recession of the cliffs has continued for at least several hundred years (since earliest records), and is currently occurring at a rate of 2m/year, leading to an annual loss of 10 hectares of land along the 55km coastline. Sea defences have been maintained at the towns of Hornsea and Withernsea from the mid-19th century. They have temporarily halted recession there, but have exacerbated erosion of the cliffs beyond. Although the towns are not immediately threatened, the steady erosion of land is the permanent loss of a national asset. Within 25 years at present erosion rates the village of Mappleton and the B.P./British Gas terminal at Easington will be threatened. Also the gradual outflanking of existing sea defences will shorten their lifespan. A Joint Advisory Committee (J.A.C.) on Holderness coast protection was formed by Humberside County Council, and East Yorkshire and Holderness Borough Councils, with the aim of evaluating low cost solutions to the erosion problem. There is insufficient cost-benefit at the coast to justify extending conventional methods of protection beyond the existing sea walls.

The recession of cohesive shorelines is now regarded as an interaction of the coastal landmass, the littoral zone and the



Figure 1.2

The Holderness coast south of Hornsea, with high energy wave conditions during a March storm

sea offshore, which must be understood to design a successful stabilization scheme (Hutchinson, 1983). Also it is realized that a scheme must be designed for an entire coast system, instead of 'piecemeal' protection at the location of the towns. Little information was available on the performance of low cost methods of protection on high wave energy coasts, the control of erosion by the beach, the beach and sediment transport processes, and the physical behaviour of the cliff. Consequently several complementary research projects were initiated by the J.A.C., including a study by the Department of Geotechnical Engineering, University of Newcastle upon Tyne, of which this thesis forms a part. The research undertaken for this thesis has been concerned primarily with the geotechnical aspects of cliff recession.

1.3 Solutions to the Erosion Problem

Any solution to the problem must seek to:

1. prevent erosion of till from the cliff, the foreshore and the offshore seabed;
2. ensure stability of the cliff under all conditions, particularly where property or revetment works could be affected.

A feasibility study by the Victoria University of Manchester (1983) examined some possible solutions. The current proposal is to construct large submarine barriers several hundred metres offshore, causing waves to break and dissipate their energy before reaching the near shore zone. Another scheme which has been considered is to construct large artificial headlands at intervals of a few kilometres, which would trap beach sediment. Erosion would continue between the headlands until an equilibrium serrated coast plan was reached. Wave energy reaching the shore would then be dissipated by the beach. Both of these solutions may still require protection of the cliff face on some parts of the coastline, to deal with storm surges, and waves which continue to reach it.

1.4 Scope and Relevance of this Study

This study was undertaken for two main reasons:

1. the relevance of research into slip behaviour and stability to geotechnical engineering practice;
2. to provide information for the conceptual design of a stable, erosion resistant coastal slope.

It provides a contrast to studies of coastal cliffs and man-made excavations in other materials. There are few case histories in which ground movement and pore water pressure

behind rapidly eroding slopes or excavations have been monitored for a long period. Also the importance of obtaining field observations to verify the results of laboratory tests and methods of analysis is often emphasized.

The work was undertaken between October 1985 and October 1988, during the tenure of a Science and Engineering Research Council 'Total Technology' research studentship. It consisted of a survey of the variation in mode of failure and cliff morphology along the coast, and the factors causing it, and a detailed study of groundwater conditions, cliff stability and changes as recession occurred at two sites, Rolston and Grimston. During the first six months of the study, the literature was surveyed and the fieldwork planned.

The fieldwork commenced in April 1986 with a survey of 28km of the coast. Instrumentation was installed at two sites during the summer of 1986, to measure pore water pressure and ground movement, and was monitored until April 1988. The ground conditions at the sites were explored by surface mapping, static cone penetration testing, drilling and soil sampling. Laboratory tests on soil samples included triaxial stress path and oedometer tests. The field and laboratory data gathered were used in analysis of the groundwater conditions and cliff stability at the sites.

A substantial part of the coast was not surveyed during the study, and uncertainty remained in many of the factors

investigated at the two sites. Some suggestions for further research are therefore made at the end of the thesis. The literature is reviewed in Chapter Two, and the general survey of the coast and detailed site investigation are described in Chapters Three and Four. In Chapter Five the results of analyses are presented and the work is discussed. Finally the conclusions are summarized and recommendations made in Chapter Six.

CHAPTER 2

LITERATURE REVIEW

In this chapter literature relevant to this study of erosion and stability is reviewed. The historical aspects and nature of the erosion are described, followed by the geology and beach conditions. Then the general behaviour of slopes in till and other materials is discussed. Finally, previous geotechnical investigations of the Holderness cliffs are reviewed, and the main research questions posed at the start of the study are outlined. Some of the previous work is described in greater detail at the appropriate place in later chapters.

2.1 History of Holderness Coast Erosion

The Holderness plain is formed by a till sequence, underlain by Cretaceous Chalk (Derbyshire et al., 1983). The cliffs are formed entirely in the tills, which are susceptible to marine erosion. The eroded cliff material coarser than silt forms a beach which covers the till shore platform along most of the coast. The beach normally provides some protection for the till from erosion, however along short sections of the coast it is frequently removed.

The Holderness coast has probably been subject to marine erosion since between 3000 to 5000 years ago, when sea levels recovered to their present levels following a eustatic depression of over 100m during the last glaciation (Hutchinson, 1983).

Records of the effects of erosion on land and property go back several centuries. These include parish records, appeals for assistance to King or Parliament, descriptions, reports of official enquiries since the 14th Century and maps and charts, measurements and soundings made between the Elizabethan period and the present (Williams, 1984). About 29 settlements have been lost to the sea (Sheppard, 1912). The destruction of one of these, Ravenserod, is described in an enquiry of the 20th year of Edward III before the Kings Commissioners:

"Two parts and more of the tenements and soil of the town by the flux of the water of the sea are beaten down and carried away; and because the said town is daily diminished, many inhabitants have withdrawn themselves, their goods and chattels, as the dangers there continue to increase from day to day, and are gone to dwell elsewhere".

(Williams, 1984)

An inquisition held at Hornsea, 1609, reported that:

"We find decayed by the flowing in of the sea since the first year of Edward VI (ie. in 63 years) 38 houses. Also we find in the same time decayed in ground the breadth of twelve score yards ... to the great hurt and impoverishment of the inhabitants of Hornsea if that a present remedy be not made for the safeguard of the said lands".

(Williams, 1984)

Dossor (1955) states that much private and public money has been spent on protective works over the years. The Royal Commission on Coast Erosion (1906-1911) found that, "the erosion in East Yorkshire was the most serious around the coast of the British Isles".

Significant factors in the coastal erosion include the structure and height of the cliffs, beach, and sea bed, the tidal range and currents, the "fetch" and strength of onshore winds, and storm surges (Dossor, 1955).

The earliest reliable maps of the Holderness coast are the Ordnance Survey maps of 1852 at a scale of 6 inches to the mile. Earlier maps exist such as T. Jeffries' map of Yorkshire, 1775, and H. Teesdales' map of Yorkshire, 1835, both at a scale of 1 inch to the mile. However these are probably not very accurate and estimates of the rate of erosion from them are two to three times higher than the rate today, which is unlikely.

The positions of the coastline between Bridlington and Aldborough have been plotted onto a drawing by Davison (1985) from the above-mentioned maps and from the 1951 2 inch to 1 mile Ordnance Survey edition and the 1984 1 to 10,000 Ordnance Survey edition. This shows the form of the coastline to be smooth in the long term, and the rate of erosion to increase uniformly to the south, except at the position of coastal defences. At Hornsea the coastal defences have slowed erosion

north of the town, apparently since at least 1775 and increased it to the south, particularly between 1951 and 1984.

The rate of retreat of the coastline calculated from maps is shown in Figure 2.1. Also included is data from a study by Valentin (1954) who measured the distances of features from the cliff edge in 1952 and compared them with the 1852 Ordnance Survey map to obtain the rate of retreat, and the measured rate of retreat by Holderness Borough Council between 1974 and 1985.

The data of Valentin (1954) gives a 100 year average rate of erosion between 1852 and 1952 of about 1.2m/year.

	(a) 1852-1952 100 years	(b) 1951-1984 33 years	(c) 1974-1985 11 years
Parish:			
Bridlington	0.4	-	-
Carnaby	0.5	-	-
Barmston	0.5	-	-
Ulrome	1.6	1.5	-
Skipsea	1.4	0.9	0.8
Atwick	1.1	0.9	1.2
Hornsea	0.8	1.8	2.8
Mappleton	1.5	1.5	2.5
Aldbrough	1.2	2.7	2.5
East Garton	1.1	-	2.3
Roos	0.9	-	2.5
Rimswell	0.9	-	1.9
Withernsea	1.1	-	0.5
Hollym	1.4	-	-
Holmpton	1.5	-	-
Easington	1.9	-	-

Source: (a) Valentin (1954).
 (b) Ordnance Survey Maps.
 (c) Holderness Borough Council.

Figure 2.1 Table of Erosion Rate Data.

The rate of retreat between 1951 and 1984 over a 33 year period is slightly higher than that of Valentin, although it is locally lower, as at Atwick and Skipsea. However, over the period 1974-1985 the rate is nearly twice as high, particularly on undefended coast south of Hornsea. Thus while the long term rate of retreat (over several decades) appears relatively uniform, in the short term (a few years) it may be much higher. This probably reflects the erosion caused during particularly severe winters, or even individual storms. Another possibility is that the erosion rate has increased in recent years due to depletion of the beaches by sea defences.

The storm surge of 31 January 1953 produced remarkable changes in the coast of Holderness according to Valentin (1954). He estimated that the retreat of the shore caused by the storm was some multiple of the annual rate over the 100 year period, and also notes severe damage at Lowestoft where between 9m and 26m of cliff recession occurred in 17 hours. Dossor (1955) records that the evening tide of 31 January 1953 was 6 feet above the predicted level at the mouth of the Humber, but did not find any increase in the average annual rate of recession between September 1952 and September 1953. However locally the erosion was much increased, as at Atwick where 8m of erosion had occurred.

Other factors which may have influenced erosion are the removal of sand and gravel for construction work prior to 1910, and the

installation of land drains which discharge water onto the cliff face (Dossor, 1955).

Valentin (1954) summarizes the land loss at Holderness by saying that between 1852 and 1952 the cliffs receded an average of 120m and approximately 720 hectares of land were lost, or about 100 million cubic metres of material. In general erosion is more severe in the south-east, and this is due to the greater exposure to wave attack.

2.2 Holderness Geology and Sedimentology

2.2.1 Pre-Quaternary

The Cretaceous Chalk underlying the till sequence has been gently folded to form a southeast plunging syncline. The Chalk outcrop forms the Yorkshire Wolds to the north and west of Holderness, between Flamborough head and the river Humber. The Chalk surface lies at a depth between -20 and -60m O.D. below Holderness, and has been dissected by erosion during Quaternary interglacial periods (Foster et al., 1976).

2.2.2 Quaternary

The Quaternary climate in North West Europe since about 2.5 million years ago has alternated between temperate and cold

periods. However extensive glaciations occurred only during the cold periods towards the end of the Quaternary since about 0.6 million years ago. Cold periods where the land was at least partially covered with ice are termed glacials. The intervening temperate periods are known as interglacials. A table of Quaternary stages in the British Isles, dates, climatic conditions and typical deposits is given in Figure 2.2. This shows there were four glacial periods: Beestonian, Anglian, Wolstonian and Devensian.

The ice limits of the most recent glaciation, which took place towards the end of the Devensian glacial stage extend south to North Norfolk. The pre-Devensian ice limits extend further south into the Home Counties. However most of the pre-Devensian deposits in Holderness were eroded by the Devensian ice, except at Dimlington cliff in southern Holderness where Wolstonian deposits may be preserved below the Devensian. A buried cliff and beach from the Ipswichian interglacial bisects the present coast at Sewerby and runs inland via Beverley and Hessle (Madgett and Catt, 1978).

The Devensian succession exposed in the Holderness cliffs consists of lodgement tills and minor facies of flow tills, sands, silts and gravels (Derbyshire et al., 1985). The lodgement tills were deposited after about 18,250 B.P. in Holderness (Penny et al., 1969). The oldest radiocarbon date for deposits above the tills is about 13,045 B.P. from The Bog, Roos (Madgett and Catt, 1978).

Dates in years B.P.	Stage	Climate	Examples of typical deposits
Recent or 10,000 Holocene	Flandrian	Temperate	Silts & peats in Lincolnshire, East Anglia and Lancashire; estuarine deposits - Thames etc.
115,000	Devensian	Cold, glacial at end of stage	Head in S & SW England; Skipsea till, Irish Sea till; Four Ashes Gravel; Dimlington Silts
128,000	Ipswichian	Temperate	Organic deposits near Ipswich; beach deposits at Sewerby & in Devon
	Wolstonian	Cold, in part glacial	Welton till, Fremlington till
	Hoxnian	Temperate	Organic deposits at Hoxne, Suffolk; Marks Tey, Essex
	Anglian	Periglacial and glacial	Lowestoft till, North Sea Drift, Corton Sands; Barham Sands and Gravels; Barham Arctic Structure Soil

Figure 2.2 Quaternary stages, dates, climatic conditions and some typical deposits after Anderton et al, (1979).

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Dates in years B.P.	Stage	Climate	Examples of typical deposits
	Cromerian	Temperate	Valley Farm Rubified Sol Lessive; peats & freshwater sediments - Norfolk
	Beestonian	Periglacial and glacial	Kesgrave Sands & Gravels
	Pastonian	Temperate	Westleton Beds) Weybourne Crag Icenian Crag,) Chillesford Clay including) Norwich Crag
600,000 hiatus 1600,000	Baventian	Cold	Marine silts & clays - Ludham & Easton Bavents
	Antian	Temperate	Shelly sand)
	Thurnian	Cold	Silt) Marine sediments in Ludhamian) Ludham borehold and Shelly sand) Suffolk
2050,000 hiatus 2450,000	Pre- Ludhamian	Temperate	Red Crag
2500,000			
PLIOCENE		Warm temperate	Coralline Crag

Figure 2.2 Continued.

There are two distinct lodgement tills, differentiated on the basis of matrix colour, clast content and grading (Madgett and Catt, 1978). The Skipsea till (formerly 'Drab till') is dark greyish brown, with local white, grey or red streaks and numerous chalk clasts. Withernsea till (formerly 'Purple till') is uniform dark brown. Withernsea till is exposed in the cliffs between Hornsea and Easington. Skipsea till underlies the Withernsea till throughout this area, however between Holmpton and Tunstall the Skipsea/Withernsea till boundary is below beach level and the cliffs are entirely of Withernsea till. North of Hornsea, Skipsea till forms the entire cliff (Derbyshire et al., 1983). Post glacial weathering has changed the colour of the uppermost 5m of the tills to a reddish brown throughout Holderness, formerly mistaken for a separate till (Madgett and Catt, 1978).

The Skipsea and Withernsea tills were shown to have several divisions in detailed geological mapping of the coast by Bisat in the 1930s (Catt and Madgett, 1981), again on the basis of colour, clast content and grading. Some of the divisions were of laminated stone-free clays and red clays. The tills are thought to have been deposited by lodgement beneath a wet-based sliding ice-sheet (Derbyshire et al., 1985). Evidence for lodgement is given by till mesofabrics, including crushed and sheared out chalk clasts, and from their geotechnical characteristics. In places more random clast fabrics indicate sub-glacial remoulding of wet till has occurred. Channels of

sand and gravel within the till result from basal drainage beneath melting ice (Foster, 1985).

The laminated stone-free clays are thought to have been deposited sub-aerially, as the ice temporarily retreated or shifted eastward. Some evidence for at least three ice advances during the Devensian is provided by a study of end moraine forms in Holderness and Lincolnshire (Straw and Clayton, 1979). The colour of the red clays may be caused by sub-aerial weathering. Derbyshire et al., (1985) referring to the stone-free clays and red clays, noted that, "sequences of this kind occur on at least two levels within the Skipsea till and at the base of the Withernsea till". The minor soil types described above are important in controlling cliff behaviour (see Section 3.2).

The layering of the till sequence (apart from the minor soil types), can be explained by a "change in the mix of lithologies deposited at any one point due to a lateral shift in ice-flow, or by the alteration of lithologies carried along a single flow line" (Foster, 1985). Analysis of the till petrography has shown that the Skipsea till was deposited from a Pennine-Lake District ice-stream, while the Withernsea till was deposited from a Southern Uplands and Cheviot ice-stream. The confluence of the two probably occurred in the Tees lowlands (Foster, 1985). A different model which involved the downwasting of a multilayered ice-sheet was originally proposed to explain the layering (Catt and Penny, 1966). However this is not supported

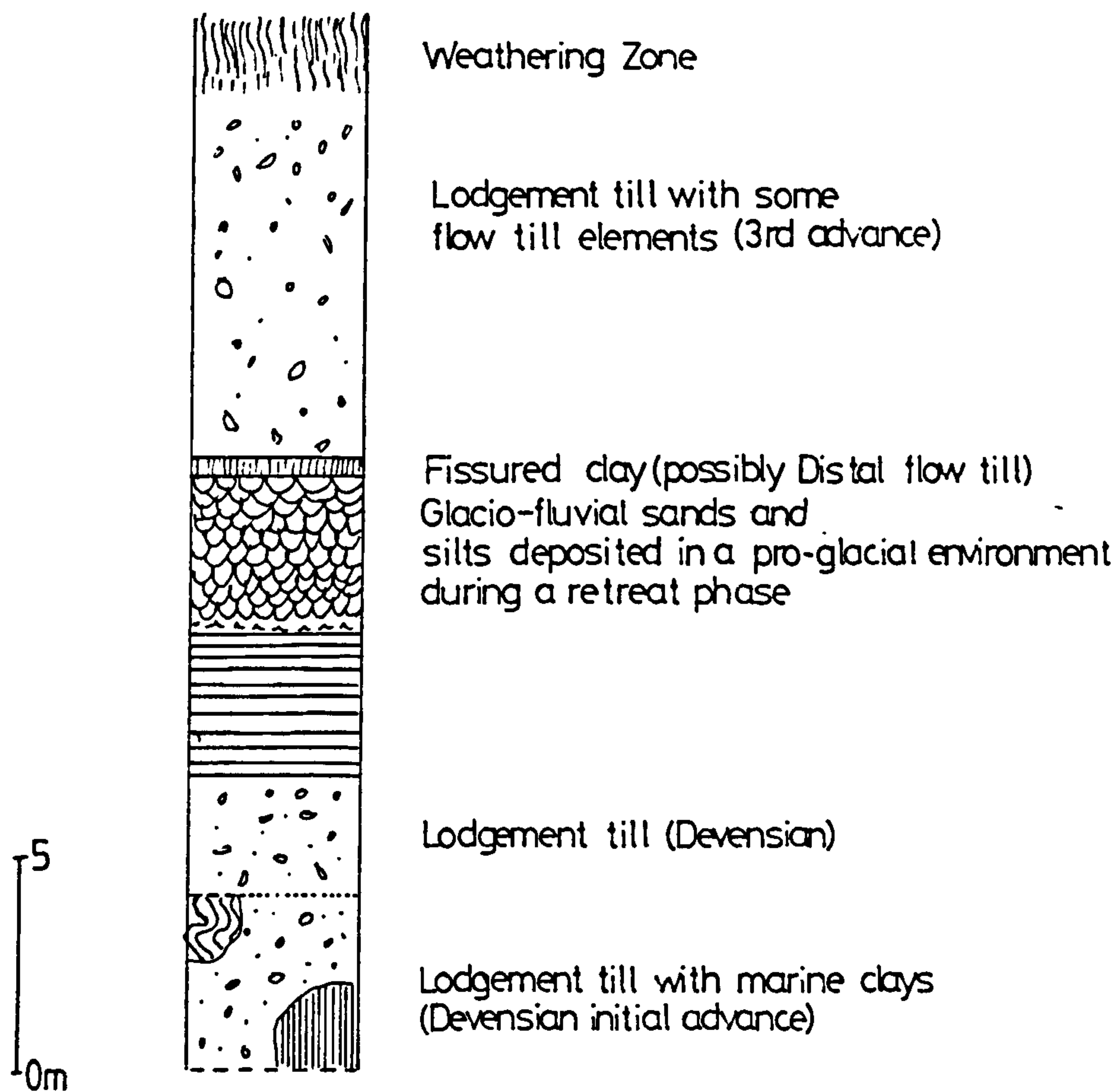
by recent sedimentological research. There is no evidence of "large scale slumping, melt out or re-sorting of the till fabric" (Derbyshire et al., 1985) which would be expected in deposits of this origin.

A section of the cliffs at Dimlington in Southern Holderness is given in Figure 2.3 showing some of the minor soil types described above (Derbyshire et al., 1983).

2.3 Beach Processes and their Influence on Erosion

Three major research projects have investigated the beach processes on the Holderness coast, including two based at Lancaster University (Scott, 1976; Pringle, 1985) and one at Hull University, currently in progress (1988). These show that the beach geometry has a large influence on the rate of till erosion from the shore and cliff foot. The rate is highest in areas where the beach is naturally lowered, locally called "ords", or artificially lowered adjacent to sea defence works.

The Holderness beaches are composed of sand and gravel derived from the till cliffs (approximately 30% of the till by weight is of sand size or above). The amount of sediment supplied to the beach is small until south of Barmston, because to the north the cliffs are low and sheltered from erosion by Flamborough head. The movement of sediment is predominantly southwards, as shown by the direction of "ord" movement and



Lithology

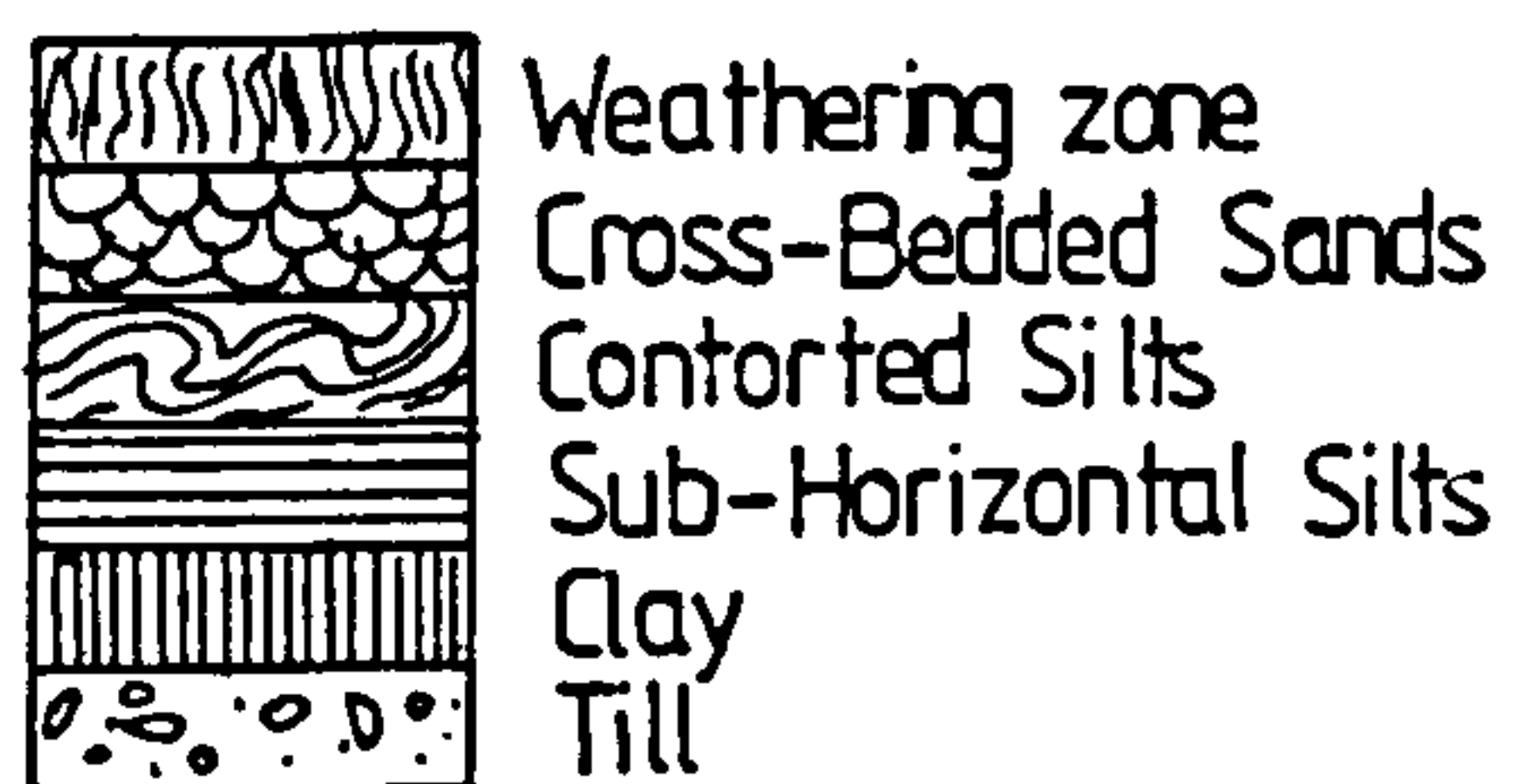


Figure 2.3
Section of the cliffs at Dimlington
(after Derbyshire et al., 1983)

calculated from wave data (Pringle, 1985). This leads to an increase in the size of beaches southwards, as the cumulative input of sediment rises. The characteristic beach profile which occurs between Barmston and Spurn is an upper beach of coarse sand and gravel, overlying a lower beach of fine to medium sand. There is usually a sharp change in profile between the two, between 40m and 80m from the cliff foot (see Figure 2.4).

At regular intervals of a few kilometres the upper/lower beach profile is replaced by an "ord". The main features of an "ord" are shown in Figure 2.4. The upper beach is removed over a length of hundreds of metres, exposing the till shore platform. Thus wave energy is not dissipated by beach sand, and the till shore platform and cliff foot are subjected to severe erosion. Pringle (1985) estimates an eight-fold increase in the volume of till annually eroded in an "ord", compared with the volume eroded from the same length of coast between them. On normal beaches neap tides rarely reach the cliff foot and spring tides occasionally, however in an "ord" over half neap tides and all spring tides reach the cliff foot.

There are two hypotheses for the mechanism by which "ords" are formed. In the first the "ords" develop on the northern Holderness coast and migrate southwards at an average rate of 0.5km/year, primarily by longshore currents. Significant southerly movement only occurs during northerly storms, which produce the highest energy waves incident on the coast. During

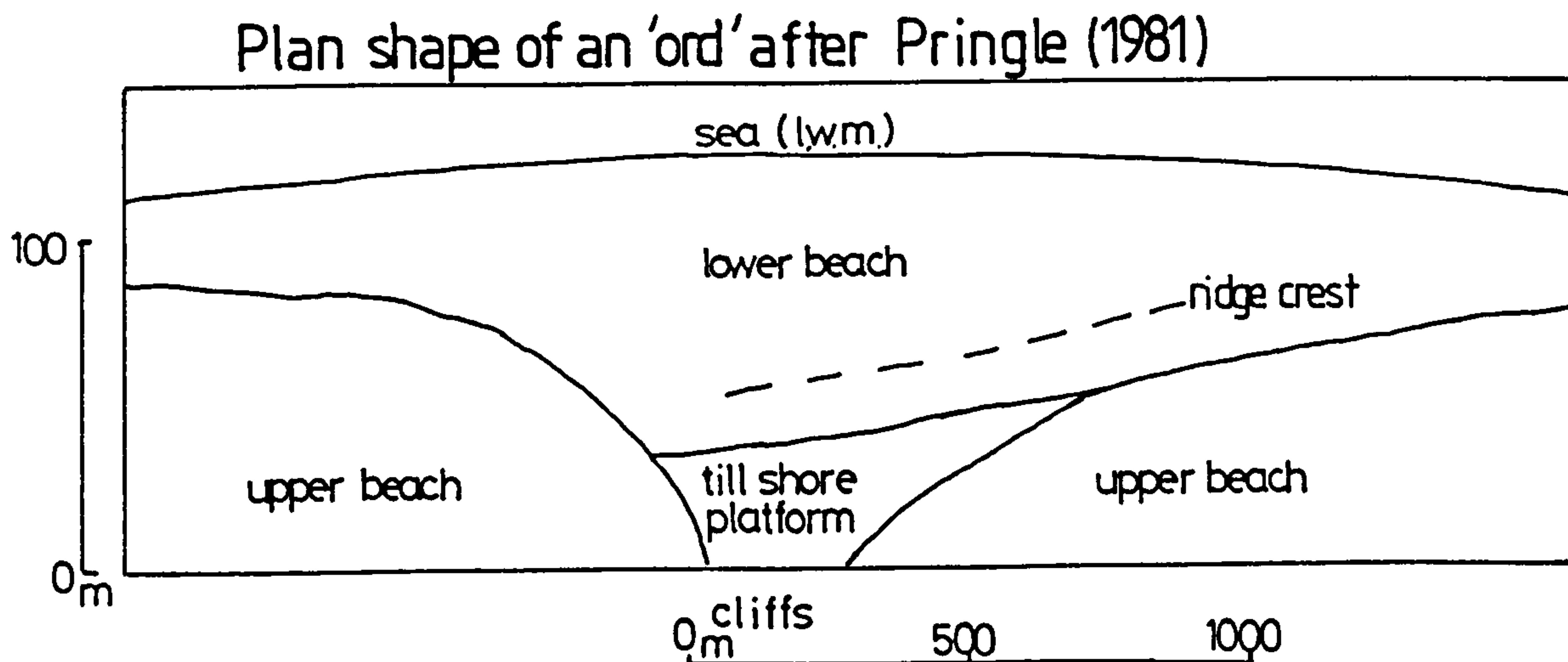
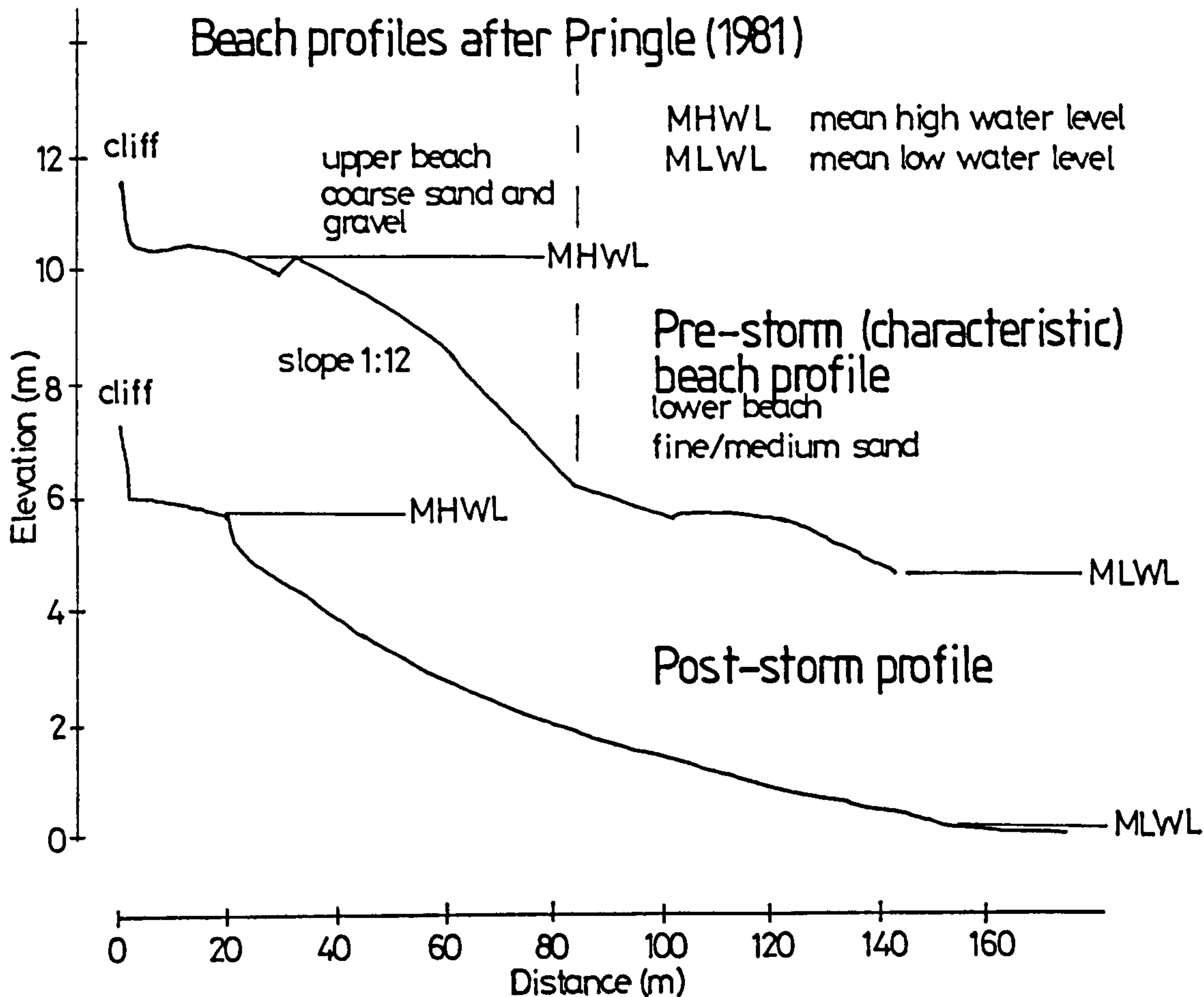


Figure 24

Beach profiles and plan of 'ord' conditions after Pringle (1981)

these conditions a tongue of coarse sand and gravel from the upper beach is formed which is moved southwards. At the same time the southern part of the upper beach is eroded. Thus the centre of the ord is moved south (Pringle, 1981).

In the second hypothesis sediment transport normal to the shore by waves is said to be responsible for the formation of "ords". During periods of high waves there is a net movement of sediment offshore, resulting in depletion of the upper beach.

Whichever of the two hypotheses is correct, observations have shown that the position of "ords" changes with time and that over a period of several years the entire coast will experience "ord" conditions. The features of an "ord" may become subdued by offshore winds and the till shore platform covered with sand. However strong northerly winds quickly expose the till again.

Pringle (1981) examined the relationship of "ords" to rhythmic coastal features found on a variety of coasts throughout the world. It was concluded that the only similar feature was the low sections between beach pads identified on St. Joseph Peninsula, Florida, U.S.A. by Entsminger (1977). The beach pads are rhythmic features which may be related to the built up sections of beach between "ords" on the Holderness coast. Erosion of dunes behind the embayments between beach pads may occur during storms, similar to the situation with "ords".

The absence of sand bars below low water mark and the absence of rip currents suggest that "ords" are not associated with cellular flows, unlike rhythmic features. It is also unlikely they are related to edge waves because of their wide, irregular spacing.

Intense erosion of the cliff foot may also occur when the normal beach profile between "ords" is lowered. During periods of onshore winds, especially from the northern quarter and over 15 knots, the upper beach may be lowered, resulting in an even profile from the cliff foot to the lower beach (see post storm profile, Figure 2.4). The upper beach is rapidly built up again once the winds have dropped (Pringle, 1981). Richards and Lorriman (1987) found a difference between summer and winter beach profiles of 2m at Easington, although the beach may be lowered by storms at any time of year. The studies of the beach have been concerned only with natural lowering of the beach, but at the towns of Hornsea and Withernsea the longshore transport of sediment is interrupted by the sea defences. Permanently high and wide beaches occur to the north of the defences, and low and narrow beaches to the south. Davison (1985) noted that the effect of this on the cliffs was to cause free degradation to the north, and severe erosion (similar to "ord" conditions) to the south.

The above studies have shown the importance of the beach in controlling erosion, and how it varies in geometry, both naturally and owing to the influence of artificial sea defence

structures. There is some indication that the mode of cliff failure is affected by the intensity of toe erosion (hence by the state of the beach), as has been observed in other coastal slopes (see Section 2.4.5). Therefore in the general survey the state of the beach was recorded, and the effect of low and high beaches on cliff behaviour studied.

2.4 Slope Behaviour and Erosion in Tills and Other Cohesive Materials

A general review of the behaviour of natural slopes and erosion of cohesive materials is given. Some of the factors affecting the behaviour of slopes in till and other materials and earlier measurements of shear strength in tills are discussed in this section. Phenomena observed in coastal slopes in other cohesive soils, which may also occur in Holderness, are also reviewed. Geotechnical investigations of the Holderness cliffs are discussed in the next Section.

2.4.1 The Behaviour of Natural Slopes

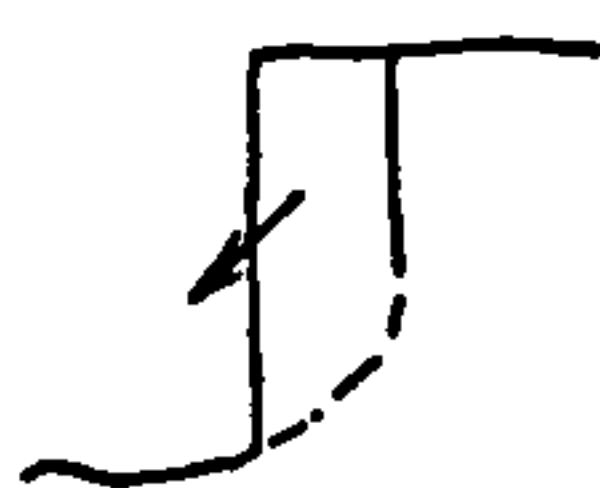
In this section a brief general review of the behaviour of natural slopes is given based on the state-of-the-art report by Skempton and Hutchinson (1969). This includes a description of the various mass movements, and a discussion of the strength

properties of clays and the discrepancies between field and laboratory strengths.

Several basic types of mass movement occur in response to gravitational forces. Other landslide forms are in general multiple or complex assemblages of these basic types. Some of these basic and complex forms are shown on Figure 2.5 and are discussed below.

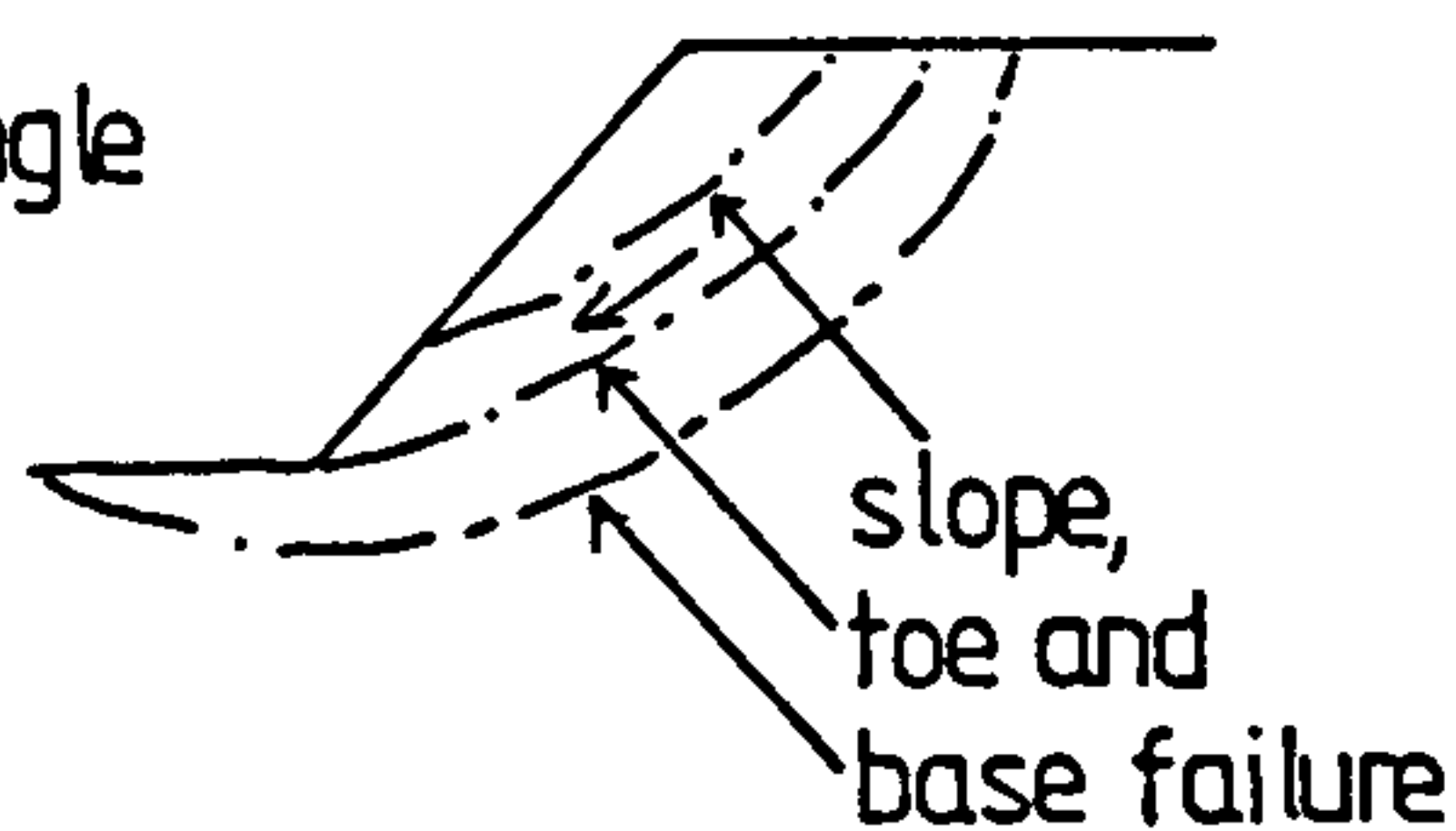
- (i) Falls generally occur as short-term failures of steep slopes in strong clays. Tension cracks may form behind the fall, and the collection of water in these can reduce stability. The size and manner of the fall can be strongly influenced by the presence of joints and fissures.
- (ii) Rotational slides are common in slopes of homogenous clay. The ratio of depth to length is usually in the range 0.15 to 0.33. The deeper and better developed slips tend to develop in steeper slopes such as cuttings and actively eroding cliffs. The shape of the slip surface is usually circular in homogenous slopes and non-circular where there is weathering or anisotropy.
- (iii) Compound slides where the slip surface has both curved and planar portions occur where there is heterogeneity at moderate depth in the slope. This may be a weak soil

FALLS



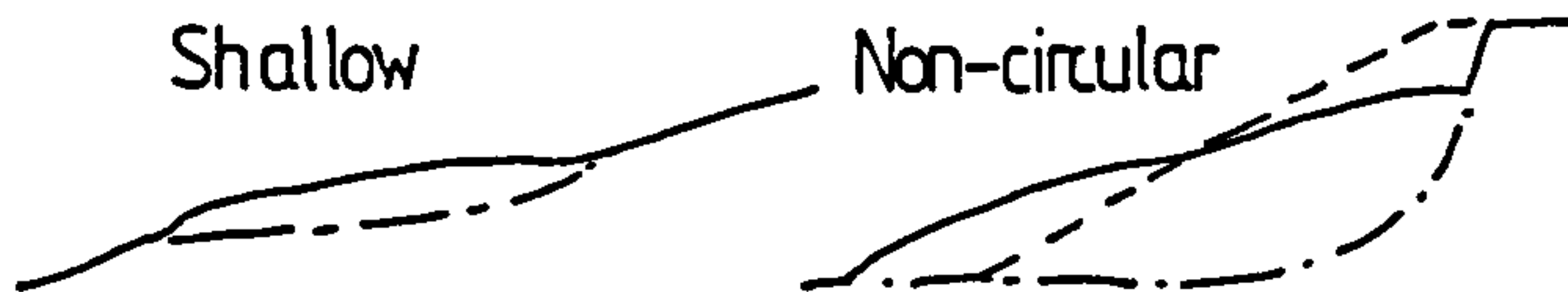
ROTATIONAL SLIPS

Single



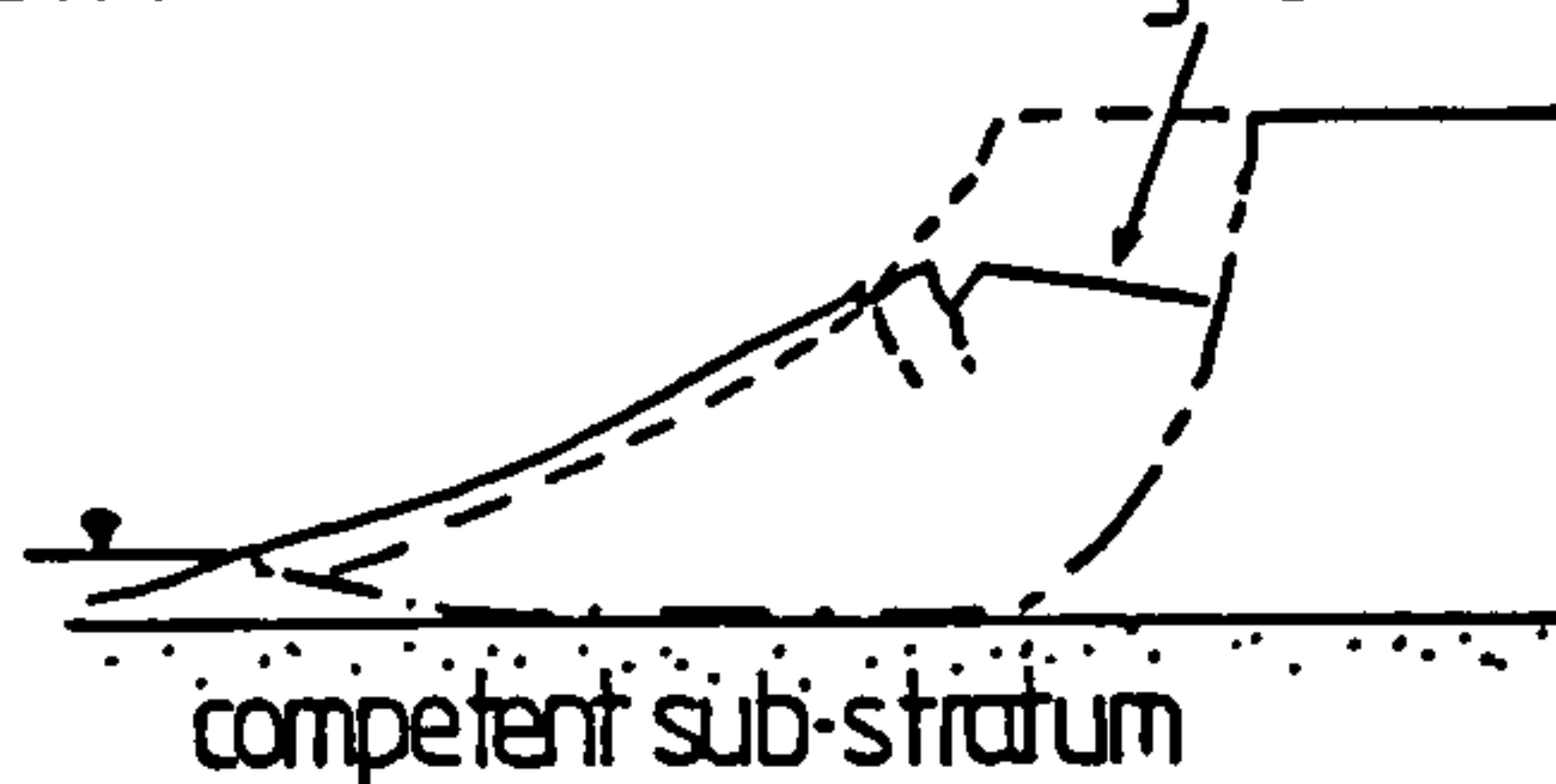
Shallow

Non-circular

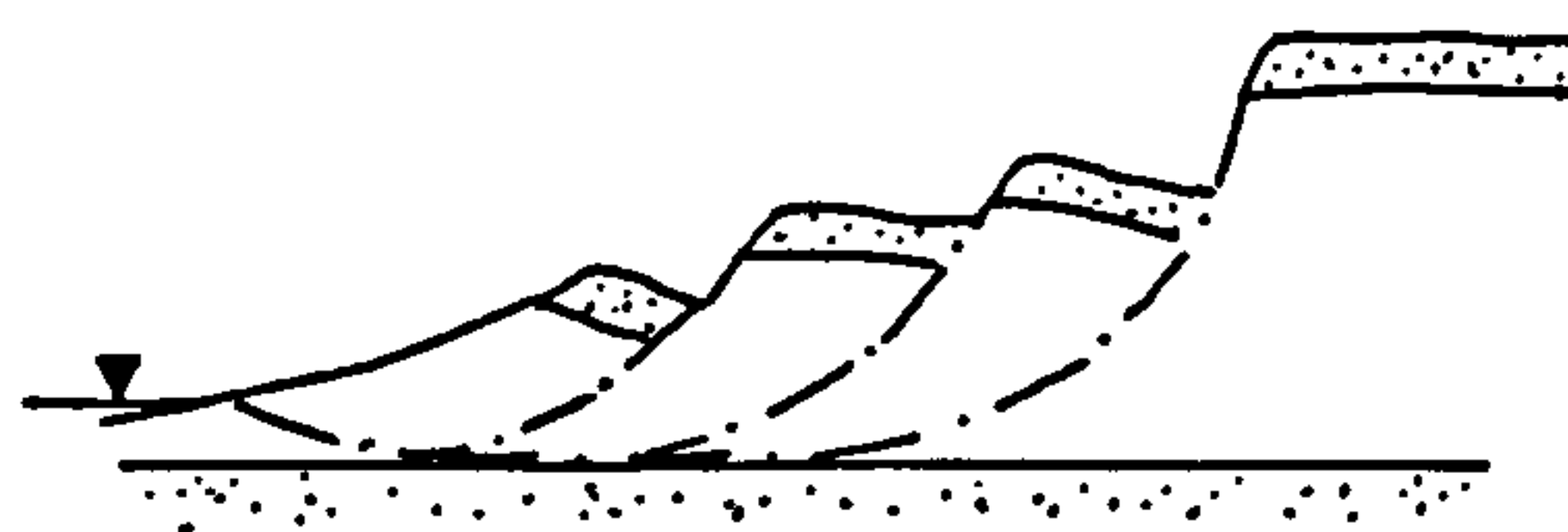


COMPOUND SLIPS

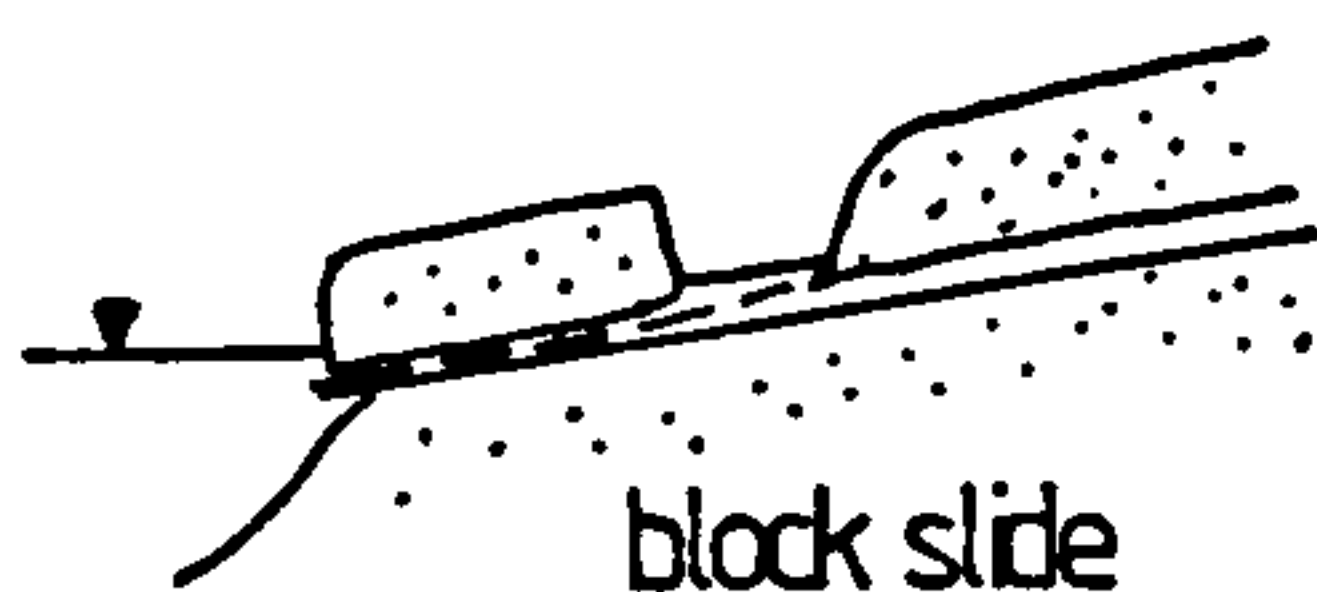
graben



MULTIPLE RETROGRESSIVE SLIPS



TRANSLATIONAL SLIPS



slab slide



MUDSLIDES

lobate



sheet or elongate lobe

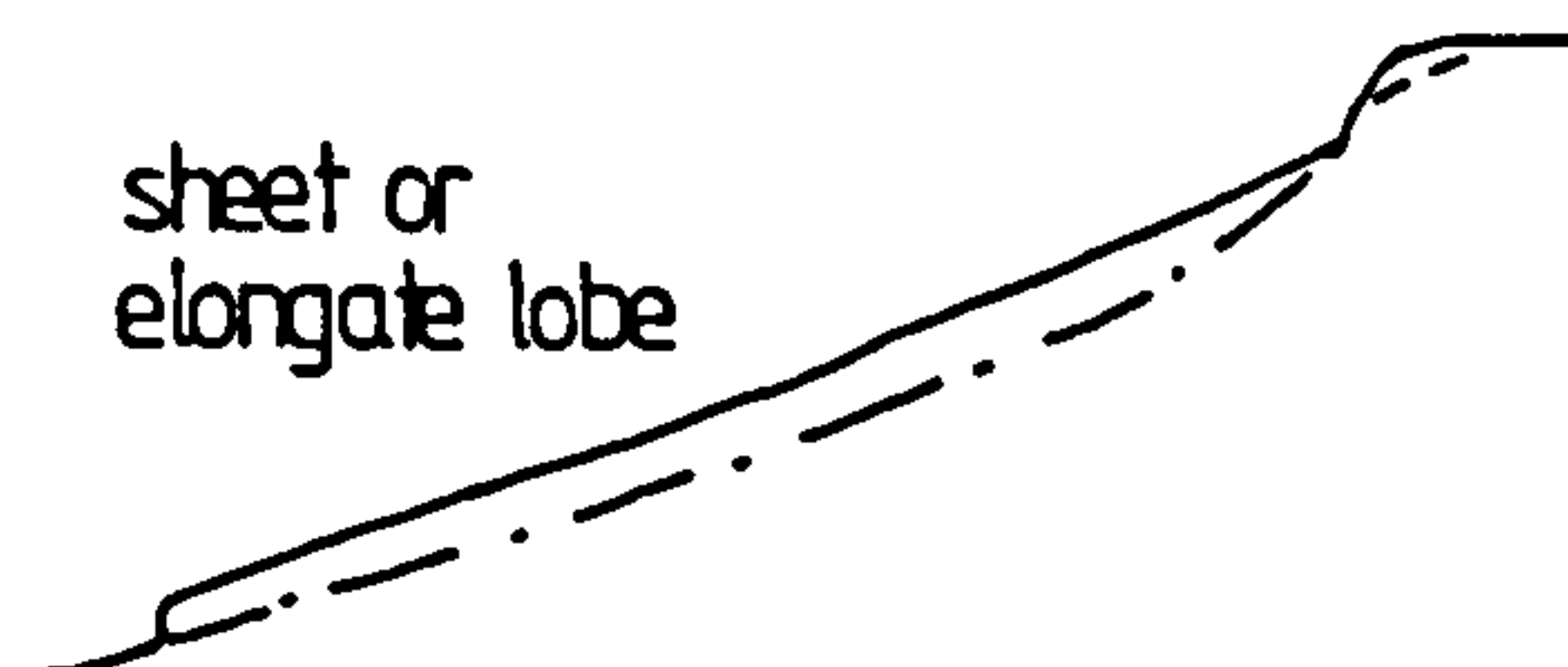


FIGURE 2.5

Basic and complex forms of mass movement on clay slopes
(after Skempton and Hutchinson, 1969 and Hutchinson 1988)

layer or the boundary of soil and rock. Distortion of the slide masses occurs at the boundary of the elements.

In compound slides in materials of low to medium brittleness the speed of failure is generally moderate. The rear part of the slide sinks down between internal shear surfaces and the rear scarp to form a graben. The middle part of the slide moves forward translationally. Compound slides may be divided into those with listric or bi-planar slip surfaces. In the more common listric type there are usually two or more internal rearward slip surfaces; in the bi-planar type only one is required (Hutchinson, 1988).

(iv) Translational slides occur where there is heterogeneity at shallow depth, eg. bedding, fault or shear zone. They may be block slides in hard and jointed material, or slab slides in weathered clay or colluvium. The ratio of depth to length is usually less than 0.1.

(v) Mudslides (formerly called mudflows) occur when argillaceous debris is softened by water and typically consist of fragments of debris in a soft clayey matrix. The boundary of the slide is generally formed by a discrete sheared failure surface. They are glacier like in form and have surface inclinations of 5° to 15° . They occur below bare slopes of fissured clay or by undermining of intact clays by seepage erosion from interbedded sand layers.

Mudslides are relatively slow moving and are commonly lobate or elongate in morphology. They generally have high clay contents and their movements are frequently highly seasonal. They are especially well developed in slopes of stiff fissured clay, and are generally bi-linear in long profile. Falls, shallow slides and mudslides feed debris from a steep backslope to a gently inclined front slope. Undrained loading by the debris can contribute to the forward movement of the slide (Hutchinson, 1988). In London Clay mudslides are best developed where the erosion is in balance with weathering and shallow mass movements, thus the slope undergoes parallel retreat (Hutchinson, 1973).

- (vi) Successive slips are an assembly of individual shallow rotational slips, and are typical of the late stages of free degradation on slopes of low inclination.
- (vii) Multiple retrogressive slips develop from single failures, followed by further retrogressive failures along a common basal slip plane. They may be of both rotational and translational type. Multiple rotational slides tend to occur on actively eroding slopes of high relief. The presence of a competent layer or cap-rock is important for their occurrence, as it inhibits degradation of the rear scarp.

Other types of failure which can occur in particular types of deposits are slides in colluvium of multiple translational type, spreading failures in varved clays and quick clay slides.

(viii) Classification of Clays

Many clays can be fitted into one of the following three groups:

- soft intact clays
- stiff intact clays
- stiff fissured clays

These groups can be further subdivided on the basis of their plasticity. Clays may also be classified by their origin such as:

- clays produced by rock weathering in situ
- sedimentary clays
- glacial clays
- periglacial clays
- clays transported by landsliding

The Holderness coast is formed mainly of glacial tills which are glacial clays and belong to the stiff intact clay group, and are of low plasticity.

(ix) Shear Strength

The basic shear strength properties of tills are discussed in a later section, but some general points are covered here. Both the peak and residual strength envelopes of a soil may be curved, that is the parameters C' and ϕ' are dependent on σ'_n . It is therefore desirable to test the soil over a range of effective normal stress likely to be encountered in the actual problem.

The failure criterion $S = C' + \sigma'_n \tan \phi'$ is two-dimensional and takes no account of the intermediate principal stress. The strength of soils in plane strain and in triaxial compression may be slightly different. A comparison of shear box and triaxial compression tests on Blue London Clay indicated that the shear box gave slightly higher results (Skempton and Hutchinson, 1969).

Anisotropy can affect both C' and ϕ' and the undrained strength of a clay. Tests on Blue London Clay showed a reduction in ϕ' of about 3° when sheared parallel to the bedding instead of at a steep inclination.

The following factors may all cause discrepancies between strengths measured in the laboratory and those operational in the field:

- sampling disturbance
- sample orientation
- sample size
- rate of shearing
- softening of fissured clays
- progressive failure

The effective stress parameters are less sensitive to disturbance than undrained strength and in stiff intact clays with little structure such as tills the effects of sample size and orientation are likely to be small. A back analysis of a failure in till at Selset showed that the effects of rate of shearing, softening, anisotropy and progressive failure are small (Skempton and Brown, 1961). This is further discussed in Section 2.4.2.

The applicability of different methods of analysis (ie. effective or total stress) to stability problems is discussed by Bishop and Bjerrum (1960). They conclude that the $\theta_u = 0$ (total stress) analysis is useful for an end of construction analysis of saturated clays subject to a rate of loading such that the dissipation of excess pore water pressure occurs. Where the pore water pressure is likely to increase with time an effective stress analysis must be carried out, using the parameters C' and ϕ' and measured or predicted values of pore water pressure. While in principle a $\theta_u = 0$

analysis could be used for a long term problem, the results have been shown to be highly unreliable.

The overall correlation between laboratory and field results using effective stress analyses is acceptable from a practical point of view with certain exceptions such as in stiff fissured or quick clays (Bishop and Bjerrum, 1960). However this correlation is based on relatively few cases of natural slopes in limiting equilibrium.

(x) Classification of Slope Movements

A classification of slope movements involving shearing by geotechnical considerations is given in Hutchinson (1988). The most important considerations are soil fabric and pore water pressure conditions on the slip surface. Slides are divided into either 'first-time' slides in previously unsheared ground or slides on pre-existing shears. They can be further divided into short-term (undrained) slides with no equalization of excess pore water pressures set up by changes in total stress, or long-term (drained) slides where there is complete equalization. A third category of intermediate lies between these two.

2.4.2 Shear Strength of Tills

Tills are generally well graded soils consisting of clay, silt, sand, gravel, cobbles and boulders. The grading is strongly influenced by the source of material and process of deposition. In the widespread lowland lodgement tills the shear strength is controlled by the fine fraction of the soil, as the coarse fraction generally forms less than 40% (Sladen and Wrigley, 1983).

The fine sandy silty clay fraction of lodgement tills is usually of low plasticity and low activity. Tills are generally overconsolidated deposits and the particles uncemented. The shear strength is a function of the composition, size, shape, distribution and orientation of the particles (Fookes et al., 1975). Undrained strengths may vary widely in the same deposit, indicating a non-uniform stress history.

The fabric of tills may show preferred orientation on a macro and micro scale owing to their mode of deposition. Discontinuities occur in some lodgement tills parallel to the till surface, or as vertical sets parallel to the direction of ice movement (Fookes et al., 1975).

An important distinction is the difference between peak and residual strength. Peak strength is the strength developed during first time shearing of a soil at small strains.

Residual strength is the strength developed during shearing at large strains and is lower than peak strength. The difference between peak and residual strength depends upon the grading, mineralogy, structure and degree of bonding of the unsheared soil (Vaughan and Walbancke, 1975). Soils showing a large difference are termed brittle and soils showing a small difference are non-brittle.

Lupini et al., (1981) postulated that there are three modes of shear:

- a turbulent mode where the soil contains a large proportion of rotund particles;
- a sliding mode where the soil contains a large proportion of platy low friction particles;
- a transitional mode between the above two.

In turbulent shear brittleness is low and is due to dilatancy of the soil, and the residual strength is dependent mainly on the shape and packing of the particles. In sliding shear brittleness is high and is due to formation of an orientated layer of clay particles along the failure surface; the residual strength is then a function of mineralogy, pore water chemistry and coefficient of friction between the particles.

Lodgement tills are of low plasticity and contain a high proportion of silt and sand sized particles. Therefore it is likely that they have a turbulent or transitional shearing

mode. This is concordant with measurements of residual strength in tills which show high residual strengths and low brittleness.

Vaughan and Walbancke (1975) give a plot of values of peak ϕ' against plasticity index for tills from Cow Green in Co. Durham and other sites. This shows a trend for decreasing ϕ' with increasing plasticity index, see Figure 2.6. Typical values are:

Plasticity Index (%)	ϕ'°
10	35
20	25

The effective cohesion intercept was estimated to be 10kN/m².

Other measurements of peak strength in tills are tabulated below:

Till locality	Plasticity Index (%)	Water Content (%)	ρ' (Mg/m ³)	C' (kN/m ²)	ϕ' ($^{\circ}$)
Selset, Yorks	13	12	2.2	8.5	32
Salford, Lancs	-	18	2.23	32	36
Taff Valley, S. Wales	-	9-17	1.8-2.1	0-40	28-42
Glasgow	-	13	2.15	9.5-24	28-32
Cromer, Norfolk	-	5-13	-	34-55	28-30

Figure 2.7 Table of peak strength measurements on tills (from Fookes et al., 1975).

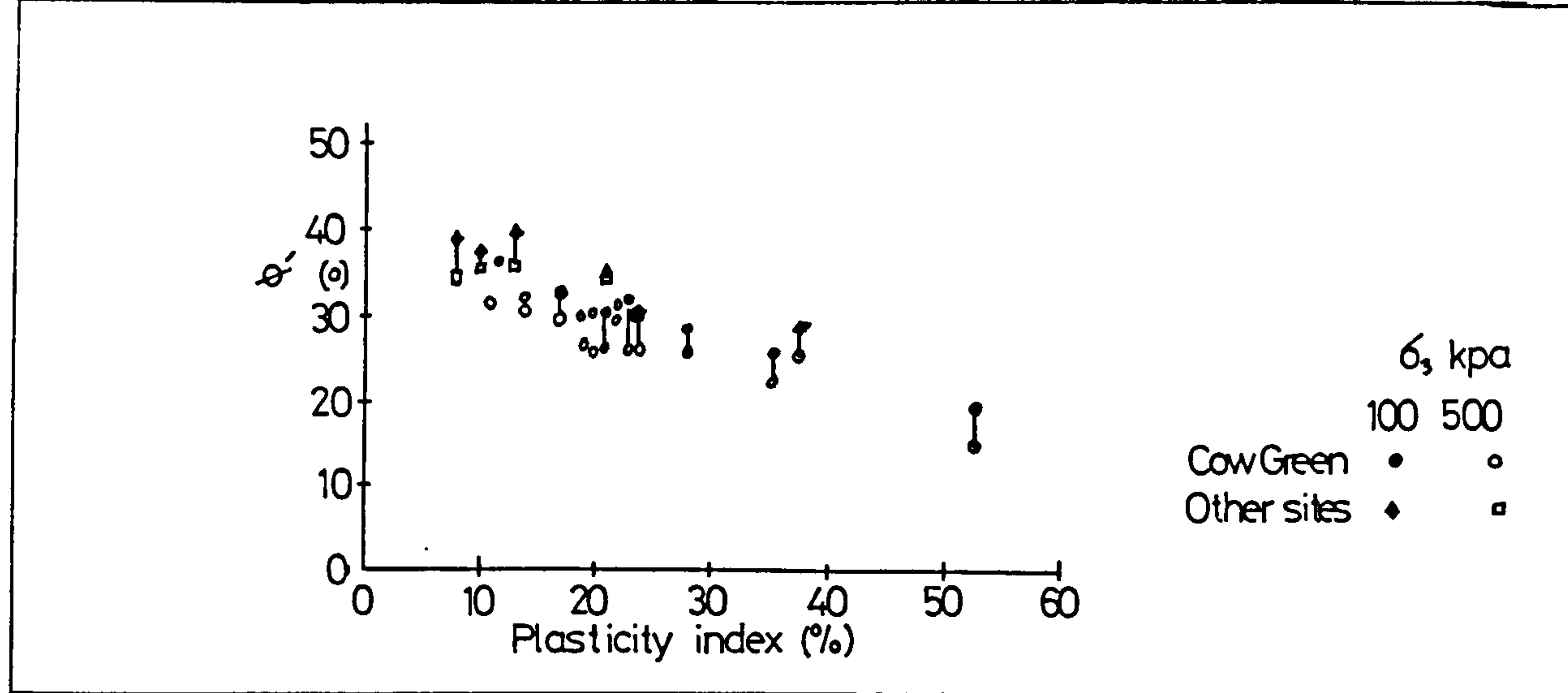


FIGURE 2.6 | Plot of peak ϕ' against plasticity index for tills (after Vaughan and Walbanke, 1975)

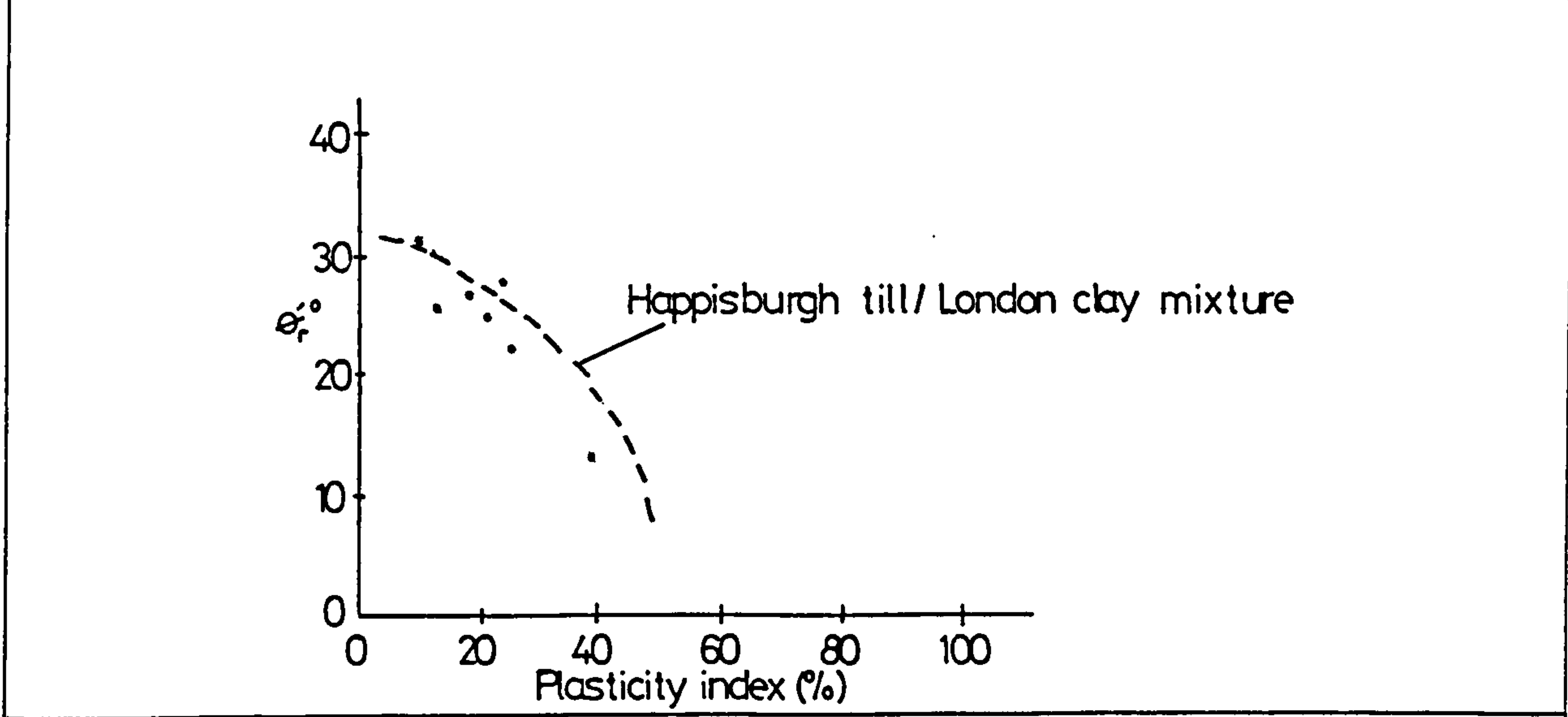


FIGURE 2.9 | Plot of residual ϕ'_r against plasticity index (after Lupini et al., 1981)

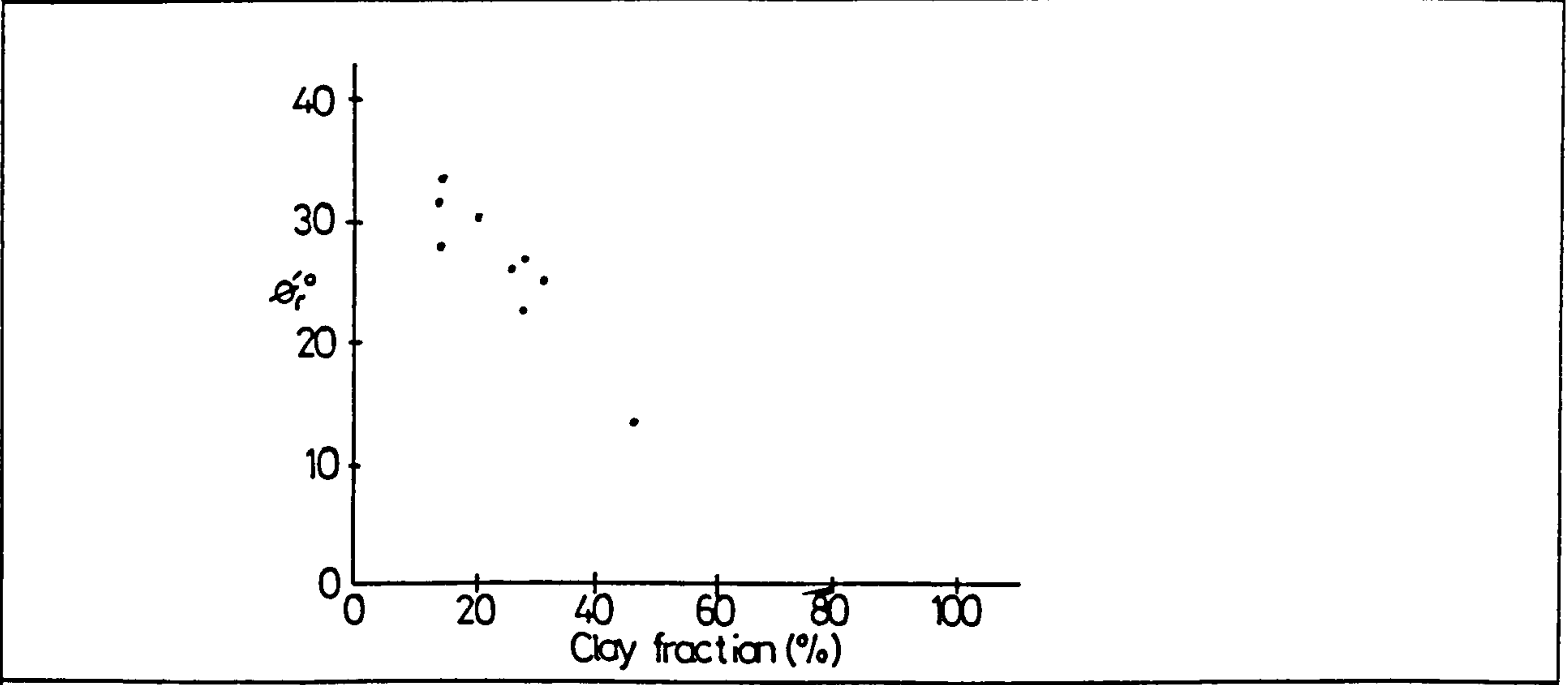


FIGURE 2.10 | Plot of residual ϕ'_r against clay fraction (after Lupini et al., 1981)

Lupini et al., (1981) gave a collection of residual strength measurements on various soils including tills. These include:

Till locality	Plasticity Index (%)	Clay fraction (<2 m)%	C'_r (kN/m ²)	ϕ'_r (°)
Bingley	13	26	1.5	25.3
Cowden,	18	28	5.8	23.8
Holderness	23	14	3.9	24.4
Perwortham				

Figure 2.8 Table of residual strength measurements on tills.

Plots of ϕ'_r versus plasticity index and clay fraction are given in Figures 2.9 and 2.10 from Lupini et al., (1981). These show a trend for decreasing ϕ'_r with increasing plasticity index and clay fraction. The values of ϕ'_r are all quite high, in the range 22° to 32°, showing that there is little reduction from the peak ϕ' range of 25° to 42°. This is in agreement with the non-brittle behaviour of tills discussed earlier.

The plot of τ / σ'_n against displacement for Cowden till in the ring shear apparatus shows that residual strength is reached at displacements of 100mm to 1m (Lupini et al, 1981).

Lodgement tills are typically stiff to very stiff in consistency with undrained shear strengths in the range of about 80kN/m² to over 500kN/m². Some typical values are given in the table overleaf:

Till location	Water Content (%)	C_u (kN/m ²)	Source
Hartlepool (upper till)	19-25	85-165	Fookes et al (1973)
Hartlepool (lower till)	10-16	245-575	"
Taff Valley	-	116	"
Selset	-	142	Skempton and Brown (1961)

Figure 2.11 Measurements of undrained shear strength of tills.

The undrained shear strength of tills is highly sensitive to changes in water content. A 2% change in water content can cause a 50% change in undrained strength (Vaughan and Walbancke, 1975).

The presence of fissures in a well graded sandy clay till will not usually affect the bulk shear strength unless they are at a critical orientation, and are coated with a material of higher plasticity and brittleness than the intact till. However there are some cases where sets of fissures in till reduce the bulk strength, as in a cutting at Hurlford, Ayrshire (McGown et al., 1974).

Skempton and Brown (1961) carried out a back analysis of a long-term slip of a river valley slope in till at Selset. The slope was 13m high and at an angle of 28°. A river undercut the bottom of the slope and periodic movement of the slip indicated that the factor of safety was very close to 1. Eight

sets of three triaxial tests were carried out on 1 inch (38mm) diameter and 3 inch (76mm) long specimens which gave consistent results. The average soil parameters were:

Liquid limit	26%
Plastic limit	13%
Plasticity Index	13%
Clay fraction	17%
Water content	12%
Cu	142kN/m ²
C'peak	8.5kN/m ²
ø'peak	32°

The back analysis indicated that near the full values of c' and ø' measured in the laboratory were operative in the field and that the c' component contributed 30% of the shear strength. This is in contrast with the behaviour of stiff fissured clays where the long term c' component is usually zero.

The work showed that the effects of rate of shearing, progressive failure, and anisotropy in tills are small, and that a high proportion of the peak drained strength measured on small samples can be used in analysis (Vaughan and Walbancke, 1975).

2.4.3 Stability of Slopes in Till

Vaughan and Walbancke (1975) in a review of the stability of cut slopes in boulder clay (till), listed some of the factors that govern slope behaviour as the drained and undrained

strength, consolidation or swelling characteristics and permeability. In lodgement tills, permeability and the rate of consolidation and swelling are generally low, therefore undrained conditions occur during rapid slope formation (by excavation), and drained conditions govern long term behaviour.

Bishop and Bjerrum (1960) show that pore water pressure is temporarily reduced in clay soil beneath an excavation or unloaded natural slope. In a fully saturated soil ($B=1$) the expression for change in pore water pressure is:

$$\Delta U = \frac{(\Delta \sigma_1 + \Delta \sigma_3)}{2} + (A - \frac{1}{2}) (\Delta \sigma_1 - \Delta \sigma_3)$$

Thus the changes in mean principal stress and shear stress both contribute to the reduction, provided $A < \frac{1}{2}$.

The changes in pore pressure and factor of safety of a cut in clay are illustrated in Figure 2.12 after Bishop and Bjerrum (1960). The pore water pressure initially falls during rapid excavation; the magnitude of the fall depending on the value of the parameter A . There is then a period of pore pressure redistribution during which time the pore pressure rises. During this period the pore pressure is said to be depressed relative to its final equilibrium level¹. The factor of safety

1. Throughout the thesis the term 'depressed' water pressure will be taken to mean pore water pressure below equilibrium value as a result of stress relief.

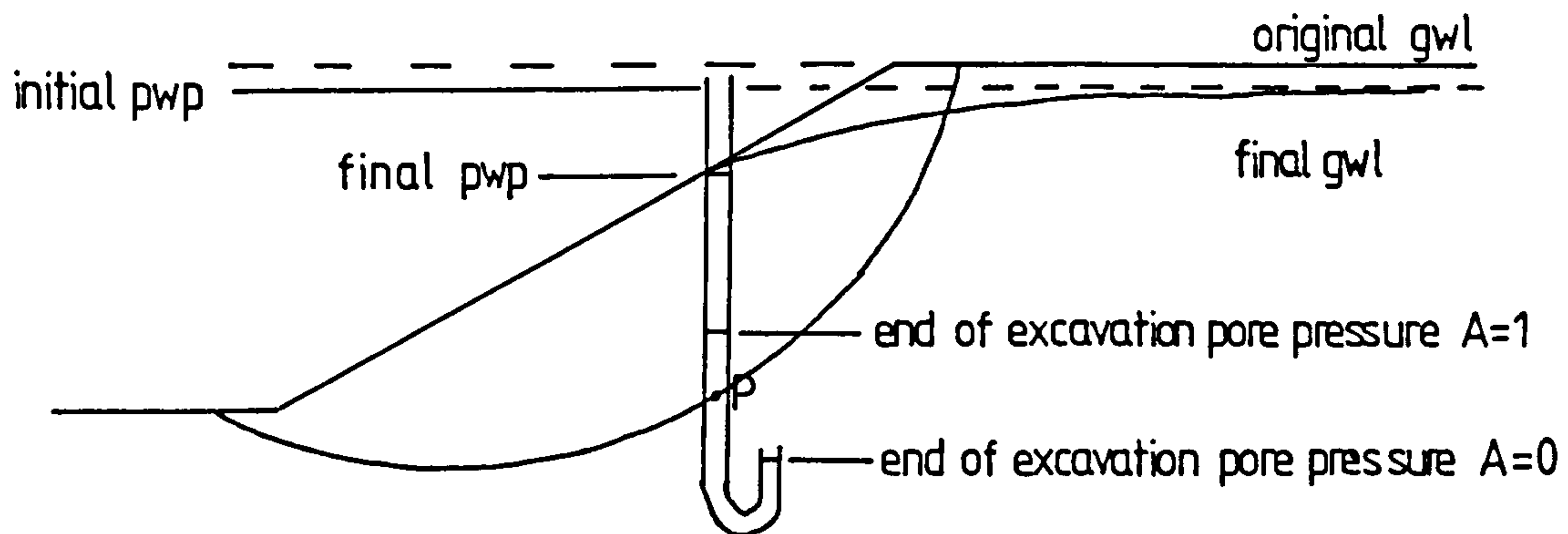
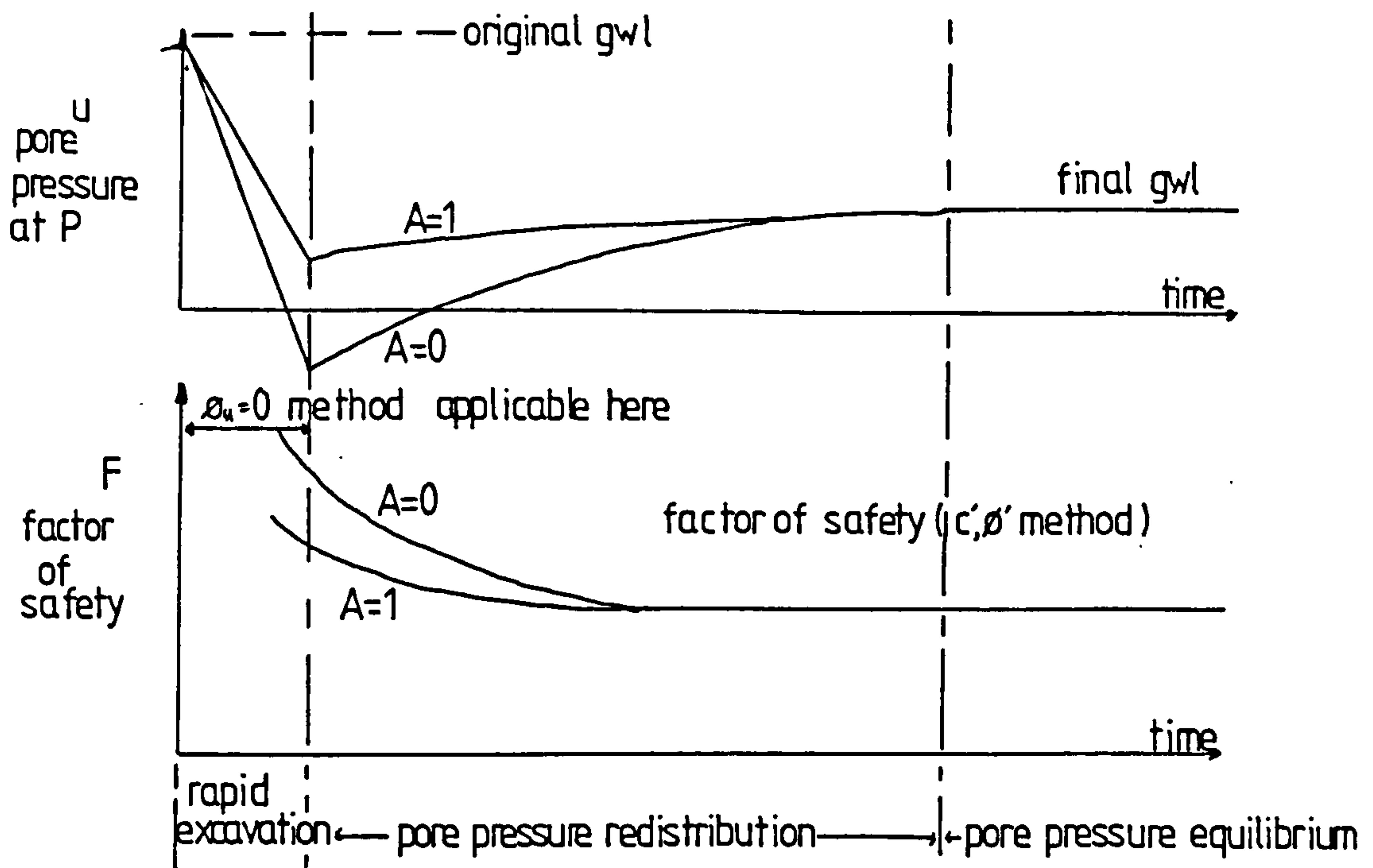


FIGURE 2.12

Changes in pore pressure and factor of safety during formation of a cut slope in clay (after Bishop and Bjerrum 1960)

decreases with time and is at its lowest value in the long term when pore pressure has equilibrated.

There are significant differences between the shear behaviour of sandy clays of low to medium plasticity, and clays of higher plasticity. Well graded sandy clay lodgement tills have a high drained strength and are non-brittle; clays of higher plasticity (P.I. > 20-25%, eg. Holderness "stone-free" silty clays (see Section 2.2.2)) may show brittle behaviour and a low residual strength (Vaughan and Walbancke, 1975). In non-brittle tills, failures are often shallow and the initial displacements of slips are small, as a small change in geometry is sufficient to restore stability. In tills of higher plasticity and brittleness, slope failures are more deep-seated and rotational in type.

2.4.4 Mechanics of Erosion of Clay Cliffs and Foreshores

Erosion of clay cliffs and foreshores occurs chiefly by physical processes, although in some cases biological agents are important as well. The susceptibility of clay soils to erosion depends upon their material properties such as mineralogy, undrained shear strength, clay content and degree of cementation, and properties of the mass such as fissuring, presence of erratics and lenses of other soil types, eg. sand. In coastal cliffs and foreshores, physical changes which occur as

the cliff recedes can change susceptibility of the clay to the agents of erosion.

In a study of erosion of till in the nearshore zone of Lake Ontario by Davidson-Arnott (1986), it was found that abrasion and fluid stressing were the chief mechanisms of erosion. Abrasion of the till surface occurred in shallow water where beach sand and gravel is mobilized by shoaling and breaking waves with high orbital water velocities. Fluid stressing leading to removal of particles of till may also occur in shallow water.

Results obtained by Zeman (1983) indicate that the critical shear stress required for erosion of particles of a till from Lake Erie is 10-20 Pa. However Davidson-Arnott (1986) estimates that the 2m high storm waves on the Lake Ontario coast produce a shear stress of 1-2 Pa at the bed. This is therefore insufficient to cause erosion unless the till has been softened by other processes. Patches of soft till, generally less than 2cm thick were found on the nearshore bed of Lake Ontario by Davidson-Arnott (1986).

The rates of vertical erosion of the till bed measured by Davidson-Arnott (1986) were greater than 30mm/yr at 3m water depth decreasing to 11m/yr at 6m water depth. This is because at greater depths the wave orbital velocities are lower and there is less sand and gravel to cause abrasion. The rate of

erosion at depths less than 3m is probably higher than 30mm/year although no measurements were made.

Another mechanism by which clay soils can be eroded is by detachment of blocks bounded by joints or other discontinuities subject to high water or air pressures generated by wave impact at the toe of cliffs. Surface blocks removed in this way tend to leave sharp angular profiles. These have been described by Hutchinson (1986) in London Clay cliffs, but are uncommon on the Holderness coast where there are relatively few discontinuities in the tills.

Clay soils on eroding shorelines may be made more susceptible to erosion by a number of processes which have been reviewed by Hutchinson (1986) and are outlined below. The erosion mechanisms of abrasion, block detachment and fluid stressing described above will then become more effective.

Swelling of the clay in the foreshore and at the toe of cliffs occurs due to unloading as the overlying clay is eroded. This results in an increase in water content and softening of the clay. At a site in Cowden, Holderness, Hutchinson (1986) reports that average water contents of till in the shore platform are about 19%, which when compared with water contents of 15% to 16% at a similar horizon inland shows that swelling has occurred.

Alternative desiccation and wetting of clay foreshores weakens the surface layer. The effect of desiccation is to induce high internal suctions, causing shrinkage of the clay and the formation of cracks. Hutchinson (1986) observed that flakes of London clay about 1mm in thickness and 5 to 10mm or less across were associated with desiccation cracks in the foreshore of the Isle of Sheppey. These flakes were then easily removed by the sea. Similar phenomena have been observed by Robinson (1977b) in Lias shales on the North Yorkshire coast.

Freezing of the soil forming the shore platform and cliff face may also weaken the surface layer of the soil. Freezing occurs intermittently in Britain but can still be an important process. Harris and Ralph (1980) report that London Clay on the foreshore at Clacton, Essex was disintegrated by frost during the hard winter of 1962/63 leading to a lowering of the surface by 300mm, in a few weeks. Freezing of the surface soil will lead to an increase in its water content and may cause detachment of flakes of soil in a similar manner to desiccation. Hutchinson (1986) observed that a 30mm thick layer of softened till was present on many Holderness lower cliff faces during January 1986, which may have been softened by frost heaving.

The influence of minor geological details on the erosion of cohesive shore platforms has been described by Hutchinson (1986). In tills on the Holderness coast glacial erratics and lithological contrasts act as concentrators of erosion.

Scouring occurs around erratics exposed in the foreshore leading to the development of depressions and pot holes.

Layers of sand and silt within the clay tills at Holderness are more easily eroded than the tills. This leads to the development of notches at the toe of the cliff which can undercut it by 0.1m to 0.3m and lead to a fall of the material above (see Section 2.5.3).

Small caves were observed in the foot of the Holderness cliffs by Hutchinson (1986). One of these near Aldbrough was 3.7m deep and had an oval cross-section about 1.7m wide and 0.9m high. The caves appeared to have originated in a more granular layer and could potentially lead to a large fall of clay.

The presence of boring organisms may be significant in some cases in weakening the upper part of shore platforms. Hutchinson (1986) observed high densities of crustaceans in the uppermost part of London Clay samples from the Isle of Sheppey.

The salinity of sea water on coastal shores will probably tend to increase the erosion resistance of clays compared with clays on fresh water shores, such as the Great Lakes. This effect should be particularly strong in active clays such as the London Clay. This is because the presence of dissolved ions in the sea water will tend to reduce the size of the radius of influence of the double layers around clay particles, thus

increasing the net attractive forces and the degree of aggregation of the particles (Hutchinson, 1986).

2.4.5 Studies of Coastal Cliffs and Excavations in Tills and Other Materials

Hutchinson (1983) identified the main features which may occur in coastal slopes as:

- toe erosion, sometimes causing notching;
- fluctuations in external water loads exerted on the cliff toe;
- seasonal variations in pore water pressure within the cliff;
- rising pore water pressures associated with swelling following earlier undrained unloading of the cliff toe;
- undrained loadings on the headward parts of the slope;
- transient pore water pressure fluctuations in the toe area deriving from the cyclical or surge affected tide levels.

Examples of some of these are described in this and the following sections.

Several studies have been made in the London Clay of cliff recession, excavation and the effects of toe erosion. While London clay is fissured and has a higher degree of over-consolidation than the intact Holderness tills, the findings may have some relevance to the behaviour of the Holderness cliffs. Hutchinson (1973) showed how the intensity of toe erosion controlled the modes of cliff failure in London Clay, and identified three categories of behaviour summarized as:

1. Type 1, where the removal of debris from the toe is in balance with the rate at which it can be supplied by mass movements on the slope.
2. Type 2, where the rate of debris removal is greater than the rate at which it is supplied by mudslides and deep-seated sliding.
3. Type 3, where there is no removal of debris and the cliffs undergo the process of free degradation.

The Holderness cliffs mainly belong to the Type 2 category, exhibiting cyclic deep-seated rotational landslides (Hutchinson, 1986).

Studies of pore water pressure distribution adjacent to excavations and in coastal cliffs have shown the effects of undrained unloading. Vaughan and Walbancke (1973), observed a depression (from the estimated long-term equilibrium value) in

pore water pressure beneath a motorway cutting in London clay, nine years after its formation. They suggested that the slow equilibration of excess pore water pressure may be the primary cause of delayed failure in cut slopes. Bromhead and Dixon (1984) also observed a significant depression in pore water pressure, attributed to stress relief induced suctions, in London Clay cliffs at Warden Point, Isle of Sheppey. They concluded that the slide behaviour was dominated by the presence of these suctions. Equilibration rates were shown to be slow by the measurement of suctions in the foreshore, which was first exposed over 50 years ago. Pore water pressures were also depressed in the body of the slope and behind the cliff crest.

The equilibration rate will be governed by the coefficient of swelling and drainage geometry. In the Holderness cliffs the time for swelling may be shortened by more permeable granular layers in the till (Hutchinson, 1986). The pore water pressure distribution in a slope may also be strongly influenced by underdrainage into more permeable underlying beds. Examples of this occur in London Clay cliffs at Herne Bay, and in a till slope at Cow Green dam (Bromhead, 1986). A 'bowed' pore water pressure distribution occurs, with low pore water pressure at depth, owing to a decrease in permeability with depth.

The points discussed above were considered in planning the site investigation, in particular the possibility of depressed pore water pressure or underdrainage occurring in the Holderness

cliffs. The distribution of soil types at the sites was carefully examined, and laboratory tests conducted to determine the stress-strain and strength characteristics of the tills.

2.4.6 Slope Stability and Erosion of Lake Erie North Shore Cliffs

The behaviour of cliffs on the north shore of Lake Erie in the Great Lakes has recently been studied in response to a need for shore protection. Some of the literature is reviewed here as a contrast to the Holderness cliffs.

The Pleistocene deposits forming the north shore of Lake Erie are variable, and include the following: lodgement till, water laid till, silts, sands and gravels (Quigley et al., 1977; Quigley and Zeman, 1980). A major difference between the Lake Erie shoreline and the Holderness coast is that lake levels show long-term cyclic variations of up to 1m over irregular periods of between 7 and 25 years. This is shown to influence the slope behaviour considerably (Quigley and Di Nardo, 1980).

The Great Lakes are freshwater bodies, and this may make their clayey shores more susceptible to erosion for a given degree of attack than saline shores (Hutchinson, 1986). The average annual input of wave energy to the coast varies between about $15,800 \text{ MJm}^{-1}$ and $63,000 \text{ MJm}^{-1}$ depending on the length of wave fetch (Quigley et al., 1977). This is comparable with a value

for the Holderness coast at Withernsea during 1969/70 of 33,293 MJm⁻¹ (Pringle, 1985).

Quigley et al., (1977) describe the slope behaviour at several study sites on the north shore of Lake Erie. They identify several failure modes dependent on the geology, cliff height and incident wave energy which are discussed below.

The cliffs at Patrick Point are 25-30m high and are receding at 0.6m/year. They are composed of lodgement till overlain by a layer of silty sand and water laid till. The cliffs are receding at a near constant profile by shallow surface landsliding, sloughing and toppling, and are in equilibrium with the rate of toe erosion. They thus exhibit Type 1 behaviour. Deep-seated slides are inhibited by a layer of strong till. An important mechanism of erosion is alternate desiccation and wetting, which can increase the water content from between 18%-25% to over 40%. This leads to the development of a thin flow slide, adding material to a central 'mud-stream'. Desiccation can also improve the stability of the slope face, leading to toppling failures of prismatic blocks of till rather than sliding.

At Iona, further to the west of Patrick Point, the rate of erosion is about 1.4m/year as the cliffs are less sheltered for South-West winds. The cliffs are composed of lodgement and water laid tills with a 3m thick silty sand layer at mid height. There are two different failure modes: instability of

the upper part of the cliff due to the presence of the sand layer, and cyclic deep seated failure occurring over an estimated 10-25 year period. The mean slope angle varies between 25° and 40° during the cycle. The cliffs exhibit Type 2 behaviour.

The process of swelling and softening due to a reduction in effective stress caused by unloading is believed to account for the occurrence of a large deep seated failure at Port Bruce (Quigley and Di Nardo, 1980). The 45m high cliffs consist of very stiff to hard clay till overlain by sands, and are receding at about 2.2m/year, normally by sloughing and small circular arc failures in the clifftop.

A large deep seated landslide began in 1964 following a long period of low lake levels. It was 200m long and extended about 40m back from the cliff edge. Stability analyses showed that although toe erosion caused by rising lake levels triggered the failure, significant softening must have occurred within the slope. However no direct measurements of rising pore water pressures associated with this phenomenon were made. During periods of high lake levels and active toe erosion it is postulated that there is insufficient time for softening to occur, which explains why deep seated slope failures do not occur during these periods.

The high susceptibility of silt sized soils to erosion is thought to be responsible for a good correlation of erosion

rate with incident wave power in the western part of Lake Erie (Quigley and Zeman, 1980).

A layer of dense silty fine sand at the foot of cliffs composed of tills and silts near Port Stanley in mid-Lake Erie is very easily eroded. This layer is usually protected by a large debris fan of material from high velocity slides. After this debris fan is removed by erosion, which can take several years, the cliff is rapidly undercut as the sand is washed out and a large slide again occurs (Quigley et al., 1977).

A comparison of the Holderness cliffs with the behaviour described above is given at the appropriate place in Chapters 3 and 5.

2.4.7 Seismicity

The United Kingdom and North Sea are situated far from plate boundaries and have a relatively low level of seismic activity. However historical records show that minor earthquakes with a magnitude of up to about 6 occur quite frequently (every few years) in the North Sea (Woo and Muir Wood, 1986). The records show patterns of seismicity which can be explained by the regional tectonic framework. The North Sea can be divided into two seismic and one aseismic zones:

1. the West Scandinavian seismic province where the crust is rebounding after melting of the ice cover;
2. a central aseismic zone running NW-SE where the crust is subsiding and stable, and the lithosphere is thin;
3. the England-Rhine seismic province undergoing long-term regional uplift.

The Holderness coast is located in the England-Rhine seismic province, but close to the boundary of the central aseismic zone.

Instrumental data reveals a concentration of seismic activity in three areas which correlate with the historical records: the northern North Sea and Skaggerak in the West Scandinavian seismic province and the Dogger Bank in the England Rhine seismic province.

A catalogue of earthquakes in the North Sea between 1834 and 1983 (Woo and Muir Wood, 1986) shows that of the five largest earthquakes, four occurred in the West Scandinavian seismic province and one occurred in the Dogger Bank. The Dogger Bank earthquake of the 7 June 1931 had the largest magnitude of all in the catalogue. The surface wave magnitude was 5.6 and the maximum observed intensity on land (on the Holderness coast) on the Modified Mercalli Intensity scale was 6.0. The earthquake was felt over the whole of England and much of Scotland,

Holland, Belgium, Denmark and Western Norway. The epicentre of the earthquake was at latitude 54.1° and longitude 1.5° , approximately 125km from the Holderness coast.

There is little documentary evidence on the effects of the 6 June 1931 earthquake on the Holderness cliffs, although there are newspaper reports of damage in the town of Bridlington. These describe chimney stacks and roofs collapsed and the swaying of telegraph poles and a church spire. Falls of rock occurred in Scarborough (Matthewman, 1931). Elsewhere in Britain, cliff falls occurred in Orkney and the Isle of Wight, and a large landslide occurred at Mundesley, Norfolk (Hutchinson, 1983).

Seismic hazard maps of the North Sea have been prepared based on historical and instrumental records, and geological information (Woo and Muir Wood, 1986). These show that for the Holderness coast the peak ground accelerations which can be expected with various annual probabilities are:

Annual probability	%g
0.02	3
0.01	4
0.0001	25

For an earthquake to cause major slope failures it is likely that peak ground accelerations would have to be between 10%g and 25%g, for which the annual probability on the Holderness coast is low (1 in 10,000). The peak ground accelerations of

3%g to 4%g which are more probable would be expected to cause only minor falls and slides.

2.4.8 Methods of Cliff Stabilization

Hutchinson (1983) outlined the main options for cliff stabilization as:

1. to do nothing. This option may be chosen for scenic or scientific reasons or where cliff retreat contributes to the littoral sediment budget.
2. to build offshore breakwaters. This is rarely done owing to expense.
3. to build forward in the form of beach replenishment or toe weighting.
4. to attempt to maintain the status quo.
5. to fall back and designate a cliff top strip as a hazard zone.

As stated in Section 1.3, options (2) and (3) are under consideration for the Holderness coast.

There are two components to stabilization of coastal cliffs, which are firstly to halt marine erosion and secondly to stabilize the cliff slopes (Hutchinson, 1983). The first can be carried out by construction of a wall or other barrier across the toe of the slope. This should be as flexible as possible to avoid damage by possible future landslides. The second may only be necessary if structures are sited on the cliff slopes, or if cliff top installations are threatened by recession of the cliff crest.

The main methods of cliff stabilization are by modifying the slope profile, drainage, prevention of seepage erosion and the construction of retaining structures. Modification of the slope profile can be done by toe weighting, flattening of the slope or by unloading of the head of the slide.

The concept of influence lines (Hutchinson, 1984) can be useful in guiding the most effective location of cuts and fills, particularly in slides on pre-existing slip surfaces. These influence lines define the effects of positive or negative vertical loadings on the factor of safety of a slope for a given slip surface under undrained, intermediate or drained conditions. Neutral lines exist which define the boundaries between areas for which fills or cuts improve or reduce the factor of safety under the given conditions.

The preferred method of slope modification is by toe weighting which can improve the stability of all slides in a complex

failure. Cuts at the head of a slide can undermine the back scarp and may only benefit one slip.

The effects of toe erosion in coastal slopes can be examined with the influence line concept. Toe erosion is equivalent to an excavation which in general leads to a reduction in stability. A failure of a slope in the Lower Lias at Lyme Regis occurred when its toe was regraded (Hutchinson, 1984). Another example is given by Bromhead (1986), where a slide occurred in London Clay in South London following the removal of about a metre of fill, which activated a slide on pre-existing shear surfaces.

Trench drains and herringbone drains can be used to reduce pore water pressures in landslides of small to moderate depth in clay slopes. In deeper slides bored drains with filters or drainage galleries are required. Precautions should be taken against clogging and the effectiveness of the scheme should be monitored (Hutchinson, 1983).

2.5 Geotechnical Investigations of the Holderness Cliffs

2.5.1 Field Investigations of the Cliffs

There have been two major geotechnical investigations and several minor surveys of the Holderness cliffs prior to this study. The Building Research Establishment (B.R.E.) has an

instrumented site on the cliffs at Cowden (TA 252 404) set up in 1984, and a test-bed site 800m inland at Cowden, established in 1976. The B.R.E. study of the cliffs was undertaken to determine the in situ engineering parameters mobilised, for comparison with field and laboratory results from the test-bed site. The study has included measurement of pore water pressure distribution, ground movements adjacent to the cliff, and in situ stresses (Butcher, 1986). The pore water pressure distribution is influenced by underdrainage to sand layers and perhaps to the Chalk, and low pore water pressures occur to at least 20m depth, adjacent to and remote from the cliff. The in situ stresses are reduced near to the cliff, in the direction at right angles to the cliffline, particularly between ground level and 4m depth. They are further discussed in Section 4.3.8.

Hull University has undertaken a study of beach processes and cliff recession as part of the coast protection project. The study of cliff recession (Richards and Lorriman, 1987) has been at Easington in southern Holderness. The study showed that the process of cliff recession has two components:

1. toe erosion, which undercuts and steepens the intact cliff, and removes failure debris;
2. a range of mass movements, chiefly deep-seated rotational slips, mudslides and falls.

These components are discussed separately in subsequent sections.

2.5.2 Erosion of the Holderness Cliff Toe and Till Shore Platform

A review of erosion mechanisms of cohesive cliffs and shores is given by Hutchinson (1986), including observations on the Holderness tills, and some points are summarized below. The shallow, outer skin of the cliff or shore platform is where erosion occurs. This outer skin may be weakened by swelling at low effective stresses, and its erosion resistance diminished by sub-aerial agencies. The principal types of erosion are corrasion, detachment of blocks on discontinuities, and detachment of flakes by sub-aerial agencies, eg. desiccation or freeze/thaw. On the Holderness coast waves may often carry suspended sand, encouraging corrasion. Discontinuities are rare, but minor geological details such as sedimentary and sheared contacts are exploited by erosion. The cliff toe is more prone to erosion than the shore platform, because of the opening of joints and fissures, softening and greater exposure to sub-aerial agencies. Thus the overall rate of recession of the coast is fundamentally controlled by the rate of shore platform erosion.

2.5.3 Modes of Cliff Failure (Mass Movements)

The mass movements which constitute cliff failure may be grouped into three main types: falls, flows and slides (Bromhead, 1986). Of these, falls and slides predominate on the Holderness coast and Gilroy (1981), recognized that their fundamental cause is continuing marine erosion at the cliff foot. Barrow (1985) observed that sliding is the chief form of mass movement, mainly in the form of rotational slips. Mudslides activated by surface and drain run-off water involve smaller volumes of till. In areas of low cliff height, falls are the predominant mode of failure (Davison, 1985). Falls are caused by undercutting of the cliff, which occurs either by seepage erosion from sand layers, or by marine erosion of the cliff foot (often aided by the occurrence of less resistant soil layers).

Hutchinson (1986) described a process leading to falls at the cliffs near Cowden. Rounded notches undercutting the cliffs were seen, some of which exploited thin sub-horizontal layers of silt or sand. It was inferred that formation of the notch had caused a shell-shaped tension failure in the superadjacent till, which in turn led to the development of a near vertical tension crack in the till upslope. Eventually it was expected that a fall would occur by shearing between the base of the crack and the notch. Pringle (1985) observed that the blocks of till from a fall lie on the foreshore and are rounded by wave action. Pebbles become embedded in their surface forming

"armoured mudballs". These are rapidly destroyed by wave action and so are only found where severe toe erosion is currently occurring.

Three types of slip were found to occur by Gilroy (1981) and Hutchinson (1986), distinguished by the position of emergence of the slip toe. These are:

1. slope failures, with the toe emerging in the cliff face;
2. toe failures, with the toe emerging at or near the cliff foot;
3. base failures, with the toe emerging in the till shore platform.

In a study of the cliffs near Easington (Richards and Lorriman, 1987) toe failures were found to be the most common. Slope failures were found to occur where the position of the slip surface was influenced by the junction between Skipsea and Withernsea tills, or weathered and unweathered tills. A prominent example of base failure at Grimston Hall (TA 289 351) has attracted the attention of several workers (Gilroy, 1981; Williams, 1982; Hutchinson, 1986). The slip has a highly unusual morphology, reflecting the shape of a stone-free clay layer (probably of the origin described in Section 2.2.2) underlying it. A study of base failures near Aldbrough and Ringbrough (Wiggin, 1987) has shown that many of these are also

influenced by the presence of stone-free clay layers. The base failures are deep-seated, often multiple, rotational slips. The shape of the slip surface of all failure types is non-circular, with a steep back portion and flat sole (Williams, 1982). In a study of cliff stability Richards and Lorriman (1987) showed that the following factors may trigger failure:

1. increasing cliff height (by lowering of the beach);
2. steepening slope angle (by toe erosion);
3. increasing pore water pressure;
4. reducing shear strength in the weathered zone.

The evidence reviewed suggests that variations in cliff geology, in particular of minor soil types, controls the occurrence of the various modes of failure and mass movements. Thus in the general survey particular attention was given to mapping them and examining their influence.

2.5.4 Geotechnical Properties of the Holderness Tills

There have been a number of field and laboratory investigations of the geotechnical properties of the Holderness tills. Marsland and Powell (1985) showed that the lodgement tills at Cowden are clay matrix dominated (~30% clay), with the clay

minerals of mica (probably illite), kaolinite, chlorite and vermiculite. The shear and index properties were found to be highly consistent, and showed no difference between Skipsea and Withernsea tills. A micro-fabric of shears was observed in thin-section, forming part of an overall pattern, although little macro-fabric was present. Close agreement was found between undrained strengths from laboratory tests and large diameter plate load tests. The undrained strength varied from about 100kN/m^2 at 6m depth to 300kN/m^2 at 24m depth.

Triaxial tests showed that the stress-strain behaviour was non-brittle and that the tills are dilatant. Stress paths from undrained compression and tensile triaxial tests on consolidated pushed samples are shown on Figure 2.13. Several types of test to determine the effective stress strength parameters of the till at Cowden were carried out by Marsland and Powell (1985) and the results are summarized in Figure 2.14.

	Details of Tests	$c' (\text{kN/m}^2)$	ϕ'°
Triaxial Tests	Consolidated undrained	0	26.7
	Consolidated drained	0	26.7
	Unconsolidated undrained	0	26.7
Simple Shear Tests	Drained and undrained	11	26
Direct Shear Tests	Drained peak	6	27
	residual	0	25
Ring Shear Tests	Drained	0	23-27

Figure 2.14 Effective stress strength parameters for tills at Cowden after Marsland and Powell (1985).

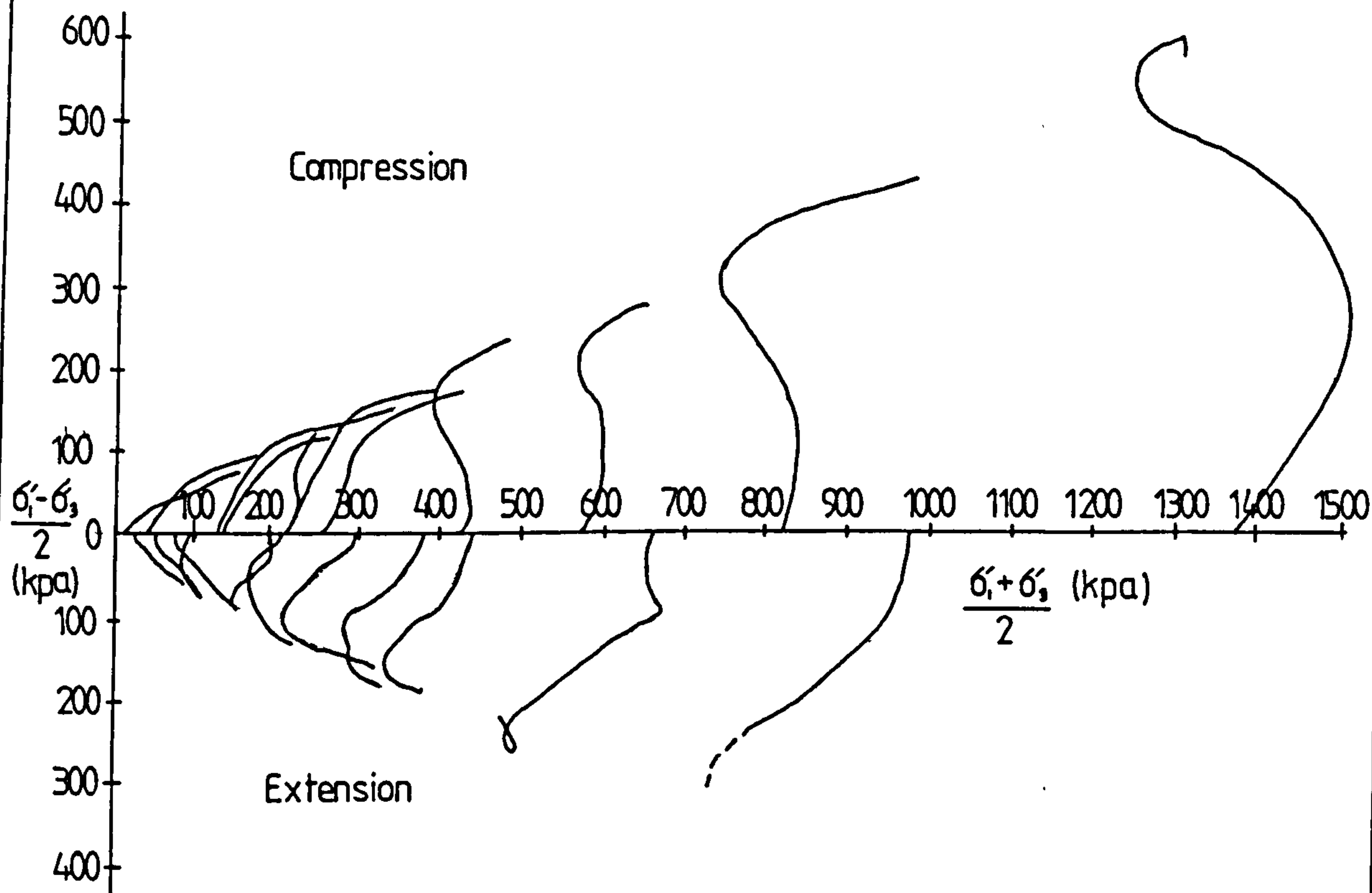


Figure 2.13

Stress paths from undrained compression and tensile triaxial tests on consolidated pushed samples (after Marsland and Powell, 1985)

The results from the various types of test are consistent and indicate a linear failure envelope with c' of near zero. As the samples tested came from different parts of the till sequence, the results also show that there is little variability in the tills.

The residual strength of the tills is high and close to the peak strength, thus the till is non-brittle in behaviour. The residual strength is further discussed in Section 5.3.3.

Measurements of preconsolidation pressure by Foster (1985) showed that the tills are moderately overconsolidated. In a review of the effects of weathering on the tills, Gilroy (1981) summarized the main changes as:

1. oxidation of pyrite and siderite, changing the colour to reddish brown;
2. formation of prismatic gleyed fissures;
3. leaching of primary carbonate;
4. increased clay content;
5. softening and disaggregation of gravel sized clasts.

However laboratory tests showed that the changes in effective stress strength parameters were small. Yeo (1986) showed that

the stress path followed to failure by the tills can influence the effective stress strength parameters, and this work is further discussed in Section 4.6.

2.6 Summary and Main Research Questions

2.6.1 Summary

The Holderness cliffs are composed mainly of lodgement tills with smaller amounts of other soil types. These include sands and gravels, silts and laminated clays, and are significant because of their lower erosion resistance and different shear behaviour and strength properties compared with the tills. They may strongly influence the mode of failure where they occur. Other controls on the mode of failure are the cliff height and state of the beach. The cliff behaviour may be drained, partially drained or undrained, depending on the rate of swelling of the cliffs compared with the rate of unloading. The reduction in stress behind the cliff as it is unloaded is important because:

1. it may set up depressed pore water pressure as observed in other cohesive cliffs of low permeability;
2. it influences the stress path followed to failure, which may affect the mobilized strength parameters.

Underdrainage of the cliffs may also be an important influence on the pore water pressure distribution. This summary forms the basis for the following research questions, which were used to guide the design of the study.

2.6.2 Research Questions

1. How is the geotechnical behaviour of the cliffs influenced by their geology?;
2. How do other controls influence cliff behaviour, in particular the state of the beach and cliff geometry?;
3. What effect do the stress changes accompanying cliff recession have on the deformation of the cliff, pore water pressure distribution, and mobilized strength parameters?;
4. How long do depressed pore water pressures persist, ie. are the cliffs drained, partially drained or undrained?;
5. What is the effect of other influences on the pore water pressure distribution, hence stability, eg. underdrainage and seasonal changes?

Some of these questions were partly answered by this literature review, but all required further investigation. Two complementary approaches were used: a general survey of 28km

of coastline, and a detailed investigation at two sites including field instrumentation and laboratory testing. The generally survey, described in Chapter 3, was aimed at questions (1) and (2) and the site investigation, described in Chapter 4, was aimed at questions (3), (4) and (5).

CHAPTER 3

GENERAL SURVEY

3.1 Aims and Method of the Survey

The chief modes of cliff failure, and their relationship to cliff geology, geometry and beach conditions were described in Chapter Two from previous studies. However it was felt necessary to conduct a general survey at the start of this study to:

1. investigate the relationship in more detail;
 2. provide an accurate record of the coast for comparison with previous data and to form a reference for future work²;
 3. show how relevant the detailed study at two sites (described in Chapter Four) is to the recession of the coast generally;
 4. provide additional information for the detailed study, by examination of similar failures at different stages in the erosion cycle.
2. The original photographs and drawings produced in the survey are stored in an archive at the Department of Geotechnical Engineering, University of Newcastle upon Tyne.

3.1.1 Introduction to Method and Analysis

The survey was limited to 28km of coast between Skipsea and Withernsea (see Figure 1.1) because of time and logistical constraints. This length was chosen because all the principal modes of failure, soil types and beach conditions were represented. The coast south of Withernsea is the subject of a current investigation by Hull University. Conventional geomorphological mapping methods were impractical, because the steepness and soft surface of the cliff prevented access to the beach except at widely spaced places. High tides reached the cliff foot at many places, allowing only a short working period. To overcome these problems photographs were used to record the cliff morphology, which could later be studied in detail away from the field. It was found necessary to record additional information onto the photographs in the field, for example soil type, which could only be accurately determined by close examination of the cliff.

Cliff failures and morphologies were studied in the field as the survey progressed, however a more objective assessment of their distribution and relationship to other factors was desired. To achieve this the cliff morphology and mode of failure were classified into a small number of types, allowing the use of a computer program (SPSSX) to handle and analyse the data. A measure of the relative proportion of any particular morphology was obtained by calculating the percentage it formed of the cliff face by area. All the other factors mapped during

the survey were also treated as variables and assigned a range of numeric values. The program could then be used to obtain descriptive statistics and plot variables with respect to location on the coast. By comparing the coastwise distribution of two or more variables, correlations between them suspected from the visual field observations could be checked. While this approach does not give proof that a relationship exists, it does provide some supporting evidence for the field observations. The survey method and analysis is described in more detail below and then the results are presented. They are given in the order 'field observations', followed by 'plots and statistics', for each factor.

3.1.2 Method of Field Survey

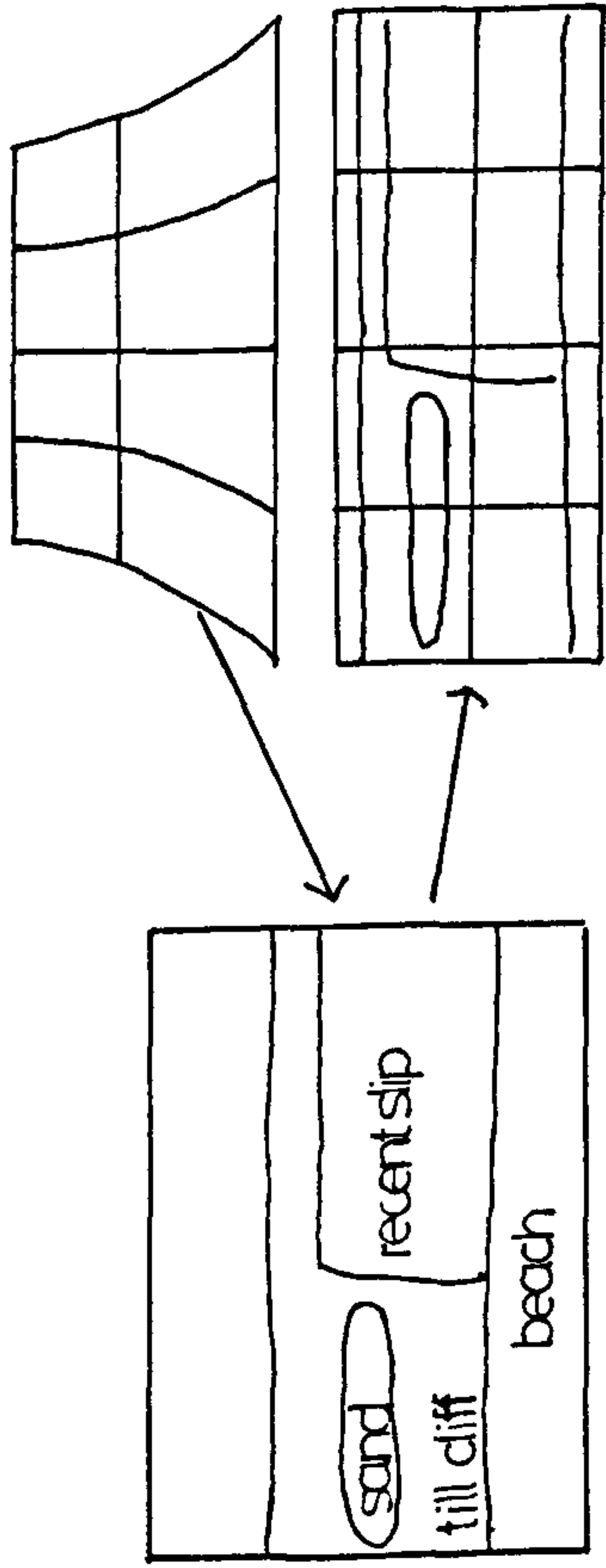
The cliffs were photographed at 50m intervals using a hand held camera with wide angle lens. Tidal and lighting conditions limited photography to a four hour period around low tide, when it occurred in the early morning. This allowed five working days per fortnight, with an average progress of 1km per day. A scale was provided by two ranging poles set 50m apart at the foot of the cliff, using a length of nylon cord to check the distance. The camera was positioned in the same place with respect to the cliff for each photograph by pacing. A numbered board was included in the field of view, to prevent mixing of the prints during developing. Prominent clifftop features, eg. pillboxes, were used to locate the photographs, and the grid

reference of each photograph was found from Ordnance Survey 1:10,000 scale maps. A4 sized black and white prints were produced and information recorded in the field with a felt marker pen. The features mapped included soil type, mode of failure, cliff morphology, seepages and dis-continuities. The photography was completed between April and June 1986 and the mapping between June and October 1986. A quarter of the surveyed coast (7.2km) was re-examined during 1987 to assess the effects of one year of erosion.

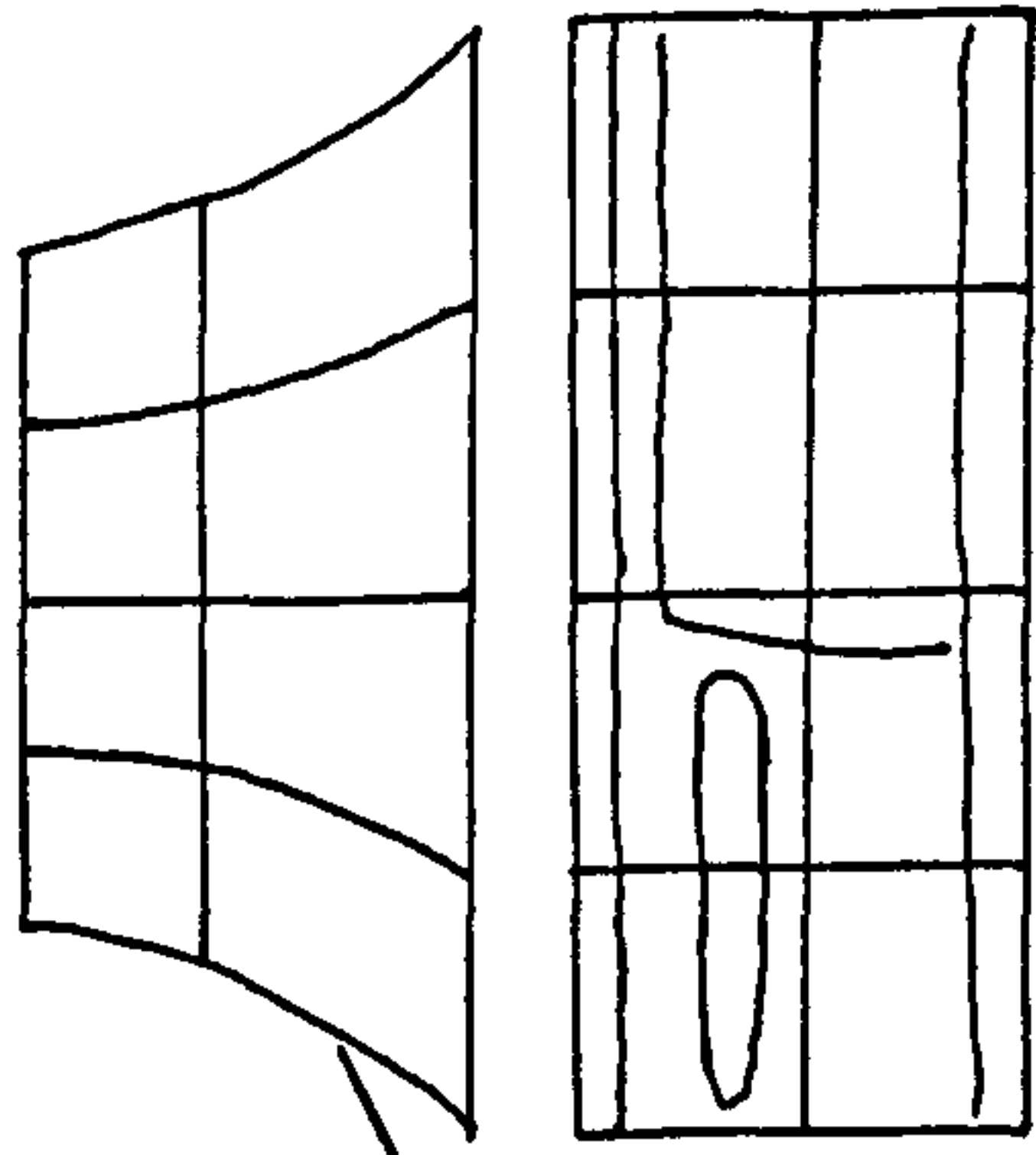
3.1.3 Abstraction of Data from Photographs

On completion of the fieldwork 577 photographs with detail recorded onto them had been produced. A description of the cliff morphologies and definition of the variables is given in Appendix A. To analyse this large dataset, the data was encoded for entry to the SPSSX batch system (see Appendix A), available on the University mainframe computer. The process of abstracting the data from the photographs is illustrated in Figure 3.2 and described below.

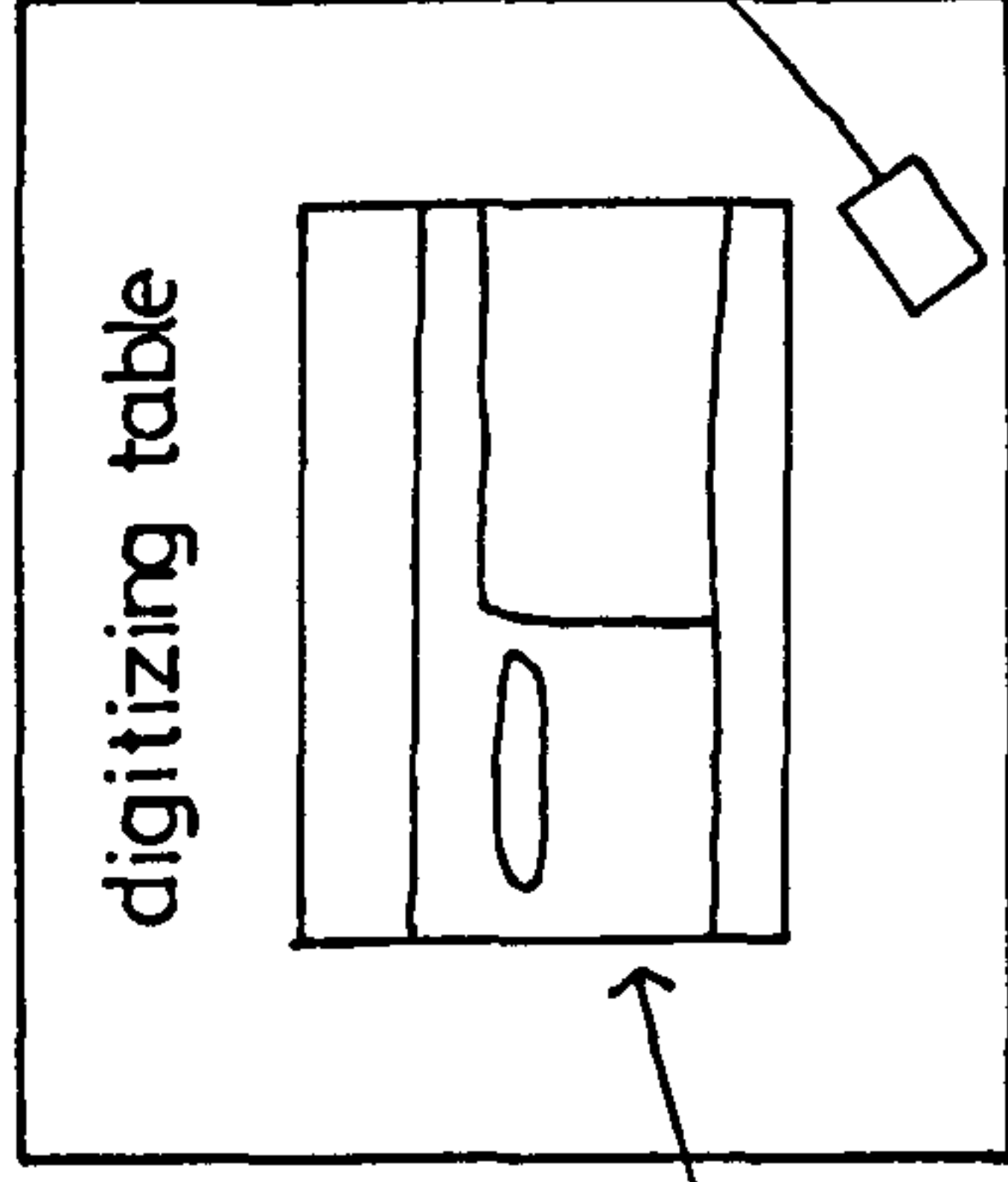
To correct the detail mapped on the photographs for scale distortion, a curvilinear grid drawn on a transparent plastic sheet was placed over the prints, and major features re-drawn onto tracing paper placed over a rectangular grid. An "Imagan" computer based image quantification system was used to quantify the drawing. The tracing was placed on a digitizing table, and



(1) A4 sized photograph of cliff with features highlighted



(2) Curvilinear grid placed over photograph and features redrawn onto rectangular grid



(3) Drawing quantified with "Imagan" program and hard copy produced at 1:500 scale

(4)

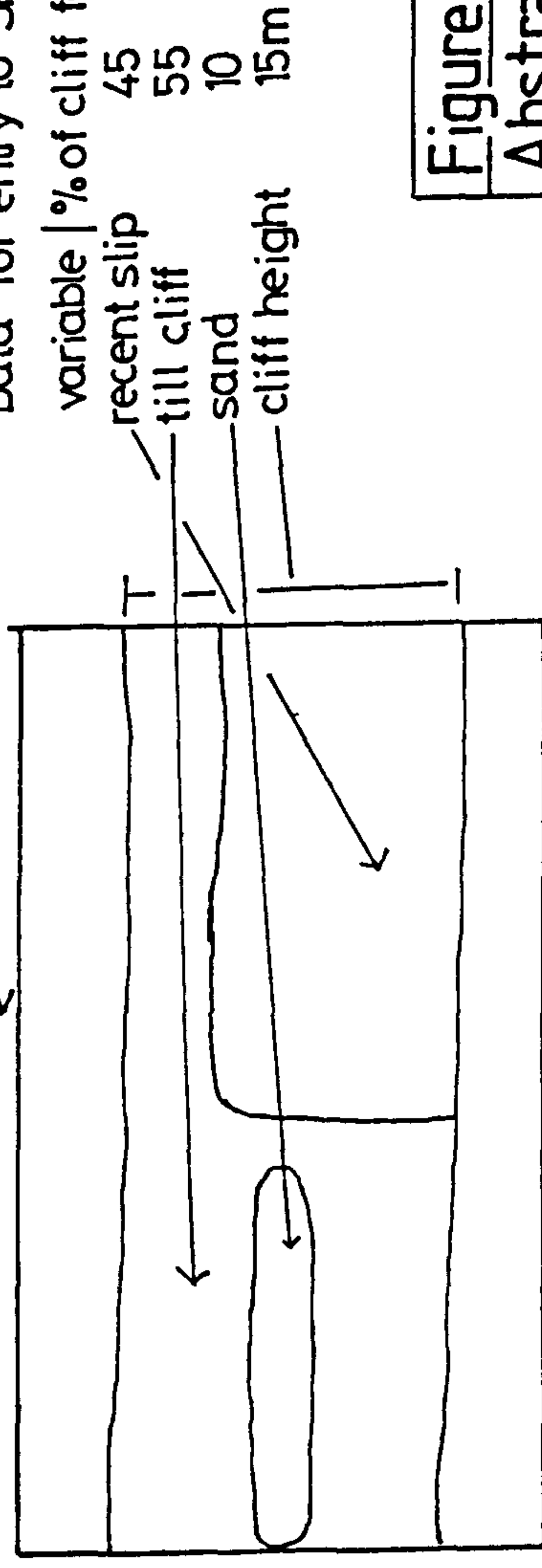


Figure 3.2
Abstraction of data from photographs

the features traced with a cursor. The program then calculated the percentage (by area) of any particular desired area compared with the whole cliff face. A hard copy of each drawing was obtained at 1:500 scale, and these were then joined together in lengths of seven representing 350m of the cliff. The lengths were then attached to boards allowing rapid inspection of any section of the coast. For entry of data to SPSSX each photograph was treated as an individual case. The values of variables associated with each case were determined from the drawings, and entered in a computer file to form a dataset. A listing of the dataset is given in Appendix A.

3.1.4 Data Analysis

Descriptive statistics of mean, standard deviation, minimum and maximum were obtained for each variable and are given in Appendix B. Selected variables were plotted with location on the coast (coastwise). For each of these variables the average value for seven cases, equivalent to a 350m length of cliff, was calculated before plotting. This was done to reduce the plots to a manageable size (approximately A4). The effects of some variables, for example soil type, were studied by selecting only those cases where that soil type was present for determination of statistics. The relationship between some pairs of variables was investigated by cross-plotting them.

The variables were divided into "control" and "response" variables for the purpose of analysis, as shown in Figure 3.3. Firstly the influence of the "control" variables on the "response" variables was investigated. Then the coastwise distribution of the "response" variables was studied and related to the "control" variables. Although there was some degree of overlap in this approach, it was felt useful as a check, and to show if there were any areas which could not be explained by the known controls. Finally the cliff was divided into zones where a particular soil type, mode of failure or beach condition prevailed.

Control variables:	Response variables:
Minor soil types present within till Beach conditions Cliff height	Cliff morphology Mode of failure

Figure 3.3 "Control" and "Response" variables.

3.2 Influence of Control Variables

3.2.1 Sand and Gravel Deposits

FIELD OBSERVATIONS

Sand and gravel deposits of various geometries occur in most cliff sections. The percentage by area of cliff face is usually small (mean 2.8%), but in a few areas is larger (10-50%). The deposits usually occur in layers, laterally extensive over tens of metres and from 0.2-2m in thickness, which were sub-glacial drainage channels (see Section 2.2.2). The layers may occur at any stratigraphic level. In some places these layers have been folded, forming irregular patches a few metres in diameter. Where sub-glacial drainage was persistent the sand deposits are larger, and take the form of channels several metres in length and thickness³.

A detailed description of the cliff behaviour in areas with a high percentage of sand (>10%) is given in Appendix C. Study of these areas showed that the presence of sand deposits may influence the mode of failure in three main ways:

1. increasing susceptibility to marine and seepage erosion, leading to undercutting and falls from the cliff;
3. "Thickness" refers to the vertical dimension, and "length" to the horizontal dimension parallel to the cliffline.

2. allowing slip surfaces to develop in unfavourably positioned layers;
3. improving drainage, thus increasing the rate of dissipation of excess pore water pressure, and preventing high seasonal pore water pressure rises.

The general effect is to make falls more common than slips, as a consequence of (1) and (3). This is because with good drainage the cliffs are stable up to a high slope angle, and with undercutting will collapse by falls before they become steep enough for slips to occur. The cliff morphology is predominantly "till cliff", with ridges and embayments developing, particularly where seepage erosion by groundwater emerging from sand layers occurs.

PLOTS AND STATISTICS

A coastwise plot of "till cliff" with "sand" superimposed is shown in Figure 3.4. There is a good correlation between areas with a high value of "till cliff" and high value of "sand", although there are a few high "till cliff" areas occurring where "sand" is low. These may be influenced by the effects of other variables. The coastwise plot of "total slips" is compared with the distribution of areas with a high value of sand (>10%) in Figure 3.5. The value of "total slips" is below the mean in these areas, and is zero near East Newton. However this agreement may be coincidental, as these areas are of low

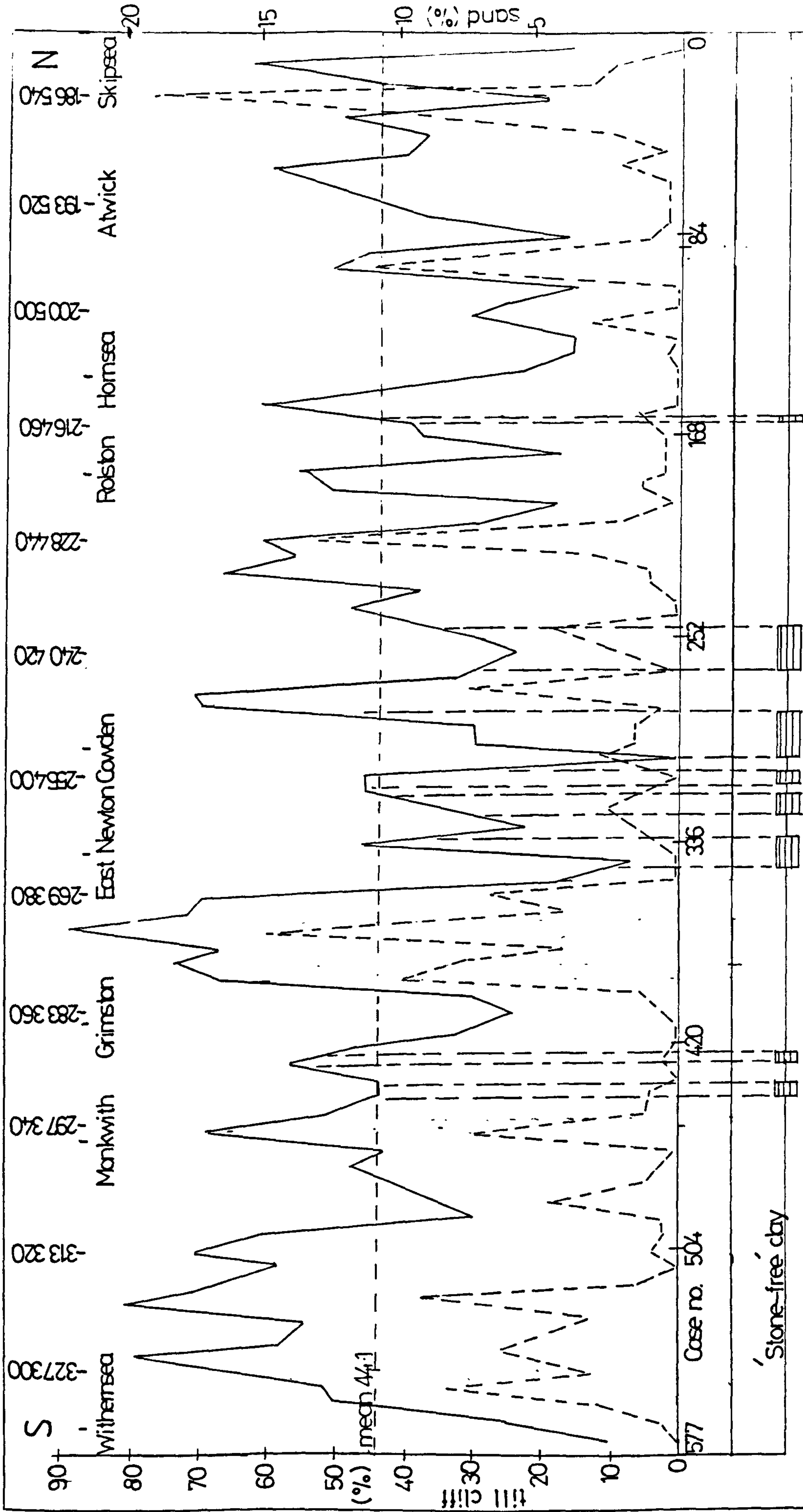


Figure 3.4
Coastwise plot of 'till cliff' with 'sand' and
areas of 'stone-free clay' superimposed

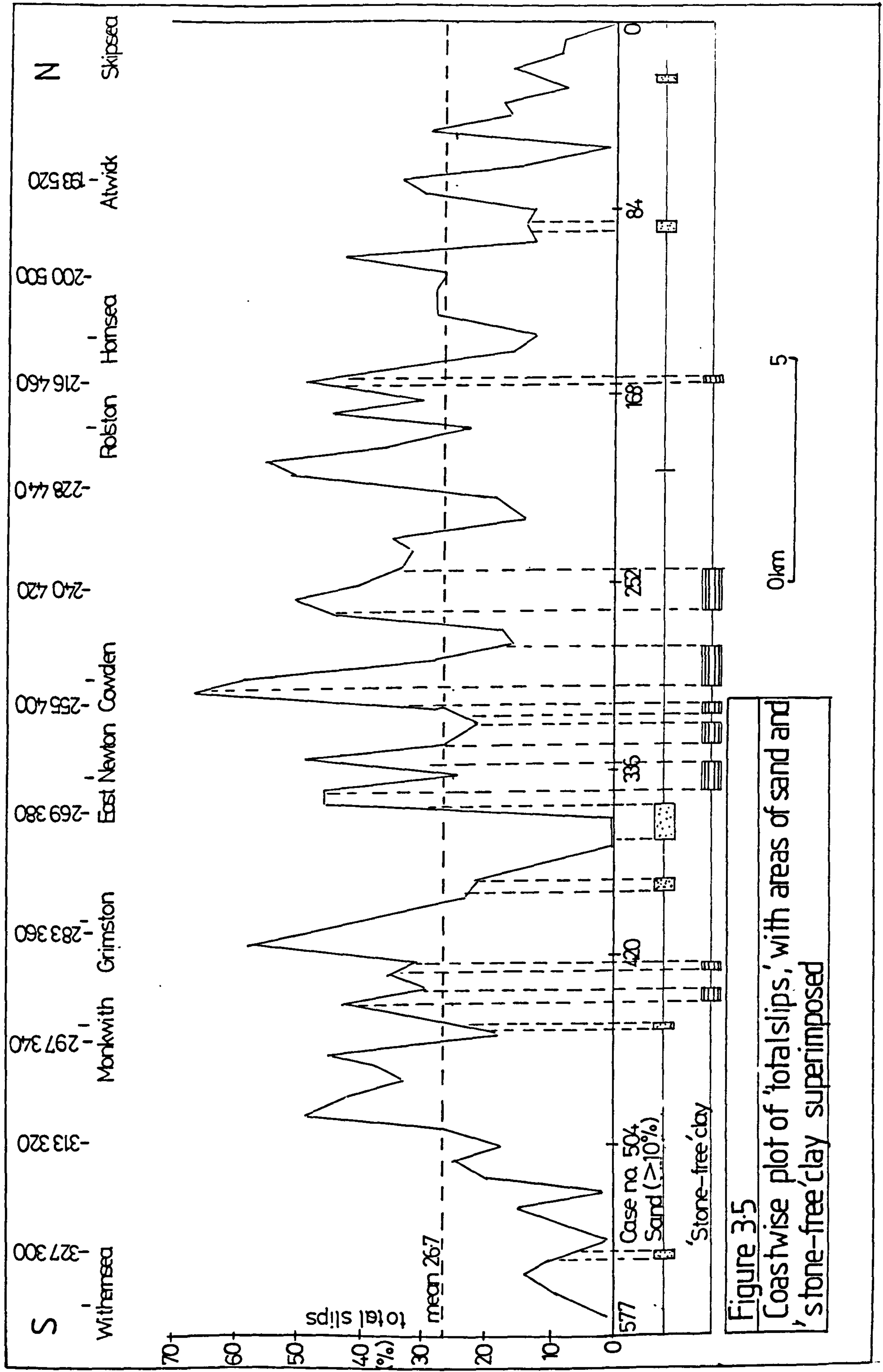


Figure 3.5
Coastwise plot of 'total slips,' with areas of sand and 'stone-free' clay superimposed

cliff height, which can also reduce "total slips" (see Section 3.2.4). The cases were sorted for analysis into three groups by the value of "sand", shown below:

1. "sand"<2.8%
2. 2.8<"sand"<9.4%
3. "sand">9.4%

Mean values of the variables "total slips", "till cliff" and "groundwater index" for each group are given in Figure 3.6.

Variable	Group of "Sand" Values		
	(1) "sand"<2.8%	(2) 2.8<"sand"<9.4%	(3) "sand">9.4%
Total Slips (%)	30.2	17.8	12.7
Till Cliff (%)	39.2	57.5	61.0
"Groundwater Index"	0.24	0.53	0.94

Figure 3.6 Mean values of "total slips", "till cliff" and "groundwater index" for cases sorted by value of "sand".

The statistics confirm the reduction in "total slips" and increase in "till cliff" with a high proportion of sand and gravel deposits, which was observed in the field. The increase in "groundwater index" indicates the sand layers are effective as drains. The statistics also indicate (by the difference between groups (1) and (2), that relatively small sand deposits

(<9.4%) can have an influence on the cliff behaviour. In the zonal scheme developed in Section 3.4, areas with a high value of "sand" (>10%) are grouped in zone "D".

3.2.2 "Stone-Free" Silty Clay Layers

FIELD OBSERVATIONS

Within the till sequence there frequently occur layers of "stone-free" silty clay, which are soft in consistency when exposed at the surface, and are often laminated (hereafter referred to as "stone-free" clays). They are laterally very extensive, often for several hundred metres, and vary in thickness from 0.1 to 2m. They were not often exposed during the survey, as they were hidden by the beach or slip debris. However their presence could be inferred, from the characteristic mode of failure and slope morphology which develops when they occur near the cliff foot (see Section 2.5.3). On subsequent visits it was confirmed at some locations, when the beach was low. Their influence on the cliff failure was most commonly observed between Cowden and Grimston in central Holderness. In this area they occurred at a distinctive boundary between two divisions of the Skipsea till. In the model of glacial deposition for Holderness proposed by Derbyshire et al (1985), they were deposited in subaerial (probably lacustrine) conditions during a temporary retreat of the ice-sheet (see Section 2.2.2).

In previous studies by Gilroy (1981) and Wiggin (1987), the "stone-free" clays were found to have a high plasticity index and low residual strength. This is confirmed by tests carried out on "stone-free" clay at Cowden (Butcher, personal communication).

In areas where "stone-free" clay layers occur near the cliff foot, the failure surfaces of the slips often appear to be partly within them, giving the slips a flat sole (see Figure 3.23). The slips are base failures with the toe emerging in the foreshore, between 5 and 30m from the cliff foot. The toe is often marked by a low ridge, arcuate in plan, visible when the beach is removed. Sheared "stone-free clay" is sometimes exposed at the ridge (see Figures 3.7 and 3.8). The cliff morphology is of long degraded slips, often multiple and of low slope angle ($20-30^{\circ}$), and continuous along the coastline for several hundred metres (see Figures 3.8 and 3.32). This contrasts with other areas where slips are separated by areas of "till cliff". A description of the main areas affected by "stone-free" clays is given in Appendix C. These include the B.R.E. cliff study site at Cowden where recent work has shown the strong influence of this soil type on the mode of failure (Butcher, personal communication). Complex failures, consisting of three blocks were found to occur.



Figure 3.7

Exposure of "stone-free" clay at lower ridge in foreshore of Figure 3.8 (a) (the toe of a previous slip). Note shear surfaces rising to the right (seawards).



Figure 3.8(a)

Recent slip influenced by the presence of "stone-free" clay. Note two ridges in the foreshore, marking the toe of the present slip and a previous one.



Figure 3.8(b)

Degraded slip influenced by the presence of "stone-free" clay. Note long, multiple, low angle nature of the slip (length 90m).

PLOTS AND STATISTICS

The coastwise plots of the variables "till cliff" and "total slips" are compared with the areas where "stone-free" clay layers occur in Figures 3.4 and 3.5. The variable "total slips" shows a sharp peak, well above the mean value, in most of these areas, with a corresponding reduction in "till cliff"⁴. This is probably because the low strength of the "stone-free" clay layers causes the cliffs to fail by slips (as the toe is eroded), before the slope angle becomes steep enough for falls to occur. The multiple form of the slips results from the occurrence of a new failure before the degraded debris of the previous one is entirely removed from the toe. This is in contrast with other areas where the degraded debris is removed, and the cliff steepened by erosion and falls before a new failure occurs. The predominantly degraded nature of the slips is shown by the coastwise plot of "degraded slips" in Figure 3.9. At each of the areas where "stone-free" clay occurs there is a peak. It is also seen that most of the base failures (see Section 3.3.1) occur in these areas.

The cases were divided into two groups for determination of statistics, those where "stone-free" clays influenced the failure mode and morphology and those where it did not. The results are given in bar-chart form in Figure 3.10. The cliff

4. In some cases the peaks are slightly offset from the "stone-free" clay areas; this could be due to the averaging procedure of grouping the cases into 350m lengths.

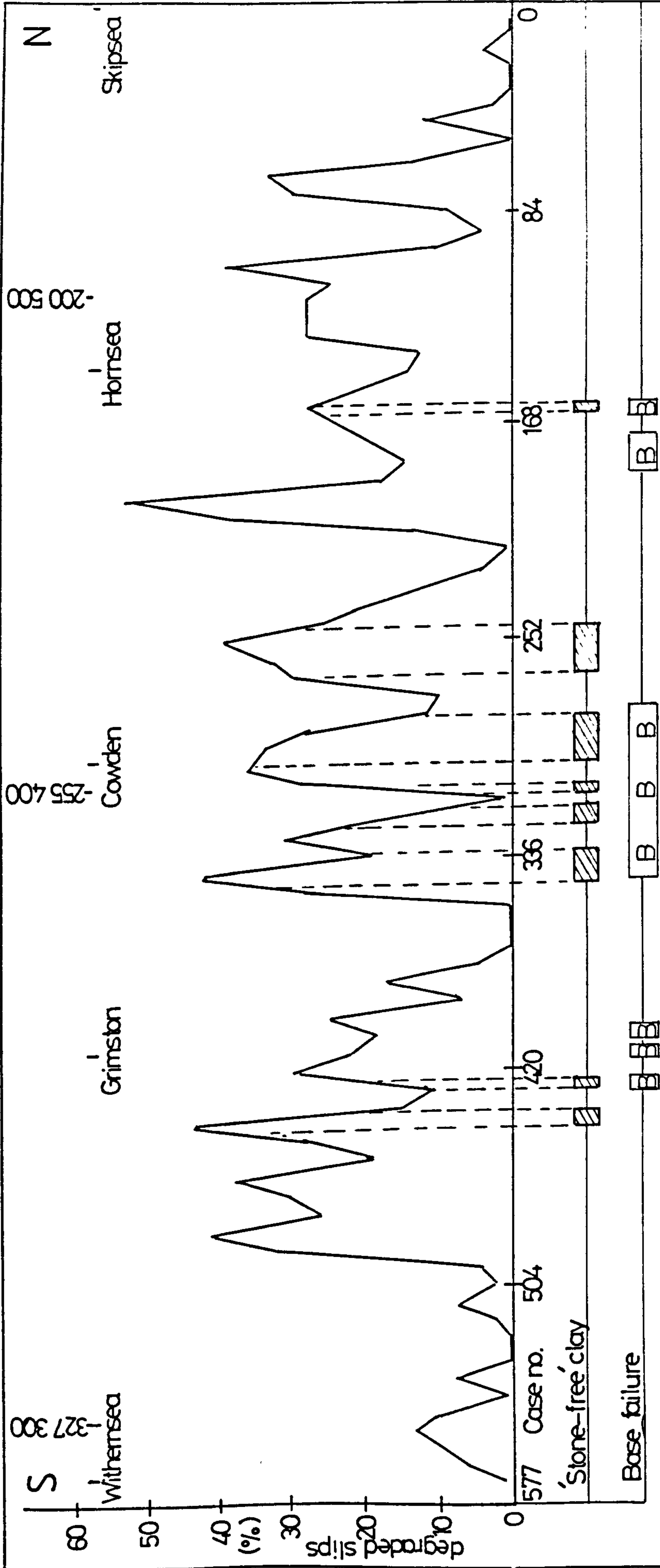
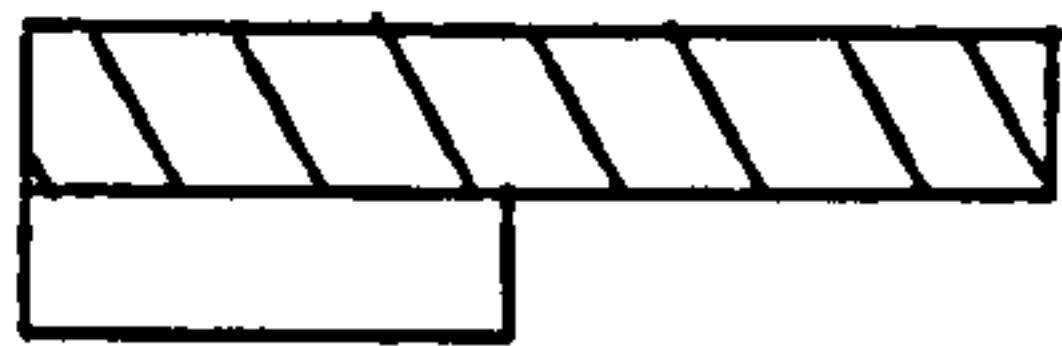


Figure 3.9
Coastwise plot of 'degraded slips' with positions of
'stonefree' clay layers and base failures shown

Stone-free clay layers
Base failures
0km 5

Cliff morphologies (mean percentage of cliff)

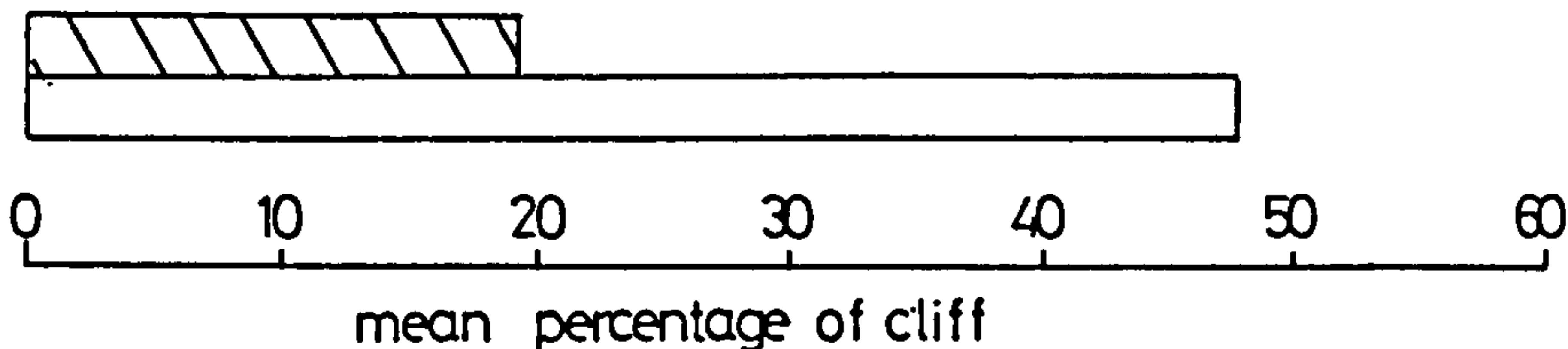
RECENT SLIPS



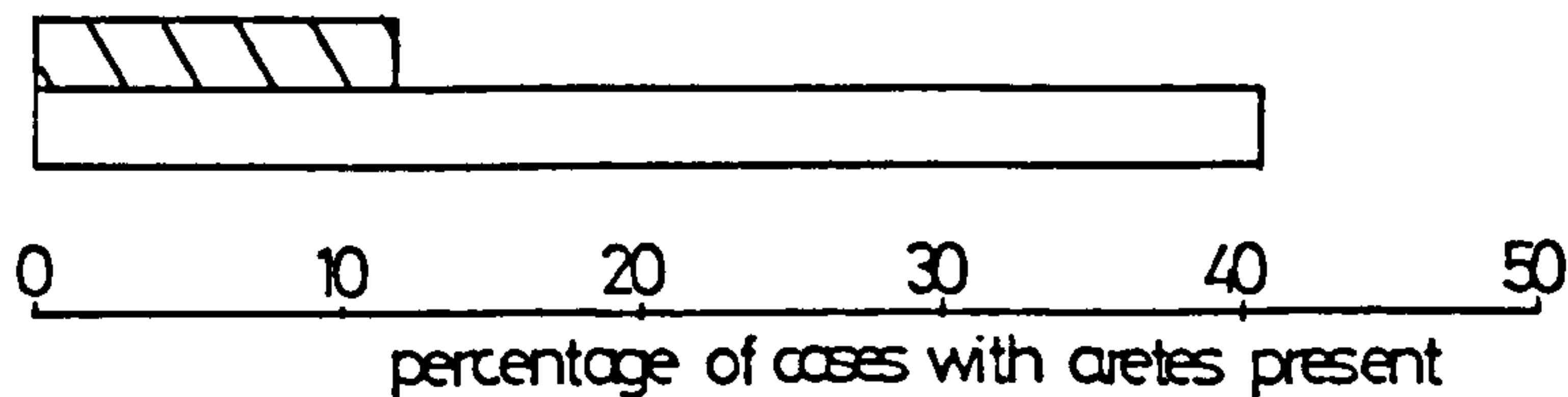
DEGRADED SLIPS



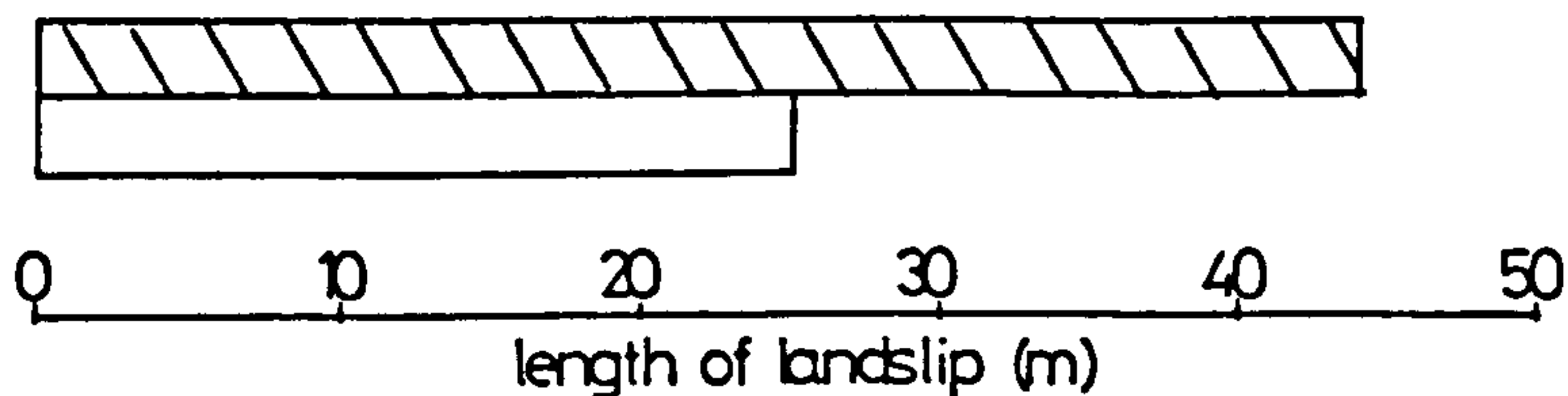
TILL CLIFF



Ridges



Length of landslip (mean)



Position of slip toe

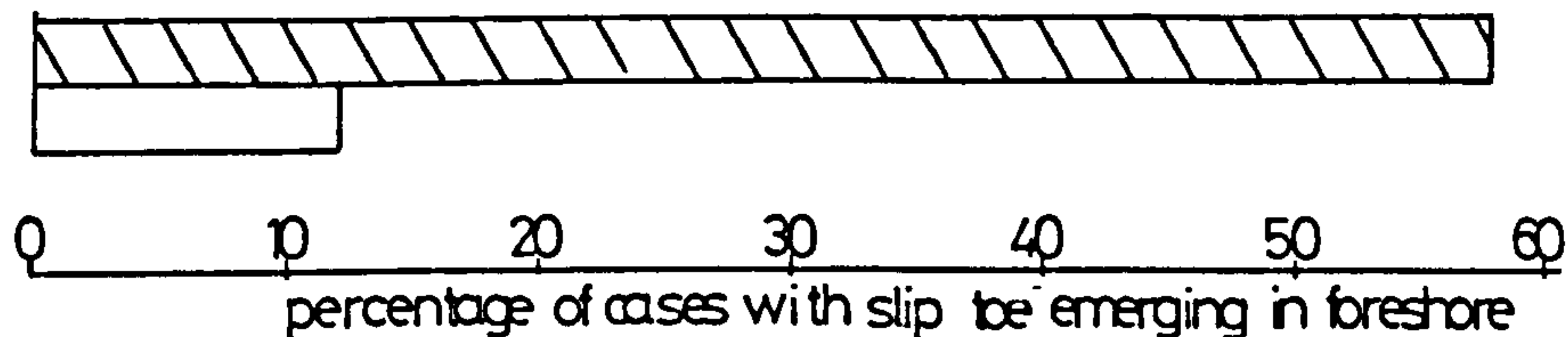


Figure 3.10

Bar chart to compare cases with and without 'stone-free' clay influence



morphology shows an increase in "degraded slips" and "recent slips", with a corresponding decrease in "till cliff" and "ridges", in the cases influenced by the "stone-free" clays. The change in slip morphology is shown by the increase in both the mean length of the landslips, and the percentage of cases with the toe emerging in the foreshore. The statistics confirm that the field observations of the effects of "stone-free" clays are representative. In the zonal scheme of Section 3.4, areas influenced by them are grouped in zone "C", and form a total length of 4.7km or 16.3% of the surveyed coastline.

3.2.3 "Red Clay" Layers

FIELD OBSERVATIONS

Stiff reddish brown clay layers, up to 1m in thickness and laterally extensive over several kilometres often occur at till boundaries (hereafter referred to as "red clay" layers). They are thought to be flow-tills, associated with the "stone-free" clays described above, and their colour may be due to sub-aerial weathering (see Section 2.2.2). They occur near the base of the cliff at the Upper/Middle Skipsea till boundary between Hornsea and Aldbrough, and at the Skipsea/Withernsea till boundary between Grimston and Tunstall (see Figure 3.17).

The "red clay" layers, together with associated thin sands, silts and gravels, are less resistant to corrasion by waves than the surrounding tills. Their presence encourages falls from the cliff face, by the process described by Hutchinson (1986, see Section 2.5.3). The falls may be several metres in length and affect the full height of the cliff (see Figure 3.11). The process has been observed at many places between Hornsea and Aldbrough, indicated by the "red clay" boxes in Figure 3.12, and occurs along 7km (25%) of the coastline. There is no correlation of areas of high or low "total slips" with the "red clay" layers, showing that they are not a major control on the mode of failure. Their influence is most noticeable in areas where "till cliff" is the chief morphology, and the cliff is steep enough for falls to occur. However these "till cliff" areas are probably caused by the influence of other variables, for example cliff height (as discussed below), and many would continue to fail by falls, even if the "red clay" layers were not present.

3.2.4 Cliff Height

The cliff height varies between 4m and 19m with a mean of 12.1m on the surveyed coastline. The highest cliffs occur around Grimston and the lowest near Hornsea, Withernsea and East Newton. During the field survey it was observed that below a height of 10-12m, falls and direct erosion of till replace slips as the dominant mode of failure, confirming the



(a) Notch forming in cliffs by erosion of 'red clay' layer.



(b) Fall developing by undercutting of the cliff

Figure 3.11

Development of falls in cliffs near Mappleton (TA 222 450)

observations of Davison (1985, see Section 2.5.3). The coastwise plots of the variables "cliff height" and "total slips" are superimposed in Figure 3.12. There is some correlation between the two, with areas of high and low values of "total slips" coinciding with areas of high and low cliff height. However "total slips" varies widely because of the effect of other variables, particularly "stone-free" clays and beach conditions (see Sections 3.2.2 and 3.2.6). At East Newton and North Withernsea, where cliff height is below 10-12m, the value of "total slips" is nearly zero. Both of these areas have a high sand content which may encourage falls (see Section 3.2.1).

To demonstrate more clearly the relationship between "total slips" and cliff height, they are plotted together in Figure 3.13, after excluding cases with unusual soil types or beach conditions. The mean value of "total slips" was calculated for each 1m increment in cliff height. The plot shows the trend for increasing "total slips" with increasing "cliff height". A possible explanation is that there is a greater increase in average shear stress along a potential slip surface, than in shear strength, with increasing cliff height. Thus higher cliffs would be more likely to fail by slipping. The mean values from individual sub-zones (see Section 3.4) are also plotted, and show a sharp increase in "total slips" above 12.5m.

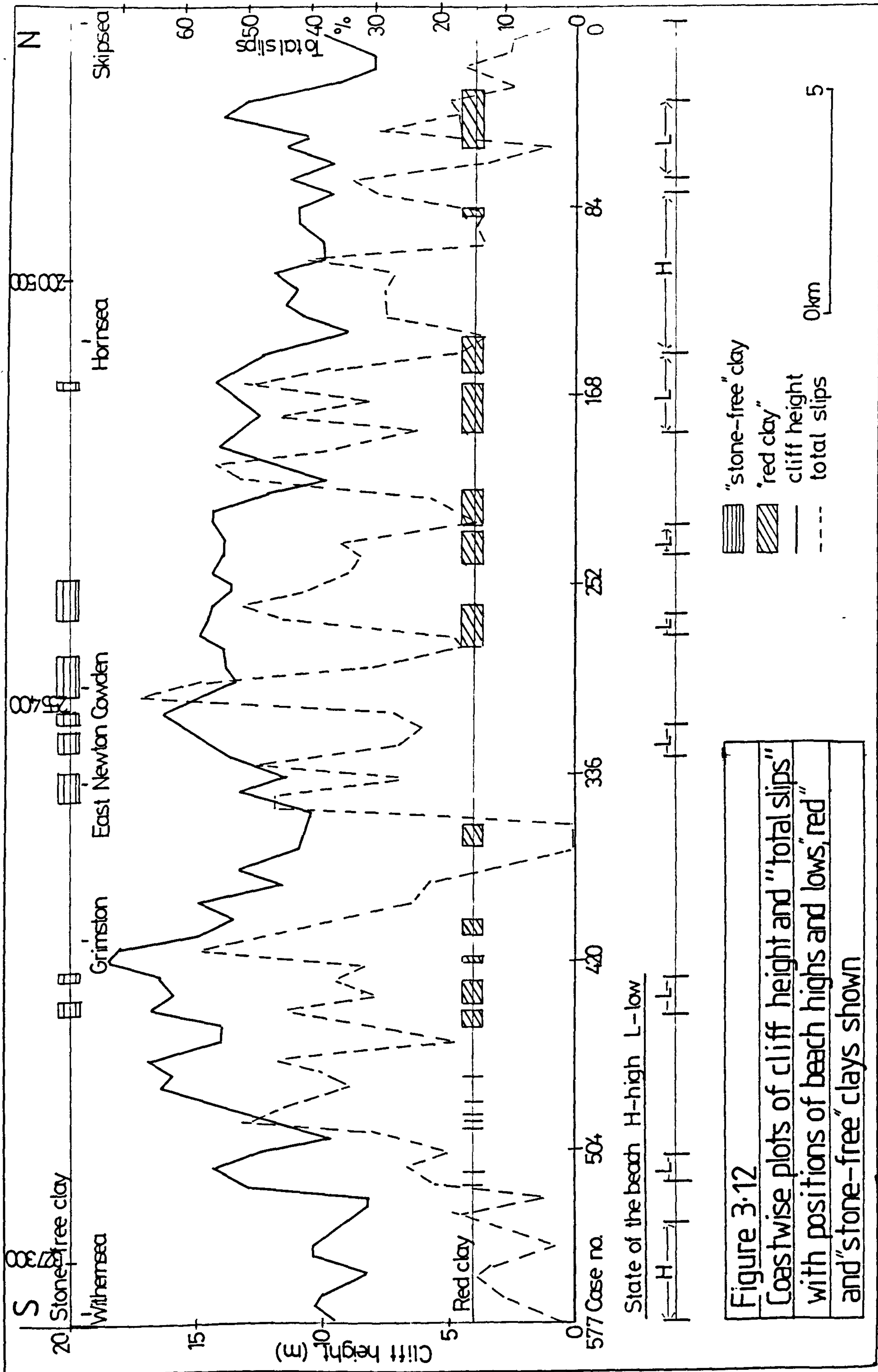


Figure 3.12
Coastwise plots of cliff height and "total slips"
with positions of beach highs and lows, "red"
and "stone-free" clays shown

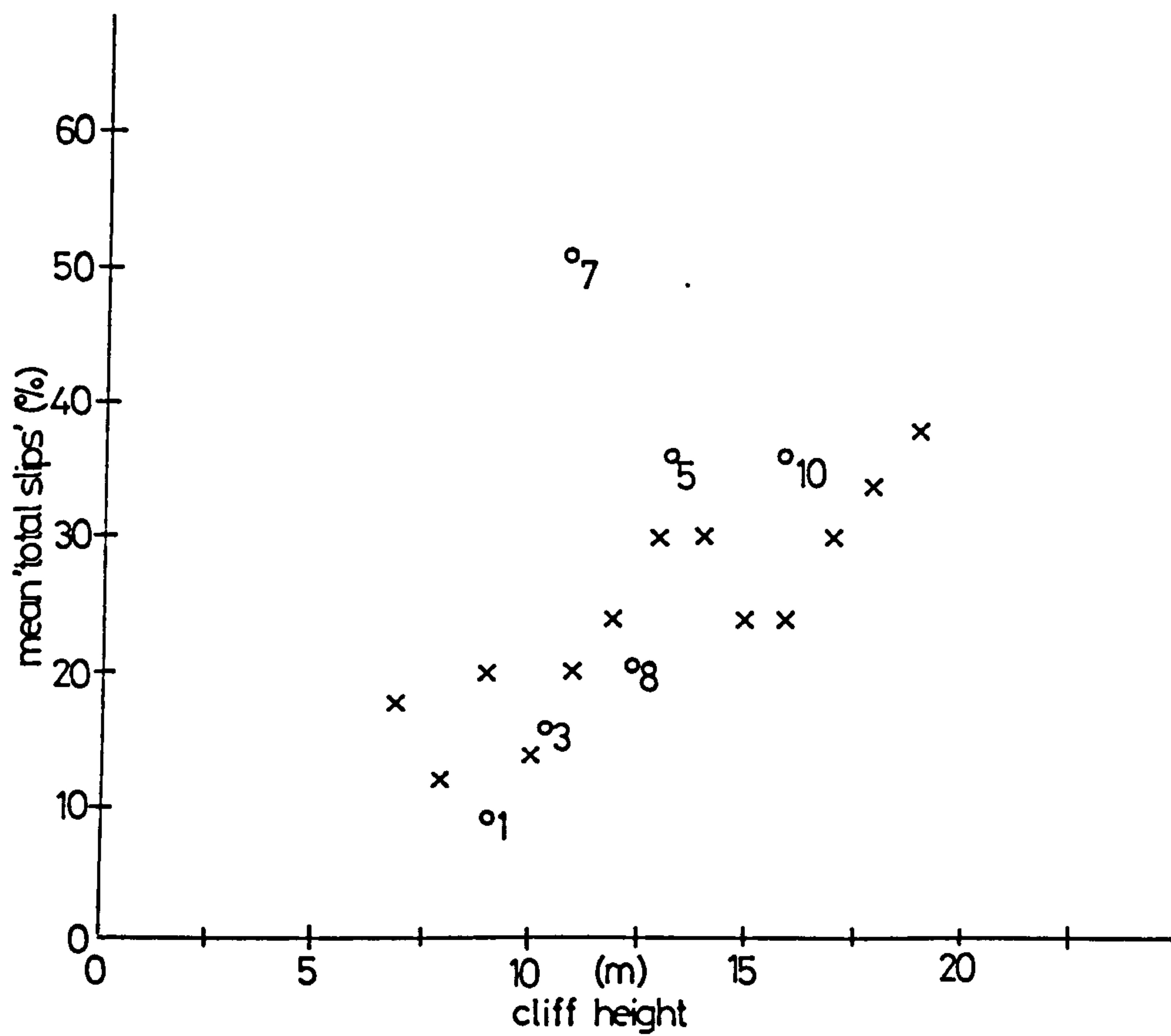


Figure 3.13
Crossplot of mean 'total slips'
with cliff height

x mean value calculated for each 1m interval in cliff height
o mean value from sub-zones (1,3,5,7,8 and 10)

3.2.5 Discontinuities

The tills are intact materials apart from the ubiquitous fissures in the weathered zone, and occasional more regular sets of fissures. These sets may occur parallel to the cliff edge, presumably formed by stress relief, or as inclined sets near the boundaries between tills, of depositional origin. Their effect is small, limited to aiding falls of weathered clay or falls from ridges. At a few places on the foreshore, irregular closely spaced fissures allowed the till to be more easily removed by marine erosion. Recent slips of several metres displacement were often broken into angular columns and blocks, allowing the slipped soil masses to be more easily eroded. The columns were sometimes bounded by discontinuities, but more often by tensile failures of the till, occurring as it moved outward.

3.2.6 Beach Level

The effects of natural and artificial variations in beach level on cliff foot erosion were discussed in Section 2.3. They were further studied during this survey, in particular to see if variations in the rate of cliff foot erosion influence the mode of failure (as occurs in coastal slopes formed in other materials, see Section 2.4.5).

FIELD OBSERVATIONS

Beach highs and lows were recorded during the survey and the effects of "ords" near the instrumented sites observed throughout the study. During the survey in April 1986, 8.0km or 27.7% (by length) of the beach was low and 3.7km or 12.7% was high. The positions of the low and high areas are shown in Figure 3.14. Six "ords" occurred between Skipsea and Withernsea, which was a typical number for the coast. Their movement during the study was difficult to follow, except near the sites of Rolston and Grimston, because of the infrequent visits to the coast. At Grimston the progress of two "ords" was followed. One moved from Ringbrough (TA 285 365) in May 1987 to Grimston Hall (TA 288 351) in April 1988, passing the Grimston site between August 1987 and January 1988 causing increased toe erosion and faster degradation of the two slips there (see Section 4.3.9). The other moved from Grimston Hall (TA 287 353) in April 1986 to Hooks (TA 293 346) in May 1987. The mean rate of southwards movement of the "ords" was estimated to be 1km/yr. This was faster than the mean rate of 0.5km/year observed by Pringle (1985), however it was difficult to determine the true centre of the "ord", so the estimate may be in error. The effects on the cliff and beach of the "ord" at Hooks were studied during May 1987. The changes caused by it are shown in Figure 3.15, and are summarized below. In the "ord":

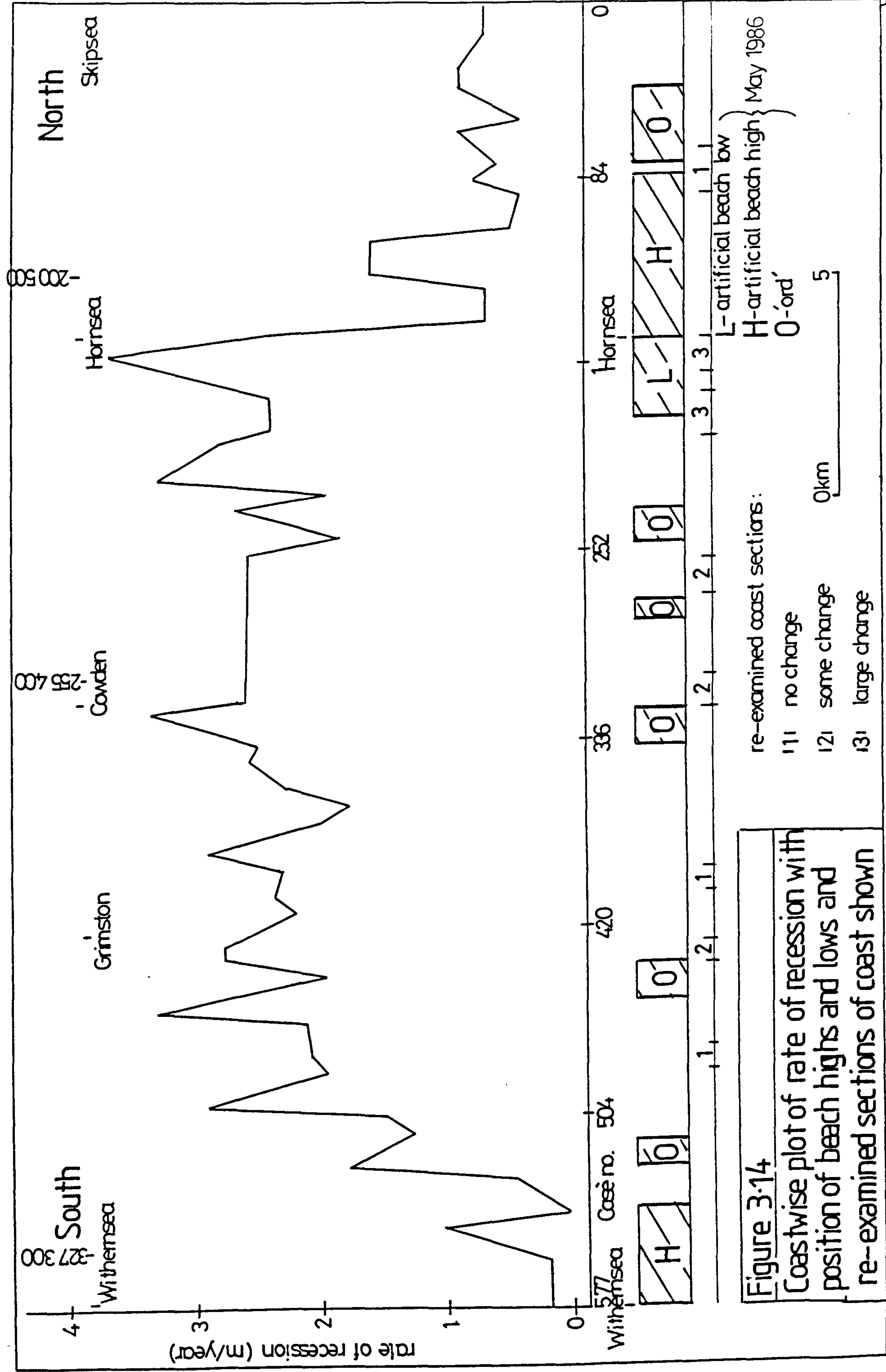


Figure 3.14

Coastwise plot of rate of recession with position of beach highs and lows and re-examined sections of coast shown

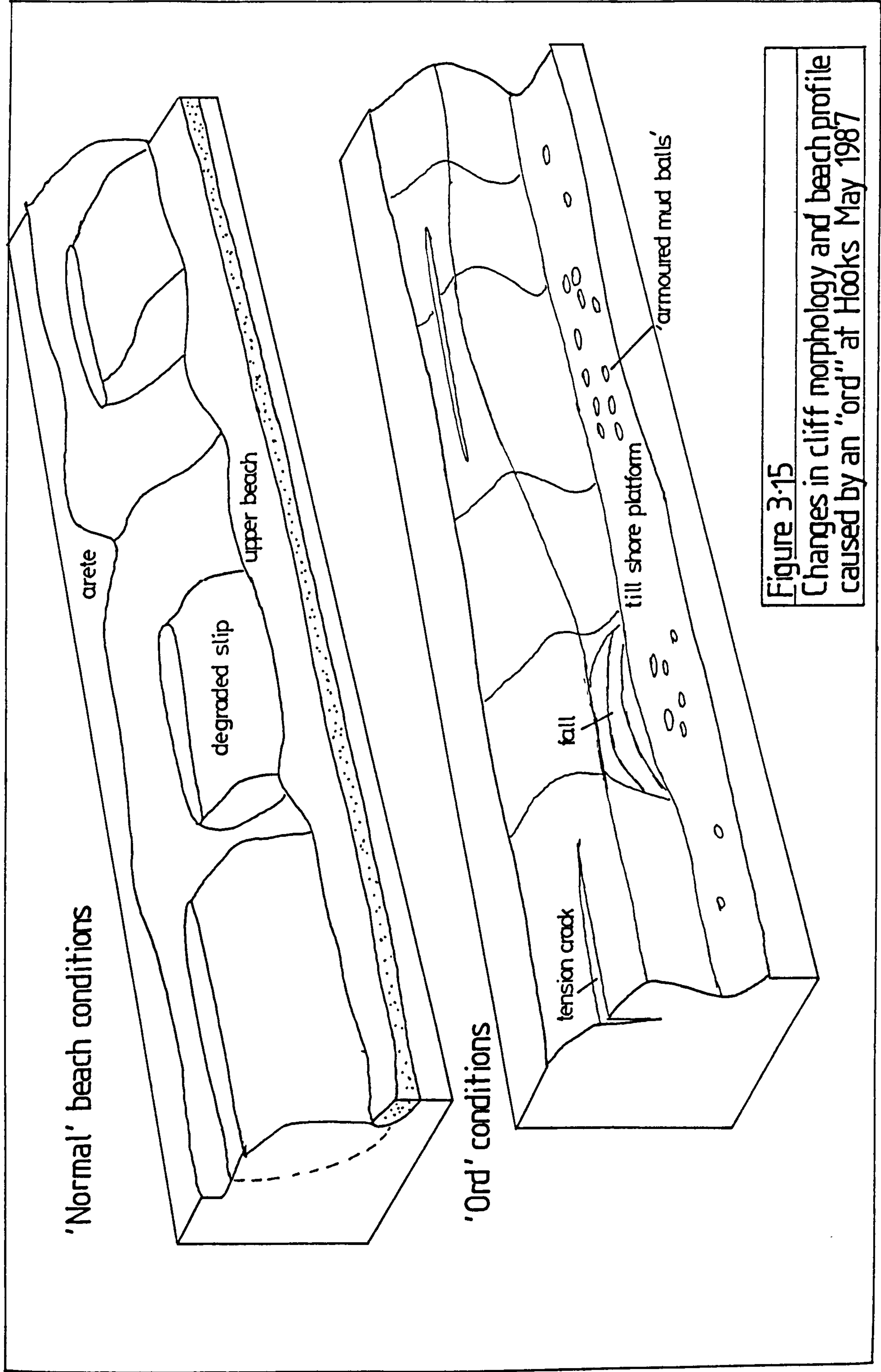


Figure 3.15

Changes in cliff morphology and beach profile caused by an "ord" at Hooks May 1987

1. the upper beach is absent, exposing the till shore platform;
2. the cliff at beach level is vertical or overhanging and falls are frequent;
3. "armoured mudballs" from falls rest on the till shore platform;
4. degraded slip debris has been removed;
5. "ridges" are less prominent.

These conditions are typical of those found in "ords" anywhere along the coast, and are also shown in the photograph of an "ord" near Cowden in February 1987, Figure 3.16.

"Ords" were not observed at the Rolston site, probably because it lies just south of the permanently depleted beach near Hornsea. However lowering of the beach was sometimes observed, owing to the effects of onshore winds (see Section 2.3). For example, in May 1988 the upper beach between Hornsea and Mableton (about 4km length) was removed, exposing the till shore platform. Erosion of it and the cliff foot was then severe, shown by the numerous "armoured mudballs" littering the foreshore. Sometimes the lower beach was also removed, again allowing erosion of the shore platform, as shown in Figure 3.17 (see Section 2.5.2). The effects of a permanent high and wide



Figure 3-16
'Ord' conditions near Cowden



Figure 3-17

Lower beach absent near East Newton (TA 270 378), allowing erosion of the till shore platform. Note the extensive eroded outcrop of a red clay layer.

beach were studied north of Hornsea sea defences. Only two slips occurred here in 18 months, and mudslides were the main degradation process, activated by seepage from sand layers and field drains. The mean slope angle was 34°, 5° lower than the mean slope angle of "degraded cliff" on the unprotected coast. The slopes were partly covered in vegetation (see Figure 3.18).

PLOTS AND STATISTICS

For determination of statistics the cases were split to distinguish between artificial and natural beach highs and lows. The results are given in Figure 3.19.

State of Beach	Rate of recession (m/yr)	Recent slip (%)	Degraded slip (%)	Degraded cliff (%)	Total slips (%)
Artificial low (S. Hornsea)	2.9	16.7	21.3	18.8	38.0
Artificial high (N. Hornsea & N. Withernsea)	0.6	1.5	11.7	45.3	13.2
Natural low ("ord")	2.1	11.9	13.7	23.3	25.5
Natural high (normal)	1.9	9.5	19.8	27.7	28.3

Figure 3.19 Mean values of selected variables for different states of the beach.

The largest differences occur between the artificial beach highs and beach lows, because these beach conditions are



Figure 3-18

Cliffs north of Hornsea, protected from erosion at the toe by a permanent high and wide beach. Zone "E"

permanent, whereas an "ord" will only affect any one place for a few months. The natural high beach is the "normal" situation with which the artificial high and low areas may be compared. In the artificial beach low south of Hornsea the value of "total slips" is higher, due mainly to an increase in "recent slips", and balanced by a decrease in "degraded cliff". The opposite effect occurs at the artificial beach highs where the value of "total slips" is lower, and of "degraded cliff" is higher. There is little change in the value of "till cliff".

The changes are illustrated in Figure 3.20 showing the coastwise plots of "recent slips" and "degraded slips", compared with the position of beach highs and lows. At the position of the high beaches north of Hornsea and Withernsea, "degraded cliff" is well above the mean value, and "recent slips" well below it (nearly zero). This is as would be expected from the study of the effects of "ords" and high beaches described earlier. The intense toe erosion in areas of low beach removes degraded slip debris, and the loss of support at the toe will be sufficient in some cases to cause a new slip, thus increasing "recent slips" and "total slips". In areas of high beach, the processes of mudslides and shallow slides cause the increase in "degraded cliff". The reduction in slope angle makes the cliff less liable to fail by slipping, resulting in the low value of "recent slips".

The rate of recession⁵ shows a steady increase from areas of artificial high beach, through normal beach, to artificial low beach, a consequence of the increase in rate of erosion from the cliff foot and shore platform (see Section 2.5.2). The rate of recession is plotted coastwise in Figure 3.14. The effect of "ords" is not shown by the plot, because as these are temporary they do not affect the long-term (11 year) rate of recession. However Pringle (1985) observed an increase in the short-term (6 month) rate of recession following the passage of an "ord". There are some unusual differences between the artificial high beaches at north Hornsea and north Withernsea. The statistics are summarized in Figure 3.21.

	Recession Rate (m/yr)	Degraded Slips (%)	Degraded Cliff (%)	Till Cliff (%)	Ridges (%)	Cliff Height (m)
North Hornsea	1.0	21.6	58.6	19.5	16.1	9.9
North Withernsea	0.4	6.0	37.6	54.0	50.0	8.8

Figure 3.21 Mean values of selected variables at artificial high beaches of north Hornsea and Withernsea.

The rate of recession is lower at Withernsea, possibly because the beach is higher and wider than at Hornsea, owing to its

5. Calculated between 1974 and 1985 from data supplied by Holderness Borough Council.

more southerly location. The values of "degraded cliffs" and "degraded slips" are higher at Hornsea, and the value of "till cliff" is higher at Withernsea. This may be explained by:

1. the greater cliff height at Hornsea, which will encourage slips;
2. the frequent occurrence of sand and gravel lenses at Withernsea, which will encourage falls by seepage erosion, hence increasing the value of "till cliff", and reducing "degraded cliff".

In conclusion the rate of cliff foot erosion does influence the mode of failure. At the position of artificial low beaches and "ords" the proportion of deep-seated slips and falls is increased, and of mudslides and shallow slides reduced. The effect of "ords" on the cliff morphology is less noticeable than the effect of artificial beach lows. The main role of "ords" is in removing degraded cliff and slip debris, and steepening the cliff prior to a new failure. Their presence may not result in an immediate increase in the rate of recession, but may cause a delayed increase. For a long-term high rate of recession to be maintained, it is essential that low beach conditions occur periodically at the cliff foot. At the position of artificial high beaches the proportion of mudslides and shallow slides is increased, and of recent deep-seated slips is reduced. Cliff recession in these areas is

continuing, but at a rate about one quarter that of the unprotected coast.

3.3 Coastwise Distribution of Response Variables

3.3.1 Slip Mode of Failure

The mean value of "total slips" for the coast was 26.7% and standard deviation 25.3%. The variable "total slips" is plotted coastwise in Figure 3.5, and shows several areas of high and low value. The distribution of these was compared with that of the control variables, and the results are summarized in Figure 3.22.

Areas with a low value of "total slips" are associated with:

1. low cliff height, frequently with a high proportion of sand;
2. the presence of easily eroded layers at the cliff foot, eg. the red clay.

All ten areas with a low value of "total slips" showed a change to a higher value of "till cliff", and the fall mode of failure. Areas with a high value of "total slips" are associated with:

Case Number	Total Slips	Cliff Height	Sand/Gravel	Red Clay Layer	Stone-Free Clay	Comment
0-25	Low	Low	Present	Present		Falls common
45-55	Low	Low				Cliff degraded
73-98	Low	Low				Falls common
174-185	Low					Falls common
205-223	Low					Falls common
265-281	Low		Present	Present		Falls common
346-378	Zero	Low				
446-455	Low	Low				
515-577	Low	Low				High beach
145-160	High	Low	Present	Present		Intense erosion
185-205	High	High				Degraded slips
248-265	High	High				Degraded slips
292-312	High	High				Degraded slips
321-327	High					Degraded slips
333-346	High					Degraded slips

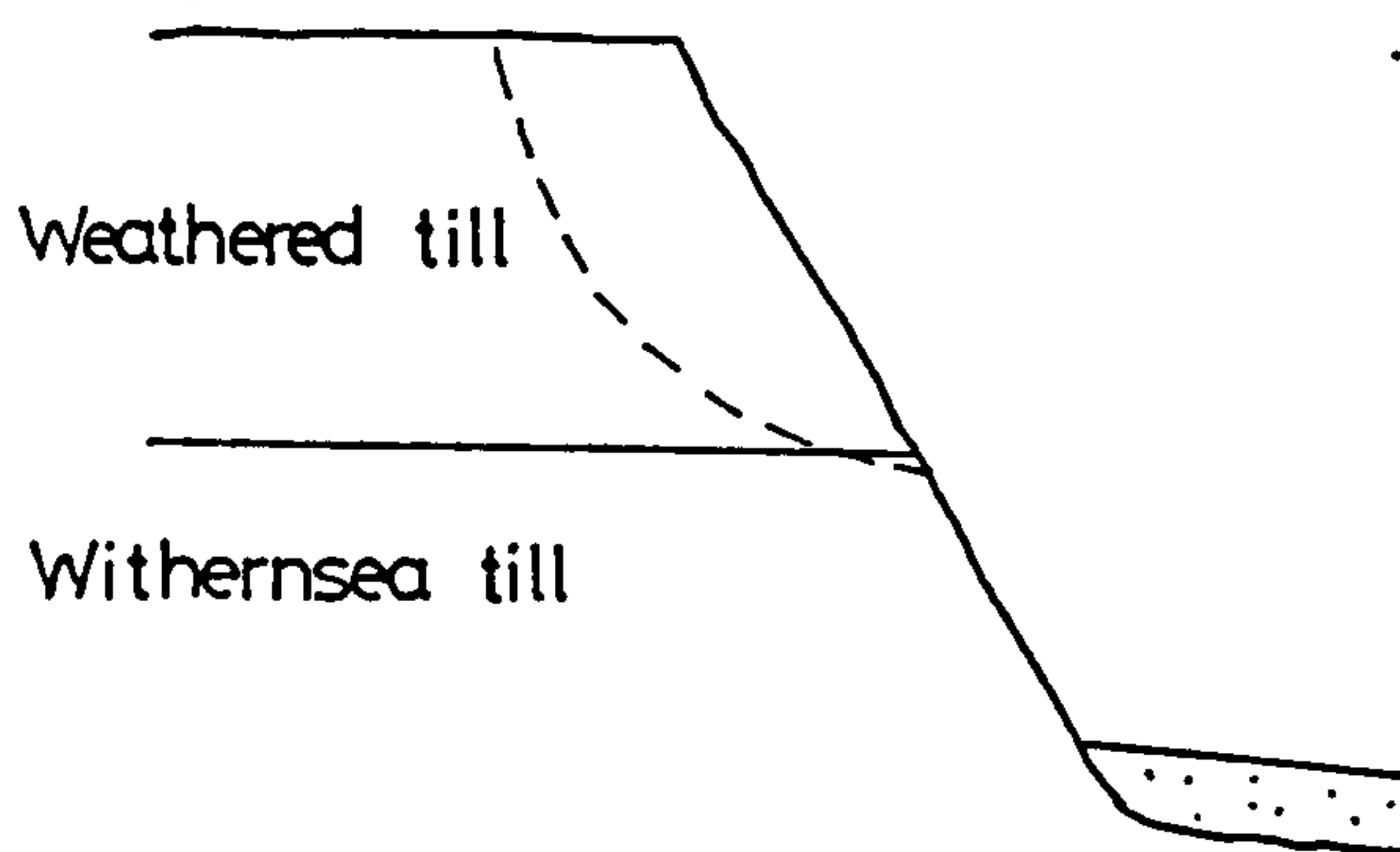
Figure 3.22 Table comparing areas of low and high "total slips" with control variables.

1. the presence of "stone-free" clays at the cliff foot;
2. intense toe erosion at the artificial low beach south of Hornsea.

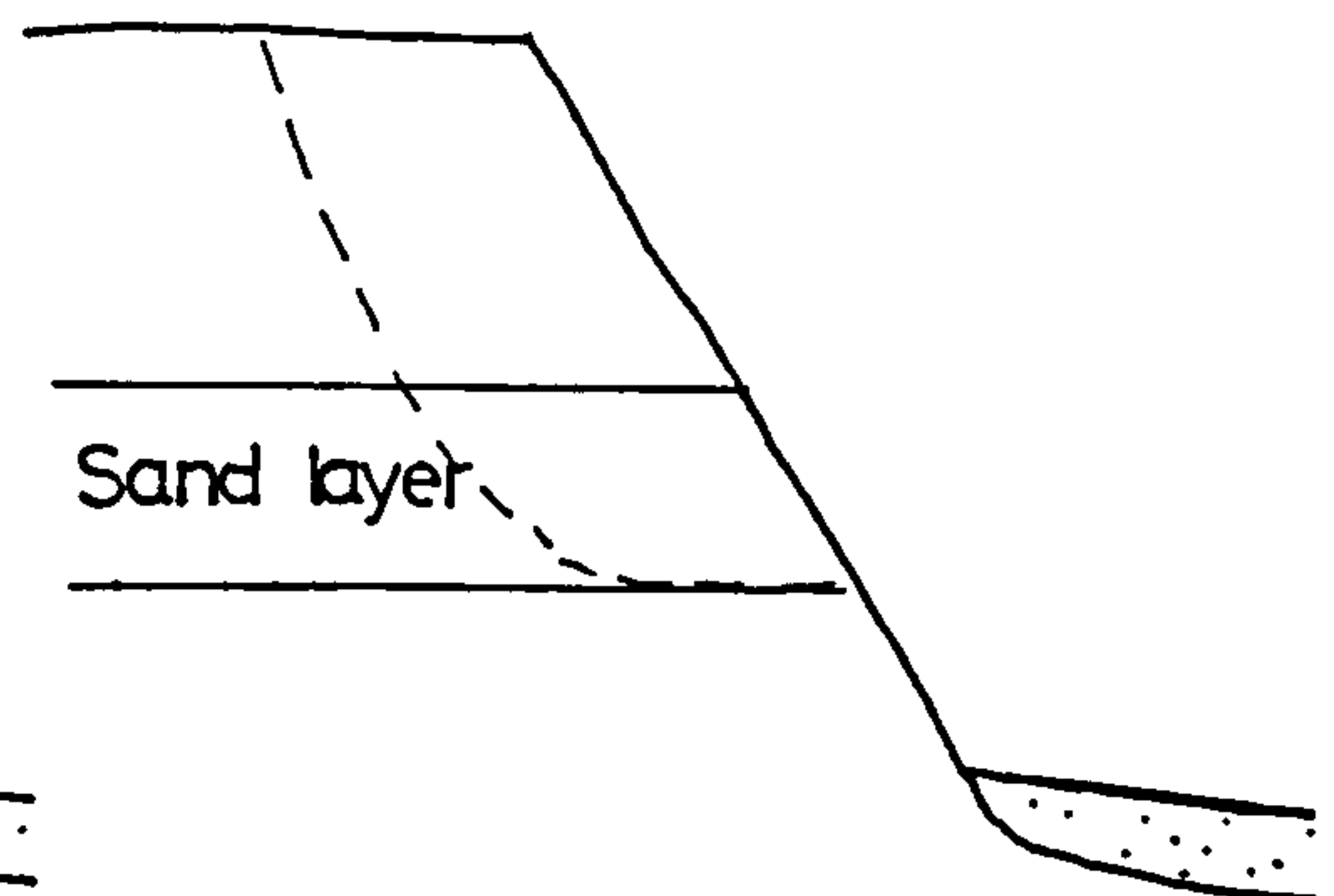
These findings are in agreement with those of the earlier section on control variables, however there are a few areas of high and low value of "till cliff" which cannot be explained by the known controls. It is possible that these areas are temporary in nature, perhaps caused by the recent passage of an "ord", or higher than usual natural beach conditions. However they could be permanent, caused by some unseen control, for example high groundwater levels in the underlying Chalk (see Section 4.3.2).

The slips are mainly of the rotational and compound types, and may be further classified as slope, toe and base failures, and examples are shown in Figure 3.23. The most common slips are rotational toe failures, deduced to be of non-circular form, judging from examination of old slip surfaces. This is confirmed by surface ground movement measurements at the Grimston site (see Section 4.3.9). The position of the slip surface of toe failures was not observed to be affected by the till boundaries, in contrast with the observation of Williams (1982) that the slip surface sometimes emerged at the Skipsea/Withernsea till boundary. However the exact position of the toe is often obscured by the beach, and can only be

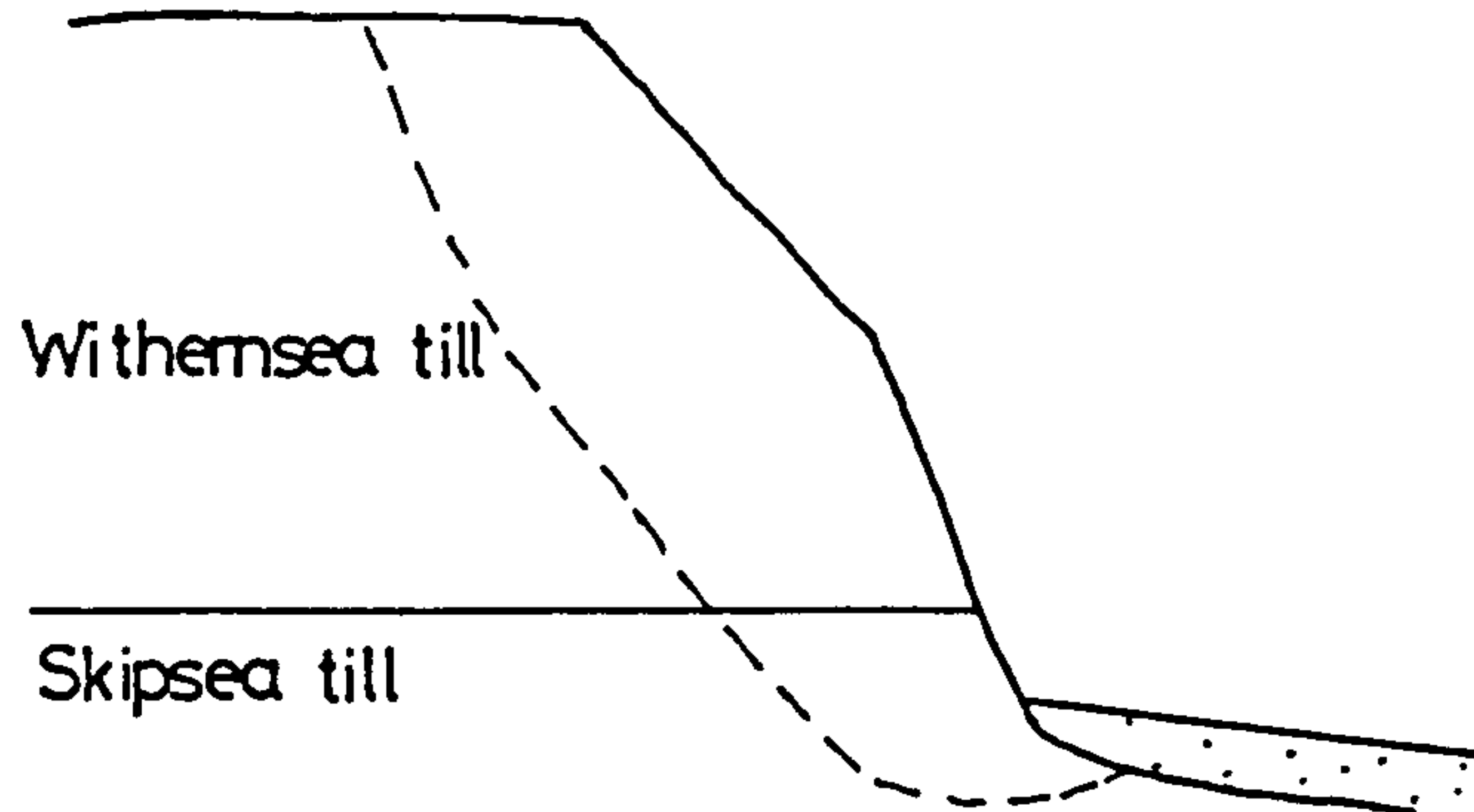
Slope failure



Beacon hill, Ringbrough (TA 279 365)



Toe failure (eg. Grimston site) TA 284 359)



Base failure (eg. South Aldbrough TA 260 393)

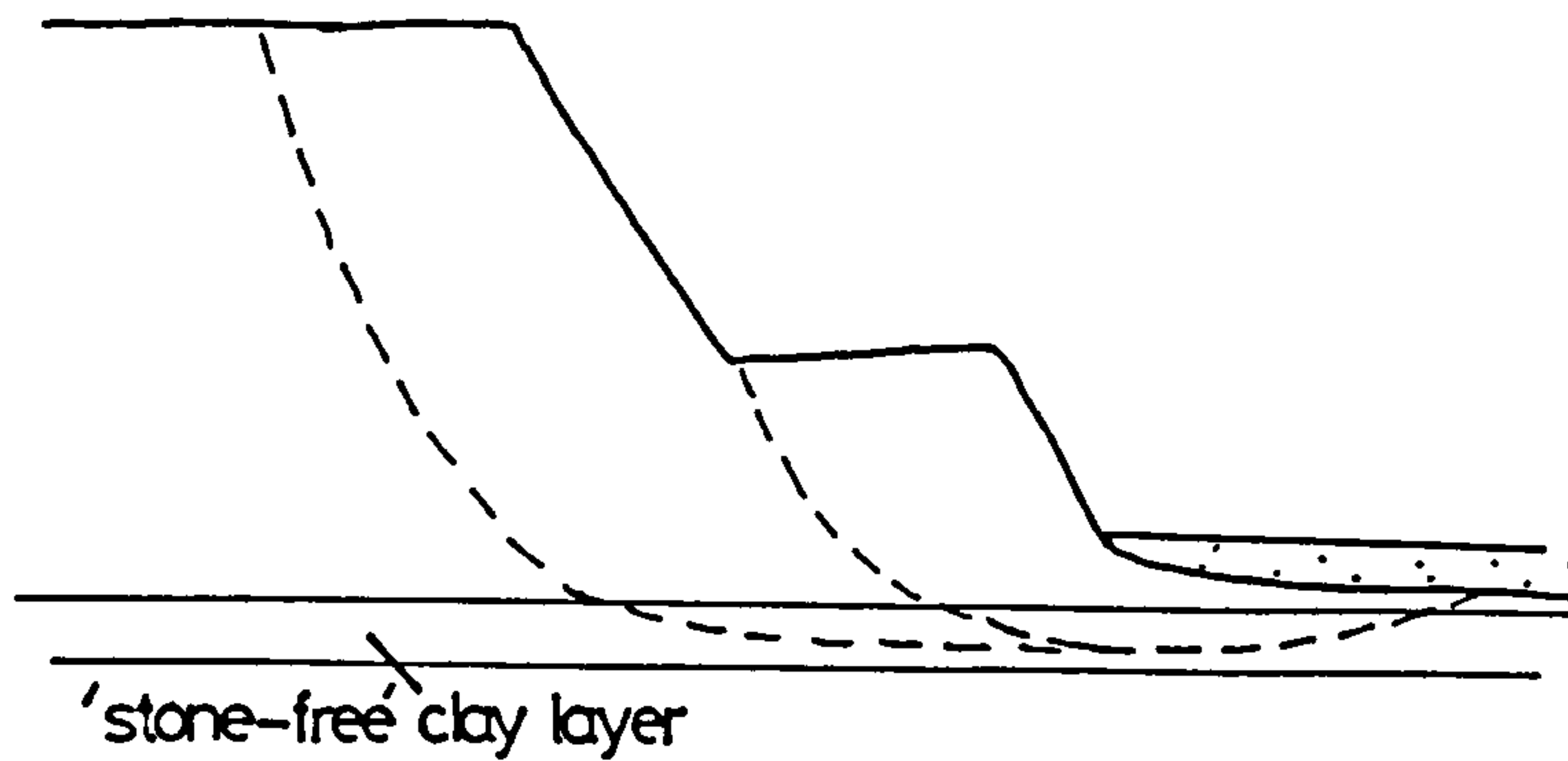


Figure 3.23

Examples of slope, toe
and base failures

Not to scale

--- slip surface

reliably determined from recent slips, unless it is marked by a prominent feature (eg. a ridge on the foreshore).

Small slope failures in the weathered zone are common along the coast. Occasionally slope failures are caused by geological conditions, eg. where a large sand layer contained the slip surface, as at Beacon Hill, Ringborough (see Appendix C). This mode of failure is similar to that described by Quigley et al. (1977) at Iona, Great Lakes, where instability occurred in the upper part of the cliff due to the presence of a sand layer. Rotational or compound non-circular base failures occur most frequently where the "stone-free" clay layers contain part of the slip surface, and are often multiple in form.

The mean landslip length/cliff height ratio is between 1.5 and 2.0 for toe failures, and increases to between 3.0 and 4.0 for base failures. Toe failures show an initial displacement after failure of 1-2m, and then continue to slide at a slow rate, taking between several months and years to be entirely removed. This behaviour is in accordance with the non-brittle stress-strain characteristics of the till, a small displacement being sufficient to restore stability.

3.3.2 Cliff Morphology

"Till cliff" occurs everywhere along the coast (mean 44.1%), but the proportion is highly variable (standard deviation

34.0%). In some areas it occurs as a stage in a cycle of failure, where the cliff is steepening prior to a new slip; in others it is permanent with the cliff receding by falls. The coastwise distribution of "till cliff" is shown in Figure 3.4, and was compared with the distribution of the "control" variables. Areas with a low value of "till cliff" occur where the beach is high, or where there are long stretches of "degraded slips" (see Section 3.2.2). Areas with a high value of "till cliff" occur where the cliff height is too low for slips. The distribution is thus the converse of that of "total slips". The mean slope angle measured in areas of "till cliff" was 48° .

Degraded cliff is formed in two main ways. Above recent or degraded deep-seated slips it is formed by falls or shallow slips from the weathered zone, typically making up 20% of the cliff. At the high artificial beaches north of Hornsea and Withernsea it is formed by shallow slips and mudslides, making up 50% of the cliff, as the debris is not removed by toe erosion.

The coastwise distribution of "ridge index" is shown in Figure 3.24. Ridges are most common in areas of low or medium cliff height ($<12.1\text{m}$), with frequent sand layers and a moderate rate of recession. In areas with a high rate of recession they are undercut and collapse, and in areas with a low rate they are degraded by falls and mudslides. Where falls are the dominant mode of failure they often occur as a ridge separating

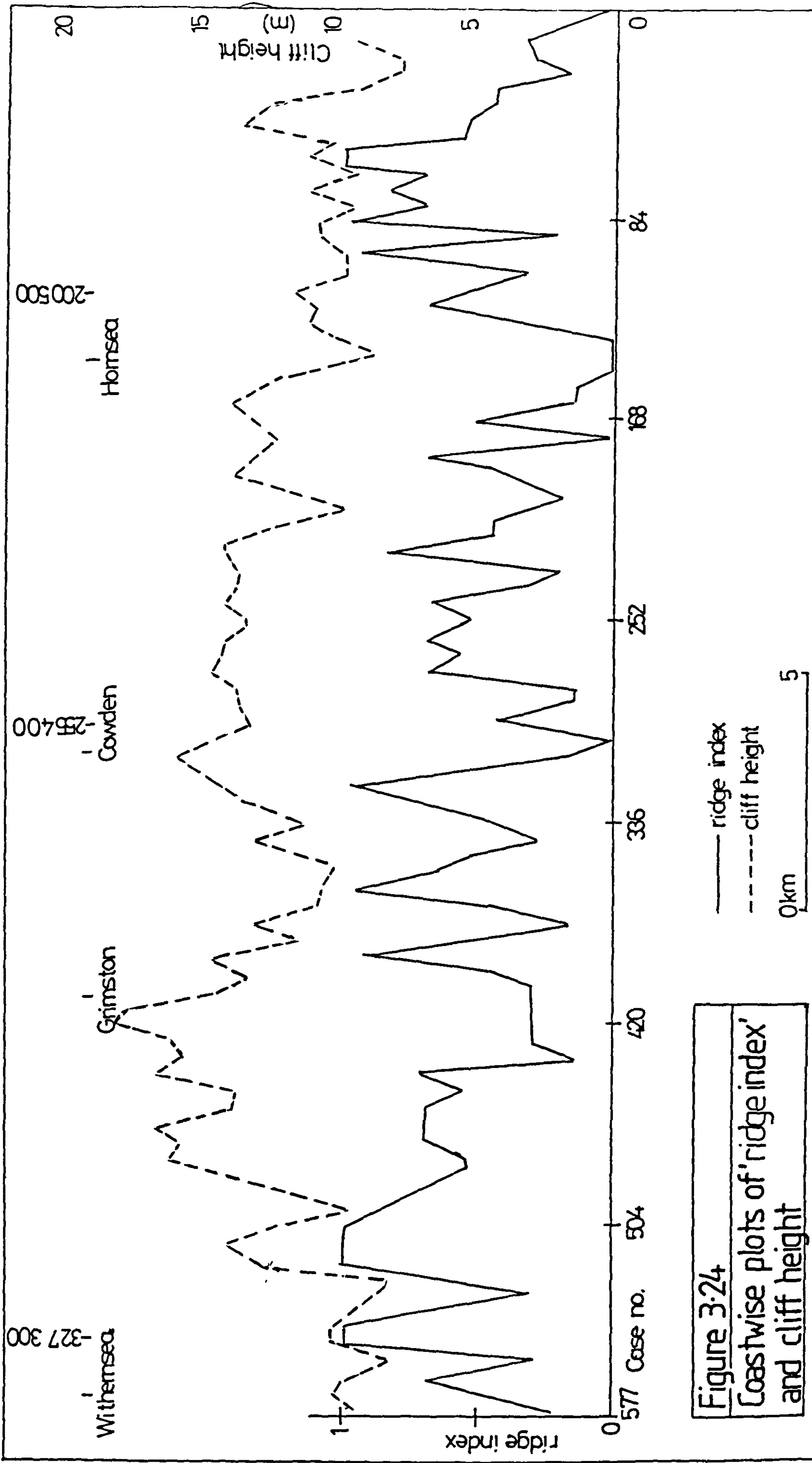


Figure 3.24
Coastwise plots of 'ridge index'
and cliff height

embayments (which are formed by seepage erosion from a sand layer (see Figures 3.28 and 3.29). In areas where slips are the dominant mode of failure, they occur at random, as the ridge of till left in situ between adjacent slips (see Figure 3.31).

3.3.3 Re-Examination of Sections of the Coast

In the summer of 1987 eight sections of the coast were re-examined, each of approximately 1km length. The survey photographs were taken into the field and compared with the cliff, allowing the effects of one year of marine erosion to be assessed. The amount of change in a section was classified into:

1. no or little change, cliff morphology unaltered;
2. some change, ie. falls, slips and degradation of old slips;
3. large change, ie. complete change of morphology, cliff face in photograph unrecognizable.

The location of the sections and the amount of change in each is shown in Figure 3.14, together with the state of the beach. Descriptions of the changes in each section are given in Appendix F. Large changes occurred in the artificial beach low south of Hornsea, elsewhere there has been some change, and



(a) Cliffs near East Newton (TA 269 379)
Note "till cliff" morphology and falls of brown Withernsea till

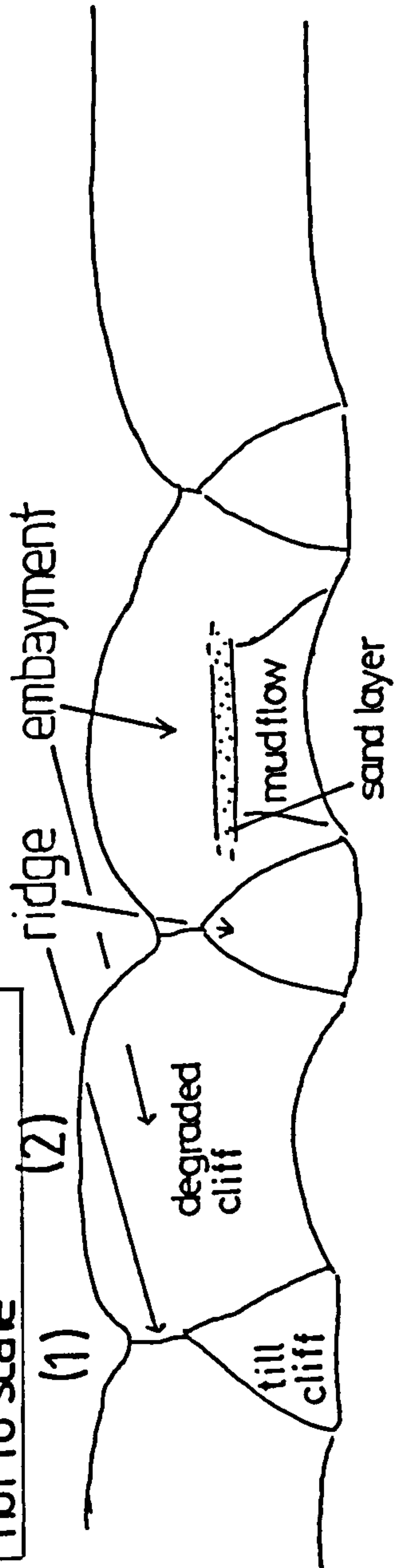


(b) Cliffs near East Newton (TA 269 379), with arete /corrie morphology.
Note seepage from sand deposit on the right side of the corrie.

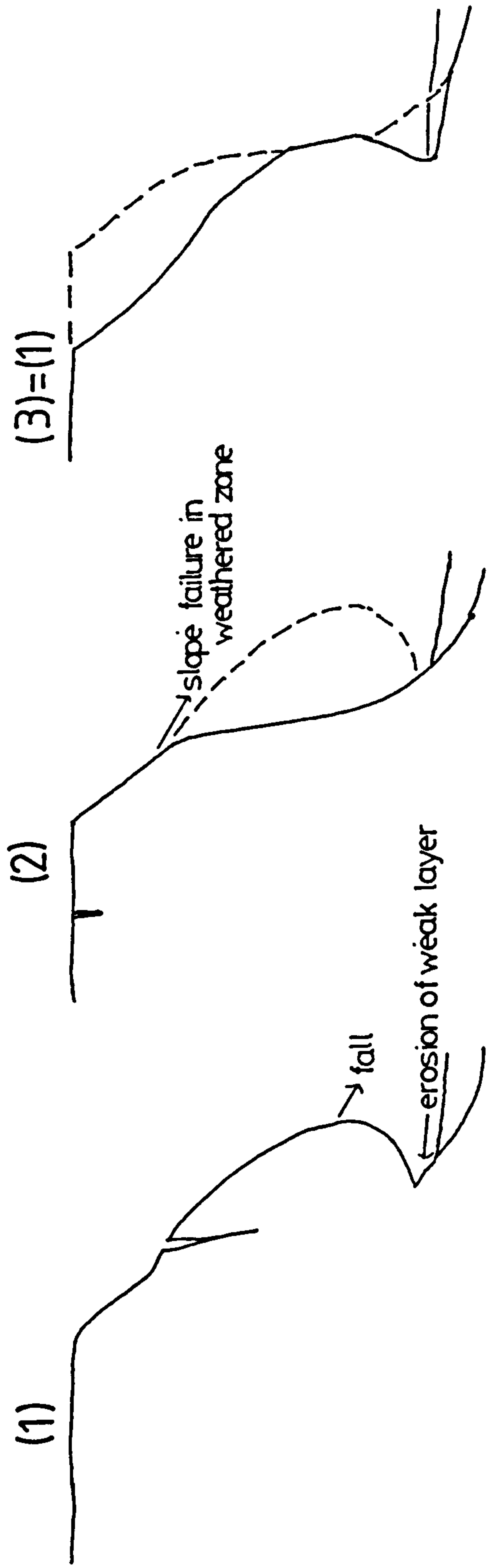
Figure 3.28

Typical cliff morphology in zone "A"

Figure 3:29
Cliff morphology and cycle
of recession in zone "A"
not to scale



Cliff Section



Changes in profile as cliff recedes (previous profile dashed)

north of Hornsea and Withernsea, little change. This result demonstrates the importance of the beach in controlling the rate of erosion and recession of the cliff.

3.3.4 Summary of Correlations Found

The study of the "control" and "response" variables described above shows that several correlations exist between them. These are summarized overleaf in Figure 3.25. Most of the variations in the "response" variables can be explained by these correlations with the "control" variables. The exceptions may be due to "unseen" controls, or temporary changes in morphology and mode of failure. Control variables are listed vertically and response variables horizontally. If there is a positive correlation between them (ie. the response variable increases if the control variable increases), a '+' is indicated, and similarly a '-' for a negative correlation (ie. the response variable decreases if the control variable increases). If there is no correlation this is indicated by 'NC'.

3.4 Zoning of the Coast

It was decided to divide the coast into zones, to clarify and summarize the relationships described above, and for planning of future investigations. Five major zones were defined, on

	<u>Response Variables</u>					
Control Variables	Total Slips	Degraded Slips	Till Cliff	Degraded Cliff	Ridges	Falls
Cliff height	+	NC	-	NC	-	NC
Beach thickness	-	+	-	+	-	NC
Sand and gravel layer present	-	NC	+	NC	NC	+
Stone free clay layer present	+	+	-	+	NC	NC
Red clay layer present	NC	NC	NC	NC	-	+

Figure 3.25 Summary of control and response variables.

NC = No Correlation.

the basis of beach and geological conditions, cliff morphology and mode of failure. The zones were defined by examining the coastwise plots of the variables in the earlier sections, and the process of defining them is described below with reference to Figure 3.26 (located in map pocket in the back cover). If a variable took a value significantly above or below the mean value on any section of coast, this was indicated on the diagram by a box. "Sections" of the coast with the same mode of failure and morphology were defined by these changes in the variables. Thirty-seven sections were defined and their boundaries are indicated on the diagram by vertical dashed lines. Several "sections" showed only slight differences in morphology, and so were grouped into 10 "sub-zones" (numbered 1-10), shown at the bottom of the diagram. Several of the "sub-zones" appeared to be influenced by a common factor (eg. soil type), with the differences between them caused by small variations in cliff height or beach conditions. These "sub-zones" were grouped together to form the five major "zones" (designated A-E) shown at the bottom of the diagram. The mean value of variables for the zones and sub-zones are given in Figure 3.27 (located in map pocket in the back cover), and the individual zones are now discussed with reference to these statistics and Figure 3.26.

3.4 Zoning of the coast

3.4.1 Major Coastal Zones

Zone "A"

Zone "A" forms 45.9% of the coast, and occurs at intervals along its entire length. The principal mode of failure is by falls. The value of "total slips" is low (mean 15.5%), and the main cliff morphology is "till cliff" (mean 56.8%). There is also a high proportion of "degraded cliff" (mean 27.4%). Sand lenses are frequent (mean 4.1%), which may cause falls and the formation of "ridges" and "embayments", by seepage and marine erosion (see Section 3.2.1). The distribution of sections with a high proportion of "ridges" corresponds closely with zone "A". Most of the zone "A" sections are of low cliff height (mean 11.0m), and this is thought to be the main control on the mode of failure. The mean values of "total slips" and cliff height are cross-plotted for the sub-zones of zones "A" and "B" in Figure 3.13. It can be seen there is a sharp rise in "total slips", and increase in cliff height, for "B" sub-zones "5" and "10" compared with "A" sub-zones "1", "3" and "8". Sub-zone "7" is unusual, probably because a very high beach occurred there at the time of the survey. This would cause cliff height to be underestimated, and increase the proportion of "degraded slips", and hence "total slips".

The typical cliff morphology in zone "A" is shown in Figure 3.28, and the cycle of recession illustrated in Figure 3.29. The slope maintains a steep profile of "till cliff" throughout

the cycle, and the slope angle alternates between approximately 40° (profile 1) and 65° (profile 2), as falls occur from the lower and upper cliff respectively. If embayments develop these may contain "degraded cliff" at an angle of 25° to 30° . The position of the embayments may be controlled by the presence of a sand layer, causing mudslides and seepage erosion. If so they will not shift position laterally as the cliff recedes. Otherwise the positions of ridges and embayments may interchange.

The three sub-zones forming zone "A" ("1", "3" and "8") show differences due to changes in cliff height and beach conditions. In sub-zone "1" the beach is relatively high, causing a low rate of recession, and the cliff height is low. These factors result in a low proportion of "total slips". In sub-zones "3" and "8", the rate of recession and cliff height increase, causing the proportion of "total slips" to rise.

Zone "B"

Zone "B" forms 24.5% of the coast, and occurs in two distinct areas, between Hornsea and Cowden, and between Grimston and Monkwith. The principal mode of failure is by slips (mean 37.6%), although secondary degradation of the slipped soil masses and the cliff between them occurs by falls and mudslides. The slips are mainly non-circular rotational toe failures. The main control on the mode of failure is thought to be the cliff height, which is generally high. The influence

of minor soil types is small. The typical cliff morphology is shown in Figure 3.30 and the cycle of failure in Figure 3.31. The cliff morphology goes through a cycle of "recent slip" (profile 1), "degraded slip" and "degraded cliff" (profile 2), "till cliff" (profiles 3 and 4) and returns to "recent slip". The slope angle varies from a minimum of 35° during degradation, to a maximum of 50° just before failure. The positions of the slips alternate with the positions of "till cliff" and "ridges" which occur between them, as the cliff recedes. The cycle of failure is discussed in more detail at the Grimston site (in zone "B") in Section 5.5.

There is little difference between the two main sub-zones of zone "B" ("5" and "10"). Sub-zone "10", located between Grimston and Monkwith, has a greater cliff height than "5", although the value of "total slips" is the same. This is contrary to the increase in "total slips" with cliff height observed in zone "A", and suggests that above a certain cliff height, the value of "total slips" is constant. The ratio of "recent slips" to "degraded slips" is high in sub-zone "5", probably because the beach is low and toe erosion is severe, between Hornsea and Rolston.

Zone "C"

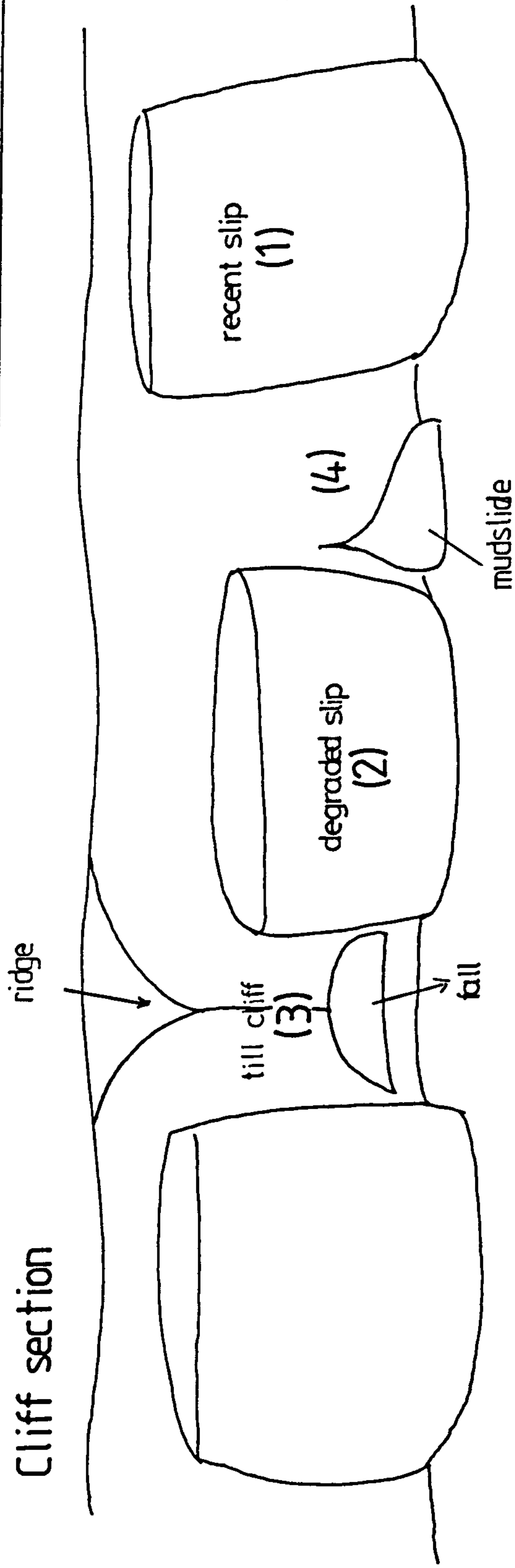
Zone "C" forms 16.3% of the coast and occurs mainly between Cowden and Grimston. The principal mode of failure is by slips (mean 48.8%), and their form is influenced by the presence of



Figure 3.30

Typical cliff morphology in zone "B" near the Grimston site (TA 284 358). Note "degraded cliff" to left, "recent slip" centre and "till cliff" to right.

Cliff section



Changes in profile as cliff recedes (previous profile dashed)

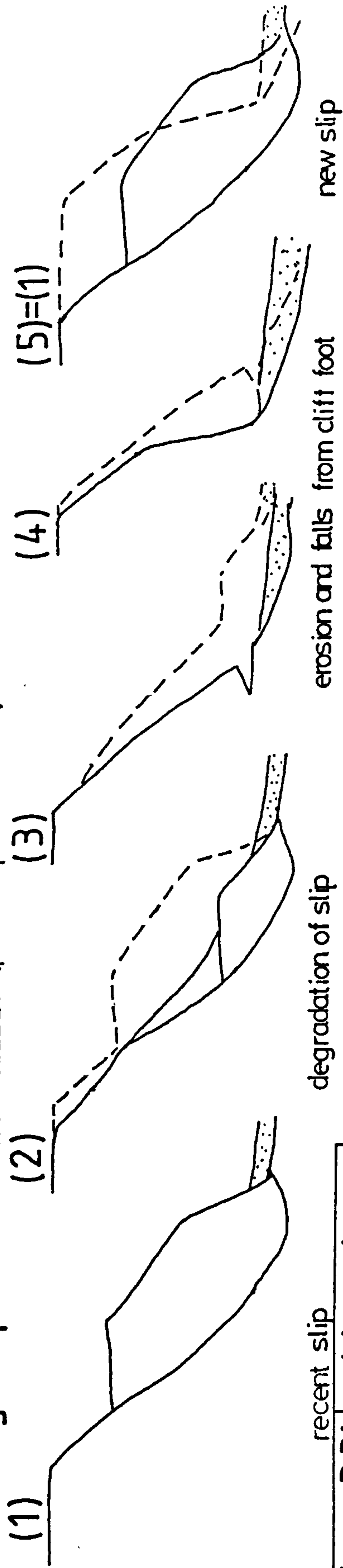


Figure 3.31 not to scale
Cliff morphology and cycle of recession in zone "B"

"stone-free" silty clays occurring near the cliff foot (see Section 3.2.2). They are non-circular rotational or complex base failures. The cliff morphology is shown in Figure 3.8, and the cycle of failure is illustrated in Figure 3.32. The cliff morphology of long degraded slips and degraded cliffs remains near constant as the cliff recedes. A new slip may occur before the previous slip is entirely removed, resulting in a multiple failure. The slope angle is low throughout the cycle, alternating between 20° (profile 2) and 40° (profile 1). The correlation of zone "C" sections with high values of "total slips" and high "degraded slips" is shown in Figure 3.26, and is confirmed by the statistics in Figure 3.27. The occurrence of zone "C" sections between Cowden and Grimston explains the absence of any zone "B" sections here.

Zone "D"

Zone "D" forms only 3.0% of the coast, where large layers of sand occur at Skipsea and Beacon Hill, Ringbrough. These areas are described in Appendix C.

In general the sand layers can be eroded by seeping groundwater or by marine erosion. For example near Skipsea a large embayment has been caused by marine erosion of a sand layer near the cliff foot. Rapid erosion of a sand layer at the cliff foot of the Lake Erie north shore was noted by Quigley et al. (1977).

Cliff section

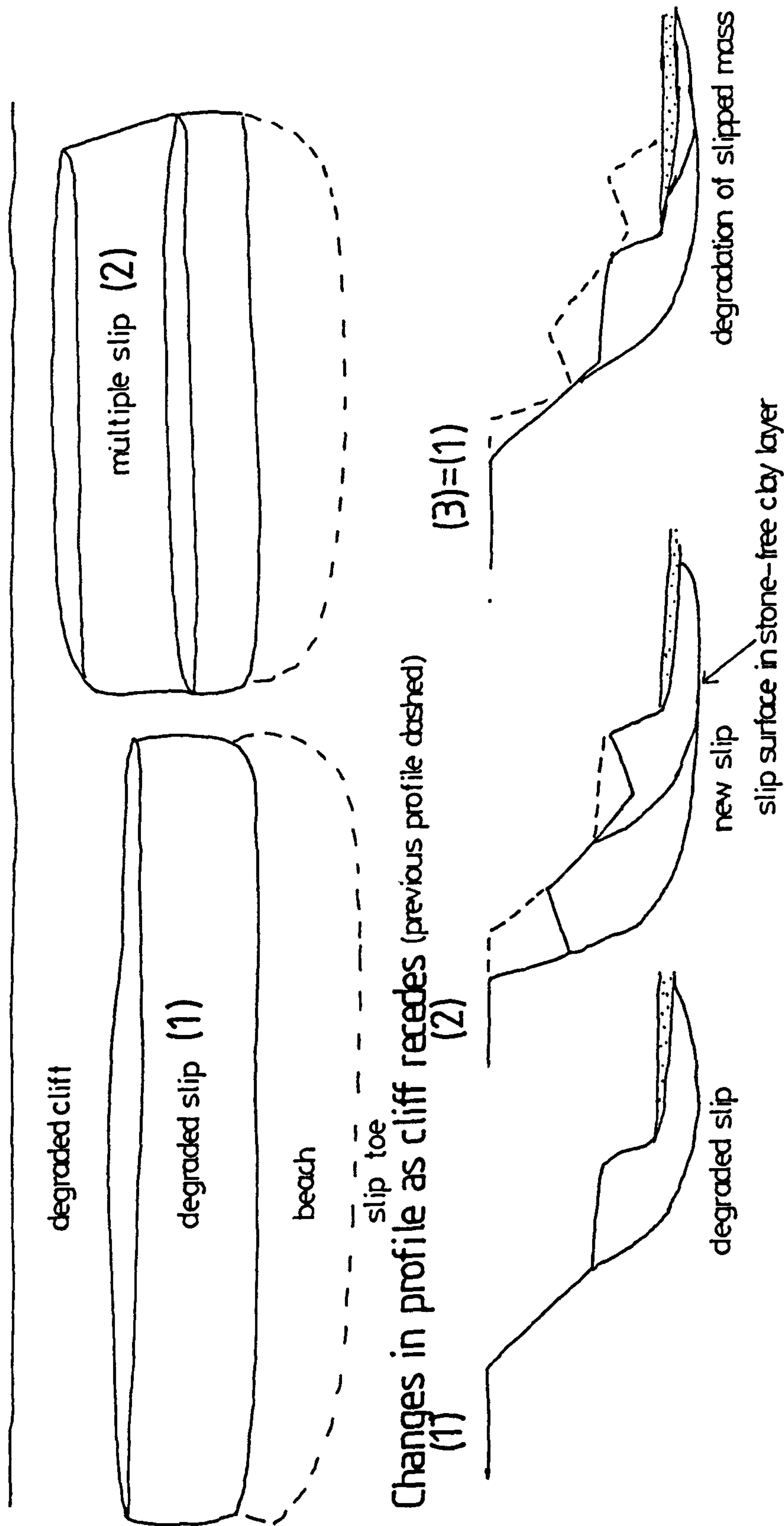


Figure 3.32 not to scale
Cliff morphology and cycle
of recession in zone "C"

Erosion by seeping groundwater can lead to falls in the overlying clay, as at Mappleton and between East Newton and Ringbrough. There are large sand layers at Ringbrough which cause some complicated failure mechanisms. Some of the slides emerge in sand layers at mid height in the cliff. Similar behaviour has been observed at Iona, Lake Erie by Quigley et al. (1977).

Zone "E"

Zone "E" forms 11.8% of the coast and occurs at the high beaches north of Hornsea and Withernsea sea defences. There is a high proportion of "degraded slips" and "degraded cliff", as toe erosion occurs only intermittently. The typical cliff morphology is shown in Figure 3.18. There is no well defined cycle of recession in zone "E". The cliff recedes mainly by shallow slides and mudslides activated by seeping groundwater from sand layers and field drains. The slope angle is lower than an unprotected coast (mean 34°) and the slopes are partly vegetated.

The slopes in zone "E" are approaching Type 3 behaviour of Hutchinson (1973), where there is no removal of debris and the cliffs undergo the process of free degradation. There is some similarity with the cliffs at Patrick Point, Lake Erie, which are slowly retreating by surface sloughing and very shallow slides. However at Patrick Point desiccation and re-wetting appears to play a prominent role in causing mudslides (Quigley

et al., 1977), which does not appear to be the case at Holderness.

3.5 Conclusions

The survey has shown that the coastwise variation of cliff morphology and mode of failure can be explained by the influence of a small number of controls. The chief ones are cliff height, state of the beach, and the presence of minor soil types within the tills. The influence of these controls on the five main coastal zones is summarized in Figure 3.33.

It would be desirable to extend the survey beyond the two-year period of this study by a similar survey in the future (see Section 6.3.3). At any place the cliff morphology may remain constant as the cliff recedes (zones "A", "C" and "E"), or change with time as part of a cycle of failure (zone "B"). The rate at which changes occur is erratic over short periods (months), mainly because of the influence of varying beach levels and storms. However over periods of several years there appears to be a near constant rate of recession.

The investigations at the sites of Rolston and Grimston were both in Zone "B". The cycle of failure identified in this zone by the survey is a useful addition to these investigations, because they were limited to an 18-month period and therefore

Zone	Percentage of Coast	Mode of Failure	Cliff Morphology	Main Control
"A"	45.9%	Falls, shallow slips and direct marine erosion	"Till cliff" standing at 40°-65°	Low cliff height and presence of 'sand' and 'red clay' layers
"B"	24.5%	Single rotational slips with secondary falls and mudslides	Cycle of "recent" and "degraded slips", "degraded cliff" and "till cliff" varying between 35° and 50°	High cliff height
"C"	16.3%	Rotational or complex non-circular slips with sole in "stone-free" clay layer	Permanently "degraded slips" or "degraded cliff" varying between 20° and 40°	Presence of "stone-free" silty clay at level of cliff foot
"D"	3.0%	Direct marine erosion and rotational slips with toe emerging in sand layers in the cliff face	"Degraded cliff"	Presence of large sand layers
"E"	11.8%	Shallow rotational slips and mudslides	"Degraded cliff" and "degraded slips" at mean slope angle of 34°	High artificial beach level

Figure 3.33 Main controls on coastal zones.

covered only part of the cycle. In the next chapter the investigations at the two sites are described.

CHAPTER 4

SITE INVESTIGATION

4.1 Aim and Scope of the Site Investigation

The principal aims of site investigation in coastal landslides were listed by Hutchinson (1983) to be to establish the site geology, the actual or potential mechanisms of failure and the hydrological conditions. Answers should be sought to questions such as: does instability involve first time sliding or a renewal of movement?; are the piezometric conditions short-term, intermediate or long-term?; what shear strength parameters obtain?

To achieve these aims, at two sites Rolston and Grimston:

- instrumentation was installed to measure pore water pressure and surface ground movement;
- soil profile and permeability were determined by field tests, geological mapping and soil description;
- failure mechanisms were identified by surveying cliff and beach changes;

- index properties, strength and consolidation characteristics were measured by laboratory tests.

This chapter describes the site investigation fieldwork and laboratory testing.

4.1.1 Selection of Sites

The location of the two sites is shown in Figure 1.1. They are both in zone "B" areas (see Chapter 3) where the single rotational slip mode of failure is common and is unaffected by the occurrence of unusual soil types. Sand layers are present at both sites, but field observations show that they do not affect the mode of failure. It was not possible to find sites without sand layers.

Areas in zone "A", where cliff height is low and falls predominate were avoided. Analysis of falls would be more difficult, and physical changes smaller and more difficult to measure. However as zone "A" represents nearly 50% of the coastline this is a considerable omission (see Suggestions for Further Research, Section 6.3.2). The Building Research Establishment has an instrumented site at Cowden (see Figure 1.1), in Zone "C", where minor soil types influence the form and behaviour of the slips. Zones "D" and "E" were thought unsuitable for investigation, because of the presence of unusual soil types (large sand layers) and the artificial

cessation of erosion respectively. Further restrictions were imposed on the location of sites by the need for security, away from towns or caravan parks, and reasonable vehicular access.

4.2 Site Description and Ground Conditions

4.2.1 Site Description

A fold out plan and cross-section of the Grimston site is given in Figure 4.1, and the general features shown on Figure 4.2. The site is in a field at Moat Farm, Grimston (TA 284 358). Cliff height is 20m and the clifftop, which is level, is at approximately 24m O.D. The cliffs are formed entirely of glacial deposits with rockhead at -33m O.D. Withernsea till overlies Skipsea till, with the boundary at 8m O.D. Two deep seated rotation slips ("D" and "F") occurred prior to the start of the study. The slipped soil masses are now displaced several metres down the slope, and are being degraded, together with the unslipped cliff between them, by falls, mudslides and toe erosion. The mean annual rate of cliff recession between 1974 and 1985 is 2.4m/yr.

A fold out plan and cross-section of the Rolston site is given in Figure 4.3, and the general features shown on Figure 4.4. The site is in a field owned by Humberside County Council (TA 221 451). Cliff height is 15m and the clifftop, which is level, is at approximately 18m O.D. The cliffs are formed



Figure 4.2
General features of the Grimston site



Figure 4.4
General features of the Rolston site

entirely of glacial deposits with rockhead at -24m O.D. Skipsea till is the only till exposed in the cliff. Failure is by rotational slips, falls and mudslides. Slipped soil masses are rapidly broken up and large falls from the lower cliff are frequent, because erosion at the toe is severe. The beach is relatively low because of the proximity of the site to Hornsea sea defences. The mean annual rate of recession is 2.4m/yr.

4.2.2 Soil Sampling, Description and Profile

U100 samples were taken from three boreholes at each site, drilled by Soil Mechanics Ltd., with a light percussion drilling rig. One borehole at each site extended to the depth of the cliff foot, and was continuously sampled to obtain a complete soil profile. Samples were taken at an average 2m spacing in the other boreholes. The samples were sealed and transported to the University for testing. The location and depth of the boreholes (numbered 1-3 at Rolston and 4-6 at Grimston) are shown on the site plans. The U100 samples were split and described after specimens had been taken for laboratory testing.

Geological mapping and soil description at the cliff face was carried out. Plans of the cliff face at Rolston and Grimston are shown in Figures 4.5 and 4.6. The sample descriptions, cliff face plans and static cone penetrometer, permeability (see Section 4.4) and index test results were examined together

for each site by plotting the information on transparent overlays. Soil boundaries were assumed to be horizontal beneath the site, as was indicated by the cliff face exposures. Idealized soil profiles are shown on the logs of Figures 4.7 and 4.8. They are used to predict zones of permeability in Section 5.1.1, and to select shear strength parameters for stability analysis in Section 5.3.3.

4.2.3 Index Tests

Water contents of small samples of soil taken from the end of the sampling tubes were determined soon after arrival of the samples in the laboratory. The variation of water content with depth is shown on the soil profiles. There is a slight decrease with depth, from about 18% near ground level to 16% near the foot of the cliff. These values are an average of 2% higher than those determined at the inland Cowden test bed site (Marsland and Powell, 1985). This difference may be caused by use of a different test procedure, or possibly by swelling of the soil near to the cliff. Between ground level and 5m depth, samples in several of the boreholes show a decrease in water content by 1-2%. This may be due to desiccation of near surface clay, and is consistent with the high values of c_u indicated by the static cone penetrometer tests at this depth.

The undrained shear strength (c_u) was determined on samples from a range of depths by unconsolidated undrained triaxial

tests. The tests were done in accordance with B.S. 1377 (1975) on samples of 38mm and 100mm diameter. The c_u values are plotted with depth in Figures 4.9 and 4.10, together with values from static cone penetrometer tests (see Section 4.4.1). Water content and c_u are plotted together in Figure 4.11, showing the value of c_u to be sensitive to small changes in water content. The results broadly agree with those from tests on tills at Cowden (Marsland and Powell, 1985).

The Atterberg limits determined on samples of till from several depths are shown on the soil profiles. They are plotted on a plasticity chart in Figure 4.12, and indicate a clay of low to medium plasticity, slightly above the A-line. Water content is at or slightly below the plastic limit, which in tills is a consequence of the wide range of particle sizes, well packed during deposition (Marsland and Powell, 1985).

The particle size distribution curves determined from some till samples are shown in Figure 4.12. The clay content is typically 30%, and sand and gravel content 32%. The curves show a well-graded soil, with little difference between till types. In a clayey sand layer at Rolston the sand content increased to 46%. A more detailed study of particle size distribution by Gilroy (1981), showed that Skipsea till is generally slightly coarser than Withernsea till, and the effect of weathering is to reduce the sand content and increase the silt and clay content.

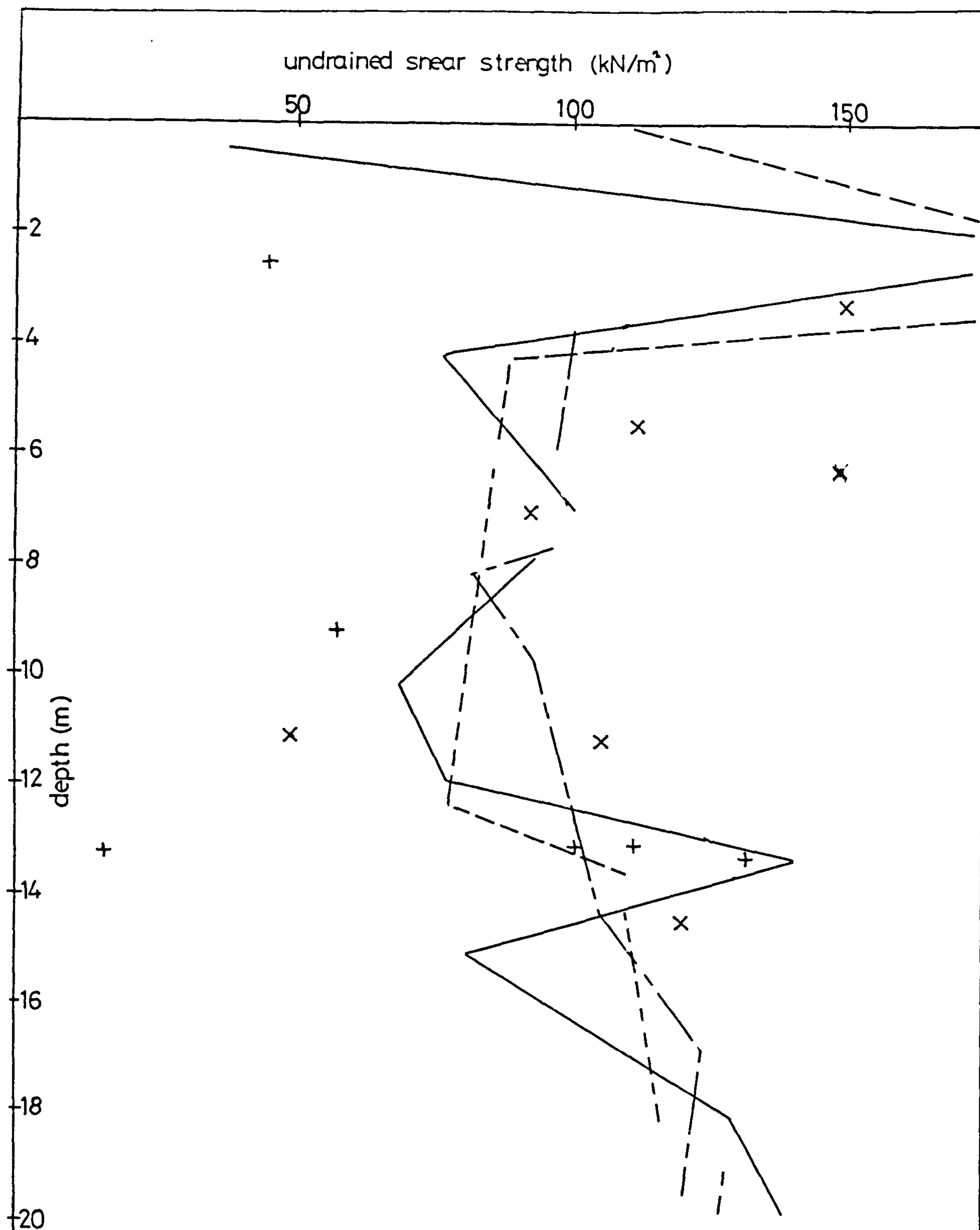


Figure 4.9

Profiles of undrained shear strength with depth from unconsolidated undrained tests and static cone penetrometer tests at Rolston

--- TEST 1 × 100 mm diameter
 — TEST 2 + 38 mm diameter
 -.- TEST 3

undrained shear strength (kN/m^2)

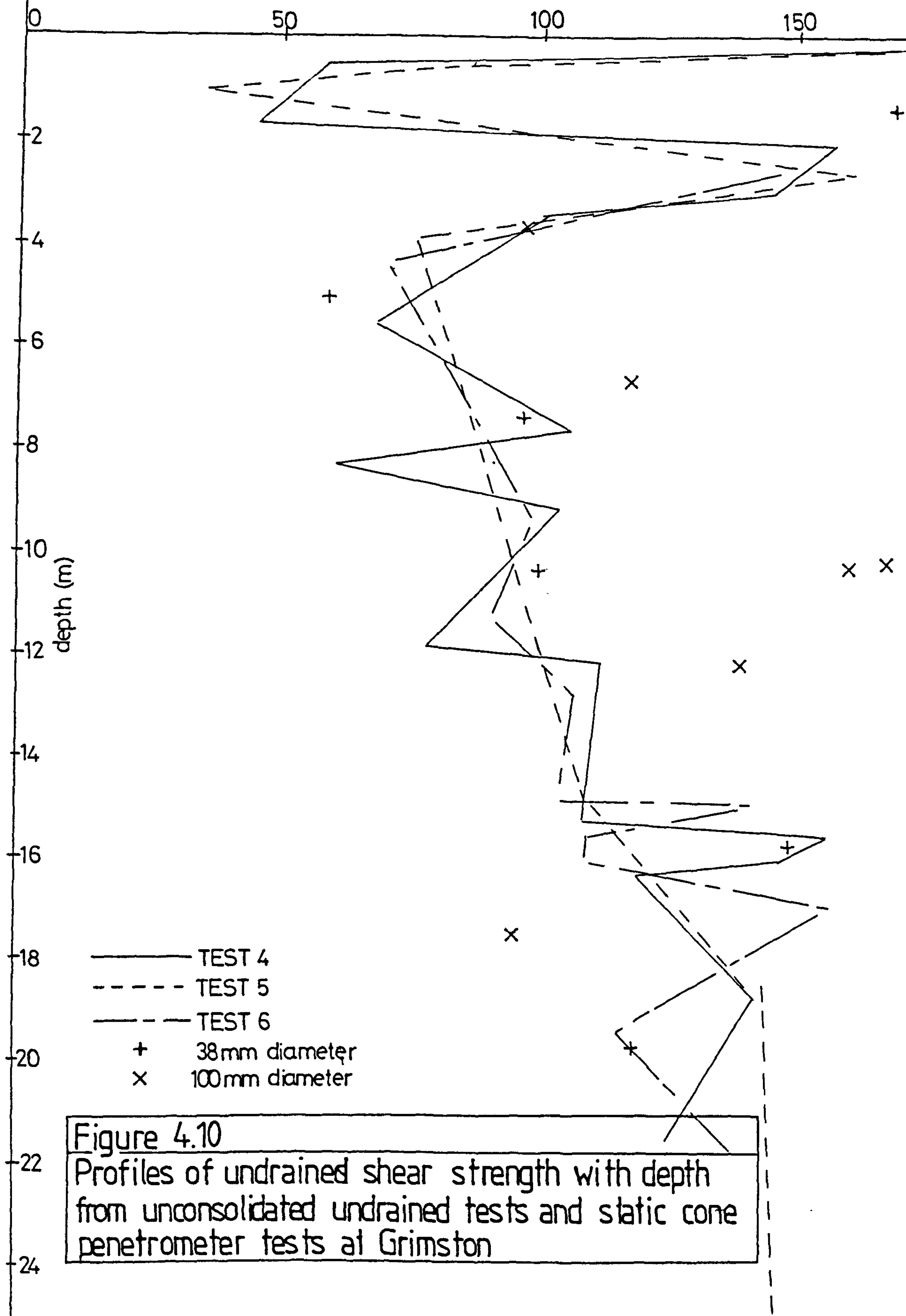
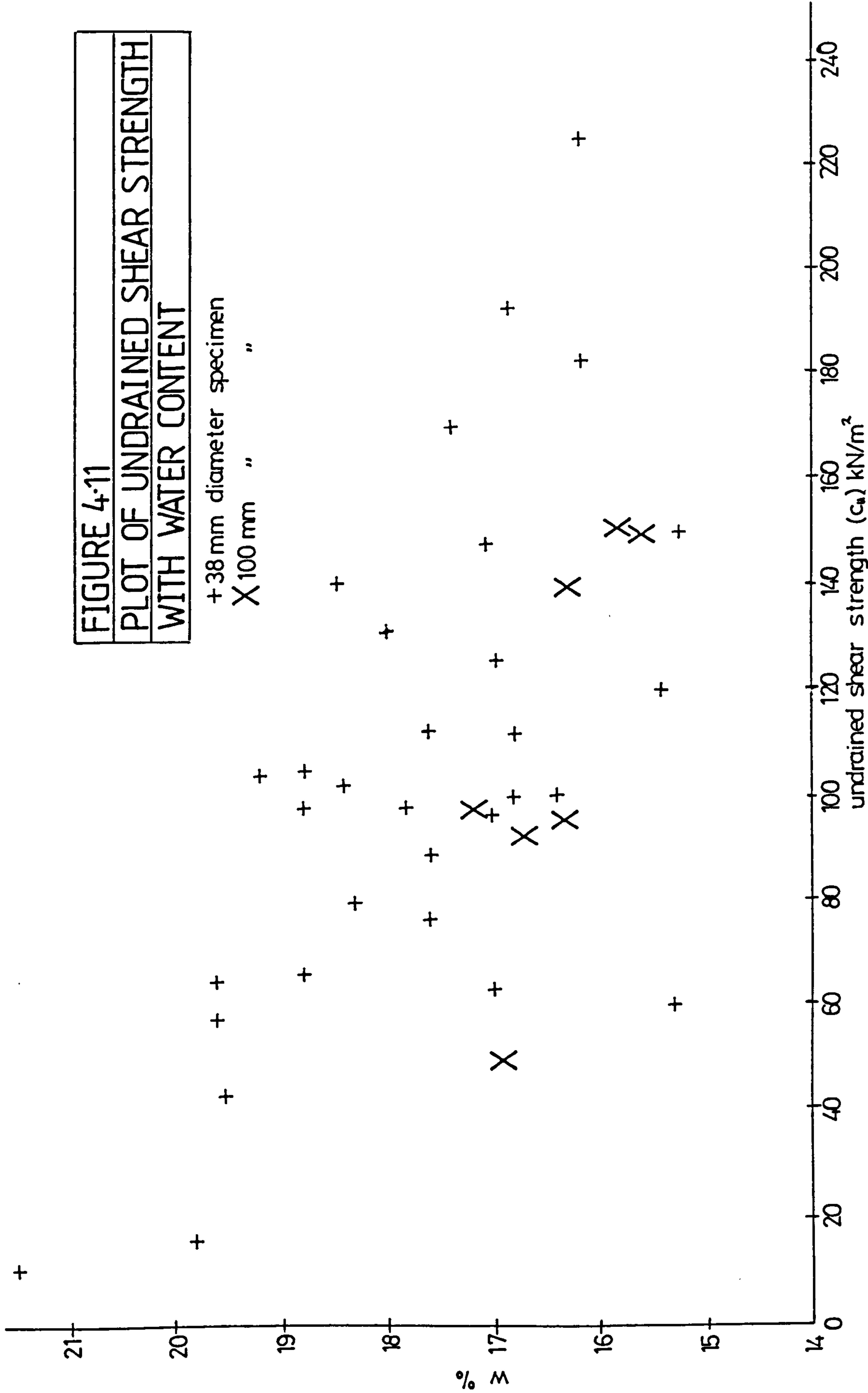
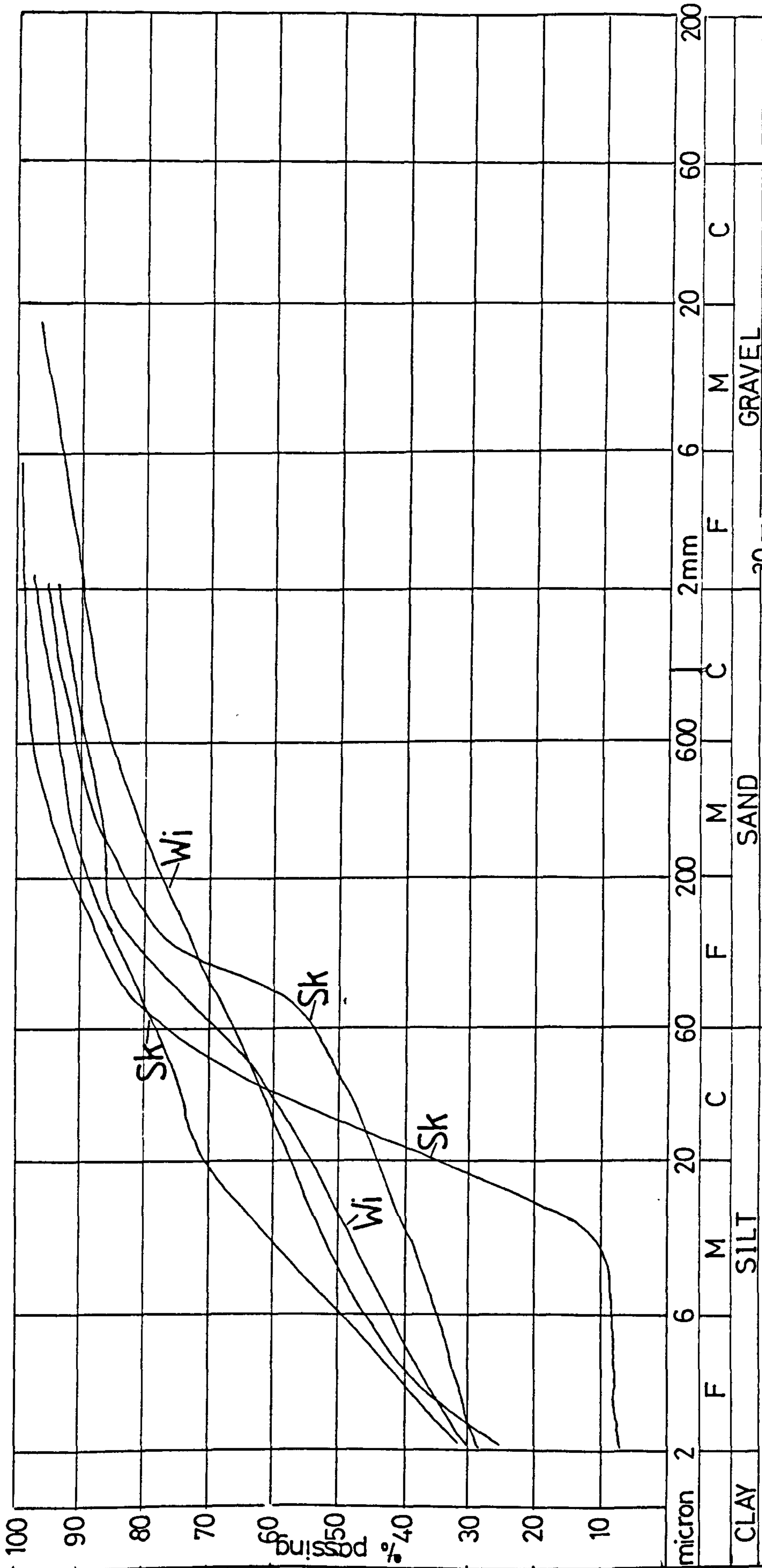


Figure 4.10
Profiles of undrained shear strength with depth
from unconsolidated undrained tests and static cone
penetrometer tests at Grimston

FIGURE 4.11
PLOT OF UNDRAINED SHEAR STRENGTH
WITH WATER CONTENT

+ 38 mm diameter specimen
X 100 mm "





Wi Withernsea till
Sk Skipsea till

FIGURE 4.12
PARTICLE SIZE DISTRIBUTION CURVES
AND PLASTICITY CHART

Bulk densities found from 38mm and 100mm triaxial test specimens are plotted with depth on the soil profiles. They show a slight variation, with a mean value of 2170kg/m^3 , giving a unit weight of 21.3kN/m^2 .

The degree of saturation of the tills was calculated from measurements of bulk density and water content and an assumed value of specific gravity of 2.7 (measured on 1 sample of Withernsea till). It is plotted with depth on the soil profiles and as a histogram in Figure 4.12A. This shows a distribution centred about 100% saturation, and with only 3 out of 19 values less than 95%. Some of the samples have a calculated degree of saturation greater than 100%, which indicates there is an error of up to $\pm 5\%$ arising from errors in measurement. Thus the majority of samples tested have a degree of saturation of at least 90%.

The expression for effective stress in partially saturated soils is given by:

$$\sigma' = \sigma - [u + (1-\chi) (u_d - u)]$$

where χ is a coefficient to be determined experimentally. However in soils with a degree of saturation greater than 90% the error in neglecting the product $(1-\chi) (u_d - u)$ is small (Skempton and Hutchinson, 1969). Therefore in this study of the Holderness cliffs effective stress is calculated by the expression:

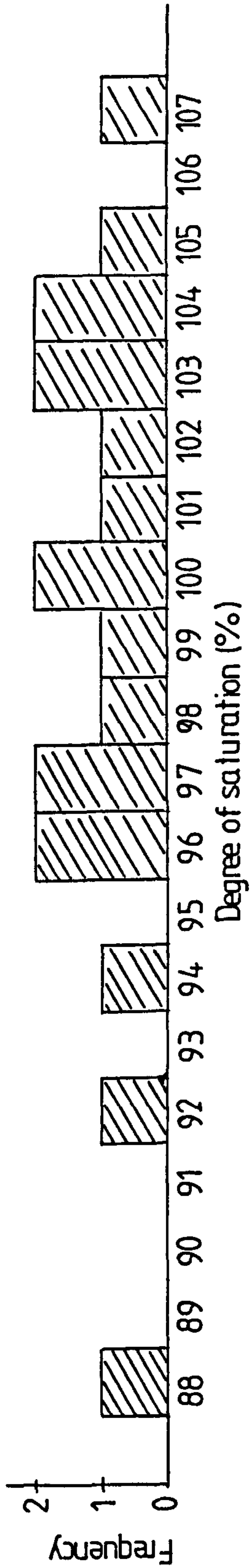


Figure 4.12A

Histogram of degree of saturation

$$\sigma' = \sigma - u$$

and it is assumed that suctions arising from partial saturation do not contribute to the stability of the slips.

4.2.4 Soil Fabric

The soil micro fabric was studied in cut and torn U100 samples and in cliff face exposures. Discontinuities observed were thought to be of three origins: weathering, stress relief and depositional. The fissures observed in the weathered zone were moderately to closely spaced to a depth of 3m, and more widely spaced below this depth. As at Cowden (Marsland and Powell, 1985) they were not seen below 5m depth. They were open up to 1mm, occasionally coated with sand and were 'gleyed' on exposure at the cliff face. The discontinuities of other origins were not frequent enough to influence failure, except for occasional falls from the cliff face.

A fine horizontal layered fabric was observed in some of the U100 samples, and at the same level on the cliff face sub-horizontal ridges of clay were often seen. These features were most common near the boundaries between till divisions. Sand layers and partings frequently occur within the clay tills (described in Section 4.4.1).

4.3 Field Instrumentation

4.3.1 Pore Water Pressure Measurement

The pore water pressure distribution in the predominantly clay soils was investigated using Casagrande type standpipe piezometers of 19mm internal diameter. A typical installation is shown in Figure 4.13A. A seal was formed using bentonite pellets and the drillhole was backfilled with bentonite/cement grout. They were chosen for their low cost, durability and ease of installation and reading.

A disadvantage of this type of piezometer is the high system flexibility, which can lead to a long time lag before equalization of the pore water pressure in the soil and in the piezometer (Gibson, 1963). The time lag in till is likely to be of the order of several weeks. However the purpose of the instrumentation was to investigate long-term changes in pore water pressure as a result of stress relief. These were expected to persist for a period long enough to be measured by the Casagrande piezometers.

The Casagrande piezometers are not capable of measuring negative pore water pressure (ie. below atmospheric pressure). However this was acceptable because pore water pressures were expected to be positive at the piezometer tip positions, even if they had been depressed by stress relief.

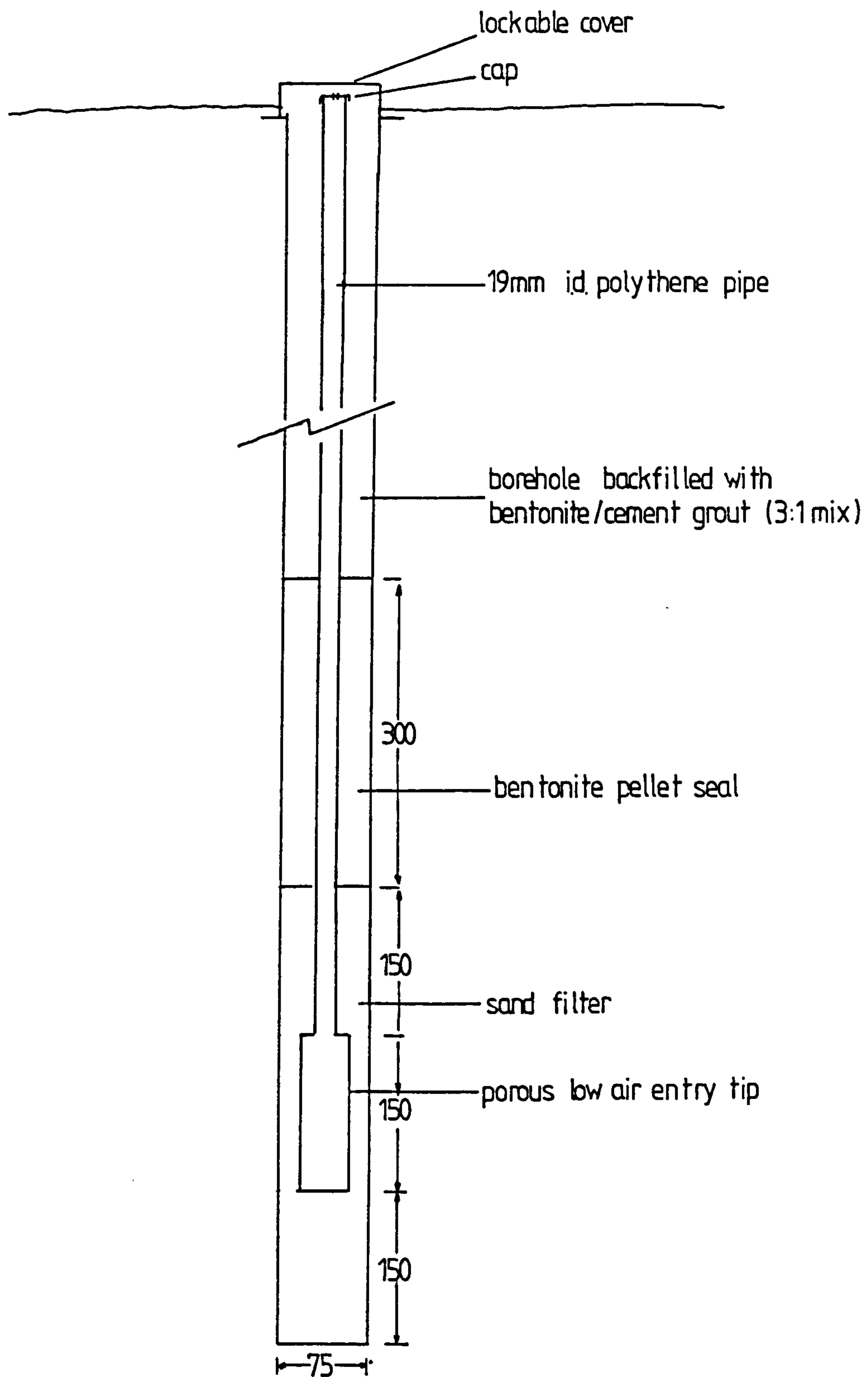


Figure 4-13A

Typical Casagrande standpipe piezometer installation

If negative pore water pressures had been encountered (indicated by dry standpipes), the use of hydraulic piezometers could have been considered.

The layout of the piezometers was designed to measure both horizontal and vertical variations in pore water pressure, adjacent to and remote from the cliff (see Figures 4.1 and 4.3).

Fourteen piezometers were installed at each site, mostly in 75mm diameter boreholes drilled using a "Minuteman" rig to a maximum depth of 10m. Three of the piezometers at each site were installed below this depth in the 150mm diameter boreholes drilled for soil sampling. Six metal push-in standpipe piezometers were installed to 3m depth, two of them at the foot of the cliff and one in the till shore platform to 3m depth. It was not attempted to measure pore water pressure on the slip surface, because access to the cliff surface was difficult, and installations would have been lost under debris, or by slip movements (see suggestions for further research, Section 6.3.1).

The pressure head of water in each of the piezometers is plotted with time in Figure A4.13, Appendix D. Typical plots for piezometers at various depths are shown on Figures 4.14 and 4.15. They show a seasonal change in piezometers at all depths. The change is smooth at shallow depths, but more erratic below 10m. The maximum pressure head occurs between

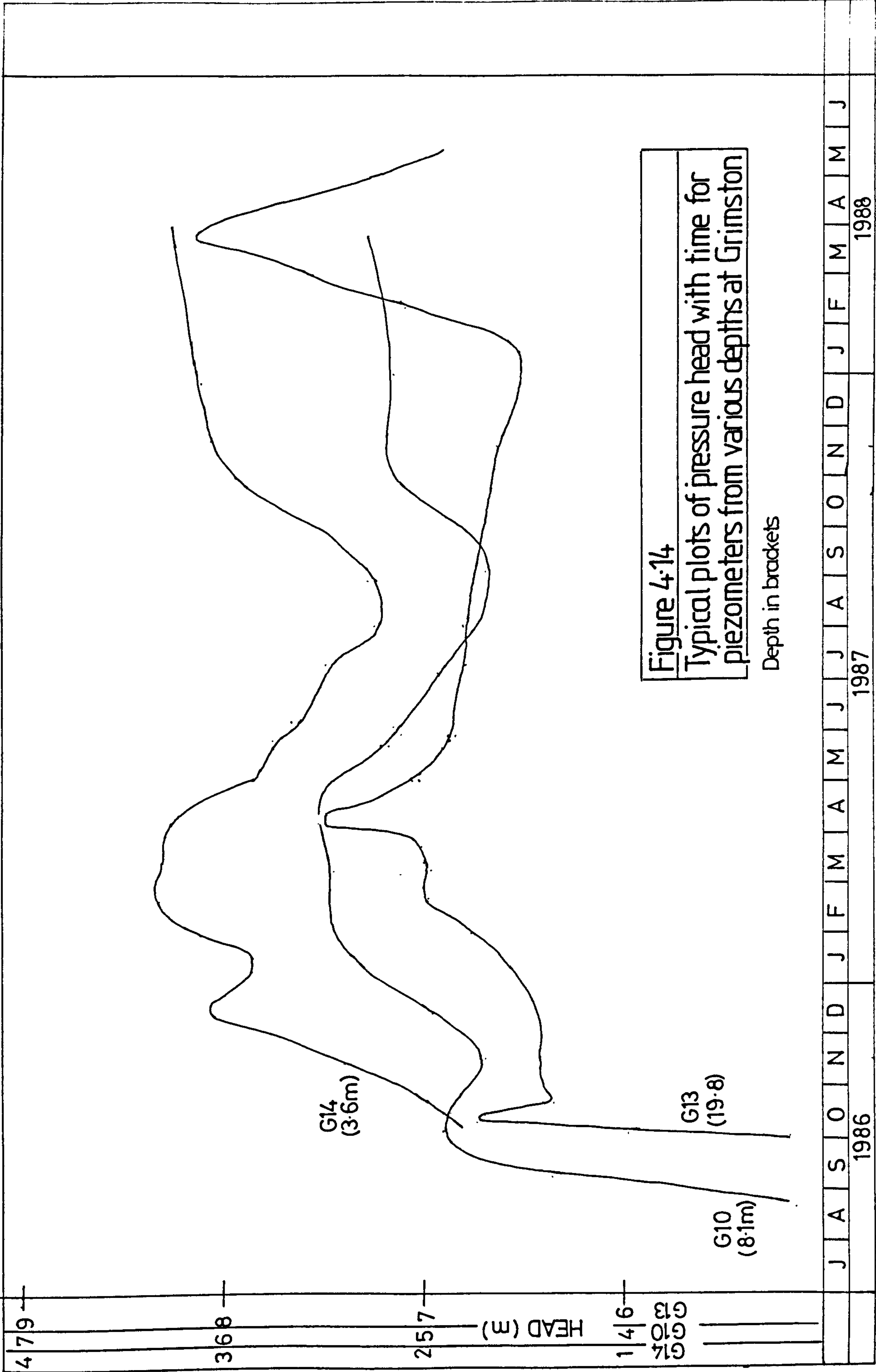
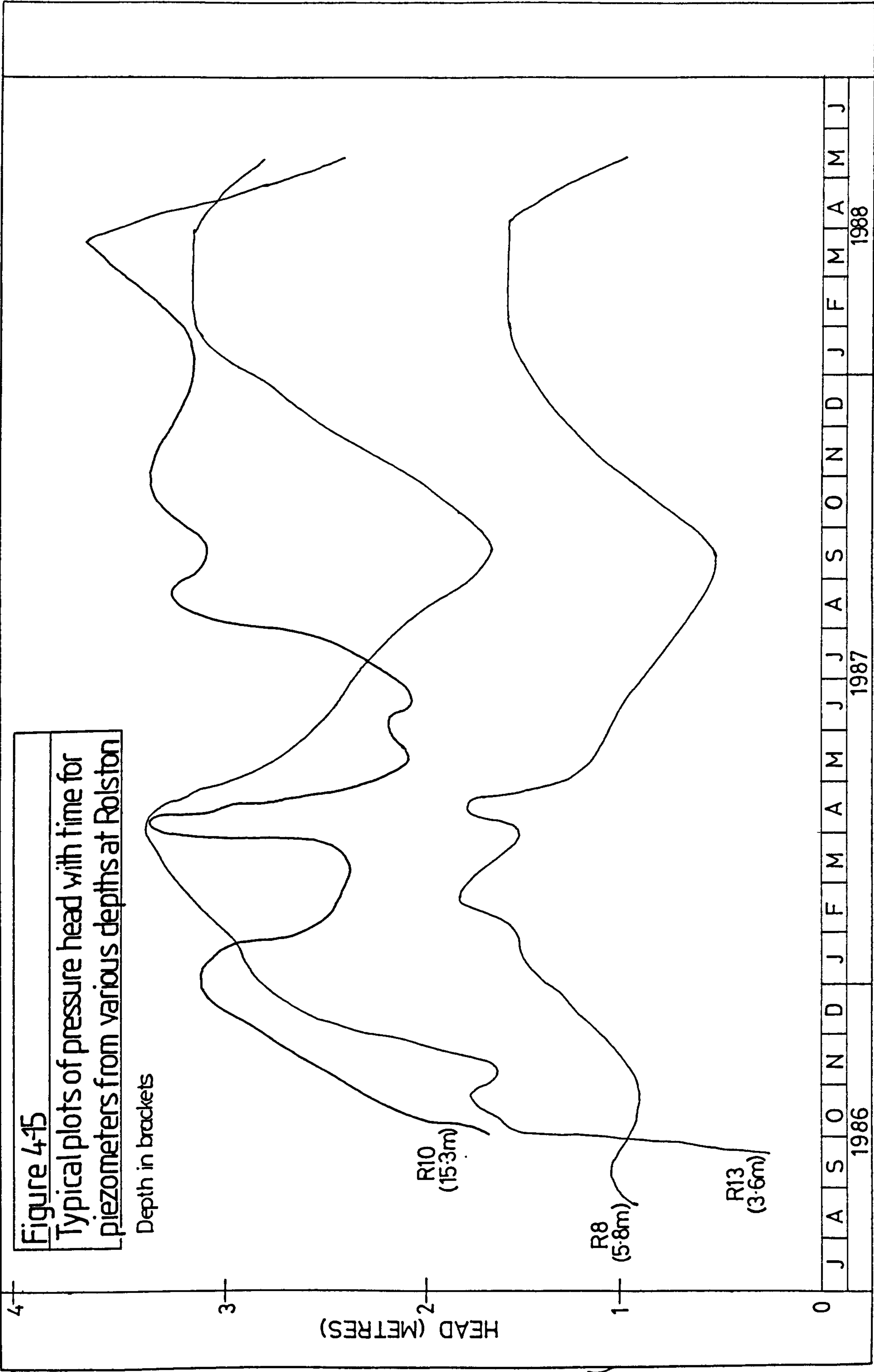


Figure 4.14
 Typical plots of pressure head with time for
 piezometers from various depths at Grimston
 Depth in brackets

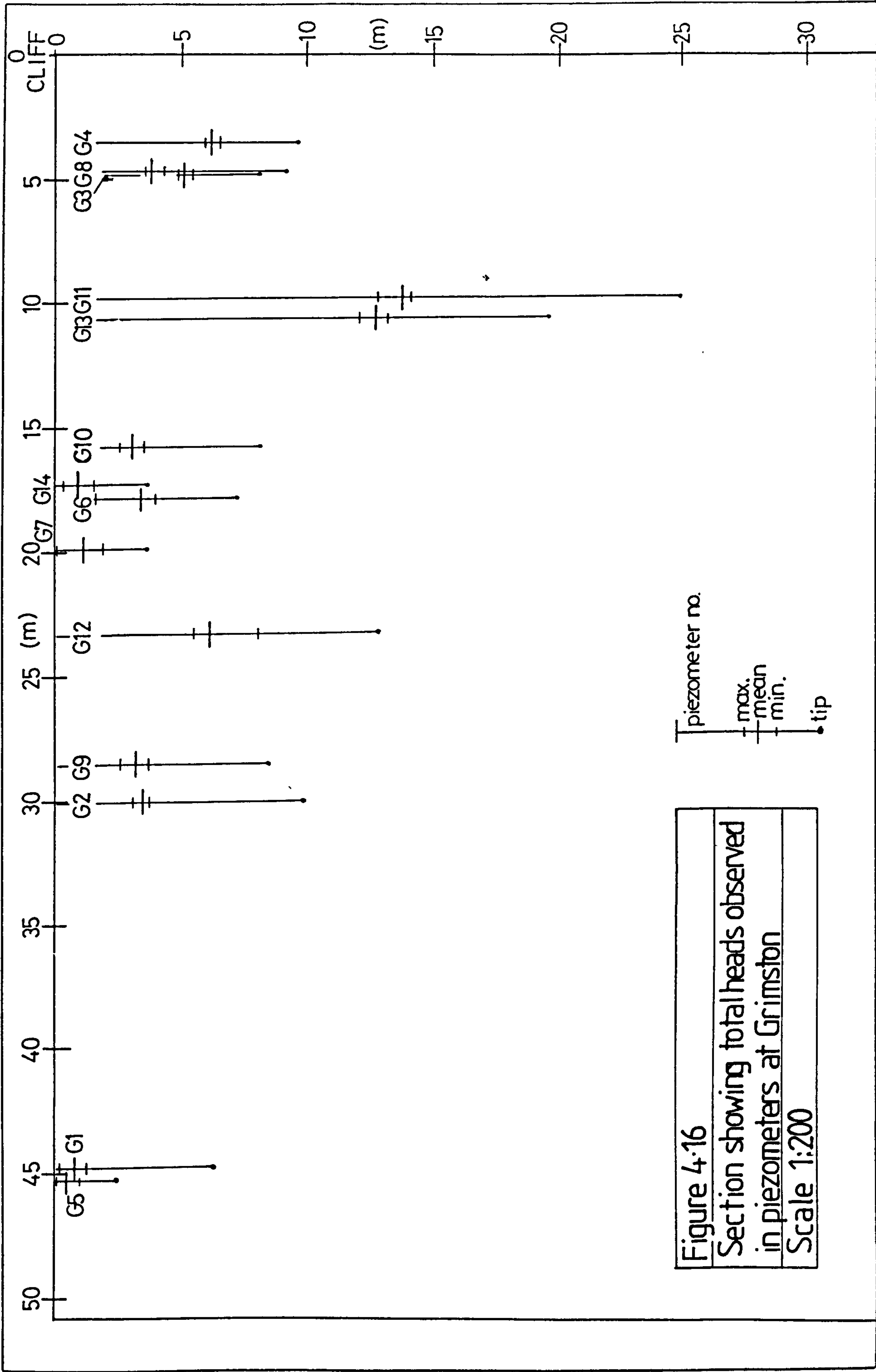


February and April and minimum between August and September. The difference is between 1 and 2m, and is greatest at shallow depths or in sandier layers.

The mean, maximum and minimum total heads observed in each piezometer are plotted on site cross-sections in Figures 4.16 and 4.17. All the piezometers (numbered R1-13 and G1-14) have been projected onto a vertical plane perpendicular to the cliff. The sections show a decrease in total piezometric head with depth and towards the cliff. At Rolston the decrease is greater than at Grimston, and is appreciable far back from the cliff. No changes associated with unloading of the cliff were observed in the piezometers. An explanation of the measured pore water pressure distribution is attempted in Section 5.1 with the use of steady seepage and consolidation analysis.

4.3.2 Hydraulic Conditions in the Coastal Slopes

The total monthly precipitation at Westlands Farm, Holderness (TA 181 409) is plotted for the period of study in Figure 4.18 (Zaidky R., personal communication). There are two peaks, in late Autumn and in late Spring. These can be correlated with the steady rise of pore water pressure from September to December, and the sharp peak in April 1987 (see Figures 4.14 and 4.15). The total annual precipitation for 1987 was 650mm. Increased evaporation between May and September would be expected to reduce the amount of infiltration (Foster et al.,



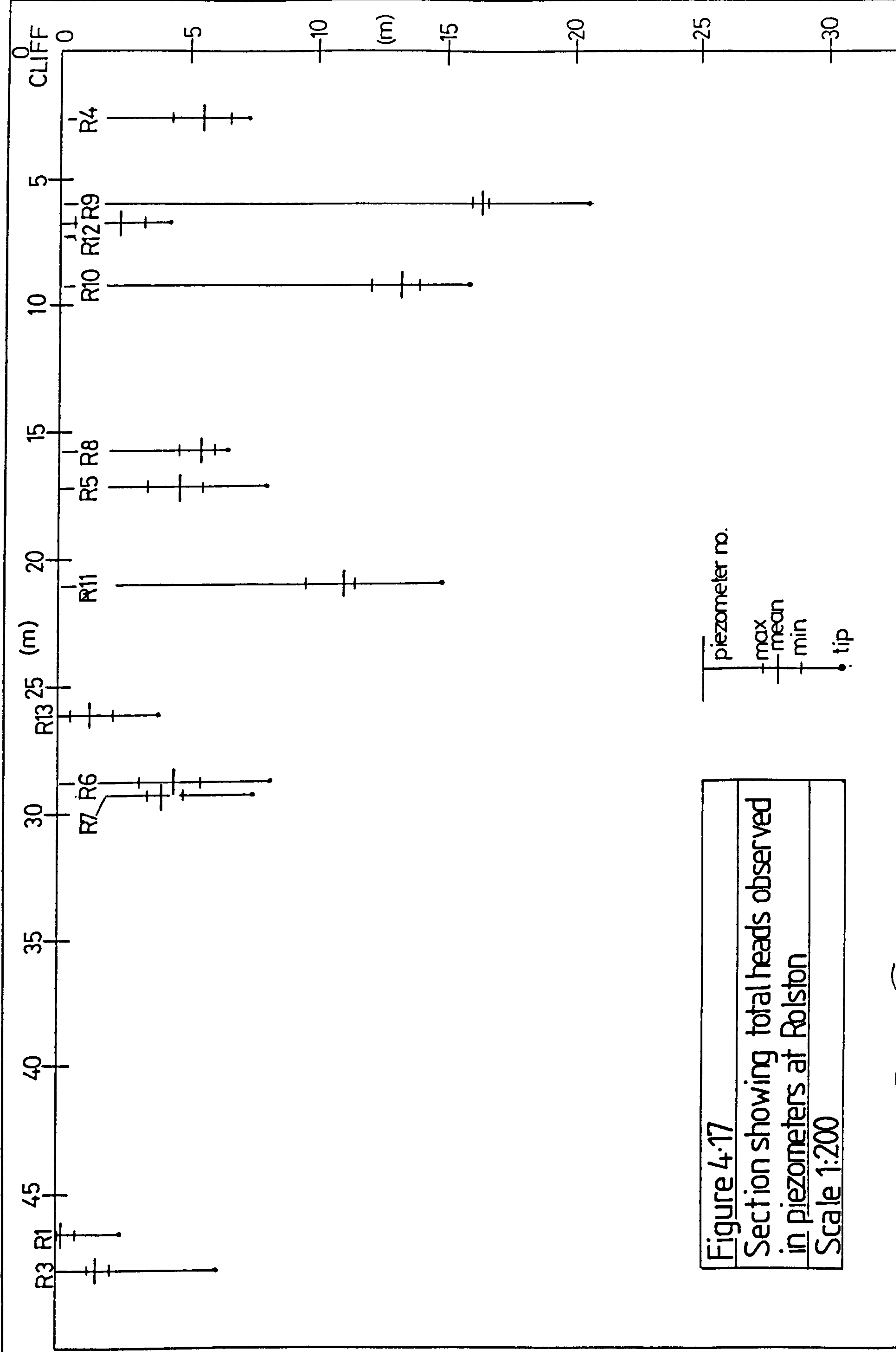


Figure 4.17

Section showing total heads observed in piezometers at Rolston

Scale 1:200

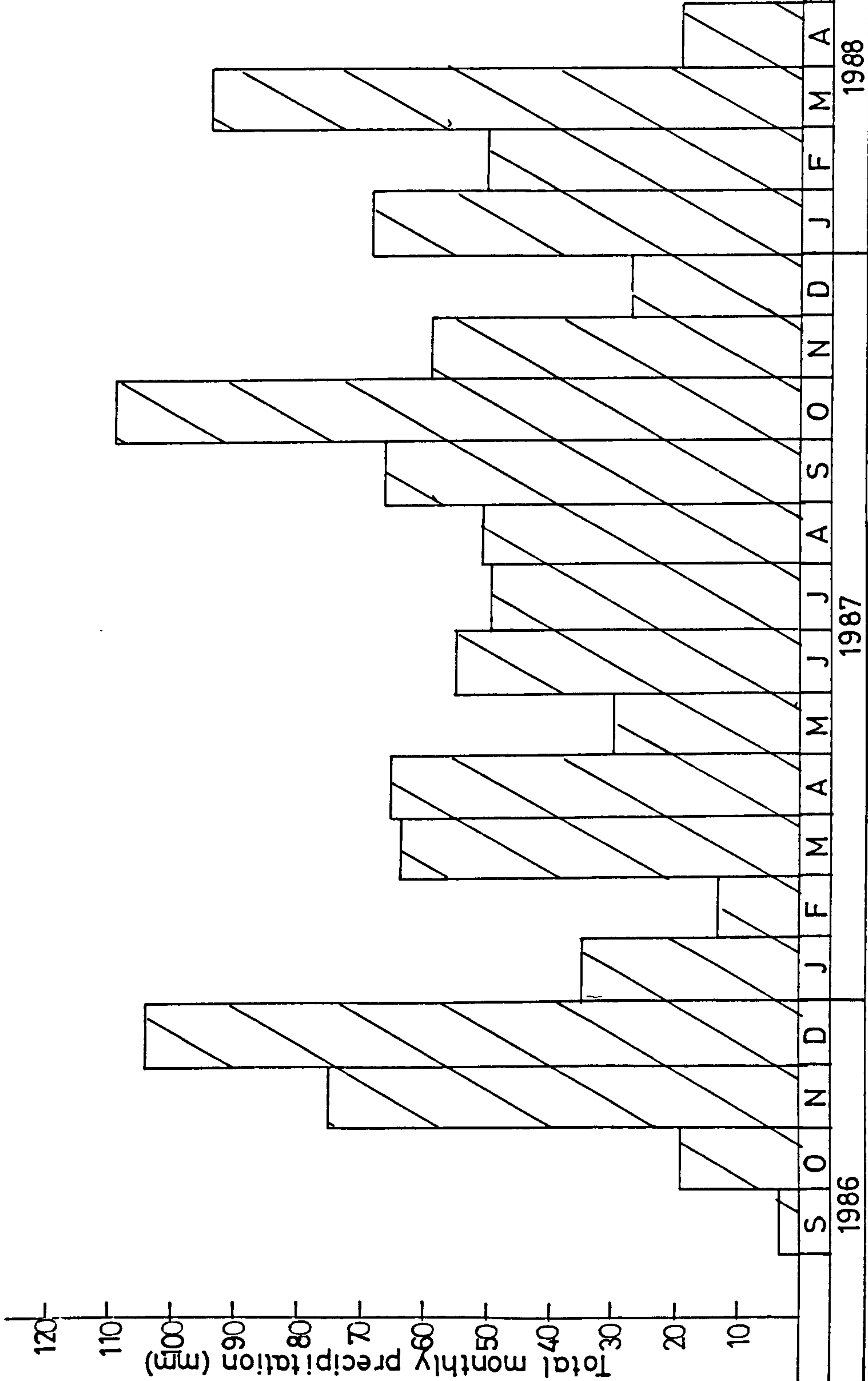


FIGURE 4-18

Histogram to show total monthly precipitation at Westlands Farm, Holderness (TA 181 409)

1976), causing the fall in pore water pressure observed between May and August 1987.

The lodgement tills are underlain by Chalk (see Section 2.2.1), which dips gently east beneath Holderness from outcrop on the Yorkshire Wolds. In the Chalk aquifer flow and storage of groundwater is confined to joints and fissures, enlarged locally by solution (Foster et al., 1976). The surface of the chalk has been affected by Pleistocene cryoturbation processes, forming a high permeability layer of broken-up chalk up to 10m thick. Most groundwater flow is confined to that layer (Chadha D., personal communication).

The groundwater levels in the chalk beneath Holderness are not known with certainty. The Yorkshire Water Authority regularly measure water levels in seven observation wells in the Holderness area. Groundwater levels at October 1987 from the Y.W.A. observations are plotted in Figure 4.19. At Hornsea near artesian conditions occur. However these conditions appear to be localized, as at the nearby Atwick and Skirlaugh wells, groundwater levels are near Ordnance Datum. Seasonal fluctuations in levels are of 1-2m, with a minimum in October and a maximum in April

There are no observation wells near the instrumented sites. At Grimston, taking the mean of the Hollym, Skirlaugh and Saltend data, a groundwater level of 1m O.D. is estimated. At Rolston the mean of the Ulrome, Skirlaugh and Atwick data gives a level

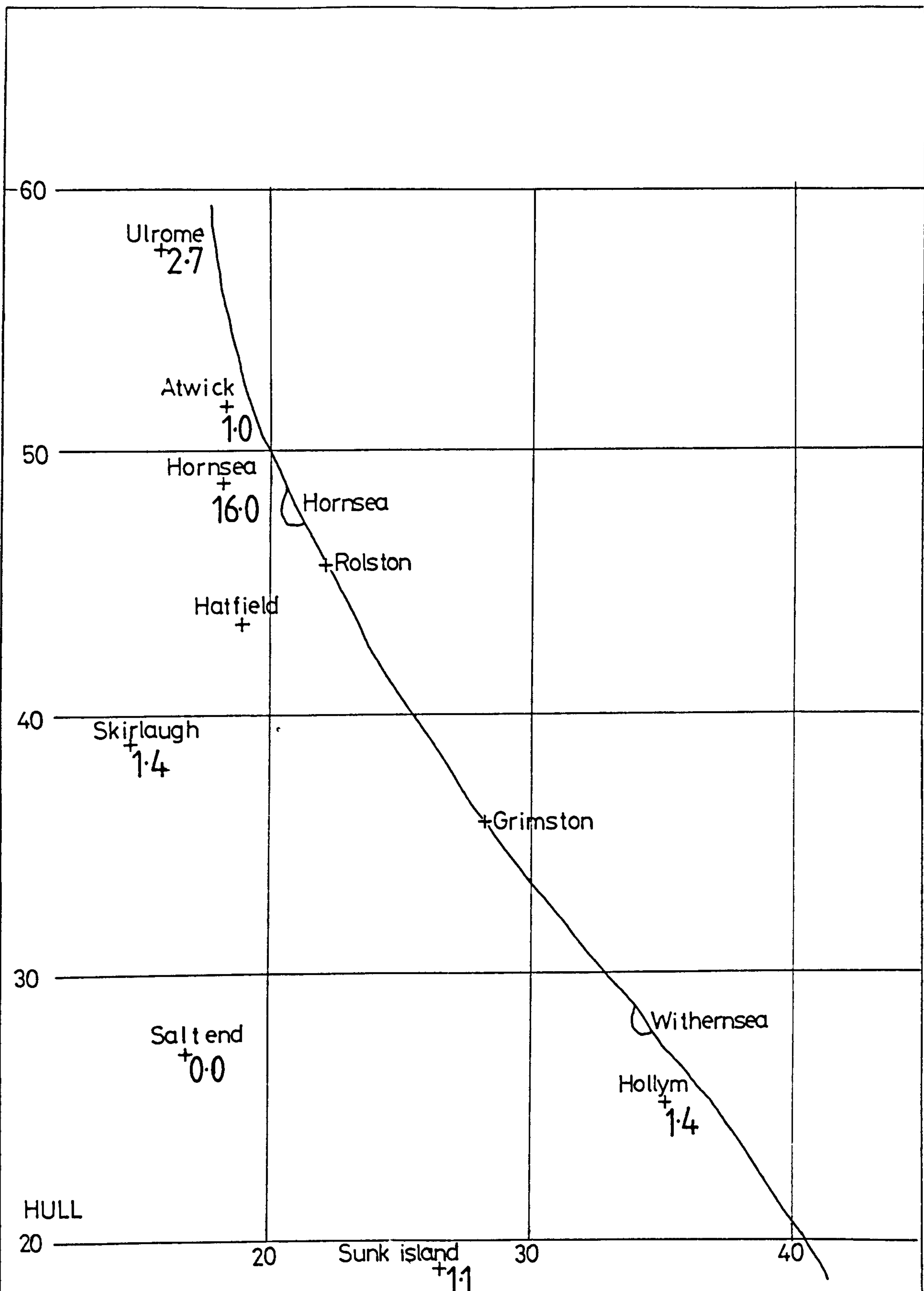


FIGURE 4-19
Y.W.A. GROUNDWATER LEVELS OCTOBER 1987 (mQD.)
SCALE 1:200 000

of 2m O.D. The low groundwater levels expected in the Chalk at both sites would result in downward hydraulic gradients in the till slopes, as the foot of the cliff is at about 2m O.D. However upward gradients could occur at Rolston, if the levels observed at Hornsea are representative of a large area of artesian conditions.

4.3.3 Surface Ground Movement Measurement

Ground movements were expected to occur adjacent to the cliff prior to and after failure from two causes:

- volume changes in the clay caused by changes in effective stress;
- deformation caused by stress relief and the opening of tension cracks.

Only surface measurements were taken. Steel pegs of 0.5m length were installed in three lines at each site (lines "A", "B" and "C" at Rolston, and lines "D", "E" and "F" at Grimston, see Figures 4.1 and 4.3). These ran perpendicular to the cliff at an average spacing of 3m, farther apart away from the cliff. A reference peg of 0.9m length was placed 40m to 60m from the cliff, where movements were expected to be negligible. The distance to each peg was measured with an MA100 Tellurometer centred on a tripod over the reference peg. An adaptor allowed

a target prism to fit over each peg. The pegs were levelled using another adaptor to detect vertical movement.

Surveys were done at monthly or bi-monthly intervals from September 1986 to March 1988. The main sources of error were in setting up the instrument level and centred. An error of $\pm 1\text{mm}$ was found in both horizontal and vertical measurements. A table of the cumulative ground movement indicated by each peg is given in Figure A4.20, Appendix E. Large stress relief movement occurred at two of the six lines of pegs, which were both at the Rolston site (lines "A" and "B"). The total displacement of these pegs during the study is shown in Figures 4.21 and 4.22, and their horizontal and vertical movements with time in Figures 4.23 and 4.24.

4.3.4 Line "A" Movements

The largest movements occurred in pegs "A10" and "A8" located 3m and 6m from the cliff edge respectively. Movements began in January 1987 when both pegs settled about 25mm (see Figure 4.23). Horizontal measurements were unfortunately not taken on that site visit. A slip occurred between line "A" and the roadend in early February 1987 (see Appendix F). About 3m width of clay was removed from in front of the line, leaving peg "A10" 20cm from the new cliff edge. The surveys of February and March showed continuing horizontal and vertical movement in pegs "A10" and "A8". Peg "A10" was lost in a small

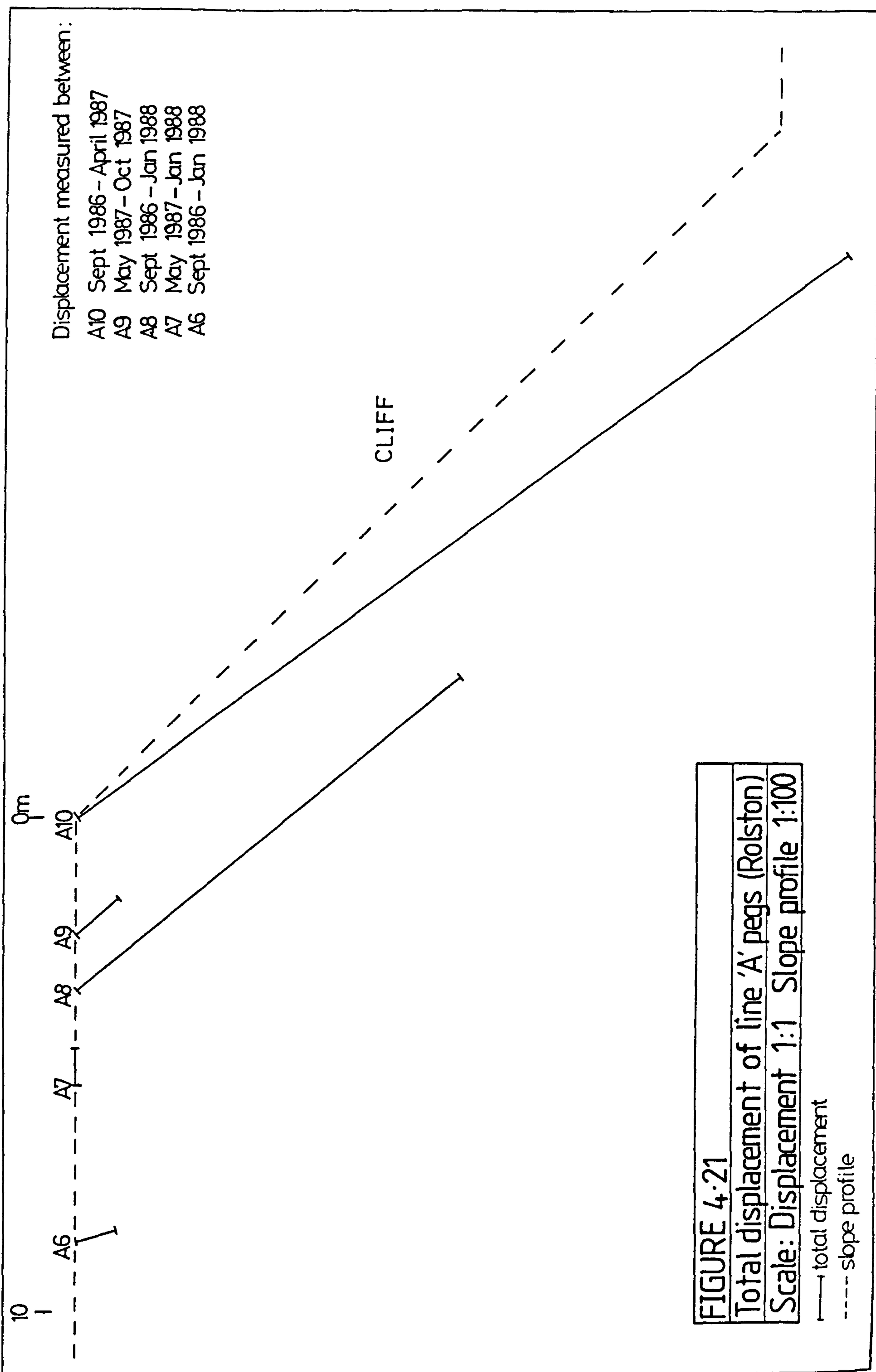
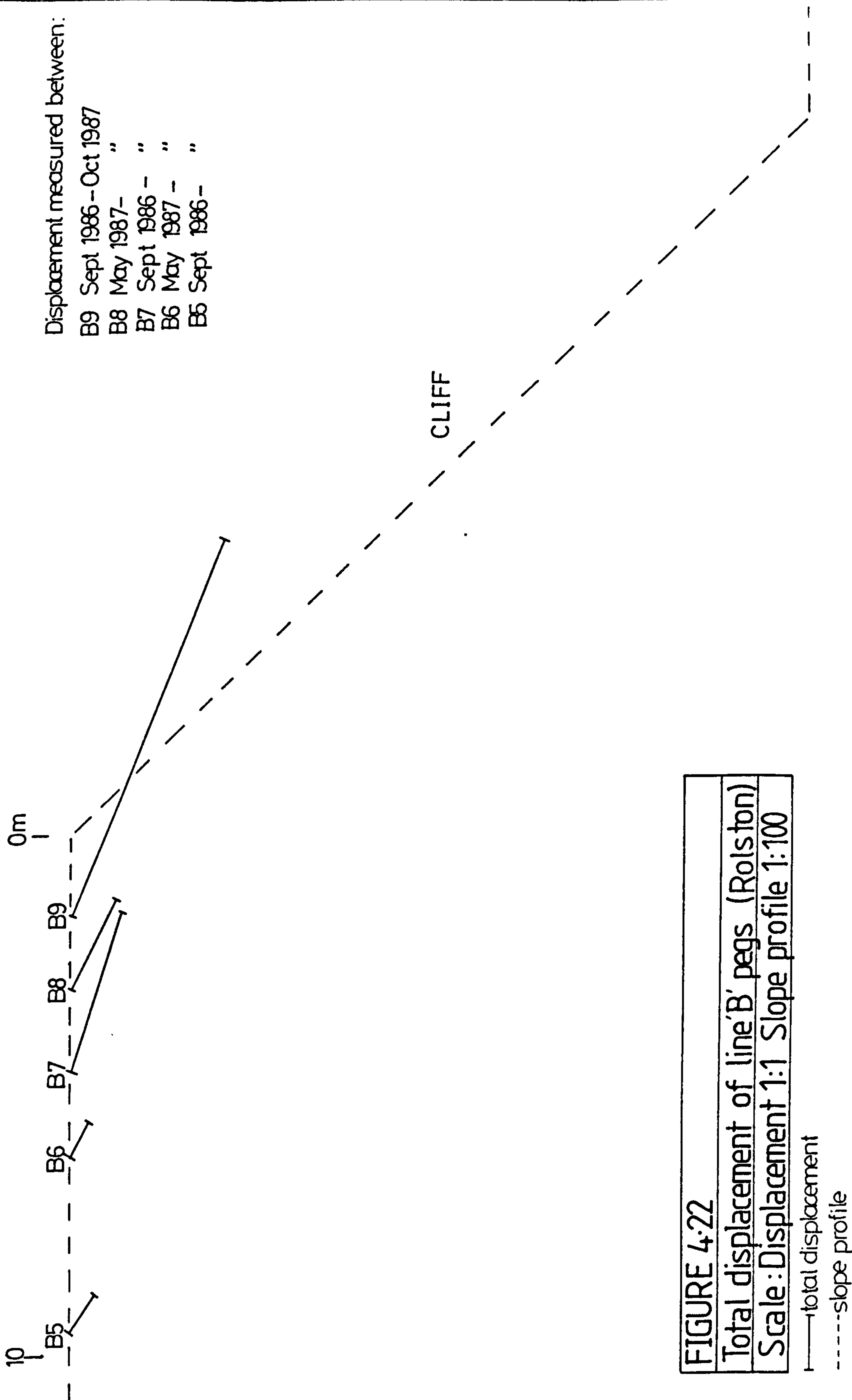


FIGURE 4.21
 Total displacement of line 'A' pegs (Rolston)
 Scale: Displacement 1:1 Slope profile 1:100



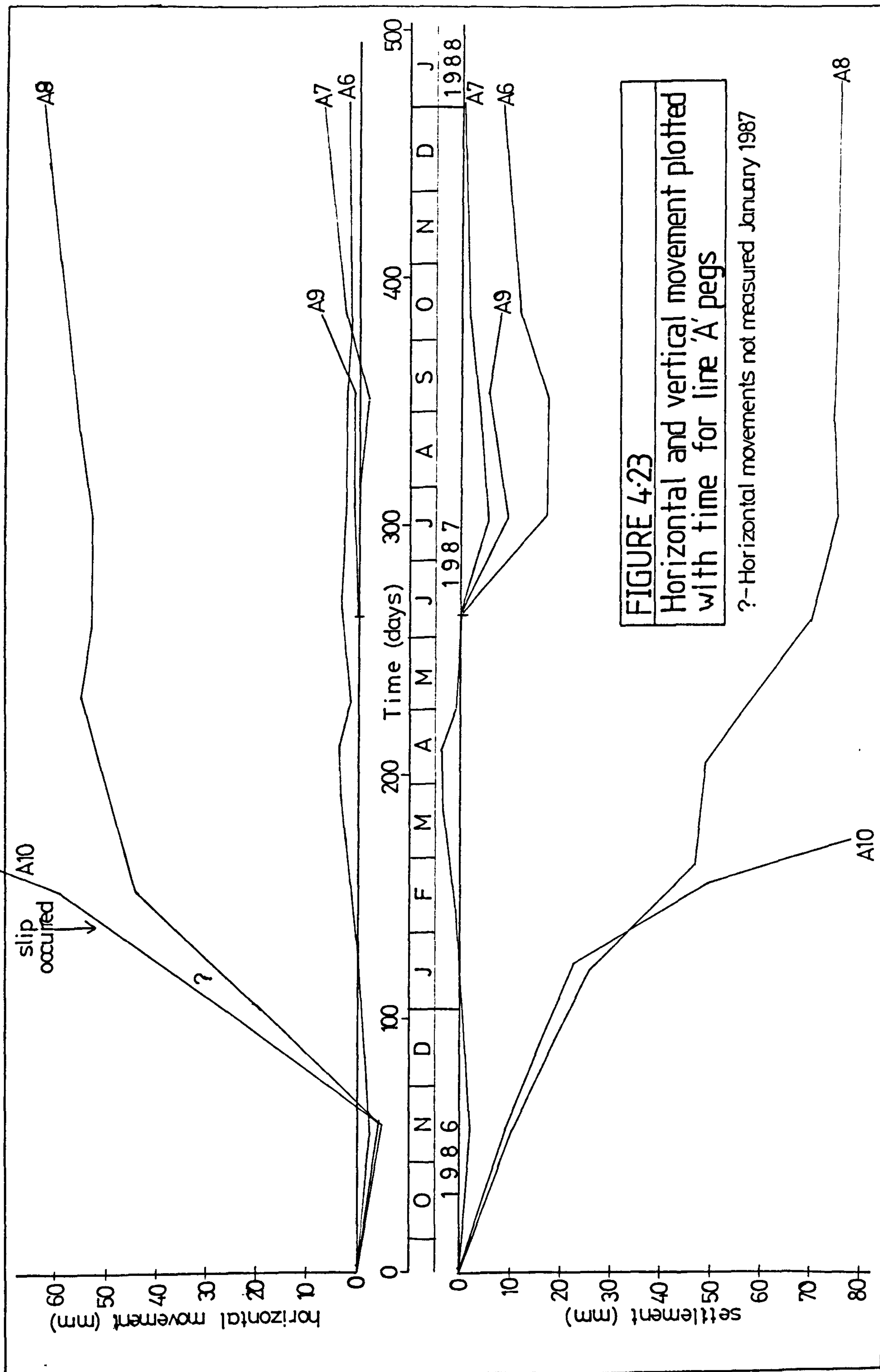


FIGURE 4:23
 Horizontal and vertical movement plotted
 with time for line 'A' pegs

?-Horizontal movements not measured January 1987

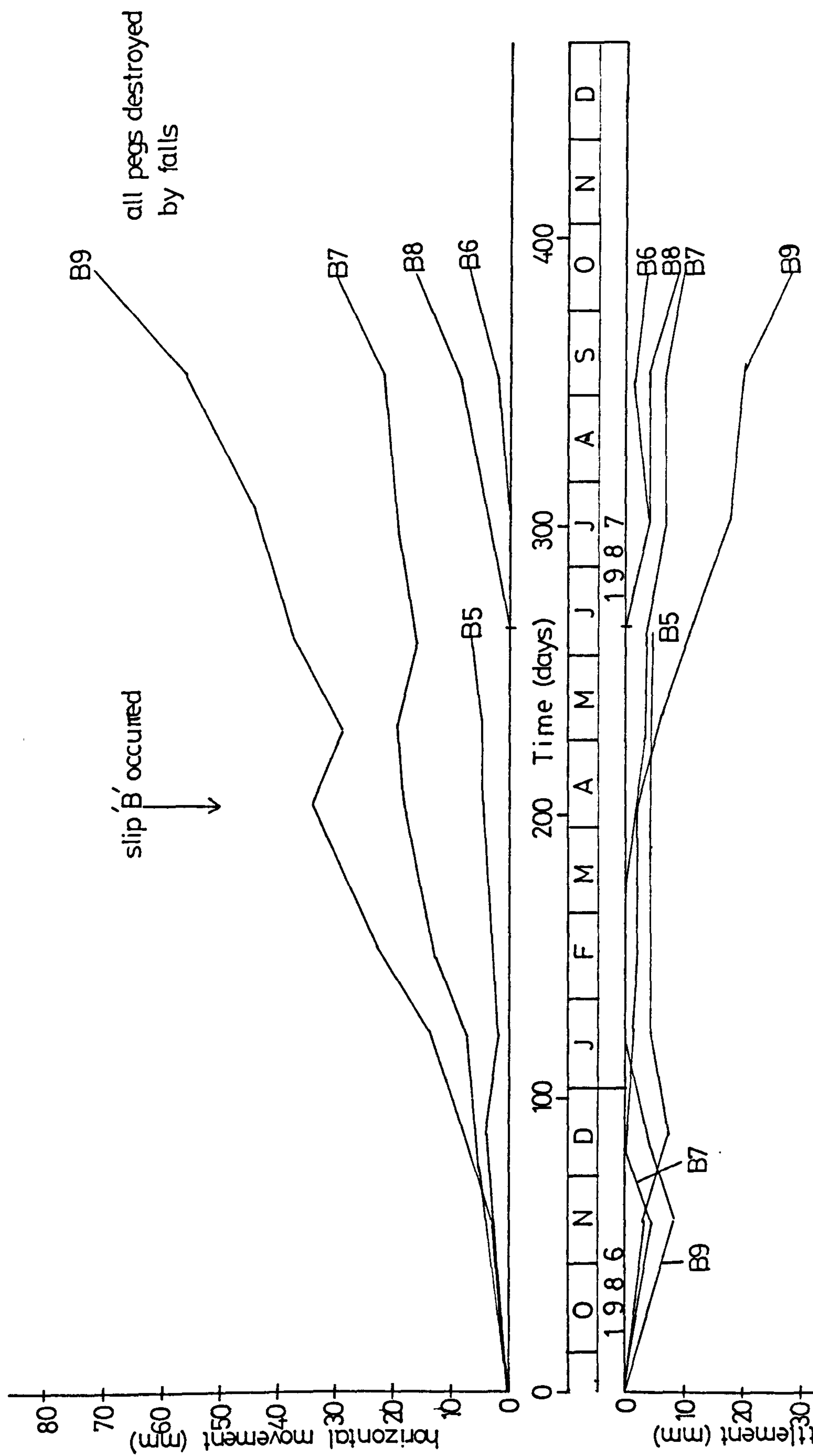


FIGURE 4-24
Horizontal and vertical movement plotted
with time for line 'B' pegs

cliff fall in April 1987. Peg "A8" continued to settle at a constant rate until August 1987, although horizontal movement was small during this period. Pegs further than 10m from the cliff edge showed no stress relief movement.

4.3.5 Line "B" Movement

The largest movements occurred in pegs "B9" and "B7", 3m and 6m from the cliff respectively. Horizontal movement of pegs "B9" and "B7" occurred at a constant rate between September 1986 and October 1987 (see Figure 4.24). A slip occurred in March 1987 centred 15m to the south of line "B" (see Appendix F). A 3m width of clay was removed from in front of the line leaving peg "B9" 30cm from the new cliff edge. This did not affect the rate of horizontal movement, but settlement of pegs "B9" and "B7" began after the slip and continued until October 1987. Two additional pegs were installed in May 1987 and showed movements consistent with the original pegs. In November 1987 large falls of weathered clay caused 5m recession of the clifftop at line "B", destroying the pegs.

4.3.6 Lines "C", "D", "E" and "F" Movements

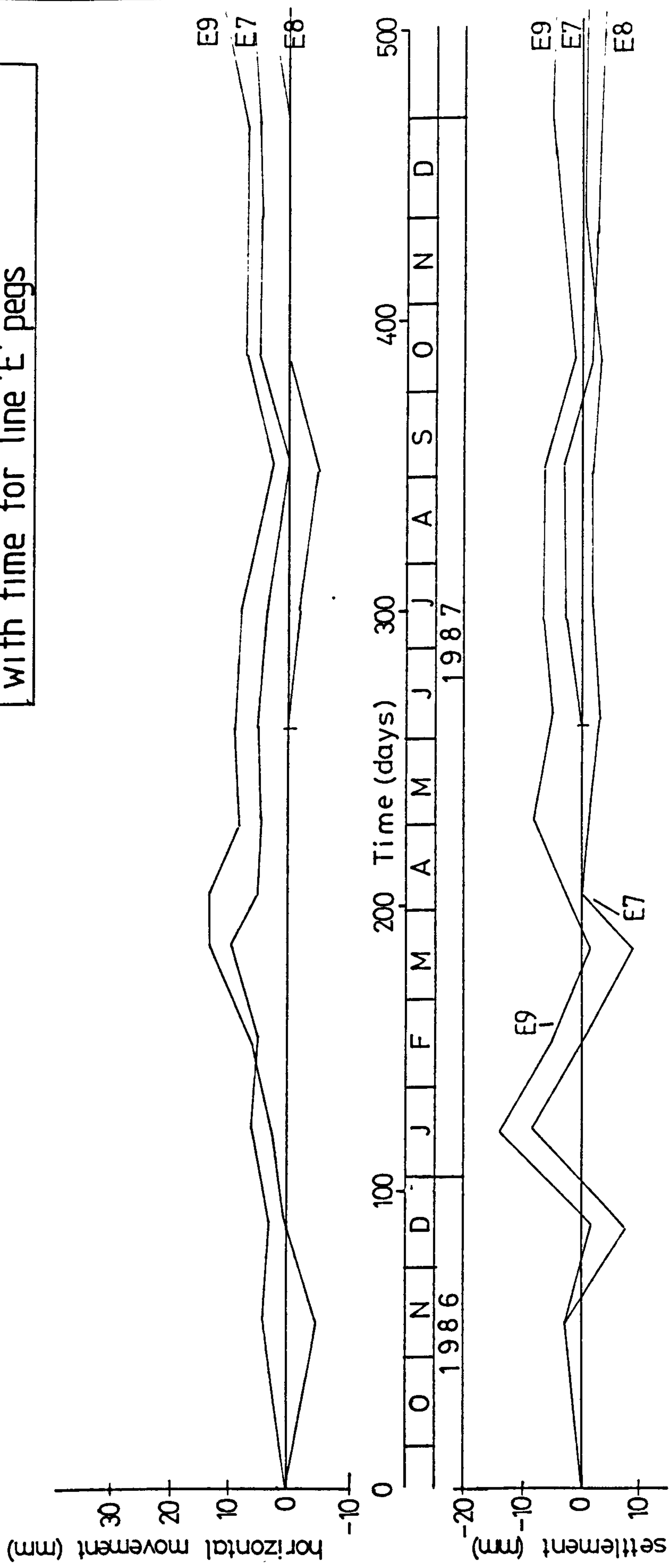
The pegs of all four lines up to 6m away from the cliff showed a gradual outward (toward the cliff) movement between September 1986 and March 1987, after which they began to move inward

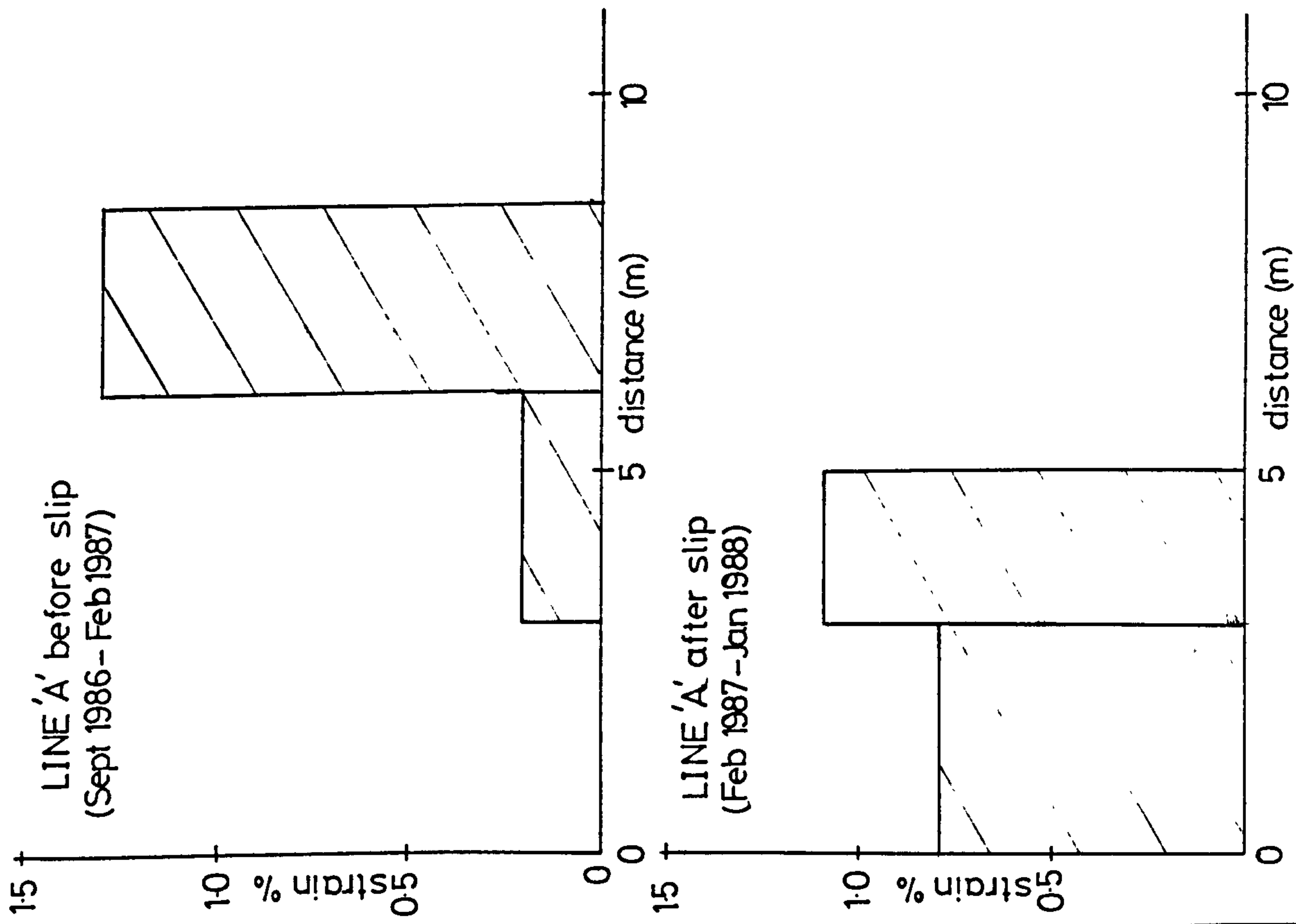
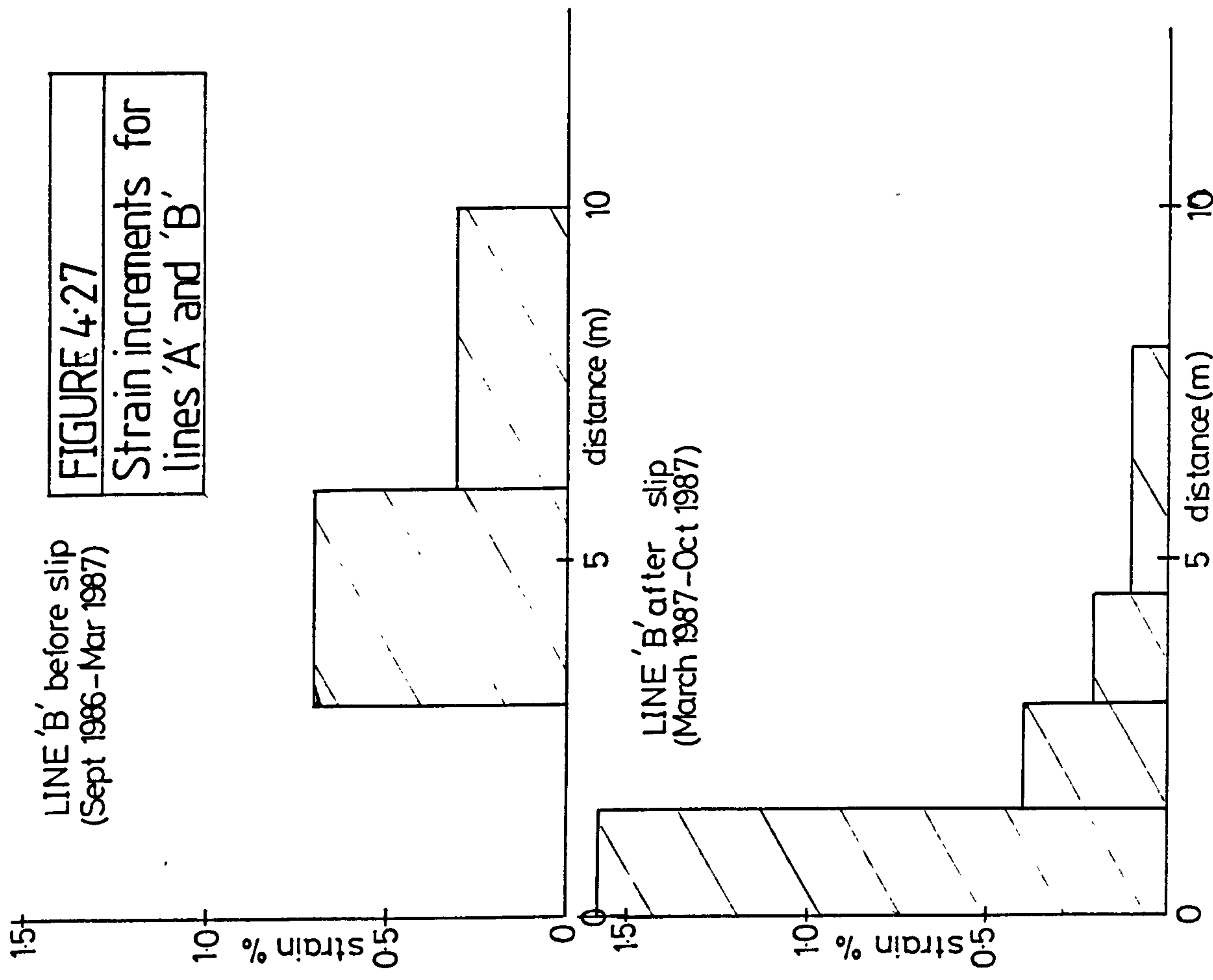
again. By September 1987 they were close to their original positions of September 1986. This is shown in Figure 4.25 by a plot of horizontal and vertical movements with time for line "E" pegs, which were typical of all four lines. This behaviour showed that most of the movement was caused by volume changes of the clay. Maximum and minimum outward movements corresponded to highest and lowest pore water pressures respectively. The pegs nearest the cliff (3m back) showed a mean outward movement of 12mm in September 1987, compared with September 1986, which is attributed to stress relief. There was little recession of the cliff by falls or slips in front of the lines "C", "D", "E" and "F" during the study.

4.3.7 Ground Strains

The movements caused by stress relief alone, corrected by subtracting the estimated seasonal movement, for lines "A" and "B", and "C", "D", "E" and "F" are given in Figure A4.26, Appendix E. The movements of lines "A" and "B" were separated into those which occurred before and after failure of the cliff in front of the lines. Horizontal ground strains between pegs were calculated from the movements. These are strain increments as considerable movement would have occurred before the measurements began. No large tension cracks were observed behind the cliff although narrow ones could have been hidden by grass. The strain increments for the lines "A" and "B", before and after failure, are plotted in Figure 4.27. The magnitude

FIGURE 4-25
Horizontal and vertical movement plotted
with time for line 'E' pgs





of strain increments observed was from 0.1 to 1.6%. At line "A" before failure, strains are largest between 6m and 9m back from the cliff. This could be caused by the occurrence of a tension crack, with the clay in front moving forward as a block. After failure the strain increments are more evenly distributed. Line "B" shows progressively larger strain increments towards the cliff. The strain increments occurring before failure in line "B" were larger than those after, and may have occurred in response to unloading by a previous failure before measurements began. Strain increments in lines "C", "D", "E" and "F" were small and only observed close to the cliff. The last major unloading of these lines took place at least one year before measurements began. Most of the settlement of lines "A" and "B" occurred after failures, and overall horizontal movement exceeded settlement. The strain increments measured in line "B" after unloading are thought to be most representative of the general changes as the cliff recedes.

4.3.8 In Situ Stresses

Knowledge of the in situ stresses was required to plan and interpret field and laboratory tests; also the changes in horizontal stress as the cliff recedes are relevant to the studies of ground movement and cliff stability. In situ stress measurements were not made at Rolston and Grimston during the study, because funds were not available. It was assumed that

the profile of K and stress changes occurring near the cliff at the sites would be similar to those measured at Cowden by the Building Research Establishment. Measurements of horizontal stress at the Cowden inland test-bed site using spade cells and pressuremeters indicate that K_0 is near unity, except in the surface weathered zone (Marsland and Powell, 1985). There it increases to a value of about two. At the Cowden cliff site measurements using spade cells indicate a slight reduction in horizontal stress parallel to the cliff edge, and a larger reduction at right angles to it. The differences are most marked in the weathered zone to 4m depth, but are noticeable to at least 8m, where the deepest measurements were made. The reductions occur up to at least 15m back from the cliff edge (Butcher, 1986).

It was attempted to estimate the in situ stresses at the Rolston and Grimston sites by laboratory tests on U100 samples to measure the initial capillary suction (Burland and Maswoswe, 1982). The test results indicated unrealistically high values of K_0 , compared with the measurements at Cowden, which may have been due to sample disturbance (eg. desiccation during storage). However the results did show K_0 decreasing with depth and near to the cliff, as was found at Cowden.

4.3.9 Grimston Site Changes and Slip Geometry

The two slipped soil masses at Grimston were degraded and moved downslope during the study, although no new slips occurred. A geomorphological map of the area made during 1981 (Williams, 1982) shows the presence of the two slipped soil masses at the same location. These are probably from the previous cycle of slipping, indicating a cycle time of 4-5 years. Falls of weathered till from the backscarp resulted in a recession of the clifftop by 1-2m, and loaded the slipped soil masses. Erosion of the cliff foot steadily removed till by large falls and corrasion. The rate of erosion was deduced to be equal to the rate at which the slips moved downslope, because the position of the cliff foot did not change during the study. Mudslides activated by seepage from sand layers eroded the till cliff between the slips during the winter months. The beach was lowered during the Summer and Autumn of 1987 by the passing of an "ord". The changes in cliff profile with time are shown in Figures 4.28 and 4.29.

The positions of wooden pegs in the slipped soil masses were surveyed bi-monthly using an AGA Geodimeter 12 and theodolite set up on the beach. The steel reference pegs for surface ground movement measurement were designated as data. Cross-sections showing displacement vectors of the wooden pegs in a vertical plane perpendicular to the cliff are shown in Figures 4.30 and 4.31. The positions of the slip surfaces were inferred from the surface movement measurements. The technique

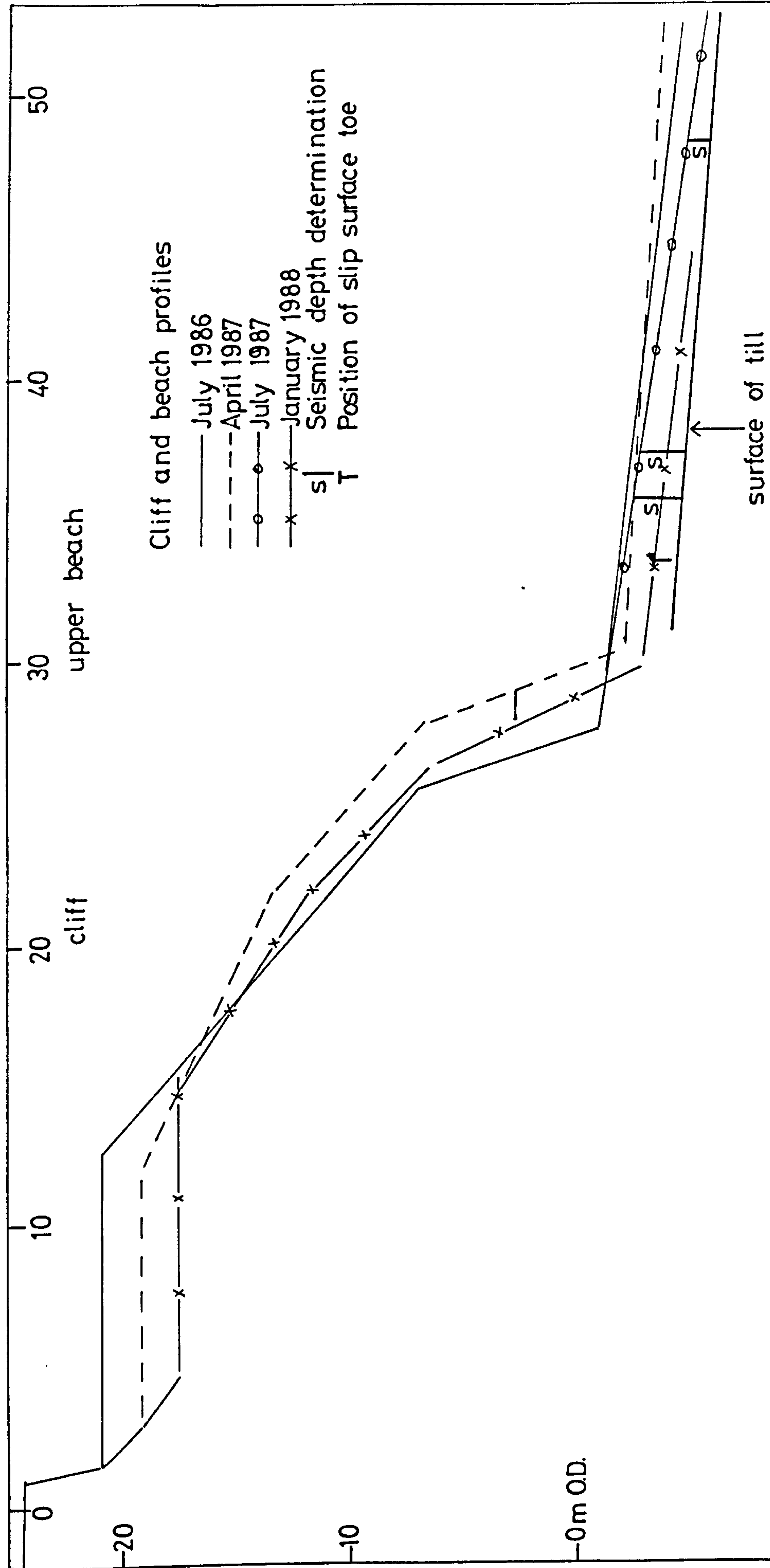


Figure 4:28
Changes in cliff profile for
slip'D' at Grimston
Scale 1:200

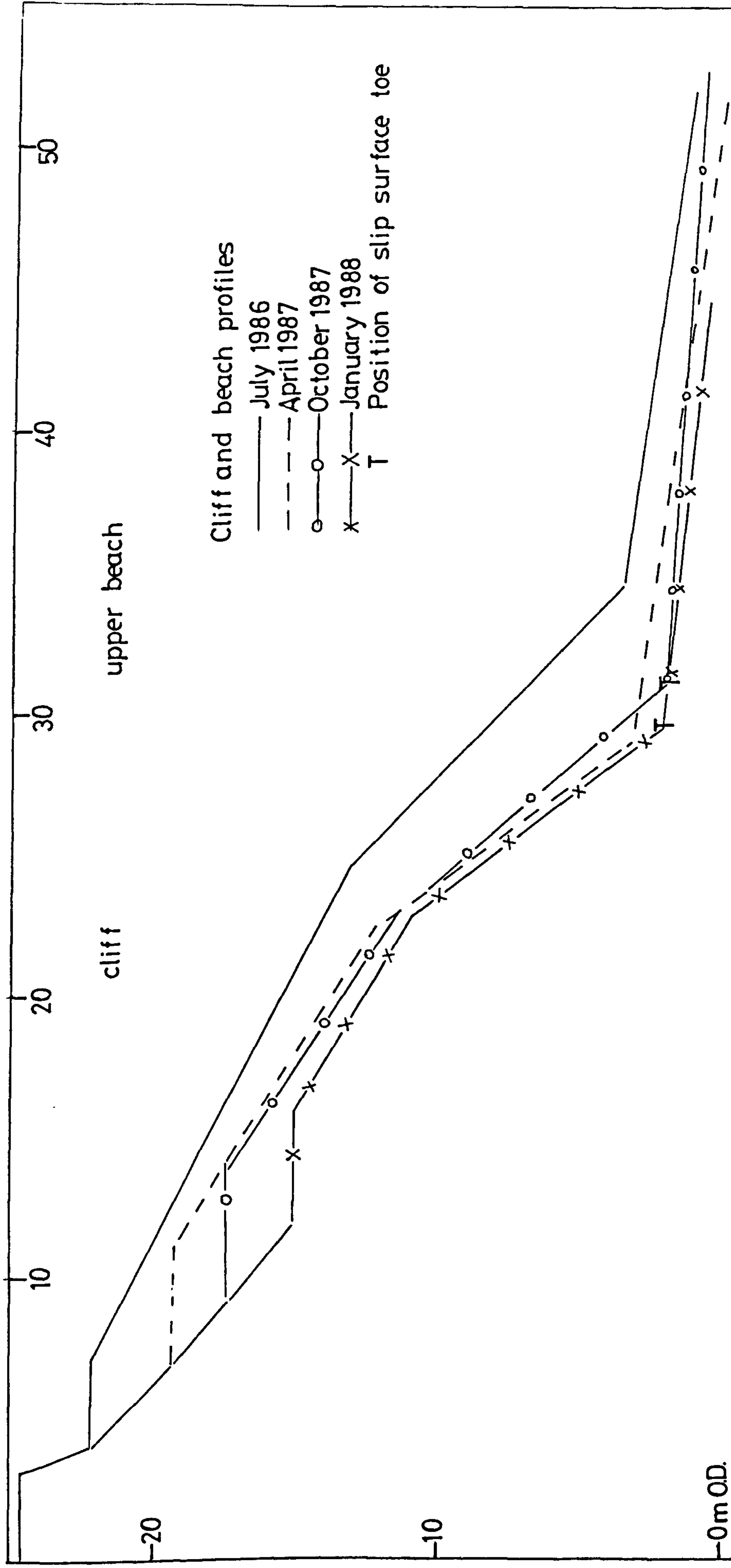


Figure 4:29
 Changes in cliff profile for
 slip 'F' at Grimston
 Scale 1:200

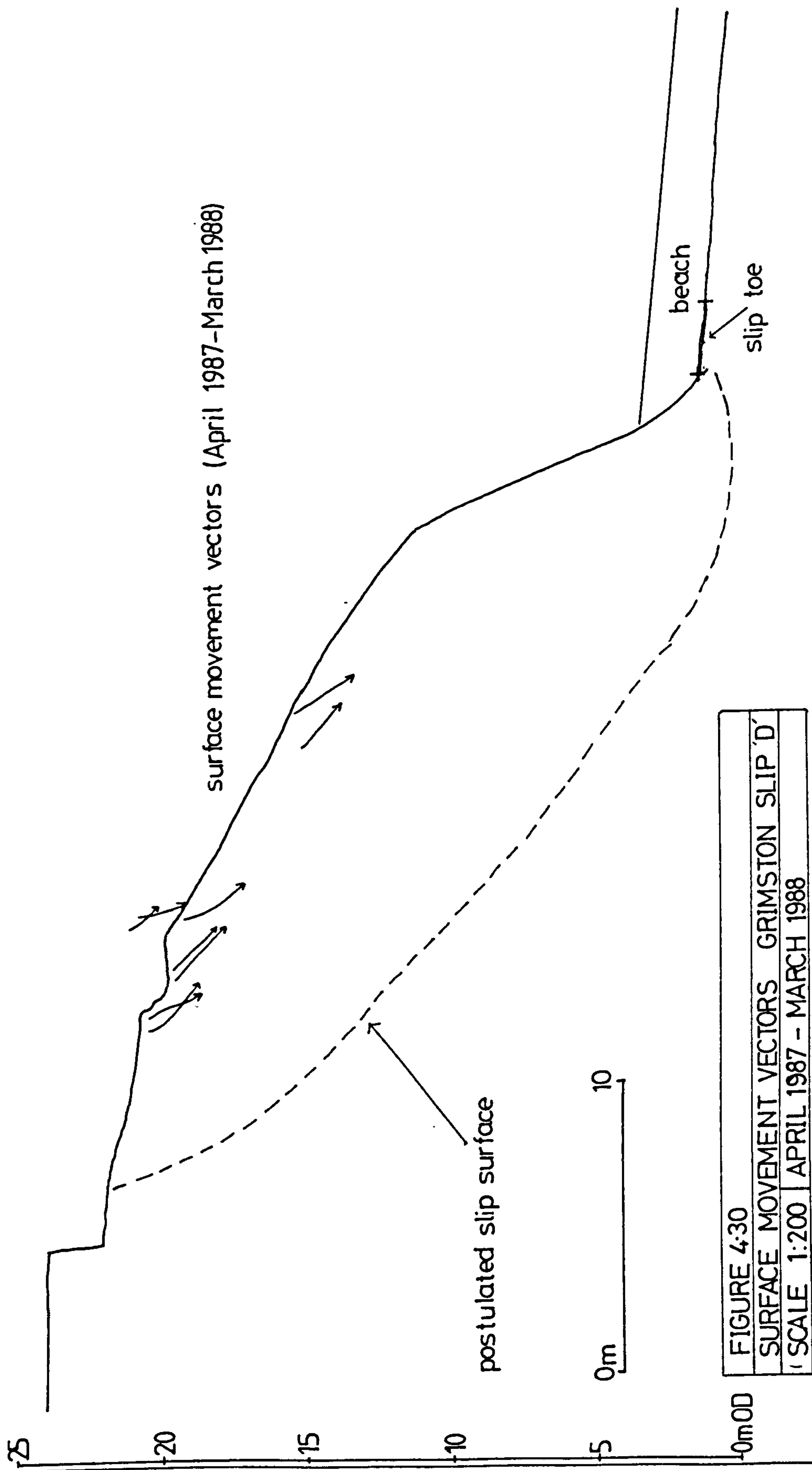


FIGURE 4:30
SURFACE MOVEMENT VECTORS GRIMSTON SLIP 'D'
SCALE 1:200 APRIL 1987 - MARCH 1988

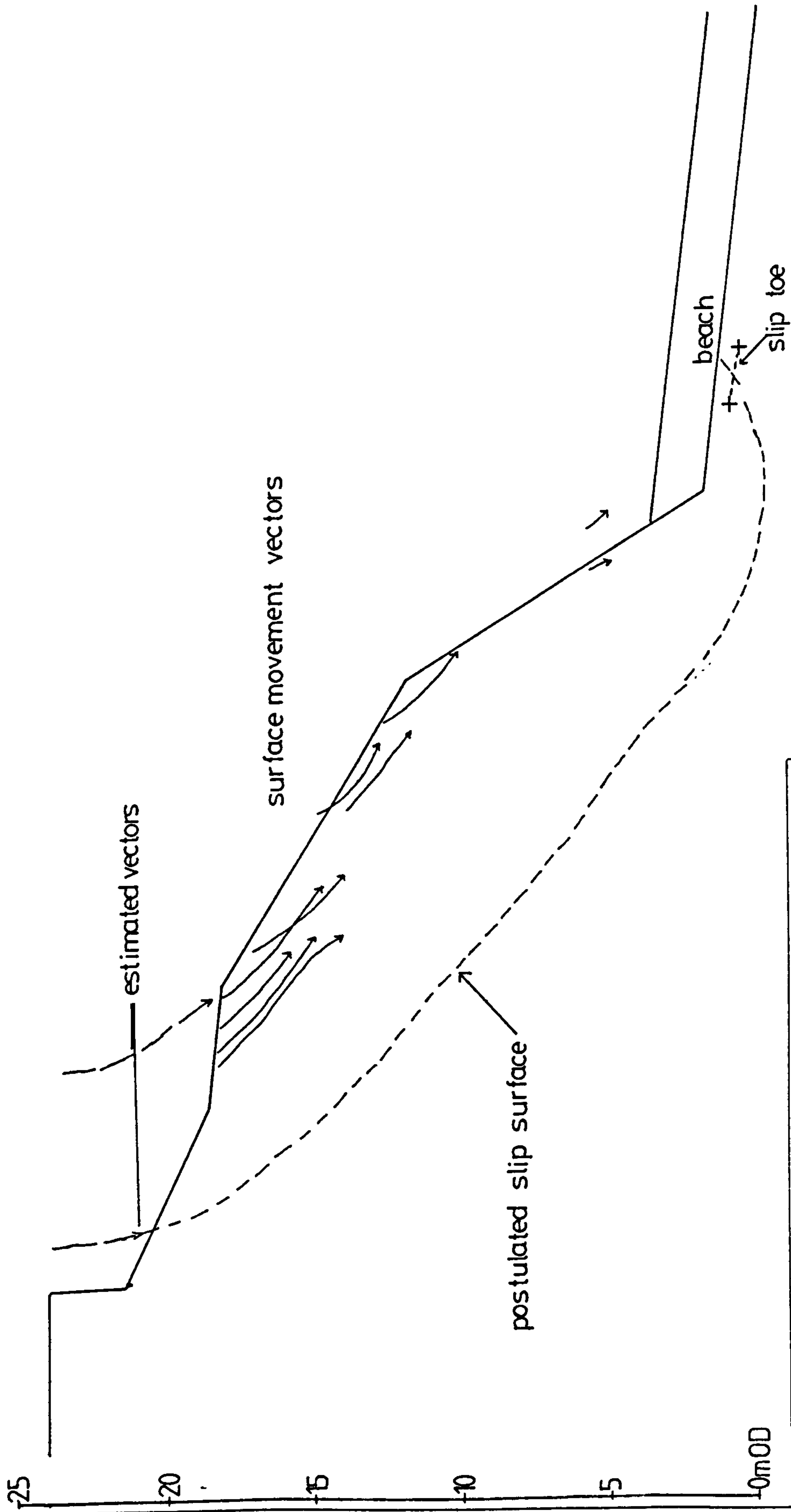


FIGURE 4-31
SURFACE MOVEMENT VECTORS GRIMSTON SLIP 'F'
SCALE 1:200 APRIL 1987 - MARCH 1988

used the assumption that surface movements were parallel to the slip surface. Tangents and normals to the surface movement vectors were drawn and considering the overall geometry of the slope, the slip surfaces were estimated. Because no definite measurements of the depth to the slip surface were made, these estimates could not be checked. Therefore in stability analyses a 'band' of slip surfaces which included the estimated one were examined.

The estimated slip surfaces comprise an approximately linear central portion with two circular portions at the head and toe. A circular failure surface could not be fitted with the observed ground movements, or be made to agree with the inclination of surfaces exposed and measured in other nearby slips. Although the slip toe was hidden by the beach during the survey, its position could be estimated between the limits shown, from exposure of different parts of the till shore platform on several visits. The rear scarp is near vertical, although its profile may have been modified by falls, and would not then represent the original slip surface.

The rate of movement of the two slipped soil masses at Grimston was calculated for the periods between surveys. The results are plotted as a histogram in Figure 4.32. Slip "F" has consistently moved at a faster rate than slip "D" throughout the period July 1986 to March 1988, probably because of the difference in size of the slips. Slip "D" is 70m long and 11m wide, compared with 30m and 5m for slip "F". The higher

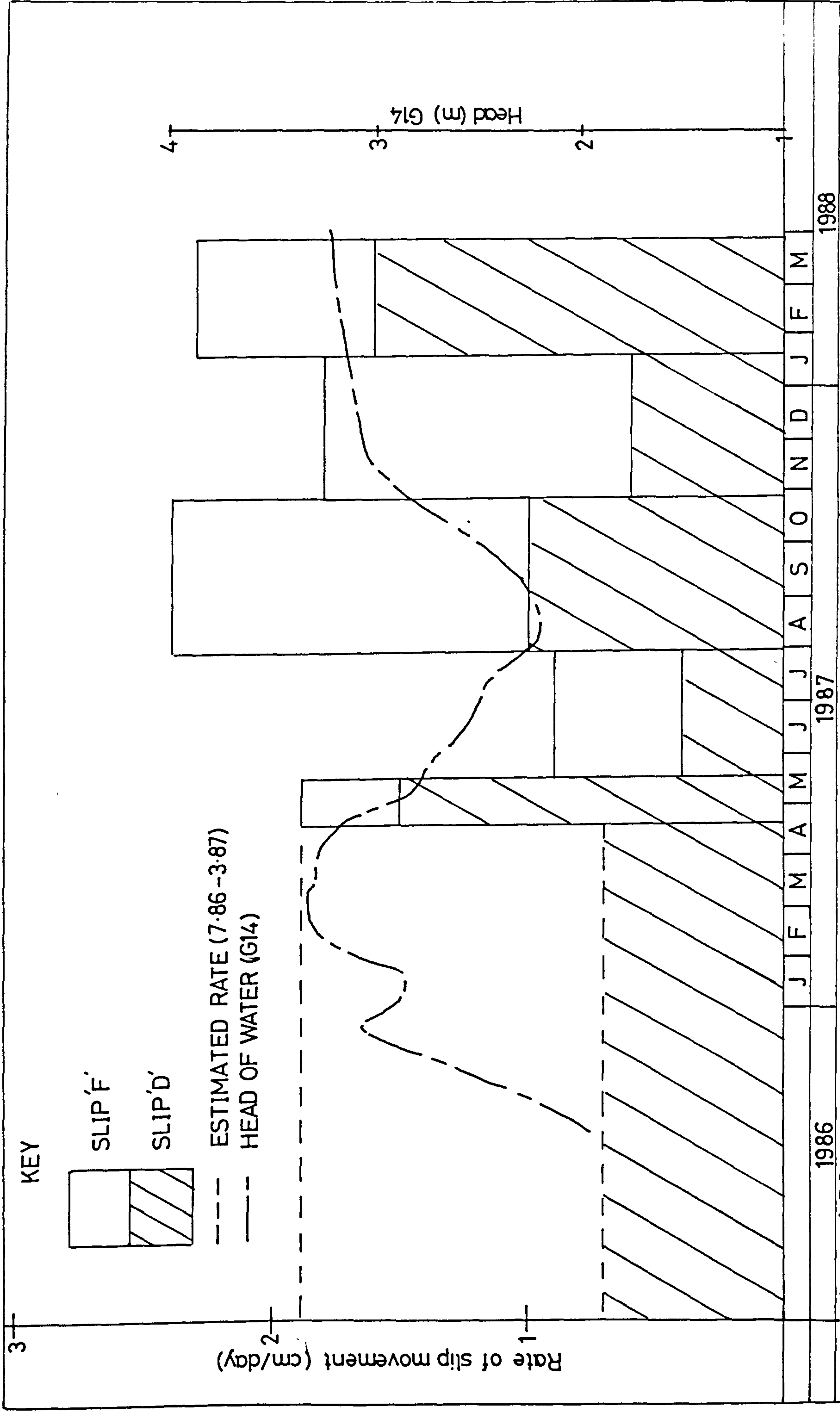


Figure 4:32
Histogram to show rate of movement of slips D and F at Grimston

profile of slip "D" (see Figure 4.28) may need a greater amount of toe erosion to activate movement. Also, erosion of the toe of slip "F" was more severe than slip "D".

The rate of movement showed considerable fluctuations, which are attributed to seasonal variation in pore water pressure, lowering of the beach and toe erosion. A typical plot of pore water pressure with time for a shallow piezometer (G14) is shown for comparison on the figure. There is some correlation of low and high rates of movement with low and high pore water pressure, particularly of the low rate between May and July 1987, and the high rate between November 1987 and March 1988. An exception is the high rate between August and October 1987 which occurred when pore water pressure was low. However the beach was lower than normal during this period, resulting in increased toe erosion and faster movement (see Section 5.5). If the rate of slip movement continues as observed, the time for removal of the slipped blocks and hence for a cycle of failure can be estimated. Cycle times of 2 and 6 years were obtained for slips "F" and "D" respectively, corresponding to a mean rate of coastal recession of 2.6m/yr. This is consistent with the rate of 2.4m/yr calculated for the site between 1974 and 1985, and is in fair agreement with the cycle time at Cowden of 5 to 6 years found by Butcher (personal communication).

4.3.10 Rolston Site Changes and Slip Geometry

Erosion of the cliff foot was more intense than at Grimston, and a complete cycle of failure occurred in 12-18 months. The clifftop has receded an average of 4m between July 1986 and March 1988, by four major landslips and several falls of weathered clay. Erosion of the cliff foot, particularly of a less resistant red clay layer, undercut blocks of till in the lower cliff face causing them to fall (see Section 3.2.3). Groundwater seeping from a sand layer at half cliff height caused seepage erosion and activated mudslides which removed fall and slip debris. Figure 4.33 shows the changes in cliff profile in front of line "B" caused by these processes. A more detailed account of site changes is given in Appendix F.

The slipped soil masses at Rolston were broken up quickly by the toe erosion, thus it was impossible to measure their movements. Two of the four major slips were surveyed soon after their occurrence. The profiles are shown in Figure 4.35, together with the estimated slip surface. The shape of the slip surface could not be found precisely owing to the lack of slip movement measurements. However it was probably similar to the Grimston slips, from examination of exposed slip surfaces nearby. Also the slipped soil masses are backtilted less than would be expected for a circular slip. The position of the toe was not influenced by the unusual red clay layer near the cliff foot.

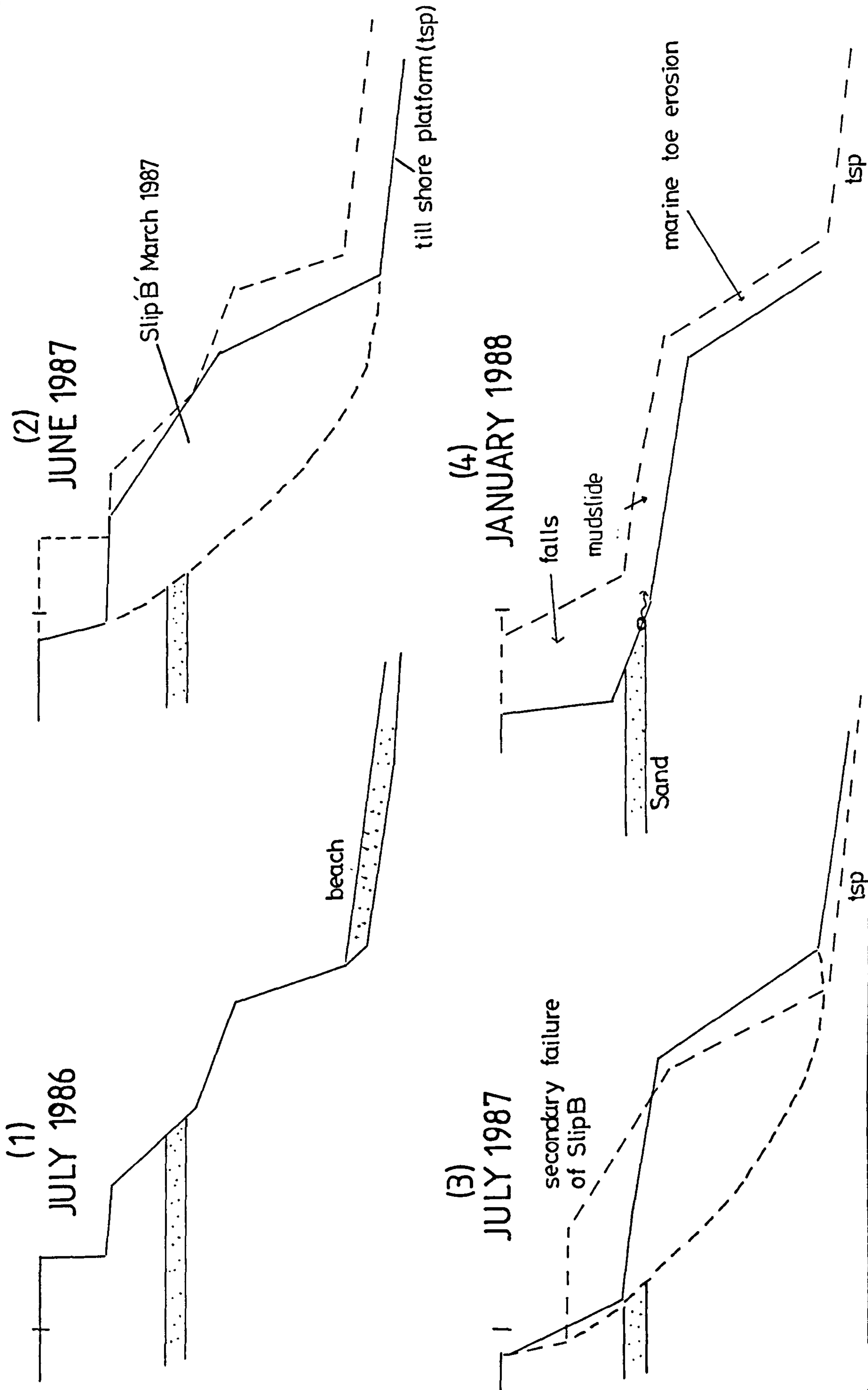


FIGURE 4-33
 ROLSTON CLIFF PROFILE CHANGES
 SCALE 1:250

previous profile dashed

SLIP 'R' JUNE 1987

SLIP 'B' JULY 1987

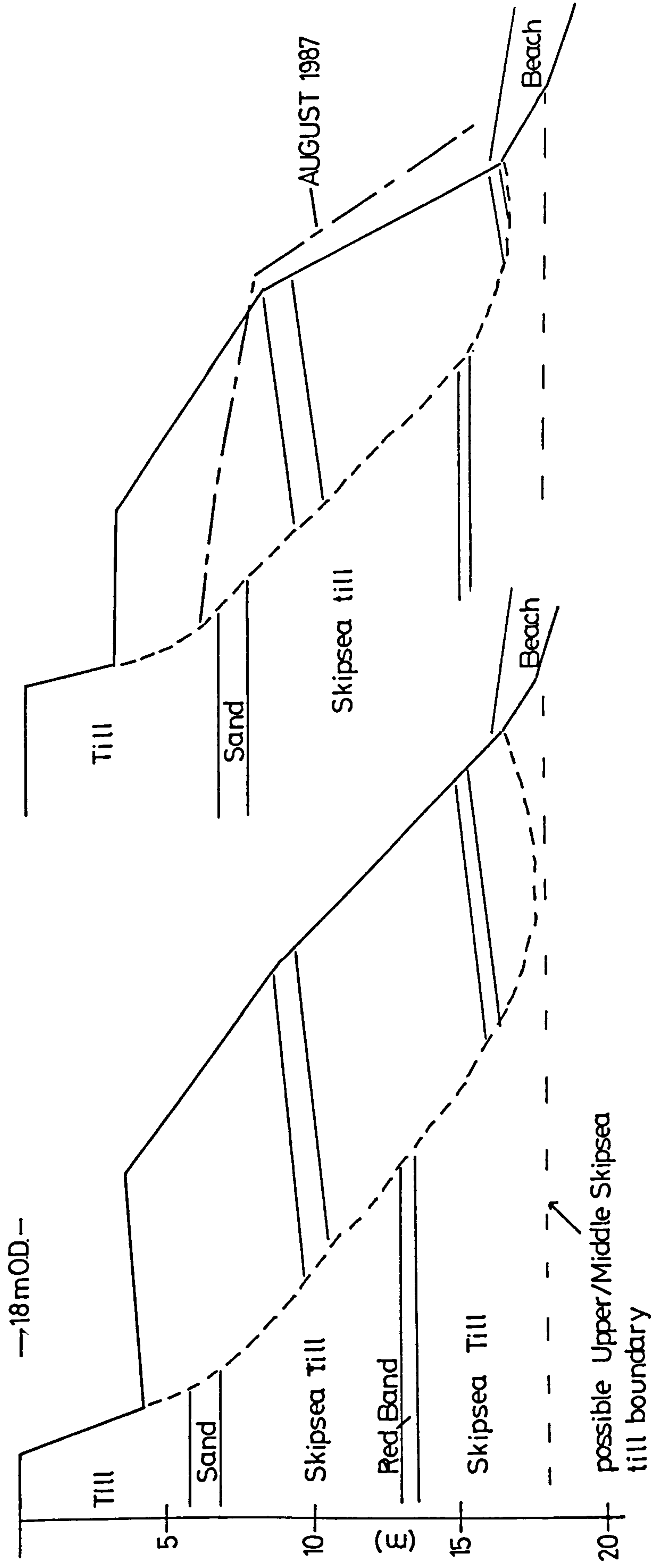


FIGURE 4.35
ROLSTON SLIP PROFILES
SCALE 1:200

4.4 Field Testing

To enable estimates of the seepage pattern and time for dissipation of excess pore water pressure to be made, more detailed knowledge of the distribution and permeability of the cliff materials was needed. Two methods of field testing, static cone penetration tests and in situ falling head permeability tests, were used for investigation.

4.4.1 Static Cone Penetration Tests

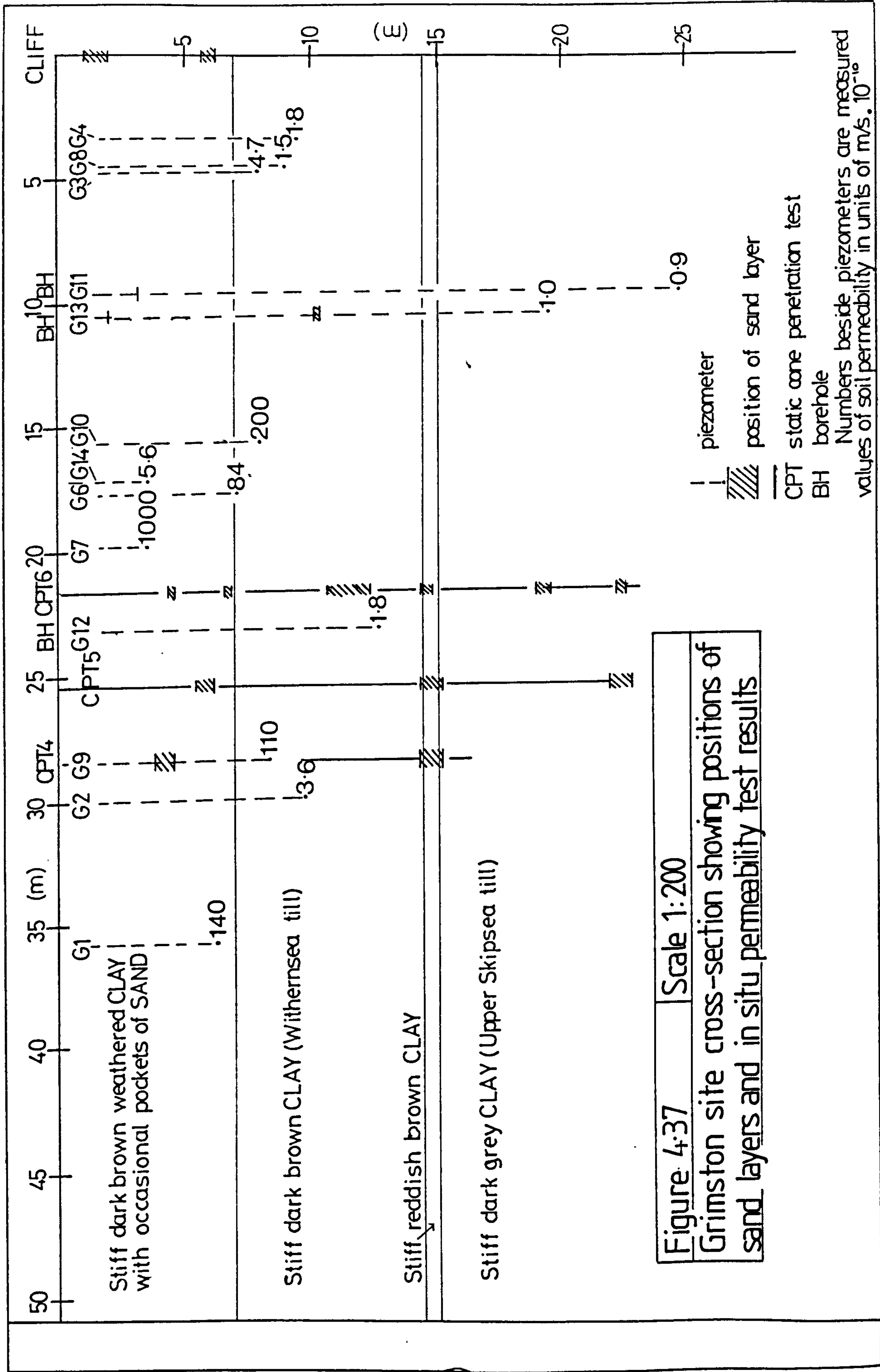
Static cone penetration tests were done by the geotechnical consultants Fugro Limited. The work was carried out at cost as a contribution to the coastal erosion project in July 1986. Three tests were done at each site, to a depth below the toe of the cliff. Two of the tests measured friction and cone resistance and the third, using a piezocone, measured cone resistance and pore water pressure. The main purpose of the tests was to identify the presence of any sandier layers within the tills. The variation of undrained shear strength with depth was also estimated. The positions of the tests (numbered 1-6) are shown on Figures 4.1 and 4.3, and the test records are reproduced in Figure A4.36, Appendix G.

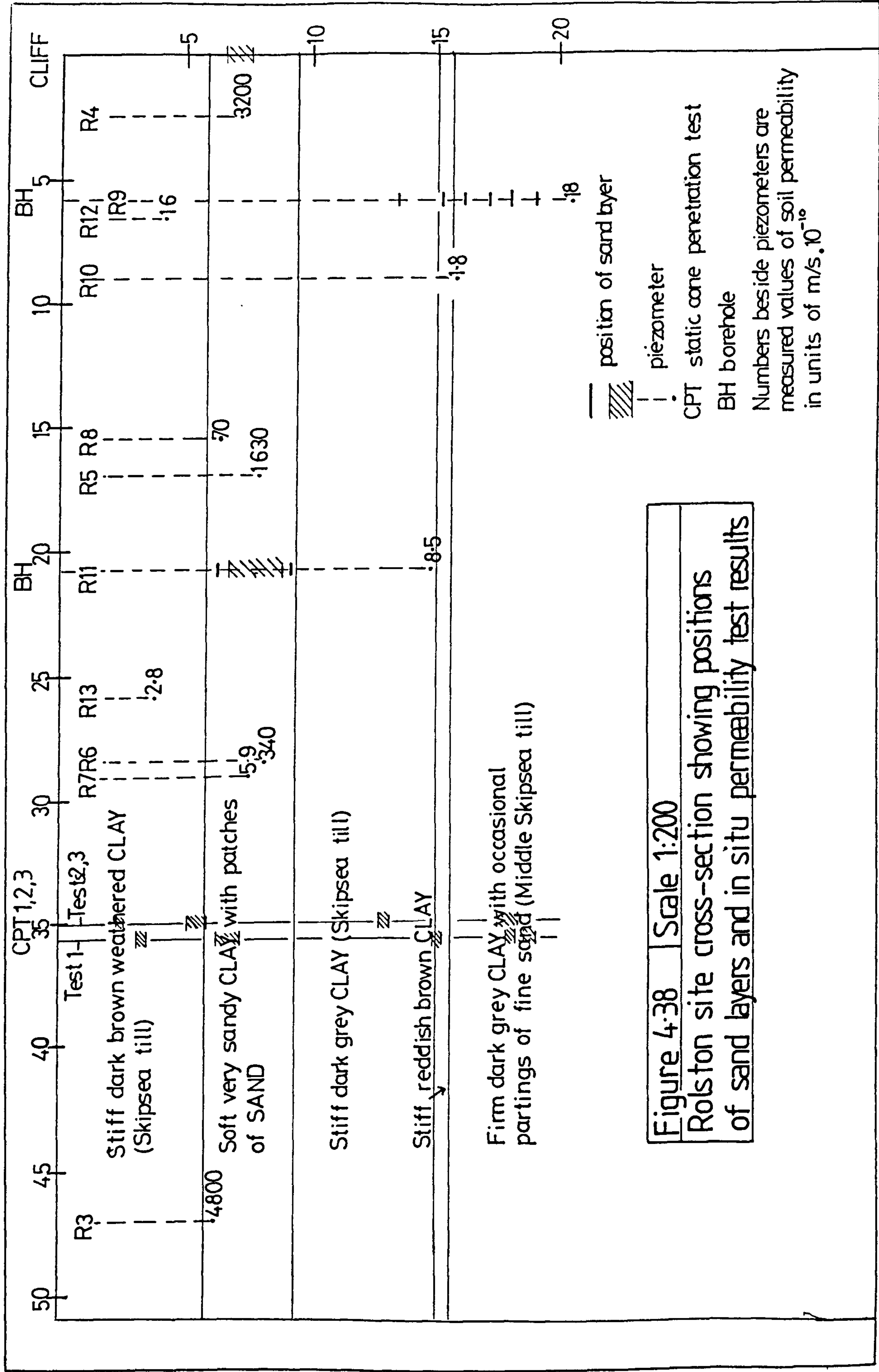
Sand layers are identified by a large increase in cone resistance (q_c), coupled with a decrease in friction ratio, and in piezocone tests by a reduction in excess pore pressure

ratio. Layers less than 100mm thick are unlikely to be detected (Meigh, 1987). The positions of suspected sand layers (from the test records) are indicated on site cross-sections in Figures 4.37 and 4.38. Clayey sand or silt layers identified from borehole samples and cliff face mapping are also shown.

At Rolston there is a laterally persistent clayey sand layer of about 1.0m thickness and varying between 6m and 9m depth. Two thinner, near horizontal layers were indicated at between approximately 15m and 16m and in tests 2 and 3 at 19m depth in all the static cone penetrometer tests. These correspond to boundaries between till divisions, frequently observed to have sand or silt layers near them when exposed in the cliff. The piezocone recorded negative pore water pressure until 14m depth, although this phenomenon could have been caused by loss of saturation of the tip. Below 14m pore water pressure was positive and increased with depth. Sharp falls in excess pore water pressure ratio (see Figure A4.36, test 3) may indicate the presence of thin sand layers, particularly at the aforementioned 16m and 19m depths. However some of the falls may be caused by the suction created when the cone encounters gravel or cobbles in the till (Lunne et al; 1986).

A sand layer was encountered from 2-4m depth at Grimston in some of the holes drilled by the "Minuteman" for installation of the piezometers. High values of cone resistance were also recorded at the same depth, but these are probably caused by desiccation of the clay. No other major sand layers were





identified at Grimston by the static cone penetrometer tests, but weak indications at several depths may correspond to isolated sand pockets or thin lenses. An indication at 15m depth was observed in all three tests, which coincided with a reddish brown clay layer at the Skipsea/Withernsea till boundary. Thin sand layers have been observed at this horizon elsewhere, but were not visible at the Grimston cliff face. The piezocone recorded positive pore water pressure from 3m depth at Grimston, and falls in excess pore water pressure ratio at 12m depth (see Figure A4.36, test 6) may show a possible sand layer. Cliff face mapping and high in situ permeability test results (see Section 4.4.2) revealed a sand layer between 6m and 8m depth, not shown by the cone tests. This may be because it is not continuous beneath the site.

The undrained shear strength (c_u) was estimated from the cone resistance (q_c) using the equation:

$$c_u = (q_c - \sigma_v)/N_k \quad (1)$$

where σ_v is the total vertical stress

and N_k is the cone factor

N_k was estimated to be 18 from the results of tests at Cowden (Marsland and Powell, 1985). The profiles are shown in Figures 4.9 and 4.10, together with the results of laboratory unconsolidated undrained tests. Undrained strength steadily increased from 70kN/m² at 5m depth to 140kN/m² at 20m. Above

5m it varied from 50kN/m^2 near the surface to 200kN/m^2 at 3m depth. The fair agreement between cone and laboratory undrained strengths is shown by the figures. Some of the laboratory specimens had low undrained shear strengths, due to high sand or silt content. There is no significant difference between the till types, other than that due to the general increase of c_u with depth.

4.4.2 In Situ Falling Head Permeability Tests

The in situ permeability of the soils was measured using falling head tests in all the Casagrande standpipe piezometers. The head of water in the standpipe was raised by 2-3m. Where the equilibrium level in the piezometer was more than 3m below the surface, the fall of water level was observed with a dipmeter. If it was nearer the surface an apparatus consisting of graduated pipes allowed the head to be raised above ground level, and the fall observed directly.

Gibson (1963) states that while the rate of equalization in piezometers can be used to determine the permeability and compressibility of soil surrounding the piezometer tip there are some disadvantages. Firstly the method is indirect for a closed system since the volume flexibility must be known. Secondly in a rising head test the soil around the tip will consolidate and then swell, and consequently the coefficient of

c will equal neither the coefficient of consolidation c_v , nor of swelling c_s but will have a value intermediate between them.

A constant head test in which a constant pressure head is maintained above the ambient water pressure in the soil while the rate of flow is measured is a more direct procedure for measuring permeability. However it was decided to use falling head tests because of the simplicity of the test compared with the constant head test. Also the flow rates in the constant head test would have been very low and difficult to measure. It was thought that the errors mentioned above would be small.

Constant head permeability tests in hydraulic piezometers were carried out by the Building Research Establishment at Cowden and are compared with the results obtained in this study later in this section.

The tests were interpreted using Hvorslev's method (Clayton et al; 1982). The shape factor was calculated from the assumed geometry of the sand cell and piezometer tip. Errors in estimating the final equilibrium level were found not to affect the calculated value of k by more than $\pm 20\%$. A linear relationship between $\log_{10}(H/H_0)$ and time was found in most of the tests. Two of the tests in piezometers at shallow depths (3m) showed a rapid fall in head during the first hour of the test. However when the results were replotted excluding this period, a linear relationship was obtained. This behaviour may have been caused by initial partial saturation of the sand cell

surrounding the piezometer tip, which then became fully saturated after water flowed in during the early part of the test. The test results for each piezometer are given on the site cross-sections in Figures 4.37 and 4.38.

The measured in situ permeability is influenced by the effective stress level and the type of soil surrounding the piezometer tip. The coefficient of permeability (k) is plotted with mean in situ effective stress (p') on a log-normal scale in Figure 4.39. The type of soil surrounding the tip was estimated from the results and the general site investigation. Bulk density, pore water pressure, and in situ stress measurements were used to calculate the mean effective stress at the tip. Two groups of results are distinguished on the basis of soil type. These are (1) clay tills of low permeability, and (2) clayey sand and till with sand partings of higher permeability.

In the clay tills a linear relationship between mean effective stress and $\log_{10}(k)$ can be idealized, shown on the figure. Some variation occurs about the line, particularly in the tests in weathered clay at low effective stresses (shallow depths). This may be caused by the presence of fissures or differences in soil grading.

The permeability of the clay tills varies from 9×10^{-11} m/s to 1.5×10^{-9} m/s. The results of constant head tests by the Building Research Establishment at Cowden gave permeabilities

of unweathered till in the range 1.6×10^{-11} m/s to 2.2×10^{-10} m/s (Marsland and Powell, 1985), ie. on average about a factor of 7 lower than the falling head tests. This difference could be due to a real difference in the permeability of the in situ tills, or due to experimental error.

The correlation of results of high permeability with the position of sand layers, particularly at Rolston, is shown on site cross-sections in Figure 4.38. However at Rolston between 15m and 20m depth, two of the piezometers in till ("R9" and "R11") gave permeabilities an order of magnitude higher than expected. This is probably because of sand partings in the till, which were seen in the U100 samples, and indicated in the cone tests at this depth.

4.5 Geophysical Surveying

4.5.1 Seismic Refraction Surveys

The thickness of the beach sand overlying the till shore platform was determined using shallow seismic refraction surveys at Grimston. It was necessary to know beach thickness for use in stability analysis, as the slip toe at the sites emerged below beach level. Problems with the equipment prevented successful use of the method at Rolston. A Hunttec seismograph with two geophones was used, and the energy source provided by a sledgehammer and plate. The maximum range in

beach sand was 30m (limited by wave noise), thus the depth of investigation was about 3m. A graph of the first arrival time plotted with distance was produced by the equipment. Four surveys were carried out parallel to the cliff, and one perpendicular, at several distances from the cliff foot. The distance-time graphs are given in Figure A4.40, Appendix H, and the results summarized in Figure 4.41. The large contrast in seismic velocity between till and beach sand is apparent, enabling the cross-over range to be found accurately, and the depth of the sand/till interface to be calculated. The beach profile was levelled at the same time as the surveys, enabling the surface of the till to be plotted as shown in Figure 4.28. The average thickness of beach sand found at the cliff foot was about 2m, when the beach was unaffected by storm breakdown (see Section 2.3).

4.5.2 Electrical Resistivity Surveys

Electrical resistivity surveys were done at the sites of Rolston and Grimston and at Monkwith, Hornsea and Aldbrough (see Figure 1.1). The aim of the surveys was to search for lateral variations in resistivity behind the cliff, which may be caused by the opening of fissures associated with stress relief. Resistivity profiles were measured with an ABEM 'Terrameter' using a Wenner electrode array. Profiles were measured perpendicular to the cliff using a 1m electrode spacing, and ran up to 40m back from the cliff edge.

Date of Survey	Distance from cliff (m)	Seismic Velocity V1 (sand) (m/s)	V2 (till) (m/s)	Crossover range (m)	Thickness of sand (m)
June 1987	5	303	1500	3.7	1.5
June 1987	5	267	1500	3.2	1.3
March 1987	7.5 (0-15)	360	2080	4.2	1.8
March 1987	7.5 (0-15)	340	2000	4.8	2.0
March 1987	8	270	2700	4.5	2.0
March 1987	8	280	2600	4.2	1.9
June 1987	18	333	1500	2.0	0.8
June 1987	30	-	1625	-	0.0

Figure 4.41 Table of Grimston seismic survey results.

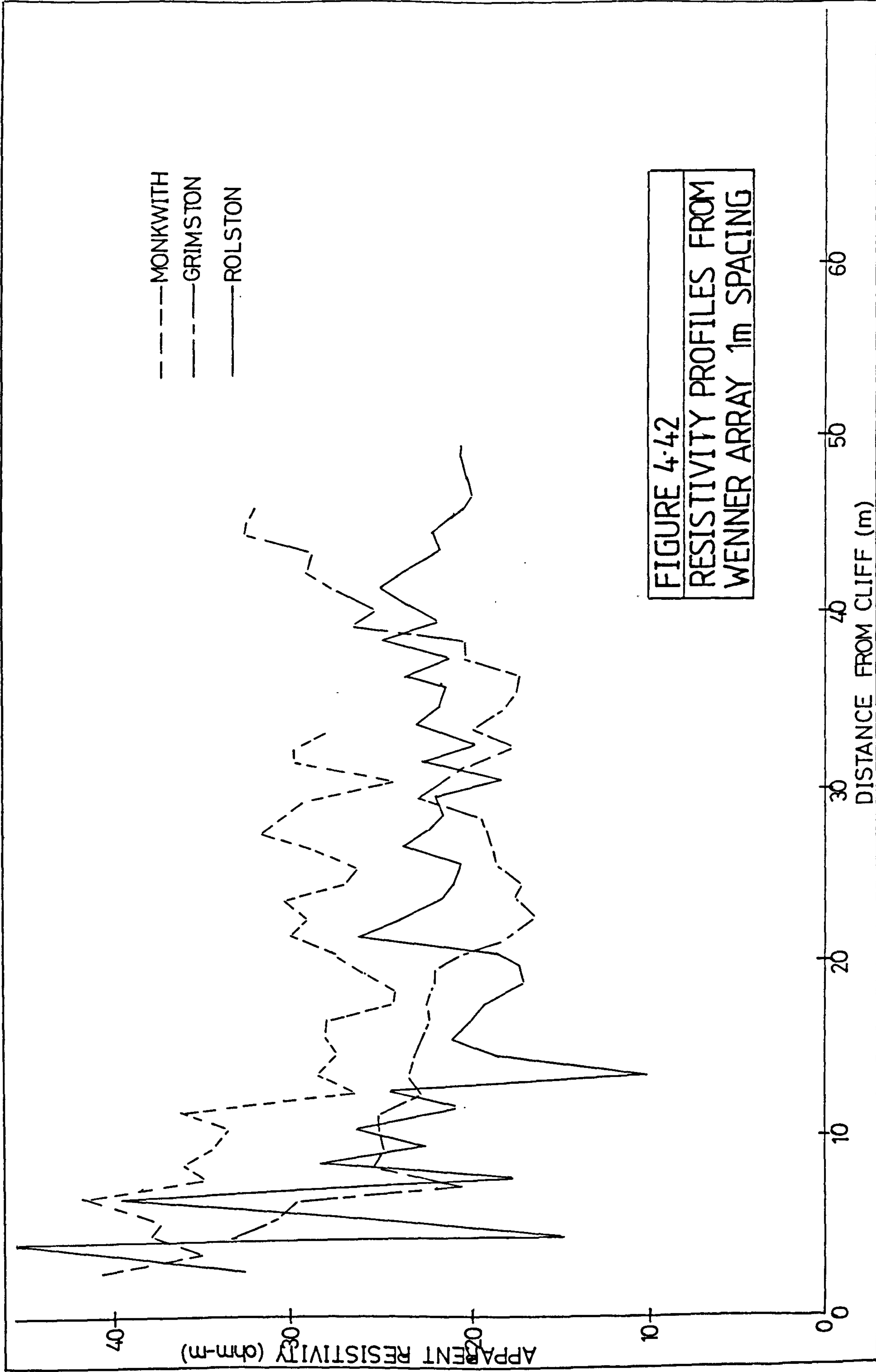
The apparent resistivity profiles are shown in Figure 4.42. At the sites of Rolston, Grimston and Monkwith, higher values of apparent resistivity were found up to 12m back from the cliff edge. The high values are probably due to the presence of fissures, but a fall in the water table near to the cliff could also have caused them.

4.6 Laboratory Testing

A programme of laboratory tests was conducted on U100 samples from the sites. Index tests have been described in Section 4.2.3. The coefficient of compressibility for use in analysis of groundwater conditions was measured by oedometer tests. The effective stress strength parameters were measured in triaxial tests, including some following a stress path where the radial stress is reduced to failure, for use in stability analysis. In the following sections the test specimens are referred to by a borehole number (see Figures 4.1 and 4.3) and a sample number separated by a slash, eg. (3/22).

4.6.1 Oedometer Tests

To estimate the time required for dissipation of depressed pore water pressure in the cliff, the field coefficient of swelling (c_s) was required. An estimate of c_s was obtained by combining laboratory coefficients of compressibility (m_v) with field



measurements of in situ permeability. The variation of permeability with effective stress was also studied, for comparison with the field results.

Ten specimens were tested, taken from samples at a range of depths and including Skipsea, Withernsea and weathered tills. The specimens were of 76mm diameter and 19mm thickness. They were tested in a Casagrande oedometer following the procedure described in B.S. 1377: 1975. The initial pressure was approximately equal to the in situ vertical effective stress. The succeeding pressures were changed by a factor of two in each stage, with two unload-reload cycles in eight stages. The settlements were measured by transducers and recorded by a computer. The water content was found at the end of the test, and used to calculate void ratio for each stage, assuming a specific gravity of 2.7. The coefficient of consolidation was calculated from the settlement curves using Taylor's method. The coefficients of c_s and m_v were combined to give k using the relationship:

$$k = c_s . m_v . \gamma_w \quad (2)$$

where γ_w is the unit weight of water

4.6.2 Oedometer Test Results

Curves of void ratio plotted with \log_{10} vertical effective stress for each sample are given in Figure A4.43, Appendix I. A typical curve is reproduced in Figure 4.44. The pre-consolidation pressure was found using Casagrande's construction where possible. It is compared with the current in situ mean effective stress (p') in Figure 4.45. The over-consolidation ratio varies from 1.3 to 2.7, showing the tills to be lightly overconsolidated.

Borehole	Sample	Depth (m)	Preconsolidation Pressure (kN/m ²)	p' (kN/m ²)	Over- consolidation ratio (O.C.R)
3	8	4.3	170	64	2.7
6	8	4.5	108	76	1.4
6	15	8.6	220	144	1.5
3	18	9.7	220	174	1.3
6	29	17.0	580	303	1.9

Figure 4.45 Preconsolidation pressure and O.C.R.

The coefficient of compressibility (m_v) has been plotted with the mean effective stress for each load stage in Figure 4.46. The results from consolidation and swelling stages are differentiated. For the consolidation stages the trend is for m_v to reduce with increase in effective stress, although there is large variation between specimens. This may be caused by sample disturbance, or variations in grading and stress history of the specimen. In the swelling stages the trend is the same,

SAMPLE 4/10 Skipsea till
Depth 24.5-25.0m

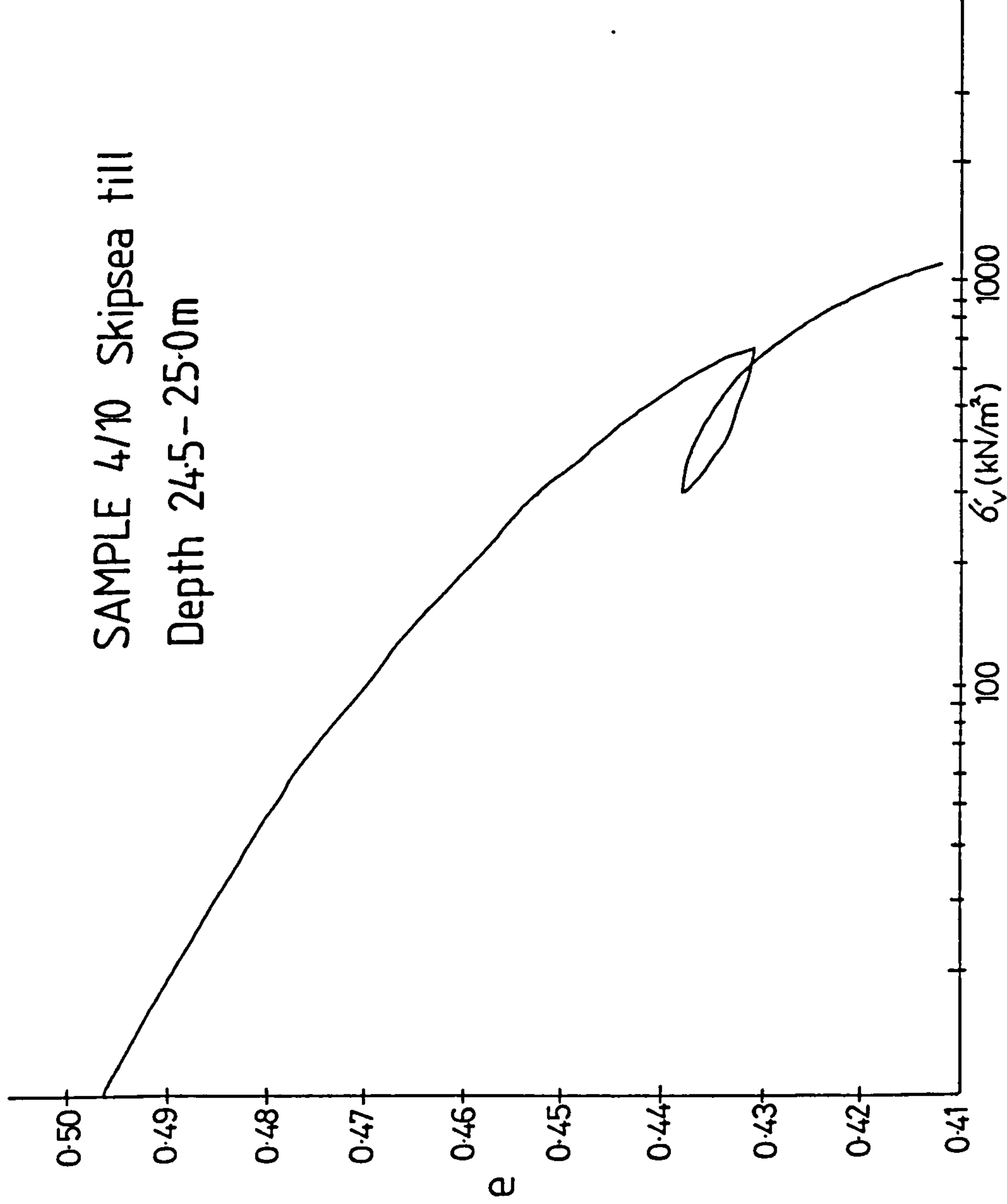


FIGURE 4.44

Typical curve of void ratio plotted with \log_{10} vertical effective stress

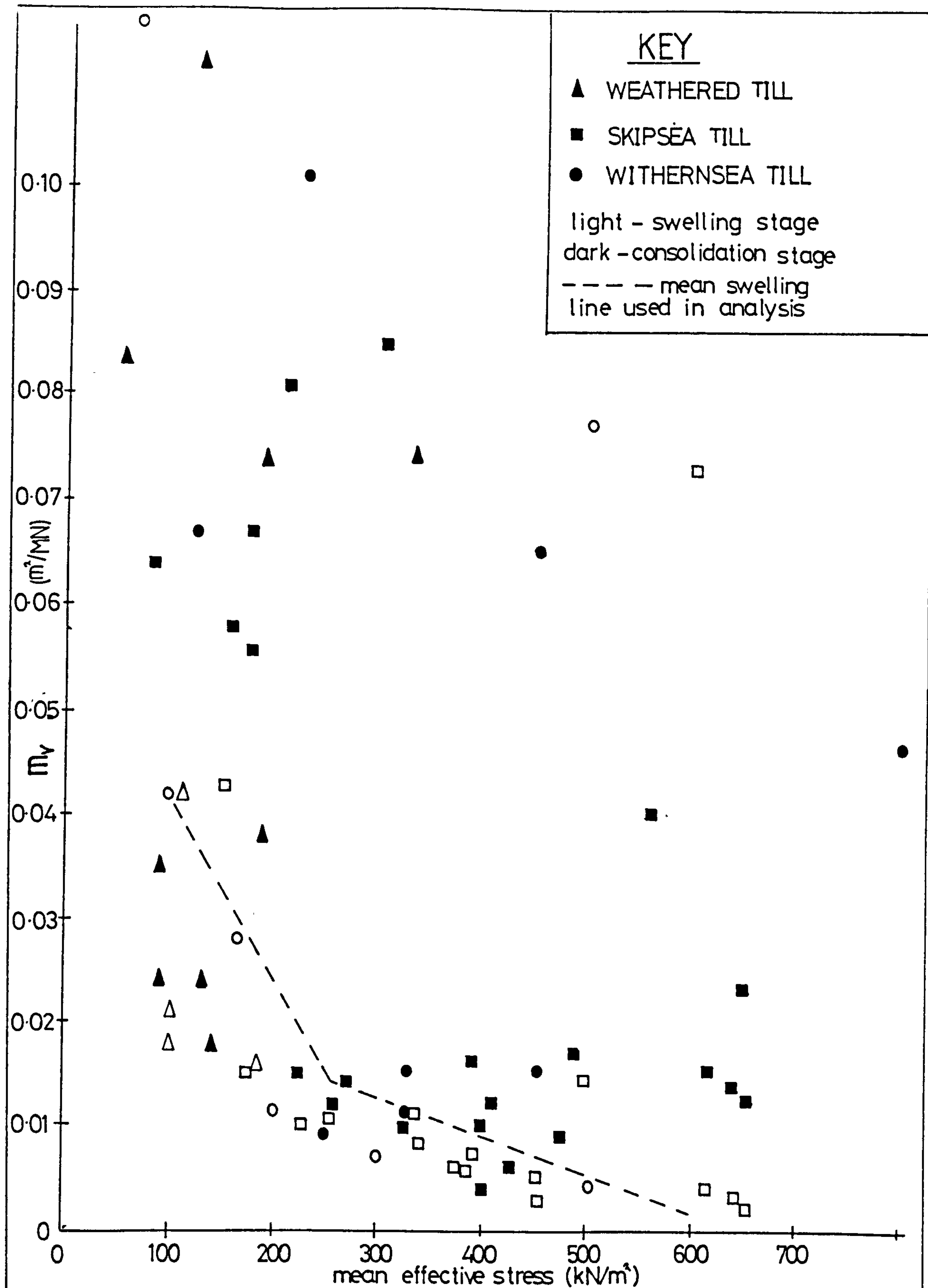


FIGURE 4.46

m_v plotted with mean effective stress for each stage of oedometer tests

but a lower compressibility is obtained. A mean line was drawn for the swelling stages to estimate average values for use in analysis.

The coefficient of permeability (k) is plotted with mean effective stress for each stage of the tests in Figure 4.47. The results are grouped by till type, and swelling and consolidation stages are differentiated. A reduction in k with increasing effective stress is shown, although there is a wide variation for the same reasons causing the variation in m_v . The relationship between permeability and void ratio shows the same trend as found in lodgement till from the Vale of St. Albans by Little and Atkinson (1988). Skipsea till appears to be slightly more permeable than Withernsea till, which may be caused by coarser grading (see Section 4.2.3). The mean line of the variation of k with mean effective stress determined from field in situ tests is shown on the figure for comparison. The laboratory values are about an order of magnitude less than the in situ values. This difference may be caused by the small specimen size, which excludes any macro fabric the till may have in situ.

4.6.3 Triaxial Tests

The peak effective stress strength parameters of the tills were required for use in stability analysis of first-time slides. Recent work has shown the major influence of the laboratory

FIGURE 4.47

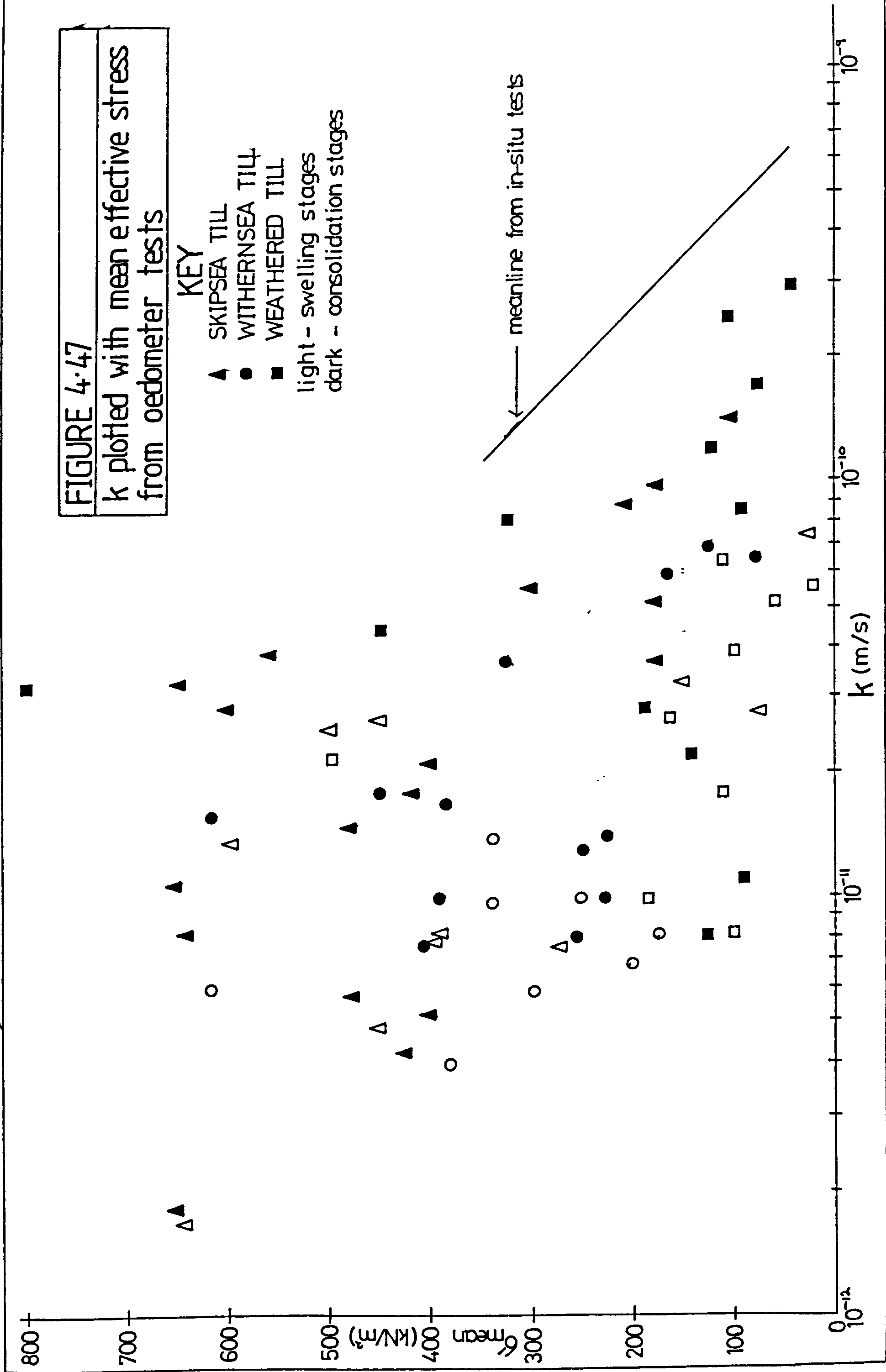
k plotted with mean effective stress
from oedometer tests

KEY

- ▲ SKIPSEA TILL
- WITHERNSEA TILL
- WEATHERED TILL

light - swelling stages
dark - consolidation stages

— meanline from in-situ tests



stress path followed on these parameters (Burland and Fourie, 1985). The strength associated with conventional triaxial tests has been established for the Holderness tills in work by Marsland and Powell (1985) (see Section 2.5.4). It was determined during this study on examples of till from the sites by seven sets of tests conducted on Skipsea, Withernsea and weathered tills. The effect of the stress path followed on the strength of the tills was investigated in three further sets of tests, in which the mean total stress was reduced to failure (by decreasing the radial stress). These tests simulate more closely the changes in stress behind a rapidly eroding cliff as discussed in Section 5.3.3.

4.6.4 Triaxial Test Procedure: s Increasing ("Standard" Tests)

Sets of three specimens of 38mm diameter and 76mm length were extruded from U100 open drive samples. The specimens were saturated under a back pressure in the triaxial apparatus for 24 hours, before consolidation to effective stresses of half, one and two times the effective in situ vertical stress. On completion of consolidation they were sheared at a constant rate of strain of 0.064mm/min, selected using the consolidation stage results. The deviator stress was applied in the standard way by increasing the axial stress. These tests are referred to hereafter as "standard" tests to distinguish them from the "reduction" tests described in the next section. The typical

time to failure (~20% strain) was four hours. The tests were undrained and pore water pressure was measured. All the measurements were taken by transducers linked to an ADU and BBC microcomputer. The current readings could be displayed during the test and recorded on disk at the finish.

4.6.5 Triaxial Test Procedure: s Reducing ("Reduction" Tests)

The specimens were of the same dimensions and were saturated and consolidated as described above (except in one test where they were anisotropically consolidated with a K_0 of 0.7). To apply the deviator stress during the shearing stage, the radial stress was reduced in decrements and the axial stress adjusted manually to remain constant, after allowing for deformation of the specimen. This was possible because a modified loading ram on the triaxial cell allowed axial and radial stresses to be varied independently. The continuous display of readings was essential for accurate control of the axial stress. The tests are referred to hereafter as "reduction" tests.

After a decrement of radial stress was applied, strain and pore water pressure changes occurred in the specimen. The next decrement was not applied until the rate of strain fell below that used in the "standard" tests, thus the average rate of strain was the same as for "standard" tests (ie. 0.064mm/min). The size of the decrements was reduced during the test to avoid high rates of strain in the later stages. Readings were taken

at intervals of one third percent strain until the sample had obviously failed or 20% strain was reached. The typical time to failure was four hours. A computer program calculated strain and deviator, mean total and mean effective stresses, after applying a correction for the rubber membrane and filter paper.

4.6.6 Triaxial Test Results: ("Standard" Tests)

Curves of deviator stress and pore water pressure with strain for a sample of Skipsea till (3/22), typical of the "standard tests" are shown on Figure 4.48. The non-brittle behaviour of the till is clear. Pore water pressure initially rose and then fell as dilation occurred, and became negative (relative to the back pressure) in tests at low radial stresses. Mohr circles are drawn for the maximum deviator stress in Figure 4.49, and the effective stress strength parameters found from the common tangent. All the standard triaxial tests were analysed in this way and the results are presented in Figure A4.50, Appendix J.

Stress paths were drawn for each test and are shown in Figures 4.51 and 4.52, differentiating between Skipsea and Withernsea till. They are consistent for each till, and have an 's' form at high stresses and an 'r' at low, which may be explained by the higher degree of overconsolidation at low stresses. The stress paths are similar in shape to those measured by Marsland and Powell (1985) in tests on till from Cowden (see Figure

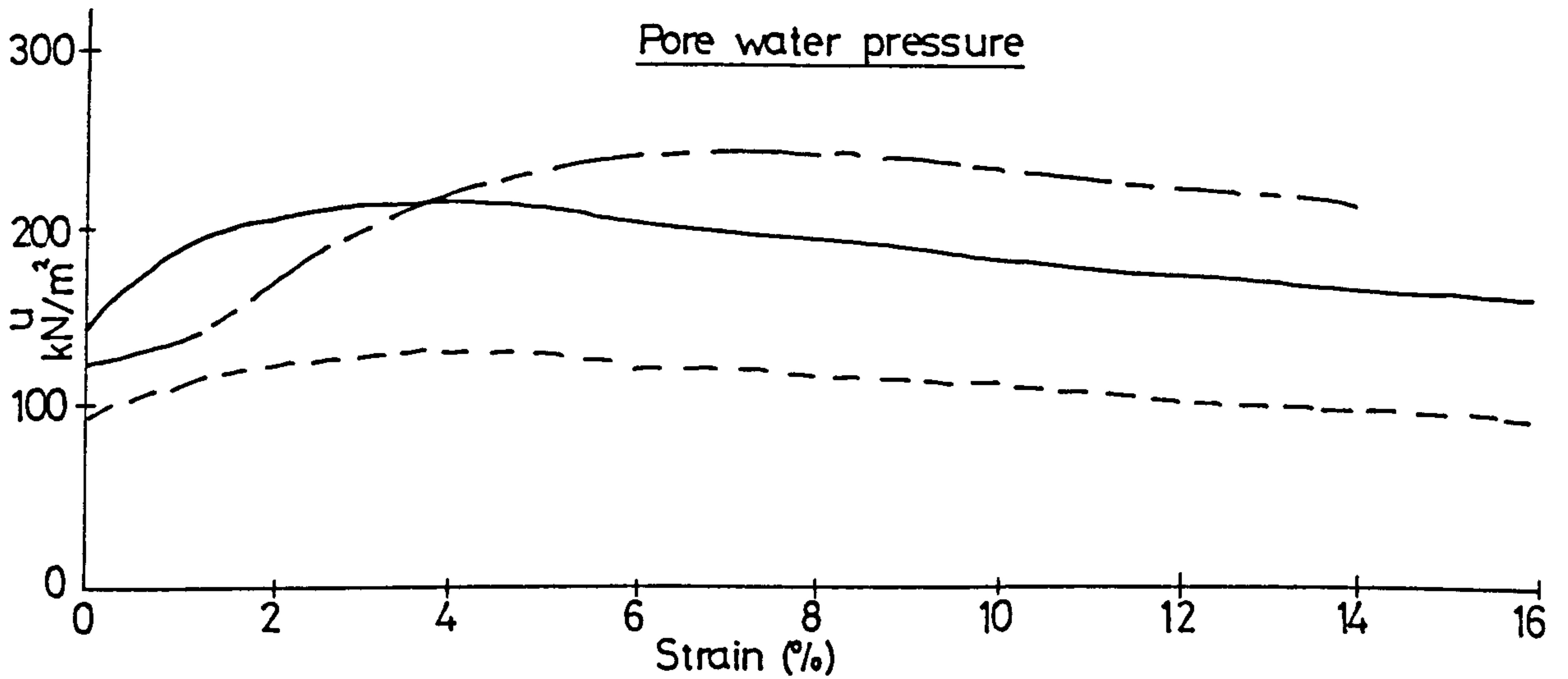
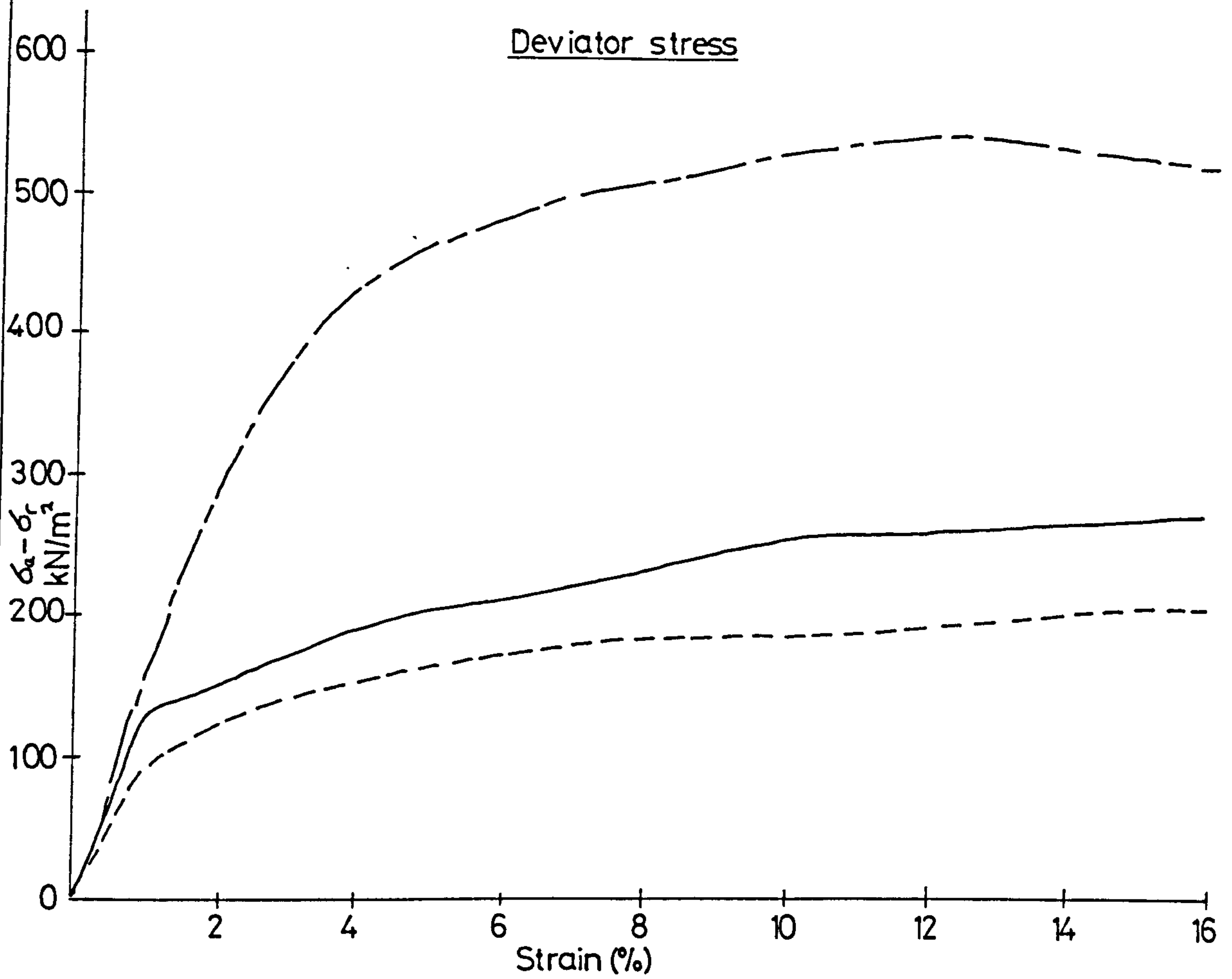


FIGURE 4.48

Deviator stress and pore water pressure plotted with strain for sample 3/22 ('standard' tests)

FIGURE 4.49
 MOHR CIRCLES SAMPLE 3/22
 'STANDARD' TESTS

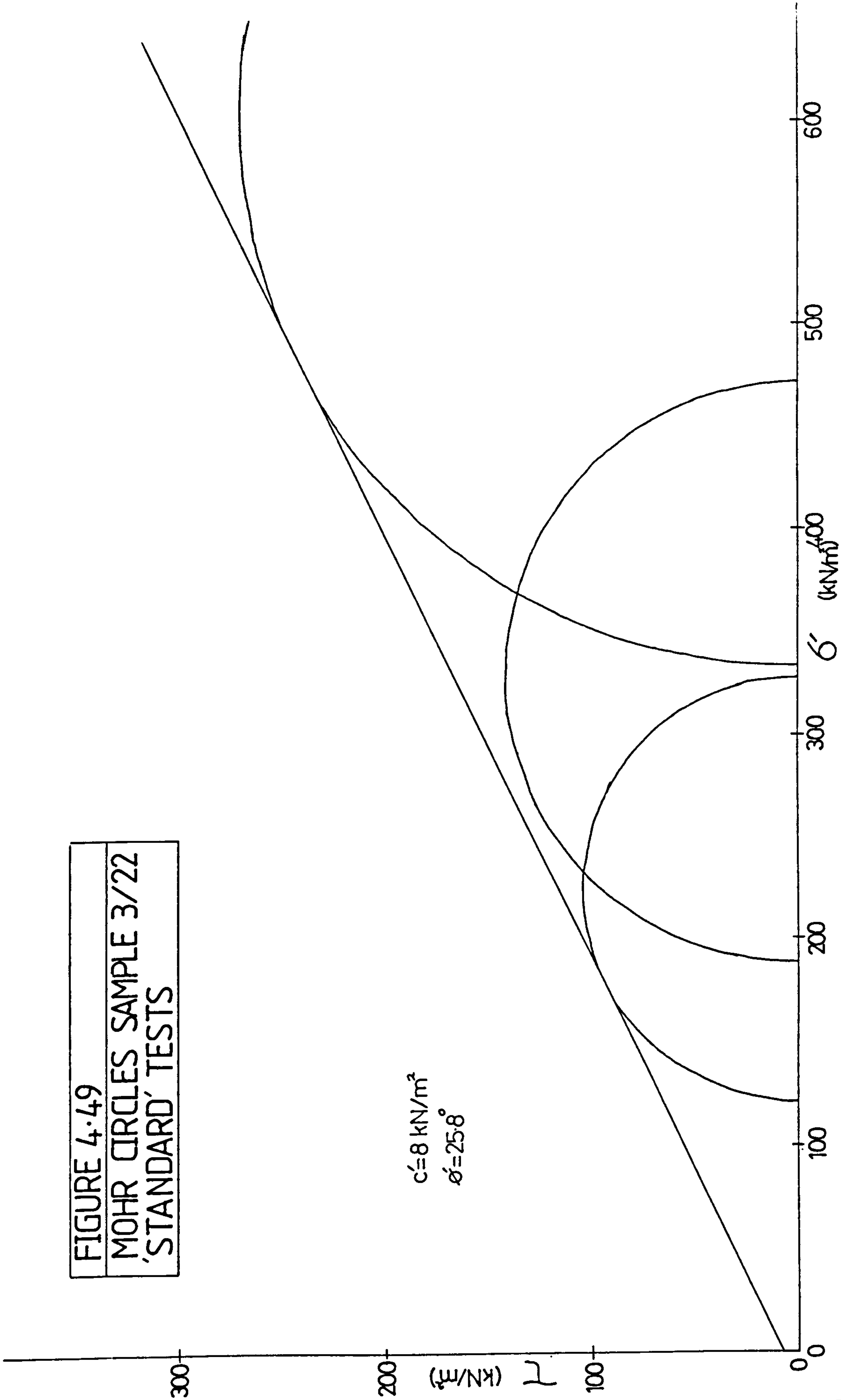
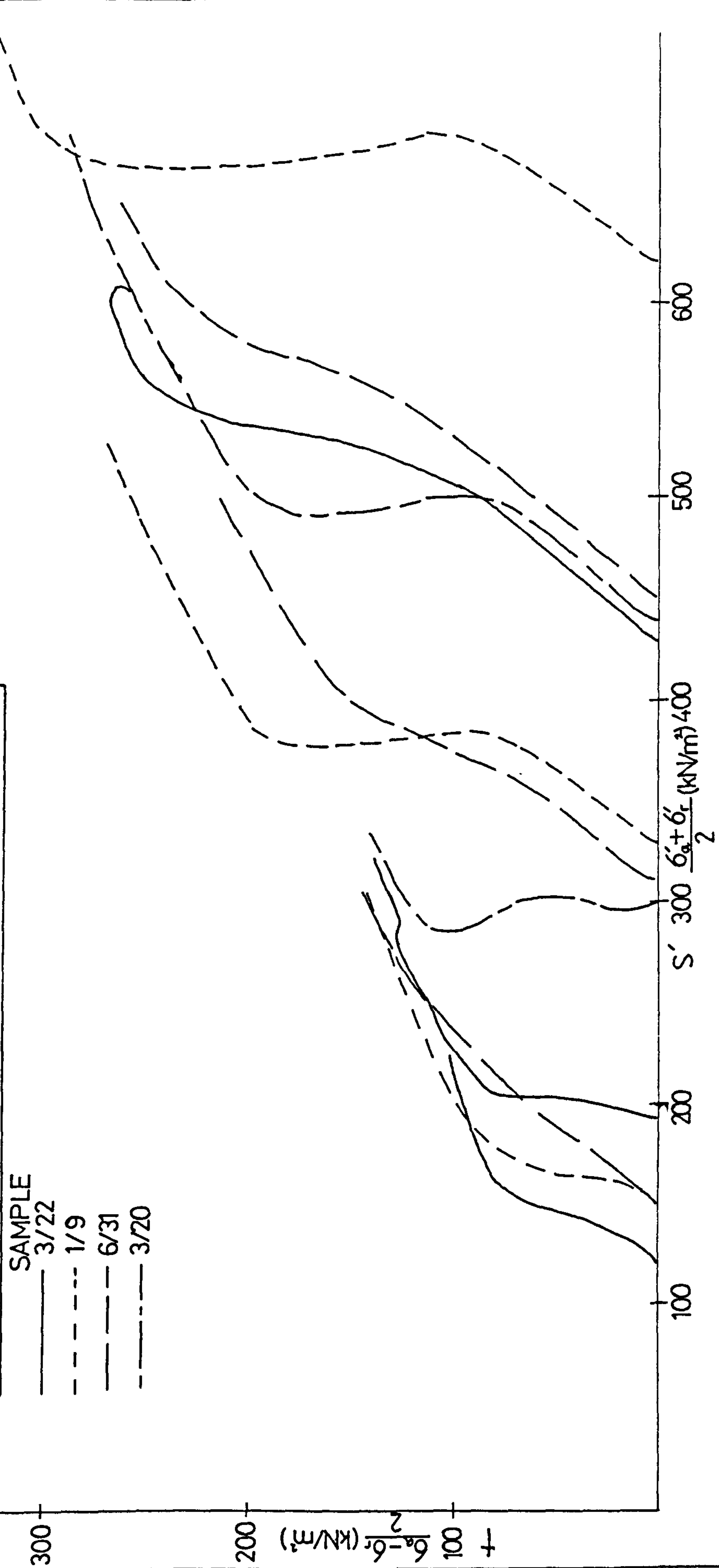


FIGURE 4-51
EFFECTIVE STRESS PATHS FOR 'STANDARD'
TESTS ON SKIPSEA TILL



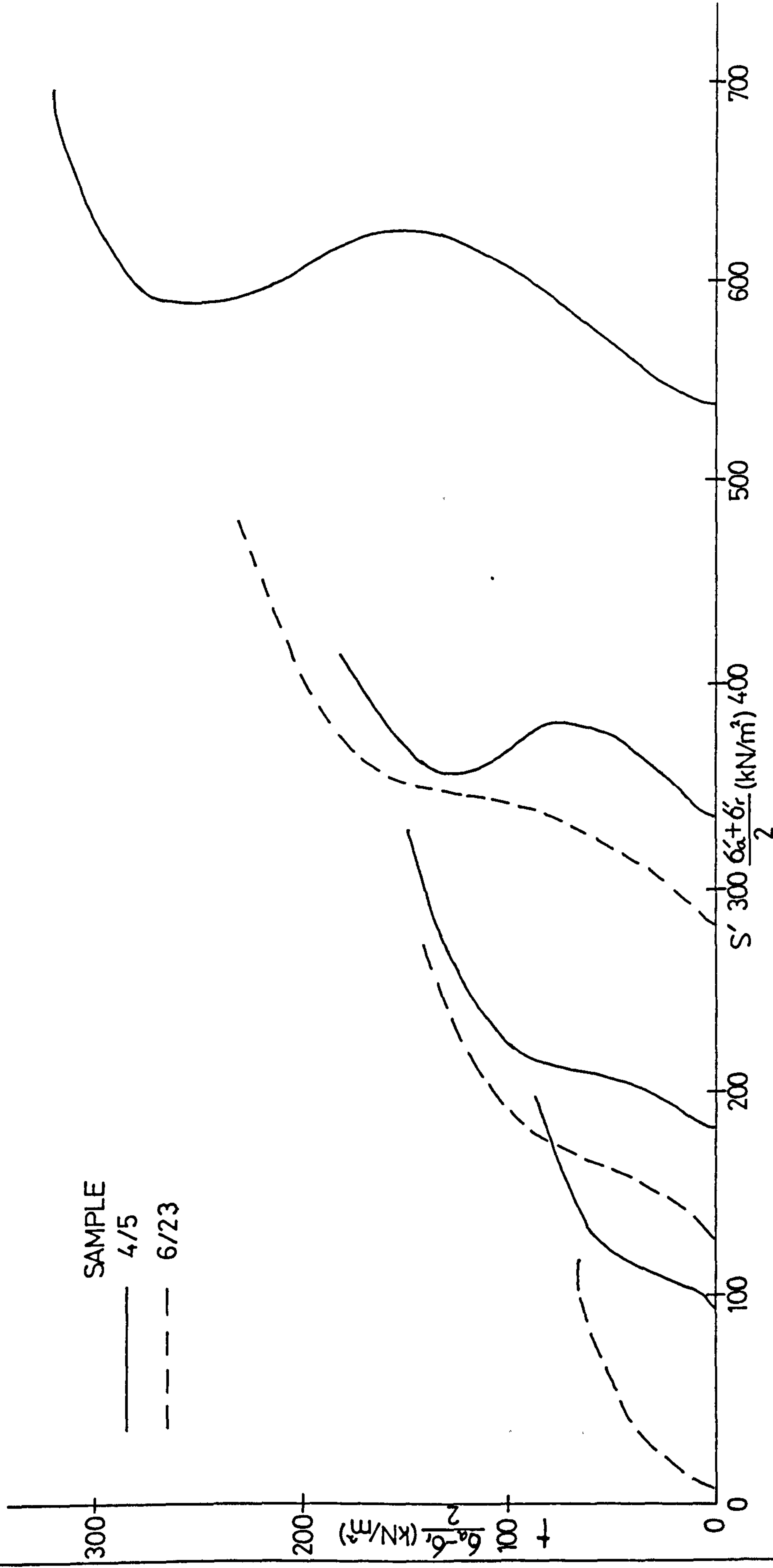


FIGURE 4.52
EFFECTIVE STRESS PATHS FOR 'STANDARD'
TESTS ON WITHERNSEA TILL

2.13). The effective stress strength parameters derived from the stress paths are given in Figure A4,50, Appendix J. They were thought to be more reliable than the Mohr circle results, and mean values for each till type are given in Figure 4.53. The mean value of the pore pressure coefficient A at failure was 0.04, range -0.3 to 0.3, typical of a lightly overconsolidated clay. The effective angle of shearing resistance is in close agreement with that obtained by Marsland and Powell (1985) for unweathered till at Cowden (see Section 2.5.4). However the effective cohesion is greater than zero, contrary to their results.

Till	c' (kN/m ²)	ϕ' (°)
Skipsea	7	25.7
Withernsea	17	26.6
Weathered	13	24.8

Figure 4.53 Mean effective stress strength parameters from stress paths.

4.6.7 Triaxial Test Results: "Reduction" Tests

Curves of deviator stress and pore water pressure for the set of four isotropically consolidated "reduction" tests on a sample of Skipsea till (1/6) are shown in Figure 4.54. The actual increase in deviator stress was stepped, due to the successive decrements in axial stress, and an average curve was drawn on the figure. Pore water pressure decreased throughout

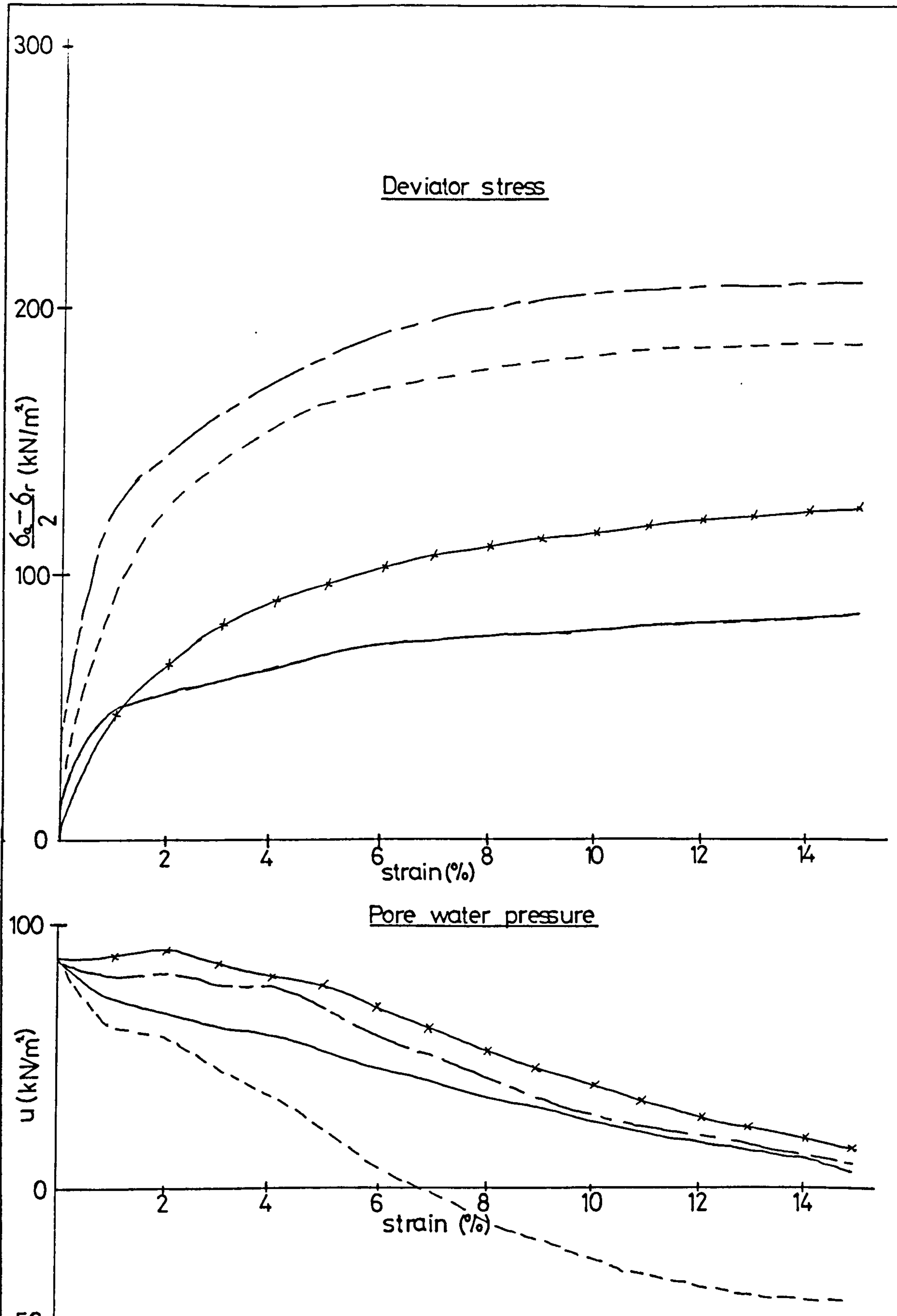


FIGURE 4.54
 DEVIATOR STRESS AND PORE WATER PRESSURE PLOTTED
 WITH STRAIN FOR SAMPLE 1/6 (REDUCTION TESTS)

the test. Mohr circles are drawn for peak deviator stress in Figure 4.55, and stress paths in Figure 4.56.

The value of s decreased as t increased, (shown by the total stress paths). This is opposite to the "standard" tests where s increased as t increased. The value of s' also decreased as t increased, until the failure line was reached. The effective stress paths then curved sharply, and s' increased as t further increased. The value of strain at this change was between 3 and 4%. The test results are given in Figure A4.57, Appendix J. They are compared, along with similar "reduction" tests carried out in an earlier study on Withernsea till (Yeo, 1986), with results from "standard" tests on samples at a similar depth in Figure 4.58. The isotropically consolidated "reduction" tests for Skipsea till (1/6) show a significant decrease in ϕ' , by a factor of two-thirds, and a small increase in c' , compared with "standard" tests. Yeo (1986) found a similar decrease in ϕ' in isotropically consolidated "reduction" tests on Withernsea till, but a larger increase in c' .

FIGURE 4.55
 MOHR CIRCLES SAMPLE 116
 'REDUCTION' TESTS

(isotropically consolidated)

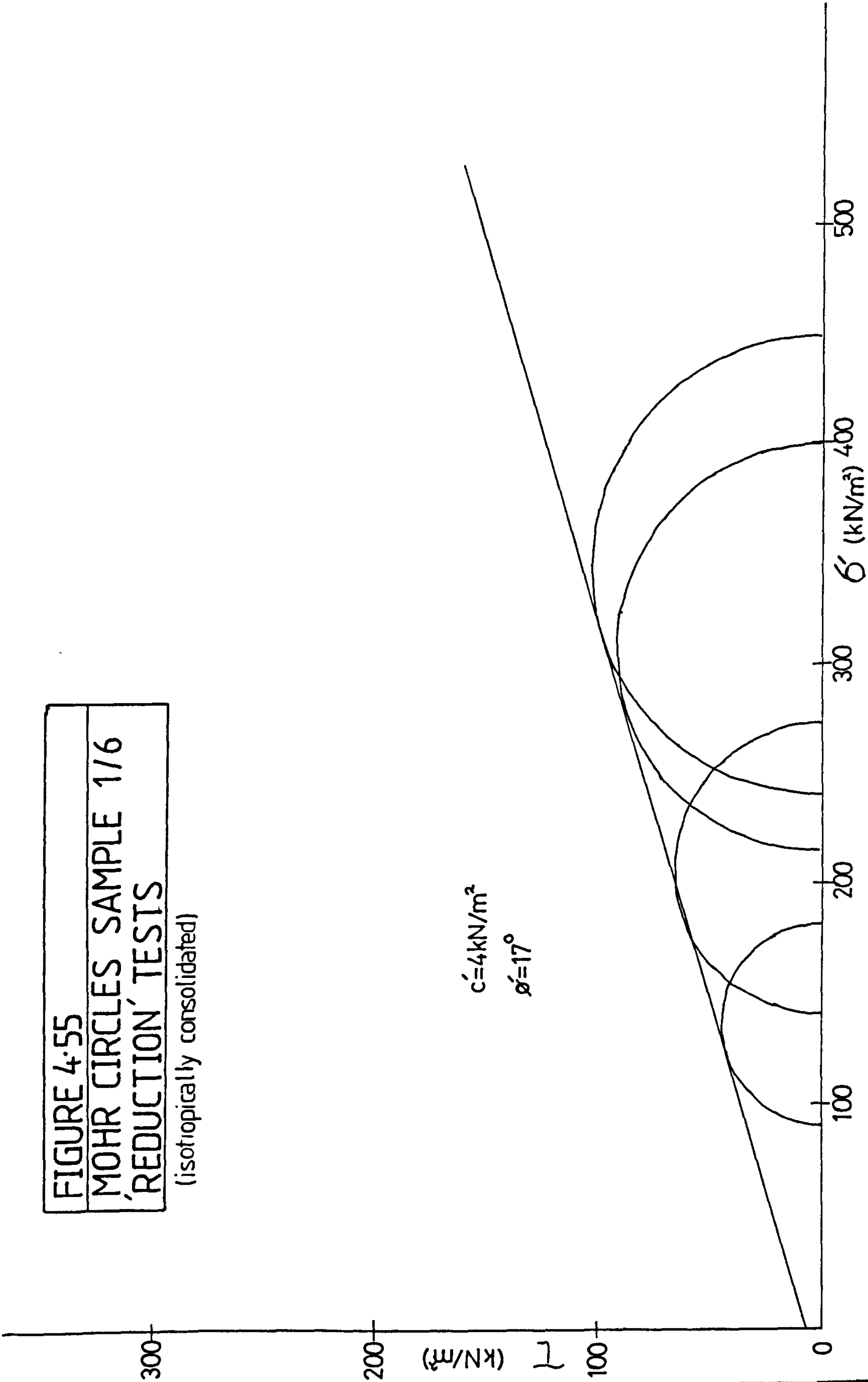


FIGURE 4-59
STRESS PATHS FOR ANISOTROPICALLY CONSOLIDATED
'REDUCTION' TESTS SAMPLE 3/24

--- Total stress path
— Effective stress path

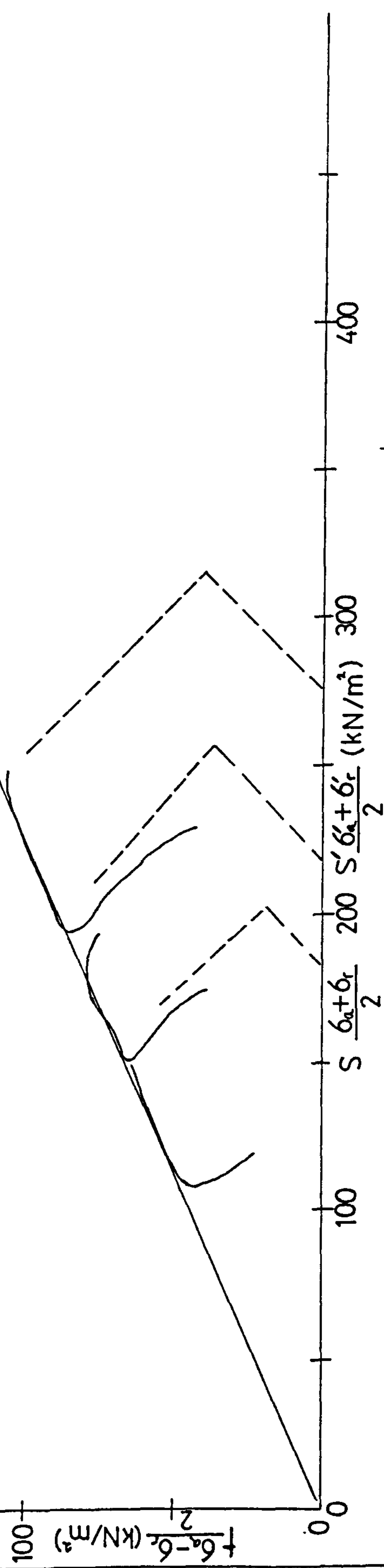
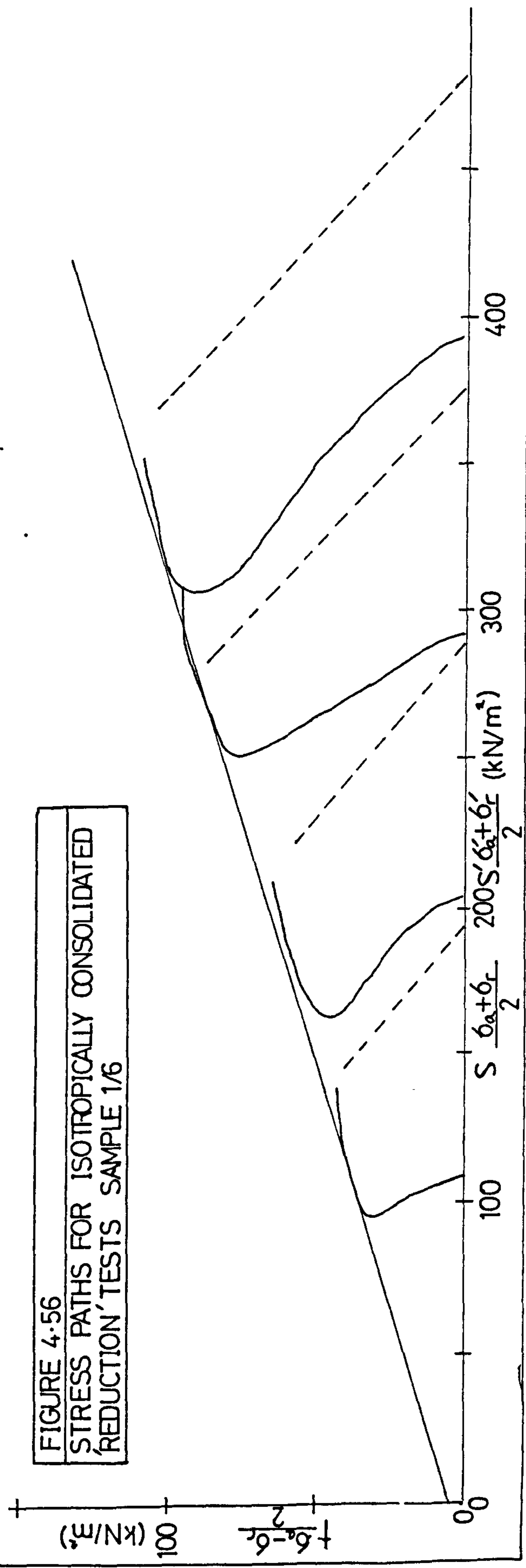


FIGURE 4-56
STRESS PATHS FOR ISOTROPICALLY CONSOLIDATED
'REDUCTION' TESTS SAMPLE 1/6



Triaxial Test Type	Borehole	Sample	Depth (m)	Till	c'kN/m ²	ø'°
"Reduction" (Ko=1)	1	6	11.3	Skipsea	4	17
"Standard"	3	22	12.3	Skipsea	0	26
"Reduction" *	5	5	10.3	Withernsea	36	18
"Standard" *	4	5	10.3	Withernsea	5	27
"Reduction" (Ko=0.7)	3	24	13.5	Skipsea	0	26
* After Yeo (1986).						

Figure 4.58 Summary of "reduction" and "standard" test results.

One more set of "reduction" tests was done during this study on a sample of Skipsea till, after consolidating anisotropically to a Ko ratio of 0.7. Stress paths are shown in Figure 4.59. The value of s increased with t during application of the anisotropic stresses at the start of consolidation, then decreased during shear, as in the other "reduction" tests. The shape of the effective stress paths is similar to the other "reduction" tests. The results are given in Figure A4.57, Appendix J, and are summarized in Figure 4.58. They show no change in c' or ø' compared with the "standard" tests, unlike the other "reduction" tests.

4.6.8 Discussion of Triaxial Test Results

The value of ϕ' determined from the isotropically consolidated "reduction" tests on sample 1/6 appears to be lower than that from the "standard" tests. However this result is not generally in agreement with the limited amount of earlier work reported in the literature. Skempton and Brown (1961) carried out some similar isotropically consolidated drained triaxial tests in which the radial stress was decreased on samples of remoulded till from Selset. There did not appear to be any significant difference in the values of c' and ϕ' derived from these tests when compared with standard tests with increasing axial stress.

Atkinson and Clinton (1986) report the results of tests on remoulded Speswhite kaolin clay following a variety of stress paths. These included one where the cell pressure was reduced with the axial stress held constant and the specimen undrained, followed by a swelling stage during which the specimen failed. They report that the effective angle of friction at failure is reasonably consistent for all the tests and that "... it is generally accepted that the effective angle of friction does not vary greatly for effective stress analysis..."

Burland and Fourie (1985) report the results of passive stress relief tests on samples of London clay, in which the mobilized ϕ' (39.5°) was higher than the value of ϕ' obtained from standard drained triaxial compression tests ($\phi'=25^\circ$) during the

early part of the test. The mobilized strength decreased to $c' = 20.7 \text{ kN/m}^2$, $\phi' = 15^\circ$ at large strains and as the effective axial stress reduced to zero.

These differences are due to swelling of the soil as the mean effective stress is reduced and dilatancy as shear strains develop. The swelling and dilatancy dictate the location of the passive stress relief path which cannot be assumed to be unique (Burland and Fourie, 1985). This therefore appears to conflict with the findings of Atkinson and Clinton (1986).

The value of ϕ' determined from the anisotropically consolidated "reduction" tests on sample 3/24 was the same as that obtained in "standard" tests. This is therefore in agreement with the earlier work of Skempton and Brown (1961), although different from the isotropically consolidated reduction tests on sample 1/6. Thus unless the difference can be explained by the different consolidation of the specimens prior to shearing it must be due to experimental error in the tests on sample 1/6.

The most likely source of experimental error is that too high a strain rate was used, leading to non-equalization of pore water pressure between the ends and middle of the specimen. As the "reduction" tests were stress controlled by hand, they are quite prone to this source of error, particularly during the later part of the test as the specimen nears failure and the strain rate tends to increase. Non-equalization of pore water

pressure could lead to an apparent reduction in ϕ' and increase in c' .

If experimental error in the series of tests on sample 1/6 is indeed the cause of the apparent reduction in ϕ' , then it is likely that the tests on sample 3/24 are correct and there is no change in ϕ' during active stress relief conditions. It was assumed that this was the case for use in stability analysis, although a few analyses were run using the lower values of ϕ' indicated by sample 1/6 in case these were genuine.

4.6.9 Summary

This chapter has presented the results of field and laboratory investigations at the sites of Rolston and Grimston. The data collected forms the basis for the analysis and discussion of the next chapter. The site geometry, ground conditions and failure mechanisms have been established, and field measurements of the pore water pressure distribution and ground movements taken. The consolidation and strength characteristics of the tills have been evaluated for analysis of the drainage conditions and cliff stability.

No measurements of residual strength of the tills were made during this study owing to time and resource constraints. However residual strength parameters for use in analysis are estimated from the literature in the next chapter.

CHAPTER 5

ANALYSIS AND DISCUSSION

In this chapter the field and laboratory data collected in the site investigation, together with information from the general survey, are used in analyses of seepage, consolidation and stability at the sites. The aim of the analyses is to answer some of the questions posed at the end of Chapter Two. The pore water pressure distribution, changes in stress and ground movements occurring as the cliff recedes are discussed. Finally the results of stability analyses of several profiles are presented, and based on these, a cycle of cliff recession is outlined.

5.1 Pore Water Pressure Distribution

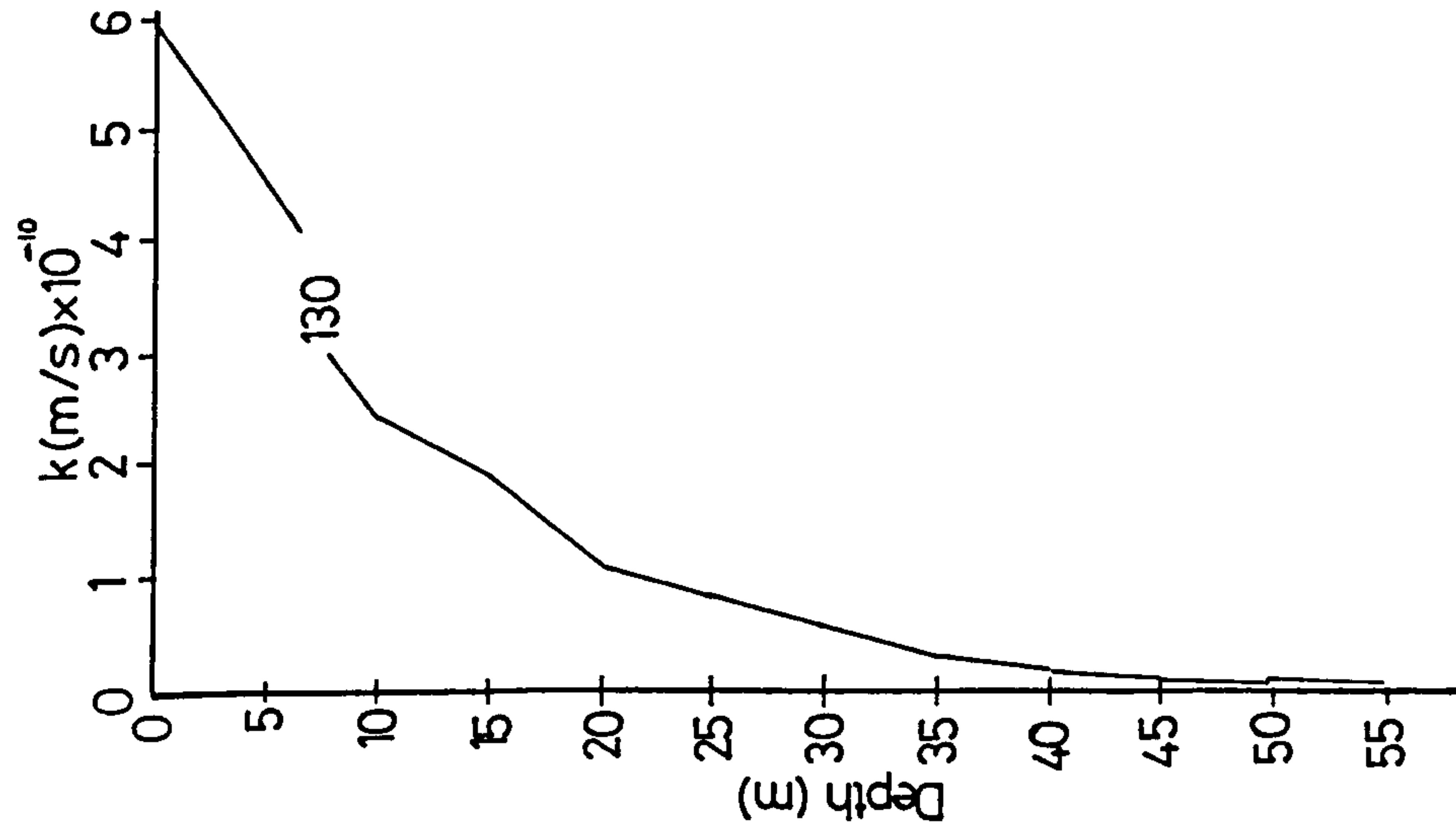
The pore water pressure distribution in the cliffs may have two components:

1. a steady seepage component;
2. an excess component caused by changes in stress as the cliff recedes.

The distribution associated with steady seepage will be governed by the slope geometry, distribution of permeability and the hydraulic boundary conditions. The excess component is likely to be negative (ie. depressed) because the cliff is unloaded as it recedes. As described in Section 2.4.5, depressed pore water pressures have a strong influence on some other coastal cliffs. To assess whether they occur at the two sites, it was first attempted to model the observed pore water pressure distribution by steady seepage flow conditions. Secondly, an estimate of the time required for dissipation of depressed pore water pressures was made.

5.1.1 Modelling by Steady Seepage Flow

The pore water pressure distribution was modelled using flownets sketched by hand. Whilst a seepage problem of this complexity can only be adequately solved for predictive purposes by using numerical methods, the sketched flownets do give a useful impression of the flow pattern which can be expected at the sites. The total heads observed in the piezometers were used to guide the position of the equipotential lines. The slopes were divided into zones of constant permeability (see Figures 5.1 and 5.2), based on the estimated variation of permeability with effective stress (hence depth and pore water pressure) and soil type (see Section 4.4.2). The permeability of the till decreases with depth, and it was assumed to be isotropic because of the



coefficient of permeability (k) m/s x 10⁻¹⁰

24mQD.

5 Weathered till
130
2 Clayey sand
Withemsea till
1 Skipsea till
0.5 Skipsea till

cliff

-33mQD.

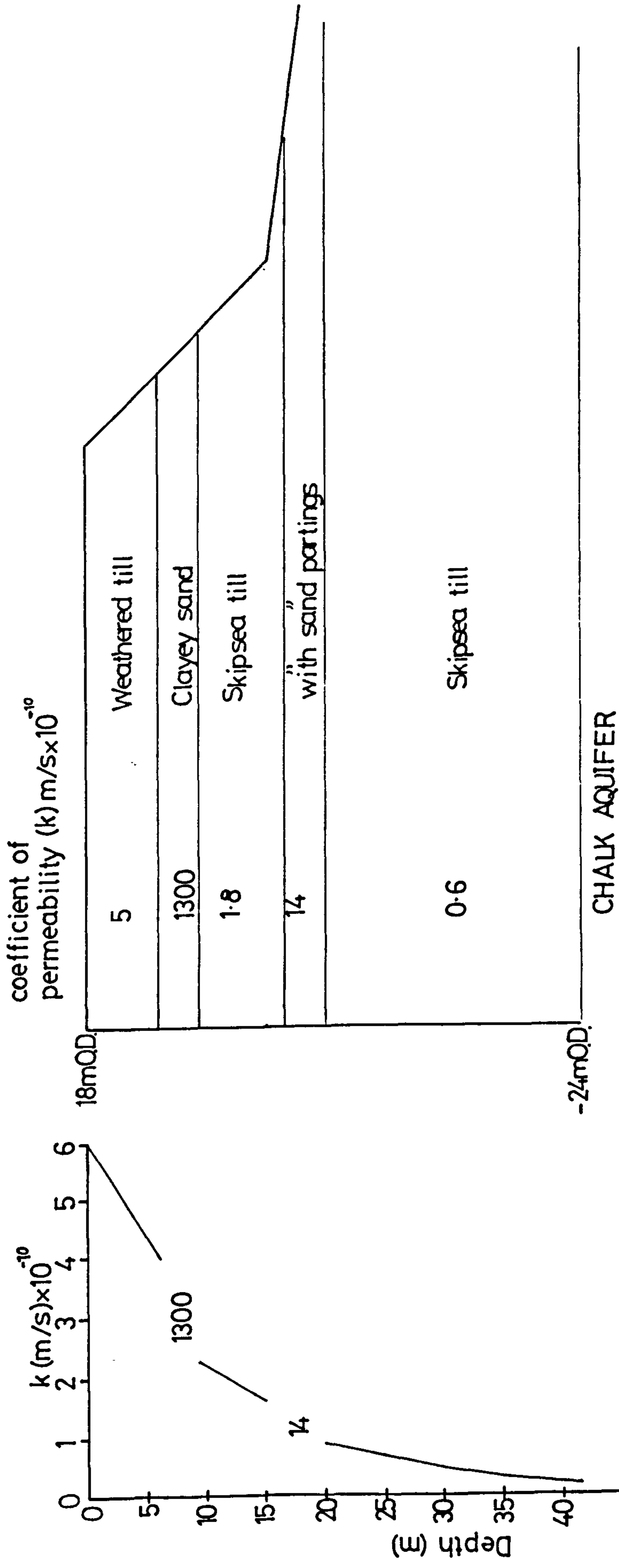
CHALK AQUIFER

Estimated variation with depth Zones of permeability used in analysis

FIGURE 5.1

DISTRIBUTION OF PERMEABILITY AT GRIMSTON

SCALE 1:500



Estimated variation with depth Zones of permeability used in analysis

FIGURE 5.2
DISTRIBUTION OF PERMEABILITY AT ROLSTON
SCALE 1:500

homogenous nature of the till. There are layers of clayey sand and till with sand partings (of higher permeability than the till) at both sites. A layered fabric was occasionally observed (see Section 4.2.4) which could cause anisotropy, but was limited to a few metres in thickness, and was disregarded. The slopes were approximated to a gradient of 1:1, and infiltration was assumed to occur at the ground surface. Groundwater levels in the Chalk were taken as 1m and 2m O.D. at Grimston and Rolston respectively (see Section 4.3.2).

The flownets sketched for Grimston and Rolston are shown in Figures 5.3 and 5.4. The flow lines show underdrainage towards the Chalk and, particularly at Rolston, to sand layers and the cliff. This could explain the low total heads observed in deep piezometers at the sites (see Section 4.3.1). As mentioned in Section 4.3.2, an upward hydraulic gradient is possible at Rolston if the groundwater level in the Chalk is higher than expected. However drainage to the lower high permeability layer could isolate the slope above from its effects (see Figure 5.4).

A comparison of measured and predicted (from the flow nets) total heads at Rolston and Grimston gives a mean head difference of 0.15m (range 0 to 1.5m) and 0.3m (range 0.1m to 1.2m) respectively. This close agreement is not surprising as the observed heads were used to guide drawing of the flownets. However it is believed that the flownets sketched are fairly realistic. The underdrained flow pattern described confirms

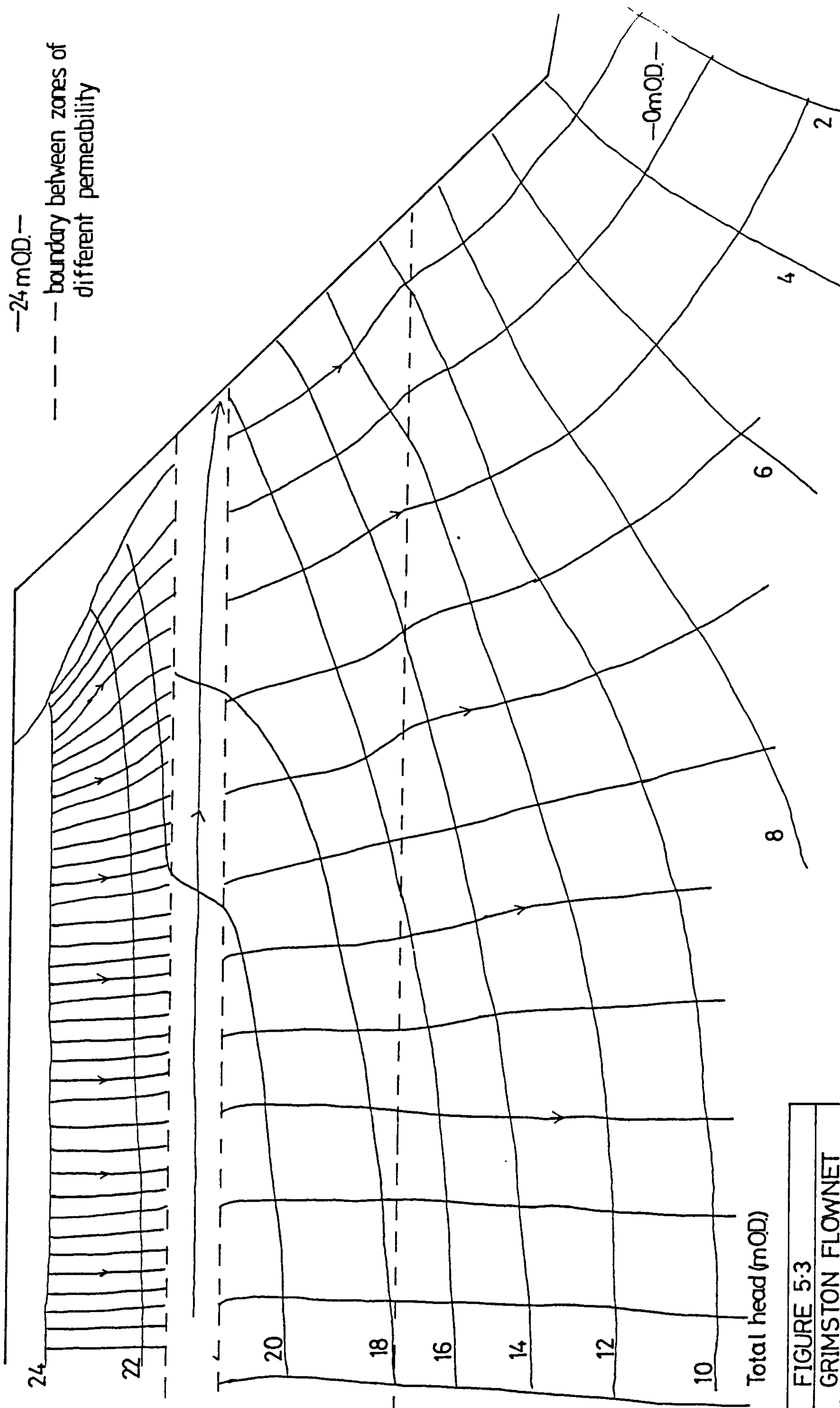


FIGURE 5.3
 GRIMSTON FLOWNET
 SCALE 1:200

Chalk at -33mOD. (Groundwater level 1m O.D.)

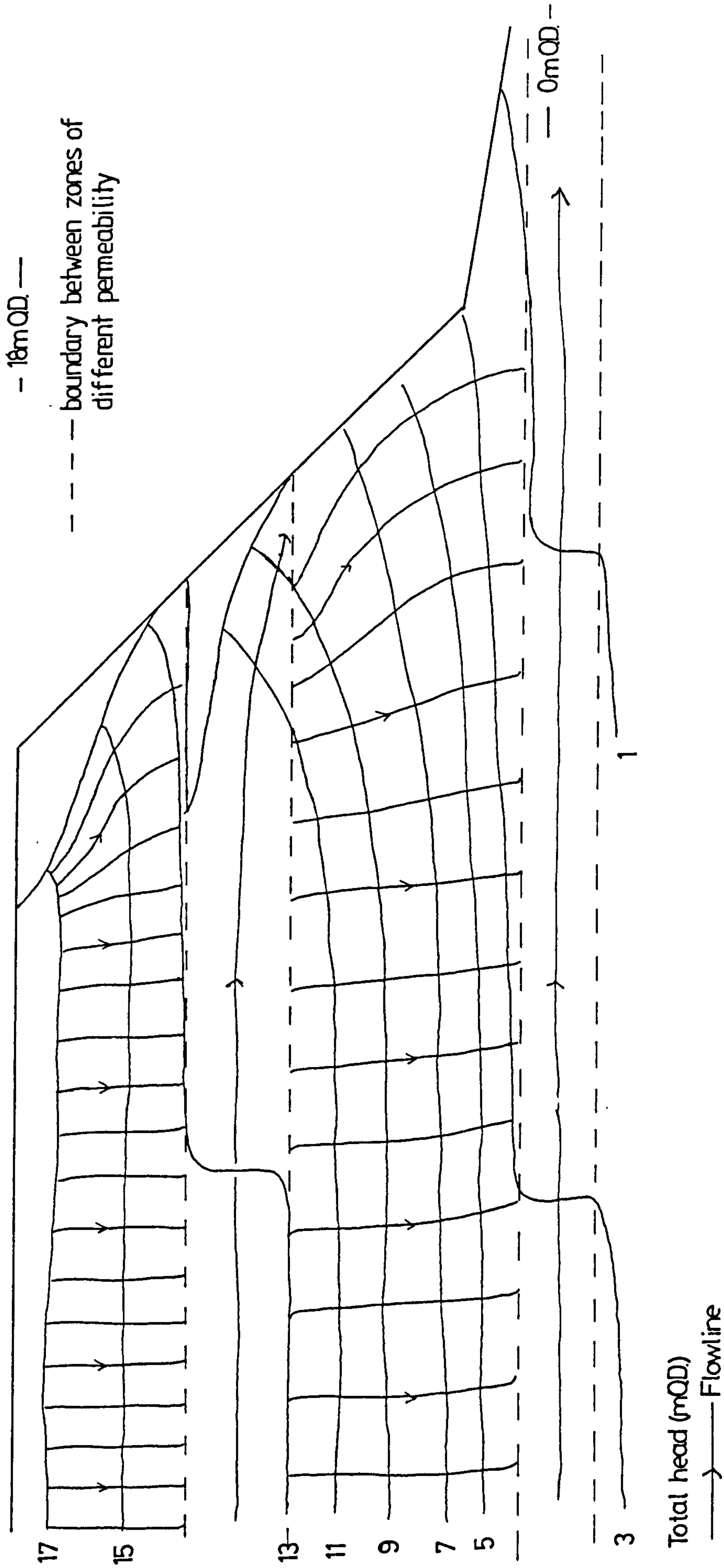


FIGURE 5.4
ROLSTON FLOWNET
SCALE 1:200

Chalk at -24mQD. (Groundwater level 2mQD.)

observations by the Building Research Establishment at Cowden (see Section 2.5.1). The modelling suggests that the pore water pressure distribution observed at Rolston and Grimston arises from an equilibrium seepage flow pattern. However this does not preclude the occurrence of depressed pore water pressure to the cliff, and therefore out of the instrumented region. This possibility is now discussed.

5.1.2 Excess Pore Water Pressure Induced by Stress Relief

All the changes in pore water pressure observed in the piezometers (see Section 4.3.1) are thought to be caused by seasonal variations, rather than stress relief or swelling associated with unloading of the cliff. This confirms the findings of the previous section. However the pore water pressure distribution in some other rapidly eroding cliffs in cohesive soil shows significant depression, owing to stress relief induced suction (see Section 2.4.2). Possible explanations that these depressions, and changes caused by unloading, were not observed at the sites are:

1. the piezometers are too far from the cliff, therefore changes are small and masked by seasonal variations;
2. no significant unloading occurred during the study;
3. the rate of equilibration is too slow to be detected.

It is believed that a combination of (1) and (2) is the correct explanation for the reasons discussed below.

From inspection of the distribution of pore water pressure changes around excavations in several case studies by Eigenbrod (1975), it is likely that the piezometers nearest the cliff (R4, R9, R10, R12 and G3, G4, G8, G11, G13) would show changes in head following major unloading of the cliff. Unloading occurred during the study at Rolston, by several falls and slips, but not at Grimston (see Sections 4.3.9 and 4.3.10). However the unloading at Rolston was small compared with the overall unloading which would occur as the slope recedes, and would not induce large pore water pressure suctions. For these to occur decrements of pore water pressure would have to persist for a long period. Successive decrements would then be cumulative. This possibility is now examined by estimating the time required for dissipation of depressed pore water pressure.

5.1.3 Time For Dissipation of Excess Pore Water Pressure

The field coefficient of swelling (c_s) was estimated from laboratory values of m_v (see Figure 4.46) and in situ values of k (see Figure 4.39), using the appropriate value of mean in situ effective stress. To correspond with the field situation of unloading, m_v from the swelling stages of the tests was used. However in the field unloading occurs in both lateral (horizontal) and vertical directions, while in the laboratory

it is only vertical. The difference was ignored in this analysis, but its effects could be assessed in future work by testing specimens cut horizontally from the samples. The distribution of c_v at the sites is shown in Figure 5.5 and 5.6. Drainage in the slope was thought to be predominantly vertical, to the more permeable soil layers (assumed to be free draining). Drainage path lengths were estimated from the spacing of these layers and the geometry of the cliff. The initial excess pore water pressure was assumed to be constant with depth, simulating the effects of lateral stress relief. The times for 50% and 90% dissipation of excess pore water pressure were estimated using Terzaghi's theory of one-dimensional consolidation. This was justified on the assumption of vertical drainage, but may not be entirely valid as drainage could also occur horizontally to the cliff face. The results are given in Figure 5.7.

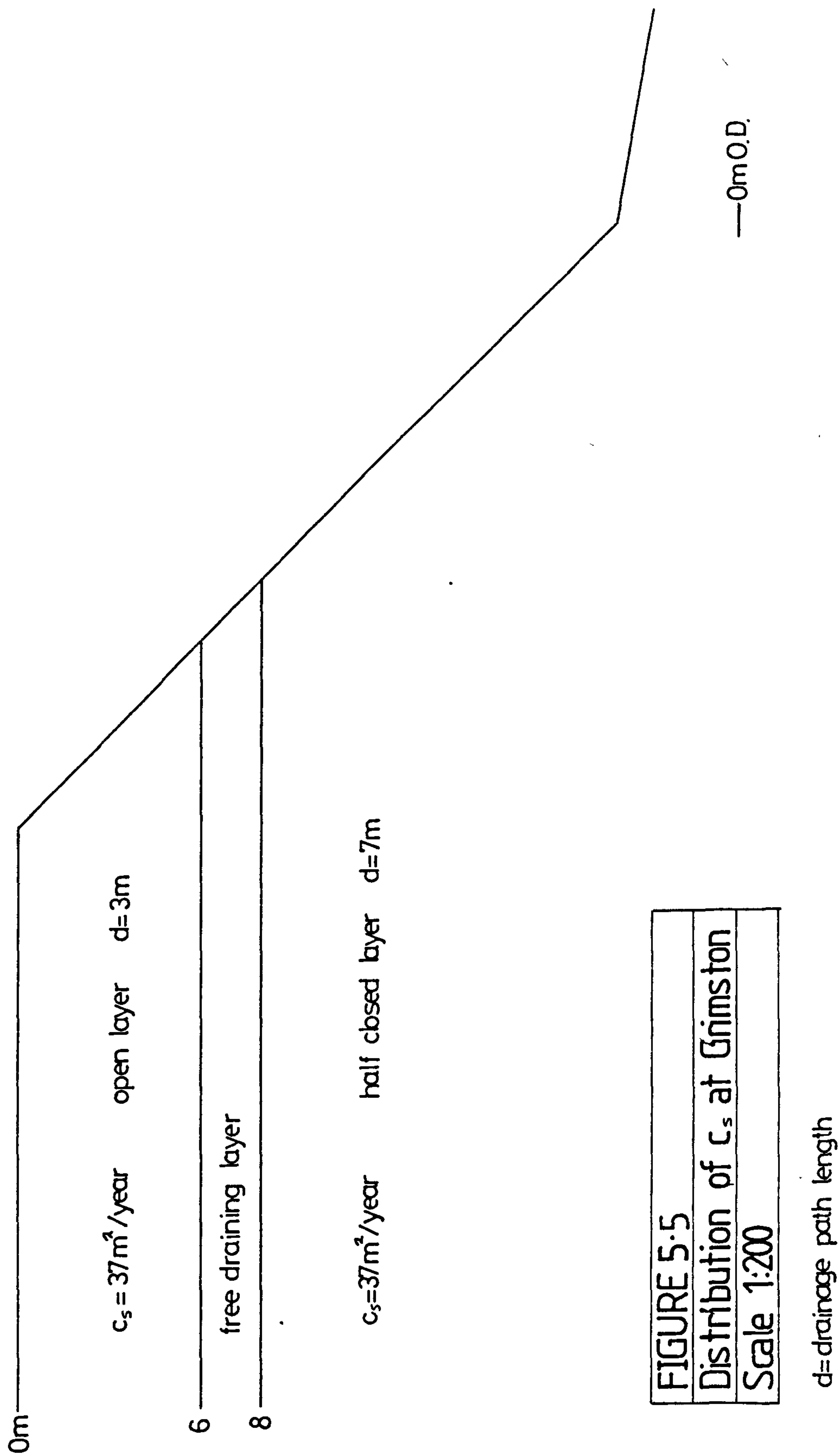


FIGURE 5.5
Distribution of c_s at Grimston
Scale 1:200

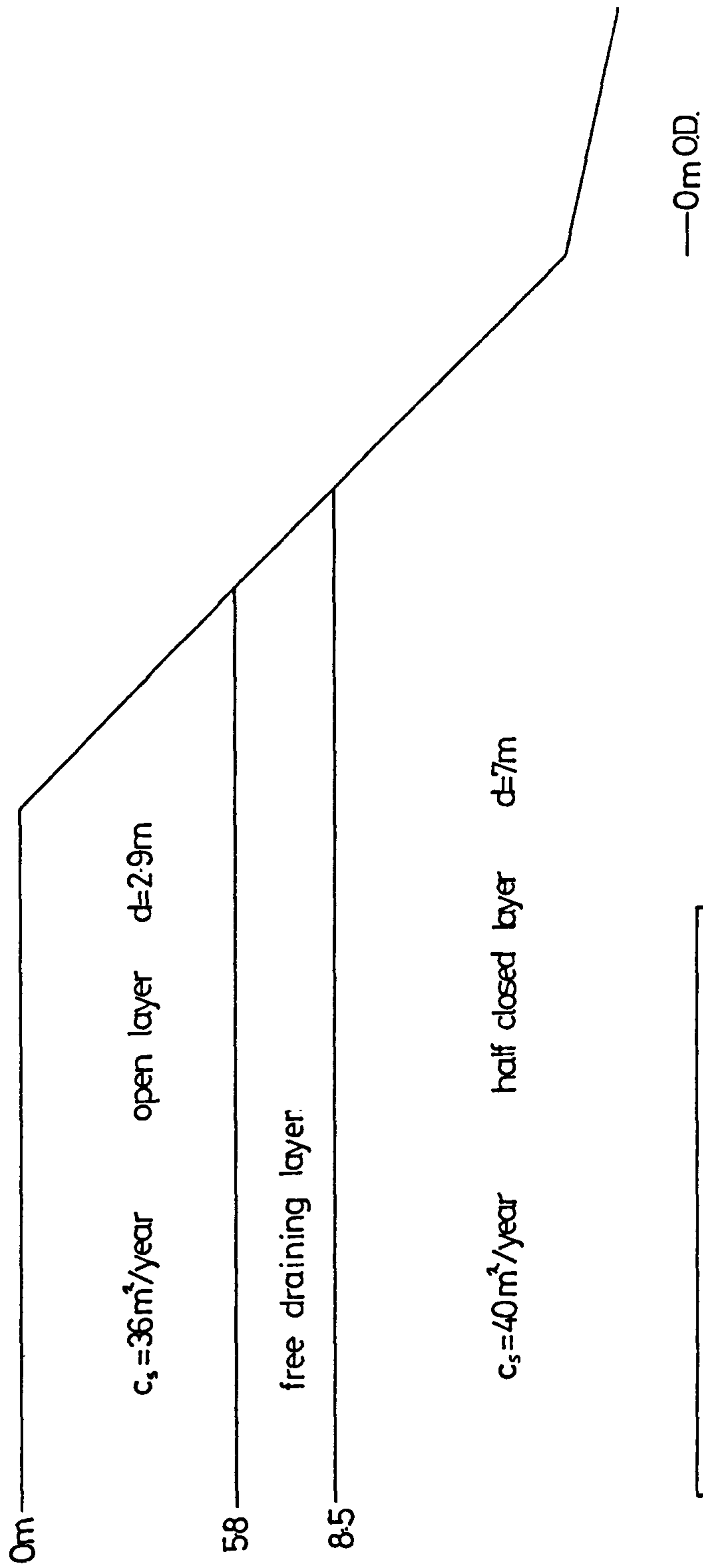


FIGURE 5.6
Distribution of c_s at Rolston
Scale 1:200

d =drainage path length

Site	Dissipation of Excess Pore Water Pressure (%)	Time (Months)
Grimston	50	4.7
	90	7.5
Rolston	50	1.0
	90	2.6
Foreshore	90	2.3
Assuming no permeable sand layers exist:		
Grimston	90	11.0
Rolston	90	6.6

Figure 5.7 Estimated times for dissipation of excess pore water pressure.

The results show that the times for dissipation are relatively short, when compared with slopes in other cohesive soils, eg. the London clay (see Section 2.4.5), even if it is assumed there are no permeable layers within the tills. This is probably due to the greater permeability of the till, and the shorter drainage path lengths in the Holderness cliffs. The times at Rolston are shorter than at Grimston for the same reasons.

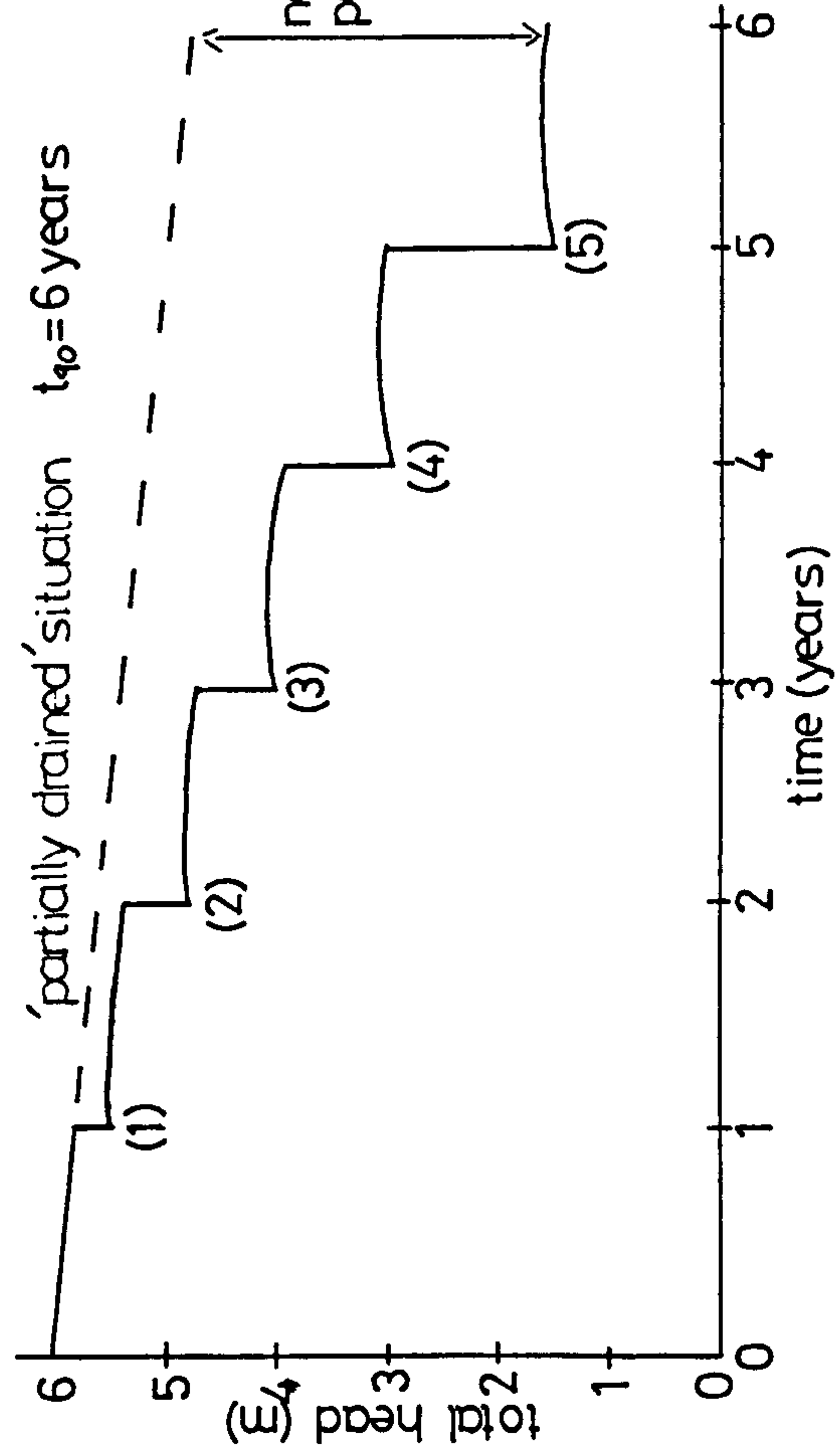
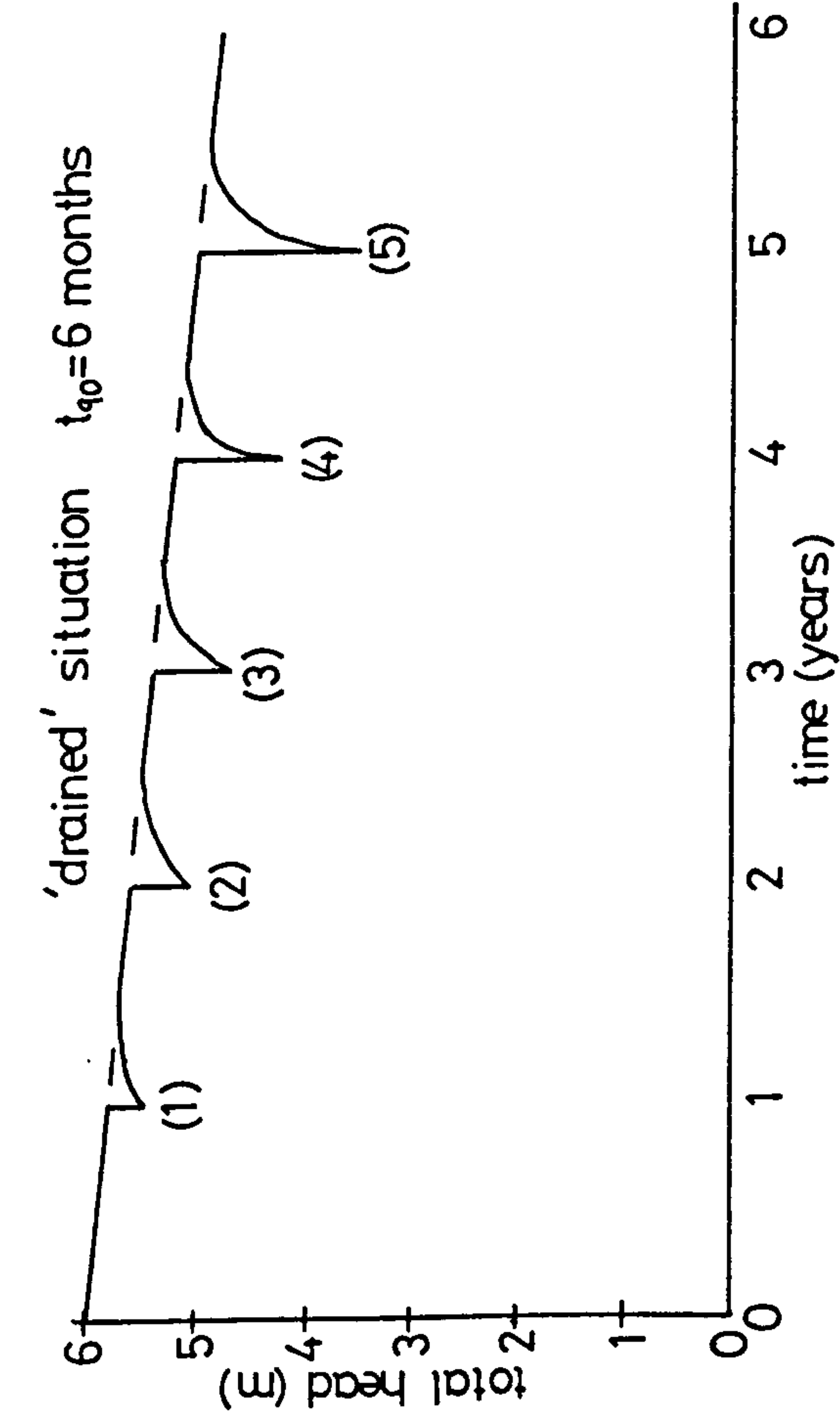
The coefficient of swelling may be increased (hence time for dissipation reduced), in the weathered till by the opening of fissures near the cliff. However some evidence that the analysis may under-estimate the time for swelling is given by a push-in piezometer in the Rolston foreshore (below high water mark) at 3m depth. It remained dry during the study, possibly

indicating the presence of suctions in an area finally unloaded 20 years ago, while the analysis indicated a t_{90} of only 2.3 months. This result must be treated with caution, as it is based on only one observation. Unfortunately no successfully measurements of the pore water pressure in the foreshore have been made by the Building Research Establishment at Cowden.

The falling head permeability tests may over-estimate the in situ permeability of the tills by a factor of about 7, indicated by the comparison with the results of constant head tests by the Building Research Establishment at Cowden (see Section 4.4.2). If this is the case this would increase the times required for dissipation of excess pore water pressure in Figure 5.7.

5.1.4 Possible Occurrence of Depressed Pore Water Pressure

To determine if a large depressed pore water pressure could develop in the slope, the time for dissipation of excess pore water pressure (dependent on the rate of swelling) was considered in relation to the time between unloading events (dependent on the rate of recession). Two hypothetical situations which could occur are illustrated in Figure 5.8. This shows the changes in total head with time at a point behind a receding cliff. The removal of wedges of soil from the cliff face induces decrements of excess pore water pressure at the point, which immediately begin to dissipate. In the



Changes in total head caused by decrements (1) to (5) of pore water pressure at point P

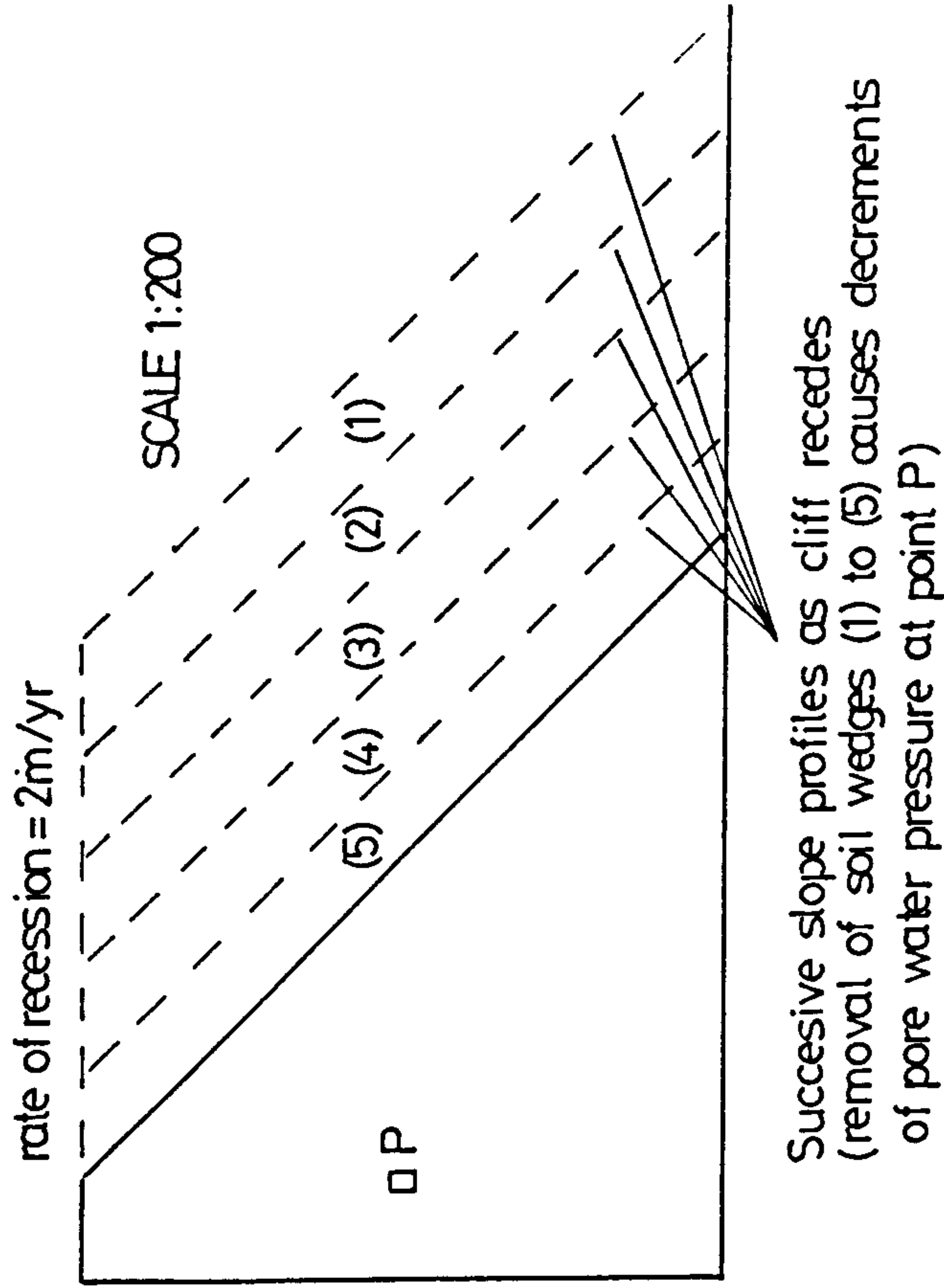


Figure 5.8

Changes in total head at a point behind a receding cliff for 'drained' and 'partially drained' situations

upper diagram, with a high rate of swelling ($t_{90} = 6$ months), all excess pore water pressure dissipates before a new decrement occurs, and the situation is drained. In the lower diagram ($t_{90} = 6$ years) only part of the excess pore water pressure dissipates, and a new decrement will add to the remainder. The situation will be partially drained. At the two sites the actual situation is thought to be predominantly drained, considering the times for swelling of a few months, compared with the time between major unloading events (slips and falls) of 2-3 years. However at Rolston, where very rapid recession sometimes occurs, it is possible the situation is partially drained. It is also possible that at both sites there is a small zone of depressed pore water pressure directly below the cliff face, due to the near continuous unloading by mudslides, small slips and the change in geometry as the slips move downslope.

In conclusion, the observed pore water pressure distribution at Rolston and Grimston is thought to arise from steady seepage flow in equilibrium with the boundary conditions. Drainage to more permeable layers and the Chalk is a dominant influence on the flow pattern. Depressed pore water pressure induced by stress relief near to the cliff is expected to dissipate quickly. Only suction resulting from very recent unloading (within a few months) will be present in the slope, and consequently its magnitude will be small. However it could make some contribution to stability. It would be desirable to confirm these conclusions by measuring pore water pressure

closer to the cliff and below the cliff face, and observing the formation and subsequent dissipation of depressed pore water pressure directly (see Section 6.3.1).

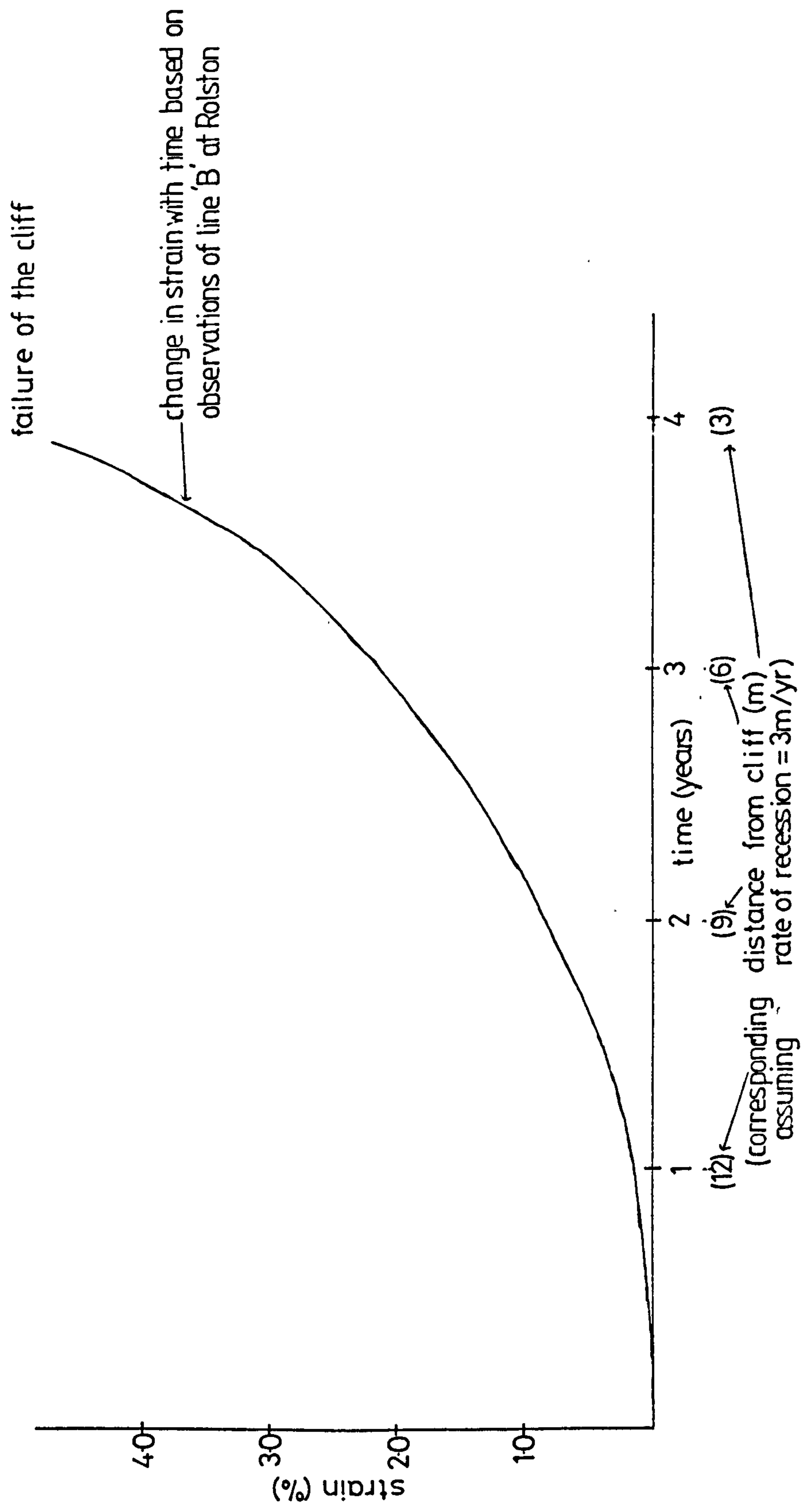
5.2 Ground Movement and Stress Relief

The results of Section 4.3.7 show that the majority of ground movement occurs in response to unloading of the cliff by slips and falls. In areas with a high rate of unloading (eg. Rolston) movement occurs almost continuously, while in low or moderate rate areas (eg. Grimston) it occurs intermittently. The changes in horizontal strain with time in a region behind the cliff as it recedes, (based on the results of line "B" at Rolston, see Section 4.3.7), are shown in Figure 5.9. Although the strain is measured on the ground surface, the value of 3-4% strain when cliff failure occurs is consistent with the results of triaxial tests, which show that failure occurs at these or slightly higher axial strains (see Section 4.6.7).

Studies of ground movement around man-made excavations in London clay (Chard and Symons, 1982) and Oxford clay (Burland et al., 1977), have shown that movements occur at a distance behind the slope up to 2.5 times the depth of excavation. At the sites the corresponding distance would be about 40m, however movements were not observed more than 10m behind the cliff. A possible reason for this difference is the smaller increments of unloading which occur by gradual erosion compared

FIGURE 5.9

Horizontal strain plotted with time in a region behind the cliff as it recedes



with complete excavation. Thus in the relatively short period of this study (18 months), only part of the overall deformation was observed. Also, the above mentioned excavations had steeper or vertical sides and hence movement would be expected to be greater, and smaller movements would be expected in the less over-consolidated Holderness tills (hence more difficult to measure). In both the aforementioned studies sub-surface measurements showed significant lateral movements at depth. No sub-surface observations were made at the sites in this study, but movements would be expected to occur.

The ground movements due to stress relief can be compared with the in situ stress and resistivity measurements. The in situ stresses at Cowden cliff (see Section 4.3.8) are reduced from the inland values up to 15m from the cliff edge. The region of higher resistivity at Rolston, Grimston and Monkwith (see Section 4.5.2) occurs up to 12m from the cliff edge. There is some correlation between these results and the ground movements, which occur up to 10m from the cliff edge. The stress changes and strain could not be compared directly because no in situ stress measurements closer than 10m to the cliff were available.

The phenomenon of stress relief and associated strains should be considered in analysis of slope stability behind excavations of eroding cliffs. It may influence:

1. the pore water pressure distribution;

2. the magnitude in stability analysis of interslice forces and normal forces on the slip surface;
3. the stress path followed to failure, and hence the appropriate strength parameters.

The effects of (1) have been discussed in Section 5.1.2. It was not possible to study the effect of (2) on the calculated factor of safety with the computer program available, but this could be done in a future study. The selection of appropriate strength parameters is discussed in Section 5.3.3.

5.3 Slope Stability

5.3.1 Introduction

It was impossible to derive from back analysis the mobilized strength parameters, or to confirm the pore water pressure distribution exactly, because of the uncertainty in both of these, and in the slip geometry. However some stability analyses were done to:

- show that the mode of failure, pore water pressure distribution and strength parameters postulated from the site investigation data are broadly correct;

- evaluate the effect on stability of natural changes which occur in the field.

The stability of four slips, two at each site, was analysed, and one of the slips at Grimston was analysed at three different stages in its cycle of failure. Apart from a hypothetical pre-failure profile at Grimston, all the slips analysed were movements on pre-existing shear surfaces. A parametric study was conducted at an early stage in the overall study (Mahjoub, 1987), and the findings are summarized below.

5.3.2 Early Parametric Study of a Pre-Failure Slope Profile

The parametric study by Mahjoub (1987) showed that the factor of safety of a typical pre-failure slope profile for the Grimston site, calculated using the Bishop simplified method, was sensitive to changes in pore pressure ratio, c' and ϕ' , but not to changes in bulk density.

The presence of a beach at the cliff foot and tension cracks in the cliff top were both found to improve stability. The factor of safety obtained was found to be generally less than one, except when a low pore pressure ratio and high strength parameters were used. It was suggested in the study that 3-dimensional effects may be partly responsible, as the central slope profile (which was the one analysed) generally has a lower factor of safety than end profiles. It was also

suggested that the factor of safety may be increased by depressed pore water pressures, induced in the slope by stress relief. The findings of this study were used to guide the stability analyses described in Section 5.4.

5.3.3 Evaluation of Parameters for Use in Analysis

The shear strength parameters, pore water pressures and slip geometry for use in analysis are evaluated in this section. A number of factors should be considered in the measurement and selection of strength parameters including:

- choice of peak or residual strength;
- choice of the test method most relevant to field conditions;
- the stress path followed to failure
- the possibility of progressive failure in a first time slide.

(i) Shear Strength Parameters

Residual strength should be used in the analysis of slips where there is a pre-existing shear surface (see

Section 2.4.2), and peak strength for the analysis of first-time failures. No residual strength measurements on the Holderness tills were made during this study. However there are several measurements reported in the literature which are summarized below in Figure 5.9A.

$c' \text{ (kN/m}^2\text{)}$	$\phi' \text{ (}^\circ\text{)}$	Comments	Source
6	24	P.I.=18%, Cowden	Lupini et al., 1981
0	25	Cowden, shear box	Marsland & Powell, 1985
0	23-24	Cowden, ring shear $\sigma_n > 400 \text{ kpa}$	Marsland & Powell, 1985
0	25-26	Cowden, ring shear $\sigma_n < 400 \text{ kpa}$	Marsland & Powell, 1985
0	25	Holderness, shear box	Gilroy, 1981

Figure 5.9A Residual strength of Holderness tills.

The above measurements agree quite closely, and also agree with the residual ϕ'_r of 24° estimated from the ϕ'_r versus plasticity index plot in Figure 2.6, assuming a typical plasticity index of between 13% and 25% for Holderness tills (see Figure 4.12). It is probable that in this range of plasticity index the till is shearing in turbulent mode, and is therefore relatively insensitive to plasticity index, see Section 2.4.2.

As the normal stress on the slip surface in Holderness slides is likely to be less than 400 kN/m^2 , a residual strength of $c'_r=0 \text{ kN/m}^2$, $\phi'_r=25^\circ$ seems most appropriate. It is likely that the residual strength is largely independent of the stress path followed to failure (Bromhead, 1986).

The effect of assuming a small effective cohesion of $c' = 6 \text{ kN/m}^2$, as found by Lupini et al. (1981) for Cowden till, was examined in some analyses. This could be an 'apparent' cohesion, resulting from curvature of the failure envelope, but the shear strength would nevertheless be under-estimated if it was omitted.

The test method which is most relevant to field conditions depends upon the geometry of the slip surface. The shape of the slip surface at the sites (see Section 4.3.9) is shown in Figure 5.10. It has a steeply inclined back portion above the level of the cliff foot, and a near flat portion at the sole of the slip below the cliff foot. The test in which the application of shear stress most closely resembles the way it is applied in the field is different for these two portions of the slip surface. Conditions in the back portion are most closely resembled by triaxial compression tests, because the steep inclination of the slip surface in the field corresponds with the inclination of the failure surface in the tests.

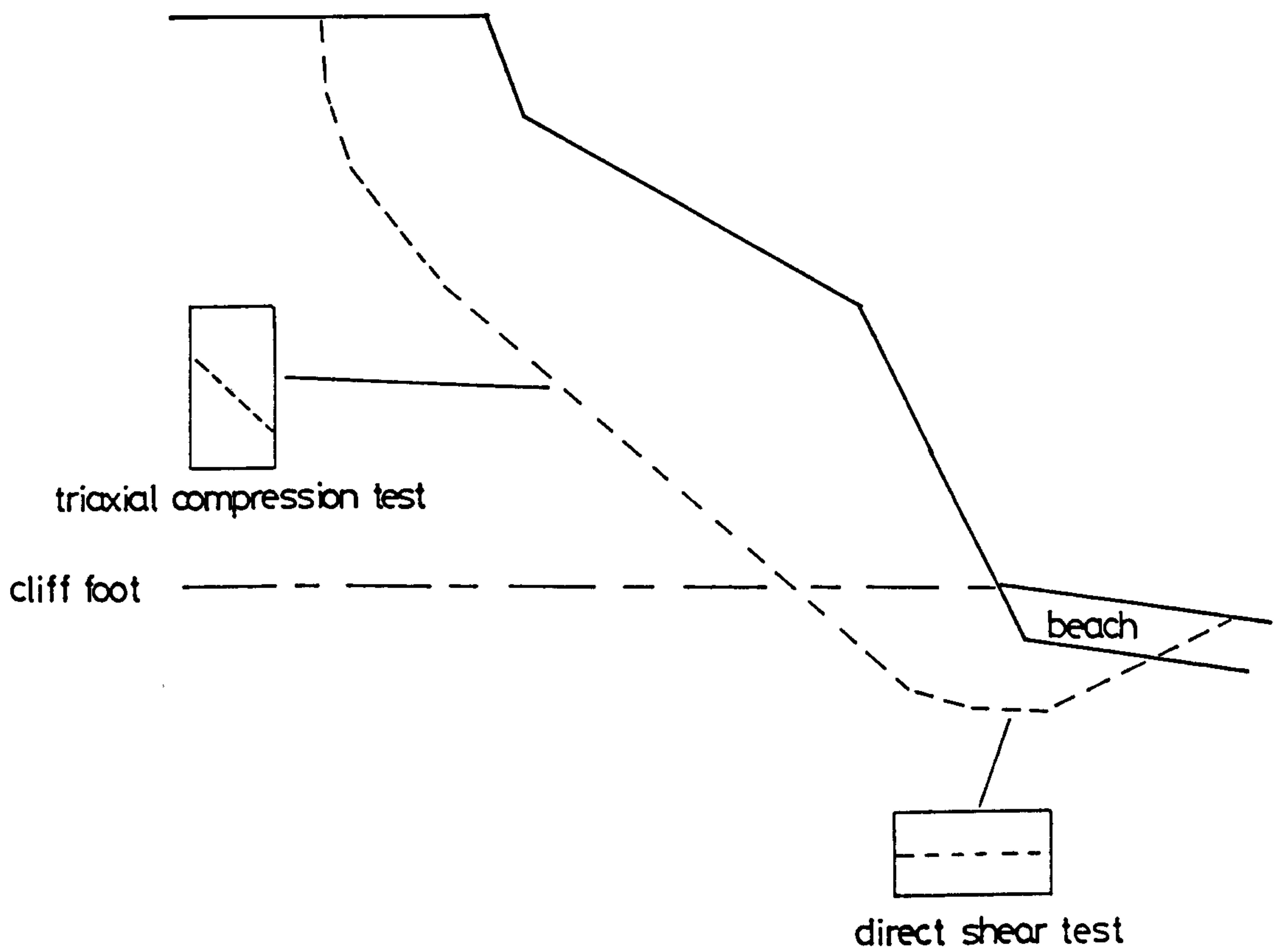


Figure 5.10
Section to show most relevant method of test
along different portions of the slip surface
Scale 1:200

Similarly, along the flat portion of the slip, direct shear (shear box) tests most closely resemble the predominantly horizontal shearing action.

Theoretical work by Wroth (1984) indicates the following relationship between the observed friction angle in triaxial compression (ϕ_{tc}) and plane strain (direct shear, ϕ_{ds}) tests of:

$$\phi_{ds} = 9/8 \cdot \phi_{tc} \quad (3)$$

No direct shear tests were carried out on the Holderness tills during this study. Previous work by Marsland and Powell (1985) on tills from Cowden indicates ϕ_{ds} less than 1° higher than ϕ_{tc} , although Gilroy (1981) found that ϕ_{ds} was about 3° higher, in agreement with Wroth's relationship. Tests on the Blue London clay showed ϕ_{ds} only slightly higher (ie. $<1^\circ$) than ϕ_{tc} (Skempton and Hutchinson, 1969).

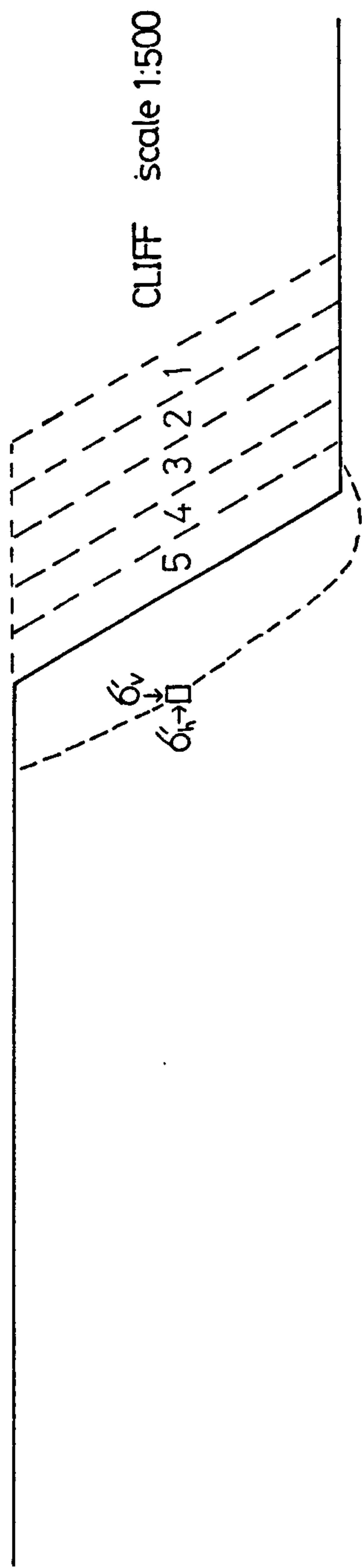
Although the evidence is inconclusive, the effect on the calculated factor of safety of using parameters from direct shear tests (hereafter referred to as "direct shear" parameters) below the cliff foot, was examined in some analyses.

The influence of the stress path followed to failure on the peak strength parameters has been discussed in

Section 4.6. Along the steeply rising back portion of the slip surface, the soil will be subject to lateral stress relief. A hypothetical stress path for an element of soil at mid-height in the cliffs as they recede is shown in Figure 5.11. The path consists of increments of undrained unloading, followed by periods of drainage. The similarity to stress paths followed in the "reduction" tests is clear (see Section 4.6.7), although the laboratory tests were undrained, whereas the field conditions are drained.

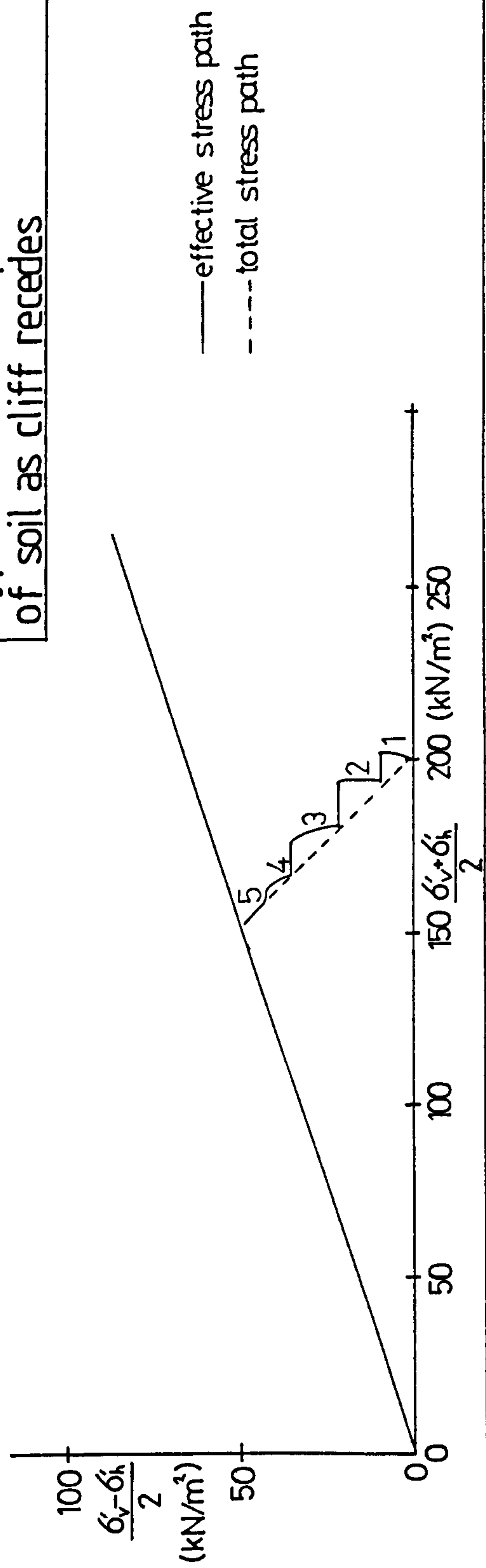
For the analysis of slopes which had not failed it was decided to use peak strength parameters from the "standard" triaxial tests, because the results of the "reduction" tests were ambiguous. However a lower value of ϕ' was used in some analyses to examine the effect on cliff stability if the results of the "reduction" tests were correct.

The peak strength operating in the field may differ from that indicated by laboratory tests, owing to the influence of sampling disturbance and sample size as discussed in Section 2.4.1. However the occurrence of discontinuities in the unweathered tills is infrequent and sampling disturbance is not thought to be large, as the results of "standard" tests agree with those on high quality pushed samples from the Building Research Establishment test site at Cowden.



Changes in cliff profile (1) to (5) cause corresponding increments of undrained unloading in stress paths

FIGURE 5.11
Hypothetical stress path for element of soil as cliff recedes



Although first-time slides in the till are probably progressive in nature (ie. with part of the slip at peak strength, part at residual strength and part between these two), this was ignored in the analyses and peak strength was used throughout. It is considered that errors arising from this would be small, because the difference in peak and residual strength is small for the till.

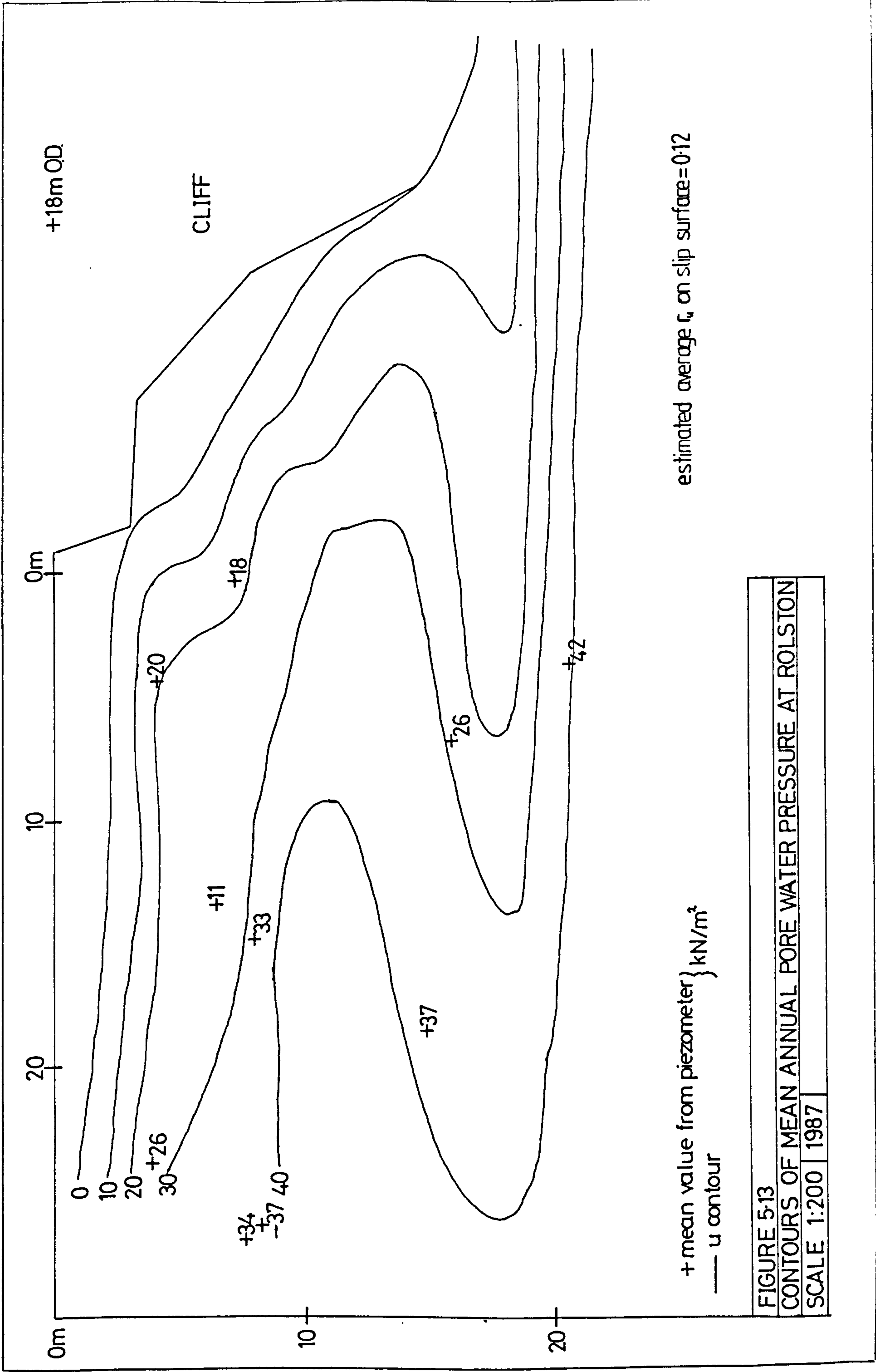
After considering all the above factors, the shear strength parameters which were selected for use in analysis are given in Figure 5.12 overleaf.

Type of Test	Soil Type	Peak Strength Parameters	
		c' (kN/m ²)	ϕ' (°)
"Standard" Triaxial	Skipsea Till	7	26
	Withernsea Till	17	27
	Weathered Till	13	25
"Reduction" Triaxial	Skipsea Till	4	17
	Withernsea Till	10	17
	Weathered Till	0	15
"Direct Shear"	Skipsea Till	7	29
Value Assumed	Beach Sand (loose)	0	28
		Residual Strength Parameters	
		c' (kN/m ²)	ϕ' (°)
Ring Shear	All Tills	0	25 *
	All Tills	6	25 **
<p>* After Marsland and Powell, 1985.</p> <p>** After Lupini et al., 1981.</p> <p>Note: The value of peak c' was set to 0kN/m² in some analyses, see text.</p>			

Figure 5.12 Strength parameters for use in stability analysis.

(ii) Pore Water Pressures

The values of pore water pressure measured by the piezometers were used to estimate the average value of r_u along the slip surface for use in stability analysis. The flow nets sketched for steady state conditions in Figures 5.3 and 5.4 were used to produce contours of the mean annual pore water pressure shown on Figures 5.13



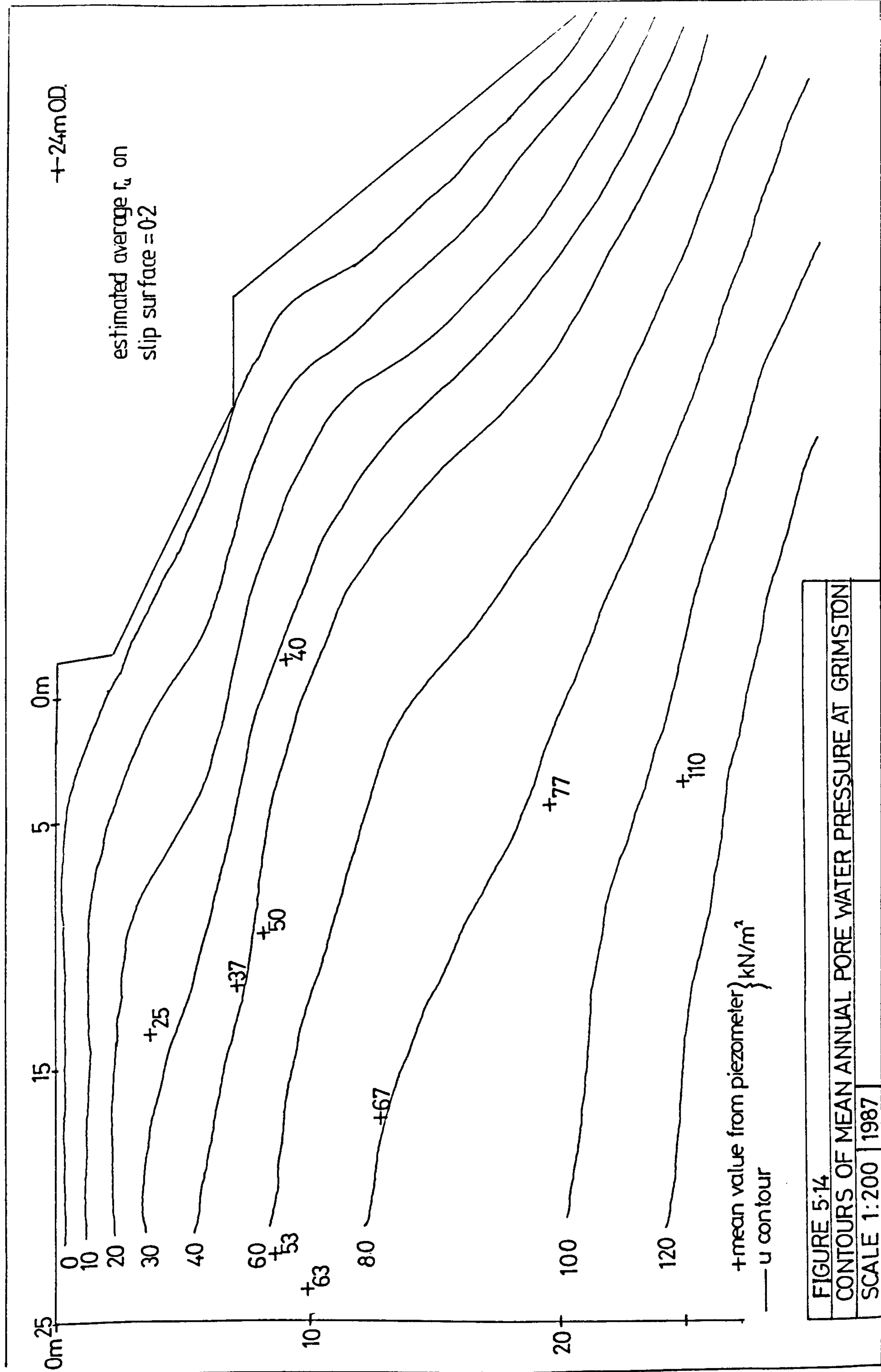


FIGURE 5.14
 CONTOURS OF MEAN ANNUAL PORE WATER PRESSURE AT GRIMSTON
 SCALE 1:200 1987

and 5.14. Mean values of r_u of 0.2 for Grimston and 0.12 for Rolston were then estimated from these. This method of representing pore pressure was chosen because of uncertainty in the estimated pore pressures at the slip surface. The more accurate methods of a piezometric line or grid were therefore not warranted. Also r_u could be varied easily to study the effects of pore pressure changes on stability.

The above values are for long-term steady state conditions. Large depressed pore water pressures were not observed in the piezometers (see Section 5.1). However the effect of assuming a small depression in pore water pressure was examined in some analyses. It is feasible such a depression could be produced along the slip surface by near continuous unloading of the cliff face by mudslides and falls.

(iii) Slip Geometry

The geometry of the slips, consisting of the surface profile of the slope and the estimated shape of the slip surface, was described in Sections 4.3.9 and 4.3.10. For analysis it was approximated to several straight lines, and the resulting slope and soil profiles are shown in Figures 5.16, 5.21, 5.23, 5.25, 5.27 and 5.29. They are from the central portion of the slips and were

modified in some later analyses to allow for changes in beach level and erosion of the toe.

The position of the slip surface was only known with reasonable certainty at the head and toe of the slips. Therefore a "band" of possible slip surfaces was analysed.

The six profiles analysed were:

FO	Slip F (pre-failure profile)						
FA	Slip F April 1987 (movement on pre-existing slip surface)						
FJ	Slip F January 1987	"	"	"	"	"	"
D	Slip D April 1987	"	"	"	"	"	"
B	Slip B July 1987	"	"	"	"	"	"
R	Slip R June 1987	"	"	"	"	"	"

Janbus' method was used to analyse the stability of the slips at the sites. A program supplied by Oasys Limited and run on a microcomputer was used, which allowed various assumptions about the distribution of the interslice forces. The assumption used in the analysis was of variably inclined interslice forces to overcome numerical problems encountered with other assumptions. Horizontal and vertical equilibrium were satisfied for each slice, and moment equilibrium for the slipped mass as a whole.

5.4 Stability Analyses Results

Six different cliff profiles were analysed, two at Rolston and four at Grimston. Five of the profiles were of slipped masses which were moving slowly downslope on pre-existing slip surfaces at the time of the survey. These were therefore analysed using residual strength parameters. One of the profiles was of the cliff prior to failure and was therefore analysed using peak strength parameters.

The effects of varying groundwater conditions, slip surface position, strength parameters, beach thickness and slope profile on the factor of safety were examined.

Analyses were first made of a hypothetical profile of slip 'F' at Grimston prior to failure (profile 'FO'). This profile was estimated from the post failure surveyed profile and measured surface movements.

Initially the profile was analysed using the $\sigma_u = 0$ method, and then effective stress analyses were carried out.

5.4.1 Analysis of 'First-Time' Failure at Grimston using the Total Stress ($\sigma_u = 0$) method

Analyses of the stability of profile 'FO' at Grimston were carried out using peak undrained shear strength parameters and

the $\phi_u=0$ method. Despite the severe disadvantages of this method for the analysis of slopes as discussed in Section 2.4.1, it was felt useful for illustrative purposes. The following mean values of undrained shear strength (c_u) were estimated for each till from the cone penetration and laboratory tests (see Section 4.4.1):

Till	Peak Undrained Shear Strength (c_u) kN/m ²
Weathered	80
Withernsea	100
Skipsea	130

A search was made for the critical slip circle which is shown in Figure 5.15. A 4m deep tension crack was included at the head of the slip. Tension cracks of about this depth were often observed behind cliff failures. The toe of the slip surface was constrained to emerge in the foreshore, as observed in the actual failure. The factor of safety obtained was 1.54.

To obtain a factor of safety of unity it was necessary to decrease the undrained shear strength by about a third. Some of this difference may be attributed to the differing undrained stress paths followed in cone tests and in the cliff. However the majority of it is probably due to swelling and softening of the till in response to stress relief, prior to failure.

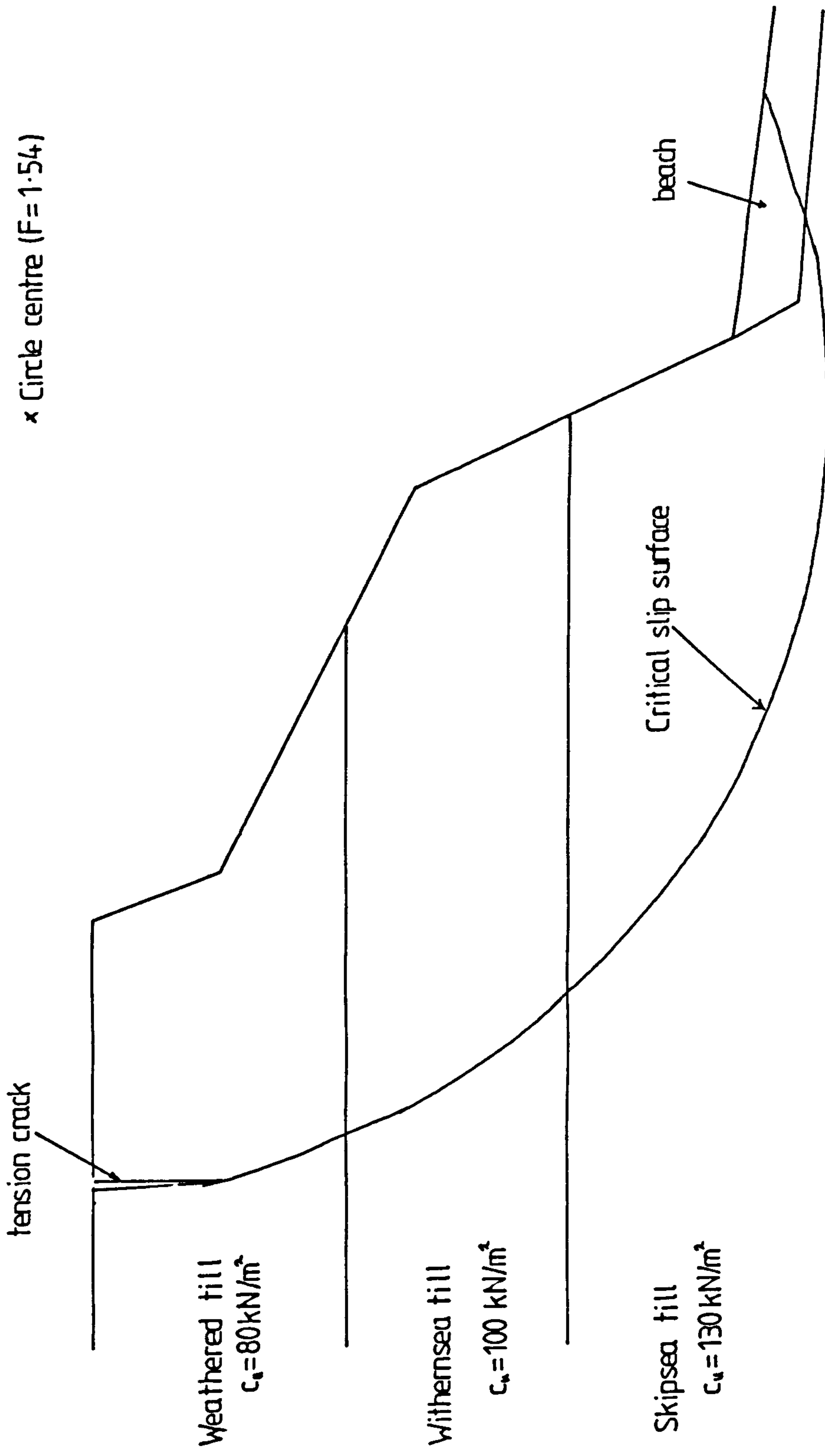


Figure 5.15	Scale 1:200
Critical slip circle for total stress $\phi_u = 0$ method	

5.4.2 Analysis of 'First-Time' Failure at Grimston using the Effective Stress Method

Analyses of the profile 'FO' using effective stress methods are discussed in this Section. Owing to the uncertainty in the position of the slip surface, a 'band' of slip surfaces was analysed to find the one with the lowest factor of safety. The approximate position of the slip surface was known at the head and toe of the slip and therefore the slip surfaces analysed were constrained to pass through these positions.

In the initial analyses a full beach, constant pore pressure ratio of 0.2 and peak strength parameters from "standard" triaxial tests were used. The slope profile, slip surfaces and factors of safety obtained are given in Figure 5.16.

The most critical surface found was a non-circular surface at moderate depth with a factor of safety of 0.78. As the profile is a pre-failure one, a factor of safety of 1 or greater would be expected. The three most probable reasons for this discrepancy are:

1. neglect of the effects of lateral variation in slope profile and end effects;
2. pore water pressures on the slip surface may be lower than than those used in the analyses;

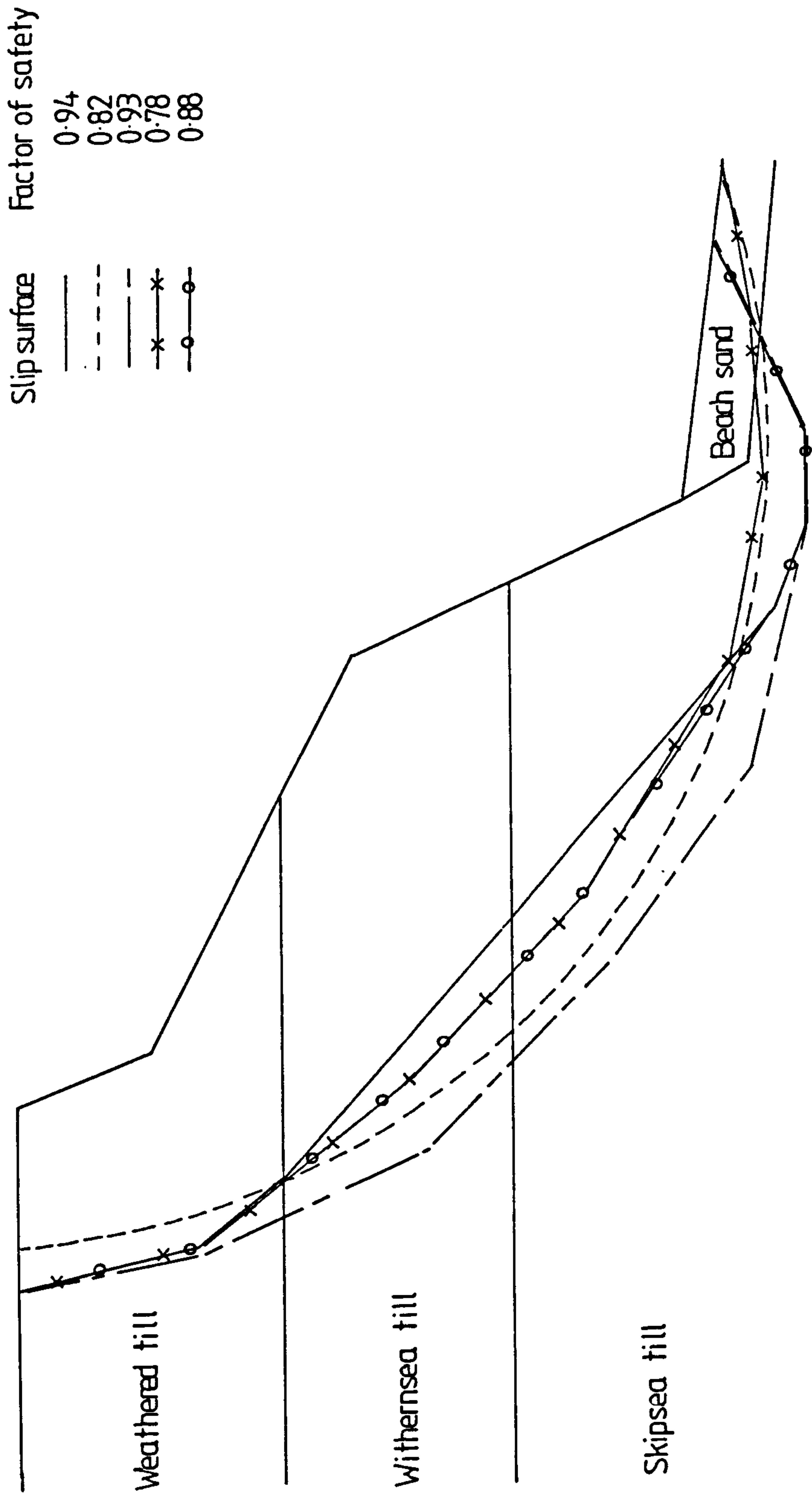


FIGURE 5.16 Scale 1:200

Profile 'FO' at Grimston with slip surfaces and factors of safety obtained

3. the mobilized peak shear strength parameters may be higher than predicted from "standard" triaxial tests.

Chen and Chameau (1983) state that for soils with cohesion the three-dimensional factor of safety (F_3) is greater than that calculated using the assumption of plane strain geometry (F_2). For a 6.1m high slope at an inclination of 2.5:1 they found that the ratio F_3/F_2 varied between 1.05 and 1.12. They state that for steep slopes the ratio F_3/F_2 is less than for shallow slopes. Gens et al. (1983) found values of F_3/F_2 varying between 1.03 and 1.23 for 6 slopes of around 30° inclination in $\phi_u=0$ analyses.

The effect of lateral variation of slope profile on factor of safety was evaluated during a parametric study of the stability of slips 'FO', 'FA' and 'D' at Grimston. The factor of safety was calculated for 'end' profiles as well as the central profile, and a weighted average by length taken as shown in Figure 5.15a. The following results were obtained:

Slip	Factor of Safety	
	Central Profile	Weighted average of central and end profiles
FO	0.66	0.70
FA	0.71	0.76
D	0.78	0.82

Figure 5.15b Results of analyses taking account of lateral variation of slope profile.

Note: These analyses were run during an early stage of the study, with a residual strength of $\phi'_r = 16^\circ$.

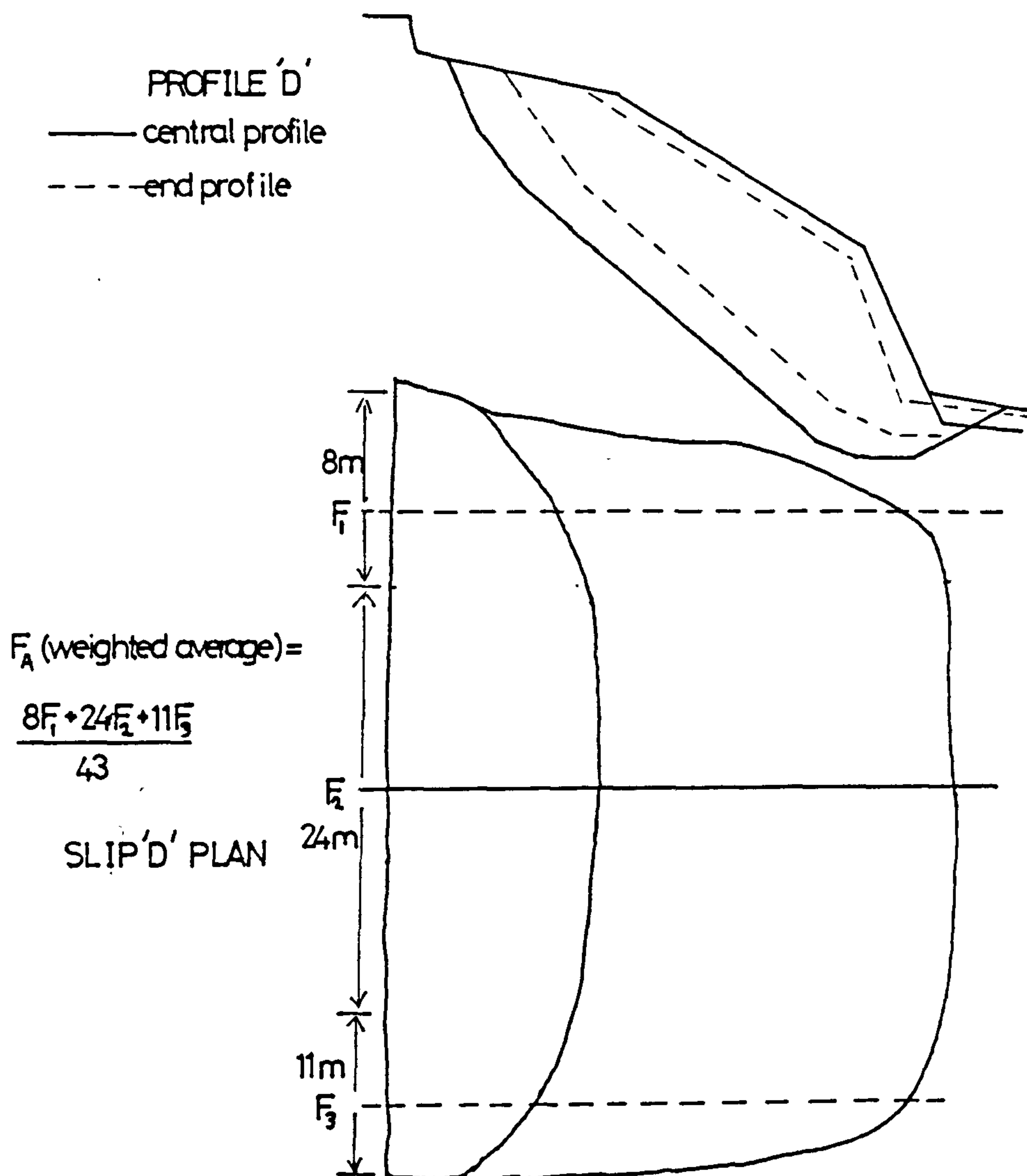


Figure 5-15a | Scale 1:400

Central and end profiles for slip 'D' and weighting procedure used to allow for lateral variation of profile

The weighted average factor of safety of central and end profiles was greater than the factor of safety of the central profile by about 6%. While this is not a true three-dimensional factor of safety, the increase is in reasonable agreement with the F_3/F_2 ratios quoted earlier.

Although neglect of end effects can explain part of the discrepancy, it is unlikely that it can explain all of it, as this would require an F_3/F_2 ratio of 1.28, while following from the above it is probably less than 1.1. The remainder is most probably due to lower pore water pressures on the slip surface than used in the analysis. This may be because of:

1. innaccuracy of the flow nets used to predict the value of r_u ;
2. depression of pore water pressure due to stress relief.

A study was made of the effect of varying r_u on factor of safety for the critical slip surface and the results are given in (1) of Figure 5.17. These show that an r_u value of 0.1 or less would be required to give a factor of safety approaching 1.

The accuracy of the sketched flow nets close to the cliff is questionable, particularly outside the zone where pore water pressure measurements are available to check them. This may result in an over-estimate of pore water pressure. The other

possibility of depression due to stress relief is discussed below.

Depressed pore water pressures are not expected to persist in the slope for long (ie. more than several months), see Section 5.1.3. However, falls of clay from the cliff face, mudslides and the change in geometry as slips move downslope could result in frequent, though small, unloading of parts of the slip surface. This could result in a near continual depression of pore water pressure, although of small magnitude.

			r_u value			
			0.2	0.15	0.1	0
(1). Peak strength parameters from "standard" triaxial tests						
Till	$c' (kN/m^2)$	ϕ'°				
Weathered	13	25				
Withernsea	17	27	0.78	0.83	0.88	0.99
Skipsea	7	26				
(2) As (1) with c' reduced to 30% of full value in weathered zone						
Weathered	4	25				
Withernsea	17	27	0.76	NA	NA	0.96
Skipsea	7	26				
(3) As (2) with c' of Withernsea till reduced to $7kN/m^2$						
Weathered	4	25				
Withernsea	7	27	0.73	NA	NA	0.93
Skipsea	7	26				
(4) $c'=0$ throughout						
Weathered	0	25				
Withernsea	0	27	0.64	NA	0.74	0.83
Skipsea	0	26				
NA = Not Analysed.						

Figure 5.17 Factors of safety for critical slip surface for profile 'FO' with a full beach.

For example if an average vertical thickness of 1m of till was removed, and it is assumed that the soil is saturated, the initial theoretical reduction in pore water pressure could be 20kN/m^2 .

The mobilized peak shear strength parameters may be higher than predicted from standard triaxial tests for the following two reasons. Direct shear tests may provide a better estimate of the value of ϕ' at the cliff foot than triaxial tests, and this is generally higher than that obtained from triaxial tests (see Section 5.3.3). The stress path followed by elements of soil below the cliff foot will probably be a passive stress relief path. Burland and Fourie (1985) found the mobilized ϕ' in the early stages of passive stress relief tests on remoulded London Clay was higher than in standard triaxial tests (see Section 5.3.3). The effect of using a higher ϕ' below the cliff foot is studied in a later Section.

The effect on factor of safety of (i) varying the effective cohesion (c'), (ii) using "direct shear" parameters, (iii) using "reduction" parameters, (iv) varying beach level and (v) changing profile to simulate toe erosion was studied for profile 'FO' and is discussed below. In all the analyses the critical slip surface from the earlier study was used (see Figure 5.16). No tension crack was assumed in the cliff top, however the effect of assuming a tension crack up to 4m deep was checked in some analyses and found not to change the factor of safety.

(i) Results of analyses varying effective cohesion (c') on factor of safety of profile 'FO'

The effects of varying the effective cohesion (c') on the factor of safety were studied. The profile was first analysed using the values of c' measured during the course of this study, in the range 7kN/m^2 to 17kN/m^2 . These values are within the range for lodgement till of $0\text{--}25\text{kN/m}^2$ quoted by Wrigley and Sladen (1983).

The tests on till from Cowden reported by Marsland and Powell (1985) indicated that c' was very low or zero. However Skempton and Brown (1961) found a value of c' of about 8.5° in till, and concluded that back analysis of a slip that this c' must be operative in the field. They also estimated that c' in the weathered till was about 30% of its value in unweathered till, and found that there was no evidence for a long term reduction in c' , as is found in stiff heavily overconsolidated fissured clays.

Considering the above it was decided to study the effect of varying the value of c' . Firstly c' was reduced to 30% of its full value in the weathered zone, which resulted in a small decrease in factor of safety (see (2) of Figure 5.17). Secondly, the value of c' of Withernsea till of 17kN/m^2 was reduced to 7kN/m^2 , as the value of 17kN/m^2 may be unrealistically high. This resulted in a further small reduction in factor of safety (see (3) of Figure 5.17).

However the factor of safety of 0.93 obtained using an r_u of 0 is still feasible.

Finally, the profile was analysed using a c' of zero throughout, which resulted in a large reduction in factor of safety to 0.64 with an r_u value of 0.2 and 0.83 with an r_u value of zero (see (4) of Figure 5.17). If it is correct that $c'=0$, then the pore water pressure on the slip surface must be much lower than estimated, and negative in places in order to obtain a factor of safety of 1.

(ii) Results of analyses using direct shear parameters below the cliff foot

The effect of using 'direct shear' parameters below the cliff foot were examined. The most critical slip surface found in the previous analyses was used, and a full beach assumed. "Direct shear" parameters of $c'=7\text{kN/m}^2$ $\phi'=29^\circ$ were assumed in the Skipsea till below the cliff foot (see Section 5.3.3). "Standard" parameters were used above the cliff foot. The effect of assuming $c'=0\text{kN/m}^2$ for the standard parameters was also examined. The results are given in Figure 5.18.

Strength Parameters Used	r_u value	
	0.2	0
"Standard" throughout	0.78	0.98
"Standard" above cliff foot, "direct shear" ($c'=7\text{kN/m}^2$, $\phi=29^\circ$) below	0.82	1.04
"Standard" with $c'=0\text{kN/m}^2$ throughout	0.64	0.83
"Standard" with $c'=0\text{kN/m}^2$ above cliff foot, "direct shear" ($c'=7\text{kN/m}^2$, $\phi=29^\circ$) below	0.69	NA
Note: NA = Not Analysed.		

Figure 5.18 Factors of safety for analyses of profile 'FO' using "direct shear" parameters below the cliff foot.

The results show an increase of factor in safety of between 5% and 8% when "direct shear" parameters are used below the cliff foot. The factors of safety obtained are closer to 1, and therefore the use of "direct shear" parameters below the cliff foot is probably more realistic than using "standard" parameters. This is in agreement with the findings of A. Butcher (personal communication) in analysis of a slip at Cowden, Holderness.

(iii) Results of analyses using "reduction" parameters

Two analyses were carried out using the parameters obtained from the "reduction" tests on sample 1/6 above the level of the

cliff foot and "standard" parameters below it. With an r_u of 0.2 a factor of safety of 0.59 was obtained, and with an r_u of zero the factor of safety was 0.73. This represents a reduction in factor of safety of about 25% when compared with using "standard" parameters throughout. Thus it seems unlikely that these "reduction" parameters are correct, and they are probably too low due to experimental error.

(iv) Results of analyses varying beach level

The factors of safety for profile 'FO' obtained with a 2m, 1m and zero beach thickness are given in Figure 5.19. "Standard" parameters were used throughout.

Beach thickness	r_u	
	0.2	0.0
2m	0.78	0.98
1m	0.71	NA
Zero	0.67	0.89
Note: NA = Not Analysed.		

Figure 5.19 Factors of safety for profile 'FO' with varying beach level.

The large influence of beach thickness on factor of safety is shown. The reduction in beach from the normal 2m thickness to zero results in an average 12% decrease in factor of safety.

If the beach is entirely removed then it seems likely that failure of the profile 'FO' would occur, as unless r_u was zero or less along most of the slip surface the factor of safety is substantially less than 1.

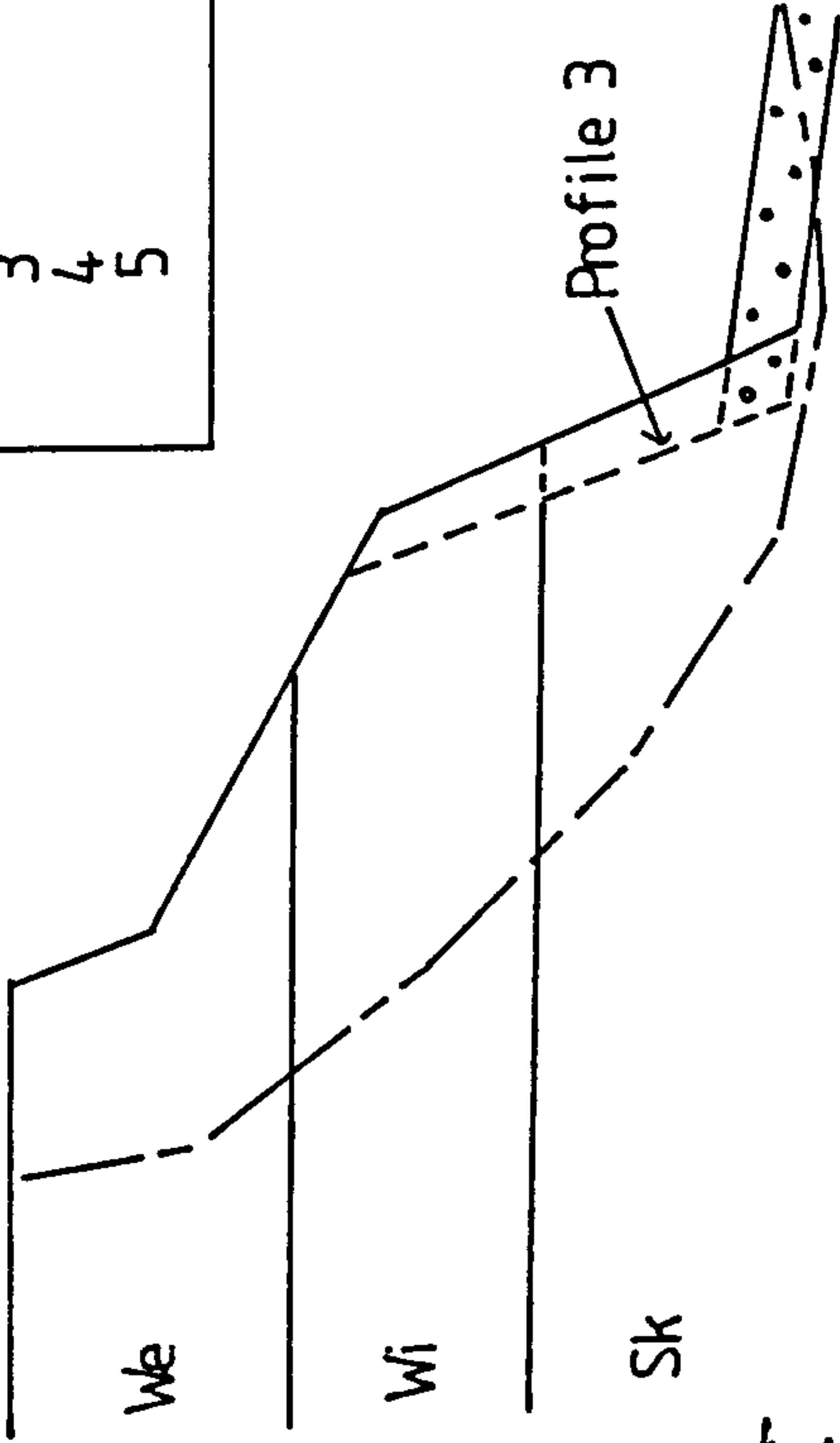
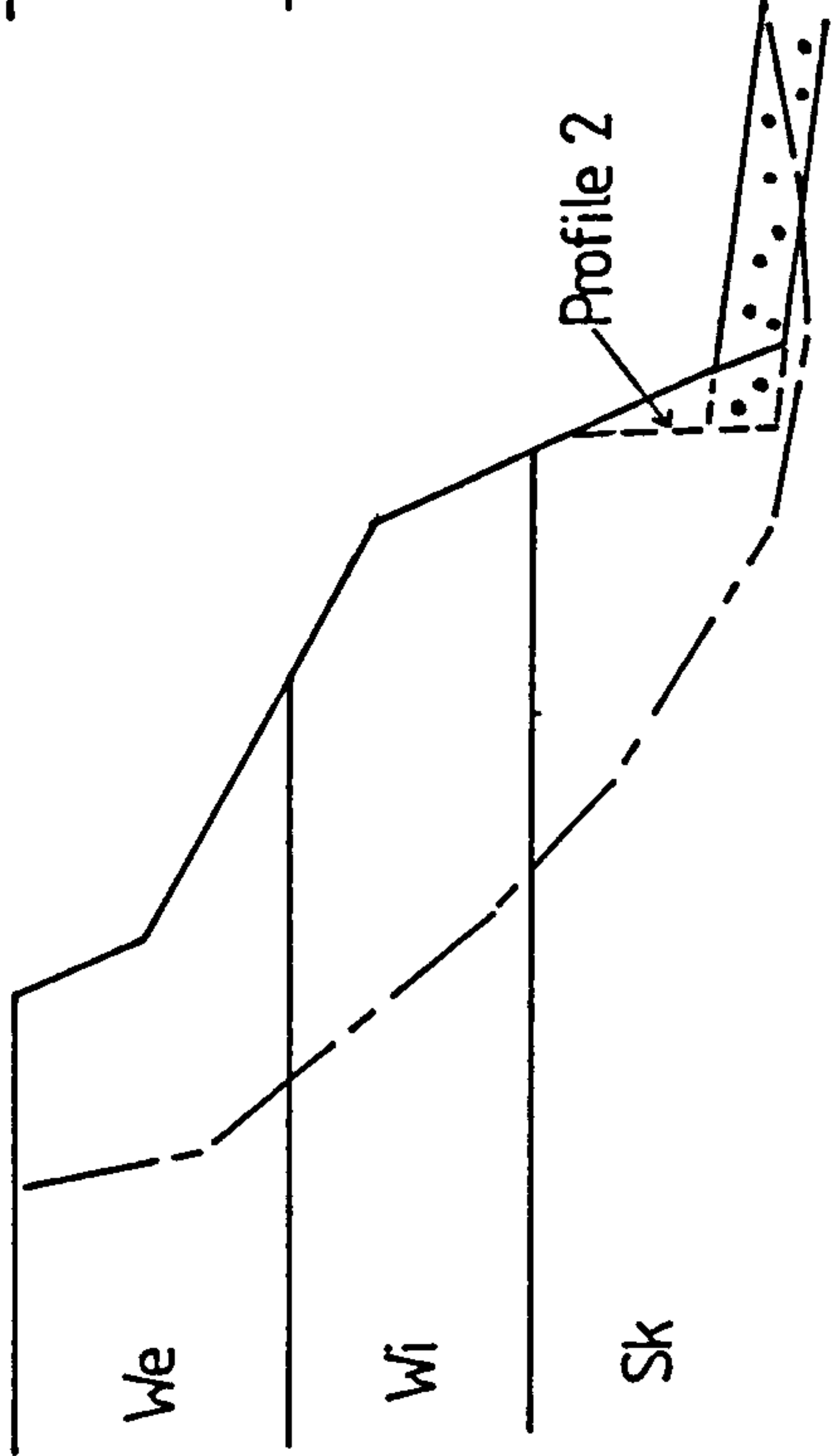
(v) Results of analyses changing slope profile

The effect on factor of safety of profile 'FO' of changing the slope profile in the lower cliff face to simulate toe erosion was examined in some analyses. An r_u value of zero was assumed in all cases and a full (2m thick) beach. "Standard" parameters were assumed throughout the slope. The changes in slope profile which were studied and the corresponding factors of safety obtained are given in Figure 5.20.

It was found that a small fall from the cliff face (Profile 2) did not affect the factor of safety much. If a 2m thickness of till was removed from the lower cliff face (Profile 3) the factor of safety was reduced by 4%. Further removal of material from the toe (see Profiles 4 and 5), caused reductions of 6% and 7% respectively, compared with the original factor of safety.

The study shows that toe erosion can be an important factor in reducing stability of the cliff, with nearly as much influence as lowering of the beach. The two processes are linked, as

Slope profile	Factor of safety ($r_c=0$)
1	0.99
2	0.98
3	0.95
4	0.93
5	0.92



KEY

Original profile(1) lines are solid
New profile (2,3,4,5) lines are dashed

We - Weathered till

Wi - Withemsea till

Sk - Skipsea till

--- Critical slip surface from Figure 5.16

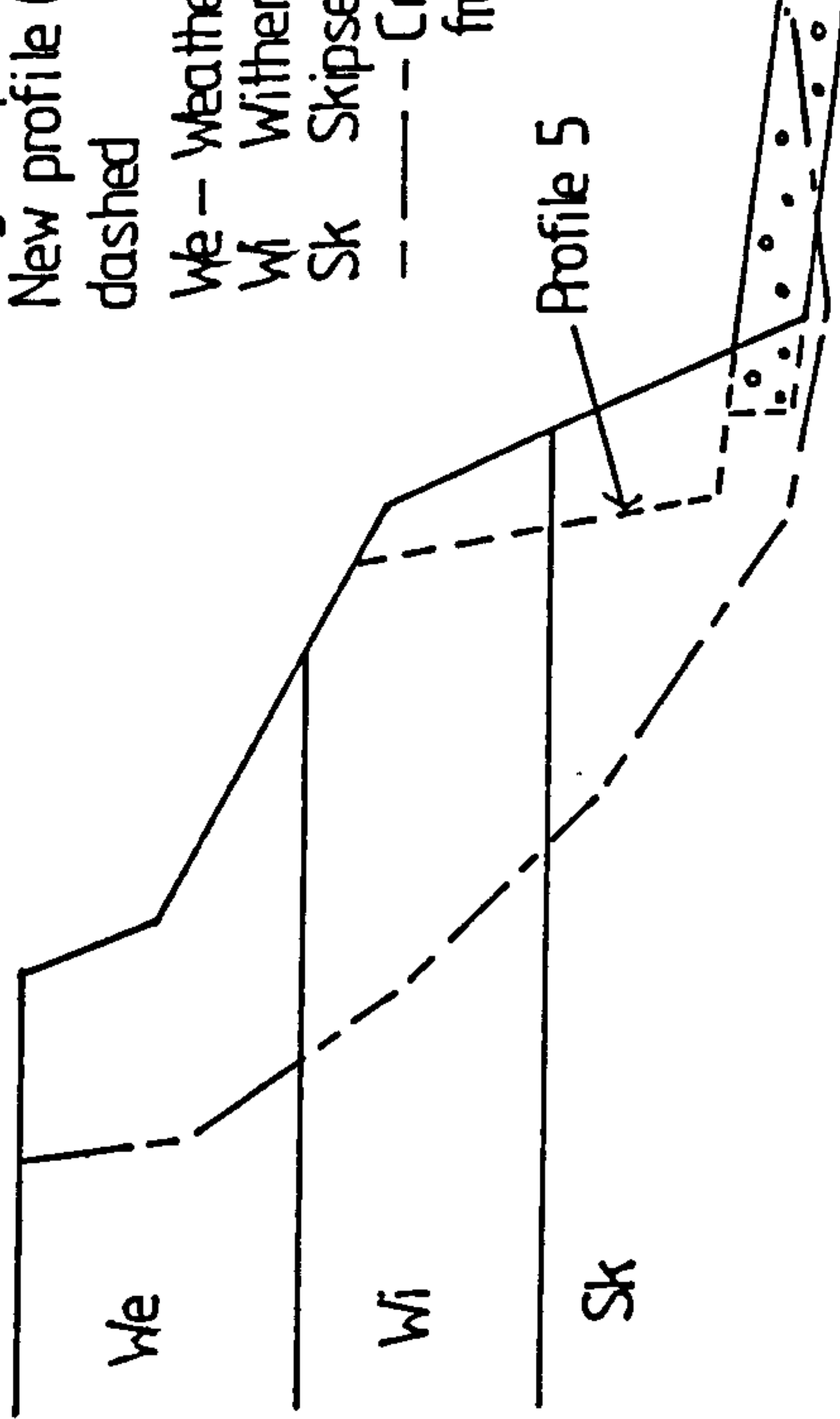
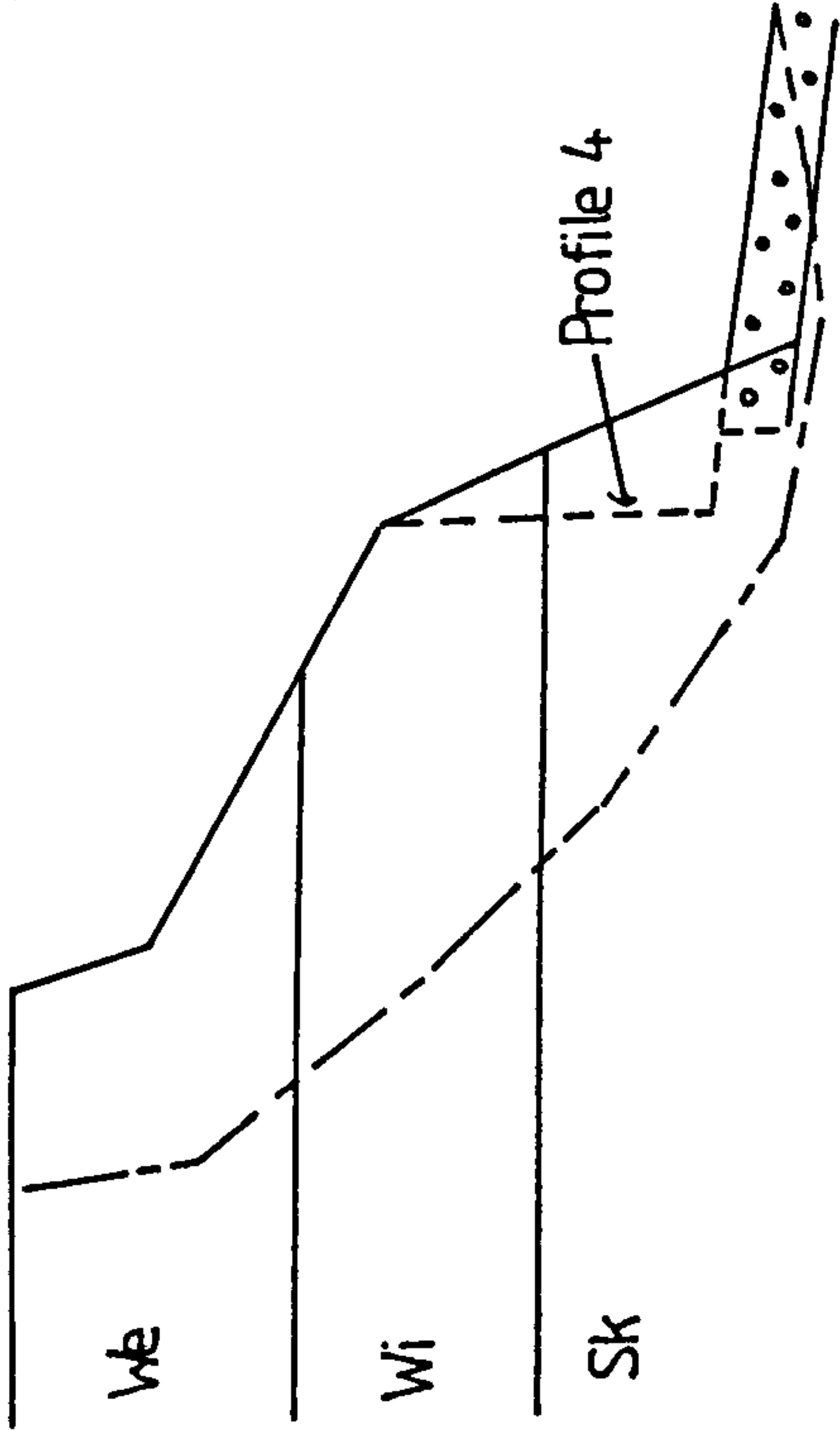


Figure 5.20 | Scale 1:400 | Changes in profile of slip 'F0' and corresponding factors of safety

lowering of the beach will cause an increase in toe erosion through the greater amount of wave energy reaching the toe.

5.4.3 Analysis of Failures on Pre-Existing Shear Surfaces at Grimston

Analyses were made of slip 'F' at Grimston at two different stages in its cycle of failure in addition to the original profile 'FO' discussed earlier. These were the profile at April 1987 ('FA') and the profile at January 1988 ('FJ') (see Section 4.3.9). Analyses were also made of slip 'D' at Grimston at April 1987 ('D').

Residual strength was used in all the above analyses. An r_u of 0.2 was used initially. Since the position of the slip surface was unknown except at the head and toe of the slip, a band of slip surfaces was analysed for each slip, corresponding fairly closely to the band of slip surfaces analysed for profile 'FO'. The most critical slip surface found was then used in a parametric study. The results are discussed below.

Measurements of surface movements show that both the slips analysed were continuously moving downslope at a rate of a few cm/day (see Figure 4.32). Therefore the factor of safety would be expected to be very close to unity.

(i) Results of analyses of Profile 'FA'

The band of slip surfaces analysed for profile 'FA' and the associated factors of safety are shown in Figure 5.21. The most critical surface found had a factor of safety of 0.77. A parametric study was then carried out and the results given in Figure 5.22.

Beach Thickness	Strength		r_u		
	c'_r kN/m ²	ϕ'_r	0.2	0.1	0
2m	0	25	0.77	0.87	0.97
2m	6	25	0.89	NA	NA
1m	0	25	0.68	NA	0.87
0m	0	25	0.62	NA	0.80
Note: NA = Not Analysed.					

Figure 5.22 Factors of safety for critical slip surface of Profile 'FA'.

With a full beach (2m) a factor of safety approaching unity is obtained only if r_u is near zero.

In general the residual cohesion c'_r of soils is taken to be zero. However in some soils a small cohesion intercept is sometimes apparent. For example the effective cohesion reported by Lupini et al (1981) for till at Cowden was $c'_r = 5.8 \text{ kN/m}^2$. An analysis was therefore carried out using a

Slip surface	Factor of safety
1 ———	0.95
2 ———	0.90
— — — —	0.91
— — — —	0.77
— x — x —	0.90
— o — o —	0.83

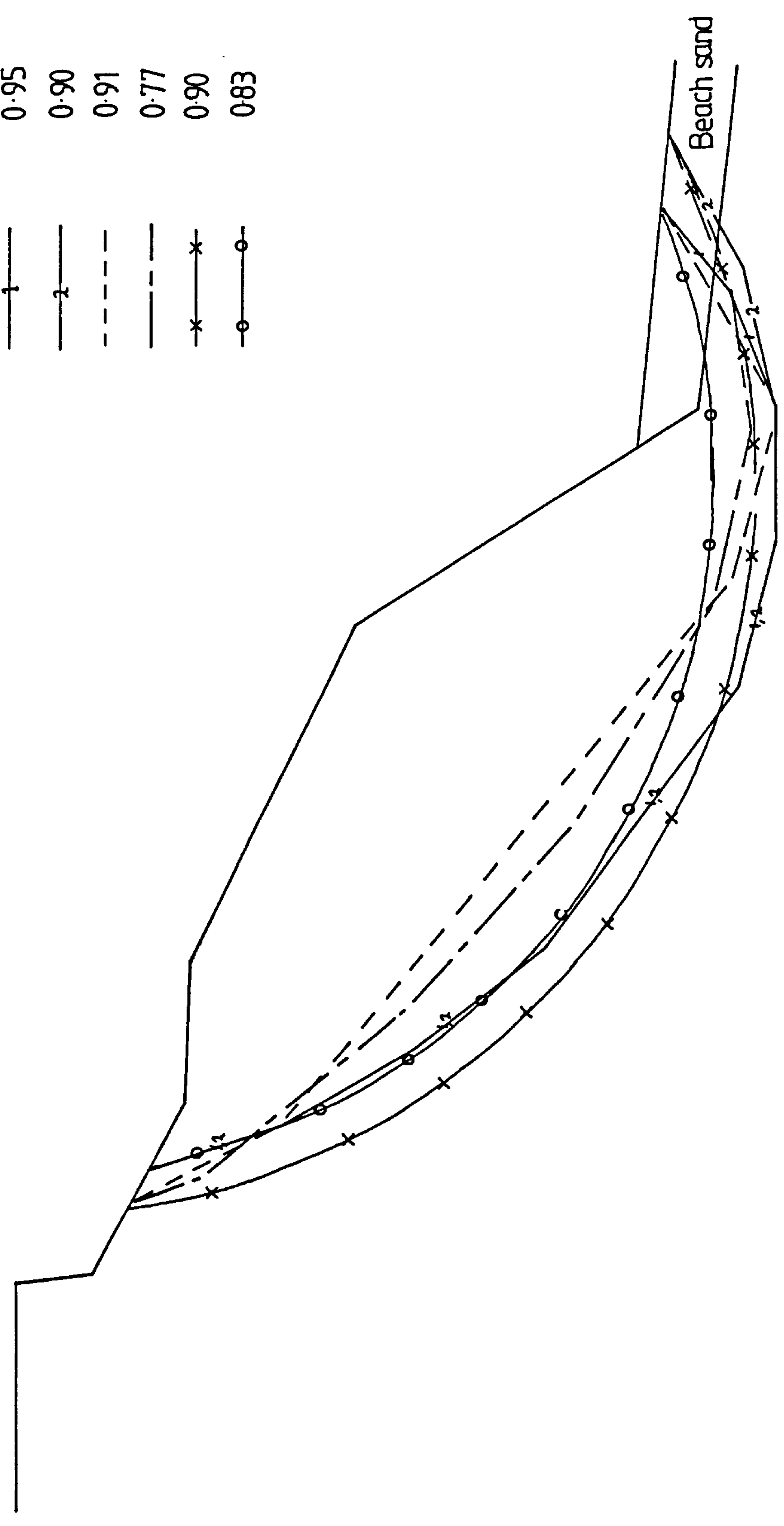


Figure 5.21 | Scale 1:200
 Profile 'FA' at Grimston with slip surfaces analysed and factors of safety obtained

c'_r of 6kN/m^2 , which gave an increase of 16% in factor of safety compared with a c'_r of 0kN/m^2 .

Analyses were carried out with a 1m beach and no beach giving average decreases in factors of safety of 11% and 19% respectively, compared with a full beach. With no beach the factor of safety obtained was very low, even with an r_u of zero. Thus it seems likely that either a small effective cohesion is operative, or that lowering of the beach would result in renewed movement. The level of the beach was normal during April 1987 (about 2m) and therefore which of the above hypotheses was correct could not be determined.

(ii) Results of analyses of Profile 'FJ'

The band of slip surfaces analysed is shown in Figure 5.23, together with the factors of safety obtained. With a full beach, an r_u of 0.2 and residual strength parameters, the lowest factor of safety obtained was 0.83.

The results of a parametric study using this critical slip surface are given in Figure 5.24.

Slip surface Factor of safety

—+—	1.12
—x—	1.05
—3—	0.99
— -- —	0.83
— — —	0.94
—*—*—	0.84

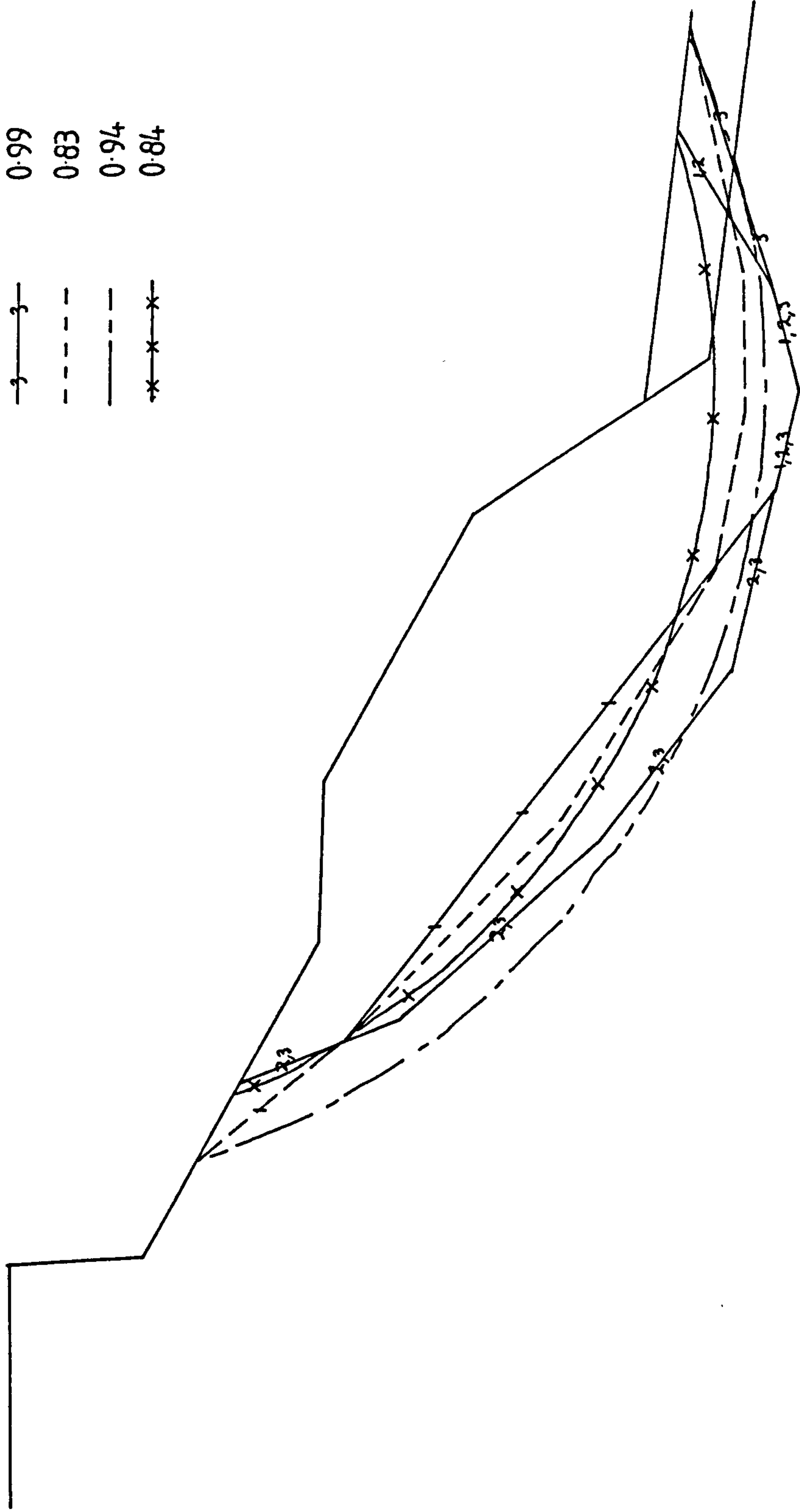


Figure 5.23 | Scale 1:200
Profile 'FJ' at Grimston with slip surfaces analysed and factors of safety obtained

Beach Thickness	Strength		r_u		
	c_r' kN/m ²	ϕ_r' °	0.2	0.1	0
2m	0	25	0.83	0.96	1.09
2m	6	25	0.99	NA	NA
1m	0	25	0.72	NA	0.95
Zero	0	25	0.66	NA	0.89
Zero	6	25	NA	NA	1.06
Note: NA = Not Analysed.					

Figure 5.24 Factors of safety for critical slip surface of Profile 'FJ'.

The results show that the factors of safety of the slip are closer to one with a full beach than Profile 'FA'. An r_u of 0.1 is sufficient to achieve a factor of safety close to unity, or 0.2 if an effective cohesion of 6kN/m² is included. The effect of lowering the beach to 1m or zero thickness is to decrease the factor of safety by 13% or 19% respectively.

With no beach the factor of safety is well below unity, even with an r_u of zero, unless a small cohesion is included. In January 1988 the beach at Grimston was about 1m thick, which would give a factor of safety of 0.95 if r_u was zero.

(iii) Results of analyses of Profile 'D'

The profile of slip 'D' at April 1987 was analysed. A 'band' of slip surfaces was analysed initially with a full beach and r_u of 0.2 and residual shear strength parameters. These are shown in Figure 5.25, with the factors of safety obtained. The lowest factor of safety of 0.69 was for a circular slip surface.

A parametric study was carried out on the critical slip surface and the results are given in Figure 5.26.

Beach Thickness	Strength		r_u		
	c_r' kN/m ²	ϕ_r' °	0.2	0.1	0
1.5m	0	25	0.69	0.83	0.93
1.5m	6	25	0.78	NA	1.02
Zero	0	25	0.66	NA	0.87
Zero	6	25	0.72	NA	0.96
Note: NA = Not Analysed.					

Figure 5.26 Factors of safety for critical slip surface of Profile 'D'.

The results show the sensitivity of the factor of safety to r_u . A factor of safety approaching unity is obtained with an r_u of zero. With no beach the factor of safety is reduced by about

Slip surface Factor of safety

—+—	0.83
—x—	0.73
— -- —	0.83
— — —	0.87
—x—	0.89
—o—	0.69

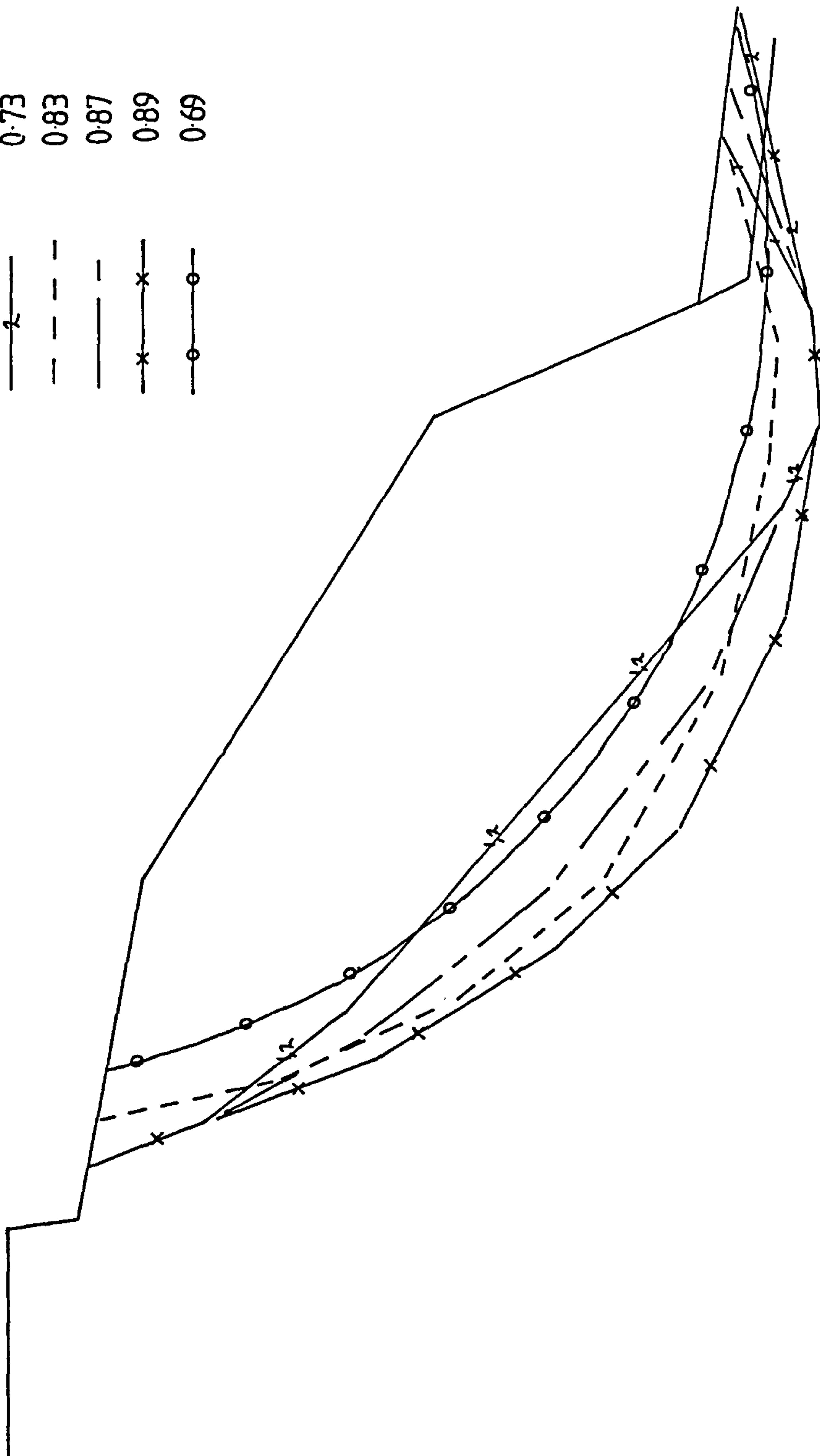


Figure 5.25 Scale 1:200

Profile 'D' at Grimston with slip surfaces analysed and factors of safety obtained

6%, showing this profile to be less sensitive than slip 'F' to beach level changes.

The factor of safety is well below unity with no beach, unless a small effective cohesion is included.

5.4.4 Analysis of Failures on Pre-Existing Shear Surfaces at Rolston

Analyses were carried out on two slips at Rolston, slip 'B' and slip 'R' (see Section 4.3.10). The profiles which were analysed were of the slips just after failure, in July 1987 and June 1987 respectively. The movement along the slip surface was about 3m, and therefore residual strength was used in the analyses.

The mean long-term pore pressure ratio was estimated to be 0.12 from Figure 5.13. The position of the slip surface was known at the head and toe of the slips, but unknown between them, therefore a 'band' of slip surfaces were analysed.

Surface movements were not measured but it was observed that the slips continued to move after failure, therefore the factor of safety would be expected to be close to unity.

(i) Results of analyses of slip 'B'

The band of slip surfaces analysed is shown in Figure 5.27. The lowest factor of safety obtained with $r_u=0.12$ and residual shear strength parameters of $c'_r = 0\text{kN/m}^2$ and $\phi'_r = 25^\circ$ was 0.78 for a circular slip surface.

A parametric study was carried out with this slip surface and the results are given in Figure 5.28.

Beach Thickness	Strength		r_u		
	$c'_r \text{ kN/m}^2$	$\phi'_r \text{ }^\circ$	0.12	0.06	0
1.4m	0	25	0.78	0.85	0.91
1.4m	6	25	0.90	0.97	NA
Zero	0	25	0.71	NA	0.84
Zero	6	25	0.82	0.90	0.95
Note: NA = Not Analysed.					

Figure 5.28 Factors of safety for critical slip surface of profile 'B'.

The results show that to obtain a factor of safety approaching unity, an r_u of zero or less is required, or the inclusion of a small cohesion.

Slip surface	Factor of safety
—+—	1.07
—2—	0.90
--- --	0.78
—- -	0.87
—x—	0.78

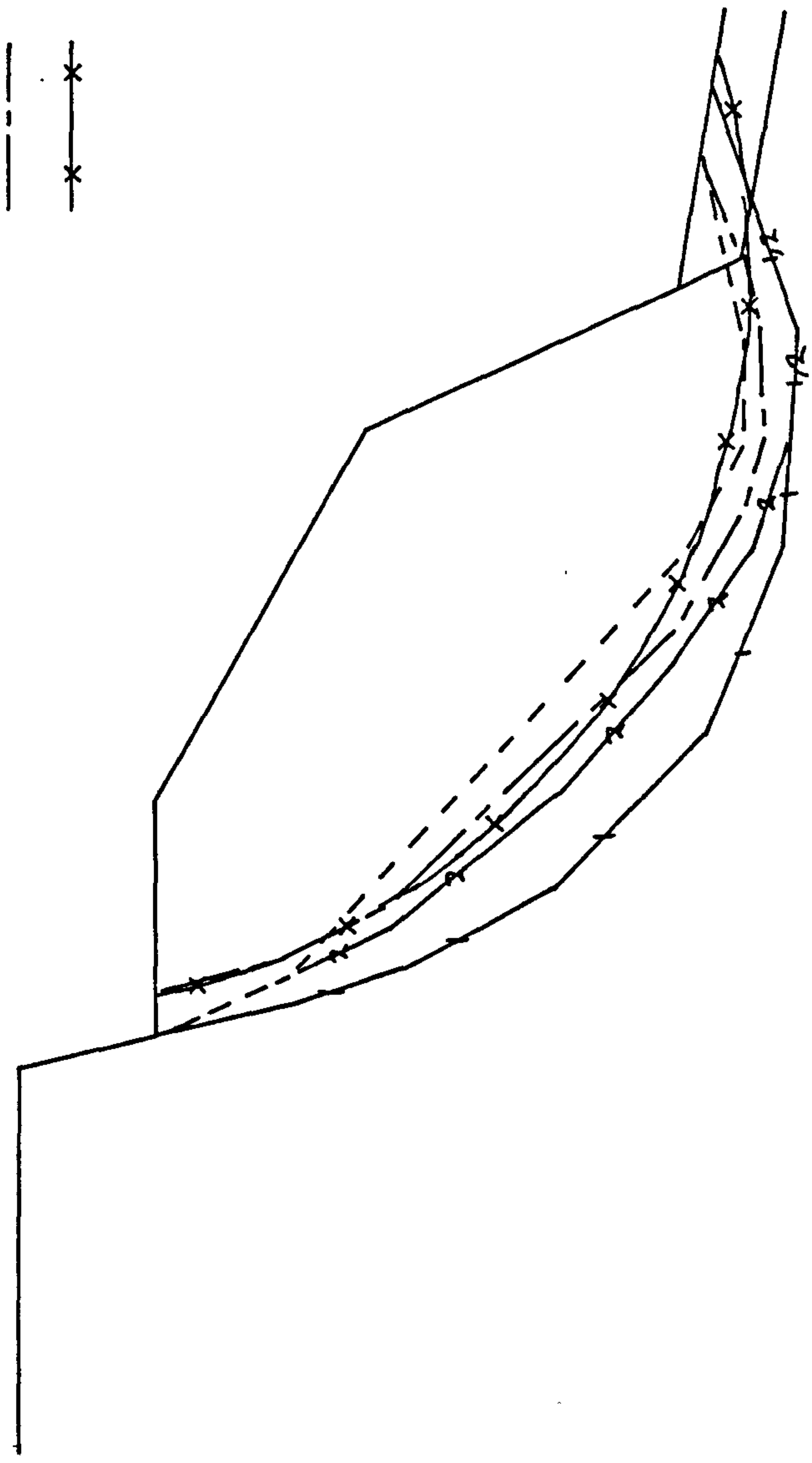


Figure 5.27	Scale 1:200
Profile B at Rolston with slip surfaces analysed and factors of safety obtained	

If the beach is removed the factor of safety is reduced by an average of 8%, and the factor of safety is well below 1, unless both an r_u of zero and a small cohesion are taken.

(ii) Results of analyses of slip 'R'

The band of slip surfaces analysed and the factors of safety obtained are shown in Figure 5.29. The most critical slip surface was circular with a factor of safety of 0.75 with an r_u of 0.2 and 0.83 with an r_u of 0.12. A parametric study was carried out and the results are given in Figure 5.30.

Beach Thickness	Strength		r_u		
	c'_r kN/m ²	ϕ'_r °	0.12	0.06	0
1.4m	0	25	0.83	0.89	0.95
1.4m	6	25	0.94	1.01	NA
Zero	0	25	0.73	NA	0.86
Zero	6	25	0.85	0.92	0.98
Note: NA = Not Analysed.					

Figure 5.30 Factors of safety for critical surface of Profile 'R'.

The factors of safety obtained are similar to those for slip 'B', although slightly higher. An r_u of zero or the inclusion of a small cohesion is necessary to achieve a factor of safety

Slip surface	Factor of safety
—+—	0.86
—2—	0.88
---	0.82
— -- —	0.82
—○—	0.75
—x—	0.82

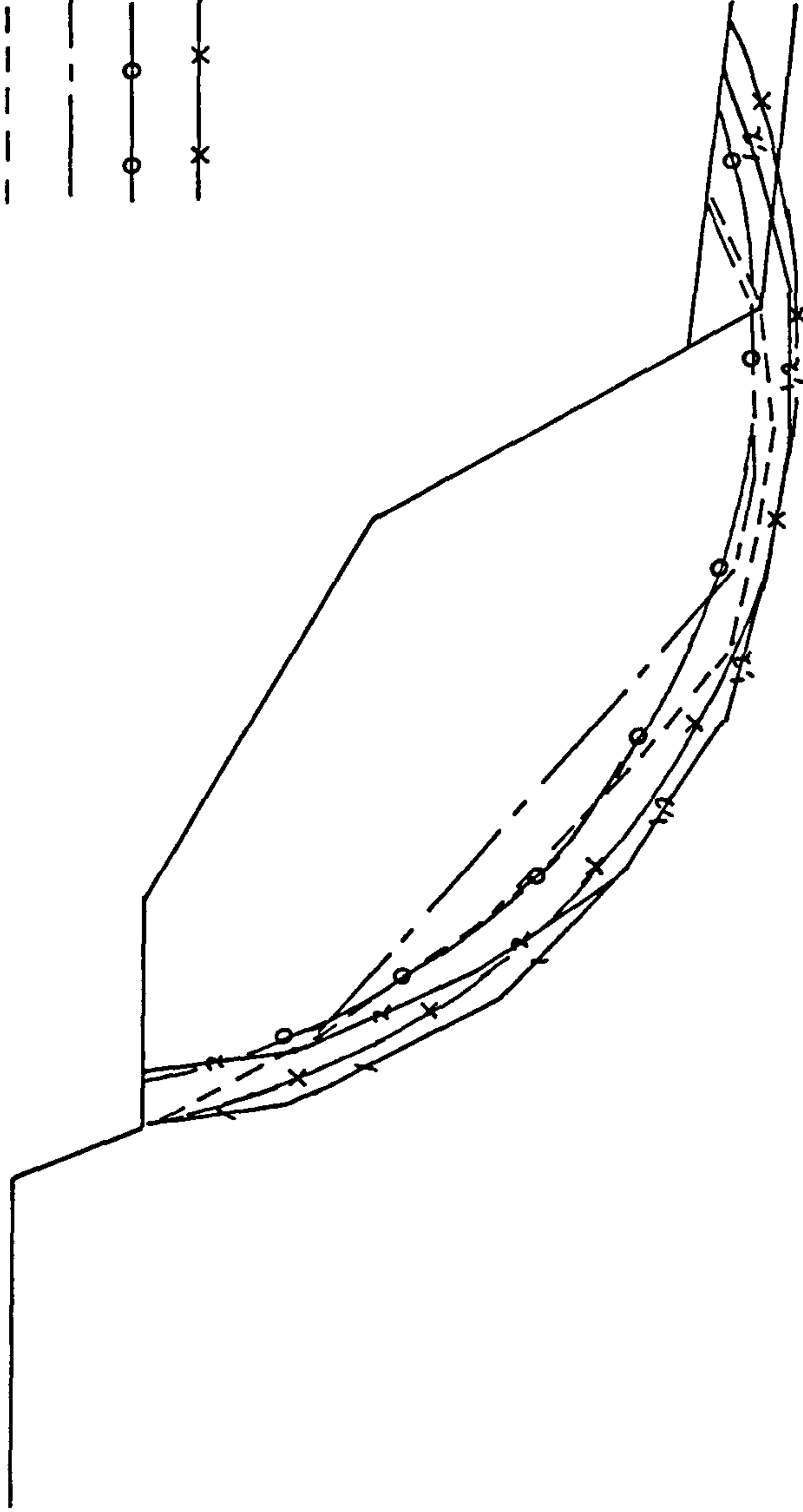


Figure 5.29	Scale 1:200
Profile 'R' at Rolston with slip surfaces analysed and factors of safety obtained	

approaching 1. Removal of the beach results in a reduction in factor of safety of about 10%.

5.4.5 Summary of Results of all Effective Stress Analyses

In this Section the results of the effective stress analyses are summarized and discussed. The lowest factors of safety obtained for all the profiles using the estimated long-term pore water pressure and the appropriate peak or residual strength parameters were all less than 1, with the range 0.69 to 0.83, mean 0.78. As the profiles analysed were either in a pre-failure condition or slowly moving, the factor of safety would be expected to be 1 or marginally greater. The reasons for this discrepancy were discussed in Section 5.4.2 and summarized below:

1. neglect of end effects and lateral variation of section;
2. possible presence of depressed pore water pressures along the slip surface from recent unloading;
3. possible higher mobilized effective stress strength parameters than measured in "standard" triaxial tests;
4. possible small c' component in residual strength.

Analyses of a failure at the Building Research Establishment (B.R.E.) site at Cowden also gave factors of safety of less than 1 (Butcher A.B., personal communication). The analyses by B.R.E. required depressed pore pressures on the cliff face to be input in order to simulate field behaviour. This is in agreement with hypothesis (2) above.

The amount by which pore water pressure must be depressed to achieve stability is now discussed. Depending upon how much the factors (1), (3) and (4) could raise the factor of safety, a depression of r_u to between about half its estimated long-term value and zero is required to achieve a factor of safety of 1. This would generally require a 20kN/m^2 to 30kN/m^2 depression in pore water pressure on the slip surface, corresponding to the removal of about 1m to 1.5m of material from the surface of the slope, if $\bar{B}=1$. As the soil is saturated, the assumption of $\bar{B}=1$ is reasonable.

Most of the required unloading would occur by mudslides and falls, and by the change in slope profile as the slipped mass moves downslope. The unloading would have to occur frequently, given the time periods of a few months estimated for dissipation of depressed pore water pressure in Section 5.1.3.

Analyses of the pre-failure profile 'F0' showed that the peak "reduction" strength parameters discussed in Section 4.6.8 are unlikely to be correct, probably owing to experimental error.

The use of "direct shear" strength parameters below the cliff foot results in a small increase in factor of safety, and is probably more appropriate than using "standard" triaxial strength parameters. This is in agreement with the findings of Butcher (personal communication) in stability analyses of the cliff at Cowden.

The assumption of a peak c' of zero in analyses of profile 'FO' resulted in a factor of safety substantially less than 1, even with an r_u of zero. Therefore it seems likely the use of a small effective cohesion c' is appropriate in the stability analyses.

The use of a residual c'_r of 6kN/m^2 in analyses of the slips on pre-existing failure surfaces gave factors of safety closer to 1 than with $c'_r = 0\text{kN/m}^2$. However the use of an r_u of zero also resulted in a factor of safety close to 1, except in the case of a low beach.

The influence of beach level on factor of safety is summarized in Figure 5.31.

Profile	Percentage reduction in factor of safety compared with a full beach (1.4m to 2m)	
	1m thick beach	No beach
'FO'	9	12
'FA'	11	19
'FJ'	13	19
'D'	-	6
'B'	-	8
'R'	-	10

Figure 5.31 The influence of beach level on factor of safety.

The large influence beach level has on factor of safety is clear, particularly for the slip profiles 'FA' and 'FJ'. These are profiles where the slipped mass has moved a long way downslope, and therefore a higher proportion of the slip surface passes through the beach.

The factor of safety of all profiles was less than 1 with no beach, unless both r_q =zero and c'_f >zero were assumed. Thus it is likely that lowering of the beach during 'ord' or storm conditions would result in renewed movement of existing slips, and cause 'first-time' slips of the cliff where the factor of safety is only marginally above unity.

Erosion at the toe of the slope results in a reduction of between 4% and 7% in factor of safety. As toe erosion is most likely to occur when beach levels are low, this will tend to exacerbate the effect of low beach levels on stability.

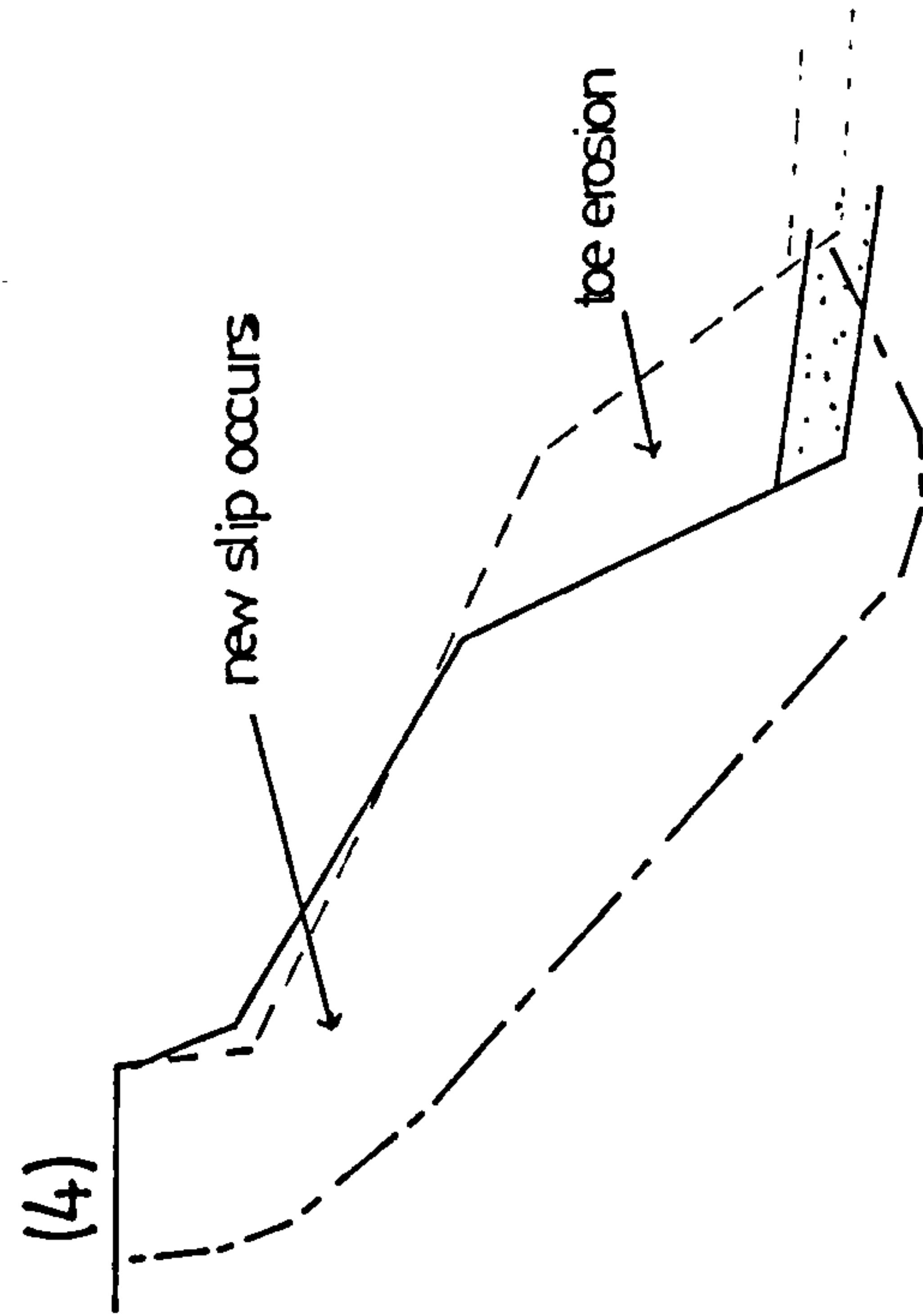
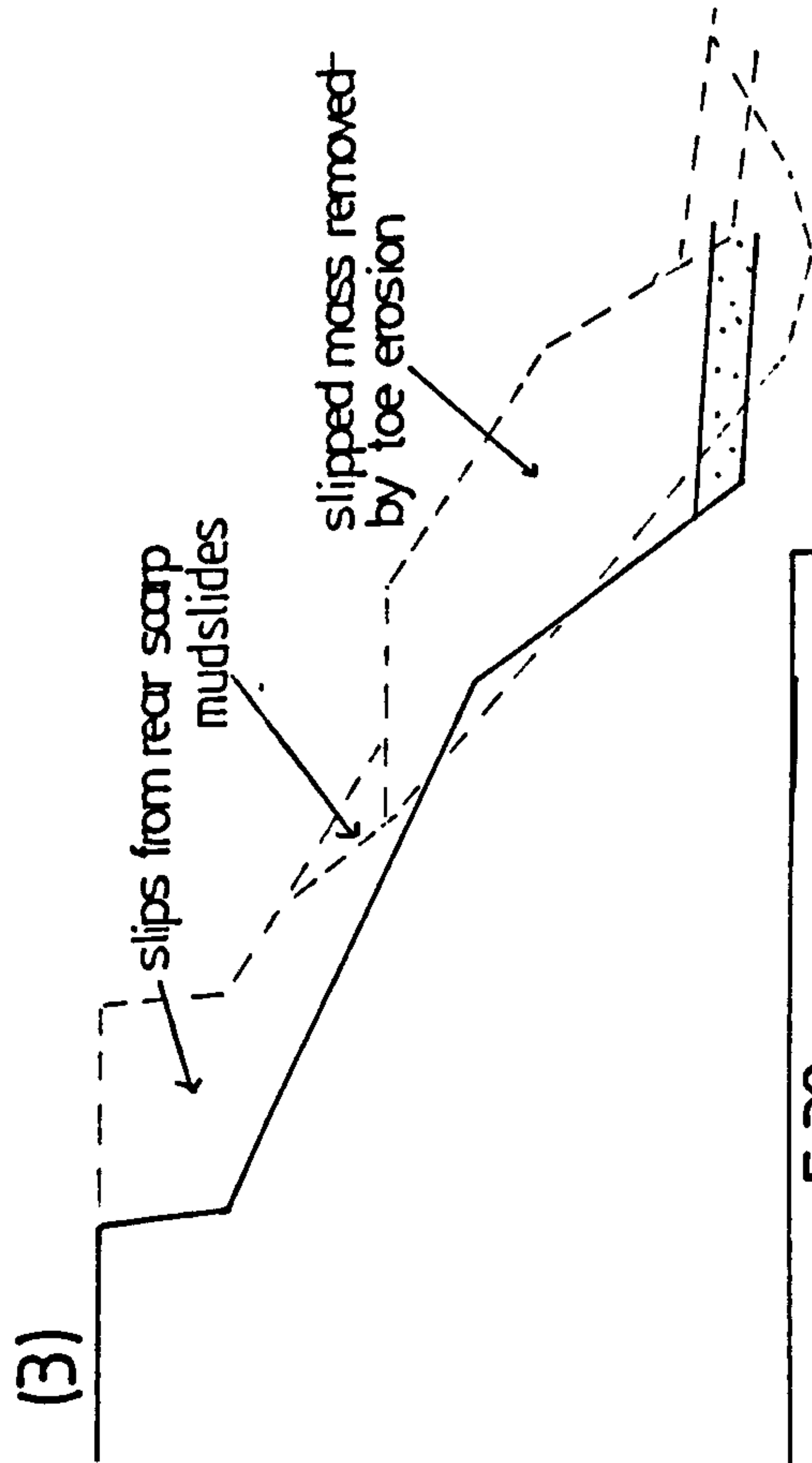
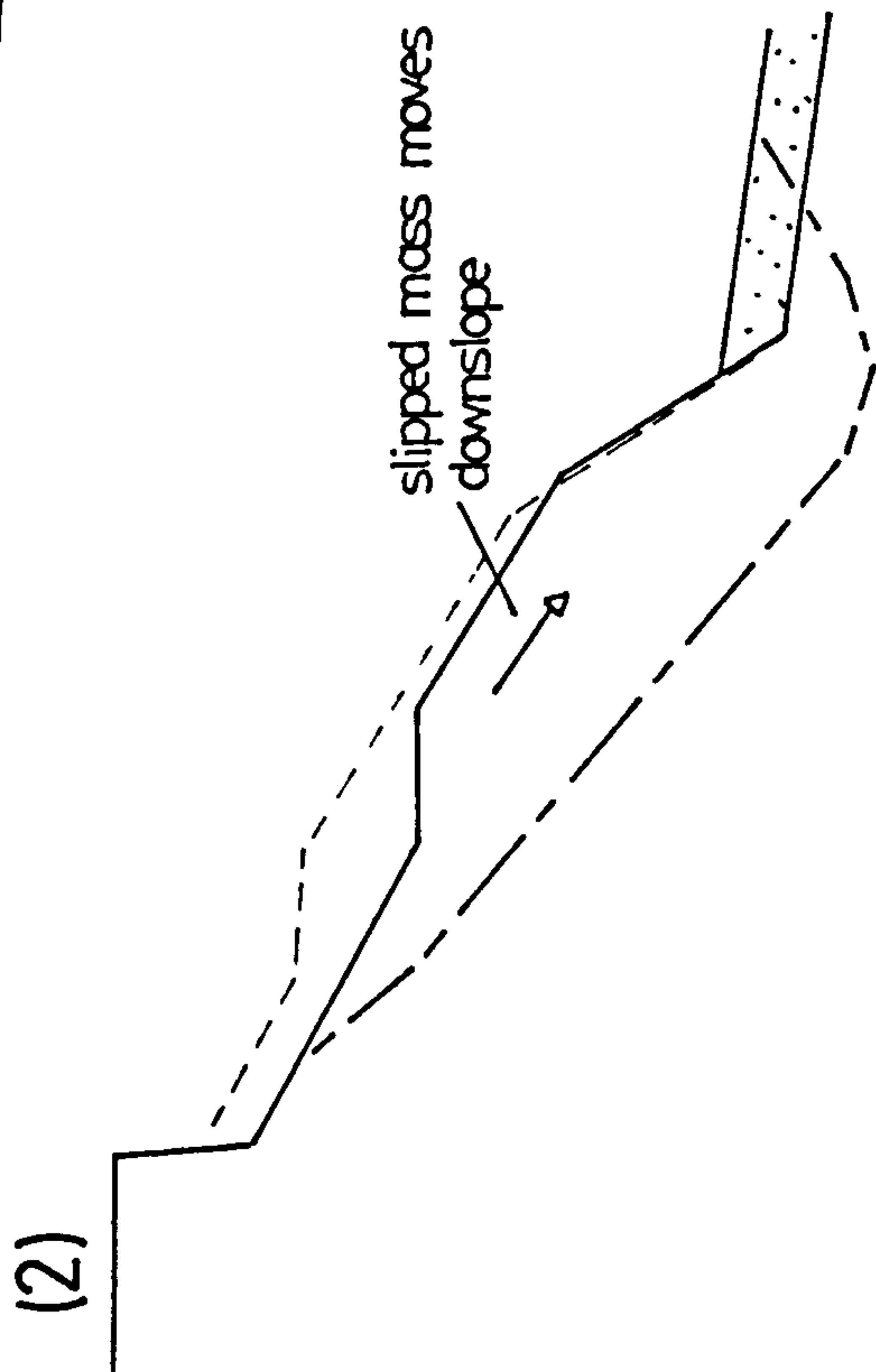
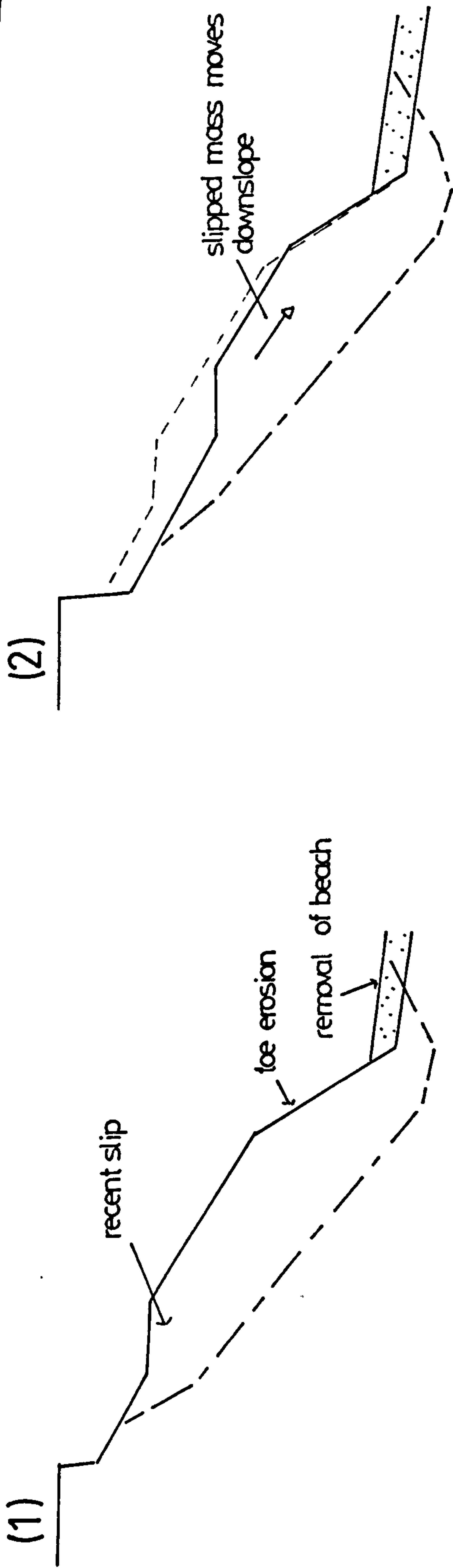
The effect of seasonal changes in pore water pressure on factor of safety were not analysed. However as these are small (see Section 4.3.1), the change in r_u would be insignificant compared with the range of r_u values which were analysed. It can be expected that the seasonal increase in pore water pressure head in late Winter would result in a small decrease in factor of safety.

The significance of the findings of the stability analyses described in this Section are discussed with reference to the cycle of failure at Grimston in the next Section.

5.5 Cycle of Cliff Recession at Grimston

In this Section a hypothetical cycle of recession for the Grimston site is outlined, based on the findings of the site investigation and the earlier sections of this chapter, and from study of other failures in zone "B" at different stages in the cycle. It is then compared with recession of cliffs on the Great Lakes shoreline. Most of the changes described will also apply to other areas in zone "B", and some will also apply to other zones.

The cycle is described with the help of the cliff profiles in Figure 5.32. The first profile (1) shows the situation just after a slip has occurred. The slipped soil mass is slowly moving downslope (at the rate of a few cm/day) and the factor



previous profile dashed

Figure 5:32
Series of cliff profiles showing cycle
of recession at the Grimston site
Scale 1:400

of safety is near unity. The movement may be accelerated by toe erosion, removal of the beach and high winter pore water pressures, which together would decrease the factor of safety by about 25%.

The slipped mass continues moving until profile (2) is reached. The change in profile will cause some stress relief adjacent to the cliff, and reduction in pore water pressure and deformation of the ground associated with it. As the depression in pore water pressure begins to dissipate, softening will occur. The factor of safety is greater than unity when a high beach is present, and further downslope movement only occurs when the beach is removed by storms or "ord" conditions. The factor of safety may again be reduced by about 25% under these conditions, and fall below unity. Meanwhile the slipped soil mass is degraded by mudslides, falls and direct marine erosion. Small slips may occur from the rear scarp, particularly during the high groundwater level conditions of winter.

After considerable time the slipped soil mass is entirely removed by erosion, exposing the old slip surface, and the overall slope inclination is about 40° (see profile (3)). The factor of safety is greater than unity. Mudslides continue to degrade the surface of the slope, and marine erosion at the toe steepens the overall slope angle. The change in profile from both these processes may cause a small amount of stress relief.

The slope finally reaches profile (4), with an overall inclination of about 48° , and now the factor of safety is marginally above unity. Under poor conditions (discussed below) a fresh slip may occur, probably in an undrained manner, displacing by 2-3m until equilibrium is restored. The cycle is then complete. The average time between each stage is about 12 months, giving an overall cycle time of four years. The photograph in Figure 3.30 of a section of cliff in zone "B" shows several of the stages in the cycle of recession.

The stress changes and deformations caused by the changing cliff profile may occur up to a distance of approximately twice the slope height behind the cliff edge. Thus an element of soil may undergo many increments of unloading from when it is first affected by stress relief until it fails. Indeed most of the stress relief on a potential slip surface probably occurs prior to the actual cycle of failure. The till will be softened in the long-term by this stress relief and associated changes in effective stress, and this will reduce the stability of the cliff. However it is thought that softening will rarely trigger failure of the cliff, and that the most common causes of failure are erosion of the cliff toe and reduction in beach level. This is because softening is a slow process compared with toe erosion and beach level changes. The latter two can cause a large reduction in factor of safety in a few days by loss of toe support and shear resistance of the sand. Increases in pore water pressure in the upper part of the cliff

due to seasonal variations may also trigger failure if the factor of safety of the slip is already close to unity.

Any one of the three phenomena of lowering of the beach, toe erosion and seasonal pore water pressure increases may be enough to cause failure, although they frequently occur in combination. During the winter beach levels are likely to be low, and consequently toe erosion more severe. Pore water pressure will also be high, because of the seasonal rise (particularly in late winter). It would therefore be expected that most failures occur during winter, and in the limited number of failures observed at the sites and in the general survey, this was the case.

The cycle of failure described above is now compared with the behaviour of cliffs on the northern shore of Lake Erie (Quigley et al., 1977). The cliffs at Iona which are composed of water laid till overlying lodgement till are undergoing a similar cycle. The slope fails by rotational sliding at an angle of about 40° , and at the end of slide movement this is reduced to 25° . The cliff is then gradually steepened by toe erosion until a new failure occurs. The cycle time is 10-25 years, which is longer than for Holderness. This difference is partly caused by fluctuating lake levels, as toe erosion is reduced during periods of low lake levels.

At a site known as the Geography Field Station near Port Bruce there is a contrasting cycle of failure. The cliffs consist of

lacustrine clayey silts overlying lodgement till. Normally the cliffs retreat by shallow sliding of the upper 12m and toe erosion. However in 1967, following a period of low lake levels a large deep-seated landslide occurred, with a vertical displacement of 1m, increasing to 4m or 5m in 1976. The mechanism of failure is probably softening of the till by a decrease in effective stress as depressed pore water pressures dissipate. During periods of high lake levels this zone of depressed pore water pressure retreats with the cliff, as the rate of erosion is high, and it does not have time to dissipate. Thus deep-seated failures of the cliff do not occur.

It is thought that depressed pore water pressures do not have such a major role in the retreat of the Holderness cliffs and that softening will occur. However they may help to improve stability for short periods (a few months). Where unloading by mudslides, falls and changes in slope profile is frequent, it is possible a near permanent zone of depressed pore water pressure could be formed, although the magnitude of the depression would be small.

CHAPTER 6

Conclusions and Recommendations

In this chapter the findings of the previous three chapters are briefly summarized. The relevance of the work to a coast protection scheme, problems which may be encountered during its construction and useful methods of site investigation are discussed. Suggestions for further research are made, and finally implications for other studies and engineering practice are briefly discussed.

6.1 Conclusions

In Chapter Three it was shown that the most important controls on the mode of failure and cliff morphology are the cliff height, presence of minor soil types and beach conditions. Falls, mudslides and slips all occur, but on any particular stretch of coast one mode of failure is usually dominant over the others. The coast was divided into five major zones, each with a relatively uniform mode of failure and cliff morphology. The difference between the two most common zones "A" and "B", is caused by increasing cliff height. Over about 12m height, slips replace falls as the primary mode of failure. There are three types of slip: slope, toe and base failures, with single rotational toe failures the most common. Slope and base

failures occur in areas where there is some geological control. In zone "C" a "stone-free" silty clay layer near the cliff foot causes base failures to occur because of its low residual shear strength. The sole of the slip passes through the clay layer, and the slips are often multiple rotational or complex in type.

In Zone "D" slope failures occur in thick deposits of sand. These may consist of undercutting by seepage erosion, or instability of the clay till overlying a sand deposit, with the slip surface emerging in the sand layer. Falls are encouraged in all zones by the marine erosion of less resistant sand, silt and "red clay" layers.

In Chapters Four and Five the behaviour of the cliffs at two sites in Zone "B" (where slips are predominant) was studied in detail. The behaviour of the cliffs is predominantly drained, as the rate of unloading was less than the rate of swelling, and excess pore water pressure could dissipate quickly. However first time failures of the cliff probably occur in an undrained manner. The pore water pressure distribution therefore results chiefly from a steady seepage flow pattern. This is strongly influenced by underdrainage to the Chalk aquifer underlying the tills, and particularly at Rolston, to sand layers within the tills. As a result pore water pressures within the slope are low, allowing it to reach a steep angle ($45-50^{\circ}$) before deep-seated failure occurs.

Stability analyses of both first-time failures and slides on pre-existing shears gave factors of safety of less than 1. This may be because the pore water pressure on the slip surface is over-estimated by the use of flow nets, or because pore water pressure is slightly depressed due to unloading of the slope face. Frequent unloading by falls, mudslides and slip movement occurs, and would probably be sufficient to cause the necessary depression.

Ground movements associated with stress relief occur up to 10m behind the cliff edge. This is consistent with results of resistivity surveying and in situ stress measurements adjacent to the cliff.

A more realistic factor of safety is obtained with the use of shear strength parameters from laboratory direct shear tests below the cliff foot. The stress path followed to failure may affect the mobilized shear strength, although the results of tests carried out to model the field stress path were inconclusive.

The stability analyses provide evidence that both the pore water pressure distribution and mobilized shear strength parameters predicted are broadly correct, although the predictions could be improved by future work (see Section 6.3.1). First-time failure of the cliff by deep seated slipping may be triggered by a seasonal rise in pore water pressure, lowering of beach level or toe erosion. However

softening of the tills owing to stress relief and consequent reduction in effective stress is also important in reducing stability. Following from the above, the effect of "ords" and winter storms in lowering beach level and allowing erosion of the foreshore and cliff toe is important in the cycle of failure. Without the opportunity for a period of intense erosion, deep-seated slip activity ceases (as in Zone "E"). The cliff then becomes degraded by mudslides and shallow slips and the slope angle is reduced, ultimately to between 20° and 30° where the cliff toe is permanently protected from erosion.

6.2 Relevance of the Study to a Coast Protection Scheme

The main requirements of any coast protection scheme, as discussed in Chapter One, are to prevent erosion of the till and ensure stability of the cliff. In the scheme currently proposed it is planned to dissipate most of the wave energy before it reaches the cliffs. However some energy will still arrive in the nearshore zone (particularly during storms) and therefore additional protection may be required. This study has shown that for a high long-term rate of recession, periods of severe erosion of the cliff foot and till shore platform are necessary, which can only occur if the beach is removed. Consequently if a high beach can be maintained, it may be all the protection that is necessary, provided it is acceptable that the cliff continues to recede at a very slow rate. However the act of preventing erosion of the cliffs will

deprive the beach of its sediment source, and cause depletion of it. "Ords" may also continue to lower the beach. To overcome this it may be necessary to artificially replenish the beach or prevent sediment transport to some way (eg. by construction of groynes or "headlands").

In some areas it may be desired to halt recession entirely, eg. where there is property adjacent to the clifftop, or at the "headlands" mentioned above. Additional protection of the cliff toe and foreshore would then be required, to guard against storm surge conditions, or occasional removal of the beach. A feasibility study by the Victoria University of Manchester (1983, see Section 1.3) proposed using revetment structures to absorb wave energy, with flat slopes to minimize wave reflection. One proposal is shown in Figure 6.1. Here a porous sloping timber revetment is constructed at the cliff foot, and filled behind with stacked tyres and crushed hardcore. The foreshore in front of the revetment and cliff above it could be given some protection using a geotextile mattress if necessary, anchored to the till bed. The cliff above the revetment would require regrading to a stable angle, probably between 25 and 30°.

Problems which could be encountered in such a scheme are:

1. degradation of the cliff face by shallow slips and mudslides, activated by water from surface run off, field drains and seepages;

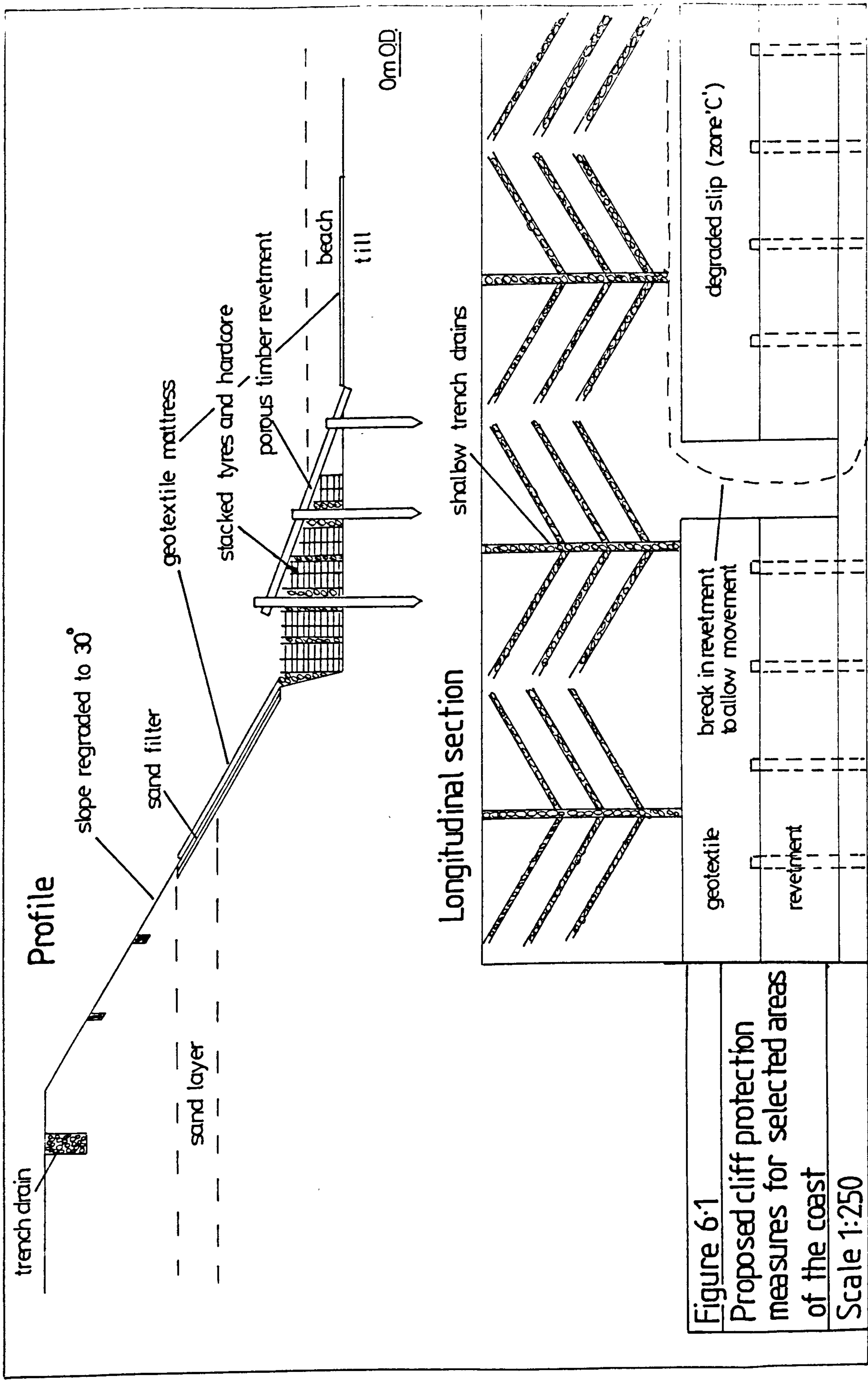


Figure 6.1
Proposed cliff protection
measures for selected areas
of the coast
Scale 1:250

2. instability and continued movement of the cliff, which may damage the revetment;
3. continued marine or seepage erosion in areas where the cliff is formed of erosion susceptible material, eg. large sand layers;
4. erosion of the till foreshore if it was exposed during a period of low beach levels, leading to undermining of the revetment.

The first problem could be remedied by using shallow herringbone pattern gravel filled trench drains on the slope face to collect surface run off and seepages. Field drains could be intercepted in a gravel filled trench drain running parallel to the cliff top, which would also limit near surface high seasonal pore water pressures and prevent shallow slips in the weathered till. A trial trench of this type is currently being monitored by the University of Hull. Seepage erosion could be prevented by the use of a geotextile or graded sand filter placed over sand and silt layers in the cliff face (Hutchinson, 1983). Deep drainage would not be effective in stabilizing the slopes, because the seepage flow pattern is already underdrained.

The second problem could arise where the cliff is fronted by long and wide degraded slipped soil masses, as in Zone "C" areas. These would be likely to continue moving, potentially

damaging to the revetment. It would not be practical to remove them, because to do so would probably cause renewed failure of the cliff by reduction of toe support. Damage could be avoided by ensuring the revetment is sited to landward of the position of the slip toe, which in typical base failures is greater than 10m from the cliff foot. A break in the revetment would have to be left at the boundaries of individual slips, in case movement did occur. In Zone "D" (ie. areas where a high proportion of sand occurs in the cliff), additional protection behind the revetment could be required. A filter with some form of facing, eg. stone rip-rap, could be used. The design slope angle may need to be adjusted, to take account of the change in cliff materials.

Erosion of the till foreshore could be prevented for a short time by the use of a geotextile. However the only feasible long-term protection is by a beach over about 1m thick.

6.2.1 Useful Site Investigation Methods

This section suggests methods of site investigation (used in this study) which could be useful in construction of revetment works as proposed above. Sand layers in the cliff are often obscured by fall or flow debris, so inspection of the cliff will not reveal their true extent. For small layers it may be possible to deal with them as they are exposed during regrading of the cliff, but the full extent of larger layers, which may

require a change of design, must be known prior to construction. The static cone penetrometer test was found useful in detecting sand layers, and a lightweight rig could be used to test near the cliff edge. The geomorphology of the clifftop and the increase in seepage from sand layers may also help to locate them.

It is difficult to determine the extent of "stone-free" clay layers which occur near the cliff foot, as they are often hidden by the beach. However their presence may be suspected from the unusual morphology which develops (see Section 3.2.2) and investigated by a borehole with continuous sampling around the level of the cliff foot. They will be most likely to occur in Zone "C" of the general survey. The extent of individual slips may be found by geomorphological mapping. The thickness of the beach overlying the till foreshore can be determined using shallow seismic refraction surveys, although it should be referred to a level on the clifftop, because it is constantly changing.

The "standard" effective stress strength parameters of the tills are well known from this and other investigations. For a regraded slope which has not previously failed, strength parameters from "standard" triaxial tests should be used above the cliff foot, and "direct shear" strength parameters below it. If the slope has previously failed then residual strength parameters from ring shear tests should be used. It may be necessary to check that the groundwater flow pattern is

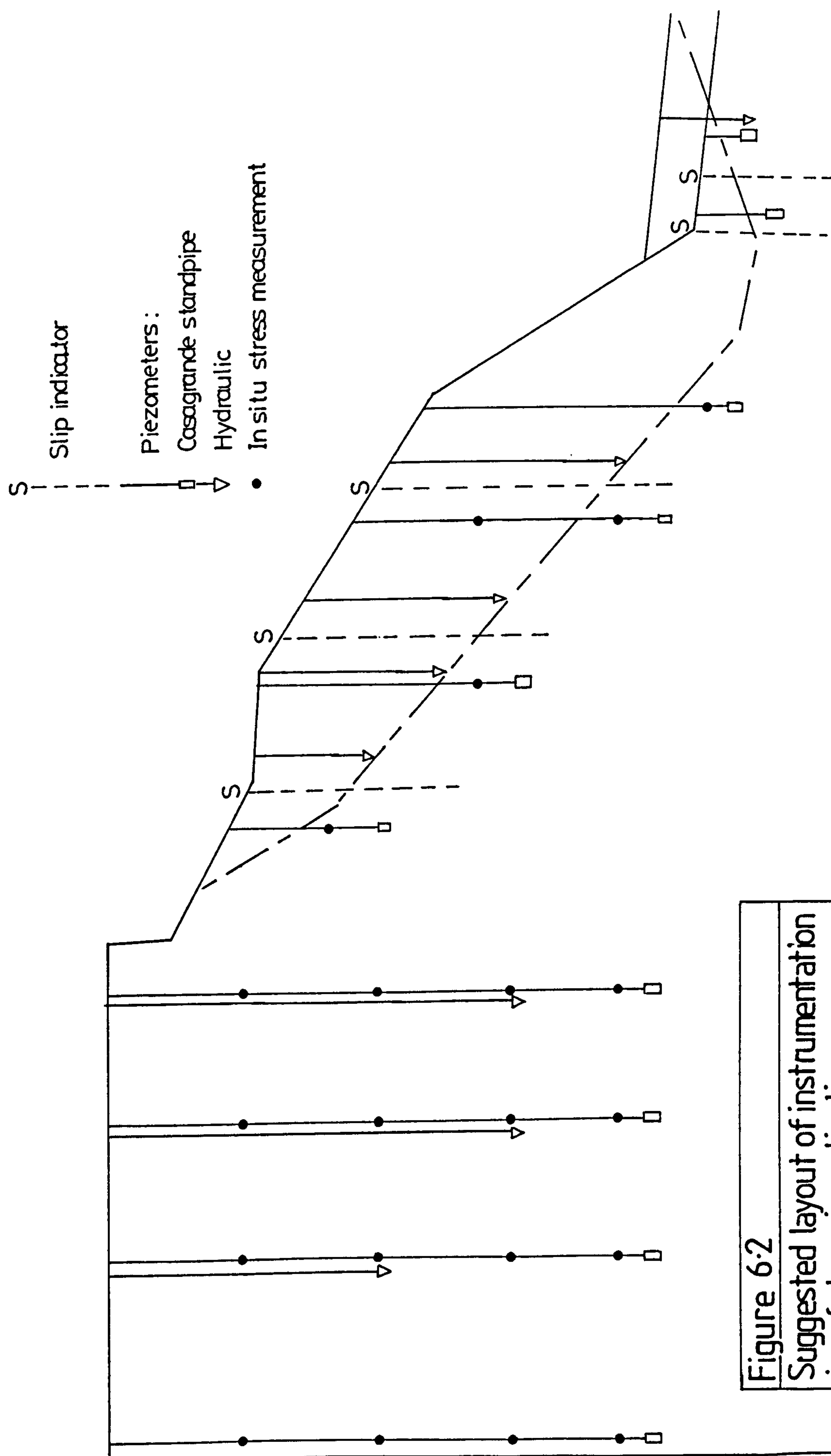
underdrained in all areas of the coast, particularly in northern Holderness where high groundwater levels in the Chalk may occur. This could be done with piezometers, some of which should be placed at least as deep as the cliff foot. In the shallow weathered zone, it can be assumed for design that the water table is at ground level during winter, unless drainage is installed.

6.3 Suggestions for Further Research

6.3.1 Research at the Existing (or Similar) Sites

The discussion of Chapter Five showed that knowledge of all the important factors affecting slope stability is uncertain, necessitating extrapolation and prediction of values for use in analysis. Knowledge of these factors could be improved in future investigations by using a different layout of instrumentation. This is now discussed for each of the factors, and a suggested layout shown on Figure 6.2.

The pore water distribution could be further investigated by installing piezometers close to the slip surface, to check the predictions of this study, and to measure changes in pore water pressure caused by unloading and swelling. This would require drilling from the surface of a slipped soil mass, which could be done with a mobile drilling rig (eg. a "Minuteman") and lifting equipment. The top of the piezometers would be



vulnerable to burial by falls, and shearing of the tubes could occur as the slip continues to move (although this would help to locate the slip surface). To minimize losses, the instrumentation could be installed in two phases. In the first phase a few Casagrande standpipe piezometers could be installed, including some below the suspected position of the slip surface. After these were sheared by continued movement, more piezometers could be installed close to but above the now known position of it. These could include some hydraulic piezometers, with a faster response time than Casagrande piezometers, to observe unloading or swelling effects, and capable of measuring negative pore water pressures if present. Readings would have to be taken frequently (eg. at daily or weekly intervals) or an automatic recording system used.

The best time for the instrumentation programme to commence would be at the end of April, to avoid damage by falls and mudslides. More deep piezometers behind the cliff would be desirable to confirm the underdrained flow pattern, and also some piezometers in the foreshore to determine if suctions are present there. Difficulties may be encountered with installation in the foreshore as the till is often covered by 1-2m of sand and high tides frequently reach the cliff foot. Push in piezometers are difficult to drive in the stiff clay and have a slow response time. The most successful approach will be to wait for an opportunity, when sand levels are low and during neap tides, to install narrow diameter standpipe or hydraulic piezometers using a mobile drilling rig. The fast

response time of these piezometers may allow equilibrium to be reached before they are buried by the returning beach.

The shape of the slip surface is uncertain except at the head and toe. It is necessary to have a better knowledge of it, to determine where "direct shear" parameters are applicable, and because it may influence the calculated factor of safety.

Its shape may be better defined using slip indicators and piezometers as described above. The variation in shape from the central profile (towards the end of the slips) should also be explored, to assess the influence of end effects on factor of safety.

Stress changes behind the cliff and in the slipped block could be investigated at the sites using push in spade type total pressure cells, as have been successfully used at Cowden by the Building Research Establishment. It would be desirable to measure in situ stresses at various depths to the level of the cliff foot, near to and remote from the cliff, to improve knowledge of the actual stress path followed by the soil to failure. The influence of stress path on the strength of a sample also needs to be further investigated. The field measurements of in situ stress could be used to guide a laboratory test programme.

Improved knowledge of the soil permeability would be useful if more advanced analyses of seepage and consolidation were to be

made. The effects of anisotropy, particularly of specimens which showed a layered fabric (see Section 4.2.4), could be investigated in the laboratory using specimens cut horizontally from U100 samples. Constant head permeability tests could be conducted in the piezometers, for comparison with the falling head tests, and would be more appropriate for analysis of steady seepage. Better definition of the distribution and continuity of more permeable layers could be achieved using static cone penetration tests, combined with continuous borehole sampling in the region of suspected layers.

The in situ stress changes and pore water pressure distribution and changes, due to unloading and steady seepage flow, could most satisfactorily be modelled using numerical methods. This would provide an improved theoretical basis with which to compare field observations. With better knowledge of some factors, eg. the pore water pressure distribution and slip geometry, back-analysis would enable the shear strength parameters mobilised in the field to be more closely defined. These could then be used to check the validity of laboratory test results and methods of analysis.

6.3.2 Other Sites

As mentioned in Chapter Four, both of the sites investigated are in Zone "B", where single rotational slips are the predominant mode of failure. It would be desirable to

investigate other zones in more detail, to contrast with this study, and because Zone "B" forms only 25% of the coast. In Zone "C" the geometry of the slips, pore water pressure distribution and stress-strain and strength characteristics of the "stone-free" silty clay (which contains part of the slip surface), are all different from Zone "B", although the same approach of site investigation could be used. In Zones "A" and "D" where falls are more common, a different approach is necessary. This should place less emphasis on instrumentation and more on detailed surveying of the cliff changes at a small number of sites. Zone "E" where active erosion has nearly ceased, is probably not worth investigating in detail.

6.3.3 The General Survey

The general survey covered 28km of the coast between Skipsea and Withernsea, but omitted a considerable distance (20km) south of Withernsea. It would be worthwhile to extend the survey to this section of the coast because of the greater cliff height (30m) at Dimlington. It would also be possible to study the changes in cliff morphology over a longer timescale. This could be done by examination of old aerial photographs of the coast, or by carrying out a repeat of this survey after, for example, 10 years have elapsed.

6.4 Relevance of the Work to Other Studies and Engineering Practice

The study of cliff recession in tills described in this thesis can be contrasted with studies in other cohesive materials. One major difference between the recession of the Holderness till cliffs and, for example London Clay cliffs, is the shorter time required for dissipation of excess pore water pressure in the former. The depression of pore water pressure in the Holderness cliffs due to unloading is relatively small and short-lived when compared with the London Clay cliffs. This is a reminder that little reliance should be placed on undrained pore water suctions to support the sides of temporary excavations, without careful investigation of the permeability of the ground. The regional underdrainage to the Chalk aquifer and to sand layers within the till, allows the cliff to stand at steep angles ($40-50^{\circ}$). Underdrainage could be a control on other coastal or inland slopes in cohesive materials, where more permeable beds occur within the slope, or where it is underlain by an aquifer.

The study of cliff stability has shown the importance of selecting shear strength parameters appropriate to the stress path followed, and the mode of shear. If "standard" strength parameters are used where "direct shear" parameters are applicable, the "apparent" factor of safety may be lower than the "true" factor of safety. In some situations the factor of safety may be largely on the "safe" side, resulting in an

uneconomic design. In back-analysis of slope failures to determine mobilized strength parameters, it may be necessary to take account of the above when interpreting the results.

Finally, the study has shown the large influence on mode of failure of soil types forming only a small percentage of the cliff by volume. These generally occur as layers of a different geological origin, and consequently with different geotechnical properties. The value of knowing the geological history of materials forming an excavation or slope, as a guide for assessing their likely geotechnical properties, is clear. Although the distribution of unusual soil types cannot be predicted exactly, their presence may be anticipated and searched for at the site investigation stage.

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APPENDIX A

DESCRIPTION OF SPSSX SYSTEM, VARIABLES AND DATASET

A.1 The SPSSX Batch System

The SPSSX batch system is a computer program for the statistical analysis of data. It has a range of facilities for data manipulation and analysis, including plotting and sorting. A set of data for analysis is referred to as a dataset, which consists of cases and variables. A case is the basic unit of analysis; in the survey it is one photograph of a 50m length of cliff. The variables are things which are measured and recorded for each case, eg. cliff height, percentage of the cliff by area of a particular cliff morphology, or state of the beach, and all take some numeric value. The variables are defined below. Each case is given a number increasing from north to south along the coast.

A.2 Definition of Variables used in the General Survey

A.2.1 Cliff Morphologies

The cliff morphology was classified into four types described below. At any place it may be solely of one type, or a combination of two or more. The proportions of types in a

photograph are expressed as a percentage by area of the whole cliff face, and this is the value of the variable for SPSSX analysis. The four types of morphology and other features are sketched in Figure A3.1.

"TILL CLIFF" - An area of cliff where slips have not occurred, and where the surface is free from mudslide or fall debris. The slope angle is generally steep (mean 48°), reaching 70° on low cliffs.

"DEGRADED CLIFF" - An area of cliff where the undisturbed till surface is buried by fall or mudslide debris to a depth of several metres. The slope is usually low angled (mean 36°).

"RECENT SLIP" - An area of cliff where a landslip has occurred, but where the slipped soil mass has not been displaced more than a few metres, or been degraded by falls and mudslides. Most recent slips probably occurred within one year prior to the survey.

"DEGRADED SLIP" - As for the recent slip, but where the slipped soil mass has been degraded by falls and mudslides, and continued movement has displaced it several metres. Most degraded slips probably occurred between one and five years before the survey, but in very actively eroding areas may have occurred only a few months before.

"TOTAL SLIPS" - The sum of recent and degraded slips.

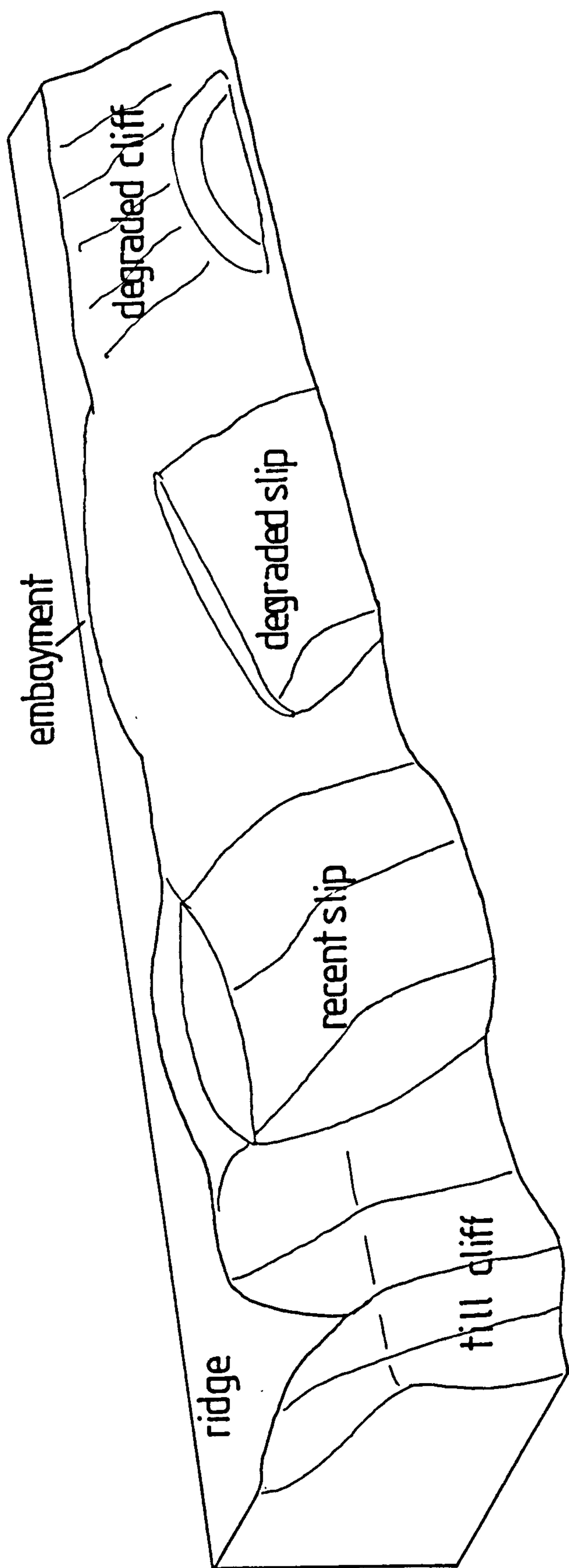


Figure A3.1 Appendix A
Definition of four major cliff morphologies

"RIDGE" - A ridge or small headland of "till cliff", triangular in plan, typically of 5m width at the base and protruding 3-4m from the general cliff line.

"RIDGE INDEX" - "Ridge Index" has a value of 1 if a ridge is present in a photograph, and 0 if none are present.

"EMBAYMENT" - A small bowl-shaped embayment, generally with degraded cliff or slip in the centre, often occurring between two ridges.

A.2.2 Other Variables

"CLIFF HEIGHT" - The vertical distance (m) between the cliff top and the level of the beach sand at the foot of the cliff. The true cliff foot is typically 1-2m lower.

"GROUNDWATER INDEX" - An approximate visual assessment of the amount of groundwater seeping from the cliff face. It has a value of 0 for no seepage, 1 for small or moderate amounts of seepage, and 2 for large amounts of seepage.

"SAND" - The percentage by area of the cliff face formed by sand deposits.

"RATE OF RECESSION" - The rate of recession of the clifftop (m/yr) calculated from data supplied by Holderness Borough Council, for the period 1974 to 1985.

"LENGTH OF LANDSLIP" - The length of a landslip (m) measured from the photographs at mid-height on the cliff.

APPENDIX A

A-3 Listing of dataset

MEANING OF VALUES

VARIABLE	UNIT	DESCRIPTION
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Position of emergence of slip toe	1 in slope face	9 no slips present
	2 at cliff foot	
	3 in foreshore	

Groundwater index	0 no seepage
	1 small to moderate seepage
Ridge index	2 large seepage

Ridge index

0	no ridges
1	ridge present

State of beach

0 no beach (till exposed)

1 normal beach

2 high and wide beach

Others	value in units indicated (see A-2 for definition)
--------	--

-1 missing value

[illegible]

Groundwater index Q - 00000110000000000011000000100000001111000000

Recession rate (m/yr)	RPE
0.7	0.7
0.7	0.7
0.7	0.7
0.9	0.9
0.9	0.9

Length of landslip(m)	LLS	-1	-1	11	13	-1	-1	-1	11	8	-1	-1	-1	-1	-1	20	16	7	-1	-1	9	-1	13	-1	13	-1	-1	-1	10	11	9	5	-1	16	-1	20	10	-1	9	19	-1
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[illegible]

Ridgeindex A 1000011010000000000101011000001010110001

[illegible]

Degraded cliff (%)	DC	99	99	74	81	99	00	00	00	00	66	99	26	00	00	00	22	00	06	87	36	46	90	80	77	94	80	99	25	99	21	60	09	15	73	00	40	54	73	99	00	65	49
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Till cliff (%)	TC	00	00	00	00	00	99	99	82	00	00	74	99	99	78	65	44	00	64	54	00	20	00	06	00	00	75	00	73	00	59	80	27	54	60	40	00	00	72	00	51
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[illegible]

Recent slips (%)	RS	00	00	26	19	00	00	00	00	18	34	00	00	00	00	00	00	07	50	13	00	00	10	00	00	23	20	00	00	00	06	40	32	05	00	46	00	06	27	00	28	35	00
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[illegible]

Cliff height (m)	CH
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10	10
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10	10
9	9
8	8
6	6
5	5
4	4
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12	12

315	311	305	301	297	292	288	283	278	274	269	265	260	255	249	244	240	235	231	227	221	216	211	207	202	198	195	191	188	183	178	174	169	165	160	155	149	144	140	135	131	127	121	116	111	107	102	98	95	91	88	83	78	74	69	65	60	55	54	49	44	40	35
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Grid reference (TA)	GR
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182225	
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182854	
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187853	
188153	
188253	

Case number	ID
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ID	GR	CH	SB	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PDTOE
41	18835330	11	0	00	15	85	00	03	1	30	13		0	9
42	18865325	10	0	00	31	69	00	00	1	-1	20		0	9
43	18875323	11	0	21	49	00	30	00	0	-1	30		0	1
44	18885318	10	0	31	00	69	00	00	0	-1	42		0	1
45	18895313	10	0	13	00	63	24	00	1	-1	10		1	3
46	18915309	10	0	00	00	10	90	02	0	-1	-1		1	9
47	18935304	10	0	32	00	68	00	00	2	-1	16		0	3
48	18955300	10	0	33	00	00	67	00	0	-1	20		0	2
49	18975296	11	0	00	00	64	36	00	2	75	-1		0	9
50	18985291	12	0	00	00	00	99	02	0	99	-1	0.4	0	9
51	19005288	11	0	05	00	90	05	00	1	30	9		0	3
52	19025284	12	0	00	00	68	32	02	1	50	-1		0	9
53	19035280	09	0	00	00	81	19	04	1	50	-1		1	9
54	19045275	10	0	00	00	27	73	00	2	55	-1		1	9
55	19065270	11	0	00	00	84	16	00	2	-1	-1		0	9
56	19075266	09	0	13	00	00	87	04	0	99	16		0	3
57	19085261	09	0	08	00	52	40	00	1	45	04		0	3
58	19105257	10	0	00	10	86	04	00	2	-1	10		0	3
59	19115253	09	0	00	00	99	00	00	0	60	-1		0	9
60	19125249	09	0	05	26	69	00	00	1	42	15	0.9	0	9
61	19145244	09	0	00	27	63	10	00	1	-1	25		0	1
62	19165239	09	0	00	25	00	75	00	0	-1	12		0	1
63	19175234	10	0	00	00	97	03	00	1	20	-1		0	9
64	19175233	11	0	00	19	81	00	00	1	75	60		0	1
65	19195228	13	0	00	30	60	10	00	1	40	-1		0	9
66	19215224	14	0	00	48	22	30	03	1	45	21		0	1
67	19225219	11	0	00	25	13	62	00	1	40	28		0	1
68	19235215	10	0	00	82	00	18	00	0	-1	-1		0	1
69	19255210	09	0	05	32	40	23	00	1	99	15		0	9
70	19275205	10	0	00	00	99	00	00	1	25	-1	0.6	0	9
71	19295200	09	0	00	00	99	00	00	1	20	-1		1	9
72	19315196	09	1	00	99	00	00	00	0	-1	30		0	1
73	19325191	09	1	00	99	00	00	00	0	-1	30		1	1
74	19335186	08	1	00	00	10	90	03	1	-1	-1		1	9
75	19355183	08	1	00	00	09	00	91	00	0	-1		0	9
76	19365178	10	1	00	00	50	50	01	2	-1	-1		0	9
77	19395174	11	1	00	28	00	72	00	0	99	26		0	1
78	19405168	10	1	00	12	29	59	00	1	42	-1		0	9
79	19425165	11	1	00	00	15	85	02	1	42	-1		0	9
80	19455159	11	1	19	11	15	55	00	1	60	17	0.8	1	1
81	19465155	10	1	00	00	19	81	00	1	30	-1		1	9
82	19475150	11	1	00	00	00	99	06	1	80	-1		2	9
83	19495146	09	1	00	20	33	47	00	2	-1	-1		0	9
84	19525141	10	1	00	00	29	71	04	0	-1	-1		0	9
85	19535137	10	1	00	34	00	66	00	0	-1	20		1	9
86	19545132	11	1	33	00	00	67	00	0	-1	15		2	2
87	19575127	12	1	08	00	92	00	00	1	-1	-1		0	9
88	19585123	11	1	00	00	52	48	00	0	-1	-1		1	9
89	19615118	10	1	17	00	68	15	21	0	-1	10		1	2
90	19635113	10	1	00	00	80	20	21	0	-1	-1	0.4	2	9
91	19645108	10	1	17	00	70	13	20	0	-1	10		1	1
92	19665104	10	1	00	00	81	19	08	2	35	-1		2	9
93	19685100	10	1	00	00	40	60	31	2	-1	-1		0	9
94	19695095	09	1	00	33	67	00	24	0	-1	-1		2	9
95	19715090	10	1	00	30	24	46	00	0	-1	-1		0	9

ID	GR	CH	SB	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PDTOE
96	19735080	10	1	00	00	30	70	00	1	40	-1		1	9
97	19745081	09	1	00	00	55	45	00	2	-1	-1		1	9
98	19755076	08	1	00	55	00	45	00	0	-1	25		0	9
99	19785072	08	1	00	55	00	45	00	0	-1	-1		0	1
100	19805068	09	1	00	63	37	00	00	1	-1	-1	0.5	2	9
101	19825063	10	1	00	35	00	65	00	0	-1	40		0	1
102	19845058	10	1	22	22	00	56	00	0	-1	37		0	2
103	19865054	10	1	00	24	28	48	00	1	-1	27		2	9
104	19885049	11	1	00	29	26	45	00	0	75	22		1	9
105	19895045	12	1	00	16	41	43	00	1	45	-1		1	9
106	19915039	12	1	00	00	76	24	00	2	-1	-1		1	9
107	19935035	12	1	20	36	00	44	00	0	-1	45		1	2
108	19965030	13	1	00	25	00	75	00	0	-1	40		0	2
109	19965025	11	1	00	10	64	26	00	0	-1	-1		0	9
110	19975021	11	1	00	38	00	62	00	1	-1	25	1.6	1	2
111	19995016	10	1	00	39	00	61	00	0	50	40		1	0
112	20015011	10	2	00	00	99	00	00	1	-1	-1		0	9
113	20025006	10	2	00	19	64	27	20	2	-1	18		0	9
114	20035002	10	2	00	51	00	49	06	0	-1	-1		0	9
115	20054997	10	2	00	46	00	54	00	0	-1	-1		1	9
116	20074993	10	2	00	24	20	56	00	1	70	40		1	1
117	20084988	11	2	00	34	00	66	00	1	-1	17		1	1
118	20104983	13	2	00	27	32	41	00	0	55	27		0	9
119	20124978	12	2	00	36	32	32	00	1	-1	44		0	1
120	20134974	11	2	00	38	00	62	00	0	-1	25	1.6	1	1
121	20174970	10	2	00	00	55	45	01	0	-1	-1		0	9
122	20184965	11	2	00	33	00	67	00	0	-1	33		0	1
123	20204960	10	2	00	00	25	75	00	1	-1	-1		0	9
124	20234956	11	2	00	55	00	45	00	0	55	35		0	1
125	20254952	11	2	00	29	00	71	00	0	-1	26		1	9
126	20274947	11	2	00	25	00	75	00	0	-1	-1		1	9
127	20294943	10	2	00	45	00	55	00	0	-1	65		0	9
128	20324938	10	2	10	15	00	75	00	0	-1	-1		0	9
129	20364934	10	2	00	37	17	46	00	0	-1	30		0	9
130	20374929	10	2	00	39	00	61	00	0	-1	30	0.7	1	9
131	20404925	10	2	00	28	48	24	00	0	-1	-1		1	9
132	20424921	09	2	00	00	40	60	04	0	-1	-1		0	9
133	20444917	09	2	00	00	00	99	00	0	-1	-1		0	9
134	20464912	08	2	00	59	41	00	00	0	-1	35		0	9
135	20484907	08	2	00	31	47	22	00	0	-1	-1		0	9
136	20504902	09	2	00	00	00	99	00	0	-1	-1		1	9
137	20524897	09	2	00	00	00	99	00	0	-1	-1		1	9
138	20534893	08	2	00	00	20	80	00	0	-1	-1		1	9
139	20564888	09	2	00	00	51	49	00	0	-1	-1		1	9
140	20574884	10	2	00	00	15	85	00	0	-1	-1	0.7	0	9
141	20584879	09	2	00	00	00	99	00	0	-1	-1		0	9
142	20604875	08	2	00	00	00	99	00	0	-1	-1		0	9
143	21224688	12	1	00	19	54	27	00	0	-1	17		0	9
144	21244684	12	1	00	27	73	00	00	0	-1	20		0	9
145	21264678	14	0	00	26	74	00	00	0	-1	23		0	9
146	21274674	14	0	17	24	59	00	00	0	-1	10		0	2
147	21304670	13	0	38	11	51	00	00	0	-1	21		0	2
148	21324666	12	0	64	00	36	00	00	0	-1	50		0	2
149	21334661	13	0	00	40	60	00	00	0	-1	10		0	3
150	21364656	14	0	00	31	69	00	00	0	-1	21	2.6	0	2

ID	GR	CH	SB	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PDTOE
151	21374652	13	0	24	00	76	00	00	0	-1	18		0	2
152	21404647	13	0	00	28	72	00	00	0	-1	15		1	9
153	21424644	14	0	00	30	70	00	00	1	-1	22		0	9
154	21444639	14	0	00	80	20	00	00	0	-1	40		0	9
155	21474635	14	0	31	12	57	00	00	0	-1	23		0	2
156	21504631	15	0	45	00	48	07	00	0	-1	20		0	2
157	21524627	14	0	00	30	70	00	00	0	-1	-1		0	2
158	21544623	13	0	00	70	00	30	05	1	-1	30		0	1
159	21564618	14	0	44	00	36	20	05	0	-1	14		0	1
160	21574613	13	0	34	10	54	00	00	0	-1	19	3.7	0	2
161	21584608	12	0	00	50	10	40	00	0	50	40		0	2
162	21624604	12	0	00	00	74	26	03	2	85	-1		0	9
163	21634600	13	0	00	26	00	74	00	0	-1	30		0	2
164	21664595	14	0	00	16	74	10	00	0	-1	17		1	2
165	21684591	13	0	00	31	59	10	02	0	-1	28		1	2
166	21704586	13	0	37	08	55	00	00	2	-1	19		0	2
167	21724582	13	0	23	22	00	55	00	0	-1	11		0	2
168	21744577	13	0	47	00	23	30	00	0	-1	19		1	2
169	21774573	12	0	00	49	51	00	00	0	-1	23		0	2
170	21794569	12	0	00	33	00	67	00	0	-1	-1	2.8	1	9
171	22824566	12	0	00	29	00	71	02	0	-1	13		0	9
172	21824560	12	0	72	00	00	28	00	0	-1	30		0	1
173	21864556	12	0	77	00	00	23	00	0	-1	20		0	2
174	21894553	12	0	00	17	35	48	00	0	35	18		1	9
175	21924549	13	0	00	30	30	40	00	2	30	19		0	2
176	21934544	13	0	00	00	84	16	00	2	25	-1		0	9
177	21964540	13	0	00	39	17	42	00	1	-1	16		0	1
178	21984536	13	0	20	00	41	39	00	0	-1	20		1	2
179	22024533	13	0	16	00	84	00	00	0	-1	10		0	2
180	22044528	12	0	11	25	64	00	05	0	-1	-1	2.4	0	9
181	22064523	13	1	13	00	65	22	00	0	-1	10		0	2
182	22194518	14	1	00	19	61	20	00	0	-1	-1		0	9
183	22124513	13	1	00	00	72	28	02	0	-1	-1		1	9
184	22144519	14	1	51	33	16	00	08	1	-1	10		1	2
185	22174505	14	1	28	00	36	36	00	1	50	35		0	1
186	22184503	14	1	00	00	84	16	00	1	-1	-1		0	9
187	22214500	14	1	25	32	43	00	00	0	-1	18		0	2
188	22234496	14	1	00	47	53	00	00	0	-1	17		0	9
189	22264493	13	1	00	69	31	00	00	0	-1	40		0	3
190	22274488	12	1	00	19	00	81	00	0	-1	-1	2.4	0	2
191	22314484	11	1	00	99	00	00	00	0	-1	50		0	2
192	22324478	11	1	18	17	00	65	00	0	-1	13		0	2
193	22344475	11	1	00	50	50	00	00	0	-1	24		1	2
194	22374470	11	1	00	50	20	30	00	1	55	-1		1	2
195	22394465	11	0	00	74	26	00	00	1	-1	-1		0	2
196	22424462	10	0	00	70	30	00	00	0	-1	30		0	2
197	22444457	10	1	00	60	40	00	00	0	35	18		0	9
198	22464452	10	1	15	31	34	20	00	0	-1	20		0	9
199	22494448	09	1	61	00	39	00	00	0	-1	27		0	2
200	22524444	09	1	00	55	45	00	01	1	40	30	2.9	0	2
201	22544440	09	1	00	47	10	43	00	0	35	25		0	9
202	22574434	11	1	12	00	00	88	18	0	-1	-1		2	9
203	22594430	12	1	40	00	60	00	26	0	-1	25		1	2
204	22614426	11	1	22	33	45	00	00	1	60	16		0	9
205	22644423	11	1	00	00	40	60	00	1	-1	-1		0	9

ID	GR	CH	SB	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PDTOE
206	22604418	11	1	00	15	85	00	22	0	-1	-1		1	9
207	22684413	12	1	18	21	57	00	18	0	-1	-1		1	9
208	22714408	13	1	00	14	73	13	23	0	-1	-1		2	9
209	22734404	13	1	22	00	78	00	07	1	-1	12		0	2
210	22764400	12	1	00	00	82	18	12	1	35	-1	3.3	1	9
211	22784395	14	1	00	00	82	18	00	1	30	-1		0	9
212	22814391	14	1	00	00	40	60	00	1	-1	-1		0	3
213	22834387	15	1	59	00	21	20	02	0	-1	27		1	2
214	22864382	14	1	00	00	80	20	04	0	-1	-1		1	9
215	22884378	14	1	53	00	27	20	00	0	-1	28		0	2
216	22914383	14	1	-1	-1	-1	-1	-1	0	-1	-1		0	9
217	22934369	15	1	00	00	76	24	00	0	-1	-1		0	3
218	22954365	13	1	00	00	82	18	02	2	-1	-1		0	9
219	22994360	14	0	00	00	66	34	00	0	-1	-1		0	9
220	23024356	15	0	43	00	30	27	00	0	-1	26	1.9	0	2
221	23044352	14	0	31	00	69	00	00	1	-1	20		0	2
222	23074348	14	0	00	00	80	20	05	1	-1	-1		0	9
223	23094343	13	0	00	22	78	00	00	2	40	12		0	9
224	23124338	14	0	15	00	42	43	00	1	-1	23		0	3
225	23154334	13	0	37	00	00	63	06	0	-1	11		0	2
226	23184329	14	0	00	22	45	33	00	0	22	22		0	3
227	23214326	14	0	78	00	22	29	02	0	-1	40		0	2
228	23234321	14	0	00	34	49	17	00	0	-1	-1		0	9
229	23274317	13	0	00	32	68	00	00	0	-1	35		0	9
230	23294312	14	0	30	00	39	31	00	0	-1	38	2.7	0	3
231	23314308	12	0	34	00	66	00	00	0	-1	45		0	3
232	23334304	13	1	00	00	50	50	00	2	17	-1		0	9
233	23364300	14	1	24	00	56	20	00	0	-1	15		0	2
234	23384295	13	1	00	43	57	00	00	0	-1	32		0	2
235	23424292	15	1	00	71	29	00	00	0	-1	21		0	9
236	23434287	14	1	28	00	29	43	00	0	-1	15		0	2
237	23464283	14	1	00	28	56	16	00	0	-1	15		1	2
238	23494278	14	1	00	26	28	44	00	1	-1	-1		0	9
239	23514274	13	1	00	31	00	69	00	0	-1	-1		0	9
240	23544270	14	1	00	31	53	16	00	1	-1	14	1.8	1	9
241	23574267	15	1	27	43	30	00	00	0	-1	10		0	2
242	23604263	13	1	00	00	65	35	00	1	55	-1		0	9
243	23624258	14	1	00	29	37	34	00	1	70	13		0	2
244	23644254	14	1	25	25	50	00	00	1	50	13		0	2
245	23684250	15	1	00	19	81	00	00	1	45	-1		1	9
246	23704246	14	1	00	18	82	00	00	1	40	-1		1	9
247	23724245	12	2	00	00	50	50	00	2	31	-1		0	9
248	23744239	11	2	00	57	00	43	11	0	-1	-1		1	9
249	23764234	12	2	00	28	00	72	16	0	-1	-1		1	9
250	23794229	13	2	00	76	00	24	07	0	-1	-1	2.6	0	2
251	23824224	13	2	00	87	00	13	00	0	-1	-1		0	9
252	23864219	12	1	00	56	00	44	05	0	-1	-1		0	9
253	23894215	13	1	00	54	00	46	00	1	-1	-1		0	9
254	23934210	15	1	00	63	37	00	00	1	50	25		0	9
255	23964205	15	1	28	17	55	00	00	2	25	17		0	2
256	23994202	14	0	00	54	00	46	00	0	45	30		0	9
257	24024197	14	1	47	00	43	10	04	1	60	45		0	2
258	24054194	14	0	47	00	34	19	00	0	-1	09		1	2
259	24084189	14	0	00	34	56	00	00	2	-1	17		0	9
260	24114185	13	0	50	00	00	50	00	0	-1	27	2.6	0	2

ID	GR	CH	SB	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PDTOE
261	24124181	13	0	44	24	00	32	00	2	65	16		0	2
262	24164177	14	0	00	39	15	46	00	0	-1	22		0	2
263	24204174	15	0	10	49	21	20	00	0	-1	25		0	2
264	24224169	14	0	00	58	42	00	00	0	-1	-1		0	9
265	24244165	14	0	00	00	90	10	00	0	-1	-1		0	9
266	24274162	15	0	00	00	99	00	10	0	-1	-1		2	9
267	24314158	14	1	00	27	73	00	05	0	-1	14		0	2
268	24344154	15	1	10	15	75	00	17	0	-1	15		0	9
269	24364150	16	1	00	00	57	43	10	2	-1	-1		2	9
270	24394146	15	0	00	00	99	00	12	1	80	-1	2.6	1	9
271	24424142	14	0	16	00	84	00	06	1	40	12		0	3
272	24464138	14	0	32	30	16	22	00	1	25	16		0	2
273	24494135	14	1	11	00	89	00	00	0	-1	-1		1	9
274	24524131	15	1	00	00	65	35	00	0	-1	-1		1	9
275	24554136	14	1	22	10	58	00	03	0	25	-1		1	3
276	24584122	14	1	00	00	87	13	00	1	-1	-1		0	9
277	24614118	14	1	00	68	32	00	00	0	-1	45		0	2
278	24634124	12	1	00	00	83	17	00	0	-1	-1		1	9
279	24664109	13	1	00	00	69	31	00	0	-1	-1		0	9
280	24694105	13	0	00	00	59	41	00	0	-1	-1	2.6	0	9
281	24724101	14	0	00	00	65	35	11	0	-1	-1		0	9
282	24754097	14	1	00	62	00	38	00	0	-1	45		0	1
283	24774093	14	1	00	49	00	51	00	0	-1	-1		0	9
284	24804089	14	1	00	45	00	55	00	0	-1	40		0	9
285	24834085	13	1	00	34	00	66	00	0	25	20		0	2
286	24864082	14	1	17	00	83	00	00	1	30	24		0	3
287	24904078	13	1	00	58	00	42	00	1	-1	40		0	1
288	24934074	12	1	35	00	65	00	00	0	-1	55		1	1
289	24964070	13	1	46	00	54	00	00	1	-1	-1		1	9
290	24994066	13	1	18	00	82	00	00	1	30	25	2.6	0	3
291	25034064	13	1	09	38	10	43	10	0	-1	20		0	3
292	25054059	14	1	39	61	00	00	00	0	-1	70		1	1
293	25064055	14	1	16	84	00	00	00	0	-1	-1		0	9
294	25104052	14	1	00	32	00	68	00	0	-1	30		0	1
295	25124048	14	1	00	76	00	24	11	0	-1	45		0	1
296	25154045	15	1	55	17	00	28	10	0	-1	45		0	1
297	25174040	15	1	37	37	00	36	00	0	-1	65		0	1
298	25204035	14	1	41	31	00	28	00	0	-1	70		0	1
299	25224032	15	1	73	27	00	00	00	0	-1	90		0	1
300	25264028	15	1	35	28	00	37	00	0	-1	-1	2.6	0	1
301	25294024	16	1	00	00	68	32	00	0	-1	-1		0	9
302	25334021	16	1	00	45	55	00	00	0	-1	25		0	2
303	25354017	15	0	00	15	64	21	00	0	-1	-1		0	9
304	25384012	16	0	00	00	22	78	00	0	-1	-1		0	9
305	25414008	16	0	00	43	57	00	00	1	30	-1		0	9
306	25434004	17	0	00	33	56	11	00	0	-1	30		0	9
307	25503998	16	0	00	54	00	46	00	0	-1	35		0	1
308	25533994	15	0	28	00	56	16	00	1	-1	25		0	3
309	25563989	15	0	00	00	99	00	00	2	-1	-1		0	9
310	25603985	15	0	00	00	00	00	00	0	-1	-1	2.6	0	9
311	25633981	15	0	62	00	00	38	00	0	-1	-1		0	1
312	25663977	15	0	70	00	00	30	00	0	-1	80		0	1
313	25703973	15	0	00	00	74	26	05	0	20	-1		2	9
314	25723968	14	0	00	00	87	13	05	1	20	-1		2	9
315	25773965	13	1	00	00	72	28	04	1	20	-1		0	9

ID	GR	CH	SB	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PD TOE
316	25783961	14	1	00	14	40	46	00	2	20	-1		0	9
317	25803956	15	1	00	26	00	74	00	0	-1	50		0	1
318	25833953	14	0	00	26	30	44	00	1	-1	35		0	1
319	25873949	15	1	00	20	80	00	00	2	20	15		0	9
320	25883944	14	0	00	22	20	58	06	0	-1	40	3.4	1	9
321	25913940	14	1	33	35	00	32	10	1	-1	30		0	2
322	25943936	13	1	00	36	00	64	00	1	-1	35		0	1
323	25973932	13	1	27	37	00	36	00	0	-1	42		0	1
324	26003928	15	1	82	00	00	18	00	0	-1	60		0	1
325	26033924	14	1	10	10	00	80	10	0	-1	15		0	9
326	26053919	13	1	00	77	23	00	02	1	-1	30		0	1
327	26083915	13	1	00	44	56	00	00	1	35	15		0	9
328	26123912	10	1	00	25	75	00	00	2	15	15		0	9
329	26143908	11	1	00	00	99	00	00	1	-1	-1		0	9
330	26183904	09	1	00	00	81	19	00	0	-1	-1	2.5	0	9
331	26213900	08	1	00	15	85	00	03	1	30	-1		0	9
332	26233896	12	0	00	19	70	11	00	1	25	-1		1	9
333	26253891	12	0	00	36	00	64	00	0	-1	30		0	1
334	26273887	12	0	33	34	00	00	00	0	-1	45		0	1
335	26313883	12	0	10	24	00	66	00	0	-1	-1		1	9
336	26333878	13	0	00	43	00	57	00	0	-1	-1		0	9
337	26353874	12	1	00	36	00	64	00	0	-1	-1		0	9
338	26383870	13	1	00	45	00	55	00	0	-1	-1		0	1
339	26423867	14	1	00	53	00	47	00	0	-1	45		0	1
340	26443863	13	1	00	53	30	17	00	2	-1	-1	2.6	0	9
341	26483859	14	1	00	42	10	48	00	0	-1	40		0	1
342	26503855	12	1	22	33	00	45	00	0	-1	-1		1	9
343	26523851	12	1	54	46	00	00	00	0	-1	45		0	9
344	26563847	11	1	42	00	00	58	00	2	-1	-1		0	9
345	26583842	10	2	52	48	00	00	00	0	-1	50		0	9
346	26623838	09	2	17	51	00	32	00	0	-1	40		0	9
347	26653834	09	2	00	00	00	99	00	0	-1	-1		0	9
348	26673830	11	1	00	18	59	23	00	1	-1	-1		0	9
349	26713826	12	1	00	00	72	28	00	1	50	-1		0	9
350	26733822	09	1	00	00	54	46	00	1	40	-1	2.3	2	9
351	26763817	09	1	00	00	00	99	00	1	45	-1		0	9
352	26793814	10	1	00	00	99	00	00	0	-1	-1		0	9
353	26823810	10	1	00	00	82	18	16	1	30	-1		2	9
354	26853805	10	1	00	00	75	25	14	1	15	-1		0	9
355	26863802	12	1	00	00	99	00	12	0	-1	-1		0	9
356	26903797	11	1	00	00	77	23	12	1	20	1		0	9
357	26923793	11	1	00	00	68	32	05	1	25	-1		1	9
358	26963789	12	1	00	00	99	00	06	1	30	-1		0	9
359	26983785	11	1	00	00	73	27	07	1	25	-1		1	9
360	27023782	10	1	00	00	74	26	00	1	17	-1	1.8	1	9
361	27053778	10	1	00	00	78	22	04	1	25	-1		0	9
362	27083774	10	1	00	00	86	14	10	1	65	-1		0	9
363	27103770	09	0	00	00	20	80	00	1	50	-1		0	9
364	27133766	11	0	00	00	99	00	00	0	-1	-1		0	9
365	27163762	10	0	00	00	76	24	11	0	-1	-1		2	9
366	27183757	10	0	00	00	78	22	27	1	-1	-1		2	9
367	27223754	11	0	00	00	99	00	03	1	40	1		0	9
368	27253749	10	1	00	00	99	00	18	1	40	-1		0	9
369	27273745	10	1	00	00	86	14	34	0	-1	-1		0	9
370	27303741	11	1	00	00	99	00	18	0	-1	-1	2.0	2	9

ID	GR	CH	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PDTOE
371	27333737	13	1	21	00	63	16	00	0	-1	17	0	9
372	27363733	15	1	00	00	99	00	00	0	-1	-1	0	9
373	27393728	15	1	27	00	73	00	06	1	-1	20	1	2
374	27423724	13	0	00	00	80	20	07	0	-1	-1	1	9
375	27443719	12	0	00	00	80	20	04	0	-1	-1	1	9
376	27473716	12	1	00	33	00	67	04	0	-1	30	1	3
377	27513712	12	1	00	00	66	34	02	0	-1	-1	1	9
378	27533708	11	1	00	14	86	00	02	0	55	-1	0	3
379	27563704	11	1	00	20	80	00	00	1	35	-1	0	3
380	27583701	12	1	00	37	63	00	00	0	-1	12	2.9	2
381	27623697	11	1	00	20	80	00	10	0	-1	35	1	3
382	27643693	11	1	00	00	99	00	33	0	-1	-1	1	9
383	27673688	10	1	00	24	56	20	04	1	43	25	1	9
384	27703684	11	1	25	15	60	00	04	2	-1	-1	0	2
385	27723680	12	0	36	00	64	00	00	0	-1	36	2	9
386	27763676	13	0	00	12	88	00	00	1	30	-1	1	9
387	27883672	14	0	00	00	80	20	10	1	20	-1	2	9
388	27813668	16	0	00	14	76	10	07	2	55	24	2	9
389	27843664	16	0	27	15	48	10	30	1	65	50	2	3
390	27873659	14	1	38	00	52	10	16	0	-1	-1	2.3	3
391	27893655	15	1	27	00	61	12	11	2	75	70	2	3
392	27933652	16	1	23	00	65	12	10	2	45	28	1	3
393	27963648	16	1	00	00	90	10	00	1	45	-1	1	9
394	27983644	14	1	21	00	56	13	02	0	-1	24	0	3
395	28023640	11	1	00	20	00	80	00	0	-1	-1	0	9
396	28053636	11	1	00	63	00	37	00	0	-1	50	0	3
397	28083631	12	1	00	60	00	40	00	0	-1	45	0	2
398	28113627	12	1	00	33	00	67	00	0	-1	30	0	9
399	28133623	13	1	50	21	00	29	00	0	-1	28	0	2
400	28173619	12	1	31	23	00	46	00	0	-1	43	2.4	2
401	28193615	13	1	43	14	00	43	00	0	-1	40	0	2
402	28223611	14	1	28	24	28	20	00	0	-1	31	0	2
403	28263607	15	1	26	30	00	44	00	0	-1	38	0	2
404	28283603	16	1	00	16	84	00	00	1	-1	-1	1	2
405	28313599	17	1	22	00	58	20	05	1	70	23	1	3
406	28333595	18	1	00	28	72	00	00	1	-1	25	0	9
407	28373590	18	1	00	38	50	12	00	0	-1	24	0	2
408	28393586	18	1	78	00	00	22	00	0	-1	58	0	1
409	28433581	18	1	73	00	00	27	00	0	-1	30	1	2
410	28463577	18	1	67	23	00	10	00	0	-1	55	2.2	1
411	28493572	17	1	38	17	45	00	00	1	-1	-1	0	1
412	28523568	17	1	00	47	53	00	00	0	-1	70	0	2
413	28543563	18	0	00	46	12	42	00	0	-1	-1	0	2
414	28583559	18	0	00	00	85	15	00	1	-1	-1	0	9
415	28613555	18	0	00	00	99	00	00	0	-1	-1	0	9
416	28633550	18	0	00	22	47	31	00	0	-1	20	0	9
417	28663546	17	0	00	77	00	23	00	0	-1	47	0	9
418	28683542	18	0	00	39	31	30	00	0	-1	40	0	9
419	28713538	18	0	00	26	63	11	00	1	75	60	0	9
420	28733533	18	0	00	35	55	10	00	0	-1	-1	2.8	9
421	28763530	18	0	00	15	64	21	00	2	30	-1	0	9
422	28783525	18	1	08	18	74	00	00	0	35	-1	1	9
423	28813520	17	0	20	00	80	00	00	0	-1	40	1	3
424	28833516	16	1	37	00	63	00	02	0	-1	50	1	3
425	28843511	14	0	22	00	66	12	00	0	-1	-1	0	9

ID	GR	CH	SB	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PD	TOE
426	28833507	12	0	99	00	00	00	00	0	-1	99		0	1	
427	28933505	12	0	99	00	00	00	00	0	-1	-1		0	1	
428	28963501	17	0	00	00	99	00	00	0	-1	30		0	2	
429	28983496	16	0	10	00	77	13	00	1	-1	-1		1	9	
430	29023492	16	1	00	00	76	24	00	0	-1	-1	2.8	0	9	
431	29053488	16	1	00	47	10	43	00	0	-1	45		0	2	
432	29063484	15	1	00	00	26	74	00	0	-1	-1		0	9	
433	29103481	16	1	00	50	20	30	00	0	-1	25		0	2	
434	29133477	18	1	00	13	77	10	00	1	-1	-1		1	9	
435	29163473	18	1	00	00	99	00	08	1	30	-1		0	9	
436	29193469	17	1	00	47	53	00	00	0	-1	-1		1	2	
437	29223465	16	1	00	81	19	00	00	0	-1	75		0	2	
438	29243461	16	1	00	35	65	00	00	1	-1	-1		0	9	
439	29263457	14	1	00	78	00	22	00	0	-1	99		0	2	
440	29303454	15	1	00	56	00	44	00	2	-1	-1	1.9	0	2	
441	29333451	15	1	00	23	00	77	00	0	-1	70		0	9	
442	29363447	15	1	00	65	00	35	02	0	-1	-1		0	2	
443	29383443	13	1	10	19	61	10	00	2	-1	-1		0	9	
444	29413439	14	1	00	17	83	00	00	1	50	35		1	2	
445	29433435	12	1	00	13	69	18	04	1	15	-1		1	9	
446	29463431	11	1	00	42	58	00	00	0	75	60		1	2	
447	29493427	13	1	00	00	99	00	00	0	-1	-1		0	9	
448	29523423	13	1	00	22	68	10	05	1	-1	-1		0	9	
449	29563419	10	1	00	29	71	00	04	1	35	30		1	2	
450	29593415	13	1	00	18	82	00	00	0	-1	-1	3.3	0	9	
451	29623412	13	1	00	28	11	61	00	1	45	25		0	2	
452	29653407	14	1	00	31	69	00	02	1	35	23		1	9	
453	29693403	16	1	00	00	99	00	40	0	-1	-1		2	9	
454	29713399	17	1	00	00	84	16	10	1	35	-1		1	9	
455	29733395	17	1	00	28	72	00	00	1	-1	23		0	2	
456	29773391	16	1	47	17	36	00	00	0	45	25		0	2	
457	29803387	16	1	00	45	55	00	00	0	-1	50		0	2	
458	29823383	15	1	00	31	10	59	00	1	70	-1		1	2	
459	29863378	17	1	00	26	74	00	00	2	30	-1		0	9	
460	29893375	18	1	00	60	40	00	00	1	55	52	2.1	0	2	
461	29923371	17	1	00	65	00	35	00	0	-1	-1		0	2	
462	29943367	17	1	00	18	72	10	02	1	-1	-1		0	9	
463	29973362	15	1	00	25	23	52	02	0	80	25		0	9	
464	30003358	15	1	00	32	68	00	00	1	30	22		0	2	
465	30043353	15	1	00	46	54	00	00	1	23	50		0	2	
466	30073348	15	2	18	50	00	32	00	0	-1	-1		0	9	
467	30103345	14	2	34	29	27	10	00	1	90	30		0	2	
468	30133341	16	2	00	10	90	00	00	1	70	32		1	9	
469	30163337	15	2	00	31	42	27	00	1	28	-1		0	2	
470	30193333	13	2	00	60	00	40	00	0	-1	42	2.1	0	9	
471	30223329	14	2	00	48	00	52	00	0	-1	-1		1	2	
472	30263325	17	2	00	11	69	20	00	1	-1	35		0	9	
473	30293322	18	1	10	16	74	00	00	1	38	27		1	2	
474	30313318	19	1	38	00	52	10	07	0	-1	30		0	2	
475	30353314	17	1	00	20	60	20	00	1	30	-1		0	3	
476	30383310	17	1	00	14	86	00	12	1	50	-1		1	9	
477	30413306	14	1	00	18	63	19	11	1	38	34		1	2	
478	30453302	13	1	00	54	00	44	06	1	-1	-1		0	2	
479	30483299	13	1	00	66	00	34	03	0	-1	60		0	2	
480	30513295	14	1	00	36	64	00	02	0	-1	40	2.0	1	2	

ID	GR	CH	SB	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PD	TOE
481	30543291	12	1	00	70	00	30	00	1	-1	22		0	2	2
482	30573287	13	1	00	50	50	00	00	0	-1	-1		1	2	2
483	30603283	14	1	00	26	74	00	02	2	-1	18		0	2	2
484	30633278	12	1	35	13	00	52	00	0	-1	20		0	2	2
485	30673276	12	1	20	40	00	40	00	0	-1	-1		0	2	2
486	30703272	12	1	13	66	00	21	00	0	-1	13		0	2	2
487	30733268	10	0	80	20	00	00	00	1	-1	50		0	2	2
488	30763264	10	0	00	17	83	00	02	1	25	15		1	2	2
489	30803261	12	0	00	20	50	30	00	1	30	12	2.9	0	2	2
490	30823257	11	0	00	30	70	00	00	1	32	14		0	2	2
491	30863253	10	0	12	00	88	00	03	1	35	12		0	3	2
492	30893249	08	0	27	00	73	00	00	0	-1	14		0	2	2
493	30923245	09	0	00	00	99	00	00	1	10	-1		0	2	2
494	30953241	07	0	65	00	00	35	00	1	-1	10		0	2	2
495	30993238	09	0	24	00	30	46	00	1	75	11		0	3	2
496	31023234	10	0	23	00	77	00	02	1	18	12		0	2	2
497	31053230	10	0	00	00	70	30	00	1	15	-1		0	2	2
498	31083226	12	1	00	00	99	00	00	1	-1	-1		0	2	2
499	31113222	11	1	37	00	63	00	04	1	33	28		0	2	2
500	31143218	12	1	52	00	30	18	00	1	-1	19	1.5	0	2	2
501	31173214	13	1	20	00	80	00	00	1	25	13		0	2	2
502	31203210	12	1	00	20	80	00	02	1	30	18		0	2	2
503	31233206	14	1	00	00	84	16	00	1	37	-1		0	2	2
504	31263201	13	1	35	00	65	00	00	1	-1	32		0	2	2
505	31283197	14	1	25	00	75	00	00	1	65	-1		0	2	2
506	31323194	14	2	00	10	90	00	00	1	20	-1		0	2	2
507	31343190	14	2	00	21	79	00	00	1	20	21		0	2	2
508	31393187	14	2	00	30	00	70	00	1	58	22		0	2	2
509	31413182	15	1	26	00	24	50	00	1	27	14		1	2	2
510	31433178	14	1	28	00	00	00	00	1	-1	40	1.3	0	2	2
511	31473173	13	1	76	00	00	24	00	1	-1	33		0	2	2
512	31503170	13	1	38	00	62	00	00	1	-1	10		1	2	2
513	31533166	12	1	00	00	99	00	00	1	18	-1		0	2	2
514	31563162	12	1	00	20	80	00	00	1	23	14		0	2	2
515	31593157	12	1	00	00	99	00	03	1	45	-1		1	2	2
516	31633154	12	1	00	00	72	28	03	2	30	-1		0	2	2
517	31653150	12	1	00	00	74	26	03	1	45	-1		0	2	2
518	31683146	10	1	00	00	80	20	10	1	30	-1		2	2	2
519	31713142	09	1	00	00	40	60	24	0	-1	-1		1	2	2
520	31733138	10	1	00	00	70	30	30	0	-1	-1	1.8	1	2	2
521	31773134	09	1	00	00	80	20	06	1	25	-1		1	2	2
522	32053097	05	0	00	00	99	00	00	1	31	-1		0	2	2
523	32083093	04	2	00	00	99	00	10	1	20	-1		0	2	2
524	32103088	05	2	00	00	99	00	00	1	30	-1		0	2	2
525	32123083	07	2	00	00	99	00	00	1	18	-1		0	2	2
526	32143079	07	2	42	00	58	00	00	0	-1	25		0	2	2
527	32173075	08	2	30	00	45	25	00	0	-1	15		0	2	2
528	32193071	09	2	00	00	53	47	00	0	-1	-1		0	2	2
529	32223066	07	2	00	00	53	47	21	0	-1	-1		0	2	2
530	32263063	08	2	42	00	00	58	00	0	-1	24	0.5	0	2	2
531	32283060	09	2	00	00	72	28	00	1	-1	-1		1	2	2
532	32323056	09	2	00	00	77	23	02	1	20	-1		1	2	2
533	32343052	08	2	00	00	30	70	00	0	-1	-1		0	2	2
534	32373047	08	2	00	00	62	38	08	2	-1	-1		1	2	2
535	32403043	08	2	00	00	54	46	21	1	47	-1		1	2	2

ID	GR	CH	SB	RS	DS	TC	DC	SG	A	WC	LLS	RRE	Q	PDTOE
536	32433040	10	2	00	00	78	22	00	1	37	-1		1	9
537	32463036	09	2	00	50	30	20	00	0	-1	-1		2	9
538	32503031	10	2	00	00	80	20	03	0	-1	-1		1	9
539	32563024	10	2	00	00	80	20	08	2	-1	-1		1	9
540	32593020	09	2	00	00	80	20	11	1	25	-1	0.1	2	9
541	32623017	10	2	00	00	80	20	09	1	22	-1		2	9
542	32643012	10	2	00	00	80	20	05	0	-1	-1		1	9
543	32683008	10	2	00	00	80	20	00	1	-1	-1		0	9
544	32713004	10	2	00	00	80	20	00	1	45	-1		0	9
545	32733000	10	2	00	00	86	14	12	1	48	-1		2	9
546	32772997	10	2	00	00	75	25	08	1	20	-1		1	9
547	32812993	10	2	00	00	77	23	09	2	20	-1		1	9
548	32832990	10	2	00	30	70	00	00	1	52	45		0	9
549	32872986	09	2	00	00	70	30	05	1	35	-1		1	9
550	32892983	11	2	00	00	77	23	00	1	-1	-1	1.0	0	9
551	32922979	10	2	00	39	00	61	00	0	-1	44		0	9
552	32962976	10	2	00	00	80	20	00	1	20	-1		0	9
553	32992972	07	2	00	38	62	00	02	1	-1	27		0	9
554	33022968	07	2	00	00	99	00	28	0	-1	-1		1	9
555	33052964	08	2	00	00	99	00	28	0	-1	-1		1	9
556	33082961	08	2	00	31	69	00	00	1	-1	-1		0	9
557	33112957	07	2	00	33	00	67	00	0	-1	23		0	2
558	33142953	07	2	00	00	35	65	00	0	-1	-1		2	3
559	33172949	07	2	00	00	00	99	00	0	-1	-1		0	9
560	33212946	09	2	00	10	90	00	00	1	-1	-1	0.2	1	9
561	33232941	08	2	00	00	00	99	00	1	45	-1		1	9
562	33262937	09	2	00	00	00	99	08	1	-1	-1		0	9
563	33302933	10	2	00	00	80	20	00	0	-1	-1		0	9
564	33332930	11	2	00	22	62	16	08	0	-1	15		0	9
565	33362926	10	2	00	00	65	35	04	1	-1	30		0	9
566	33392922	08	2	00	33	57	10	00	1	-1	-1		1	9
567	33422918	10	2	00	00	00	99	00	0	-1	-1		1	9
568	33452914	10	2	00	20	60	20	00	0	-1	-1		0	9
569	33482910	10	2	00	20	00	80	05	0	-1	10		0	9
570	33522906	10	2	00	00	00	10	90	00	1	-1	0.2	0	9
571	33542902	10	2	00	00	00	50	50	00	1	30		0	9
572	33582898	10	2	00	00	00	20	80	00	1	-1		0	9
573	33612894	09	2	00	00	00	30	70	00	0	-1		0	9
574	33632890	09	2	00	00	00	23	77	00	0	-1		1	9
575	33662886	08	2	00	00	00	00	99	00	0	-1		0	9
576	33702882	08	2	00	00	00	00	99	00	0	-1		0	9
577	33732878	10	2	00	00	00	20	80	00	1	-1	0.2	0	9

Variable	Mean	Standard deviation	Minimum	Maximum	Units
Recent slips	9.3	17.8	0	100	%
Degraded slips	17.4	21.8	0	100	%
Till cliff	44.1	34.0	0	100	%
Degraded cliff	28.9	28.9	0	100	%
Sand	2.8	6.7	0	56	%
Length of landslip	28.1	16.2	4	100	m
Width of embayment	41.6	19.5	10	100	m
Rate of recession	1.8	1.0	0.1	3.7	m/yr
Cliff height	12.1	2.7	4.0	19.0	m
Categorical variables					
Ridge index	% of cases				
0	56				
1	44				
Groundwater index					
0	70				
1	24				
2	6				

Appendix B

Descriptive statistics for all variables

APPENDIX C

DESCRIPTION OF CLIFF IN AREAS OF UNUSUAL SOIL TYPES

C.1 High Proportion of Sand and Gravel

SKIPSEA (TA 1859 5407-1862 5398) CASES 24-26

The sand layer occurs as a channel 150m long and several metres thick, forming about 50% of the cliff face. The slope is uniform at an angle of 38° . Seepage erosion causes embayments to form, and marine erosion leads to undercutting and falls of the cliff. The cliffline retreats 20m at the position of the sand layer, forming an embayment.

ATWICK ROAD END (TA 1961 5118-1969 5095) CASES 89-94

The sand occurs near the clifftop in a layer 2-3m thick, and forms about 25% of the cliff face along a length of 200m. The effect on the cliff is the same as at Skipsea, except below the sand layer the till forms steep cliffs.

MAPPLETON (TA 2257 4434-2259 4430) CASES 202-203

The sand occurs as a layer 3m thick at mid-height in the cliff along a length of 100m, forming 20% of the cliff face. Seepage erosion is common, causing falls in the overlying clay.

EAST NEWTON TO RINGBROUGH (TA 2682 3810-2730 3741) CASES 353-370

The sand occurs as an irregular layer from 1-3m thick over a length of nearly a kilometre, forming 15-30% of the cliff. This increases to 35% in some large channels near Ringbrough. No slips occur owing to the low cliff height and the presence of the sand (which encourages falls to occur and embayments to develop).

BEACON HILL, RINGBROUGH (TA 2788 3672-2793 3652) CASES 387-392

Sands and silts occur in channels several metres thick in the upper cliff, forming 10-30% of the cliff face along a length of 250m. Large embayments form by slipping, with the slip surface emerging in the sand. Slides and falls of sand are encouraged by seeping groundwater.

MONKWITH (TA 2965 3407-2971 3399) CASES 452-454

Sand occurs in a channel several metres thick, forming 10-40% of the cliff along a length of 100m. The cliff is steep (45°) and recession is by seepage erosion and falls.

WITHERNSEA (TA 3299 2972-3308 2961) CASES 553-556

A 2-3m thick sand layer forms 30% of the cliffs along a 150m length. The cliffs are steep with falls caused by seepage erosion. Many smaller sand layers also occur in this area, and the cliff morphology is mainly "till cliff".

C.2 "Stone-Free" Silty Clay Layers Present

All outcrops occur at the Upper/Middle Skipsea till boundary unless otherwise stated.

ROLSTON RIFLE RANGE (TA 2158 4608) CASES 160-163

Soft light brown laminated "stone-free" CLAY occurs in a layer at the cliff foot, from 0.1 to 0.5m thick, and along a 200m length of cliff. Two long (50m) degraded slips occur with their toe emerging in the "stone-free" clay (which contains slickensides). The area forms an embayment and the slope angle is low ($\sim 30^\circ$).

COWDEN RANGES-NORTH END (TA 2372 4245-2424 4165) CASES 247-265

Soft brown laminated "stone-free" CLAY occurs in a layer at the cliff foot, near continuous over 900m length of cliff. It was hidden by the beach, except during August 1987 when it was exposed in several places. The cliff morphology is of degraded slips, continuous along the coast, forming a low angle ($20-30^\circ$) slope. The cliff retreats up to 20m from the general cliffline and slip surfaces and slickensides were observed in the "stone-free" clay.

COWDEN RANGES-SOUTH END (TA 2475 4907-2526 4029) CASES 281-300 and (TA 2550 3998-2566 3977) CASES 307-312

Soft brown laminated "stone-free" clayey SILT, occurs in a layer below the cliff foot. The exposure is poor, but it is occasionally seen in the foreshore during low beach conditions,

at low arcuate ridges marking the position of the toe of slips. The cliff morphology is of long ($\sim 100\text{m}$), sometimes multiple, degraded slips, continuous along the coastline.

ALDBROUGH ROAD END (TA 2577 3965-2600 3928) CASES 315-324

Soft-firm brown laminated "stone-free" silty CLAY occurs as a layer 1-2m in thickness below the cliff foot over a length of 450m. Six long ($\sim 70\text{m}$) multiple degraded slips occur with the slip surface emerging in the "stone-free" clay.

EAST NEWTON (TA 2625 3891-2662 3838) CASES 333-346

Soft light brown "stone-free" CLAY occurs in a layer 0.5m thick at the cliff foot over a length of 650m. It is poorly exposed (only seen in the north of the area in November 1987) but is inferred to be present throughout by the morphology of long ($\sim 80\text{m}$) degraded slips at a low slope angle ($25-30^\circ$).

GRIMSTON HALL (TA 2883 3516-2896 3501) CASES 424-428

Soft laminated "stone-free" silty CLAY occurs in a large V-shaped outcrop, probably within the Withernsea till. The unusual geometry of the layer has resulted in a multiple slip of low slope angle, 100m in length and receding up to 60m from the general cliffline. The "stone-free" clay shows slickensides and contains part of the slip surface.

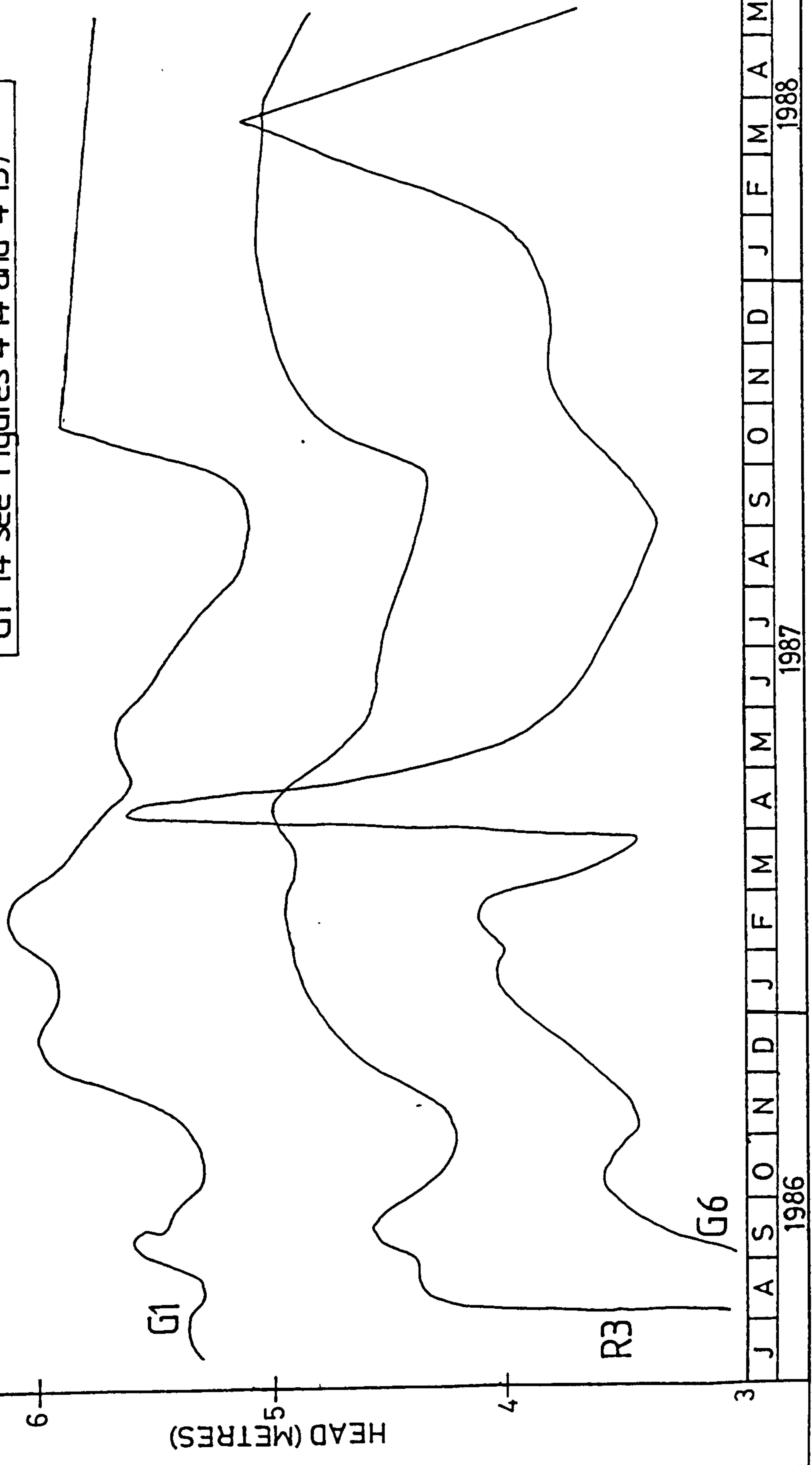
HOOKS (TA 2919 3469-2936 3447) CASES 436-442

Soft light brown "stone-free" CLAY was observed at the cliff foot during low beach conditions in April 1988. The cliff

morphology is of long ($\sim 80\text{m}$) multiple degraded slips and the slip surface emerged in the "stone-free" clay layer.

"Stone-free" clays were observed in the foreshore during low beach conditions at many other places near to those above, however they were at too low a level to affect the mode of failure of the cliffs.

Figure A4.13 Appendix D
 Plots of pressure head with time
 (for location of piezometers R1-13 and
 G1-14 see Figures 4.14 and 4.15)



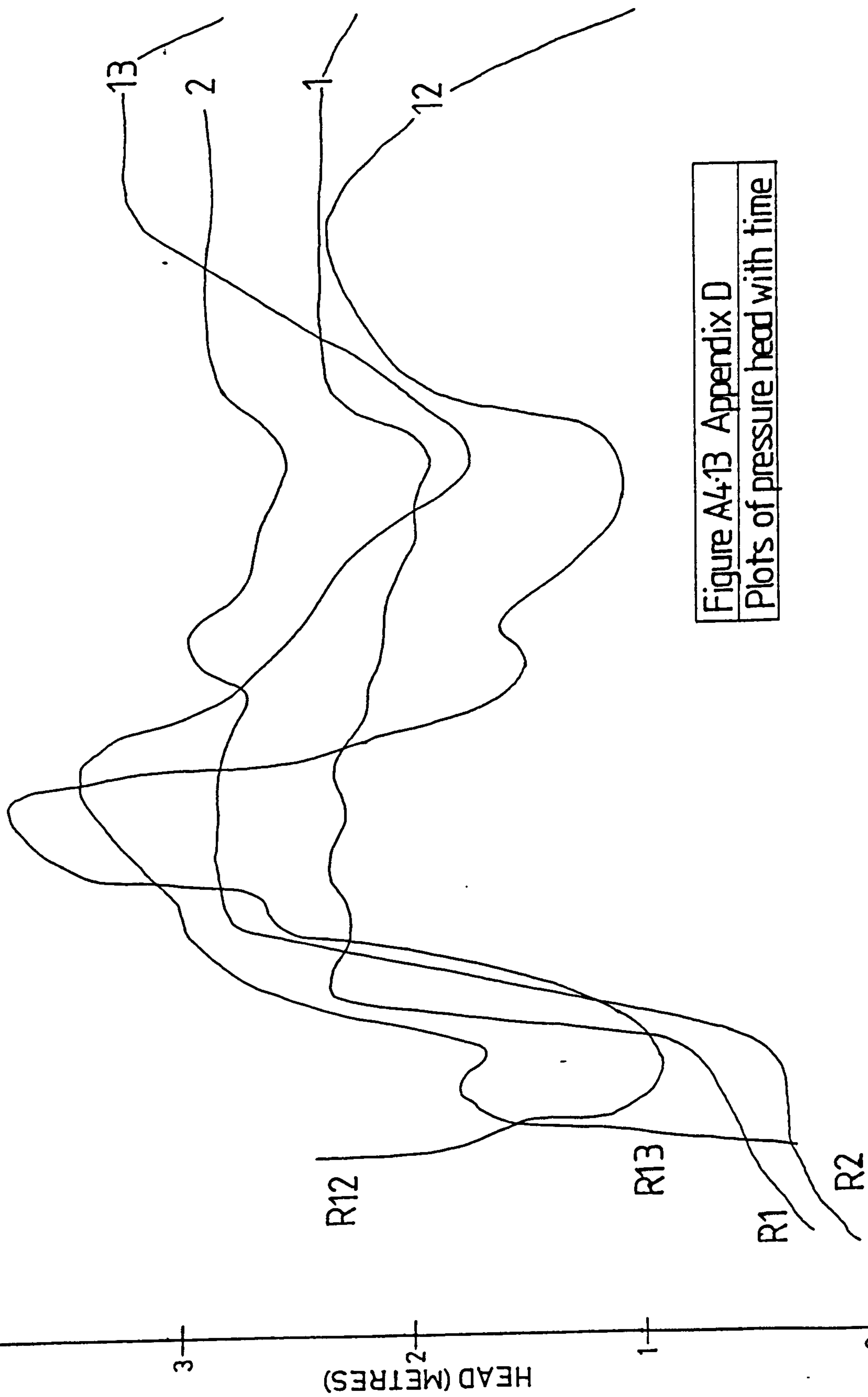


Figure A4.13 Appendix D
Plots of pressure head with time

J	A	S	O	N	D	J	F	M	A	M	J
1986					1987					1988	

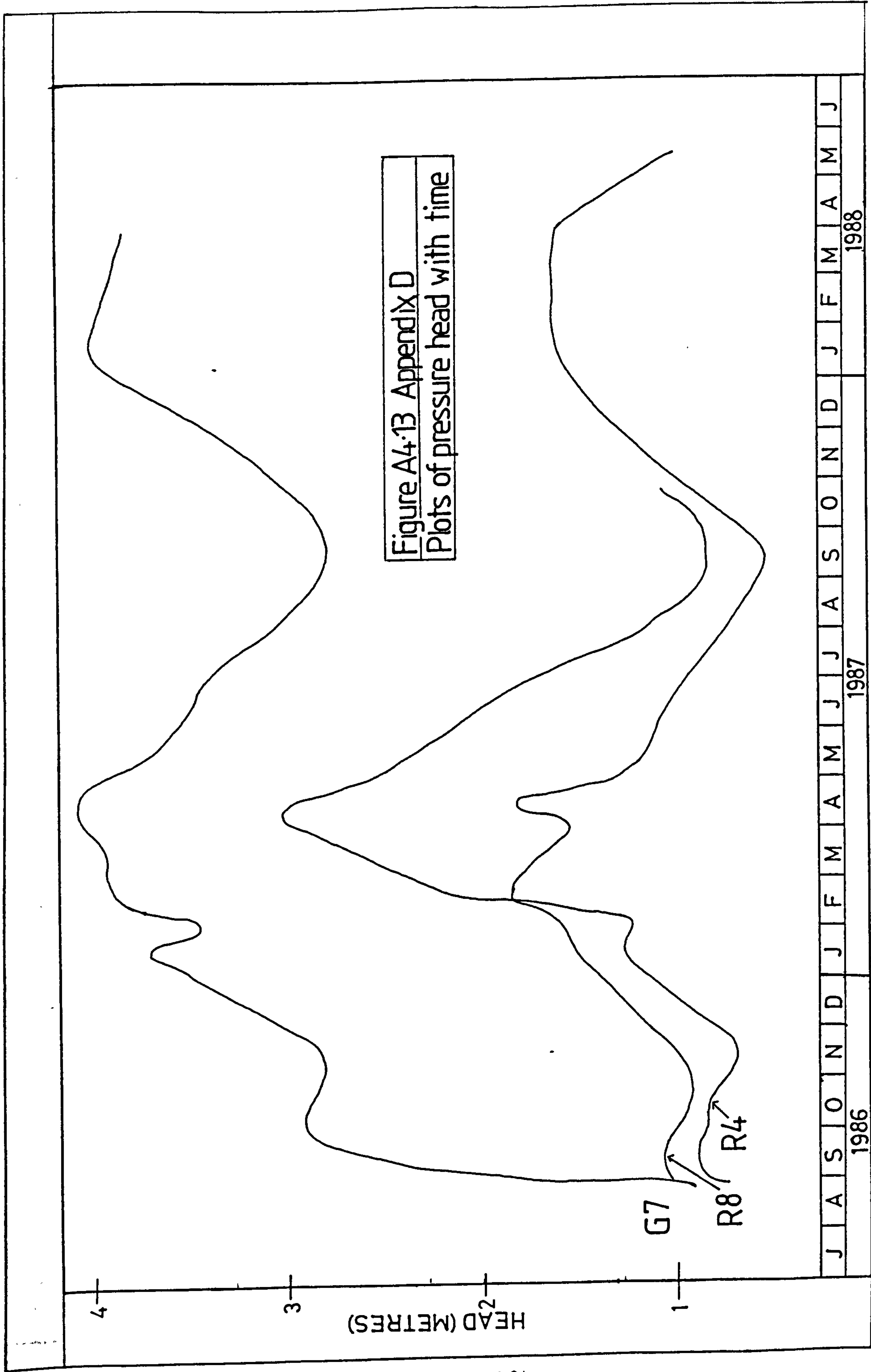


Figure A4.13 Appendix D
Plots of pressure head with time

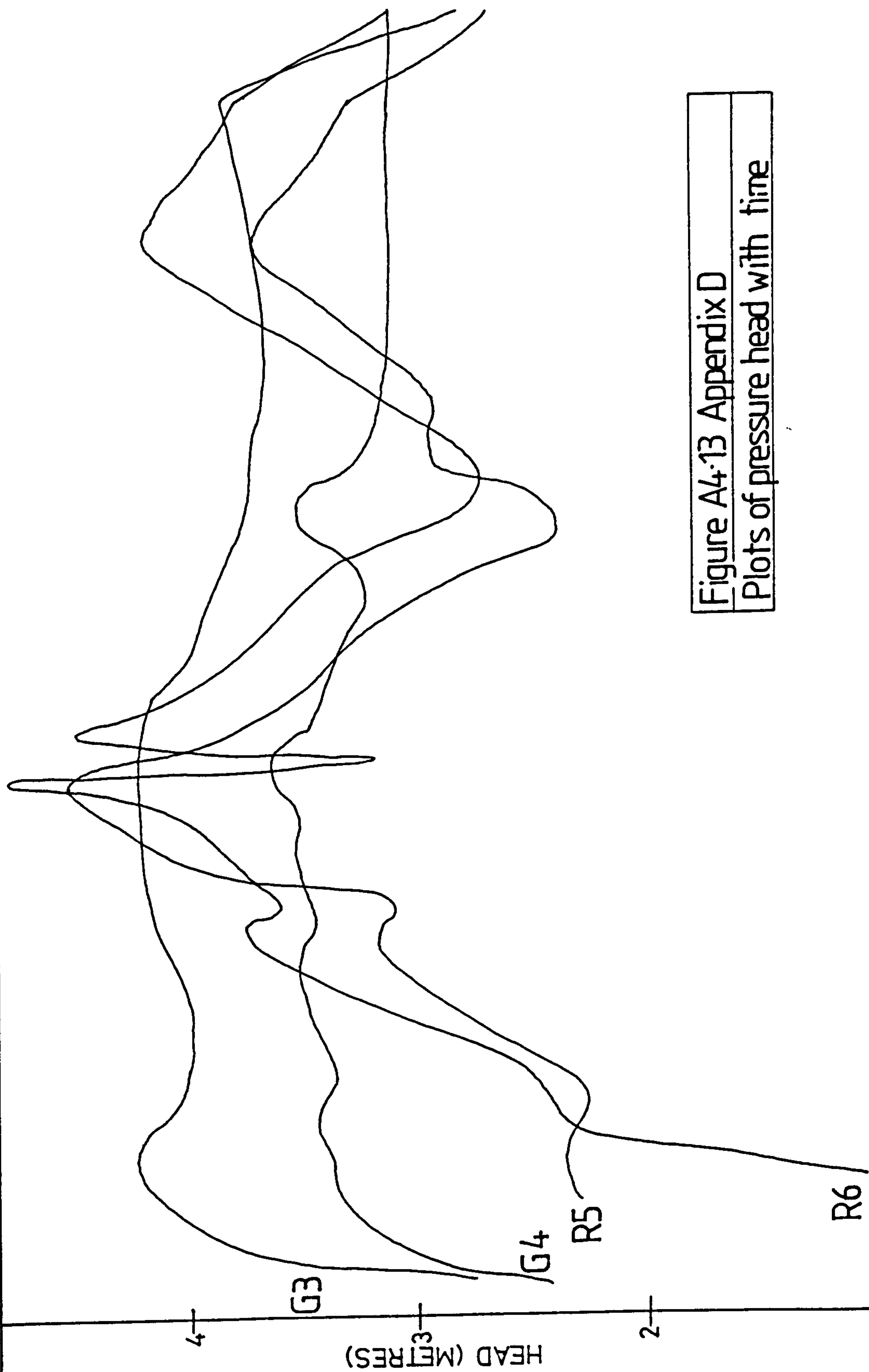


Figure A4.13 Appendix D
Plots of pressure head with time

J	A	S	O	N	D	J	F	M	A	M	J
1986						1987					
						1988					

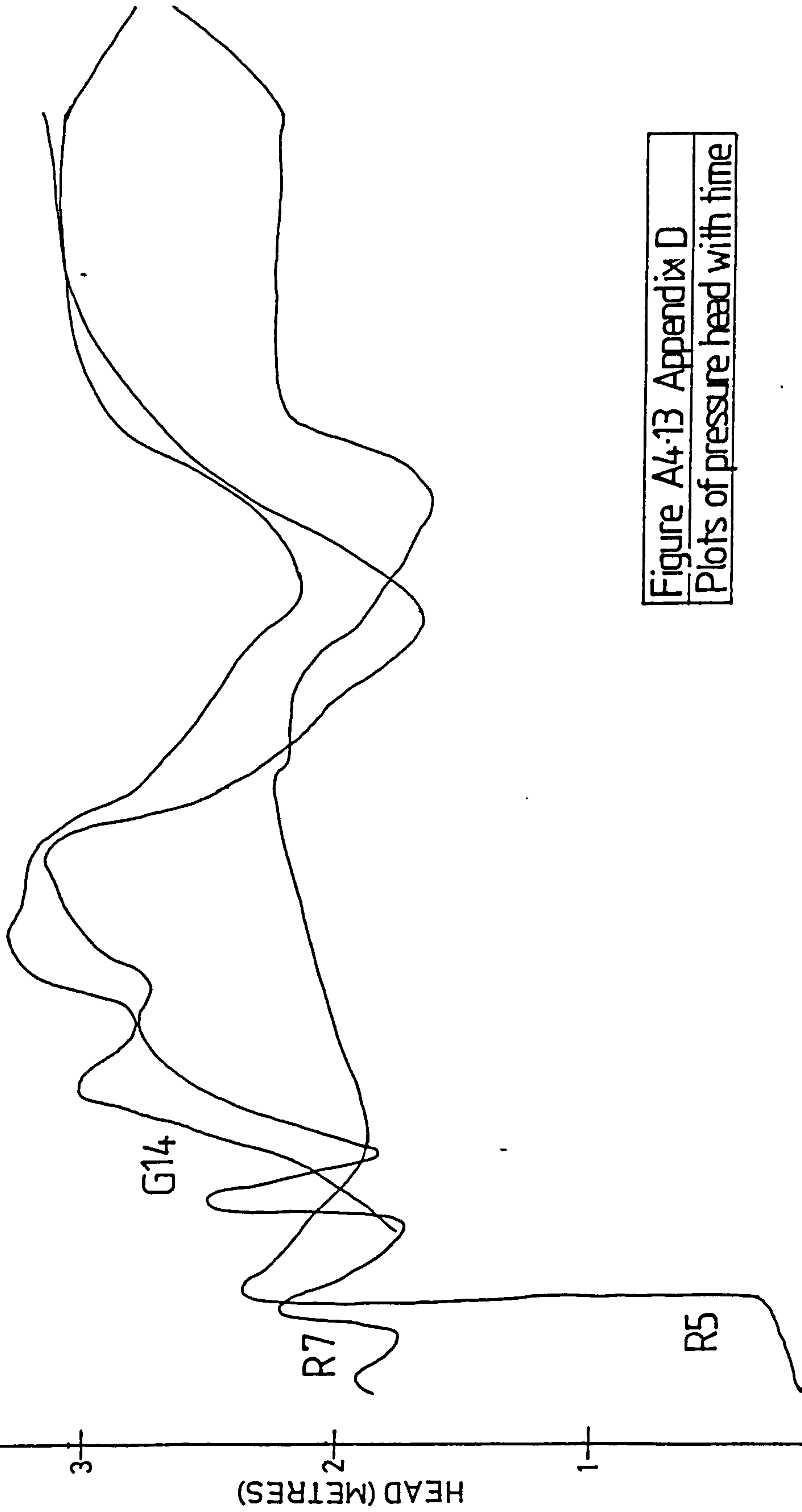


Figure A4.13 Appendix D
Plots of pressure head with time

J	A	S	O	N	D	J	F	M	A	M	J
1986					1987					1988	

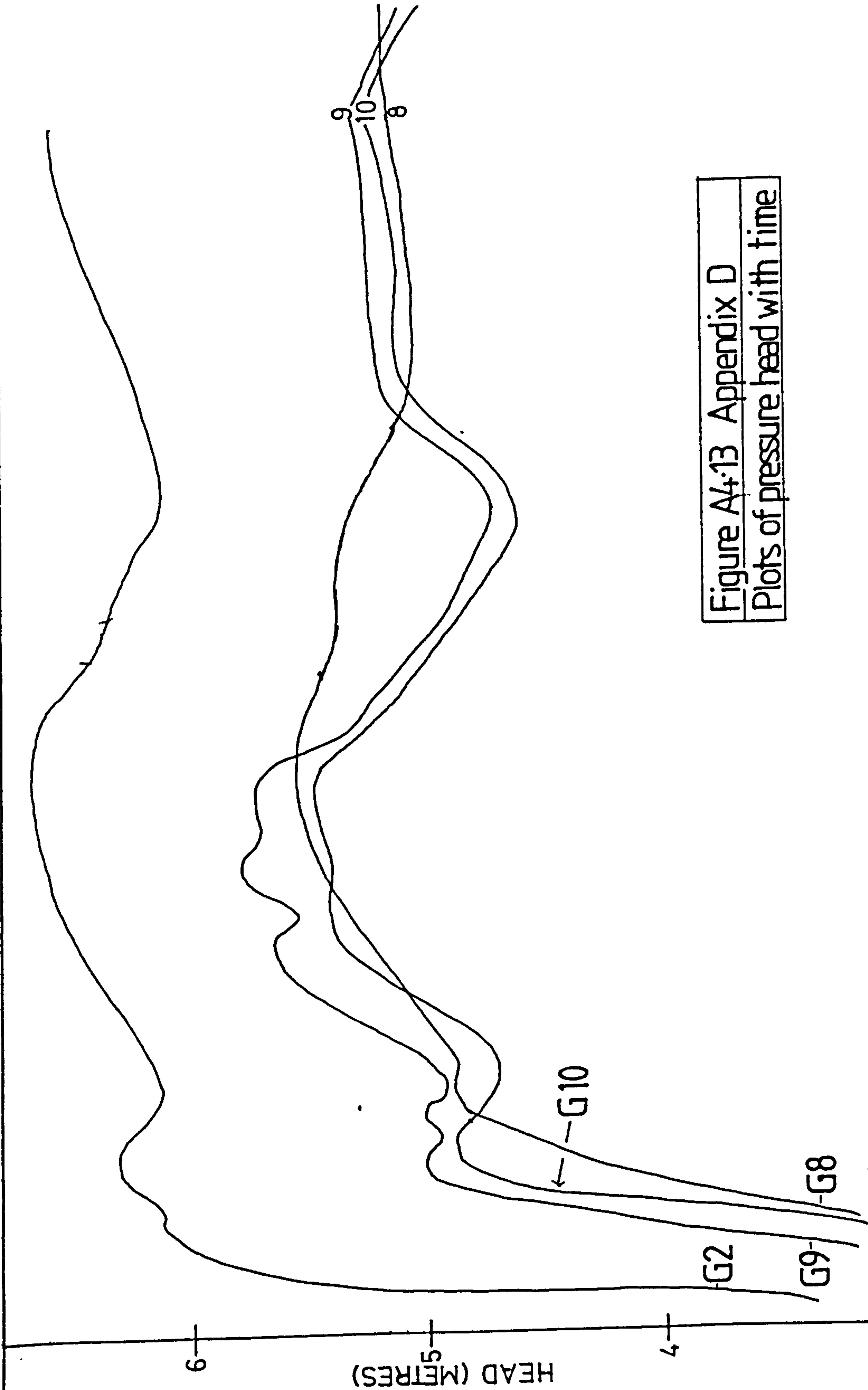
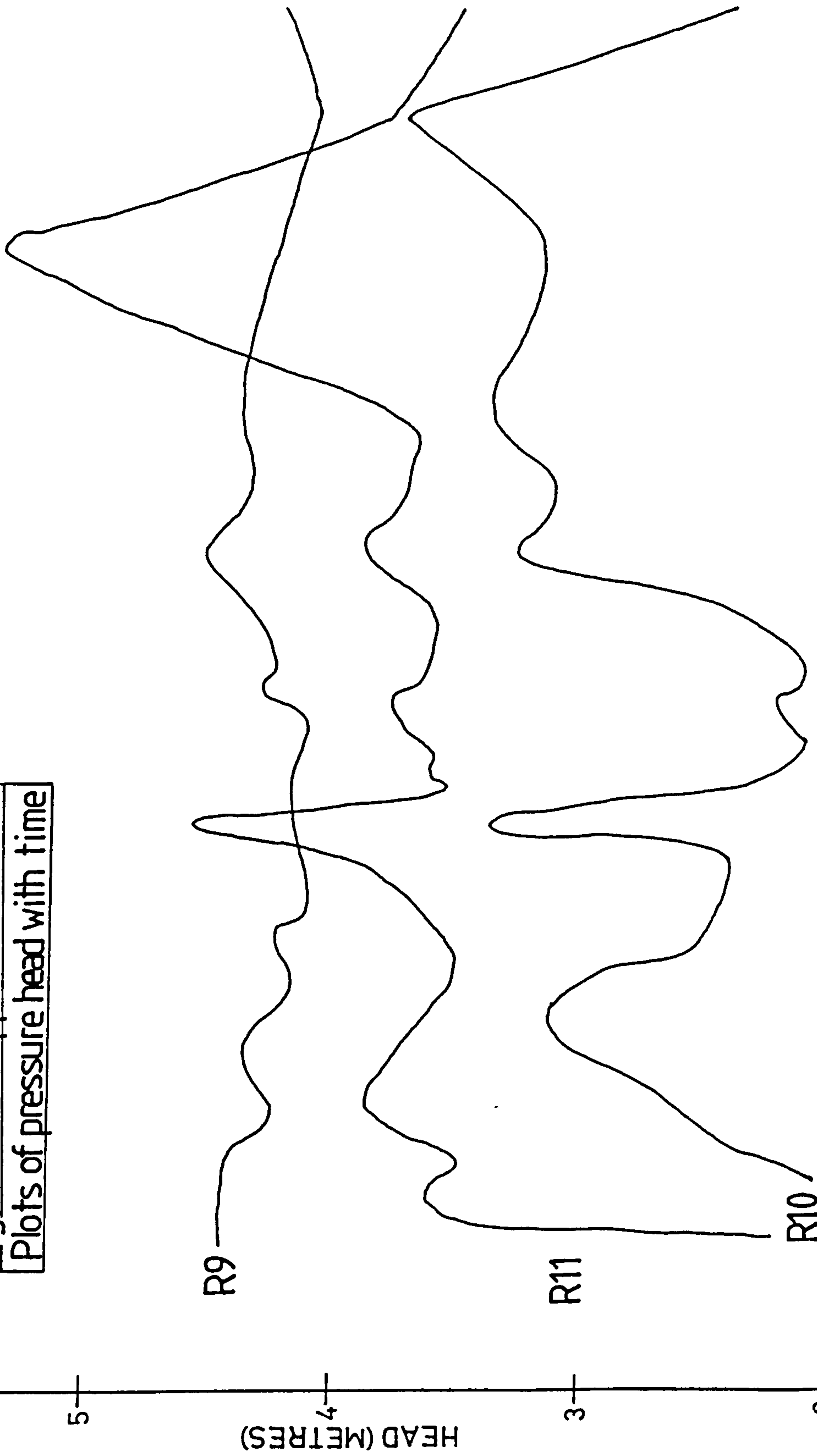


Figure A4.13 Appendix D
Plots of pressure head with time

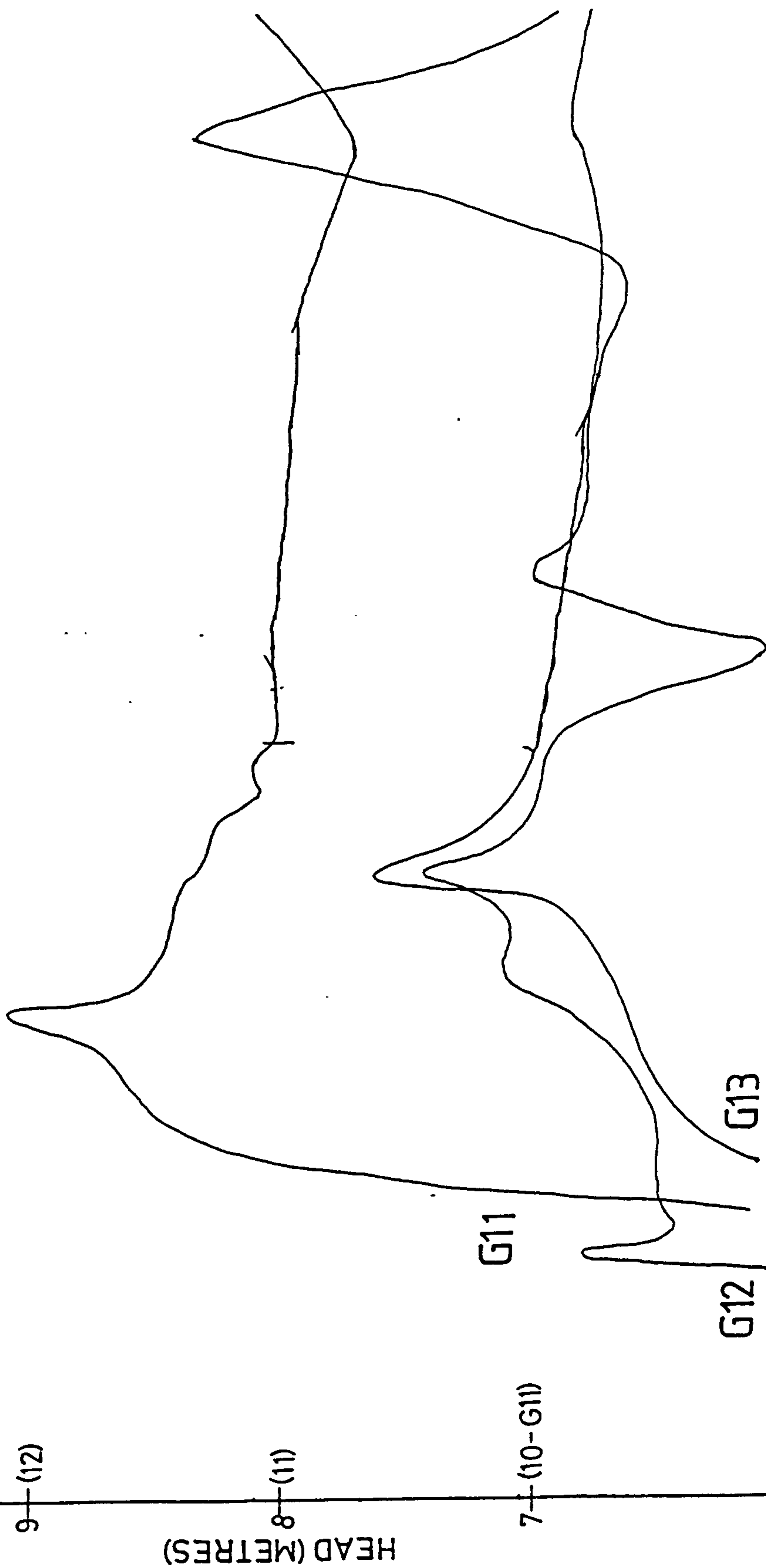
J	A	S	O	N	D	J	F	M	A	M	J
1986					1987					1988	

Figure A4.13 Appendix D
Plots of pressure head with time



J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J
1986						1987						1988											

Figure A4.13 Appendix D
Plots of pressure head with time



9--(12)

8--(11)

7--(10-G11)

6

HEAD (METRES)

G11

G12

G13

	15-11-86	20-1-87	21-2-87	27-3-87	12-4-87	19-5-87	22-6-87	30-7-87	20-9-87	
A2	5 2	— 1	2 0	3 0	3 2	-1 0	-3 -1	— —	0 0	
A3	-2 1	— 1	-2 2	-5 3	-2 1	-4 1	-3 1	LOST		
A4	-3 -3	— 0	0 1	1 0	-1 0	1 0	1 0	LOST		
A5	3 -5	— -4	9 0	10 20	10 4	12 1	11 0	LOST		
A6	-2 -10	— -1	1 3	4 4	4 6	2 3	3 0	2 -17	2 -17	
A7	— —	— —	— —	— —	— —	— —	— —	0 -5	-2 -3	
A8	-4 -10	— -26	45 -47	50 -12	52 -49	55 -58	53 -71	53 -76	56 -75	
A9	— —	— —	— —	— —	— —	— —	— —	1 -9	1 -5	
A10	-5 -9	— -23	60 -48	114 -154	LOST					
B2	-1 5	1 0	1 -1	— —	0 -1	-1 -1	LOST			
B3	2 -6	2 -1	2 -1	— —	3 -1	4 0	4 0	LOST		
B4	0 -3	1 -5	4 -5	— —	4 -1	4 -4	4 -2	LOST		
B5	3 -3	2 -4	3 -4	— —	5 -3	5 -4	7 -4	LOST		
B6	— —	— —	— —	— —	— —	— —	— —	0 -3	2 -1	
B7	4 -8	8 0	14 0		19 -1	20 -3	17 -3	20 -6	22 -6	
B8	— —	— —	— —	— —	— —	— —	— —	4 -3	9 -3	
B9	3 -5	13 -1	3 -	— —	34 -2	29 -5	37 -11	43 -18	56 -20	
C2	0 0	-2 -2	1 1		0 1	2 4	1 -1	0 2	-1 0	
C3			-6 -3		-5 -3	-7 -4	-7 -3	-7 -7	-5 -5	
C4	-1 -1	-1 16	1 -4		2 -3	2 -5	2 -1	3 0	2 -1	
C5	1 1	-2 0	1 3		1 1	3 0	3 2	2 2	2 5	
C6	4 7	2 17	3 8		4 9	7 -2	6 -2	3 -1	5 1	
C7								-4 4	-2 4	
C8								-2 3	-2 2	
C9	1 8	4 9	13 13		21 -1	20 -4	24 -4	23 -5	26 -5	
C10	9 8	LOST								
	27-10-87	19-1-88			27-10-88	19-1-88	FIGURE A4:20 APPENDIX E			
A6	1 -11	2 -8		C2	2 -7	1 -1	Table of cumulative ground movements (mm)			
A7	2 -1	7 0		C3	-5 -2	-6 -3				
A8	58 -76	62 -76		C4	1 -3	3 -2				
A9	7 -8	LOST		C5	1 4	3 2	left figure—horizontal movement (+ve outwards) right figure—vertical movement (+ve upwards)			
B6	7 -3	LOST		C6	9 -4	12 -2				
B7	30 -9	LOST		C7	2 -2	7 -1				
B8	17 -8	LOST		C8	7 -3	15 0	for location of pegs see Figures 4.1 and 4.3			
B9	72 -28	LOST		C9	35 -9	LOST				

	16-11-86	16-12-86	21-1-87	22-2-87	27-3-87	13-4-87	17-5-87	24-6-87
D2	4 -1	0 0	0 -1	— —	— —	— —	3 3	2 2
D3	-3 1	0 0	1 0	3 -1	— —	0 -1	-1 -1	-1 0
D4	-5 -2	0 1	4 4	2 4	— —	4 3	7 6	4 5
D5	-16?0	-1 -1	2 1	3 2	— —	3 1	7 0	6 0
D6	— —	— —	— —	— —	— —	— —	— —	— —
D7	-5 -3	4 0	10 4	12 8	— —	11 9	6 9	3 7
D8	— —	— —	— —	— —	— —	— —	— —	— —
D9	-17 -4	-1 -2	5 0	7 5	— —	6 6	5 6	4 5
E2	-1 -2	-2 -1	-2 0	-1 0	-1 0	-1 0	-2 0	-1 -1
E3	0 2	0 1	1 0	0 0	0 0	1 -1	2 -1	-1 0
E4	3 1	2 -1	0 0	1 1	1 1	0 10	1 11	1 9
E5	-2 3	0 -1	-1 11	2 2	2 -1	2 -1	-1 1	-1 0
E6	— —	— —	— —	— —	— —	— —	— —	— —
E7	4 3	3 -7	6 9	4 -1	5 0	10 -9	5 0	4 -1
E8	— —	— —	— —	— —	— —	— —	— —	— —
E9	-5 3	0 -1	2 14	6 5	13 -1	13 6	8 8	9 5
F2	-2 -1	-1 1	-2 1	-2 1	-5 0	— —	4 1	4 0
F3	1 3	-1 1	0 0	-1 0	1 1	— —	1 -1	1 0
F4	0 2	-1 -1	-1 -2	-1 -1	4 -2	— —	3 -3	4 -1
F5	-3 2	8 0	8 4	7 5	10 3	— —	11 5	14 4
F6	— —	— —	— —	— —	— —	— —	— —	— —
F7	1 1	-1 -4	-2 2	-5 1	0 -3	— —	-2 2	-1 2
F8	— —	— —	— —	— —	— —	— —	— —	— —
F9	-6 0	2 -3	4 -3	9 1	15 -4	— —	14 1	16 0

FIGURE A4:20 APPENDIX E

Table of cumulative ground movements (mm)

	30-7-87		20-9-87		27-10-87		19-1-88		29-3-88	
D2	0	-2	2	-3	0	-2	-6	-1	—	—
D3	-1	-2	-3	0	-3	0	-3	0	-3	-1
D4	4	3	4	4	4	3	5	3	5	3
D5	11	-2	8	0	4	0	4	0	4	0
D6	-1	-1	-2	0	-3	-1	-2	0	-2	0
D7	1	6	4	6	10	3	10	7	10	7
D8	-3	1	-1	-1	-4	-4	0	-2	0	-2
D9	3	4	4	2	6	-1	7	2	7	-1
E2	-1	-1	1	-1	-2	-1	-2	0	0	0
E3	0	0	-2	0	0	1	1	0	1	0
E4	1	10	-2	11	0	11	1	11	0	10
E5	0	2	-3	2	-2	2	-1	2	3	1
E6	-1	0	-1	1	-1	-2	0	0	3	-3
E7	4	-1	0	-2	5	-3	5	0	8	-1
E8	-2	3	-5	3	0	-2	0	-3	5	-5
E9	8	7	3	6	7	1	7	5	15	4
F2	3	1	3	1	2	0	0	1	2	-1
F3	1	-1	1	-2	2	0	4	-1	6	-2
F4	5	-2	4	-2	4	-1	6	-2	6	-2
F5	16	1	17	0	17	2	15	2	14	3
F6	6	-2	4	-3	1	-4	0	-3	1	-2
F7	-1	0	-1	-1	0	-1	-1	0	0	-1
F8	-3	1	-3	0	-1	-3	3	-2	6	-1
F9	14	0	16	0	19	-4	22	-3	29	-2

FIGURE A4.20 APPENDIX E
Table of cumulative ground
movements (mm)

PÉG	DISTANCE FROM CLIFF (m)	(mm) HORIZONTAL MOVEMENT	(mm) VERTICAL MOVEMENT	HORIZONTAL STRAIN INCREMENT (%)	DISTANCE CALCULATED OVER (m)
Line B before failure (Sept 1986-Mar 1987)					
B9	3	20	-2	0.7	3-6
B7	6	10	-1	0.3	6-10
B5	10	0	0	—	—
Line B after failure (Mar 1987-Oct 1987)					
B9	0	34	-26	1.6	0-1.5
B8	1.5	10	-8	0.4	1.5-3.0
B7	3	4	-8	0.2	3.0-4.5
B6	4.5	2	-3	0.1	4.5-8
B5	8	0	0	—	—
Line A before failure (Sept 1986-Feb 1987)					
A10	3	39	-30	0.2	3-6
A8	6	32	-30	1.3	6-8.5
A6	8.5	0	0	—	—
Line A after failure (Feb 1987-Jan 1988)					
A10	0	47	-125	0.8	0-3
A8	3	22	-47	1.1	3-5
A7	5	0	0	—	—
Line E (no failures occurred)					
E9	3	12	0	0.4	3-6
E7	6	0	0	—	—

Figure A4.26 Appendix E

Table of stress relief movements and horizontal ground strain increments

APPENDIX F

F.1 Detailed Description of Changes at Rolston

A detailed account of slope erosion and degradation at the Rolston site is given here. The changes are described with reference to the areas shown in Figure 4.3. The cliff edge and toe position are shown for July 1986 and July 1987. The cliff top has receded from 2-8m at an average rate of 2.4m per year, while the toe has remained near stationary. The level of the beach has fluctuated up to 2m during the study but has been generally low because the site is in the zone of depleted sediment south of Hornsea. Particularly low levels occurred in August 1986 and May 1988 when the till shore platform was exposed. Erosion of the toe indicated by the presence of "armoured mudballs" on the beach, was most severe during periods of storms. The changes are now described for each of the four areas shown on the plan (Figure 4.3).

F.1.1 Area 1

A slip had recently occurred at the ridge in front of line "C" prior to the study in July 1986. It rapidly degraded and after one year had reached beach level. The backscarp degraded by small falls of weathered clay which formed a debris slope, loading the top of the slip. A mudslide of 4m width developed

on the north side of the slip, activated by seepage from a sand layer. The cliff top was unstable above the mudslide and subsided with formation of tension cracks. Overall recession of the clifftop was 7m in 18 months.

F.1.2 Area 2

A large slip occurred between lines "B" and "C" (slip "B") in March 1987 and its subsequent history is outlined in Figure A4.34 (overleaf).

F.1.3 Area 3

A slip of the weathered till in the cliff top occurred in January 1987. An area of 20m length and 2m width was displaced 2m downslope. The toe of the slip was not visible but probably emerged in debris from falls about mid-height in the slope. A slip occurred to the north of line "A" in February 1987 of 20m length, and 3-4m width, and initial displacement of 0.1 to 0.4m. The toe of the slip emerged at the foot of the cliff. Tension cracks opened in the face of the slipped mass, forming blocks which could then fall. As a result the slip was quickly degraded and was unrecognisable two months later. A failure of the weathered clay in the backscarp occurred in June 1987 of 11m length and 1m width.

<u>Date</u>	<u>Event</u>
March 1987	Clifftop tension cracks and a backscarp of up to 40cm height developed, enclosing an area 38m long and 3-4m wide.
April 1987	Backscarp height varies from 1m at south end to 2m at north. Slipped ledge of cliff top began to break up.
June 1987	Backscarp was 2-3m in height and horizontal displacement 2-3m. Secondary failure of the northern half of the slipped block occurred.
July 1987	Northern half of slip fell 3m while southern half remained stationary. Northern half became broken up and a mudslide activated by groundwater seepage removed debris.
Autumn '87	Continued mudslide activity entirely removed northern half of slip. Falls of backscarp thus undermined caused 5m recession of cliff top.

Figure A4.34 Summary of slip "B" changes.

F.1.4 Area 4

A slip occurred in April 1987 (slip "R") centred on the road end, of 25m length and 6m width, with the toe emerging at the foot of the cliff. The initial displacement was 0.2m, which increased to 0.3m in May. In June this increased rapidly to 4.5m, and the lower part of the slipped mass became degraded in the same way as the slip described in Area 3. During the Autumn of 1987 the backscarp collapsed and the cliff edge receded 2-3m.

F.2 Changes in Sections of Cliff Re-Examined in 1987

This section describes the changes in eight areas which were re-examined one year after completion of the survey (see Section 3.3.3).

ATWICK - Little change. Major cliff features are recognizable. Slight erosion has occurred at the toe, and mudslides have been active, fed by groundwater from sand lenses.

SOUTH HORNSEA - Large change. Several slips and debris have been removed to leave "till cliff" morphology. Other slips have been degraded by erosion at the toe.

ROLSTON-MAPPLETON - Large change. Slips and debris have been removed to form continuous "till cliff" morphology at steep angle.

COWDEN - Some change. A few recent slips have occurred and degraded ones have been entirely removed by erosion, but overall the morphology is unaltered.

ALDBROUGH - Some change. Degraded slips and debris have been eroded and new slips have occurred near Aldbrough road end.

NORTH GRIMSTON - Little change. The section has been protected by a high upper beach.

SOUTH GRIMSTON - Some change. The beach has been lowered by an "ord", and falls have occurred at the cliff foot. Degraded slips have been further displaced and recent slips have occurred.

TUNSTALL - Little change. Occasional recent slips have occurred and some degraded slips have been removed, but the cliff morphology is unaltered.

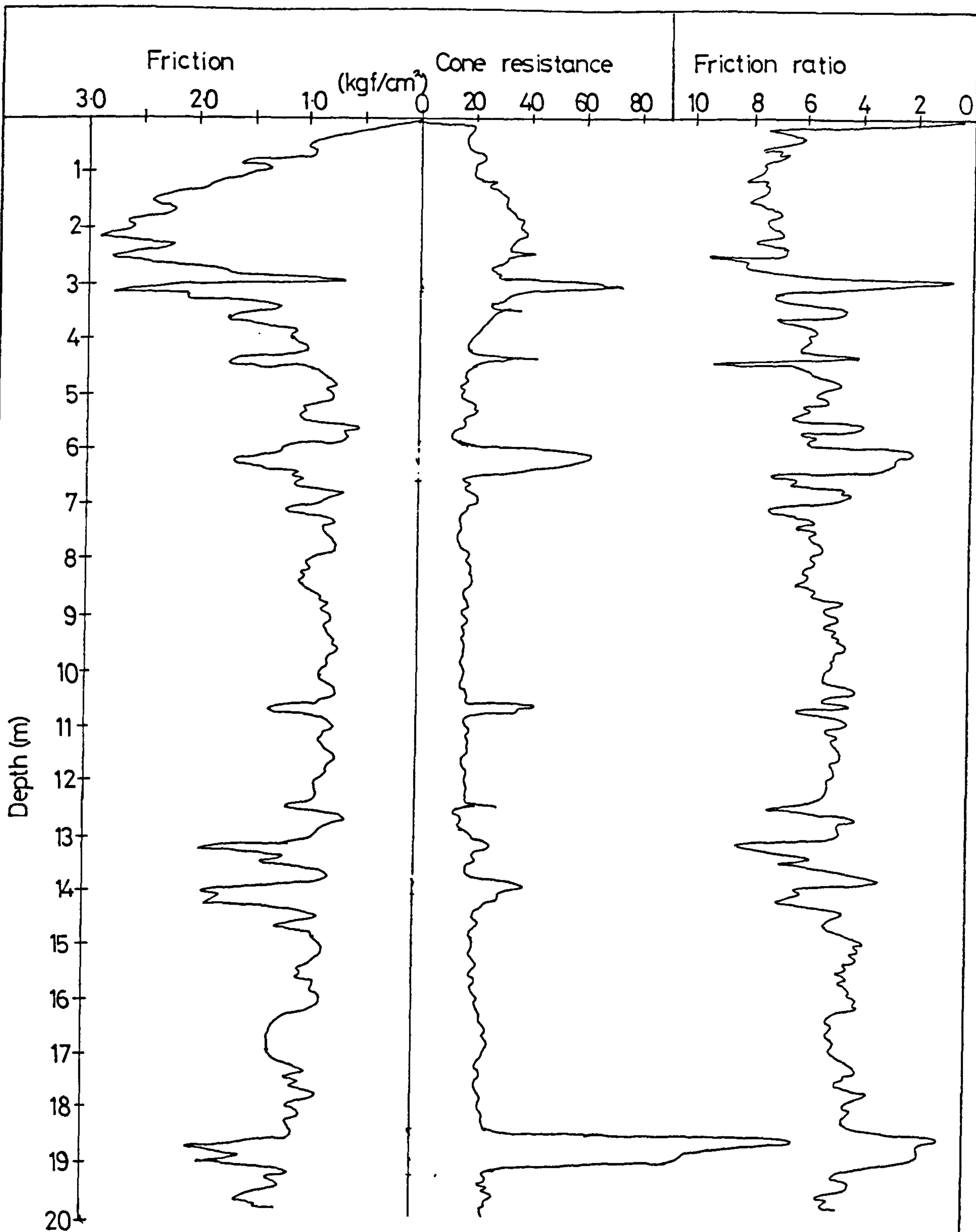
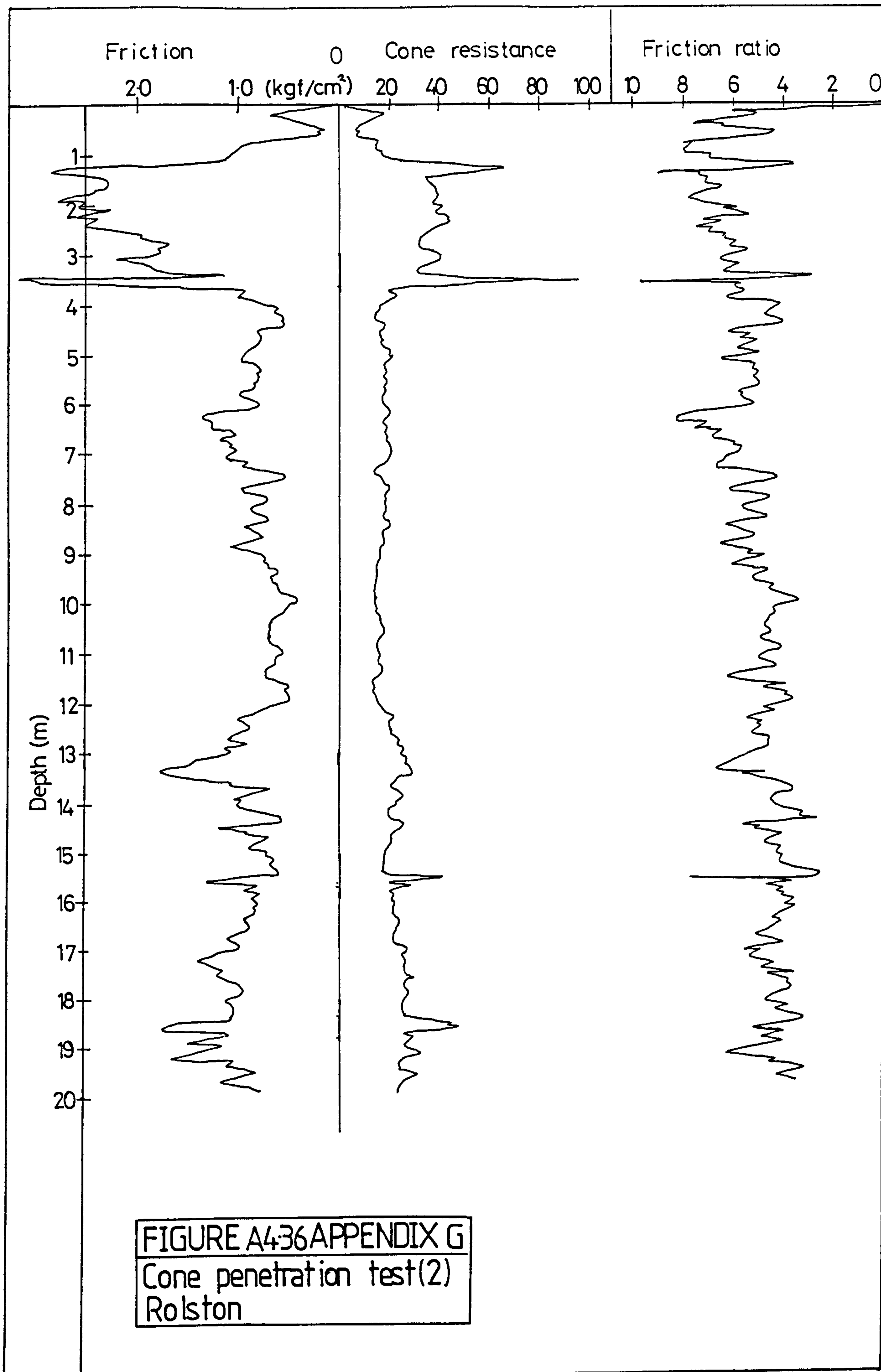


Figure A4-36 Appendix G
Cone penetration test (1)
Rolston



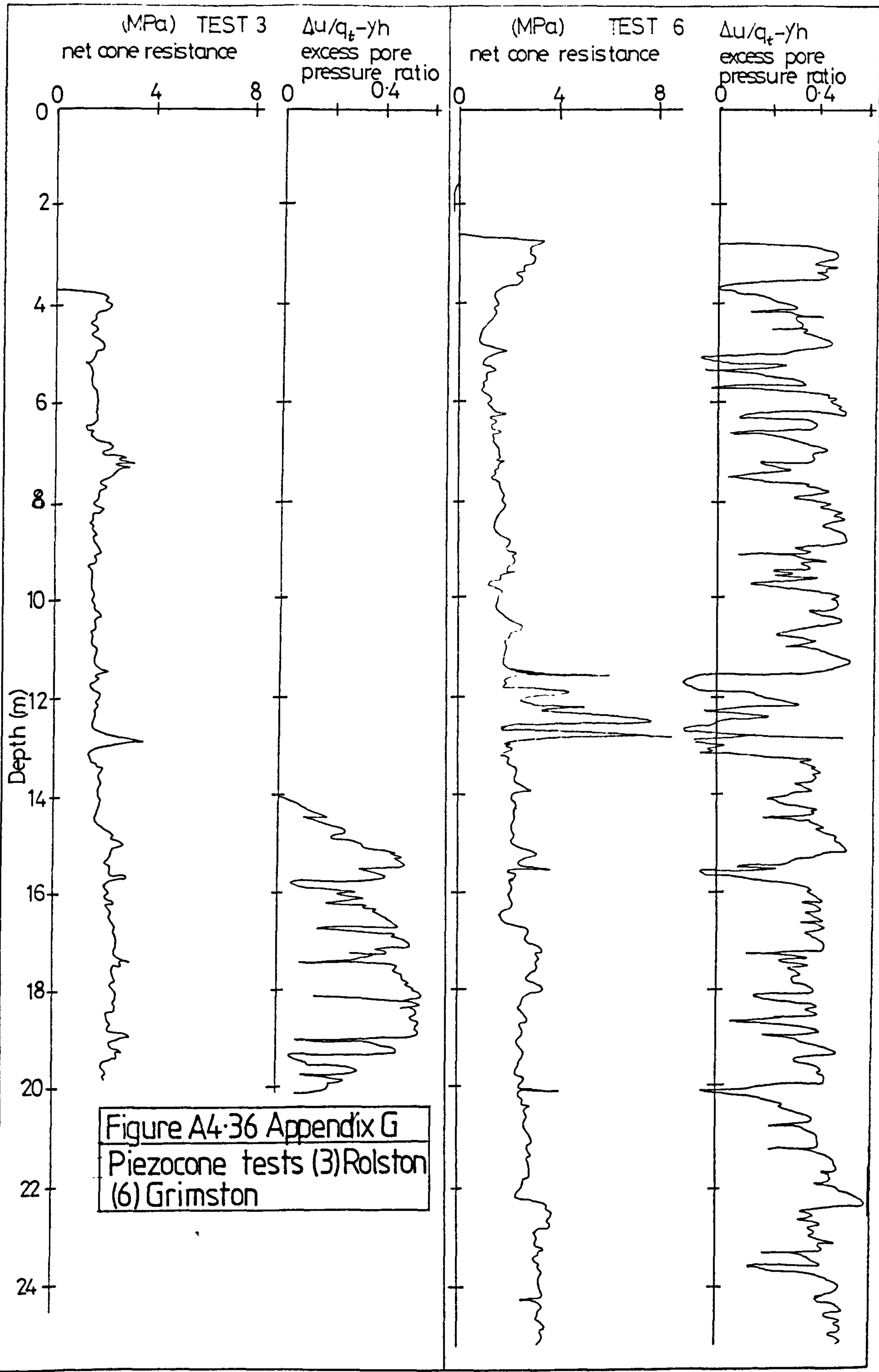


Figure A4.36 Appendix G
Piezocone tests (3) Rolston
(6) Grimston

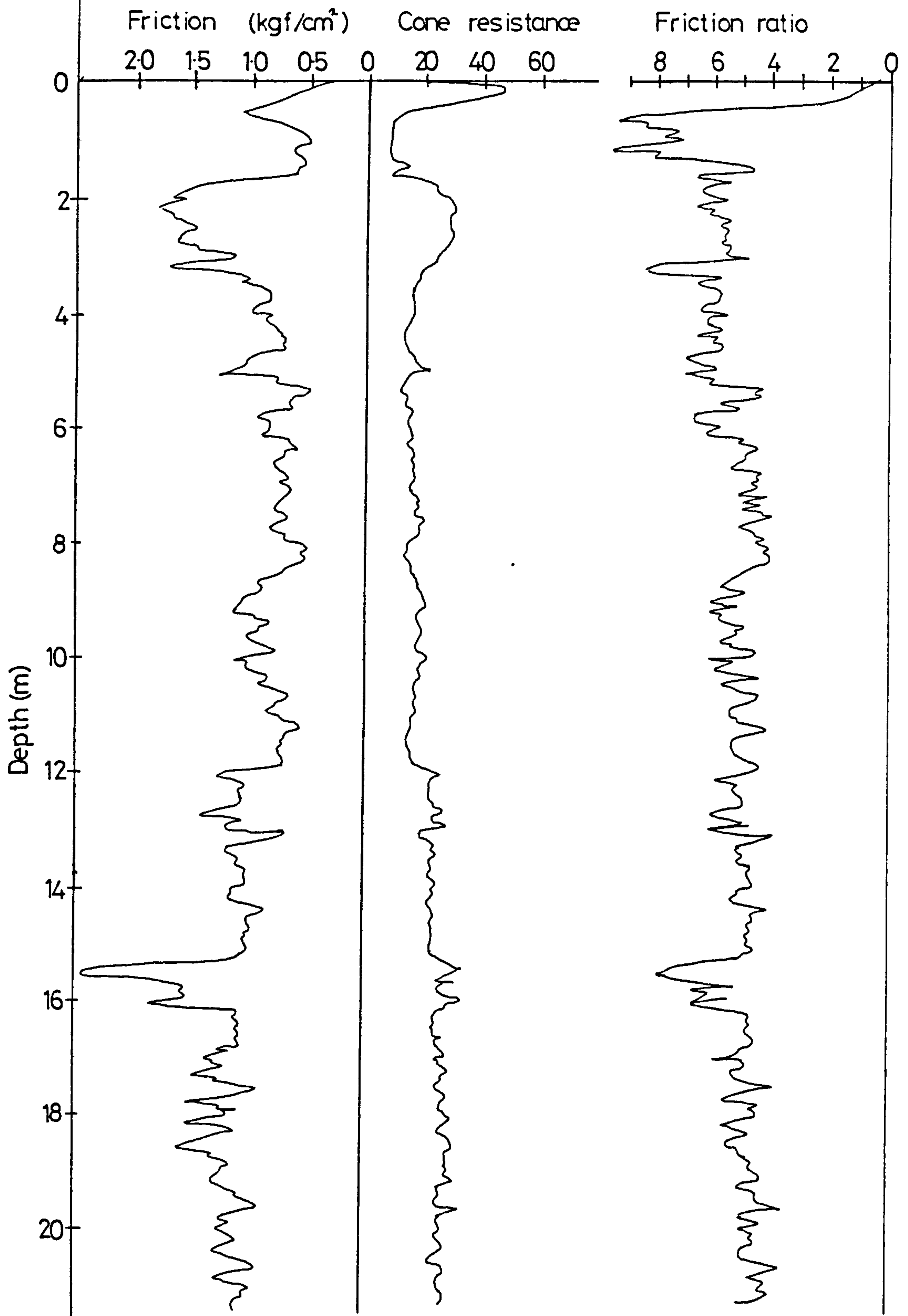
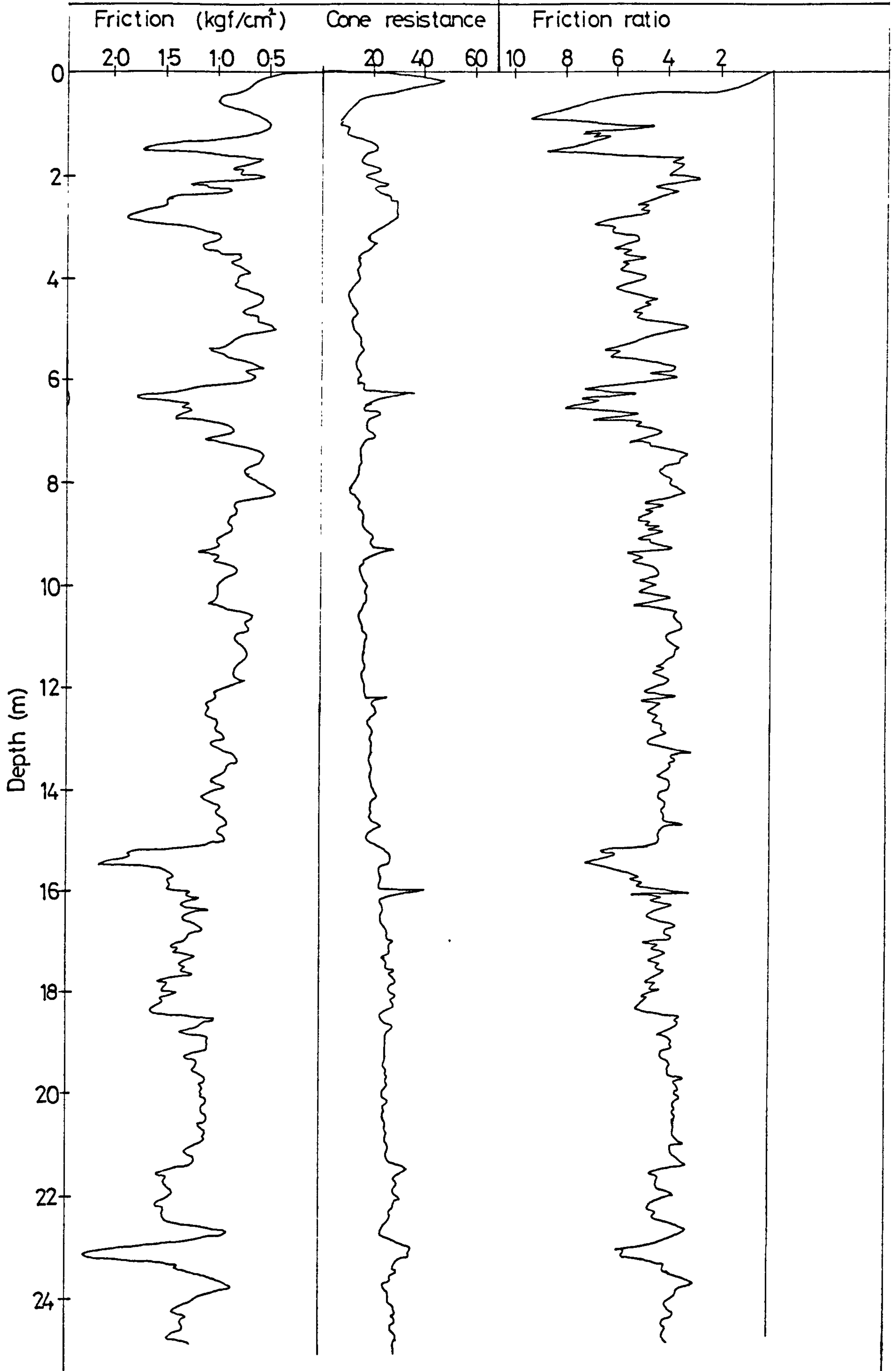


Figure A-36 Appendix G
Cone penetration test (4)



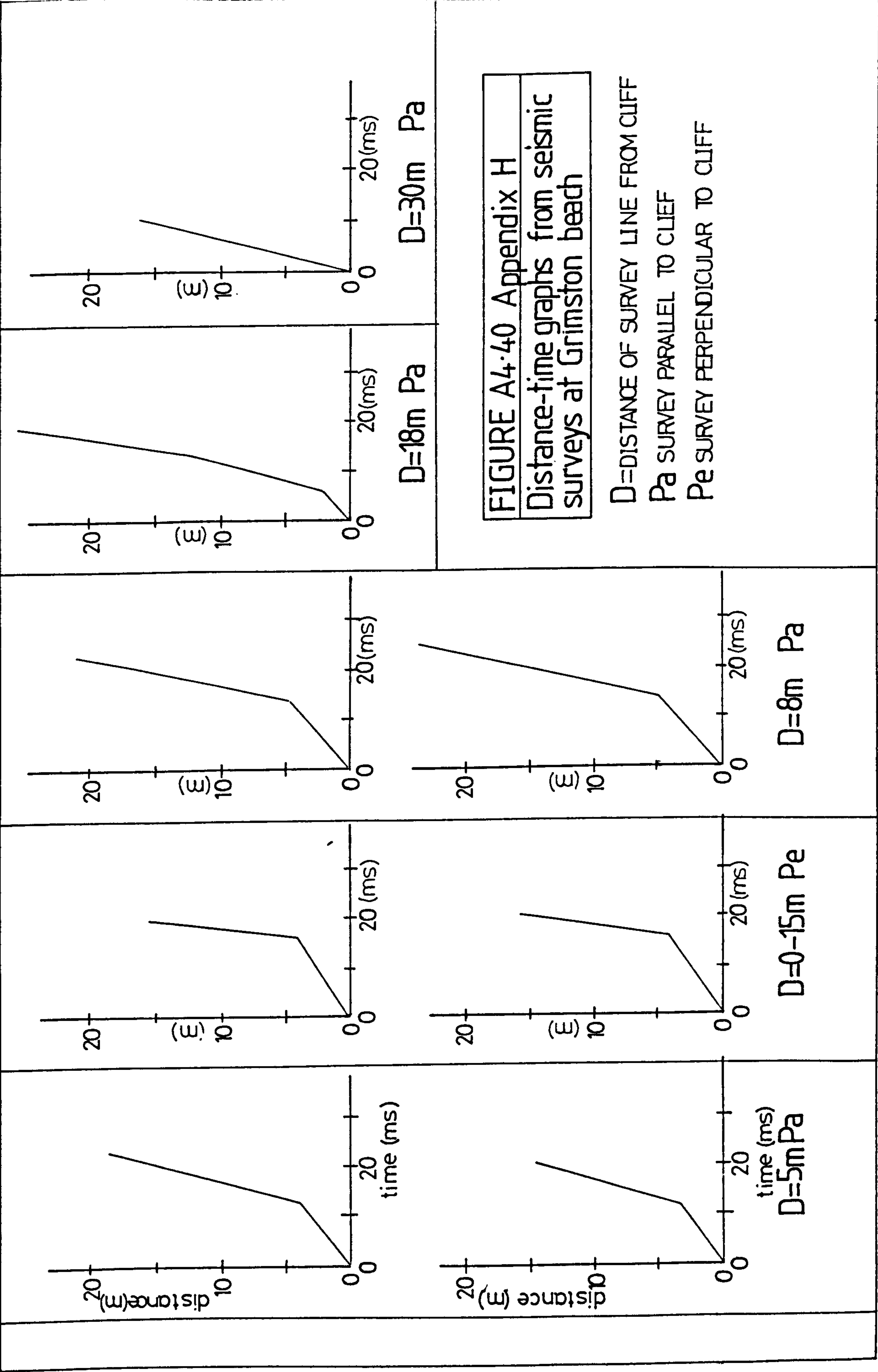


FIGURE A4.43 APPENDIX I

Curves of void ratio plotted with \log_{10} vertical effective stress

Depth of sample in brackets
SAMPLE 6/27 (15.8m)
(Skipsea till)

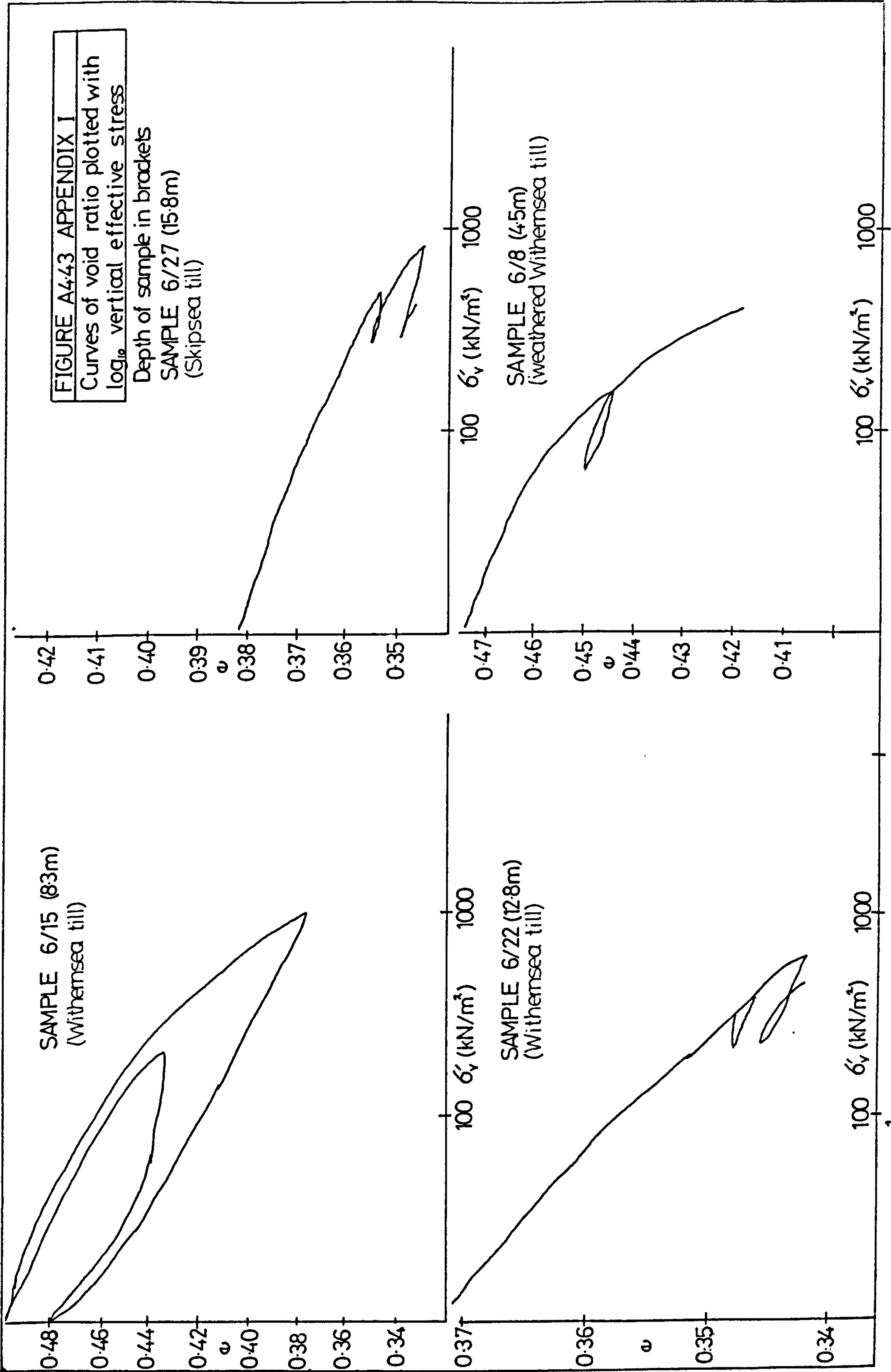
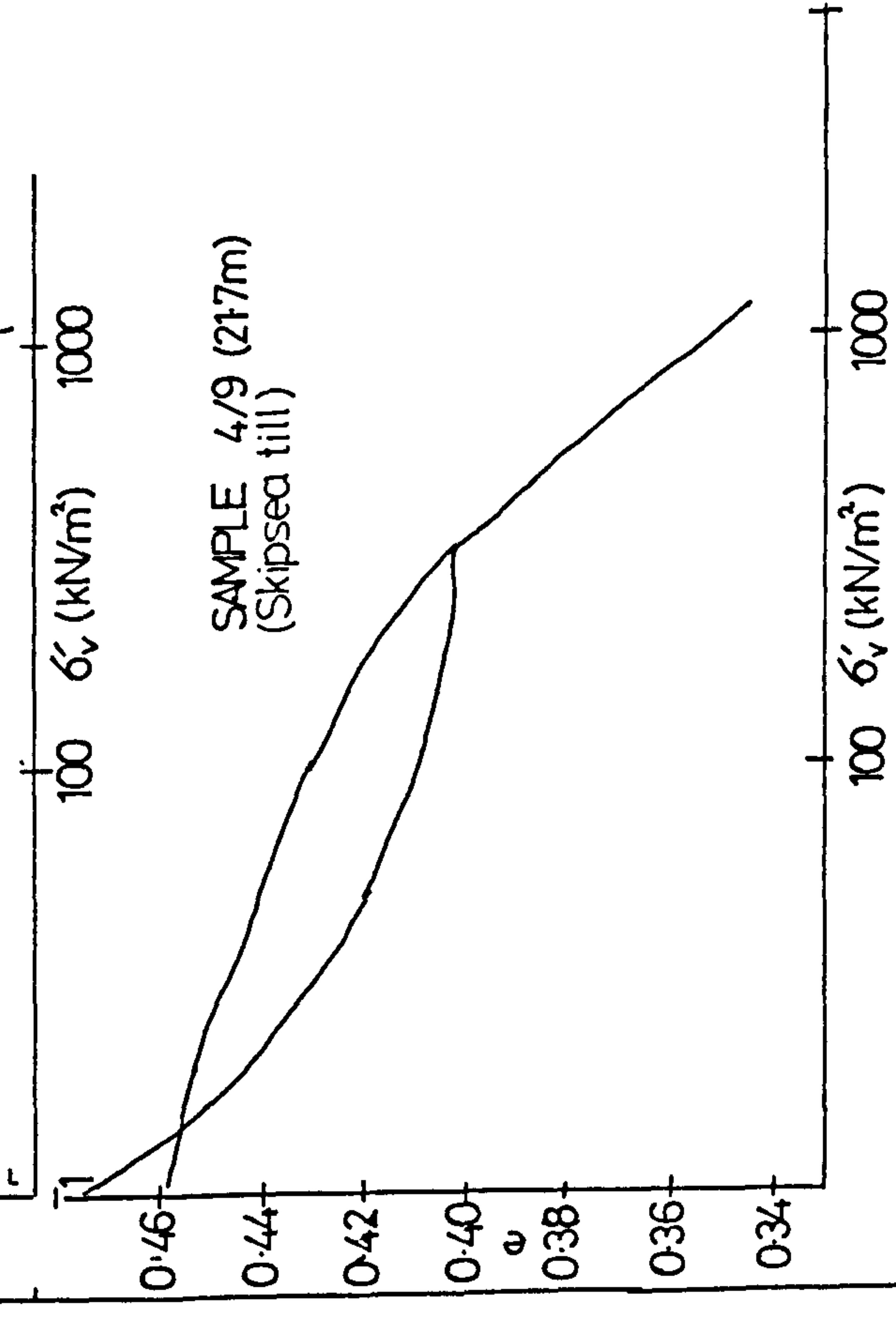
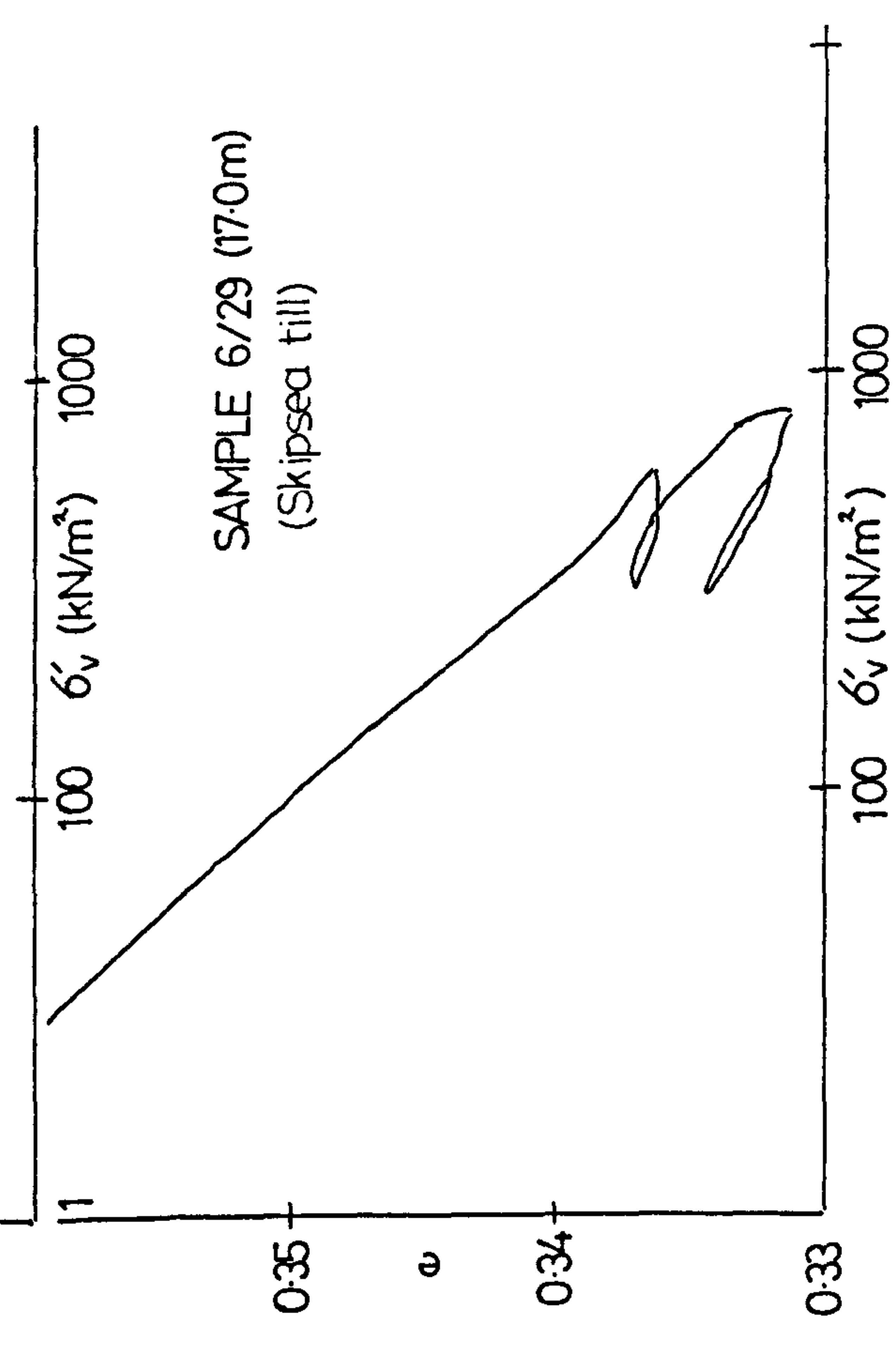
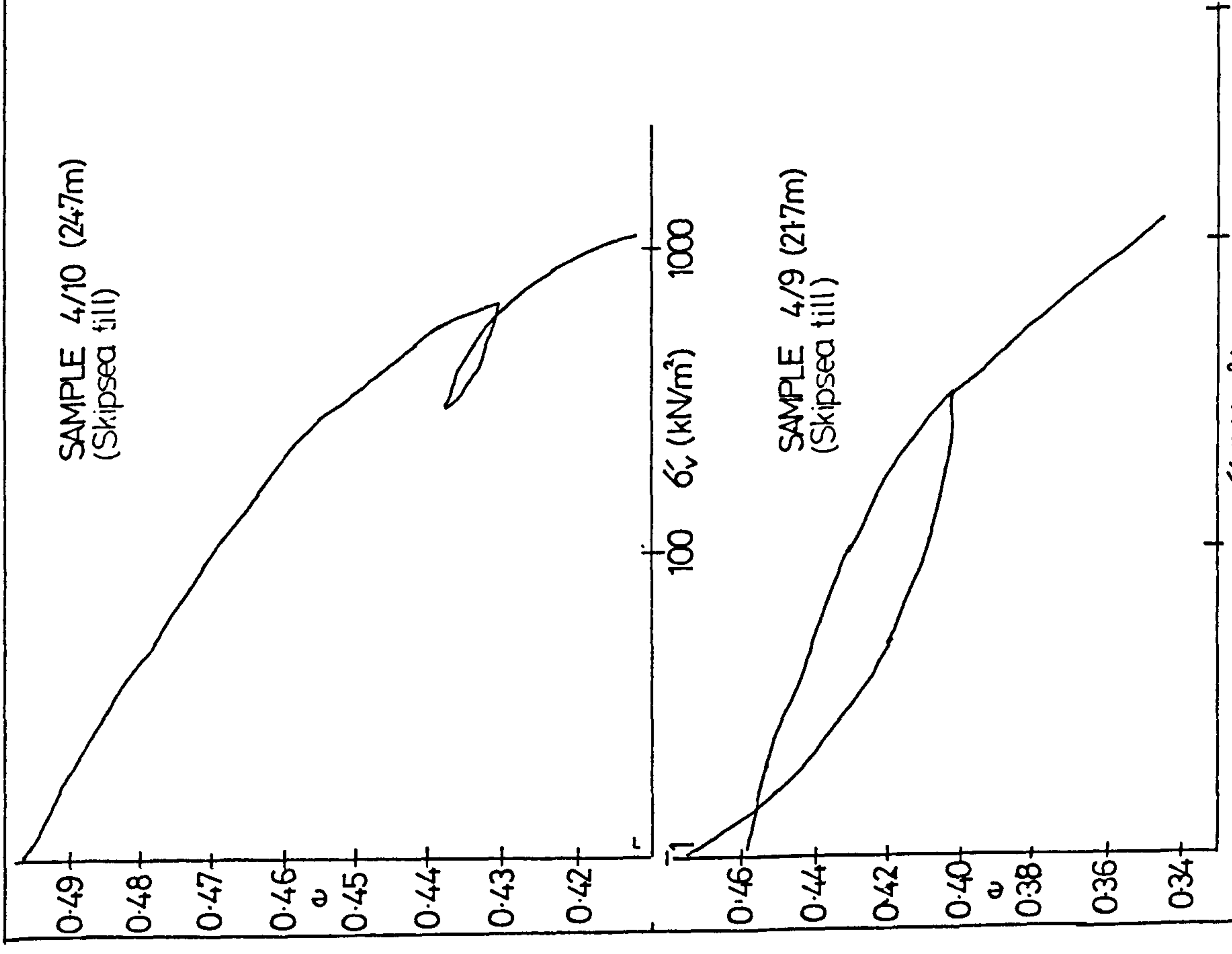
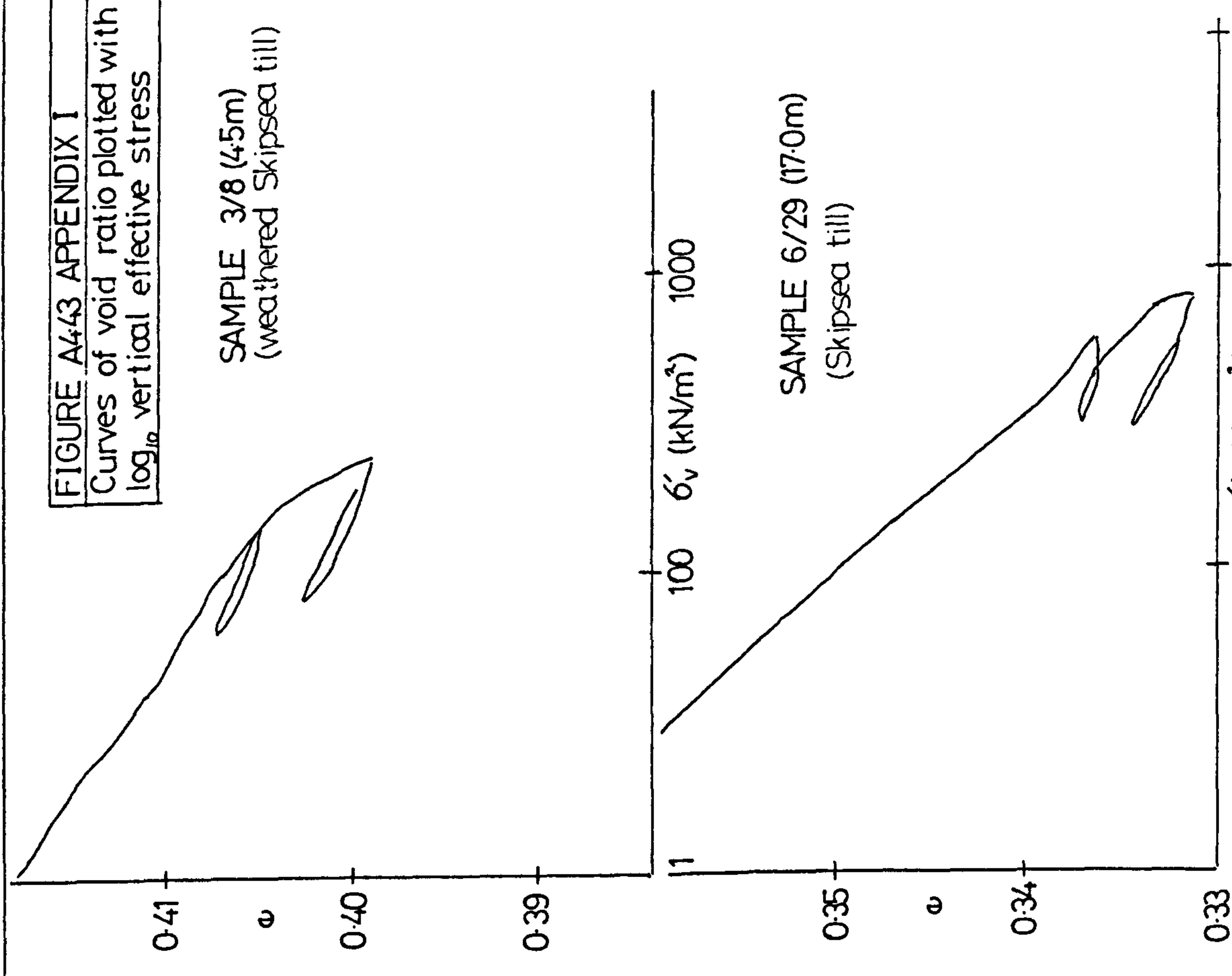


FIGURE A443 APPENDIX I
Curves of void ratio plotted with
 \log_{10} vertical effective stress



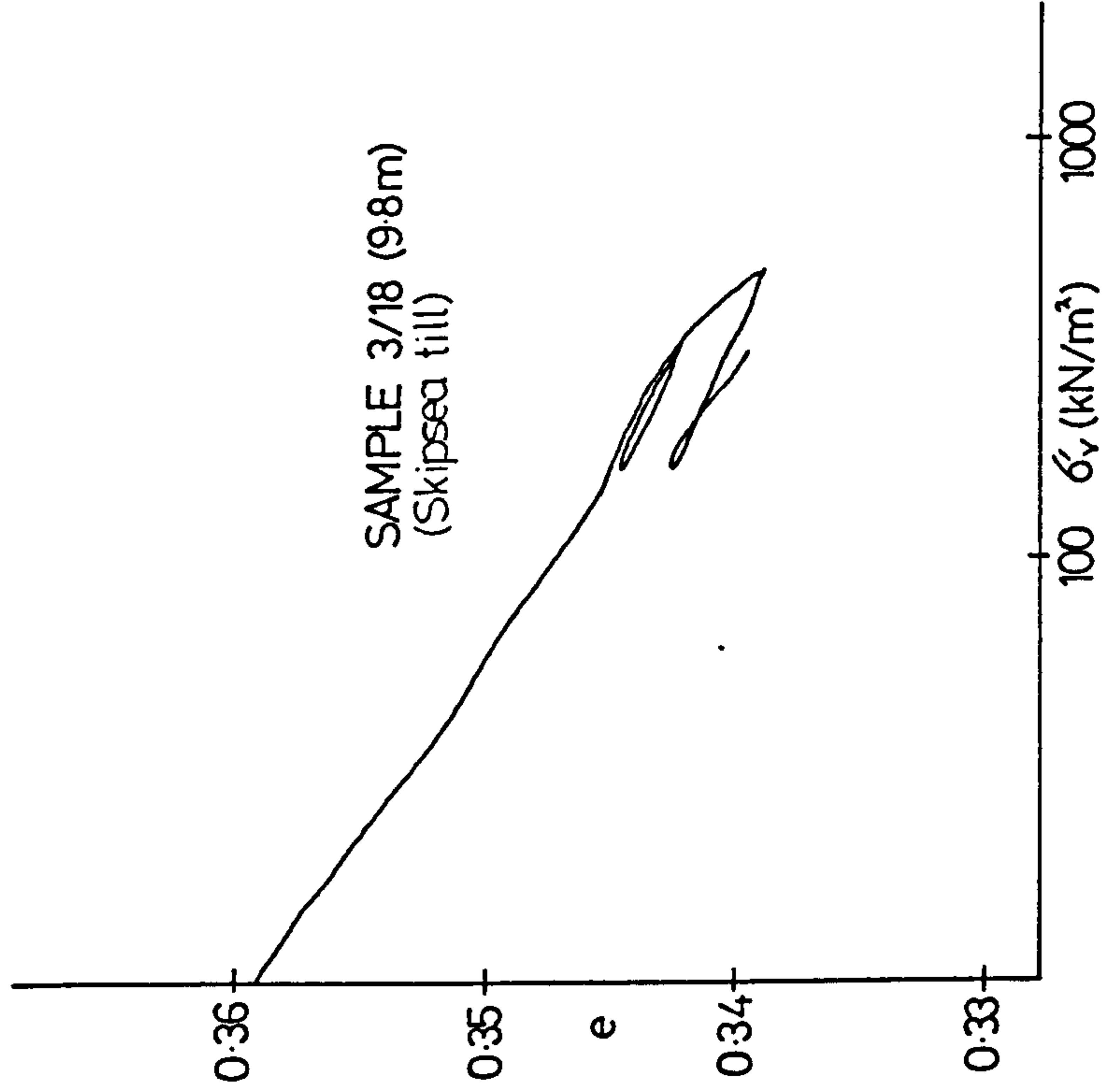
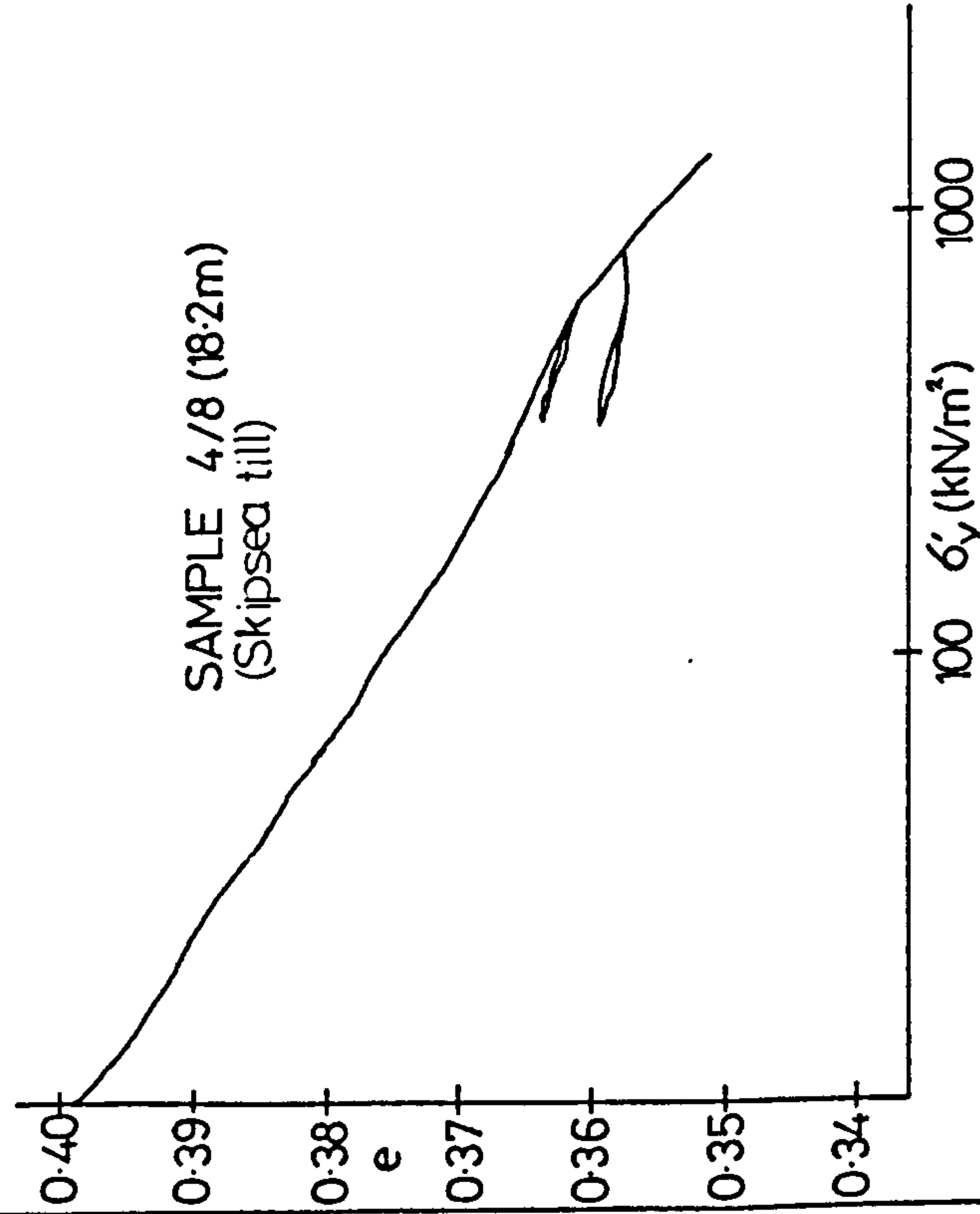


FIGURE A4-43 APPENDIX I
Curves of void ratio plotted with
 \log_{10} vertical effective stress

BOREHOLE SAMPLE TILL		DEPTH	c'	ϕ'	B	A_F	c'	ϕ'
		(m)	(kN/m ²) ($^{\circ}$)		(kN/m ²) ($^{\circ}$)		from stress paths	
			from Mohr circles					
3	20	SKIPSEA	11.3	12	26	0.87 0.92 0.98 0.98 0.97 0.97	-0.12 0.06 0.30 0.00 -0.01 0.18	0 25
3	22 [*]	SKIPSEA	12.3	8	26	0.98 0.97 0.97	0.00 -0.01 0.18	7 26
1	9	SKIPSEA	17.3	25	25	0.90 0.89 0.92	-0.11 -0.21 0.27	7 28
6	31	SKIPSEA	18.3	40	19	0.86 0.90	0.08 0.10 -0.53	0 25
4	5 [†]	WITHERNSEA	10.3	5	27		-0.01 -0.06 0.23	9 26
6	23	WITHERNSEA	13.8	27	26	0.86 0.84	-0.06 -0.10 -0.32	29 25
3	9	WEATHERED SKIPSEA	5.7	11	24	0.83 0.81 0.85	0.07 -0.16 0.22	13 25

* Tests carried out by I Robertson (other tests by MSc students)
† Tests by E Yeo

FIGURE A4.50 APPENDIX J
TABLE OF 'STANDARD' TRIAXIAL TEST RESULTS

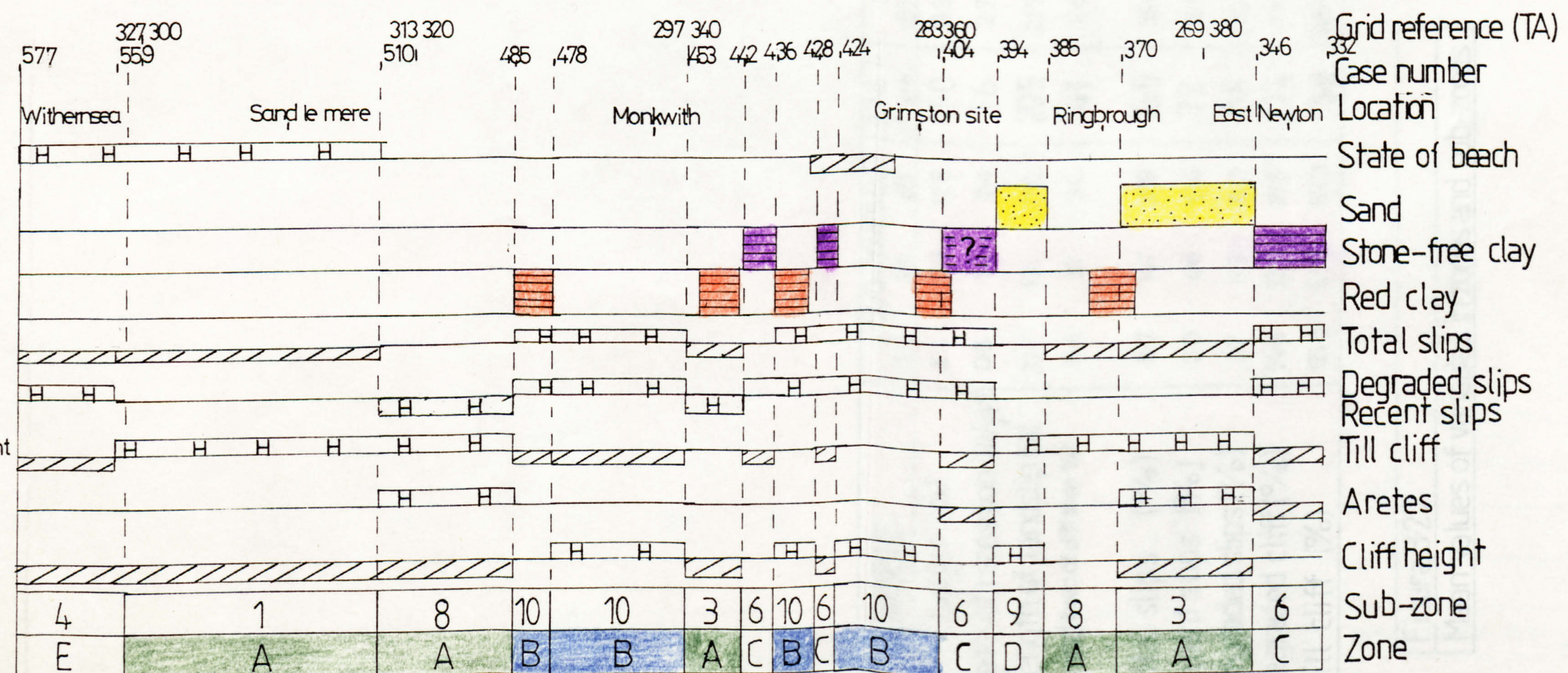
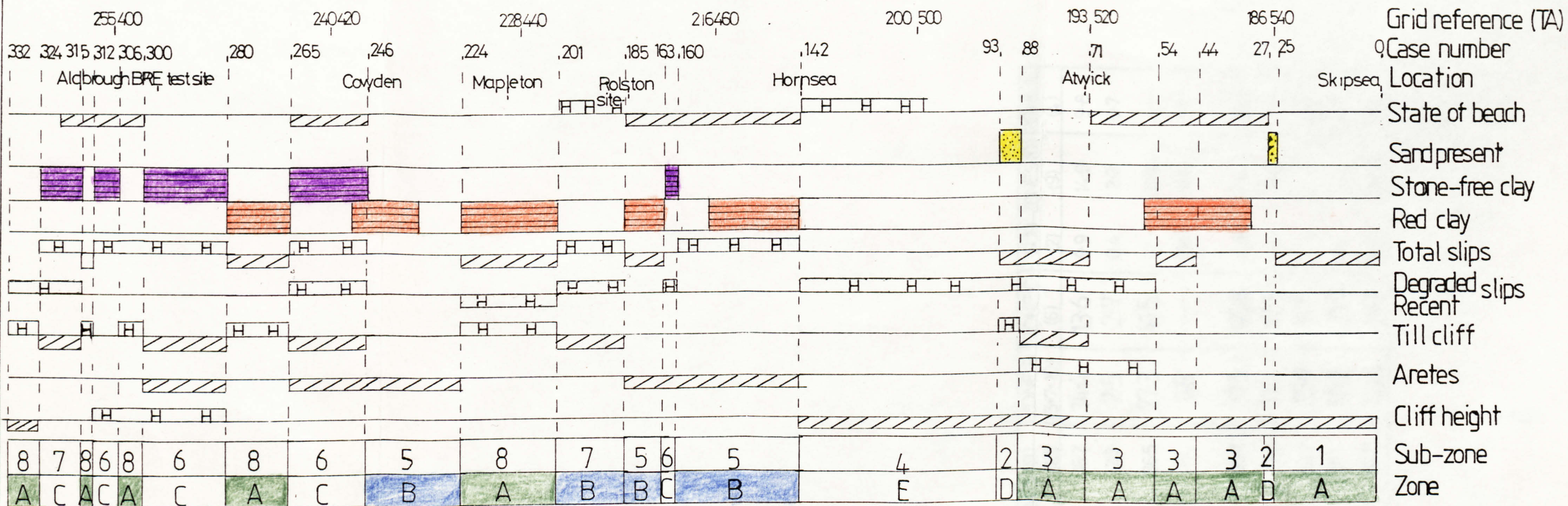
BOREHOLE SAMPLE TILL		DEPTH		c'	ø'	c	ø	w	B	σ ₃ failure	u failure
		(m)	(kN/m ²) (°)	(kN/m ²) (°)	(°)	(%)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)
		Mohr circles		stress paths							
1	6	SKIPSEA	11.3	4	17	8	17	18.9	0.89	145.2	5.5
								19.5	0.92	88.3	-23
								19.1	0.94	244.5	42
								17.8	0.92	172.3	-41.6
3	24	SKIPSEA	13.5	0	25	0	26	19.2	0.87	143.1	13.7
								19.6	0.90	83.6	-68
								18.3	0.88	126.3	18.3

Both tests carried out by I Robertson

FIGURE A4.57 APPENDIX J
TABLE OF 'REDUCTION' TRIAXIAL TEST RESULTS

Kilnsea old church, ere the wild sea took,
Cliff, church, God's acre all to its breast,
In the ardour of its fierce unrest,
Twenty years ago I trod the sand,
In summer, one of a gay young band,
Just to see, at lowest ebb of tide,
Two or three stones by the sea marge wide,
Never dry, sand buried, seldom seen,
Last worn relics where the church had been.

E. Lamplough



VARIABLE	SUB-ZONES (A)		ZONE A	SUB-ZONES (B)		ZONE B	ZONE C	SUB-ZONES (D)		ZONE E
	(1)	(3)	(8)	(5)	(7)	(10)	(6)	(2)	(9)	(4)
Cliff height (m)	8.8	10.9	12.8	13.3	11.4	16.2	13.6	9.9	14.9	9.8
Rate of recession (m/yr)	0.8	1.4	2.4	2.7	2.7	2.3	2.7	0.4	2.3	0.7
Length of landslide (m)	20.7	19.1	21.0	21.2	26.7	36.5	41.5	—	39.2	—
Sand (area of cliff face %)	5.6	3.7	3.4	0.6	1.2	1.2	—	29.5	9.6	1.2
Total slips (%)	9.7	14.7	20.9	35.6	51.0	35.7	48.8	6.8	—	19.7
Recent slips (%)	6.0	4.8	10.6	15.5	8.6	9.0	17.1	—	15.1	—
Degraded slips (%)	3.7	9.9	10.5	20.1	43.4	26.7	31.7	—	—	—
Degraded cliff (%)	34.6	33.1	15.9	19.3	17.1	17.1	39.9	41.4	—	56.4
Till cliff (%)	55.5	51.9	62.9	45.0	31.8	47.0	19.3	51.8	68.4	23.8

Figure 3.27

Mean values of variables in zones and sub-zones

CONTAINS PULLOUTS

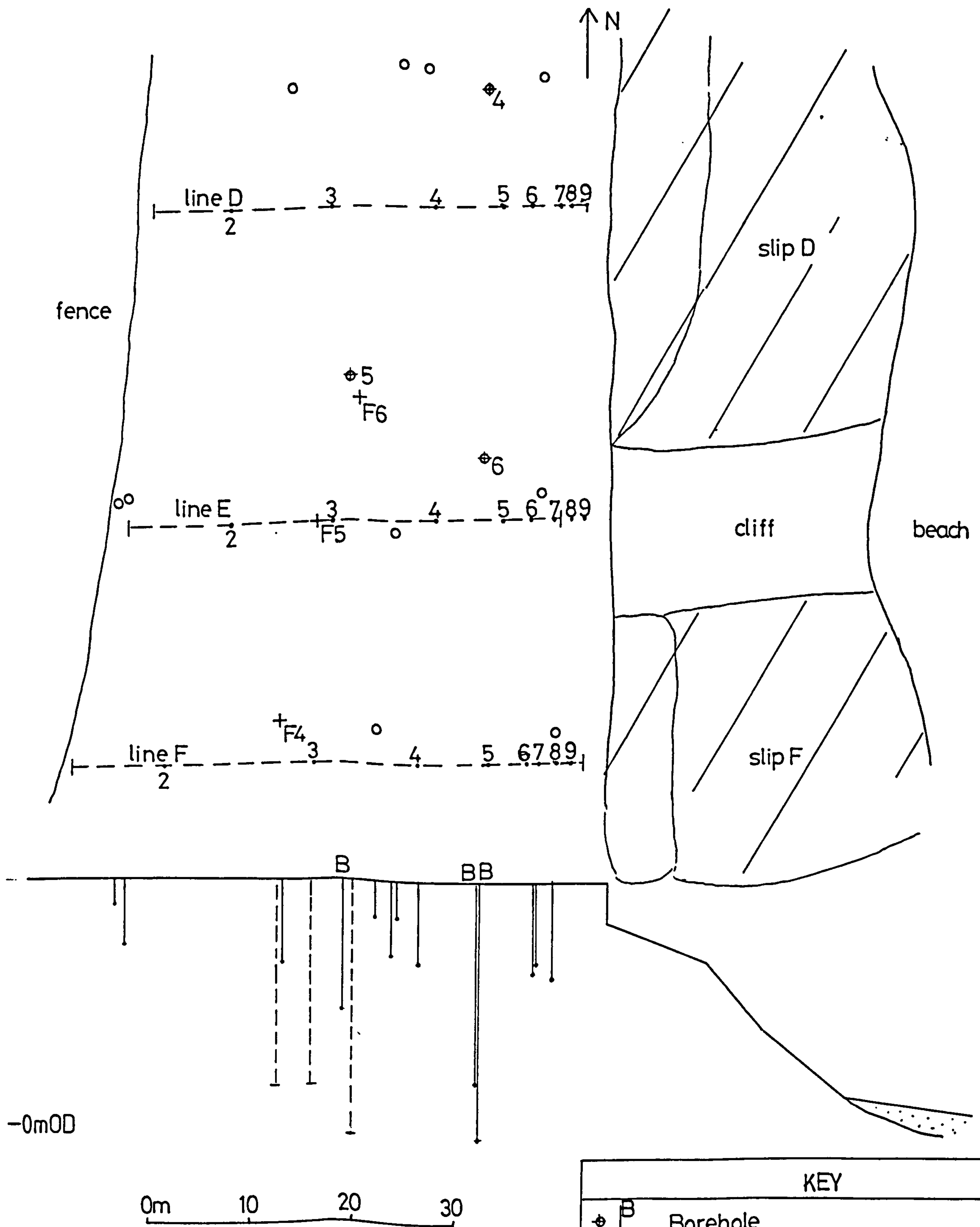


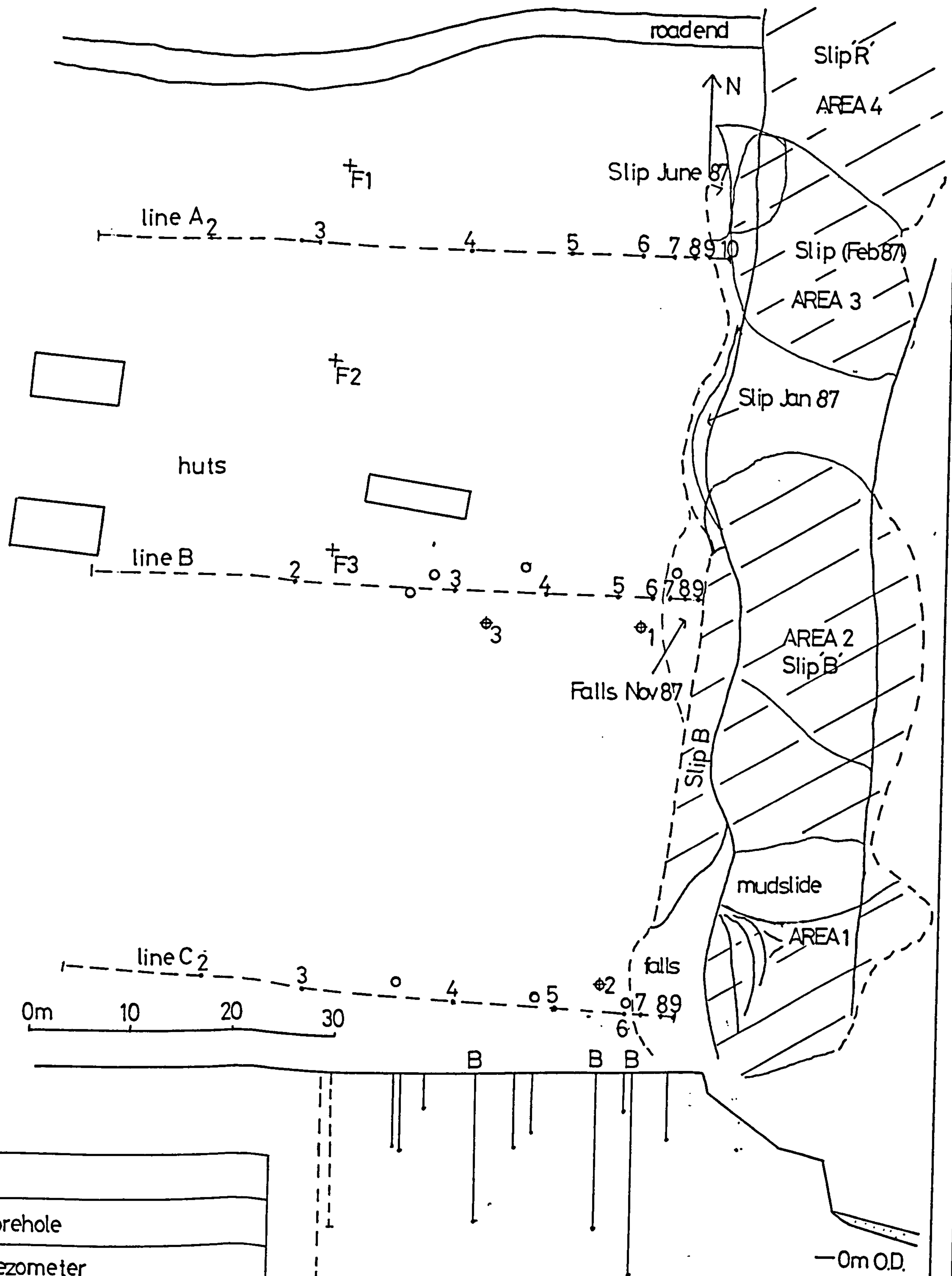
FIGURE 4.1

GRIMSTON SITE PLAN AND CROSS SECTION

JULY 1986 SCALE 1:500

KEY

Φ B	Borehole
o	Piezometer
+ F	Static cone penetrometer test
---	Line of steel survey pegs




KEY	
Position of cliff — July 1986 - - - July 1987	⊕ ^B Borehole
	○ Piezometer
	+ _F Static cone penetrometer test
 slip	- - - Line of steel survey pegs

FIGURE 4.3

ROLSTON SITE PLAN AND CROSS SECTION

JULY 1986 SCALE 1:500

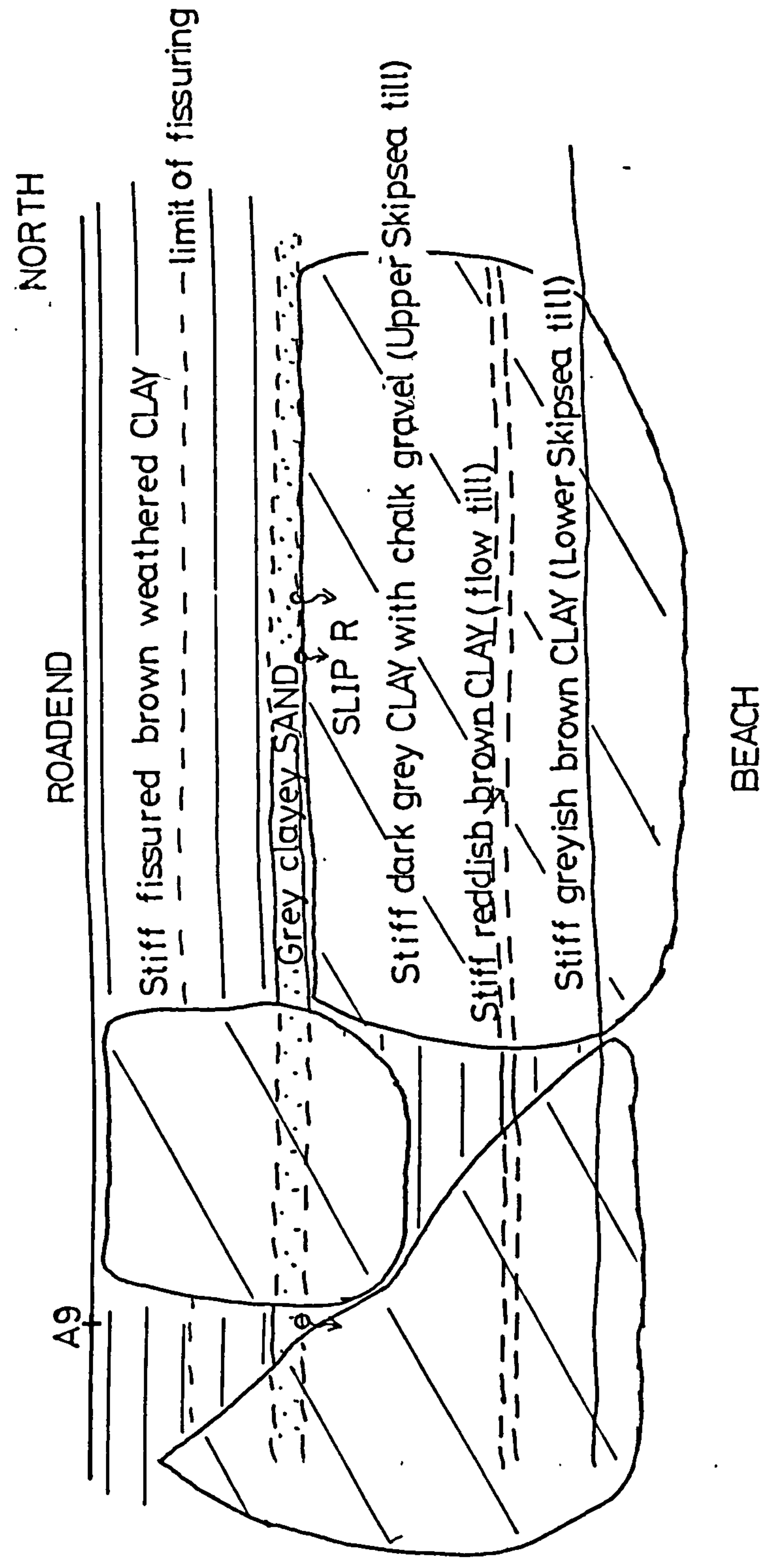
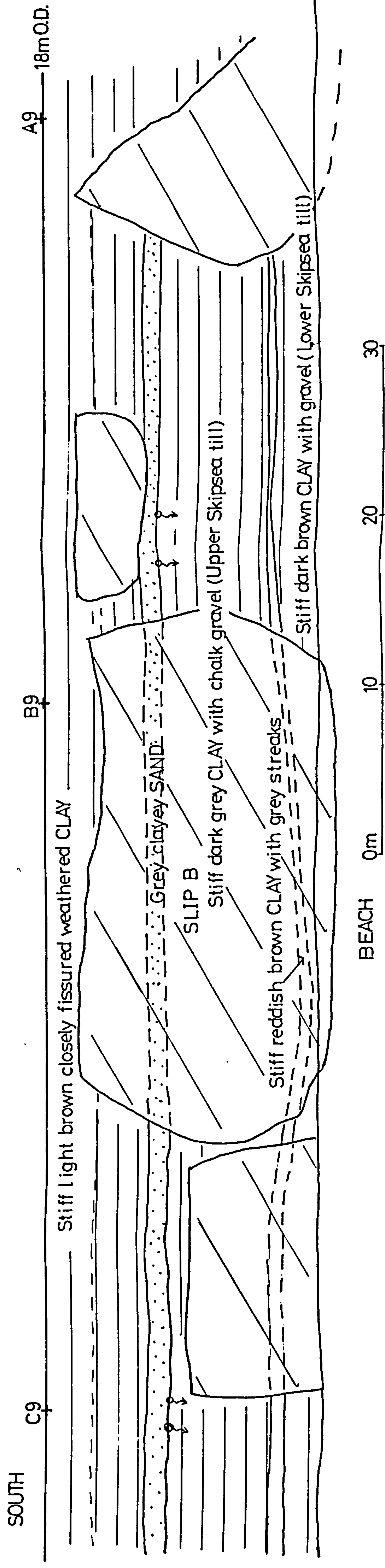
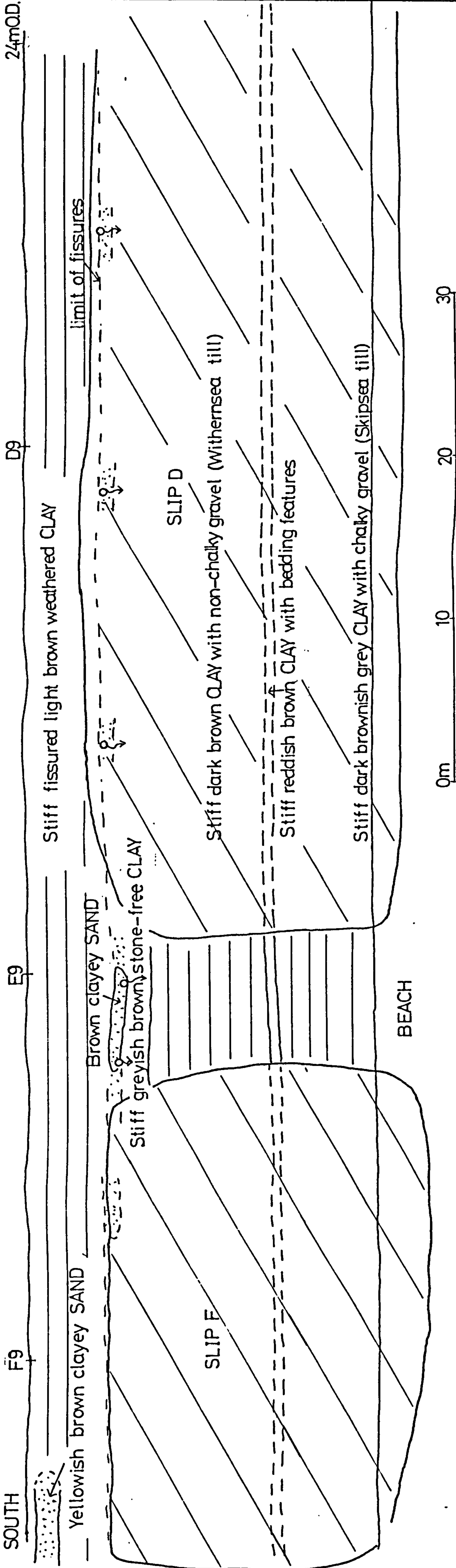
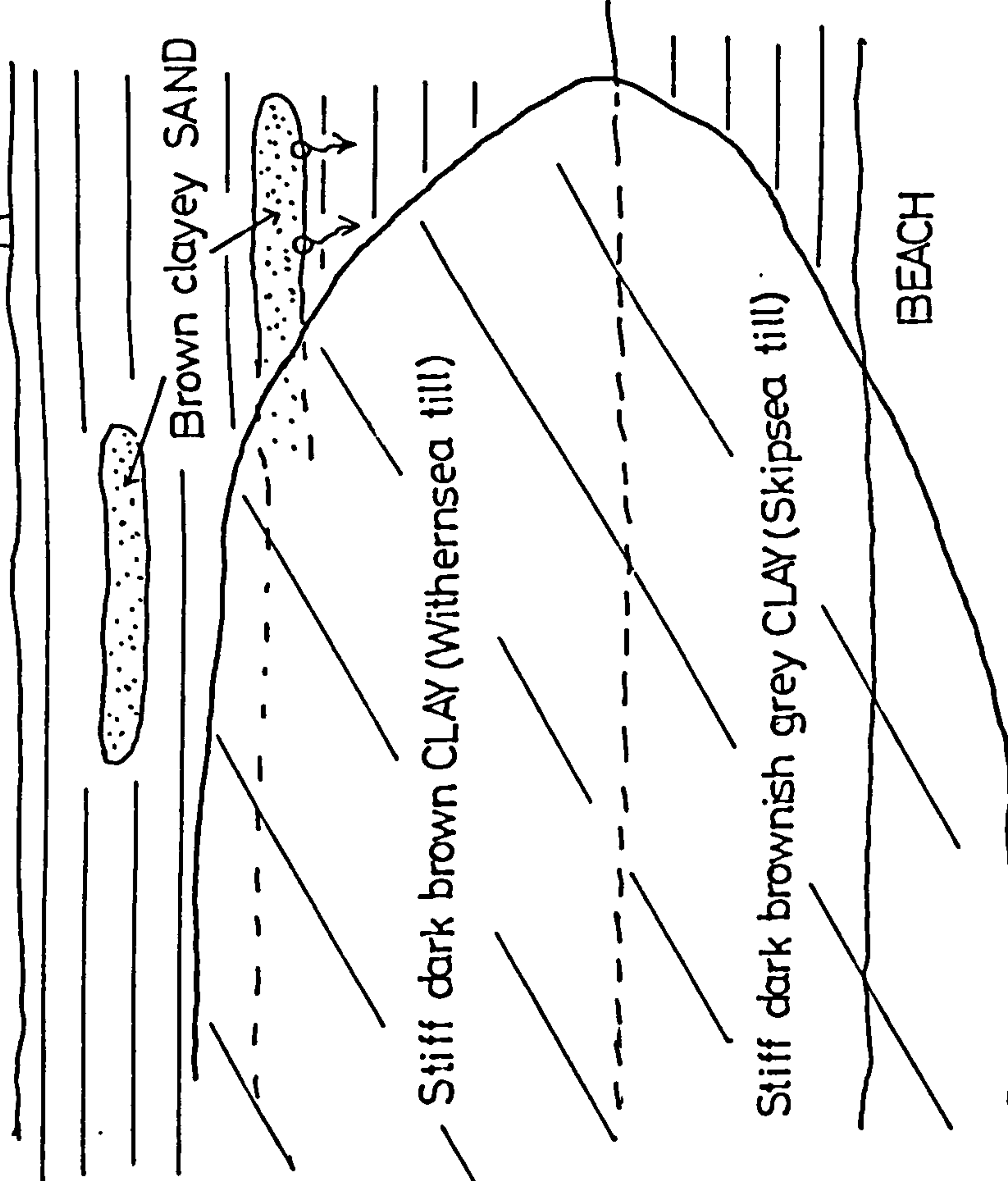


FIGURE 4-5
PLAN OF ROLSTON CLIFF FACE
JUNE 1987 SCALE 1:250

Cont. below→



Hoist NORTH








KEY	
	Slip
	Seepage
	Geological boundary certain
	uncertain
	Steel reference pegs

FIGURE 4.6
PLAN OF GRIMSTON CLIFF FACE
JUNE 1987 SCALE 1:250

