Landslides in the Lower Greensand Escarpment of South Kent

Anna Hopper

A thesis submitted in partial fulfilment of the requirements of Kingston University for the degree of Doctor of Philosophy

December 2000

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Abstract

In 1988, an area of landslides, known as The Roughs, situated on the Lower Greensand Escarpment, in the Wealden district of South Kent, started moving, after an extended period of stability. The Roughs had degraded to a low angled slope since being abandoned by the sea. This thesis undertakes to discover what triggered this reactivation, to clarify the mechanisms, magnitude and rate of the landslides and deduce the significance of the reactivation on other dormant landslides in the UK.

The investigation was approached by the undertaking of a detailed desk study, a comprehensive series of fieldwork campaigns, and a statistical analysis of rainfall and a stability analysis of the slope.

Although many investigations have been carried out in and around the research site, this thesis further contributes to the knowledge of the subject in the following ways.

i. Confirmation of detail in the geotechnics of land movement and how they relate to geological structure in the Lower Greensand Escarpment, Bilsington to Folkestone.
ii. New geological sections through The Roughs.
iii. Indications of the topographic, hydrological and other reasons for the exact location and extent of landslide activity in 1988 and later.
iv. Qualitative discussion on the concept of cyclic variations in the factor of safety.
v. Quantitative analysis of the effect of wet weather periods on The Roughs.
vi. The use of both the infinite and the "finite" slope methods to analyse the stability of the degradation zone.
vii. Examination of the effect of raised ground water levels on the stability of the site using other stability analysis methods.
viii. An explanation of the sequence of events during the land movements.
ix. New insights into the events at Stutfall Castle.

Acknowledgements

I would like to offer my gratitude to the following people, without whom, this thesis would not be.

Firstly to Nigel, Sancta, my parents and parents-in-law for their support, encouragement, patience and time.

To Eddie Bromhead for his never-ending stream of ideas and his help and guidance throughout my work on this thesis.

To the Sappers of the Royal School of Military Engineering, for their weeks of work in the field.

To Nick Lambert (of Soils Ltd.) for his help during the first fieldwork campaign.

To the staff of the Hythe Ranges, for permission be on The Roughs and their interest in the project.

To the staff at Kingston University, especially Roger Curtis, Wilf Schofield and Malcolm Ince, for their help and support throughout the years.

Finally, to Manoj Patel, fellow student and ally, for his assistance during fieldwork and photography.

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Introduction

1.1 The overall problem

In some ways, the UK is far from ideal as a place to study landslides. Seismic activity is rare and of low-intensity. Nowhere is there an extreme landscape in terms of slope heights and gradients, and rapid erosion (except, perhaps along the coast) is unusual. Land-use changes have been largely benign: major deforestation happened in the Neolithic, and industrialisation/ urbanisation seems to have generally improved slope stability with groundwater abstraction. As a result, the UK landscape is generally considered relatively stable. Fortunately in the UK, land movements of any sort are rarely the cause of fatalities and damage is usually limited to financial. However, the recent flooding throughout the country in the year 2000, has demonstrated to the public one effect of "global warming". The highly controversial cause of this, whether it is due to natural climate variation or humankind, is, in fact, irrelevant to the resulting documented increase in rainfall. This rainfall, in turn, makes future landslide activity in the south of the UK very relevant.

In the past, there were many more landslides than today. Periglacial conditions (during the Devensian) allowed sliding of even very low angled slopes, due to increased pore water pressures. Time has hidden the signs of these past movements, to leave the gentle rolling hills that are a feature of today's landscape. These past movements, however, have left the ground in a weakened condition and this, in combination with increased ground water levels, could be a recipe for disaster. We will probably see the reactivation of numerous landslides and it is inevitable that this will result in expensive damage to property and roads.

The following work is an investigation into one of the first of these climate driven reactivations: a series of landslides in an area known as The Roughs, situated near Hythe on the Lower Greensand Escarpment of South Kent. The Roughs is especially interesting because it has a shallow slope, of approximately 11°, and was thought to be stable under present-day conditions. However, in 1988, a series of rotational-translational type slides occurred there after a period of especially wet weather.

There are a number of problems to address, namely:

- What, exactly, triggered The Roughs landslides?
- What was the mechanism of these landslides?
- Why did they move relatively 'fast' and 'far'?
- What significance this has on other similar 'stable' slopes of similar origin?

An unusual feature of the Lower Greensand Escarpment, is that it was once part of a series of coastal slopes. Recent (post-Roman) history has seen the formation of Romney Marsh, which has progressively protected the toe of the escarpment slope, from west to east, allowing the former coastal slopes to develop as an "abandoned cliff". Slopes along the Escarpment show various aspects of this development.

Particular attention was played to the role of climate, especially rainfall, on the reactivation of The Roughs. A statistical analysis, **Ibsen and Brunsden**, 1996, on climate data and landslide frequency from the Isle of Wight, suggested a connection with extended wet-weather periods and the onset of landsliding. A similar analysis was applied to The Roughs with analogous results.

1.2 The desk study

Much of the initial work on this project involved a desk top study to assemble relevant information. This material can be grouped into four categories:

- Establishing the geology and structure of The Roughs and surrounding area. *(Chapter 2)*
- Research into the history of the site. *(Chapter 3)*
- A study of local and related landslides. *(Chapter 4)*
- Literature study relating to landslide theory. *(Chapter 5)*

References are cited in connection with the detailed discussion in Chapters 2 to 5 rather than in this introduction. It was first necessary to establish the morphology of the site. This commenced with a general overview of the Wealden District and a more specific account of the Lower Greensand Escarpment and its geological units. The strata present on The Roughs were established to be Weald Clay, Atherfield Clay and Hythe Beds. An account of the formation of the abandoned cliff in the Weald Clay at the toe of the escarpment then follows. Importantly, the Lower Greensand Escarpment from Adlington to Hythe, known as the Lympne Escarpment, was protected from erosion by the sea by the formation of Romney Marsh and the process by which this occurred is outlined. The chapter concludes with a description of the variation of the geomorphological units along the Lympne Escarpment.

The significance of the history of the site is not immaterial. At first inspection, the site seems to be all but deserted. The only obvious feature is a large concrete sound mirror which is a relic of an Acoustic Research Centre, dating from between the wars. Closer inspection yields another, collapsed mirror and the foundations of the associated buildings. The landscaping and drainage works connected with this development have affected the natural development of The Roughs to this day.

To predict the nature of the landslide mechanism on The Roughs, an investigation of regional landslides was carried out. There were found to be two main types of slides in the immediate area, both with bedding-controlled, sub-horizontal, basal shear surfaces. The first exhibited a basal shear surface at or below sea level and the second type had shear surfaces "perched" high in the cliff. Establishing the critical beds in these systems was also important. Using this information, it was easy to predict that the slip surface, in the landslide on The Roughs, was in the Atherfield Clay, high up the slope. Debris sourced from the upper (perched) landslide has cascaded over the lower slopes, and accumulated on the former beach and marsh. Thus the landslide system on the Lower Greensand Escarpment is shallower, and more sensitive to weather-related groundwater changes, than the larger-scale, deep-seated landslides elsewhere in the district.

It was also helpful to clarify the many terms and ideas associated with landslide theory. The authorities often differ over the relative importance of many factors in slope stability work, particularly relating to trigger factors, and this necessitated a detailed review in this thesis. The chapter includes discussions on landslide type, position, age, shape and trigger-mechanisms and concludes by introducing the principals of stability analysis.

This extensive study was necessary to understand the likely situation on The Roughs. Once this is established, then the fieldwork can be more effective.

1.3 Fieldwork methodology

The site investigation, the details of which are described in *Chapter 6*, can be split into three sections:

- An investigation of two cross sections consisting of a number of boreholes and trialpits
- A series of comprehensive surveys
- Monitoring instrumentation installed on the site

This investigation took place during a series of three fieldwork campaigns. *Table 6.1* summaries the work carried out during each campaign. *Figure 6.1* shows the position of all the trial pits and boreholes. Samples were taken from the boreholes for laboratory testing in order to establish soil properties for use in subsequent stability analyses in *Chapter 9*.

Two surveys were carried out on The Roughs with help of the Royal Engineers. These are discussed in *Section 6.6*. These were used to determine the ground profiles in the geological cross-sections, AA and BB shown in *Figure 6.2* and *Figure 6.10*, respectively.

Instrumentation, in the form of piezometers and inclinometers, was monitored on a regular basis over the next few years. The piezometer readings are shown graphically in *Figures 6.11-13* and the inclinometer readings are discussed in *Section 6.5.2*. Piezometer data was used in conjunction with the ground model revealed by the subsurface investigations in a series of slope stability analyses. This work is discussed in *Chapter 9*.

1.4 Laboratory investigation

To gain full benefit from the fieldwork, it must be followed by analysis of the field samples obtained. Undisturbed samples were recovered from many of the boreholes and extruded in the laboratory. These samples were carefully logged to help in the construction of the cross-sections AA and BB. A series of tests was carried out which are detailed in *Sections 6.7-6.7.5*. The ring shear tests were used to find the angles of shear resistance, to be used in the stability analyses discussed in *Chapter 9*. The graphs of moisture content against depth, plotted for all of the shell and auger boreholes, shown in *Figures 6.14-15*, indicate the possible position of slip surfaces. This information was also used in the stability analyses.

1.5 Analysis

The analysis of the situation on The Roughs was carried out in five sections.

- An examination of slow moving slopes. *(Chapter 7)*
- A statistical and theoretical consideration of the effect of climate on the stability of the slope. *(Chapter 8)*
- A stability analysis on the translational zone using both the finite and infinite slope methods. *(Chapter 9)*
- A stability analysis on the rotational zone using the Morgenstern Price method. *(Chapter 9)*
- An assessment of the sequence of events during the reactivation of The Roughs in 1988. *(Chapter 9)*

Chapter 7 is a re-appraisal of the assumptions made in the analysis of slope movement magnitude and rate.

A statistical analysis of rainfall was carried out to attempt to find a correlation between wet weather periods and the onset of landsliding. It is believed that the landsliding on The Roughs, and other sites, is triggered by a wet weather period, possibly followed by a very intense shorter period of rainfall. Three sites were examined: Folkestone, Ventnor on the Isle of Wight, and The Roughs. Historical records and more recent reports and rainfall records, were used to try to make the connection. Two of the most useful approaches were examining the accumulated rainfall over varying periods of time and also smoothing the curves, using the method of moving averages. It was then a simple process to match the resulting peaks to any coinciding land movements.

Information gathered from the desk study and the fieldwork and laboratory investigations, was incorporated into a stability analysis of The Roughs, to attempt to explain the reasons and mechanisms behind the landsliding. The landside profile was then split into two sections, the long translational slide and the rear rotational slide, for separate analysis. The former was examined using both the infinite and finite slope methods. Parameters were varied to study their effect on the stability of the slope and to find the critical values to induce sliding. The rotational section was analysed using the Morgenstern Price method.

By deducing which section of the slope is least stable, it is hoped to determine the sequence of events on The Roughs. If the translational zone is destabilised more easily, then it is likely that it moved first, removing the toe of the rotational slide and instigating the movement there. Conversely, if the rotational zone is more easily triggered then it is probable that as it moved, head loading was increased on the translational slide, destabilising it. Therefore, by using this information in conjunction with eyewitness accounts, the sequence of events on The Roughs can be ascertained.

The analyses were carried out on the slope profile that existed after the movements of 1988 took place. This profile is, obviously, different to the profile before the reactivation. Slopes usually reach a more stable position of equilibrium after a movement. The model used for the analyses is, therefore, more stable than that which actually moved and reacts differently to changes. It is safe to presume that The Roughs of 1988 would have been more easily activated than The Roughs of today.

1.6 Original aspects of the investigation approach

There has been much work, both academic and professional, done on and around The Roughs. Each successive investigation contributes to the accumulated knowledge of the area. As landslide theory progresses, so does the nature of examination carried out. It follows that each piece of work can be more comprehensive than the last. How does this research differ from previous pieces of work?

- Confirmation of detail in the geotechnics of land movement and how they relate to geological structure in the Lower Greensand Escarpment, Bilsington to Folkestone.
- New geological sections through The Roughs.
- Indications of the topographic, hydrological and other reasons for the exact location and extent of landslide activity in 1988 and later.
- Qualitative discussion on the concept of cyclic variations in the factor of safety.
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- An explanation of the sequence of events during the land movements.
- New insights into the events at Stutfall Castle.

Geology and geography of the area

2.1 The Wealden District and The Roughs landslides in their geological context

In order to understand the geological controls on landsliding at The Roughs site, it is important to understand the regional geographical and geological context. This context is described in the opening sections of this Chapter. Subsequently, the detail of the geology of The Roughs site and the natural landscape features which have had a bearing on slope development are discussed. Most, if not all, of this is based on a literature survey. Since much of the literature predates the development of plate tectonic theory, and is more interested in, for example, fossil content that in the detail of lithology so important to geotechnical engineering, it is often of limited value to this study. However, nineteenth century geologists (e.g. **Topley**, 1875) concentrated on lithology and thus their accounts are preferred to modern interpretations, where lithology is seen as less significant.

The geological sequence present in The Roughs is due to its position in the Wealden succession; the general geological structure to its location in the gently sloping outer limits of the "Wealden Dome" and its geomorphology and geotechnics to its recent geological history, taking into account the one-time effects of the sea before the toe of the slope was afforded the protection from marine attack by the growth of Romney Marsh.

The Roughs are part of the Lower Greensand Escarpment of the Wealden District, in Southern England. *Figure 2.1* gives the location of the research site on the map of the Wealden District. Table *2.1* shows the table of formations in the Wealden District, although this complete sequence does not exist in any one place. The time periods and epochs referred to in *Table 2.1* are defined in the geological time scale, shown in *Table 2.2*. All of the rocks in the sequence are sedimentary including both fresh water and marine deposits. A ribbon diagram depicting the strata of the Lower Greensand Escarpment is shown in *Figure 2.2* and a more detailed geological section from Bilsington to Folkestone is shown in *Figure* 2.3. The geological information in this chapter is based on work by **Topley** (1875) and **Smart** (1966).

The Chapter concludes by pointing out that the geomaterials and structure have a bearing on the mechanism of the land movement on the site, but that a comprehension of this is only the first stage in the understanding of the complete situation.

2.1.1 The formation of the Wealden District

A great lake, known as the Wealden Lake, existed over Southeast England and part of France and Belgium during Purbeck (mid-early Jurassic) times. *Figure* 2.4 depicts the likely location of this lake. The extent of the lake is not known and it has been proposed that it actually extended across France to the Tethys Sea, since, the Wealden deposits have the characteristics which suggest that they were laid down in a large delta. Deposits of shallow water origin were deposited with a thickness of more than 800 metres. These deposits were sorted by the action of water and on erosion , formed clays, silts and sandstones. These Wealden Strata are shown in *Table* 2.3.

After the formation of the Wealden Strata, the sea to the south of the Wealden Lake then advanced into the lake at the end of the Wealden period and converted the freshwater lake into a saltwater bay. Due to further land movements, the lake spread northwards and approached the sea to the north. These land movements also caused localised changes in depth within the lake and hence resulted in a variation of sedimentation type and depth throughout the lake. This variable shallow-water sedimentation resulted in the formation of the Lower Greensand.

2.1.2 The structure of the Wealden District

Four counties are included in the extent of the Wealden District: Kent, Surrey, Sussex and Hampshire, which technically continues across the Straits of Dover into the Bas Boulonnais. The Weald-Boulonnais anticlinal dome structure in Southeastern England and Northeast France is deeply eroded, exposing successively older rocks towards the centre. It is separated into a larger UK part and a smaller, French part by the English Channel. Chalk hills form the most prominent feature of the geography. This ring of Chalk hills has gentle dip slopes trending away from the Weald with much sharper scarps facing towards it. A second, less well developed, inner ring (or "cuesta") of hills, marks the outcrop of the Lower Greensand. Most of this escarpment is inland, but in the area between Aldington in the West, and Sandgate in the East, the escarpment forms a cliff line along the northern edge of Romney Marsh, which has been exposed in the past to marine erosion. The growth of Romney Marsh has protected the toe of the slope from the effects of direct erosion, except in the extreme East at Sandgate. Now abandoned by the sea, this former coastal cliff has long been recognised as landslipped.

During the Tertiary era, differential movements of plates caused the formation of the Alpine mountains and created the dome structure of the Weald (**Edmunds, 1954,** reinterpreted in light of the development of plate tectonic theory). The Wealden Strata were affected to a lesser extent than many areas but their arched structure was nevertheless formed. It is thought that this movement commenced in pre-Eocene times but attained maximum elevation in the Miocene epoch, see *Table 2.2.*

The Weald generally shows an anticlinal structure with an east-south-east to west-north-west axis shown on *Figure 2.1,* although borehole evidence from East Kent points to a synclinal, or trough-like, structure at a deeper level (**Edmunds,** 1954). According to Edmunds, the broad Wealden structure may therefore be considered to be that of an anticline superimposed on part of the northern limb of an extensive syncline which is itself affected by minor folds.

Generally, the Wealden Dome has a dip of 1.5-2° from the central axis. There are however much greater dip angles locally due to the existence of many smaller monoclinal folds which generally have the deepest dipping limb to the north. These folds also have east-west axes.

2.2 Geological units of The Weald

The following descriptions, of the geological units of The Weald, are included because of their relevance to the fieldwork study, for the identification of drilling samples. It is not a simple process to define the exact junction between the Atherfield Clay and the top of the Weald Clay. Ultimately, the positions of these strata are needed for the stability analysis, which is fundamental to the project.

Not all of the described geological units are present on The Roughs, but are included, partly because some are present in other relevant local landslides (***Chapter 4)*** and partly for completeness.

2.2.1 Weald Clay

The Weald Clay consists mostly of light brown or blue grey clay, often weathered, to a maximum depth of 6 metres, to a yellow tint. Towards the top layers, the Weald Clay is brackish in nature and contains thin layers of black clays. At greater depths, the Weald becomes shaly in nature and darkens to dark grey to brown in colour. These clays are often mottled with red (catsbrain). Red or crimson coloured clay is found in the Weald in association with seams of sand.

Subordinate beds of limestone, sand, sandstone and clay-ironstone (as nodules) are found in the Weald Clay but these are localised. The Limestones are formed from the shells of the freshwater snail *Viviparus*, and these are known as Sussex Marble.

The Weald also contains seams and lenses of grey siltstone and brown ironstone. Ironstone nodules (crowstones) are associated with sand beds and have been deposited by percolating ferruginous-water.

2.2.2 Lower Greensand Escarpment

In the Wealden District of the South of England, the Lower Greensand Escarpment extends in an elliptical belt from Hythe across to Maidstone, Sevenoaks, Reigate and Dorking west to Farnham and Petersfield and back down to Eastbourne as is shown in *Figure 2.2.*

The Lower Greensand consists of clays, sands, sandstones and sandy limestones of a shallow water marine origin, grouped into, in ascending order, Atherfield Clay, Hythe Beds, Bargate Beds, Sandgate Beds and Folkestone Beds, as shown in *Table 2.1.* These beds were all given names from the locality with the exception of Atherfield which is named after Atherfield Point on the Isle of Wight. Only the Atherfield Clay and the Hythe Beds, with isolated zones of Sandgate Beds, are present on The Roughs.

The green mineral glauconite is present in small quantities in some of the beds, hence the name "Greensand". It is however, rare and by far the most common colours present are red, brown and grey.

A change in fossil types clearly defines the junction between the bottom of the Greensand, the Atherfield Clay, and the top of the Weald Clay. Freshwater fossils are present in the Weald whereas saltwater fossils are found in the Atherfield Clay. Fossil shells are also present in the Sandgate Beds and less so in the Hythe Beds.

2.2.3 Hythe Beds and Bargate Beds

At the research site, the Hythe Beds, which are approximately 15 m thick, consist of layers of Ragstone (Kentish Rag) and Hassock of thickness 0.2 m to 0.7 m. Ragstone is a hard, greyish blue, glauconitic sandy limestone and hassock is a grey to brownish grey, glauconitic, argillaceous, calcareous sand or soft sandstone. They are thickest in the north west of the Wealden area although they are more distinct from the other strata of the Lower Greensand in Kent.

There are very few references to Bargate Beds. **Topley**, 1875, describes a variation in the Hythe Beds around the south west of Dorking. Here, the beds contain more Sandstone and in the higher part there appears to be a calcareous sandstone or grit known as Bargate Stone.

2.2.4 Atherfield Clay

The Atherfield Clay consists of a bluish grey, occasionally brown mottled, sandy clay and a pale grey slightly glauconitic clay. At depth, the Atherfield Clay is reddish brown or chocolate brown in colour. In places, the Atherfield Clay is chocolatey brown at its junction with the Weald Clay.

Atherfield Clay outcrops are often boggy due to the many springs present at its junction with the Hythe Beds. This clay is thickest in the western part of the Weald and gradually thins out as it follows the southern boundary of the Weald to the east. Atherfield Clay has not been recognised in east Sussex but it can be traced throughout Surrey and Kent, again thinning in an easterly direction.

2.2.5 Folkestone Beds and Sandgate Beds

The Folkestone Beds consist of brown and yellow stained quartzose sand and are thickest in the north west of the Wealden area.

A spring line clearly marks the presence of the boundary between the Folkestone Beds and the Sandgate Beds.

In East Kent, West Sussex and the Kent Surrey border the Sandgate Beds are most developed. They are mostly glauconitic clays and silts. It is presumed that at the time they were laid, the area was somewhat unstable, since there is a great variation in rock types present. In places along the rear scarp the Hythe Beds are overlain by between 1 to 4 metres of Sandgate Beds composed of glauconitic silty clays and silts, with a thin layer of small cream coloured phosphatic nodules occurring at the contact. In Kent and Surrey the Sandgate Beds contain Fuller's earth. The colour of the Sandgate Beds varies from dark green and greenish grey to brown.

2.2.6 The Gault and Upper Greensand

After the formation of the Lower Greensand, the land of the Wealden District submerged and was flooded. A thick layer of mud was deposited by marine currents. In some areas the upper layers of the Lower Greensand were already eroded before the mud was deposited. This mud, now a clay, is now known as the Gault. Nodules of a lime phosphate mark the junction between the Lower Greensand and the Lower Gault. Due to variations in the rates of subsidence leading to an uneven deposition of sediment, some of the new sediment was washed away and another layer of phosphatic nodules was formed. The type of sediment changed and the initial calcareous muds were replaced by silt and then sand. These calcareous muds formed the Upper Gault and the sands and silts, the Upper Greensand.

The Gault is a stiff inky-blue clay. Consisting of layers of sand, sandstone and clay, the Upper Greensand actually has a greenish glauconitic tinge.

2.3 The effect of sea levels and ice on the escarpment

One of the interesting features of the lower Greensand Escarpment, between Aldington and Hythe, known as the Lympne Escarpment, is the abandoned cliff at the base of the slope, above the marsh. The Quaternary climatic fluctuations, as shown in *Table 2.4*, led indirectly to the formation of this cliff. The glacial limit in the UK, shown in *Figure 2.5*, is situated north of the research site. Areas in southern Britain, which have not been affected by glaciation, tend to be more greatly weathered than comparable areas in the north, which have been eroded by ice sheets in the past. This weathering in the uneroded areas often results in land more prone to landsliding. In addition, the cyclic advancing and retreating of the ice sheets resulted in dramatic raising and lowering of sea levels. Each lowering of the sea level led to the abandonment of coastal cliffs. This allowed the cliffs to freely degrade. As the sea returned, the debris would be washed away and the steepened cliffs would be under new attack. It is believed that each inter-glacial high sea level may have eroded a few kilometres of coastline from the soft cliffs of Southern Britain. This process formed the abandoned cliff on the Lympne Escarpment, which divides the Hythe Beds plateau from the marsh.

The last of these sea level rises, the Flandrian Transgression, took place as the Late Devensian ice sheets melted from around 14,000 BP to 8,000 BP. During this period the sea levels rose from -100 m to -20 m. Present sea levels were attained in approximately 5000 BP. It is believed that the existing coastal landslide systems were initialised during the period between 8,000-3,000 BP. Most have remained active since apart from those with a stabilising influence at the toe, such as the formation of Romney Marsh at the toe of The Roughs. These figures have been taken from **Jones and Lee, 1994.**

2.4 Formation of Romney Marsh

An integral part of the creation of The Roughs and the Lympne Escarpment from Aldington to Hythe as we know them today, was the formation of Romney Marsh. By protecting its toe from erosion by the sea, the formation of Romney Marsh partially stabilised the escarpment. The escarpment has since been degrading to a more stable slope angle but these changes are not significant in comparison to the past damage wrought by the action of sea.

A map of Romney Marsh is shown in *Figure 2.6*. It extends up to Hythe in the north and down to the Pett Level Wall in the south. The area known as Romney Marsh actually includes Walland Marsh, Kete Marsh and Denge Marsh and has an area of 260 km^2. Ten thousand years ago the whole area was under water and parts of the marsh are still below sea-level. This account of the formation is an amalgamation of work by **Edmunds (1954)**, **Green (1968)**, **Cunliffe (1980)** and

Eddison (1983).

During Palaeolithic times, a series of sand ridges, Midley Sand, was formed off Fairlight Head due to local river estuary deposition during a time of elevated sea-levels. Shingle beaches build upon the seaward edge of these sands forming a barrier which was aligned in a northeast southwest direction and positioned west of where Lydd now lies. This barrier, protecting the shallow water from the force of the waves, allowed the deposition of marine clays which formed the basis of the marsh.

As the ridges extended rapidly, by Roman times new beach was formed as far as Hythe. Here the only break to the shingle beach existed due to the volume of water emerging from the combined drainage system for all the local rivers. This cleared the exit of blockage by shingle which drifted over from the east, creating a cuspate headland which curved towards the main beach. The Roman fort at Lympne was built at the entrance to this estuary in 340-350 AD.

From early Saxon times (600 AD) the shoreline was breached in two places where rivers entered the sea: in the north the Rother, now emerging at Romney and in the south the Tillingham and the Brede which emerged south of Rye. The shingle was washed away from the area around Rye to form the Dungeness headland. This allowed the sea to flood the marshland and capture the Tillingham, the Brede and the Southern Rother which where then forced to flow southwards. The outlet at Rye was kept clear by the volume of water flowing from it.

Clearance of river sediment was prevented by the decrease in the volume of water from the northern estuary. The shingle was not washed away from the entrance to the estuary and the shingle barrier eventually extended, blocking the outlet.

Around the break in the barrier at Romney the currents eventually washed the shingle south. This added to the volume of material at the headland at Dungeness. Eventually an estuary was formed at Romney which was fed by water draining off the marsh. In time, this estuary extended back to the Northern Rother river and captured some of the water which was flowing to the northern estuary, exasperating the silting problem.

By the early medieval period most of the Northern Rother flowed through Romney Creek, the estuary at Hythe having silted up totally. The peat around Appledore was shrinking and threatening to divert the course of the Rother to the south. To counter this, a canal called the Rhee Wall was dug which is shown in *Figures 2.6 and 2.7* The section of the canal to Old Romney was dug around 1200 AD and the extension to the sea was dug in 1257.

To the south of the Rhee Wall, the land was flooded in 1287, during one of the many great storms of the period. At Dungeness, the headland was much advanced by the build up of shingle deposited during these storms. The flood captured the Northern Rother into the southern estuary. At Romney, the port quickly silted up, there being no water flow to clear the estuary. By the beginning of the seventeenth century, most of the flooded land had been reclaimed.

The shoreline is still constantly changing, but at a greatly decreased rate, due to a scheme of moving shingle and the construction of walls and groynes. The shingle deposition at Dungeness has been decreased by the building of a harbour at Hastings although shingle still flows south from New Romney.

2.5 Geology and structure of The Roughs and the surrounding area

The nature of the landsliding on The Roughs is largely, although not wholly, controlled by the geology and structure of the local escarpment. The nature of the landsliding here differs from nearby Sandgate, where the movements are to some extent affected by the tides and Folkestone where the Atherfield Clay, the critical bed at The Roughs and Sandgate, has dipped below sea level. An understanding of the structure of The Roughs is integral to understanding how and why the land is moving.

The Roughs is located towards the north eastern edge of the Wealden anticline, resulting in a regional dip of approximately 1° to the east. Therefore, the abandoned cliff reduces in height (OD) from 150 metres at Aldington to 30 metres at Hythe. With an overall slope angle of about 9 ° to 11 °, increasing west to east, the slope comprises of the cliff line, which is a steep escarpment (>30 °) in the Hythe Beds caprock, with a shallower slope running to the marsh across the outcrop of Atherfield Clay and Weald Clay. Generally, the slope morphology becomes less bumpy and more subdued in a westerly direction along the length of the abandoned cliff. This is consistent with the earlier growth of the shingle headland in the west preventing active marine erosion of the cliff, allowing areas west of Lympne to achieve long term stability.

Howland, 1986, believes that the most important factor influencing slope formation has been climatic change in the recent geological past. He has investigated the effect of time, since abandonment, on the slope angle, on five slope cross sections on the Lower Greensand Escarpment between Hythe and Aldington along with one at Linton, south of Maidstone. Since Romney Marsh developed from the west, as described in *Section 2.3*, the escarpment shows a progression of natural degradation from west to east.

The slope angle of the most westerly cross section, along with that of the profile from Linton, is below 7°, the ultimate angle of stability for Weald Clay. **Howland**, 1986, suggests that these slopes developed under periglacial conditions. He concludes that the slopes east of Easting 090 on the escarpment, have been re-steepened by rising sea-level, followed by re-abandonment and that slopes west of Easting 090, are soliflucted and have not been toe eroded since sea level lowered in the Pleistocene and have been stable and unmodified since. His estimation of the period for natural degradation of the Weald is circa 10, 000 years.

Solifluction or gelefluction is the flow of debris during periods of partial thawing of snow above frozen subsoil. In some areas this flow took place on very low angled slopes (1.5-2°), whereas in more temperate climates, an angle of at least 8° would be required. The thawing of the permafrost led to high pore pressures. This besides the absence of root stabilisation and rainwater permeating through the melting snow, allowed the movement at such low angles. The solifluction deposits, known as head deposits are common all over the UK and can often be several metres thick

The solifluction sheets often have internal shears and overlie basal shear surfaces. They can be associated with rotational landslides and mudslides. An example of this in the Lower Greensand Escarpment can be found in Sevenoaks, Kent. Work done by **Skempton and Weeks** (1976) showed the existence of solifluction sheets of 2 km in length flowing from the escarpment, which was inclined at an angle of only 1.5°. They found evidence of large landslips that occurred in the Devensian age which have been reactivated in the post glacial period.

For most of its length, the approximately 100 metres high slope is capped with Hythe Beds. On the Roughs the Hythe Beds are approximately 21 metres thick. Underlying the Hythe Beds and the Atherfield Clay is the Weald Clay. It is present to depths in excess of 100 metres below present sea level and underlies the recent deposits which form Romney Marsh.

Under the Hythe Beds is the Atherfield Clay which is also approximately 15 metres thick. This is a critical bed for landsliding, often containing the basal slip surface. Due to the local dip, the Atherfield Clay is perched progressively higher in the cliffs in a westerly direction and therefore landslides also occur at these higher levels.

East of The Roughs, housing development on the slopes obscures some morphology, but it is evident from a study of the air photographs (**Brunsden *et al.*** 1996) that the landslide forms continue into the housing estate west of Turnpike Hill. They probably also exist throughout much of Hythe, and it is certain that much of the town of Sandgate is built on similar landslides. However, to the east of Hythe, the cliffs become progressively capped by Sandgate Beds, Folkestone Beds, Gault (at Copt Point) and Chalk (Folkestone Warren), and the regional dip to the east and north puts the Weald Clay below sea level. Many investigations into the landslides at Sandgate have been undertaken. These have shown that the basal slip surface of the landslides is almost invariably in the Atherfield Clay, and is controlled by the occurrence of a weak bed or beds within the deposit.

2.5.1 Geomorphological Units of the Lower Greensand Escarpment

The landslides along the Lower Greensand escarpment, from Lympne to Hythe, can be considered in six geomorphological units. Much of the data in this section is taken from **Geomorphological Services Ltd., 1988.** Trends along the Lower Greensand Escarpment are summarised in *Table 2.5.* According to **Smart et al, 1966,** the Hythe Beds are often affected by cambering, especially near Maidstone. Where cambering has taken place, the Hythe Beds appear to extend further down slopes than would be expected from the regional dip. Probably, the sea-cliff retreat and marine erosion during the Flandrian, part of the Holocene epoch, will have erased the signs of cambering from most of the Lower Greensand Escarpment between Lympne and Hythe.

In the east, the slope of the free face of the rear cliff of Hythe Beds is steeper, becoming shallower in the west, with slope angles ranging from 27-40°. The taller cliffs also tend to be found at the western end. These cliffs have undergone rotational failures involving both the Atherfield Clay and the Hythe Beds, in the past, resulting in the rotated units of Hythe Beds at the base of the cliff. Degrading back scars of these past failures have formed the free face. The former Weald Clay undercliff is completely buried in the west, becoming more prominent in the east.

From west to east, along the escarpment, the increased period of degradation also affects the zone of rotated blocks remaining from the rotational failures. To the west, the width of this zone is over 100 metres wide in places whereas, near Hythe, it is far narrower. Also, the number of springs emerging from the zone of rotated blocks and the incidence of ponding, increases from east to west. The block disruption decreases markedly towards the cliff line. In the east, the linear benches and steep scarps are more prevalent.

Normally, the degradation zone, which varies in angle from 6-11° in Lympne Park Wood to 12-14° in the east, is seasonally active, more so in the east, which is to be expected considering the steeper slope angles and greater incidence of springs. The thickness of the debris covering the degradation zone, which is being transported from the rotated bock zone to the accumulation zone, is about 5-10 metres. Partly translational and partly shallow rotational failures account for the mechanism of debris transportation

The boundary between the degradation and the accumulation zones is extemely difficult to identify. Generally, the slope angle in the accumulation zone is a couple of degrees shallower than that in the degradation zone. The former undercliff can be identified in the accumulation zone. Also present, are many lobes of landslide debris which are more prevalent around West Hythe. *Plate 4.1*, shows one of these lobes. At the base of the slope, the accumulation zone is interleaved with the Romney Marsh deposits. The marsh is at an angle of 1-2 °.

2.6 Summary

To determine the likely nature of the landslide mechanism on The Roughs it is first necessary to establish the geology and structure of the site. To summarise, the salient points are:

- The site is located on the Lower Greensand Escarpment of South Kent, on the outer limits of The Weald.
- In ascending order, the geological units present on the site are, Weald clay, Atherfield Clay and Hythe Beds.
- At one time actively undergoing marine erosion, the Lympne Escarpment is now protected from the sea by the formation of Romney Marsh.
- The Lympne Escarpment is subject to the process of free degradation, which is more advanced in the west due to the earlier protection of the marsh.

The description of the geomorphological units on the Lympne Escarpment point to the classic combination of rotational slip (due to the presence rotated blocks) and translational slip. The streamlines indicate the junction of the Hythe Beds and the Atherfield Clay. Once all this information has been established it is then useful to examine those landslides in the vicinity which have already been documented. The crucial facts are the critical beds in which the slip surfaces are formed and the likely trigger or triggers for the movements. From this material, is it possible to infer the likely slip scenario on The Roughs before the commencement of the fieldwork. The resulting investigation can then be better targeted. It is also necessary to know the history of the site to establish whether there has been any human interference which may have changed the natural course of events. All of these topics are examined in subsequent chapters: a more detailed site description and history in *Chapter 3*, related landslides in *Chapter 4* and fieldwork in *Chapter 6*.

Figure 2.1. Map showing the Wealden District of South East England. (After Bromhead, 2000).

Figure 2.2. Ribbon diagram of the Lower Greensand Escarpment. (After Gallois et al, 1965).

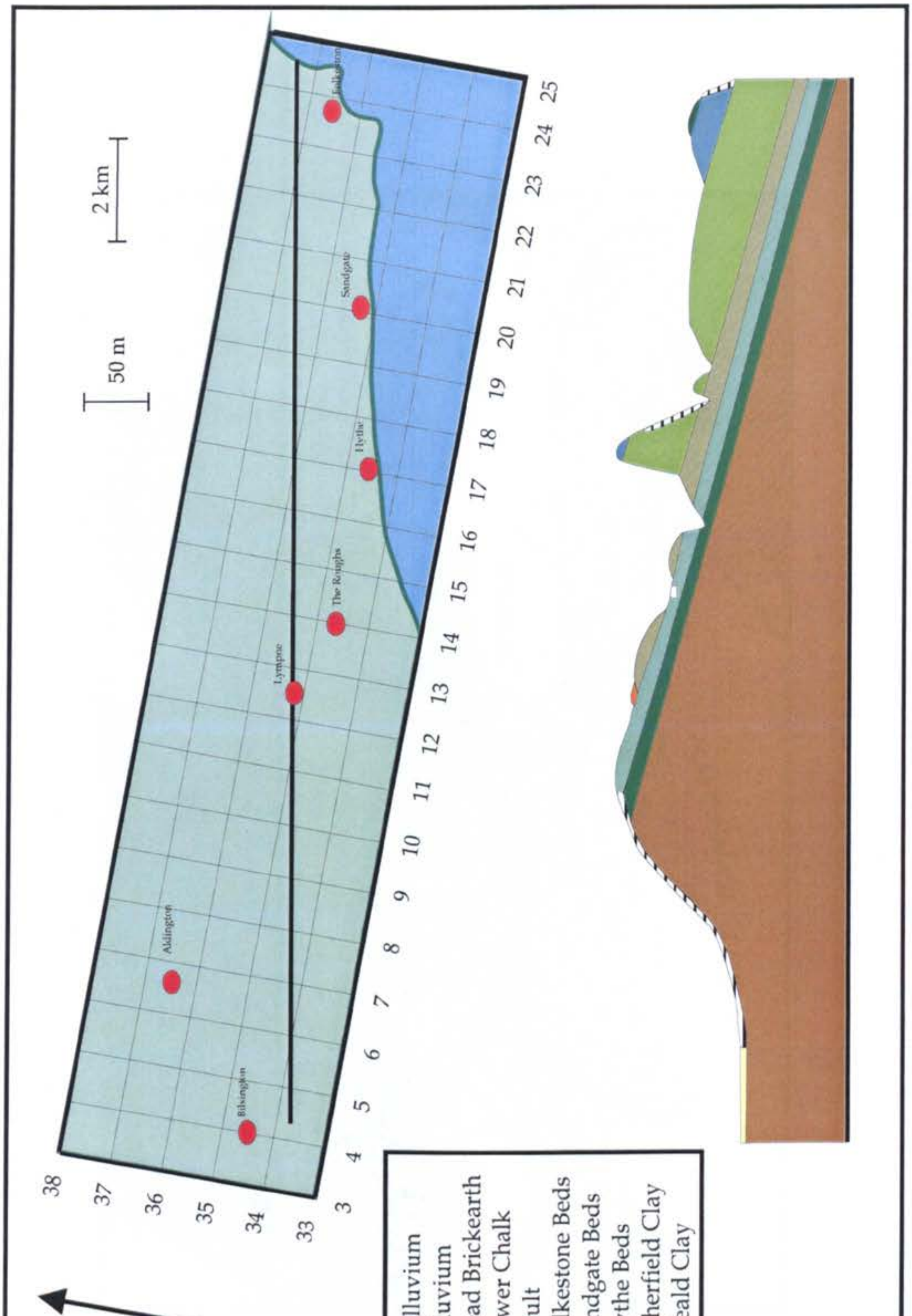


Figure 2.3. Geological section from Bilsington to Folkestone.

Figure 2.4. Sketch map showing the probable extent of the Wealden 'Lake'. (The shaded portions represent the present land surface of part of England, France and Belgium). (After Edmunds, 1954)

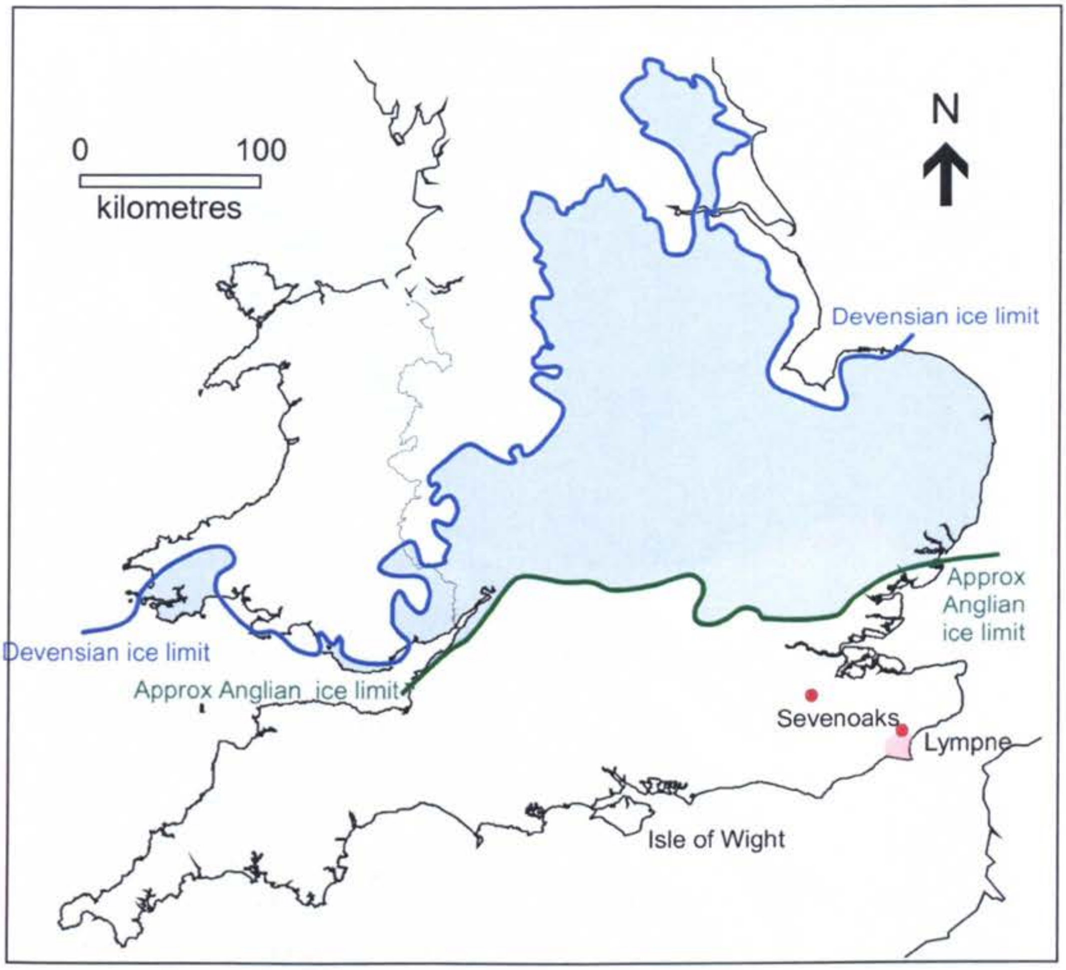


Figure 2.5. Map showing the glacial limit in the UK

Figure 2.6. Map illustrating the historical development of Romney Marsh. (After David Ovenden, 1999).

Figure 2.7. Sketch map illustrating the development of Dungeness. (After Edmunds, 1954).

Table 2.1 Formations in the Wealden District (After Edmunds, 1954).

	Period	Epoch	Deposits and Formations		
Strata exposed at surface	Quaternary	Recent and Pleistocene (Superficial Deposits)	Sand and Shingle; Alluvium River Gravels; Dry Valley and Nailbourne Deposits; Head; Clay-with-flints		
	Tertiary	Pliocene and early Pleistocene	Lenham Beds, etc.		
		Eocene	Bagshot Beds Claygate Beds London Clay Oldhaven Beds Woolwich and Reading Beds Thanet Beds		
	Mesozoic	Upper Cretaceous	Chalk	Upper Chalk Middle Chalk Lower Chalk	
		Lower Cretaceous	Upper Greensand		
			Gault	Upper Gault Lower Gault	
			Lower Greensand	Folkestone Beds Sandgate Beds Bargate Beds Hythe Beds Atherfield Clay	
			Wealden Series	Weald Clay	
				Hastings Beds	Tunbridge Wells Sand Wadhurst Clay Ashdown Sand
Strata proved in borings only		Upper Jurassic	Purbeck Beds Portland Beds Kimmeridge Clay Corallian Beds Oxford Clay Kellaways Clay		
		Middle Jurassic	Cornbrash Great Oolite Series Inferior Oolite		
		Lower Jurassic	Upper Lias Middle Lias Lower Lias		
		Triassic	Rhaetic		
	Palaeozoic	Carboniferous	Coal Measures Carboniferous Series		
		Devonian	Old Red Sandstone		
		Silurian	Wenlock Series Llandovery Series		

Table 2.2. Geological time scale.

Eon	Era	Period	Epoch	Approximate time boundaries
Phanerozoic	Cenozoic	Quaternary	Holocene	10,000
			Pleistocene	1,640,000
		Tertiary	Pliocene	5,200,000
			Miocene	23,300,000
			Oligocene	35,400,000
			Eocene	56,500,000
			Palaeocene	65,000,000
	Mesozoic	Cretaceous		145,600,000
		Jurassic		208,000,000
		Triassic		245,000,000
		Permian		290,000,000
	Paleozoic	Carboniferous		362,500,000
		Devonian		408,500,000
		Silurian		439,000,000
		Ordovician		510,000,000
		Cambrian		570,000,000
Proterozoic	Neoproterozoic			1,000,000,000
	Mesoproterozoic			1,600,000,000
	Palaeoproterozoic			2,500,000,000
Archaean	Late			3,000,000,000
	Middle			3,500,000,000
	Early			4,000,000,000
Priscoan				4,650,000,000

Table 2.3. Strata formed from the Wealden Lake. (After Edmunds, 1954).

Formation	**Formation**	**Thickness/metre**	**Subordinate Beds**
	Weald Clay	120-425	
Hastings Beds	Tunbridge Wells Sand	40-120	Lingfield Beds Upper Tunbridge Wells Sand Grinstead Clay Lower Tunbridge Wells Sand
	Wadhurst Clay	30-70	
	Ashdown Sand	50-210	
	Fairlight Clays	0-120	

Table 2.4. Periods of glaciation affecting parts of Great Britain.

Glaciation		**Approximate period of maximum extent (bp)**
Loch Lomond Glaciation	Devensian Glaciation	11,500-10,800
Dimlington Glaciation		18,000
Wolstonian Glaciation (Paviland/Welton)		270,000
Anglian Glaciation		450,000

Table 2.5. Summary of trends along the Lympne Escarpment.

Feature	**East**	**West**
Natural degradation	Decreasing	Increasing
Former Weald undercliff	Prominent	Buried
Depth of material in accumulation zone	Thinner	Thicker
Width of 'rotated block' zone	Narrower	Wider
Slope angle	Increasing	Decreasing
Slope of rear scarp	Steeper	Shallower
Height of rear scarp	Higher	Lower
Linear benches and steep scarps	Less prevalent	More prevalent
Springs and ponding	Greater incidence	Lesser incidence
Seasonal activity of degradation zone	More active	Less active
Lobes of landslide debris	Less prevalent	More prevalent

The Lympne Escarpment: past and present

3.1 Location of the site and its surroundings

The section of the Lympne Escarpment under investigation, is known as The Roughs. It is to the west of Hythe, in the Wealden District of South Kent, at Grid Reference TR 140343 as shown on the location map in *Figure 3.1*. Although, fortunately, the recent landslides have occurred in a non-built up area, this does not imply that future landslides would not damage something of great significance, since the district is of "*high societal value*". It is seeped in history, a site of scientific interest as well as an economic, built up area.

The escarpment itself is broken, sloping ground, of variable height (rising to the west), formed by the outcrop of Weald and Atherfield Clays capped by Hythe Beds. This geological sequence is discussed in *Chapter* 2. The Lympne Escarpment has many small springs and a few major ones, which are more numerous towards the eastern end of the escarpment. As part of the Hythe Ranges, The Roughs are owned by the Ministry of Defence and are currently leased to a farmer to graze animals but are also used for limited military exercises. The Lympne Escarpment, is a Site of Special Scientific Interest (SSSI). This is dealt with more fully in *Section 3.3.*

North of the site, there is a high-level plain. This is an erosion surface on top of the Hythe Beds. It falls to the East and North in accordance with local dips. To the north of the site the flat land is used for crop-growing.

As depicted on the map in *Figure 3.1*, the landslide site is bordered to the east by the small town of Hythe, which is in the district of Shepway. One of the original Cinque Ports, Hythe dates back to the Doomsday Survey. The history of Hythe is outlined in *Section 3.6.1*. Much of it is built on the same escarpment as The Roughs. Details of ground investigations in Hythe and the surrounding area are given in *Chapter 4*. To the west of the site is the hamlet of West Hythe (at the foot of the slope) and the village of Lympne with the mediaeval Lympne Castle (at the crest of the slope).

To the south, an accumulation of shingle beaches and reclaimed marshland forms a low elevation coastal plain (Romney Marsh). The beach and marsh rest on an eroded surface in the Weald Clay. Part of this area is cordoned off for army training in the Hythe Ranges. These, and the waters around them, are considered a "danger zone", due the army firing exercises which take place there. The Ranges are on the most northeastern corner of Romney Marsh. *Section 2.3* gives details of the formation of the marsh. Between the escarpment and the Hythe Ranges, running in an east-west direction, is the Royal Military Canal. This was excavated at the beginning of the 19^{th} Century

(1804-15) as part of a scheme envisaged by the Duke of York (Vine, 1972). He planned to defend the peninsula, against a French attack, by constructing a thirty mile canal and adjacent road to move troops quickly. Only four miles of the Royal Military Road was ever built. The canal was never used for defensive purposes, but only to transport supplies to the Martello towers. These famous towers were also built at this time and one survives on the southern boarders of the Hythe Ranges. A housing estate and camping site are situated between the Military Canal and the Ranges.

Running from Hythe, across Romney Marsh to Dungeness, is the Romney Hythe and Dymchurch Railway. This was built in the 1920's for a millionaire racing driver and is the World's smallest public railway. The steam from the one-third of full size locomotives can be seen clearly from The Roughs.

To the west is the Wild Animal Park which is home for black rhino, Timber wolves, Asian elephants, Barbary lions, monkeys and cats. Also to the west, at Grid Reference TR 118343 are the remains of the Roman Portus Lemanis also known as Stutfall Castle. This has been the subject of a detailed archeological/geotechnical investigation. More details are given about the fort and the investigation in *Section 4.5.2.*

3.2 Comparison of six geological sections along the Lower Greensand Escarpment

Section 2.4.1 covers the geomorphology of the Lower Greensand Escarpment and the variation thereof along the length of the Escarpment, in some length. These trends are summarised in *Table 2.5*. To examine this topic further, sections along the escarpment have been looked at in more detail.

The positions of six geological sections through the Lower Greensand Escarpment, including four from *Chapter 4 (Figures 4.9, 4.13 and 4.16)*, which have been redrawn and rescaled to facilitate easy comparison, are shown on the location map in *Figure 3.1.* As far as possible, the sections have been aligned using the position of the abandoned cliff at the base of the slope. Obviously, this is not a perfect solution, but due to the differing extents of each section and the natural variation along the escarpment, this was the most obvious "neutral point". These sections have been reproduced in order, from west to east along the Lympne Escarpment in *Figures 3.2 to 3.7. Figure 3.8* shows the six sections superimposed. Using the six colours of the rainbow, in their natural sequence, was a practical method of distinguishing the individual sections in *Figure 3.8*, while being able easily to deduce the order in which they appear, along the escarpment.

Allowing for misinterpretation of data in the construction of the original sections and other errors which may have occurred, *Figure 3.8* demonstrates extremely well the aforementioned trends along the Lympne Escarpment. It is demonstrated how the rear cliff of Hythe Beds decreases in height from west to east as expected from the regional dip in the area. The buried undercliff at the base of the slope is indeed far deeper in the west.

3.3 The Escarpment as a Site of Special Scientific Interest

The Lympne Escarpment, which has a total area of 143.1 ha or 353.6 acres, was notified as a Site of Special Scientific Interest under section 28 of the Wildlife and Countryside Act of 1981. The following is extracted from Shepway District Council documents, giving reasons for the status of the Lympne Escarpment as a SSSI.

The site consists of a steep escarpment of Kentish Ragstone formed by the Hythe Beds of the Lower Greensand. Ragstone is a hard sandy limestone which produces calcareous soils. The grassland and woodland of this site are among the best remaining examples of semi-natural habitats in Kent. Wet ash-maple is the predominant woodland type with a small area of calcareous ash-wych elm wood. Many plants usually associated with chalk soils occur in the grassland. The south-facing slope is close to the sea and the resulting mild humid conditions encourage the growth of ferns and mosses. Many springs and flushes occur at the base of the escarpment at the junction of the Ragstone and the Atherfield Clay.

Outcrops of Ragstone are frequent on the upper slopes of the escarpment. The vegetation here is dominated by grasses such as fescues *Festuca* species, cock's-foot *Dactylis glomerata*, false oat-grass *Arrhenatherum clatius* and tor-grass *Brachypodium pinnatum*. Grazing helps to minimise a diverse flowering plant community including cowslips *Primula veris*, carline thistle *Carlina vulgaris* and hound's tongue *Cynoglossum officinale* which are associated with the calcareous soils. Due to the high humidity of the area wood sedge *Carex sylvatica* and stinking iris, species usually restricted to woods, are able to grow in the open grassland.

Past landslips have produced much scree at the foot of the escarpment and the grassland here is dominated by tor-grass. The marshy ground below the spring line has tall herb vegetation including plants such as great horsetail *Equisetum telemateia*, great willowherb *Epilobium hirsutum*, ragged-robin *Lychnis flos-cuculi* and water figwort *Scrophularia auriculata*.

3.4 Description of The Roughs

In 1988 The Roughs started to move again after a period of heavy rain, which is examined further in *Chapter 8*. This reactivation is shown clearly by contrasting series of aerial photographs taken by Kent County Council in 1985 and 1990 *(Plates 3.1 and 3.2)*. These stereo pairs have been used for field reconnaissance and allowed the ground to be mapped after the recommencement of sliding. The geomorphological map derived from the 1990 photograph is shown in *Figure 3.9*. It shows the extent of the 1988 slide as well as the degraded features remaining from previous movements. Many water-related features are depicted. These are especially abundant in the area around the most recent landslide.

Figure 3.10 depicts an oblique view of The Roughs. The area of the site under investigation, centrally placed in the diagram, is currently the most active portion of the landslide, although other sites have been more active in the past. Moving in an approximately north south direction the main landslide is flanked by two large mudslides, also shown on *Figure 3.10*. The site can be split into six regions:

- the rear scarp of Hythe Beds
- rotational slide zone/zone of rotated blocks which is part of the degradation zone
- translational slide zone which is also part of the degradation zone
- accumulation zone which contains many lobe-like features
- toe, the end of the accumulation zone which over spills on to the marsh
- marsh

3.4.1 Description of the research area

There is no public vehicular access to the site and access with MoD permission, is limited to a rough road along the canal leading to a gate at the south western corner of the site. A public footpath or track runs from this gate, northwards up the escarpment and along to the east along the top of the escarpment to the local housing estate. Driving a limited distance up this track is possible, but a four wheeled-drive or track vehicle is needed. Bad weather can make the journey extremely hazardous or even impossible. With the permission of the farmer who owns the land, driving equipment through the fields at the top of the site to be carried down manually is possible. Unfortunately, bringing the Shell and Auger drilling rigs to the site by this route is not possible.

To the east of the track is a large mudslide which has to be traversed to reach the main body of the landslide. A rough track was created through the landslide while the investigation was in progress to facilitate access of the Pilcon rigs to the site. This disintegrated rapidly during the wet weather. The mudslide does not reach the marsh at the base of the Escarpment but runs out on an area of land which has a high density of tension cracks.

At the top of the site there are three relatively smooth and even plateaus which are ideal sites to set up drilling rigs. These are situated at the top of the rotational slips and hence tend to tilt. The highest plateau has the remains of an Acoustic Research Station which is described in *Section 3.6.2.* Between and below these plateaus, is rough, uneven ground with occasional ponding. One of these ponds is shown in *Plate* 3.3. The rough area was formed by the degradation of the ground which had been previously thrust upwards by the rotational slips and the accumulation of the debris lower down the slope.

The rear scarp consists of Ragstone and Hassock which is described in *Section 2.1.5. Plate 3.4* clearly shows the remains of the access road to the Acoustic Research Centre in the cliff face .

At the base of the slope the landslides toe out. Many toe features on site are extremely well developed and an example is shown in *Plate* 3.5. The toe features are well covered with tension cracks. They run out onto Romney Marsh, which is generally level and runs along the base of the slip bordered, by a small water-filled ditch to the south. The marsh is, obviously, extremely wet and boggy in the winter months.

3.5 Present and past land usage

Currently, The Roughs are used to graze sheep and as a training ground for the army and the police. The rough ground and vegetation afford ideal cover for "stalking" and camouflage. The area is also used for hunting. To the north of The Roughs the fields are used for agriculture and therefore no buildings are in immediate danger of collapse as the rear scarp degrades further. However, this is not true along the length of the escarpment and buildings as near as Hythe may well be under threat.

Development of The Roughs site was inhibited by its use as an Army training area. Foundations of a house are present on a bench on the accumulation zone, this was in use in the early half of this century. At the top of the slope, foundations for huts and an access ramp built in the 1920-30's in connection with experimental sound mirrors for long-range aircraft detection may be seen. One mirror (Scarth, 1995) remains to the west of the active area, but another, of a different design, cast in concrete against the scarp of the Hythe Beds, was undermined and toppled over during the 1988 movements. It is believed that some ground reprofiling on a small scale was made in connection with this experimental work at the time, accounting for the very smooth ground formerly present. Water collection from springs near the junction of the Hythe Beds and the underlying clays for the Folkestone and District water company ceased in the middle of this century .

3.6 History of the escarpment

The Escarpment is littered with evidence as to its former usage. The most obvious, to the west of the landslides under current investigation, are the ruins of the Roman Port Lemanis, or Stutfall castle. There is evidence such as realigned tiling, that the land was unstable even while the castle was under construction.

A pre-radar Second World War listening station, situated on the most active part of the landslide, fared less well than the castle. Apart from one large listening dish, nothing remains but the foundations and broken drainage pipes.

In the summer, when the soil has dried out, the foundations of what were probably a shepherd's cottage can be traced on one of the lower plateaus.

3.6.1 The History of Hythe

Hythe is one of the five original Cinque Ports, the others being Dover, Sandwich, Romney and Hastings. Shepway Cross, on Lympne Hill, was erected in 1923 to mark the site of the meeting point of the dignitaries of the Cinque Ports. The ports existed before the Norman conquest but not as a formal body. They were expected to protect the southeast shores and to provide cross-Chanel passage to the monarch. In return, the Ports were granted varying degrees of autonomy and ranges in privilege and honours at court. The earliest Cinque Port charter was granted to Hythe in 1278 during the reign of Edward the First.

During the next few centuries the town declined to the relentless silting up of the port. Hythe was only able to provide a small (less than 25 tonnes) ship against the Spanish Armada in 1588 due to this problem.

"Who names us SANK and not our SINK is forever foe. His ships be engaged and Bloody Battle SUNK. No prisoners be taken!!" Anonymous.

To be expected with its long history, there are many old and interesting buildings in Hythe. The Town Hall, formally the Guildhall, dates from 1794 and was built on the site of the covered market place. The Parish Church was originally a Norman structure but has been expanded in 1175, and in the 13th and 18th Centuries.

3.6.2 Twentieth century

One of the most eye catching features of The Roughs is a large concrete listening dish. This is a relic of an air-defence experiment, researched by **Scarth**, 1995. This experiment was abandoned due to the development of radar.

Between the two World Wars, six listening mirrors and an Acoustical Research Station were constructed as part of the "Hythe System". The area was chosen because of its ideal position to test the new concrete listening mirrors, since civil aircraft used Lympne as a reference point as they set a course for France. A listening station and two mirrors were built on The Roughs. The research centre and the first mirror, which was twenty foot in diameter, date from 1922/3. A thirty foot mirror, was built in 1929, to the west of the first (*Plate 3.6*). An access road to the earlier site was built on the cliffs to the buildings of the research centre: a handful by the first mirror and one bunker by the second. Natural springs were used to supply water to the site. This has affected the natural water courses to the present day.

The larger of the two mirrors has survived but the smaller mirror, which is situated on the active part of The Roughs, toppled over in the 1980's. None of its associated buildings survive although their distorted foundations are clearly visible. The remains of the access road are only just distinguishable in the cliffs.

This illustrates that The Roughs were not considered unstable at the time of the mirrors' construction and indeed proved to be stable until the 1980's. Only the site of the smaller mirror has moved to any great extent since the twenties although tension cracks are now opening below the surviving mirror.

3.7 Description of the 1988 reactivation

In his MSc thesis, **Anderson**, 1990, gives an account of the reactivation of The Roughs landslide complex as noted by Jill Eddison. Her version of the sequence of events is summarised here.

i. 15th January, 1988. The gateman of the Hythe Ranges observed movement on The Roughs and Jill Eddison inspected the site. She noted a rotational/translational type slide occurring at Location I. A 4.6 metre scarp had been formed.

ii. Movement continued, affecting an increasing area downslope.

iii. 6th February, 1988. Jill Eddison observed the initial stages of the rotational/translational slump.

iv. 15th February, 1988. Jill Eddison noted the initial stages of Landslides II and III.

The three landslides mentioned are shown on the map in *Figure 3.11*. Each of the three landslides formed below a streamline emerging at the base of the zone of rotated blocks. A seasonally active mudslide also existed above the site of Landslide I. This would indicate the significance of water to the reactivation of the site. From information received from Jill Eddison, Anderson concluded that the rotated blocks were undercut by seepage erosion resulting in the formation of two gullies (located in the zone of rotated blocks, above and to the east of Landslide I) and a mudflow. Between the landslides were relatively stable zones, especially between Landslides I and II which were both bounded by "prominent lateral shears". Landslide III had a pronounced lateral shear along its western edge and a 'hummock overridden edge' at its base. The section X-X' shown on *Figure 3.11*, is shown in *Figure 3.6*.

This account is interesting but inconclusive. It is not immediately apparent which came first: the rotational or the translational slide. If the base of the rotational block zone was indeed eroded, then this *'toe unloading'* may have triggered a rotational slide. This in turn would have added head-loading to activate the translational slide. Alternatively, since the soil was at or near saturation, see *Chapter 8*, a primary translational slide may have unloaded the toe of the rotational slide, causing the secondary movement. The true scenario may or may not be fully deduced, but the matter is investigated further in *Chapter 9*. Here, stability analyses are carried out to investigate the more likely solution.

3.8 The importance of The Roughs as a research topic

Of all the landslides, both past and present in the UK, why chose those on The Roughs for special attention? At first, they merely seem to be a relatively small set of movements in the middle of nowhere with little or no bearing on the local area or its population. This is far from the truth.

One of the reasons why the movements on The Roughs site are potentially interesting, is because toe excavation is clearly definitely not the cause of the reactivation. This fact has led to a rethink of the causes of other landslides on comparable slopes. At Hadleigh (*Section 5.4.1)*, a phase of activity in the 19th century was attributed to digging a brick pit at the toe of the slope. At Lympne, it was thought that the Romans had excavated away at the toe.

The landslides seemed to have occurred rapidly, and to be large in scale. At the time, Hutchinson, discussed the nature of landslides on pre-existing shears on low-angled slopes (**Hutchinson, 1987**). He suggests six reasons why these movements may be not be slow and limited, *Section 5.4.4*. His discussion on the effects of water has a different emphasis to the ideas presented in this paper *(Chapter 8)* and do not revolve around the concept of climate change and wet weather periods. The Authors of the Lympne paper (**Hutchinson et al.**, 1985) also appear to be perplexed by the magnitude of the deformations on this low-angled slope, since most of the failures in pre-existing slipped slopes have a big disturbing force, a load or excavation. This is usually associated with road construction such as the infamous case on the Lower Greensand at Sevenoaks (**Skempton and Weeks, 1976**). All The Roughs had was a period of wet weather!

The behaviour of this site has a bearing on public safety, being intimately related in its geology and geomorphology to the nearby housing. It also has SSSI status and is of archaeological interest.

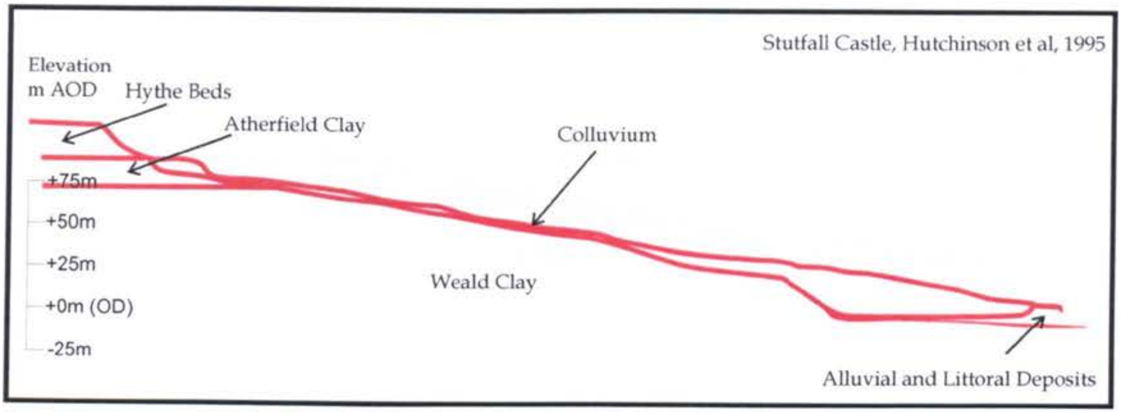
3.9 Summary

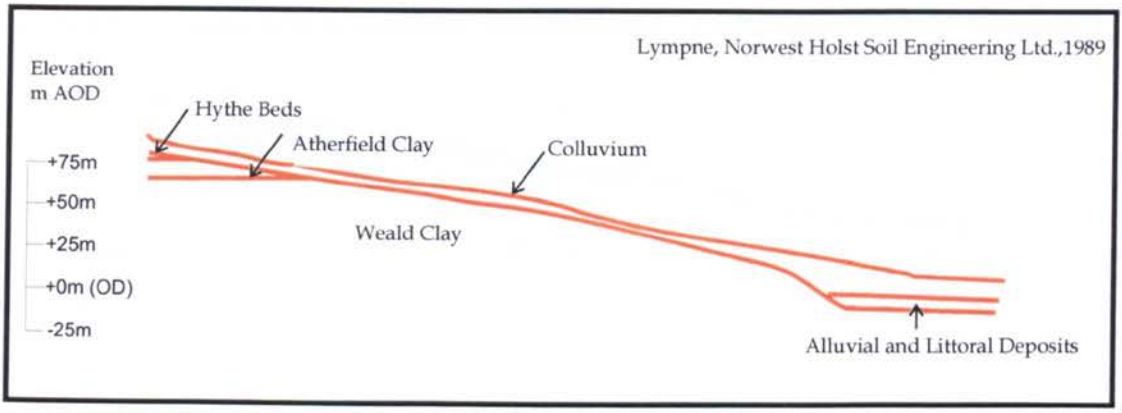
The Roughs was chosen as a research site because it was believed that it had not been interfered with to any degree. On further investigation, this premiss has been found untrue. The primary cause of damage has been the building of the Acoustic Research Centre. It is obvious on closer inspection that there was a significant amount of landscaping associated with the building work. Drainage pipes from the station have become misaligned and are a probable cause of the western mudslide.

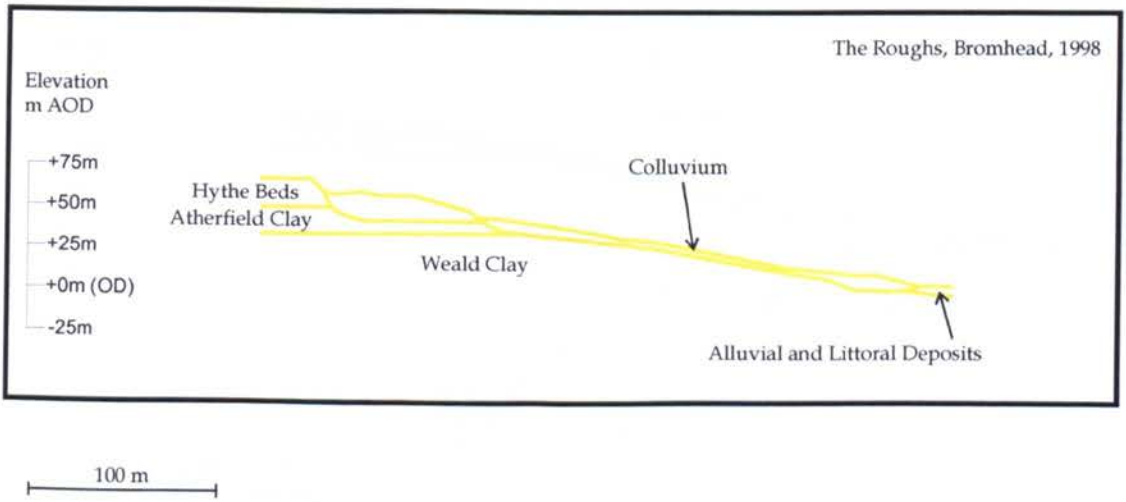
It is also clear that parts of the Lower Greensand Escarpment are not moving, nor have they done so for centuries. The presence of the 18th century Town Hall and the 12th century Parish Church in Hythe illustrate this fact.

The movements on The Roughs, a low-angled and apparently stable slope, have contributed to new ideas in landslide theory. Most importantly, they have emphasised the importance of climatic change *(Chapter 8)* in the reactivation of ancient landslides. This is of great relevance to the local population and ultimately, to numerous other communities in the UK.

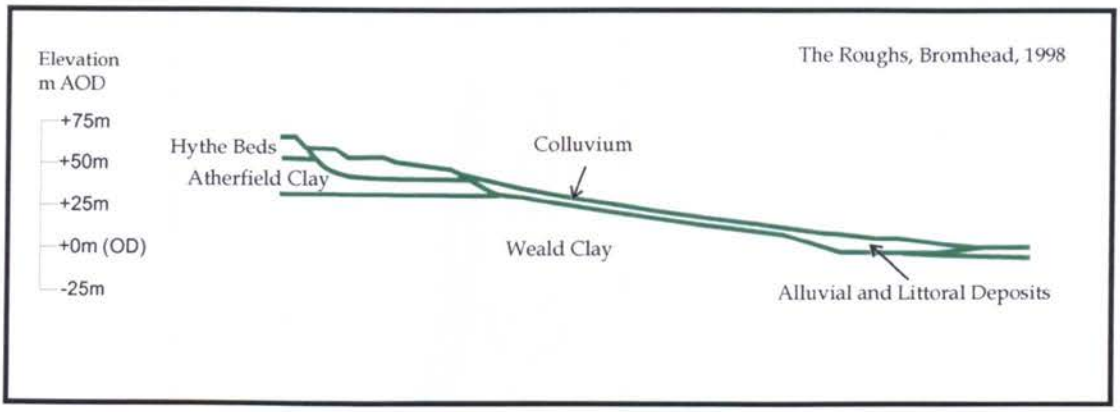
Figure 3.1. Location map showing the positions of geological sections, Hythe and local landmarks. (After Hutchinson et al., 1985).

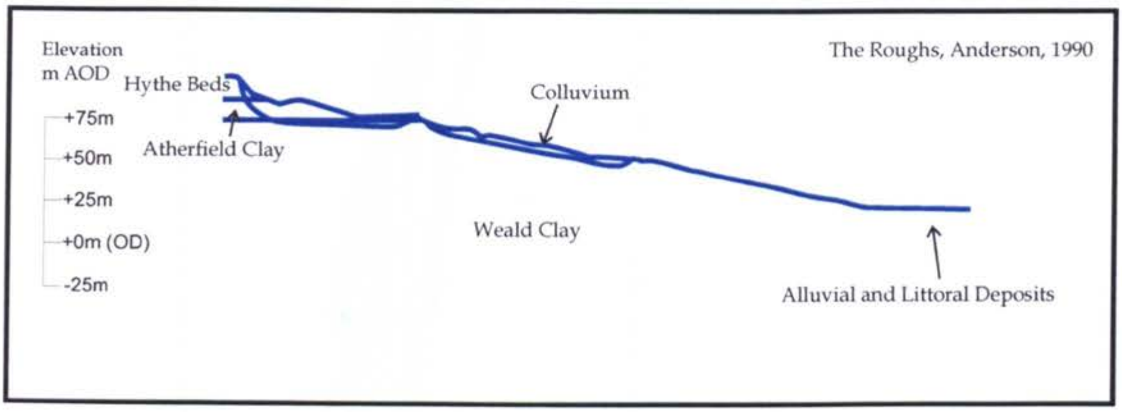


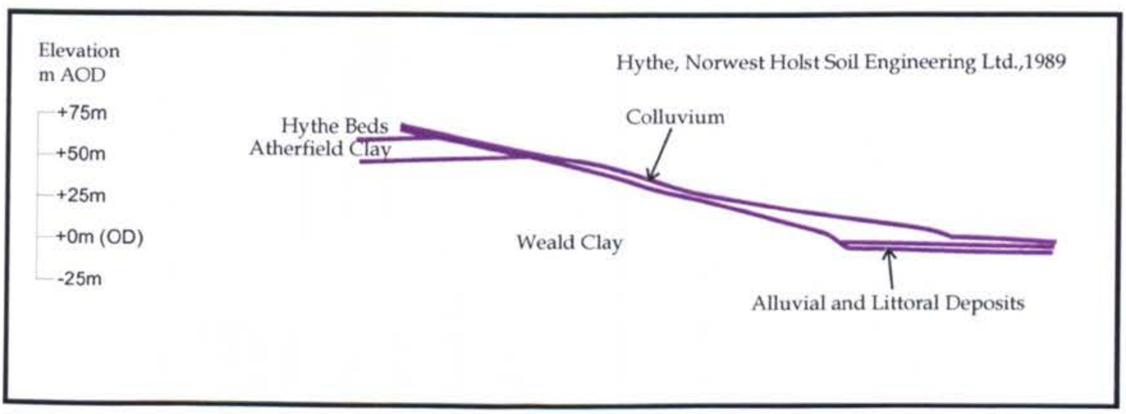


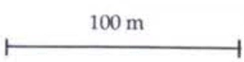


Figures 3.2 (top), 3.3 (middle) and 3.4 (bottom). Geological sections through the Lympne Escarpment, from west to east.









Figures 3.5 (top), 3.6 (middle) and 3.7 (bottom). Geological sections through the Lympne Escarpment, from west to east.

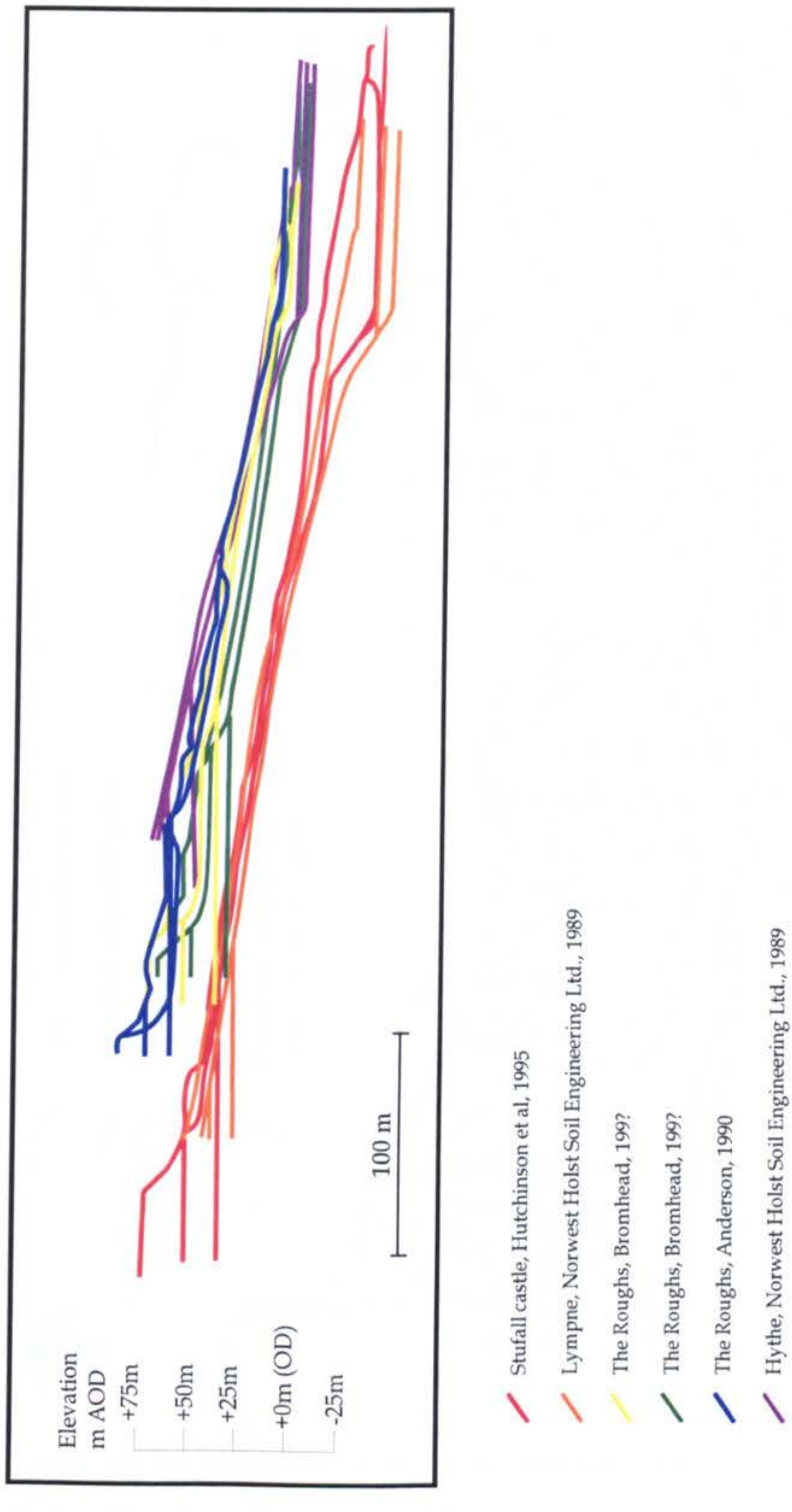
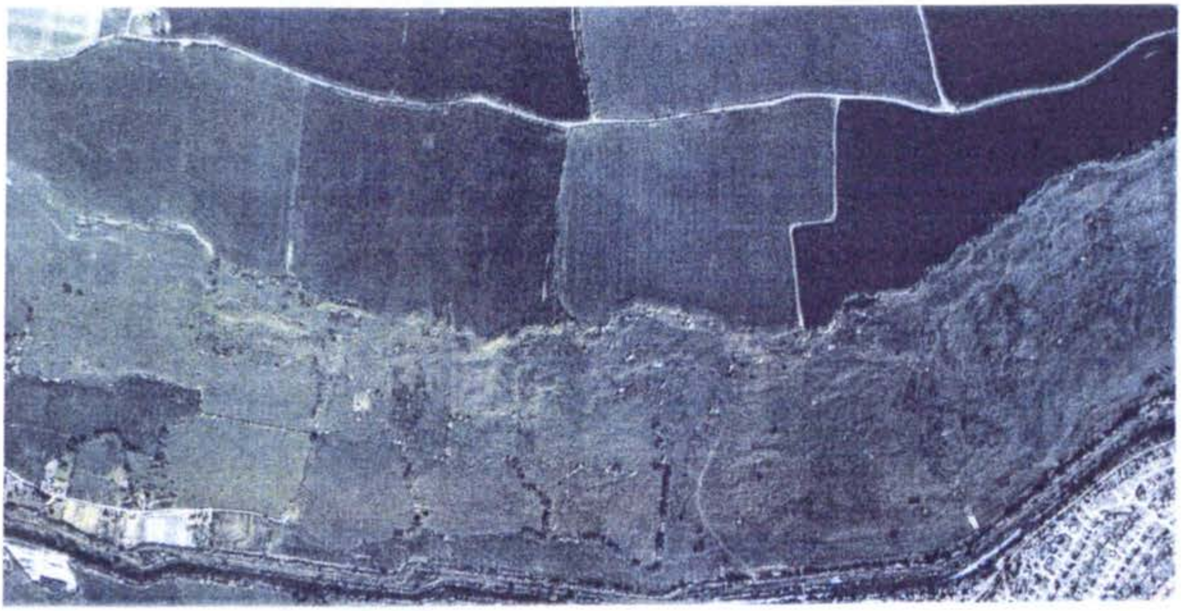


Figure 3.8. The six geological sections along the Lympne Escarpment, superimposed.

Figure 3.9. Preliminary outline geomorphological map of the surrounding landslides, derived from vertical air photographs taken in 1990. Mapping was done by Professor D. Brunsden. (After Bromhead, Hopper and Ibsen, 1998).

Figure 3.10. Interpretation of the oblique aerial view of The Roughs, taken from an oblique aerial photograph, and showing the section lines AA and BB. (After Bromhead, Hopper and Ibsen, 1998)

Figure 3.11. Map showing the geomorphology of the 1988 reactivation of The Roughs. (After Anderson, 1990).



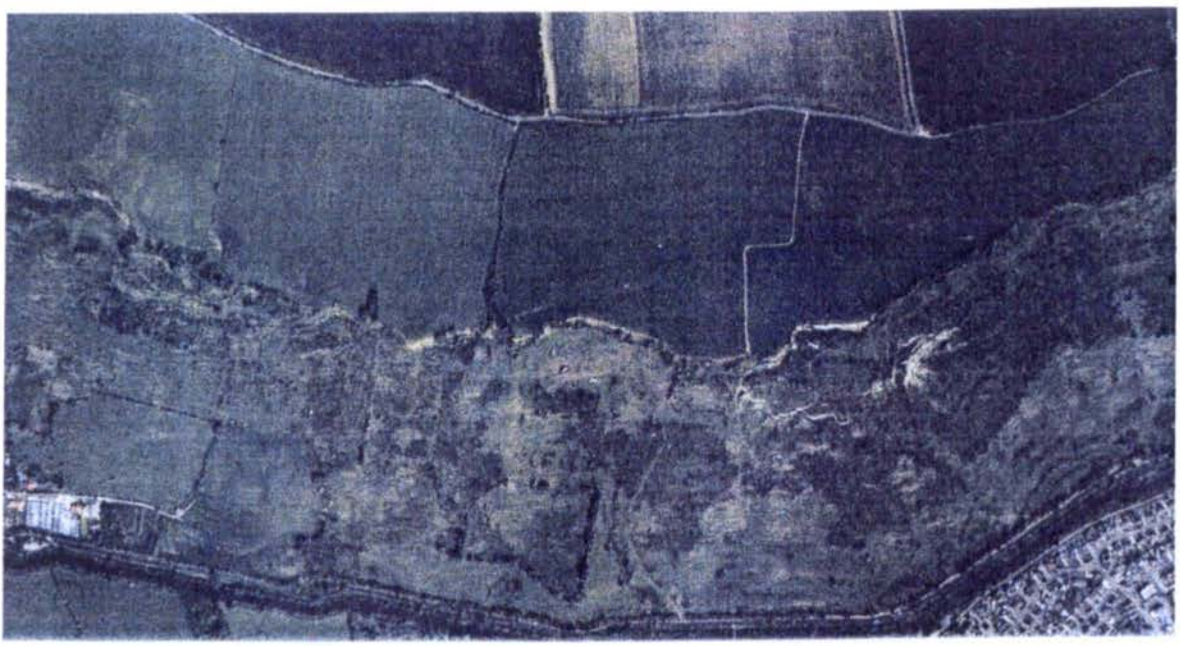


Plate 3.1 (Above). Aerial photograph of The Roughs taken in 1985, before reactivation of the landslides.
Plate 3.2 (Below). Aerial photograph of The Roughs taken in 1990, after reactivation of the landslides.

Plate 3.3 (Above). An example of ponding on The Roughs
Plate 3.4 (Below). The remains of the access road to the Acoustic Research Centre, in the Hythe Beds.

Plate 3.5 (Above). An example of a toe feature on The Roughs.
Plate 3.6 (Below). The surviving listening mirror to the west of the landslides on The Roughs.

Review of landsliding and related sites

4.1 Introduction

Large scale landsliding is usually the result of movement along particular (often low-dip) bedding planes. **Jones and Lee, 1994** define the four types of landslide concentration in the Wealden district and in each case suggest the critical bed responsible for the movements. This is summarised in *Table 4.1.* Definitions of these failure mechanisms are given in *Section 5.2.* At least part of The Roughs landslide complex is formed by a flat-soled, bedding-related, compound landslide. Evidence to support this comes from the widespread occurrence of such landslide types in the district, most importantly at Sandgate, with one of the critical horizons occurring within the geological section. Such a correlation is supported by the fieldwork described in *Chapter 6.*

Review of landslide activity and its correlation with the rainfall record have been undertaken by **Bromhead, Hopper and Ibsen, 1998** and by **Anderson, 1990.** Ibsen, in particular, refers to landslide activity throughout South Kent, not merely in the Lower Greensand Escarpment. The critical sites are identified in this Chapter. Potential hazards resulting from landslides local to The Roughs are shown in *Table 4.2.*

Figure 4.1 shows the locations of the landslides described in this chapter and *Figure 4.2* and *Figure 4.3* respectively, show the locations of the boreholes and trialpits sunk and excavated during the ground investigations mentioned. A summary of the site investigation data is given in *Table 4.3.* There is a large amount of data available but only space to include selected examples here. This excludes information derived from The Roughs site for this project, which is covered in *Chapters 3 and 6.*

4.2 Folkestone, Folkestone Warren and related inland landslides

4.2.1 Geotechnical information

Folkestone Warren stretches from Copt Point (Grid Ref. TR 243365) in approximately a north north easterly direction to Abbot's Cliff (Grid Ref. TR 275386) and is situated on the northern limb of the Wealden uplift. A map of the area is shown in *Figure 4.4.* The exposed strata at the Warren are more recent than those at The Roughs and consist of Middle and Lower Chalk overlying the Gault and Lower Greensand (Folkestone Beds) and dip at about 1° to the NE/NNE. At its western end, the base of the Gault emerges above the shore platform and disappears beneath sea level at the eastern end. "The Warren" is the area of landslipped material between the 100 m high Chalk rear cliff and the sea and is 2.7 km long and 50-350 m wide.

This geological sequence, combined with steep slopes and shallow dips towards the coastline, also exists in Ventnor, Isle of Wight and on the northern coast of France. This is caused by the Weald-Boulonnais anticlinal dome as described in *Section 2.1.2.*

At Folkestone, there is a gentle in-land component of dip due to the local structure (there is a component of dip coastwise to the east as well). At St. Catherine's Point, Isle of Wight, there is a gentle coastward component of dip(as well as the coastwise component). This may be relevant in affecting the head-to-toe size of the slips.

In his survey of coastal landslides of Kent, **Hutchinson**, 1962, covers in great detail, the history of the landslides at Folkestone Warren. A summary, extracted from Hutchinson's survey, of the historical slides in this area is given in *Appendix 4.1.* This is included to illustrate the significance of the coastal landslides of Folkestone Warren *"as among the most important in southern England"* (**Hutchinson**, 1969) and for the sake of pure interest.

Hutchinson, 1969, categorised the land movements into three groups:

- M-type, renewal of multiple rotational land slips
- R-type, smaller rotational slips
- F-type, sliding and falling of chalk masses

Investigations into the causes of the landslides carried out by **Hutchinson**, 1969 and **Hutchinson et al**, 1980, conclude that the landslides are subject to active toe erosion while still being seasonally affected by high water levels. In fact, all eleven of the deep-seated landslides during the past two centuries, have occurred within the four-month span between December and March, during the period of maximum piezometric levels. It is concluded that the increased frequency in the occurrence of landslides during the second part of the nineteenth century and the early part of the twentieth, is due to the interruption of the littoral drift by the Folkestone harbour works. **Hutchinson**, 1969, suggests that *"retrogression of the rear scarp of landslides occurs by a mechanism of progressive failure associated with seaward expansion of the Gault on the removal of its lateral support by marine erosion."*

Trenter and Warren, 1996, concur with the findings in the aforementioned papers in their comprehensive study of the area. They concentrate heavily on the effect of water, in the Chalk and Chalk rubble rather than that in the Folkestone Beds, on the degree of land movement. They point out that the size and mobility of the landslides are greater in the west than the east and attribute this to the weight of the chalk falls from High Cliff and from the undrained loading of the undercliff. From the stability analyses which they carried out, they conclude that:

- The calculated value for φ'_r depended on the slip being analysed (due to variations in the Gault)
- The rate of movement and the factor of safety of these slips was affected by this calculated value of φ'_r
- There was a difference between these back-analysed and the measured values of φ'_r

The cross sections which they constructed through Warren Halt and Horsehead Point are reproduced in *Figures 4.5 and 4.6.* These show clearly the multiple rotational slides present which have basal slip surfaces in the Gault.

4.2.2 Historical references

John Sackette (1716) gives *"an account of a very uncommon sinking of the earth near Folkestone"*. He describes the *"pressing forward of the cliffs and sinking of the hills in the neighbourhood."* **Sackette** is probably referring to the Folkestone Beds when he says, *"The cliffs consist of great ragged sand-stones till we come to near a yard (at some places more) of the bottom; then we meet with what they call a slipe, ie a slippery sort of clay always wet. Upon this slipe at the bottom, they presume that the hard stony land above slides forwards toward the sea, as a ship is launch'd upon tallow'd planks. "*

The cliffs sank down 12 m, the movement forcing up the rocks on the shore resulting in a very high dip angle. The sketch map drawn by Sackette seems to suggest a location around Copt Point where sandstone does indeed exist in the Folkestone Beds, but below the Gault and not above as Sackette describes. Alternatively, he could be referring to the Hythe Beds and Atherfield Clay present in Sandgate. Most likely, however, the area between Folkestone and Sandgate, where the first clays of the Sandgate Beds emerge from beneath the Folkestone Beds, to that to which he refers.

4.3 Sandgate (the Encombe Landslip)

4.3.1 Geotechnical information

Sandgate is the site of well documented historical landslides as well as much land movement to the present day. It is situated on the outer limits of the Weald belt, as can be seen in *Figure 2.1.* Sandgate Beds and, in places, Folkestone Beds are present above the Hythe Beds which are found on The Roughs. The situation at Sandgate is complicated by possible regional dips. There does not seem to be a consensus in opinion as to the exact nature of the land movements in the area.

Bowdler, 1972, proposes a variety of mechanisms to account for the land movement at Sandgate:

- The classical circular-type slip involving the beds of the Lower Greensand series, which form the steep escarpment face.

- A *"solifluctious"* creep (cambering) involving the Atherfield Clay at the foot of the Hythe Beds. (This lobe movement is a similar mechanism to that found at Sevenoaks.)
- Piping and washouts below ground level.
- A combination of the above.

Bowdler concluded that most of the movement was due to cambering and hence, was not looking for the position of a basal slip surface. However, the information derived from his borehole logs, indicated the presence of slip surfaces in the Atherfield Clay although the boreholes were not deep enough to examine the Weald Clay.

An M.Sc. thesis by **C.J. Foster** (1980) contains a review of site investigations. From these, he reached five conclusions:

- He confirms the presence of existing multiple rotational slides and the translational sliding of a superficial mantle of structure-less 'landslide debris.'
- This superficial mantle has a well defined basal slip surface at its interface with the undisturbed, often deeply inclined, strata below.
- This mantle is thicker at the top of the slope, masking large movements at the rear scar of the landslip.
- The rotational movements have resulted in much of the strata being back tilted.

A section through the Encombe landslide suggested by **Sir William Halcrow and Partners**, 1986, is shown in *Figure 4.7*. This section shows the basal slip surface to be in the Weald Clay but this was probably a misinterpretation of, or insufficient data.

Palmer, 1991, describes the Encombe landslip as a multi-rotational progressive slip which displaces Atherfield Clay, Hythe Beds, and Sandgate Beds, which can be seen on the cross section of the landslip shown in *Figure 4.8*. This diagram shows the basal slip surface to be in the Atherfield Clay which is the interpretation recognised by the Author.

According to **Palmer**, 1991, there is a strong correlation between 6-monthly and 12 monthly rainfall accumulations and land movements, a fact which is examined in further detail in *Section 8.5*. He also concludes that the land movements are directly related to the volume of shingle present on the beach and tidal variation, which had also been noted by **Topley**, 1893. **Topley** also says of the landslides along the stretch of coast at Sandgate *"Special local causes may possibly have some influence* *in determining the exact position and origin of any landslip along this coast; but the main cause is always the same - the saturation of the land by heavy rains."*

There are, in fact records of the beach level at Sandgate, dating from 1720. In more recent times there have been records kept of rainfall levels and, post 1969, careful monitoring of land movement.

4.3.2 Historical references

Topley (1893) refers to the earliest slides which took place in 1827 in the east part of Sandgate. The major landslide in 1893 affected the west of the Sandgate whereas the present day slides are causing greatest problems to the centre, between the two previous sites.

The major landslide took place between seven and eight o'clock on the evening of 4th March, 1893. It resulted in great destruction of property and according to the Folkestone Express article titled "Terrible Landslip at Sandgate" more than seventy houses were damaged. The extent of the damage varied from negligible to complete destruction. There was no reported loss of life.

Topley (1893) describes this landslip. He notes that the slipped faces of the clay were fresh and *"were streaked with true slickensides running obliquely down the face"*. He quotes rain gauge readings taken by Mr H.B. Mackeson that recorded that the average reading for February 1883-92 at Hythe was 1.95 inches (0.049 m) (13.8 wet days), compared with a reading of 4.3 inches (0.109 m) (24 wet days) in 1893. Topley discounts local rumour that the landslide was influenced by the blowing up of two off shore wrecks, the Calypso in the summer of 1891 and the Benvenue in the autumn of 1892. However, he does mention that groynes built on the east of Hythe depleted the sea front of Sandgate of shingle. The effects of the lack of shingle should have been felt in the wet weather and the lack of foreshore support may have determined the time of the slides: low spring tide and the following low tide.

Topley (1893) then describes the plan to prevent further slips recommended to the local board by Mr Baldwin Latham. *"Deep drains must be carried along the back of the undercliff, to carry off the water percolating from the Folkestone Beds above; the undercliff must also be thoroughly drained, and no surface water, other than that due to the rainfall on the area itself, must be allowed to enter the ground. "*

Blake (1893) writes about the Sandgate landslide, *"In the case before us, given that a landslip was bound to occur in a spot which is largely covered by buildings, it could not have occurred in a milder or more favourable manor."* He attributes the landslip to sliding over the dampened Sandgate Beds.

The **Folkestone Express,** 1893 describes the massive but localized damage to property caused by the slide, the blame was entirely on the heavy rains.

The present day movements were believed to have commenced in the 1930's and have been only marginally decreased by the drainage scheme installed in 1975.

4.3.3 Recent investigations

South Eastern Soils Limited carried out a series of investigations for Shepway District Council to assess the ground conditions as part of a scheme to construct a foul sewer and a pumping station. The first stage, in the autumn of 1982, involved the excavating of three trial pits and thirteen boreholes, the positions of which are shown on *Figures 2.2 and 2.3.* Samples were recovered to log and test.

In August, 1983 the work for Phase 1 focussed on the area around the Pumping Station and the sewer under the Royal Military Canal, (Grid Ref: c. TR 160345). The ground investigation consisted of eight boreholes and one hand excavated trial pit. Samples were recovered and underwent testing in the laboratory and in situ falling head permeability tests were carried out.

During the winter of 1984/85, work was carried out on Phase 2 of the scheme. This concentrated on the coastal plain area of Hythe. The investigation included the sinking of ten boreholes. The positions of the boreholes and trialpits are shown in *Figure* 4.2 and *Figure* 4.3, respectively.

Bowdler, 1972, gives borehole logs for three boreholes in the Sandgate area. The position of these is shown in *Figure* 2.2.

4.4 Hythe area

4.4.1 Geotechnical information

Before Hythe is reached, the Atherfield Clay, which is the critical bed comes up to sea level and then rises up the slope, all this is a function of doming of the Weald. Landslides are perched high (becoming higher) in cliffs in a westerly direction.

The town of Hythe itself it built on a landslipped slope. Drainage works, however, have prevented almost all damage to property in living memory. The formation of Romney Marsh and the geology and geomorphology of the area is covered fully in *Sections 2.3 and 2.4.* A cross section through the Lower Greensand Escarpment, between the research site and Hythe, is shown in *Figure 4.9.*

4.4.2 Historical references

In his writings about the Encombe Landslide, **Topley, 1893**, refers to the frequently occurring slips along the Atherfield Clay, near Hythe.

4.4.3 Recent investigations

The M.Sc. by **R.J. Anderson, 1990**, includes borehole and trial pit information from The Roughs. A series of seven very shallow hand-augured boreholes were drilled on a cross section (Grid Ref. TR 142341-140345), which is on the site of current research and will form an interesting comparison.

A ground investigation was carried out by **A.G Weeks and Partners, Ltd.**, on behalf of R.P Furlong, on a house in Seabrook Road, Hythe (Grid Ref. TR 180348). The investigation was requested after a landslip occurred behind the retaining wall at the rear of the property.

Three boreholes, which are shown on *Figure* 4.2, were sunk, two using a cable percussion tripod rig and one using a hand auger. Atterberg limits and residual shear strength parameters were determined for samples recovered from the head deposits and the landslipped Atherfield Clay. A cross section through the site, which shows the probable slip surface in the Atherfield clay, is shown in *Figure 4.10.*

Another ground investigation was carried out by **A.G. Weeks and Partners, Ltd.**, on behalf of M.A. Jackson, on the most northeastern corner of The Roughs, (Grid Ref. TR 145349), in October, 1988. The investigation was carried out in preparation for the building of four detached houses on the sloping site. This site was known to be situated on a landslip mantle with visible standing water. It was anticipated that the stability of the landslip mantle could be adversely affected by house construction particularly if the slip surface on the underlying Weald Clay was shallow and the groundwater table was high.

Nine trial pits and four boreholes were excavated on the site. These confirmed that landslip debris existed to depths of 5 to 6 metres at the west end of the site and 2 metres at the east end. This debris varied from silty and sandy clays to ragstone boulders. Slip surfaces were identified at the top of the Weald Clay within the landslipped Atherfield Clay in the top 4 to 6 metres of the formation.

An investigation into the causes of distress on his property, was carried out by **A.G. Weeks and Partners, Limited.**, on behalf of Dr Wells. The investigation was carried out in 1988 when the distress had been occurring for the previous 18 months. The house was located on a platform formed by cut and fill on the north of Cliff Road, (Grid Ref. TR 179 351).

It was thought possible that the steep slope behind the property may have been formed by regression of the backscar of a mapped landslip, which was believed to be 60 metres away and that the backscar of an associated minor slip or even the main slip, could be much closer than the maps would suggest.

Three trial pits, four hand-augured boreholes and one shell and auger borehole had been excavated on the site, in June 1988 by Pynford South Ltd. A further five boreholes were drilled by A.G Weeks and Partners Ltd to determine the position of the backscar of the major landslip in relation to the position of Ty-Fry. These were continuously sampled and the samples taken were logged and examined for slip surfaces. Three of the boreholes were instrumented with inclinometers and two with piezometers. A geological section was compiled and is shown in *Figure 4.11*. It was found that a major slip surface existed which emerged close to the front of the house.

A.G. Weeks and Partners, Limited later carried out an investigation on a neighbouring site to Ty-Fry, also on Cliff Road (Grid Ref. TR 178351). A series of five shell and auger boreholes were sunk, the positions of which are shown on *Figure 4.2*. The boreholes showed the presence of steeply inclined slip surfaces immediately above the Hythe Beds.

An investigation in connection with a proposed marina by E.N. **Bromhead** (1990) includes a study of local landslips and detailed maps (Grid Ref. TR 180347). The marina was due to be constructed at the eastern end of the Military Canal. There were local concerns that the construction of the marina would impede the movement of beach shingle, unloading the toe and hence cause land movements.

The investigation was carried out in three stages: first a desk study and literature review, secondly a series of walk over surveys and thirdly a sub-surface investigation. It was concluded that the construction of the marina would not cause the destabilisation of any existing ancient landslides since, landwards of the marina site, the Atherfield Clay was at too high an elevation for slip surfaces to break out under the marina and therefore any excavations in conjunction with the marina would not unload any of the existing toes.

4.5 Lympne

4.5.1 Geotechnical information

Lympne is also situated on the Lower Greensand Escarpment. The slope at Lympne is at a lower angle than at Hythe due to earlier abandonment and is considered more stable. This is covered more fully in *Section 2.4.1*. The rear scarp has been used for quarrying and is therefore more degraded than would be expected.

Work on Lemanis Castle by **Hutchinson et al**, 1985 (see *Section 4.5.2*) has revealed details of the geomorphology of the escarpment at Lympne. *Figures 4.12 and 4.13* show the geomorphology of the site in plan and cross section respectively. The four distinctive zones on the slope are:

- Rear scarp of inclined Hythe Beds
- A degradation zone with landslide debris of thickness 1.5-3.5 metres
- An accumulation zone with landslide debris of thickness 8-9 metres
- A toe over lying the edge of Romney Marsh

4.5.2 Historical references

There are documents available, one a letter (**Anon.**, 1728) and the other by **Topley** (1893) referring to a landslide which took place in 1725, at French House (Grid Ref. TR 112347), Lympne, to the west of the research site. This may have been the landslide responsible for the toppling of the remaining walls of the ruined Stutfall Castle. *Figure 4.14* shows an engraving by Stukeley of the still intact northwest walls of the castle below, what is, probably French House in 1722.

An anonymous man (**Anon**, 1728) who describes himself as a "*rude Designer*", apologising for his diagram which is reproduced in *Figure 4.15*, describes what appears to be a rotational slip in a letter to Mr P. Collinson. He says that the slip took place in 1726 due to a "*very wet season*" when undrained waters on the uplands caused a "*quick sand at some considerable depth in the earth.*" A slide occurred at night which caused the brow of the hill to sink by 40-50 feet and the lower half to be raised by a similar height. The wooden farmhouse on the brow of the hill was not only lowered but moved forward, it survived. A strongly built stone barn was destroyed. *"The ground sunk at night, and was not perceived by the farmer's family till they found the change in the morning, by the door-cases not suffering the doors to open."*

The position of the ruins of the Roman Fort Lemanis, also known as Stutfall Castle, (Grid Ref. TR 118343) are shown in *Figure 4.1*. Lemanis was one of a series of ports with direct road links to Canterbury, used as supply routes. The walls of the castle were thought to have finally collapsed in the eighteenth century. Investigations have been taking place on the site since the beginning of the sixteenth century. More recent research has been carried out by archaeologists including the excavations by Charles Roach Smith in the 1850's, Barry Cunliffe in the 1970's and Hutchinson et al in 1985. These excavations and related research indicated that the fort was built in c. A.D. 275-280 to protect Fort Lemanis and abandoned in c. A.D. 340-350. The marsh was almost fully formed at this stage but a narrow entrance to the estuary still existed, which had disappeared by the start of the second millennium. An investigation of the ruins of Stutfall Castle was carried out by **Hutchinson et al.,** 1995. The mechanics of collapse of the uppermost sections of the Roman masonry walls were re-evaluated by **Hutchinson and Bromhead,** 1996.

At the time the fort was built, the slope was not at equilibrium: the degradation zone would have been more active and hence the accumulation zone would have been steeper and the toe would have been further back. The Romans built the fort on the accumulation zone and used wooden piles in an attempt to stabilize the walls. An indication of the original positions of the walls is given by the position of these piles.

Therefore, the escarpment has been active for many centuries. A similar set of conditions to that which now exist on The Roughs may have been present at the time of the collapse of Stutfall Castle. The morphology of the rotational slide elements in the slopes north west of Stutfall Castle is obscured by construction works to redevelop French House, and to provide sewage treatment works for Lympne Airfield in the 1940's. However, they are much clearer in the slopes beneath Lympne Castle, Church and churchyard. No doubt a Roughs-scale landslide here could have affected the north east walls of Stutfall Castle. However, a further possibility is of a mudslide, originating from springs near the present-day pond reaching the very toe of the slope, as does the eastern mudslide at The Roughs. This would account for many of the distortions in the eastern wall of the Roman fort.

4.5.3 Recent investigations

Slightly to the east of the castle is the site of the A259, a new link road to the M20. Two large scale ground investigations have been carried out in connection with the new road. The road was planned to run in approximately a north south direction through the escarpment (Grid Ref: TR 130342-60). Both investigations were carried out for the consulting firm **Owen Williams and Partners Limited**.

A ground investigation in 1989 by **Norwest Holst Soil Engineering Limited** into the area between Lympne and The Royal Military Canal and along the escarpment, east to Hythe, has provided trial pit and borehole data and piezometric readings in addition to the results of laboratory tests on soil samples. A total of fifteen boreholes was sunk, twelve of which were located in six pairs. One borehole in each pair was instrumented with a piezometer and the other with a slip indicator.

Three trial pits were also excavated. The positions of the boreholes and trialpits are indicated on the map in *Figure 2.2* and *Figure 2.3.* Cross sections of the escarpment compiled with data from these boreholes are shown in *Figure 16.*

A more detailed investigation, once the route of the A259 had been determined, was carried out by **Exploration Associates** between April and July, 1992. Twenty-seven boreholes were sunk by light cable tool percussion methods, twenty by rotary coring methods and five more by a combination of methods. Twenty-seven of these boreholes were instrumented with standpipe piezometers, two with inclinometers and six with slip indicators. Nine holes were hand-augured.

Thirty-eight trial pits were sunk using a back-hole excavator and four were hand excavated on the Roughs themselves. Electrical static cone penetration testing, water monitoring and permeability testing were also carried out followed by extensive laboratory testing on recovered samples. The positions of the boreholes and trialpits are shown on *Figures 4.2 and 4.3.*

4.6 The Roughs in context

To investigate the landsliding on The Roughs, taking into account other landslides in the locality is helpful. Generally, two types of bedding controlled, flat-base, landslides exist:

i. Folkestone Warren and Sandgate as models, where the basal shear is at or below sea level.
ii. Lympne, (The Roughs too), Folkestone East Cliff as models - where the basal shear is high up in the cliff (i.e. "perched" landslides)

In both cases, the critical horizons for sliding have been identified (sometimes very tentatively, as in Sackette). Therefore, it should be possible based on the literature alone, and certainly when backed up by the surface morphology, to predict the existence of bedding-controlled perched slide surfaces at The Roughs.

Secondly, the literature contains many references to intermittent slide activity. Water is almost always implicated as a mechanism, usually in combination with heavy rainfall. Tidal effects are also thought to be a contributory factor at many sites. Toe erosion is less often mentioned. Only at Folkestone Warren does the literature refer to loading from collapses of the rear scarp - but then the movements here are uncharacteristically large. Even the classic Sandgate landslides only moved a few metres.

Conversely, the big difference between The Roughs and Lympne and the other sites is the absence of toe erosion in recent times, allowing the build up of the accumulation zone.

4.7 Summary

The Roughs is therefore situated in a region where the combination of geology, topography, climate and coast erosion combine to cause frequent landslide movements. The major landslide systems, at Folkestone Warren and Sandgate, are deep-seated landslide complexes, with bedding-controlled basal slip surfaces, and with toes which break out in the foreshore. They are therefore subject directly to toe erosion. **Hutchinson,** 1969 and **Hutchinson *et al.*,** 1980 argue that the interception of litoral drift has had a major impact on the stability of the Warren complex. Hutchinson follows this toe erosion theme with the model for the movements affecting Stutfall Castle (**Hutchinson, et al.** 1985), and therefore needs a remote date for the damage, at a time when marine erosion at the foot of the slope was active.

However, there is an equal body of evidence and opinion to support the view that the landslide activity is due primarily to rainfall which is discussed in *Chapter 8*. Certainly, toe erosion is not involved at all in the activity of landslides on the inland slopes analogous to Folkestone Warren at the Channel Tunnel terminus, although these have not shown the degree of activity of the Warren, either.

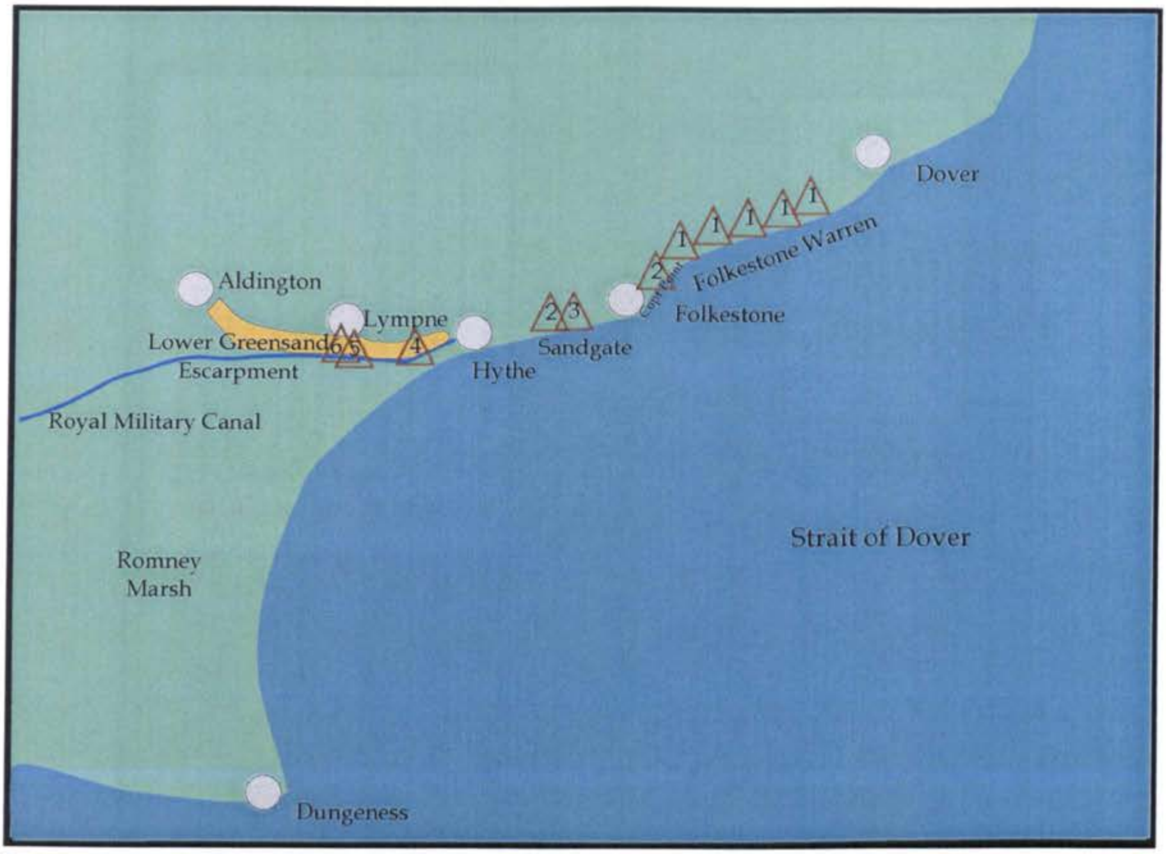
Only at Folkestone Warren is the rear scarp high enough to generate a rock-fall hazard, and the evidence from the 1915 landslide is that major landslide displacements occurred in response to a significant rear scarp collapse.

If Folkestone Warren and the Sandgate landslides have bedding control of the position of the basal shear surface(s) then these surfaces are located at an elevation lower than the slope toe. There are also numerous examples of perched slides. In the Folkestone East Cliffs, the Gault caps the slopes, and slide failures occur in this upper stratum. Coast defence here is comparatively recent, and the Folkestone Beds cliffs are still exposed, and to a certain extent, still fail. Similar perched landslides occur between Sandgate and Folkestone, but on a scale intermediate between the small slips of Folkestone East Cliff and the Sandgate and Folkestone Warren systems. In addition, the density of housing development makes them more difficult to study. There may be bedding control over slip surface location, but this is not certain.

Slides of Chalk and Gault on the site of the Channel Tunnel terminus do not appear to exhibit the strong basal shear of the other major systems. Past activity of these slides, is, however, unconnected with marine erosion.

Finally, the Lower Greensand Escarpment exhibits:

- Perched, bedding-controlled, landslides active intermittently at the present day
- Absence of marine erosion (in historic times)
- A substantial accumulation of side debris on the main part of the slopes and at their toe.



1	Folkestone Warren, numerous slides, especially 1765, 1877, 1886, 1896, 1915, 1937 and 1940.
2	Possible sites of 1716 slide described by John Sackette.
3	Encombe landslip, 1827 and 1893.
4	The Roughs landslide, 1988.
5	Site of Stutfall castle, destroyed by landsliding during the last two millennia.
6	French House, 1725.

Figure 4.1. Location of landslides in the vicinity of The Roughs.

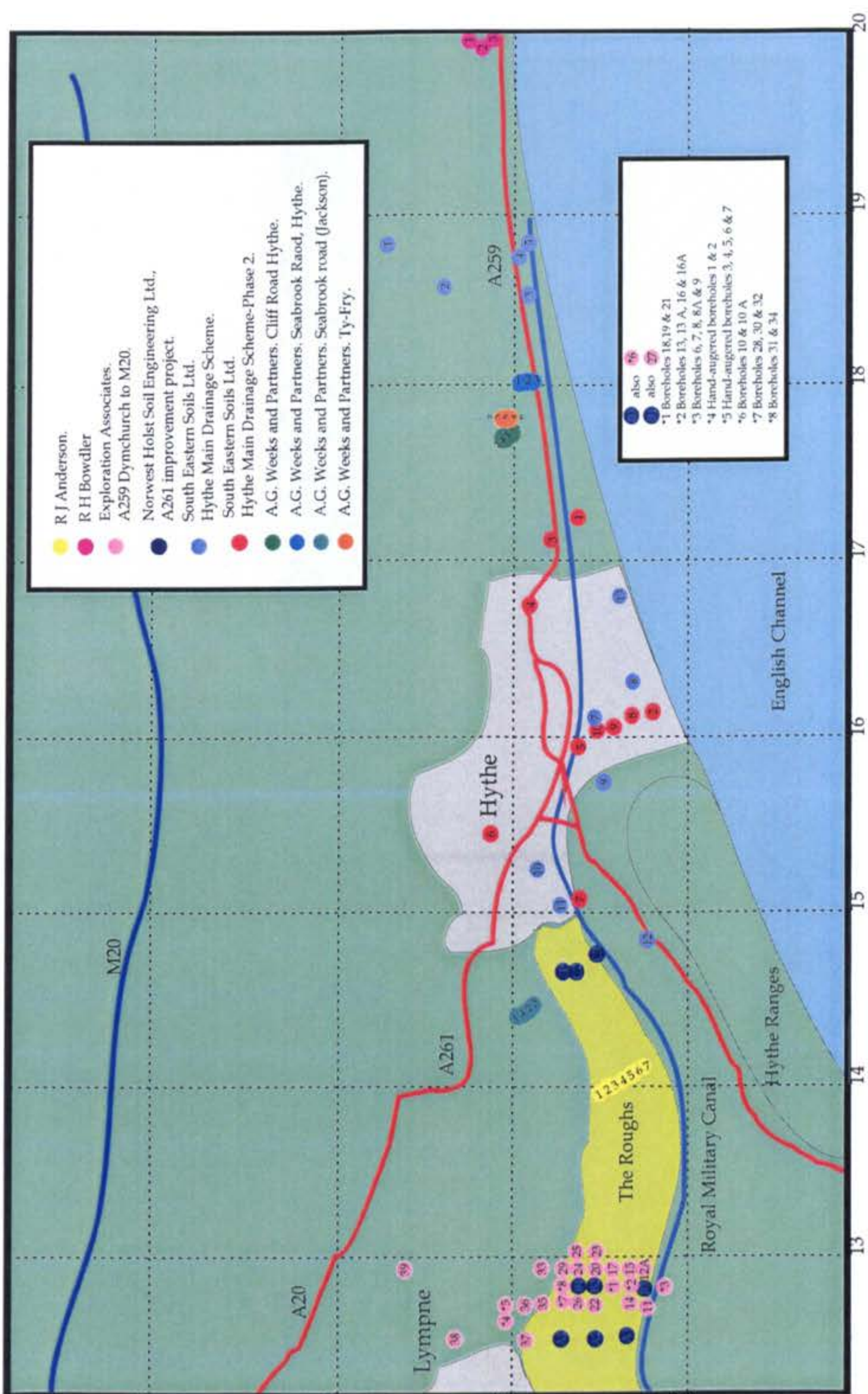


Figure 4.2. Location of boreholes in the vicinity of The Roughs.

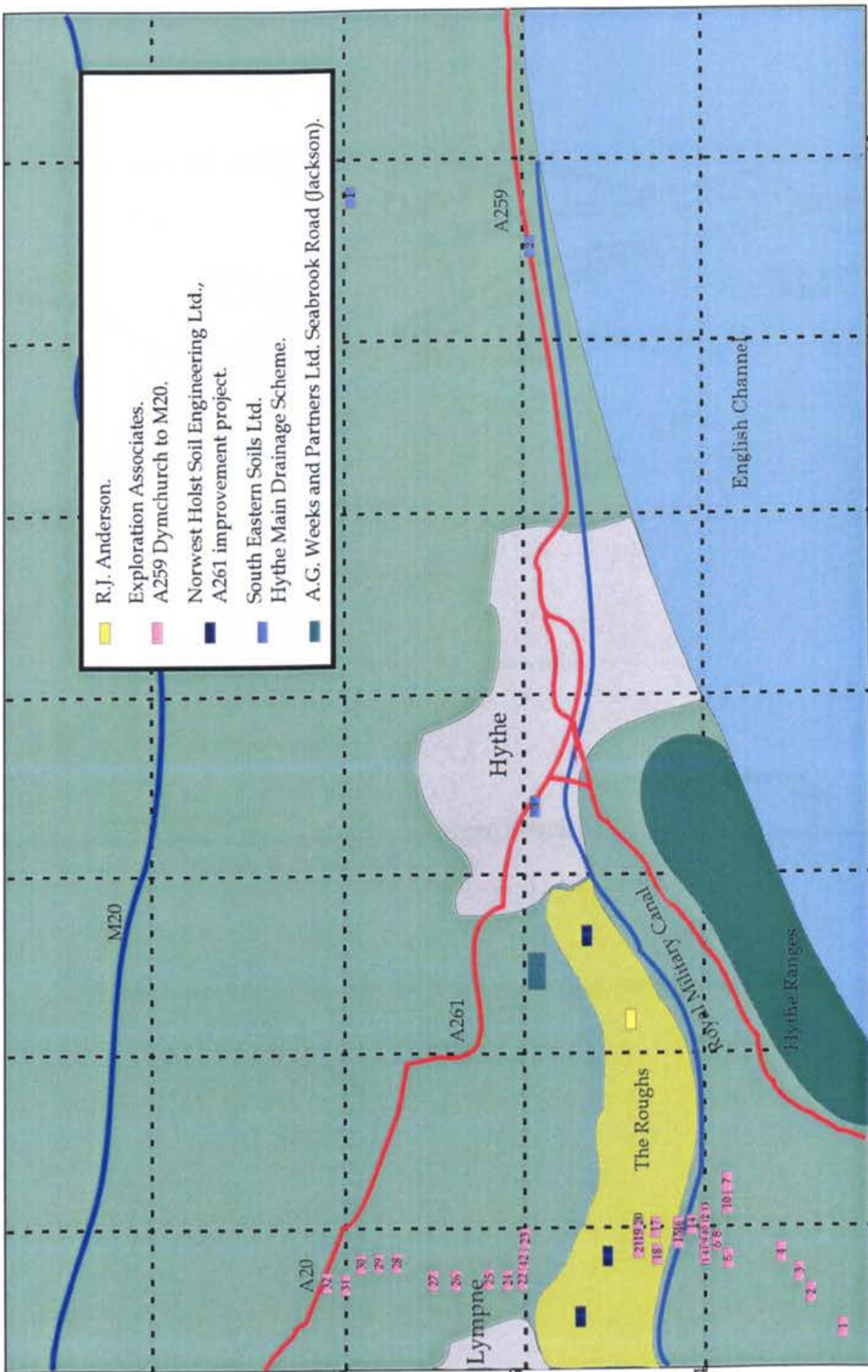


Figure 4.3. Location of trialpits in the vicinity of The Roughs.

Figure 4.4. Map of Folkestone Warren. (After Hutchinson et al, 1980.)

Figure 4.5 (above) and Figure 4.6 (below). Cross sections through Warren Halt and Horsehead Point respectively. (After Trenter and Warren, 1996.)

Figure 4.7 (above) and Figure 4.8 (below). Two cross sections through the Encombe landslip. (After Sir William Halcrow and Partners, 1986 and Palmer, 1991 respectively).

Figure 4.9. Geological section through The Roughs. (After Anderson, 1990).

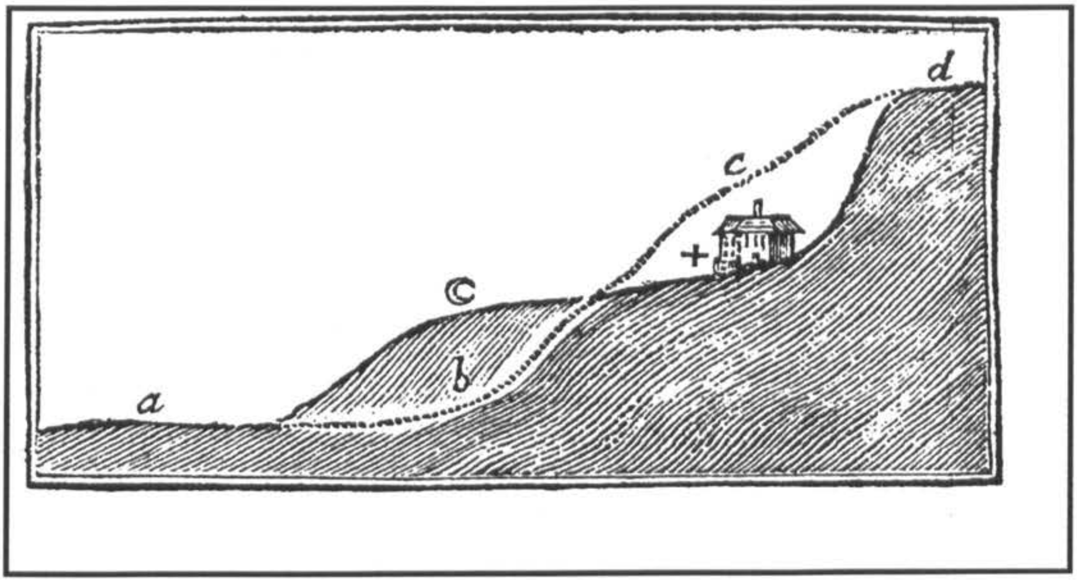
Figure 4.10. Cross-section through Seabrook Road, Hythe. (After A.G.Weeks & Partners Ltd.)

Figure 4.11. Geological cross-section through Ty-Fry. (After A.G.Weeks and Partners Ltd., 1989)

Figure 4.12. Plan view of Stutfall Castle showing geomorphological areas. (After Hutchinson et al., 1985).

Figure 4.13. North-south cross-section through the Lympne Escarpment immediately west of Stutfall Castle. (After Hutchinson et al, 1995).

Figure 4.14. Representation of the engraving by Stukely of Stutfall Castle.



"a b c d the profile of the Land.
a the flat Land at Bottom 3 or 4 Mile from the Sea.
d the flat land at Top, stiff Ground and rocky.
+ The Place of the farm at present, which not
only sunk down from d 40 or 50 Foot, but was
also moved somewhat towards a.
b the lower Part raised to ℂ "

Figure 4.15. Diagram of the rotational landslide at French House. (After Anon., 1728)

Figure 4.16. Profile and interpreted stratigraphy from boreholes performed by Norwest Holst. (After Anderson, 1990).

Table 4.1. Table showing the four main landslide concentrations in the Wealden District of South-East England (based on information published by Jones and Lee, 1994).

Area of the Weald	Type of failure	Critical bed
Lower Chalk	Large rotational slides and rock falls	Gault Clay
Upper Greensand	Large rotational slides and compound failures	Gault Clay
Lower Greensand	Cambering and multiple rotational slides	Weald Clay, Atherfield Clay & Hythe Beds
Central Weald	Shallow rotational and translational slides	Weald Clay, Wadhurst Clay, Ashdown Beds & Purbeck Beds

Table 4.2. Hazard potential of landslides in the areas local to The Roughs

Location	Landslide hazard potential
Folkestone Warren	Railway, walkers, the railway infrastructure of seawalls etc
East cliff	The public, seawalls
Sandgate	Residential, business
Hythe	Residential, business
Roughs, Lympne	Residential, archaeological interest, Port Lympne

Table 4.3. Table summarising details of site investigation information.

Company	Number of trialpits	Number of boreholes	Location
Anderson, 1990	1	7	Lower Greensand Escarpment
Bowdler, 1972	-	3	Sandgate
Exploration Associates, 1992	42	52	Lower Greensand Escarpment South of Lympne
Norwest Holst Soil Engineering Limited, 1989	3	15	Lower Greensand Escarpment
South Eastern Soils Limited, 1982	3	13	Hythe
South Eastern Soils Limited, 1983	1	8	Hythe
South Eastern Soils Limited, 1984/5	-	10	Hythe
A.G Weeks and Partners Ltd. July, 1988	-	3	Seabrook Road, Hythe TR 180348
A.G Weeks and Partners Ltd. Nov, 1988	9	4	Seabrook Road, Hythe TR 154349
A.G Weeks and Partners Ltd. Dec, 1988	-	5	Ty-Fry, Cliff Road, Hythe TR 179351
A.G Weeks and Partners Ltd. Sept, 1989	-	5	Cliff Road, Hythe TR 178351

Landslide types and theory

5.1 Introduction

In the limited space available in this chapter, the author does not aim to replace the number of comprehensive books on landslide theory, but merely to clarify some relevant terms and theories. In earlier chapters, The Roughs landslides have been referred to as reactivated rotational/transitional, climate-triggered landslides with an abandoned cliff, which were once coastal and are presently inland. This chapter puts these terms in context. It then goes on to describe some processes which occur at or below surface level, which contribute to the landslide process. The chapter then concludes by a brief explanation of terms relating to landslide stability analyses, which are examined further in *Chapter 9*.

Skempton and Hutchinson, 1969, introduce landslide theory succinctly, *"Mass movements occur chiefly in response to gravitational forces, sometimes supplemented by seismic activity. The manner in which a slope yields to these forces is controlled by a multitude of factors, of which geology, hydrology, topography, climate and weathering are the most important."*

5.2 Landslide types

This section should, perhaps, be titled "types of land movement" but the term landslide tends to be used as a generic term. There are other methods of classifying land movements, but the simplest seems to be a categorization into the following three groups:

- Falling
- Flowing
- Sliding

This method was devised by **Skempton and Hutchinson**, 1969. Most landslides are a combination of two or more or the above and are known as complex landslides. According to **Jones and Lee**, 1994, of those mass movements which have been split into their component classifications in the UK, 63% are slides, 18% are flows and the remaining 16% are falls (presumably 3% are undefined).

Falling occurs from steep slopes and cliffs under the action of gravity. In Great Britain "falling" most commonly occurs from coastline cliffs as the sea undermines the base of the cliff. Inland, rocks may be dislodged by the action of water in joints as it freezes, or by the action of tree roots. The debris formed can form talus or scree slopes.

Sliding involves the movement along an inclined surface of a mass with well-defined boundaries.

Depending on whether the sliding surface is planar or curved, these slides can be classed as translational or rotational.

Translational slides are generally shallow and are caused by the presence of a heterogeneity. Types of translational slides include:

- rock slides, movements of rock masses along discontinuities
- debris slides, shallow slides of weathered materials and superficial deposits
- slab slides, shallow slides on weakened clay slopes
- mudslides, slow moving softened debris over basal shear surfaces
- peat slides, caused by peat bogs swelling due to rain
- block slides, material remains intact and undeformed as it move over shear surfaces

Rotational slides which can be circular (normally in slopes of uniform clay), or non circular in nature, can occur singularly or in multiple groups. They are often *retrogressive* as successive rotational slides eat back into the slope.

Compound slides or translational (non-rotational) slides are hybrid of translational and rotational slides. Their characteristic shape is the result of the mechanism outlined in *Section 5.4.2. Figure 5.1* shows diagrams of typical rotational and translational slides. Various methods of analysing these slides are briefly discussed in *Section 5.10*

Flowing is the least understood type of land movement. **Skempton and Hutchinson**, 1969, divide this class into three divisions: earthflows, mudflows and solifluction lobes and sheets. Flowing involves more internal deformation than sliding.

Cruden and Varnes, 1996, class the speed of landslides into seven categories, varying from extremely rapid to extremely slow. These are shown in *Table 5.1.*

5.2.1 Coastal landslides

Along the coastline, the action of the sea is the major contributing factor to both erosion and deposition. In some locations on the coast, it is necessary to construct sea defenses to prevent the eroding action of the sea on coastal cliffs. These stabilisation works can take the form of sea-walls, groynes and breakwaters. Because of litoral drifts, distant alterations along the coastline, such as harbour construction, can add to or deplete existing beaches. Occasionally, such as on The Roughs, the coastal cliff may be abandoned altogether and assume a state of free degradation.

According to **Hutchinson, 1967**, landslides in coastal cliffs which are subjected to fairly strong marine erosion, tend to be rotational or compound, sliding on deep or moderately deep slip surfaces. Under less severe conditions, shallow slides and mudflows dominate.

5.2.2 Inland landslides

In the UK, landslides on inland slopes, which are not caused by human activity, are usually caused by the action of a river at the base of the slope. Weathering and creep can also be factors. Others are often the reactivation of pre-existing landslides by human intervention or by even by climactical changes. Some landslides are totally man-made, such as the collapse of industrial waste or slag heaps, with sometimes catastrophic consequences, such as that in Aberfan in 1966.

5.3 Landslide triggers

Landslides can be activated or reactivated by a number of both artificial and natural causes or a combination thereof. The most important trigger is water. Rising ground water levels, whether attributed to global climate change or more obvious man-made causes such as reservoir building, causes a reduction in effective stress in the soil and hence a reduction in resistance to shear. Water can also be driven into cracks in rock, freezing and forcing open joints. An unbalancing of forces acting on a slope, by removing weight form the toe of a slope or adding it to the head, will result in land movement to regain equilibrium. Toe removal may be a result of erosion by the sea or by man-made excavations for construction. Head loading can also be caused by natural or artificial means such as rock falls or construction. Vibration, which can be caused by earthquakes or even heavy traffic, can raise stress levels in soils or fracture rocks. Deforestation has had serious consequences in recent times and resulted in large loss of life. Strength reduction, in the form of weathering and creep (*Section 5.5*), will weaken soils and may eventually lead to collapse.

5.3.1 First time failures

Table 5.2 gives a geotechnical classification of landslides, differentiating between first time slides and reactivations. First time failures often occur due to the undermining of previously unsheared ground by river or tidal action or by man-made excavations. If the soil is brittle, that is there is a large difference between the peak and residual stresses, this first time movement may involve large displacements. When a load is applied to previously unsheared ground, the peak strength may be mobilised. The soil must then redistribute the stress if it is to be sheared further, the stress is then transferred to neighboring soil elements and shear surfaces are formed. The result is a landslide of some description, along with pore water pressure dissipation and reestablishment of stress equilibrium.

First time failures occasionally involve the mobilisation of the peak strength of the soil however they usually occur in clay which has reached the fully soften state. Slow movements may precede a slide

.

5.3.2 Reactivation of landslides

The Department of the Environment have developed a scheme to classify landslides into five categories:

i. Active (currently moving or having moved within the last five years)

ii. Recent (last 100 years)

iii. Relict (100-1000 years)

iv. Fossil (historic and prehistoric)

v. Unknown (presumably relict or historical since movements are unrecorded)

A survey of landslides was then carried out which recorded the number of coastal and inland landslides in each category. These results are presented in bar chart form in *Figure 5.2.* Unsurprisingly, since most are under continual attack, the coastal slides were primarily active or recent. Inland, many of the "youthful" slides occurred in the South Wales Coalfield, the London Clay of Essex, the Lower Greensand of Kent and the Ironbridge Gorge of Shropshire. There is only a handful of active sites in Scotland. Landslides in the Lower Greensand (Hythe Beds) also feature strongly in the ancient list. Clearly, there are many "inactive" landslides in the UK. Do these landslides pose a problem?

Once a landslide has occurred, the clay along the slip surfaces is likely to be at residual strength, rather than at peak strength. This implies that less energy is needed to reactive the landslide, in its weakened state, than to cause the initial movements, but the causes of landslide reactivation are much the same as those for a primary activation:

- head loading
- toe unloading
- an increase of ground water levels
- earthquakes

One problem is that it is not always obvious that a slope is landslipped, once the characteristic features, such as grabens and lobes, have degraded over the years. Consequently, when the site is altered in some way such as new drainage outlets or construction, it moves. This is usually an expensive mistake but is sometimes also a dangerous one. Another problem, as discussed in

Chapter 8, is the changing climate. It is likely that as rain increases, so is the likelihood of the reactivation of many sites around the country.

5.4 Factors defining the shape of a landside

Many contributory factors define the shape and position of a landslide. These include the action of the sea, or not, the presence of discontinuities in the soil structure and the geological strata among many others. Here, the formation of abandoned cliffs is discussed, as well as bedding control and perched landslides. The terms "discontinuity" and "shear surface" are often referred to in the context of landslides, and they too are examined.

5.4.1 Abandoned and defended cliffs

Slopes subject to toe erosion may be found in several locations: on the coast, lake shores, and where rivers are actively downcutting or meandering towards hillslopes. Under the precise combination of erosive intensity, groundwater and geology, a variety of forms of landslide can occur. The lifetime of these and their geomorphology at any time in their lifetime may follow a cyclic pattern. Toe erosion can defy the processes which combine to flatten a slope.

Should the toe erosion cease for any reason, the sub-aerial processes take over. There are a variety of reasons why toe erosion may cease. One reason is the growth of a marsh or beach at the toe of the slope which prevents landslide debris from being eroded, so that it accumulates. Another reason is tectonic land uplift. A river may meander away from the toe of a slope. A lake may dry up, or sea level may fall with the onset of glaciation. There are a variety of reasons for the occurrence of an abandoned cliff. Man-made defences lead to the same result: the slope is then a defended one.

Under toe erosion conditions, failures are frequent. Of course this depends on the intensity of erosion. However, when the accumulating debris is not transported, the slope is flattening - its head continues to retreat landwards, and the toe pushes out in the "seaward" direction. Eventually, activity will be confined to wet season only movements, and then to not-every-year wet season movements, and then to infrequent movements. Eventually, we get to a long return period between movement events, and such a slope is normally considered stable.

However, movements of the slope could occur if the geometry is changed, for example by reestablishment of erosion conditions, inappropriate earthworks (cut or fill), artificial recharge of groundwater or by a change in the climate. This is because the equilibrium which is reached is critically dependent on moisture balance.

The Lympne Escarpment has an abandoned sea cliff running along its length. Abandoned cliffs, such as these, were formed during the Pleistocene, a period of sea-level and temperature variation (*Table 2.4*). The process of abandonment is as follows:

- The sea levels rise and the toe of the slope is removed by marine erosion. (*Figure 5.3 (upper)*)
- Regression of the cliff. (*Figure 5.3 (upper)*)
- Sea levels fall and the newly formed "cliff" is abandoned. (*Figure 5.3 (upper)*)
- The slope degrades naturally, to a point of equilibrium, alluvium covering the abandoned cliff. (*Figure 5.3 (middle and lower)*).

This process continues until the sea-levels stabilise. The higher the sea, the further back on the escarpment the abandoned cliff is formed. During this period, periglacial conditions existed in the south of England. This allowed the formation periglacial landslides. These can occur in very low angled slopes due to the high pore water pressures in the soil which exist during the annual thawing process.

The most well documented case of an abandoned cliff is at Hadleigh, Essex, at the junction of the River Thames and the North Sea. It is the site of the mediaeval Hadleigh Castle, which is slowly collapsing as the spur of London Clay, on which it was built, retrogresses. *Figure 5.4* shows a section through the cliff, deduced by Hutchinson and Gostelow as part of their comprehensive investigation of the site (**Hutchinson and Gostelow**, 1976).

The abandoned cliff at Hadleigh was formed during the Devensian era (*Table 2.4*) and similarly to The Roughs, covered by marshland. Since formation, it has been subjected to four primary periods of instability.

- Late glacial/periglacial mudsliding. (Toe at -19 m OD, 10, 000 years BP)
- Early Atlantic temperate mudsliding. (Toe at -9 m OD, 270-6500 years BP)
- Early Sub-Atlantic temperate mudsliding. (Toe at 3 m OD, 2100-2000 years BP)
- Moderately deep-seated sliding at the crest of the slope. (Late nineteenth century)

The last set of movements was thought to be caused partly by human interference. In light of the movements on The Roughs, this may now be attributed to climatic changes.

5.4.2 Bedding control and perched landslides

Barton, 1977, seeks to demonstrate "*the extent to which a preferential control on the form of landsliding can be exercised by such surfaces irrespective of their level within the slope profile.*" He gives reasons why a particular bedding plane may be the preferred surface of shearing:

- internal erosion along a sand/clay junction
- tectonically produced bedding plane slip giving a residual strength condition
- stress relief by erosion giving a shear deformation in some cases augmented by excavation
- shear strength differences for shear along bedding planes compared with the adjacent soil
- discontinuity in the shear strength profile with a marked increase below the layer in which sliding is taking place
- mineralogical and chemical changes

An important example of the second option listed by **Barton**, above, is given by **Skempton**, 1966, in correspondence about the Vaiont disaster in which he refers to '*bedding plane slip*'. He surmises "*...that the combination of the two fundamental and simple phenomena may have led to an almost complete reduction in strength to the residual, along bedding planes, during tectonic folding* ."

Bromhead et al, 2000, describe how the control exerted by bedding on slip surface location is very clearly seen where the slip surfaces are perched in the slope. Perched systems of landslides are related to the occurrence of a weak bed or beds (usually clays) in the geological sequence and located above beach level. In these cases, the landslide morphology follows the geological structure, often depicting a stepped pattern. In the simplest cases, the sequence contains only one weak bed, and a single perched slide appears in the slopes. Even where the elevation of the weak bed is only a little above beach level and the sea cliff is therefore low, any debris which spills over the sea cliff from slide activity at a higher level is readily removed, thus keeping the sequence clean. Even if debris is not removed, then it takes the form of mudslides and screes (depending on the nature of materials present and their water content), which are easily recognised as different from the higher-level slide morphology.

Single perched slides may also occur in abandoned or defended cliffs, for example, the slopes at The Roughs, Hythe (**Bromhead *et al.* 1998**) or at Hadleigh Castle (**Hutchinson & Gostelow**, 1976). In such cases, the elevation of the basal slip surface may be obscured by the build-up of slide debris and vegetation. In some cases, the slopes underneath the rotational slide may be occupied by a *transport zone* where mudsliding occurs, predominantly parallel to the terrain slope, and an *accumulation zone*.

The shape of the slip surface in compound landslides is partly bedding plane controlled. **Barton,** 1984, discusses the typical shape of compound landslides, which can be split into three sections:

- the translation section which is parallel to the bedding
- the steep rear ward section which is most probably controlled by a stress relief joint
- a sharp radius of curvature which links the other two sections.

A diagram of a typical compound slide is shown in *Figure 5.5*. The elevation of the translational section is governed by the occurrence of the preferred bedding plane, its attitude being determined by the dip of that plane. He concludes that in areas of over-consolidated clay and soft rock with flat lying bedding, compound landslides are the norm and should be assumed.

5.4.3 Discontinuities

Discontinuities or imperfections in the ground structure, can be trigger points for the commencement of movement. They may indicate some weakness in the soil structure and allow water to entre the ground. **Skempton and Petley**, 1967, define the discontinuities found in stiff clays. This is reproduced in *Table 5.3*. They outline the five successive stages clay undergoes as it is subjected to simple shear:

i Continuous non-homogenous strain

ii. Formation of '*Riedel*' shears at or just before peak strain is attained.

iii. After further movement, Riedel shears are no longer kinematically possible and the clay develops displacement shears. '*Thrust shears*' develop.

iv. Displacement shears link to form a '*principal displacement shear*' or '*slip surface*'.'*Thrust shears*' develop.

v. With yet further movements, the slip surface undergoes appreciable flattening.

The clay particles undergo orientation which gives the shear surfaces their polished or slickensided appearance.

5.4.4 Shear surfaces

Hutchinson, 1987, states that *"landslides occurring on pre-existing shears generally exhibit limited, slow displacement on failure, which is consistent with the normally non-brittle nature of the associated slip surfaces."* He lists reasons why under some circumstances, reactivation would not be slow and limited:

- Water-induced mechanisms
- Changes in the surface profile of the slide
- Mechanisms involving brittleness within the slide mass
- Mechanisms involving modification of pre-existing shears
- Seismic effects
- Coalescence of landslides

The author approaches the problem from the opposite angle believing that the question should be why landslides on pre-existing shears are not all large and rapid. This is discussed in depth in *Chapter 7.*

5.5 Creep

Creep in soils can be defined as any movement which is imperceptible except by measurements over long periods of time. According to **Carson and Kirkby**, 1972, the three main causes of creep are:

- a systematic re-working of the surface soil layers due to variations in moisture and temperature
- random movements due to organisms or micro-seisms
- a steady application of a downhill shear stress

Continuous creep is an integral part of progressive failure but can occur at great depths for long periods without leading to failure. Creep effects may produce tension cracks, pressure ridges, and radial crack patterns.

Skempton and Hutchinson, 1969, however, discuss the distinction between only two main types of creep highlighted by Terzaghi:

- Mantle creep
- Mass creep

The former is highly seasonal and dependent on temperature and moisture content. Higher creep rates are due to the freeze-thaw cycle and should be regarded as peri-glacial solifluction movements. This form of creep can affect ground up to a depth of 1 metre. At greater depth, mass creep has more effect, being caused by the action of gravity alone. They note that pre-failure creep movements are indicated by accelerating and finally high rates of movement. The rates of movement which develop are dependent on the thickness of the clay in which the slip surface exists and on the type of clay involved.

Saito and Uezawa, 1961 have shown that there is an inverse relationship between strain rate and creep-rupture life and that this relationship can be used to forecast slope failure and landslides. By monitoring stakes driven into the slope, the authors were able to prove that the strain in the soil increased rapidly just before failure but were unable to predict the approach of failure from their field measurements.

Saito, 1965, continued his previous research by developing a method for forecasting the time of slope failure by measuring the surface strain of slopes. This was done in four stages:

i. Taking measurements of land displacement.
ii. Using the relative displacement curve to find the onset of instability.
iii. Using the relative displacement curve to determine the constant strain rate.
iv. Applying this constant strain rate to estimate the creep rupture life (using relationship developed for the previous publication).

5.6 Solifluction

Carson and Kirkby, 1972, describe solifluction as a rapid form of seasonal creep found in periglacial areas, produced by annual freeze-thaw cycles.

Waltham, 1994, describes solifluction as the downslope movement of saturated debris - a type of wet soil creep moving about 1 metre per year. It can occur on any saturated slope, but is most common in the summer-thawed active layer of periglacial slopes which cannot drain through the permafrost. Periglacial conditions in the Pleistocene caused many slope failures in Great Britain. Solifluction of the active layer was widespread on slopes >4° notably in clays, mudstone and chalk. The postglacial melt of permafrost permitted drainage and marginal stabilisation, leaving shear surfaces with residual strength of φ_r=0-15°. Many slides were reactivated by deforestation. Any slope >5° in clay, which was in the Pleistocene periglacial zone, is likely to have head debris prone to reactivation. The classic example of a reactivated periglacial landslide is that at Sevenoaks, Kent.

5.7 Residual strength

The relationship between shear strength and normal stress is given by the Coulomb- Terzaghi equation:

$$\tau = c' + \sigma' \tan \varphi'$$

where

τ = shear strength

c' = apparent cohesion

σ' = effective normal stress

φ' = angle of shearing resistance

The cohesion of clay is formed from inter-particle bonding or cementation. It is independent of compressive forces.

Figure 5.6 shows how the reduction in strength from peak to residual also reduces the internal angle of friction. The peak strength of clay reduces to residual strength due to the restructuring of the clay particles over time. All clays undergo this loss of strength but it is most significant in clays which have a low plasticity index. Progressive failure of clays occurs due to this drop in strength to the residual.

5.8 Progressive failure

Progressive failure is an important phenomenon in landslide theory and is discussed at some length in the relevant literature.

Skempton and Hutchinson, 1969 explain that the ratio of strength to shear stress is far from uniform along the length of a potential slip surface. Therefore the peak strength, in a first time slide, must be reached at some points before others. Moreover, unless the clay is an ideally plastic non-brittle material, the strength at these points must decrease as further movements take place, and when overall failure finally occurs the mobilised strength will be less than the slip surface. The more brittle the clay the greater the difference is likely to be.

Bishop, 1967, discusses the mechanism causing progressive failure. He lists the four main contributory factors to progressive failure:

i. The time lag in readjustment of pore pressures.

ii. Weathering effects on soil structure affecting both strength and permeability.

iii. The delayed release of strain energy due to a time lag in the rebound curve.

iv. The reduction in strength values, particularly at the peak, due to the rheological component of shear strength.

Potts et al, 1990 used finite element analysis to determine why the Carsington dam failed when, at the time of collapse, the factor of safety was 1.2. The average shear stress mobilised was less than peak strength, the discrepancy being accounted for by the nature of progressive failure. This was an ideal situation to test the suitability of the analysis to cases of progressive failure, because the embankment was well instrumented and its exact dimensions were known both before and after failure. The results of the analysis gave:

- The height of the dam after collapse.
- A reproduction of the progressive element of the failure surface.
- A reproduction of the observed collapse mechanism.
- A reasonable agreement with the data from the instrumentation.

It was concluded, therefore, that this finite element analysis was effective in predicting progressive failure.

5.9 Factor of safety

The factor of safety, F, is the ratio of the strength available to the strength mobilised. At the onset of movement, F is equal to one. For stable slopes, the strength available is greater than that mobilised and therefore F is greater than one. There are anomalies to this when progressive failure is involved, the factor of safety can be greater than one during failure, see *Section 5.8*

5.10 Stability Analysis

The stability analysis of a landslide may be by assessment of forces in two dimensions in individual slices of the mass: these vary across the slide and may include artificial constraints. Full landslide stability analysis is more complex due to:

- breaking the slide into small units
- reaction forces between these units
- variable water pressures
- estimated values of c and φ
- reaction in three dimensions

Amongst the best-known methods of stability analysis are Coulombe's 'wedge', Haefeli's infinite slope method (**Haefeli,** 1948), Skempton's 'φ_u=0' method (**Skempton,** 1948), Bishop's slip surface analysis (**Bishop,** 1954) and Morgenstern and Price's analysis of general slip surfaces (**Morgenstern and Price,** 1965, 1967). *Chapter 9* includes a stability analysis on The Roughs.

5.11 Summary

The terms used to describe The Roughs landslides have now been clearly defined. Previous chapters have set the morphology, structure and geology of the site and surrounding area and outlined its history. Enough background knowledge has been acquired to take the investigation to the next stage: the fieldwork. This enables theories about the landsliding to be proved or disproved by investigating the position of the strata, shear surfaces and piezometric levels within the slide mass. Information from the fieldwork will, in turn, allow stability analyses to be carried out on the site. It is hoped that these analyses will explain the nature of the 1988 reactivation of The Roughs.

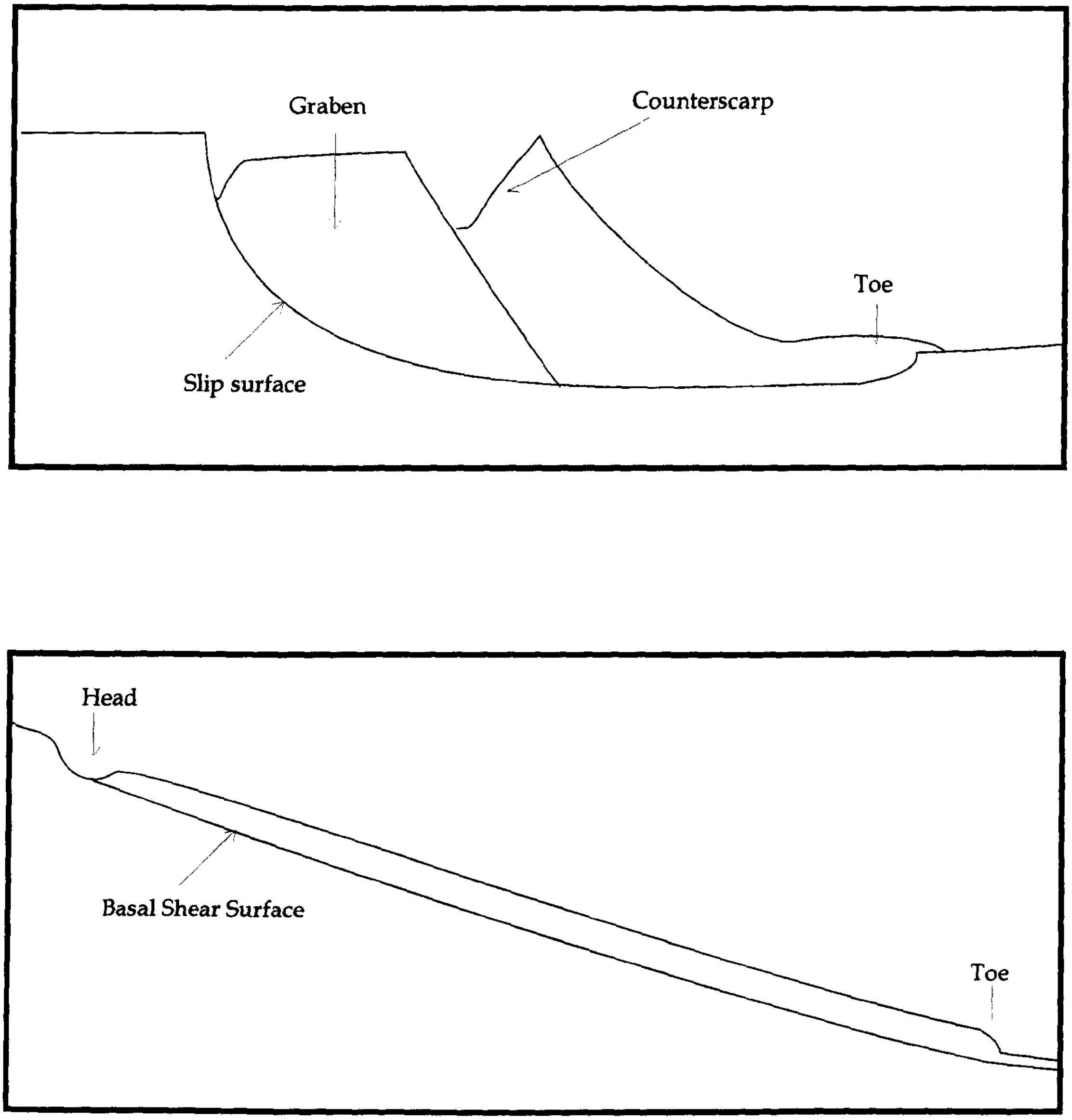


Figure 5.1. An example of a single non-circular rotational slide (upper) and a translational slide (lower).

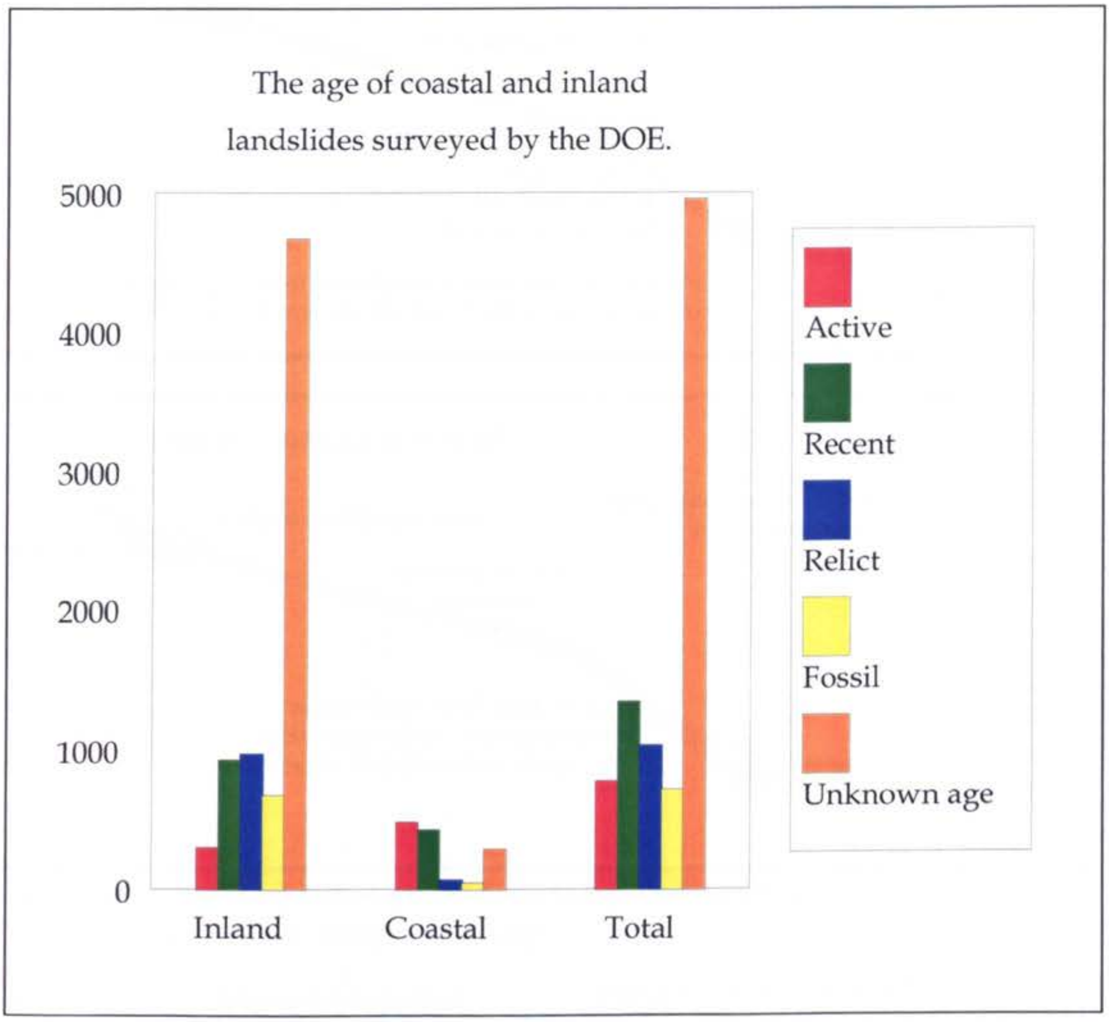


Figure 5.2. Bar chart showing the numbers of coastal and inland landslides in each age category specified by the DOE.

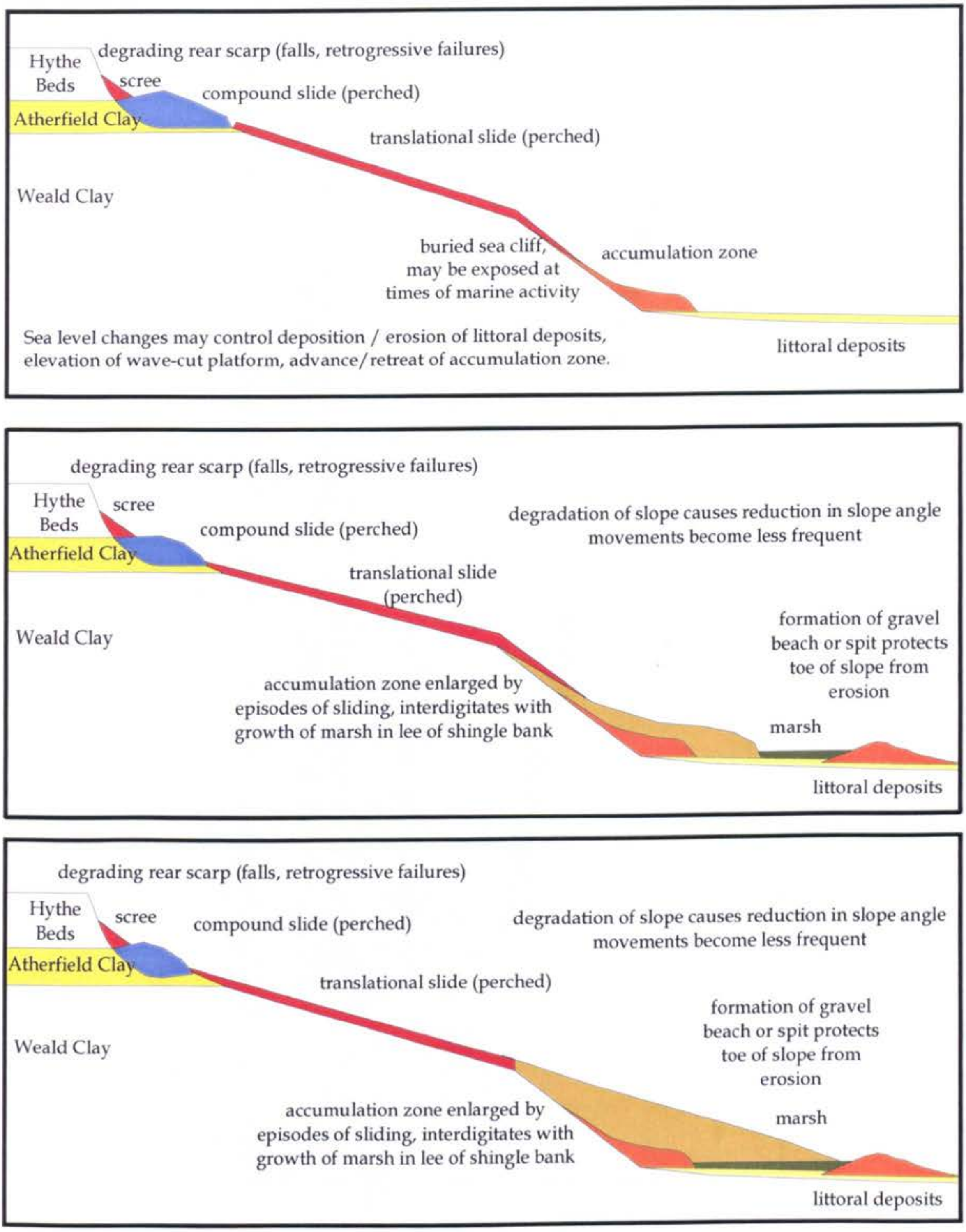


Figure 5.3. Formation of the abandoned cliff on the Lympne escarpment.

Figure 5.4. Evolution of the abandoned sea cliff at Hadleigh Castle. (After Hutchinson and Gostelow, 1976).

Figure 5.5. The main characteristics of compound landslides with flat-lying bedding. (After Barton, 1984).

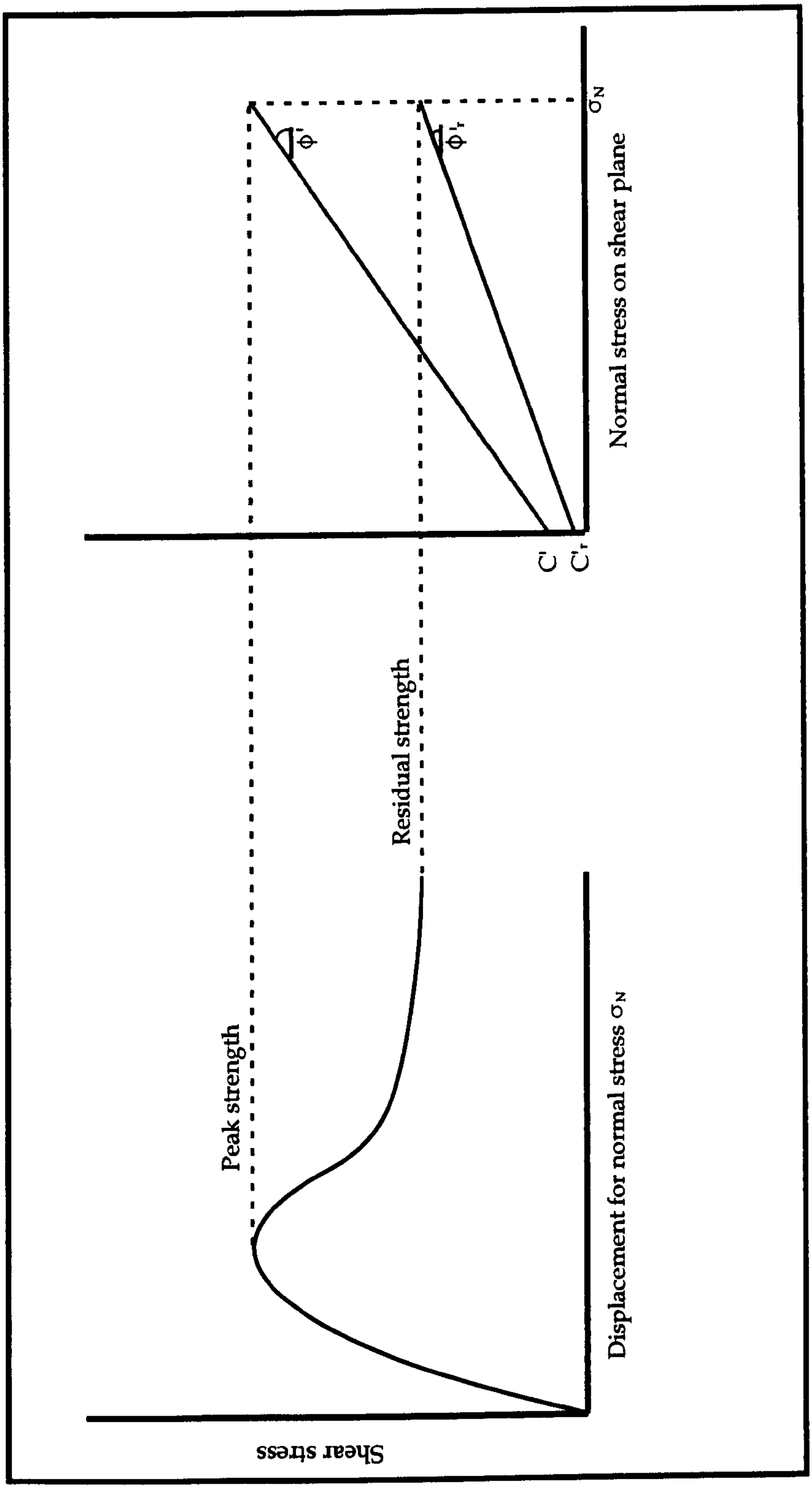


Figure 5.6. Graphs showing the relationship between shear strength and both stress and strain for an overconsolidated clay.

Table 5.1 A classification of landslides in terms of velocity. (After Cruden & Varnes, 1996).

Velocity Class	Description	Velocity (mm/sec)	Typical Velocity
7	Extremely Rapid		
		5×10^{3}	5 m/sec
6	Very Rapid		
		5×10^{1}	3 mm/min
5	Rapid		
		3×10^{-1}	1.8 m/hr
4	Moderate		
		5×10^{-3}	13 m/month
3	Slow		
		5×10^{-5}	1.6 m/year
2	Very Slow		
		5×10^{-7}	16 mm/year
1	Extremely Slow		

Table 5.2. A geotechnical classification of landslides, based on the shear strength mobilisation during failure. (After Jones and Lee, 1994).

Classification	**Geotechnical explanation**
First time slides	In preciously unsheared ground; soil fabric tends to be random. Shear strength parameters are at or close to **peak strength** or **residual strength.**
Reactivation of earlier landslides	Soil fabric along the shear surface is highly orientated in the slip direction. Shear strength parameters are close to the **residual value.**
Initiation of landsliding on discontinuous pre-existing shears	Produced by processes other than landsliding: • tectonics • glacitectonics • solifluction • rebound • non-uniform swelling

Table 5.3. Partial classification of discontinuities in stiff clays. (After Skempton and Petley, 1967)

Group	Type	Occurrence	Relative movement
Depositional or Diagenetic	BEDDING SURFACES	Bedding planes laminations Partings	Zero
Structural	JOINTS 'Brittle fracture'	Systematic joints 'Fissures'	Practically zero
	MINOR SHEARS Non-planar, slickensided	Small displacement shears and thrust shears	Less than 1 cm
	PRINCIPAL DISPLACEMENT SHEARS Subplanar, polished	Principal slip surfaces in : landslides, faults, bedding plane slips	More than 10 cm

Site investigation and laboratory testing

6.1 Introduction

However comprehensive the desk study for a project has been, there is no substitute for a detailed investigation in the field to confirm or refute formulated hypotheses. Evidence of landslide mechanisms on adjacent sites does not necessarily infer that the same holds true for the site under investigation. A hidden geological abnormality, such as a fault, or even human intervention, such as drainage, could drastically alter the entire system. Field investigations can be expensive, especially when a lot of drilling is involved. They are, however, necessary.

In order to determine the nature and mechanism of the landslides on The Roughs certain parameters had to be ascertained. It was important to find the position of any existing slip surfaces to identify the shape of the landslide ie rotational or translational. These were found from boreholes and trial pits. The water levels on The Roughs were fundamental to any stability analysis and were monitored in a series of boreholes. Soil properties were found by a series of tests on soil samples. All this data was necessary for the stability analyses described in *Chapter 9.*

Fieldwork on The Roughs was undertaken during three periods. These are referred to as field work "campaigns". Completed during the summer and the early autumn of 1994, the first campaign was carried out in conjunction with Soils Ltd. The second and third campaigns were undertaken in the autumn of 1995 and the spring of 1996 respectively, in conjunction with The Royal School of Military Engineers (RSME). Funding for the project has been received from:

1. the Institution of Civil Engineers' Research and Development Fund
2. the Department of the Environment (DoE), (matching the ICE R&D contribution)
3. the Polytechnic and Colleges Funding Council (PCFC)/Higher Education Funding Council for England (HEFCE).
4. internal funding from the School of Civil Engineering, Kingston University

Help in kind has been provided by Soils Ltd. and the RSME work has been carried out under the auspices of the joint Natural Environment Research Council (NERC)/Ministry of Defence scheme: Joining Forces for the Environment (JFE).

Boreholes have been drilled to determine the depths at which the slip surfaces exist, and their relationship with the undisturbed strata. They also allowed instrumentation to be installed. Trial pits were dug to give an overview of the site at a much shallower depth. Surveys located the relative positions of the boreholes and trial pits. Contour maps of the site have also been constructed with Royal Engineers surveyors as part of the Joining Forces for the Environment collaboration. The instruments have been monitored at regular intervals and piezometric levels and ground movements have been recorded. Samples recovered from the site have undergone a series of tests, either in a mobile laboratory on site or in the laboratory at Kingston University.

Table 6.1 contains a summary of the boreholes and trialpits investigated during all three fieldwork campaigns and *Figure 6.1* shows their approximate locations on site.

6.2 First Campaign

During the first fieldwork campaign it was decided to concentrate the investigation on a single north-south cross-section, which is shown as Section AA on the oblique aerial view of the site, *Figure 3.10* and on *Figure 6.1*. The position of the cross section was chosen due to its location, approximately half way along the slip between the two mudslides, which are clearly described in *Chapter 3*. A total of ten boreholes was drilled on this section, which are shown on *Figure 6.2*, the geological section AA through the slope. Boreholes were drilled using Soils Limited's man-portable flow-through sampling equipment, a percussive system which takes a core in a window sampler. The percussive action is provided by a generator-driven hydraulic system and hydraulic hammer.

A series of seven flow-through boreholes was drilled along this cross-section, in the degradation and accumulation zones and in the marsh. Henceforth, these boreholes are identified as Boreholes 1-7 or BHs. 1-7. They were drilled to depths ranging from 2.7 m to 6.0 m and yielded disturbed samples which were logged on site. All of the boreholes were instrumented with piezometric tubing and secured with padlocked metal tops. The piezometric levels in these boreholes have been monitored on a regular basis since. *Figures 6.1 and 6.2* show the positions of these boreholes and sample descriptions are given in *Appendix 6.1*.

The boreholes revealed mostly landslide colluvium with some in situ material only at depth. This was to be expected since the boreholes were drilled on the degradation and accumulation zones of the slope. Borehole 6, which was drilled in the marsh, yielded inter-digitated layers of landslide colluvium and beach alluvium. It is believed that this was formed as the toe of the slide moved over the beach, as the slope was destabilized by the eroding action of the sea and was covered by water as the sea level rose and fell, and is analogous to that found at Lympne (**Hutchinson et al., 1985**).

Three Shell and Auger holes were also drilled using a Pilcon percussive rig to depths of 10 m to

16.5 m. Henceforth, these boreholes will be identified as SA 1-3. *Figure 6.1* shows the positions of these boreholes. The first borehole, SA1, was drilled on one of the lower plateaus, in the accumulation zone and the other two, SA2 and SA3, were drilled at the top of the slope, in an attempt to find the slip surface of the rotational slide. These boreholes were continuously sampled using U100 tubes with liners. The ends of the tubes were logged, coated in wax and sent back to the laboratory for analysis. Piezometric tubing was then installed into the Shell and Auger boreholes. The piezometric levels in the boreholes have been monitored on a regular basis. *Appendix 6.2* contains the borehole logs from these three boreholes .

A series of six trial pits was dug using a Hy-mac tracked backhole excavator. The positions of these trial pits are shown in *Figure 6.1*. Each pit was approximately one metre wide, three metres long and three metres deep. For safety, the sides of the trial pits were supported by hydraulic props. One side of each pit was carefully logged. This was done by marking out a base line with string and measuring from this any points of interest such as changes in strata or moisture content, the position of shells, rocks and fossils and the line of any slip surfaces discovered. Slip surfaces were found by "picking" at the surface of the clay with a trowel, until a lump slid freely out along a slip surface. The surface could then be followed along the side of the trial pit. Slip surfaces in clay have a shiny, polished appearance due to the alignment of the clay platelets; this polished appearance is termed "slickensided" An example of this "slickensided" surface is shown in *Plate 6.1*. The rear scarp was also logged in a similar manor. Diagrams depicting the trial pit and rear scarp face are given in *Figures 6.3-6.9*. Photographs of the trial pits are shown in *Plates 6.2-6.3* and a photograph of the rear scarp is shown in *Plate 6.4*.

6.3 Second campaign

A second cross section, BB, was chosen for investigation during the second fieldwork campaign. This was located between the first cross section, AA, and the western mudslide, to obtain geological and engineering information along the width of the site. The second cross section is shown on *Figure 3.10* and *Figure 6.1*. Data obtained from this investigation was used to construct a geological section, BB, which is shown in *Figure 6.10*.

Drilling was carried out by the RSME as part of the Joining Forces for the Environment scheme. The RSME agreed to use The Roughs because:

- of its location in Southern England, not excessively remote from their Chatham barracks, with convenient Army accommodation nearby,
- it is in Army ownership

- of the challenges which it would pose to their trainee drillers.

Three pairs of holes, P01A-B, P02A-B and P03A-B, were drilled to a depth of approximately 12 m using a Pilcon cable percussion rig. This Pilcon cable percussion rig is shown in *Plate 6.5*. The positions of these boreholes are shown in *Figure 6.1* and in the diagram of Section BB *Figure 6.10.* At the first site, the boreholes were sunk at a distance of 10 m apart. Problems occurred due to a layer of limestone (slipped and displaced Hythe Beds) and a combination of California and Flat chisels and a Clay cutter had to be employed to cut through it. This disrupted the continuous sampling as well as being extremely time consuming. On the second and third sites, the pairs of boreholes were drilled at a distance of 2 m apart. The problem with the limestone was also encountered during the drilling of these holes. One of each pair, P02A-B and P03A-B were instrumented with piezometer tubing and the other with inclinometer tubing. Solid gas barrels were concreted in over these boreholes and all the previously drilled holes to protect them from vandals. Borehole logs are given in *Appendix 6.2.*

Besides the cable tool rig, two other drilling rigs were used on The Roughs. A *Minuteman* rig was used to take samples every 50 m along the access track to the west of the site. This type of rig is a small bore continuous flight auger rig. As well as part of the research, this was an exercise to train the Royal Engineers to redesign the road. Large areas of sand were found towards the top of the track but these may be remains from the construction of the listening station, which is described in *Section 3.6.2.* Figure 6.1 shows the position of the Minuteman boreholes. The Minuteman borehole logs are given in *Appendix 6.3.*

Three trial pits were dug by hand along the track, in areas thought to be not naturally occurring. Bulk samples were taken from these pits. The positions of these pits are given in *Figure 6.1.*

A *Skidster HDE 105*, a mobile one man drilling rig, was tested on The Roughs. It can produce both continuous and disturbed samples. Three boreholes were sunk in the marsh to depths of 2-85-2.9 m. Unfortunately, the Skidster was unable to penetrate a layer of beach alluvium occurring at that depth. The Skidster was also tested on sites along Section BB, shown in *Figure 6.1* which shows the positions of all the Skidster boreholes. *Appendix 6.4* contains the Skidster borehole *logs.*

The RSME also provided and manned a mobile laboratory. This allowed the U100 tubes to be extruded and logged almost immediately, index tests to be carried out and moisture contents to be determined. The results of these tests are shown in *Table 6.4.*

6.4 Third campaign

The lower half of the second cross section, BB, was targeted during the third fieldwork campaign. Continuous U100 samples were obtained by the shell and auger technique. Borehole depths ranged from 5.5- 16 m. Four boreholes were completed: P05-8. Two of the boreholes, P07 and P08, were instrumented with piezometers and one, P05, with an inclinometer. *Figure 6.10*, geological section BB through the site, shows the positions of these boreholes. The borehole logs from these boreholes are included in *Appendix 6.2*.

Five trialpits were opened using a Hydrema 806 Medium Wheeled tractor. Bulk samples were taken for testing. The positions of these trialpits are shown in *Figure 6.1*.

A selection of samples was analysed in the mobile laboratory and others transported to the laboratories in Kingston University and the results are given in *Table 6.5*.

6.5 Instrumentation

The instrumentation on site consists of inclinometers and piezometers. The ability to monitor the latter single-handedly enabled more frequent readings to be taken than for the inclinometers, which require assistance to manhandle the readout equipment. Readings from the piezometers are also more likely to show variations over shorter periods, due to the response of the piezometric levels to rainfall, whereas the inclinometer readings can go for extended periods with minimal or no change.

Difficulties were encountered with the monitoring. Padlocks became rusted and difficult to open and the screw caps to the gas barrels became wedged shut. Some of the instrumentation was destroyed by vandals.

6.5.1 Piezometer readings

The piezometric readings from the flow through boreholes (1-7) and the Shell and Auger boreholes in Section AA and Section BB were measured and plotted (*Figures 6.11, 6.12 and 6.13)*.

Figure 6.11 shows the readings from the flow-through boreholes. Borehole 1 is situated in the translational slide. The water levels were at a peak in this borehole during February, 1995. Borehole 2 is at the foot of the small scarp at the top of the translational zone. It is too shallow to be able to take readings and the water level only raised high enough to do so for a short period during February, 1995. The levels in Borehole 3, situated above Borehole 1, ascended in a similar fashion to those in Boreholes 1, 5 and 7 before the piezometer was removed by persons unknown. Borehole

4 is situated on the second terrace up from the marsh and is too shallow to take readings. The only data collected was during the wet spring in 1995. However, Shell and Auger Borehole 1 is situated on the same terrace and is deep enough to take readings. Borehole 5 is situated on the first terrace above the marsh. Water levels in this hole also rose in the wet spring of 1995. Borehole 6 was sunk in the marsh itself. The ground water levels gradually rise and stabilize at 0.9-1.0 m. Here, the strata dip to the south and the water can drain away. Slight fluctuations can be seen with the rainfall levels. Borehole 7 is also within the translational zone and its readings behave similarity to those from Boreholes 1, 3 and 5. Missing readings are due to the borehole covers becoming stuck.

Figure 6.12 shows data from the three Shell and Auger Boreholes on Section AA. Situated on the second terrace, the first borehole shows little variation, the shape of the graph being similar to Boreholes 1, 3, 5 and 7. The second borehole is situated close to the rear scarp, next to standing water. This suggests a perched water table, hence the high levels of water. Below the second, the third borehole is in the rotational slip zone. The level has settled at about 9 m, a probable slip surface location.

Figure 6.13 shows readings from Section B. P07 and P08 are located on the first terrace and on the marsh itself and mimic the findings from Section AA. P01 is at a level with Shell and Auger Borehole 3 and P02 on the terrace below. Both water levels have settled at 6.5-7.0 m below ground level.

6.5.2 Inclinometer readings

Initial readings from the inclinometer tubes were recorded after the tubes had been given time to settle down. These were used as the baseline against which later readings could be compared.

After heavy rains in August, 1996, the inclinometer casings were found to have moved. The inclinometer in P02 had *kinked* at 10 m, in the area of rotational sliding, and that in P05 was found to have *kinked* at 4 m, in the translational slides.

6.6 Surveying

A closed traverse in the shape of a rough pentagon was marked on the site using permanent stations. Two stations were situated on the marsh, one on the scarp behind the western mudslide, one on the rear scarp and the final one on a smaller scarp to the east of the first cross section. Two further stations were positioned inside this pentagon. The survey was checked after some time and one of the permanent stations on what was thought to have been stable ground, was found to have moved (the eastern most station) further than the two stations on unstable ground. A set of co-ordinates were assigned to the stations.

The positions of the boreholes and trial pits on the first cross section were surveyed from the permanent station on the rear cliff overlooking the mudslide. This afforded the best view of the cross section. Another of the stations has been removed by persons unknown.

In the summer of 1995, the Royal Engineers conducted a larger scale survey involving more than 900 points. This was done by siting a total station on the scarp above the western mudslide, which afforded a view of most of the site. Two reflecting poles were used to allow one to move as the position of the other was being recorded. The positions of boreholes, trial pits, tension cracks, scarps and changes in slope were taken and the data downloaded into a computer to give a contour map of the site using the programme, *Secure Data Recording* This also allows the cross section along any given line to be plotted. The survey was repeated in the summer of 1996.

6.7 Laboratory Work

Much of the routine sample testing was carried out on site by the RSME to the direction of the Author. This consisted of moisture content determinations, particle size determinations, index tests and basic field sample logging (detailed logging was done by the Author where the sample condition was good enough to merit it). All of the samples from the Minuteman drilling and the Skidster drilling as well as P01A-B and P02A were analysed in the mobile laboratory. *Tables 6.2 and 6.3* summarise the identification codes given to the samples tested by the RSME, during the second and third campaigns, and provide the location of the borehole or trial pit from which each sample was taken and a brief description of that sample. *Table 6.4* shows the results of the testing. The remaining samples were analysed at Kingston University and these results are shown in *Table 6.5*. There, in addition to the aforementioned tests, ring shear tests were also carried out to determine the residual strength of the samples, the results of which are shown in *Figure 6.16*. Much of the testing at Kingston University was carried out by laboratory technician, Malcolm Ince.

6.7.1 Logging

The U100 samples from the Shell and Auger boreholes were extruded, split, photographed and logged in the laboratory. The samples were logged by drawing a 1:1 picture of the split surface and giving an engineering soil description of all the layers present, noting the presence of slip surfaces, rocks and shells.

6.7.2 Moisture Content

The moisture contents of the samples give an indication of the disturbance of the soil, together with information on the strata through which each borehole passes. Moisture contents of the samples from each borehole were plotted against depth *(Figures 6.14 and 6.15)* which can be used to confirm the visual identification of probable positions of slip surfaces. *Figure 6.14* shows the variation of moisture content with depth for the Pilcon boreholes in Section AA. Shell and Auger Borehole 1, situated on the second terrace above the marsh, shows an increase in moisture content from 6 m downwards. The material with the lower moisture content is the looser, more drained landslide colluvium whereas the material with the higher moisture content is the in situ clay. The results from Shell and Auger Borehole 2, which is in the rotational slip zone, show a decrease in moisture content from 7-8 m downwards. This is the contact between the Hythe Beds and the Atherfield Clay. The moisture content in Shell and Auger Borehole 3 shows a peak about 10 m. This is a likely position for a rotational slip surface.

Figure 6.15 shows variation of moisture content with depth for the Pilcon holes in Section BB. Borehole P02A shows a peak of moisture content at about 10 m. This is a probable location of a slip surface. Unsurprisingly, since the two boreholes were drilled 2 m apart, Borehole P02B also shows this peak at approximately 10 m. It can be seen that the correlation between the two graphs is very good. Borehole P02B is on a similar level to Shell and Auger Borehole 3 in Section 1. It is interesting that the results from both suggest similar locations of slip surfaces. The readings become lower with depth where the more silty bands of Weald Clay are found. Boreholes P03A and P03B are on the same terrace and would be expected to show similarities. P03A dries out near to the surface as would be predicted, samples at that level were not recovered from P03B. Both vary between moisture contents of 17-23%. Borehole P05 also dries out towards the surface. The low moisture content at depth represents the silty Weald Clay. Samples were not recovered at the surface of Borehole P06. At 3 m, the low moisture content represents a very sandy area and the peak below 12 m represents a band of very soft clay. Borehole P07 also dries out towards the surface. The peak below 2 m is a probable location of translational sliding surface.

A summary of the sample moisture contents determined by the RSME, during both fieldwork campaigns, is given in *Table 6.4*. Those determined at Kingston University are given in *Table 6.5*.

6.7.3 Ring Shear Tests

Ring shear tests were carried out on the samples in accordance with a simplified procedure (**Bromhead, 1992; Harris & Watson, 1997**) as the full BS 1377: Part 2: 1990 procedure gives information of little relevance to this project. Ring shear tests are carried out to determine the residual strength (Skempton, 1964, 1985) of the soil sample, this is relevant since land slipped material is usually at residual strength. The sample is packed into a thin ring shaped mould and topped with a rough platen which is subjected to a torque. It is then sheared under a series of normal loads. In situ land is likely to be at peak strength whereas landslides are likely to be at residual strength, hence the relevance of the test.

Twenty-four samples were tested and one hundred and sixty-one measurements taken. The results of the tests are shown in *Figure 6.16* and can be split neatly into four groups. Firstly those with a $\varphi'_r = 30°$, these are the least plastic of all the samples and represent the Hythe Beds. The second group are more plastic and have $\varphi'_r = 15°$. These are samples of the more silty Weald Clay. The less silty Weald Clay and the Atherfield Clay have $\varphi'_r = 10°$. Finally, the most plastic sample has $\varphi'_r = 5°$ is the slip surface in the Atherfield Clay.

6.7.4 Particle Size Distribution

Particle size distribution determinations were carried out on the samples in accordance with BS 1377: Part 2: 1990. The results of the particle size distribution tests had corrections (BS 1377) applied to them using a spreadsheet and were plotted on a graph. These show the percentage of sands and silts in each sample and will confirm identification of samples. A summary of the particle size distributions determined by the RSME is given in *Table 6.4*. Those determined at Kingston University are given in *Table 6.5*.

These results were plotted on a bar chart, *Figure 6.17*. This shows a predominance of fines, from the clays and silty clays, closely followed by sands, from the Hythe Beds and sandy layers in the clays. The gravels have a small percentage of the overall fabric of the samples and are derived from the landslipped Hythe Beds.

6.7.5 Index Tests

Index tests were carried out on the samples in accordance with BS 1377: Part 2: 1990. These give the liquid and plastic limits of the samples and hence their Plasticity Indices. The index results determined by the RSME during both fieldwork campaigns are summarised in *Table 6.4*. Those determined at Kingston University are given in *Table 6.5*. All the results from the site have been plotted on a Casagrande Plasticity Chart, shown in *Figure 6.18*, to give an indication of the plasticity of the soils on the site and hence their susceptibility to develop low residual strengths. The majority of samples fall in the two categories of clays of intermediate to high plasticity. This is to be expected when sampling landslipped clays.

6.8 Summary

With the help of the RSME conducting a series of very comprehensive fieldwork campaigns was possible. The information collected can be summarised as follows:

- logs from seven flow-through, thirteen shell and auger, twenty-seven Minuteman and eight Skidster boreholes
- logs and/or samples from twenty-three trial pits
- two detailed site surveys
- piezometer readings
- inclinometer readings
- two geological sections through the site
- a large amount of data from tests on soil samples in the laboratory

A summary of information from the boreholes logs in given in *Table 6.7*. At times, interpreting the borehole logs was very difficult, due to the nature of the landslide system. Much of the material has moved in large, intact slabs, and it is not always possible to tell what is in situ and what is not. Detecting the position of the top of the Weald Clay when the logging sequence is incomplete is especially difficult. Although much Atherfield Clay was found, nearly all of it was colluvium.

Section BB is the more accurate of the two since the shallower boreholes in Section AA make it difficult to determine the position of the in situ Weald which is deeper than this section suggests. Section BB shows the abandoned cliff in the Weald Clay extremely well. The piezometric level follows the top of the Weald until the position of the abandoned cliff. Inclinometer readings are used in section BB to position the slip surface. In Section AA, the location of the slip surface is presumed to be where the moisture content is high.

The geotechnical information derived from the field study confirmed the predictions derived from the study of the geology of the site and other local landslides.

- The geological units found are Weald Clay, Atherfield Clay and Hythe Beds.
- The rotational landslide is "perched" in the Atherfield Clay at the head of the slope.
- The degradation/accumulation zone reaches from the toe of the rotational slide to the marsh at the toe of the slope.

Most of this information is used in the stability analyses in *Chapter 9*. Before these are attempted, giving some thought to the mechanics of slow-moving landslides is necessary. This is discussed in *Chapter 7*. The other question to be asked is what triggered the reactivation of The Roughs landslide, after being inactive for such a long period. All the evidence points to water and in particular a period of very wet weather. This wet weather is a feature of climate change and is considered in *Chapter 8*.

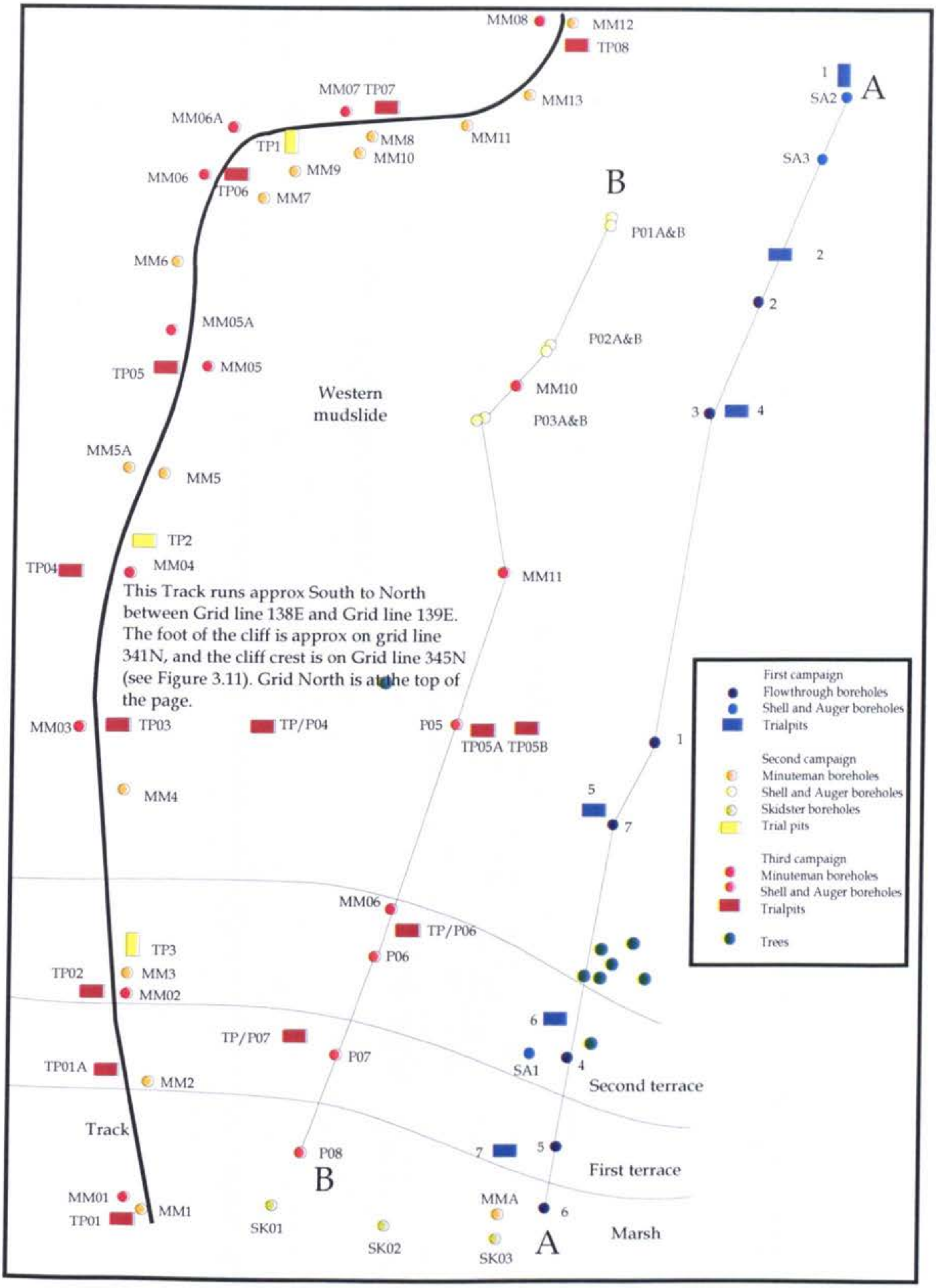


Figure 6.1. Sketch map of The Roughs showing the position of boreholes and trialpits.

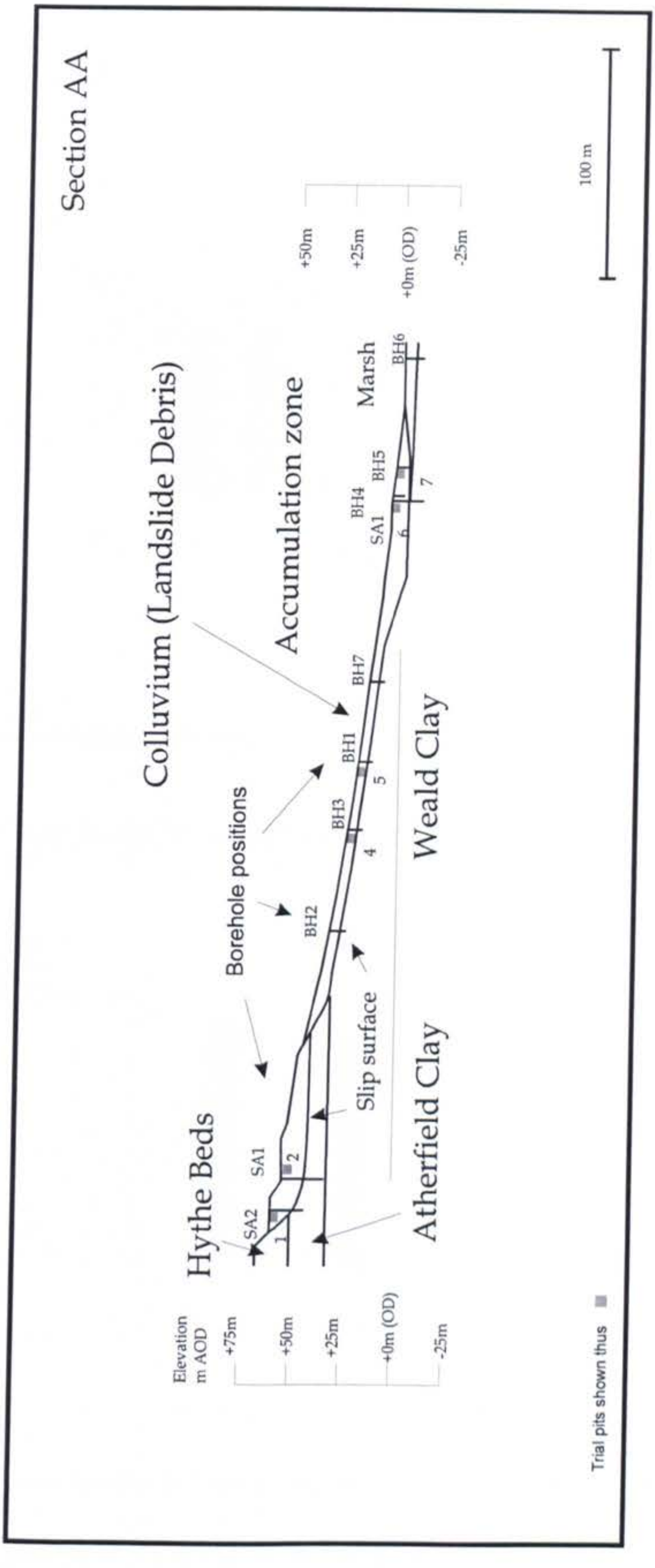
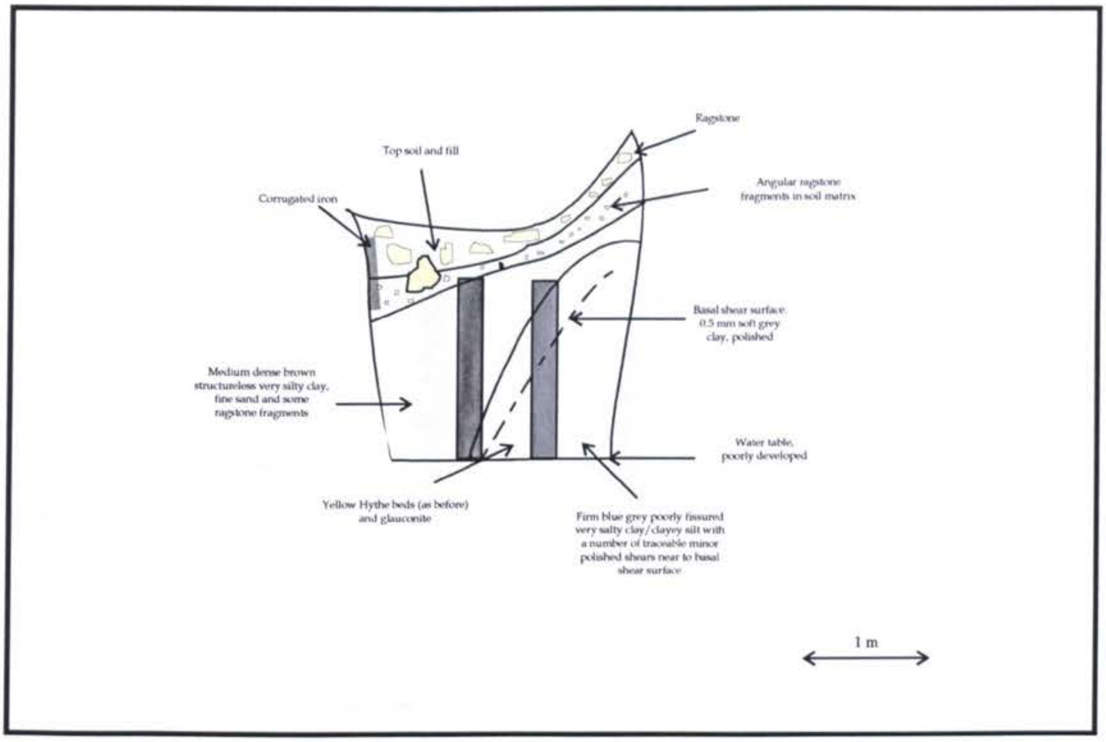


Figure 6.2. Section AA through The Roughs.



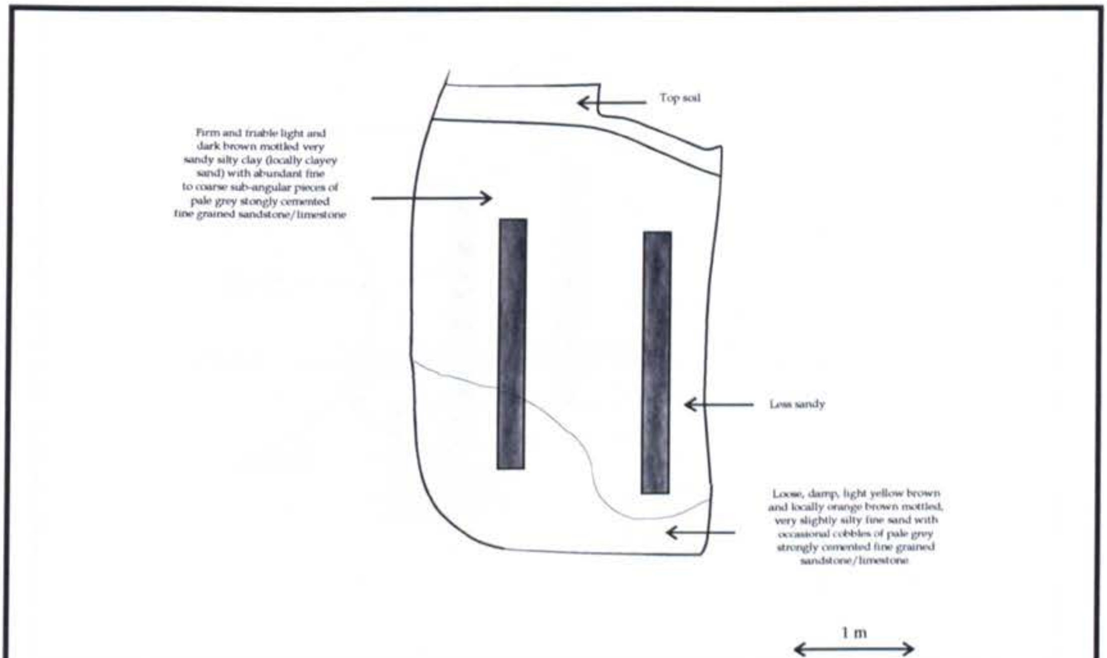
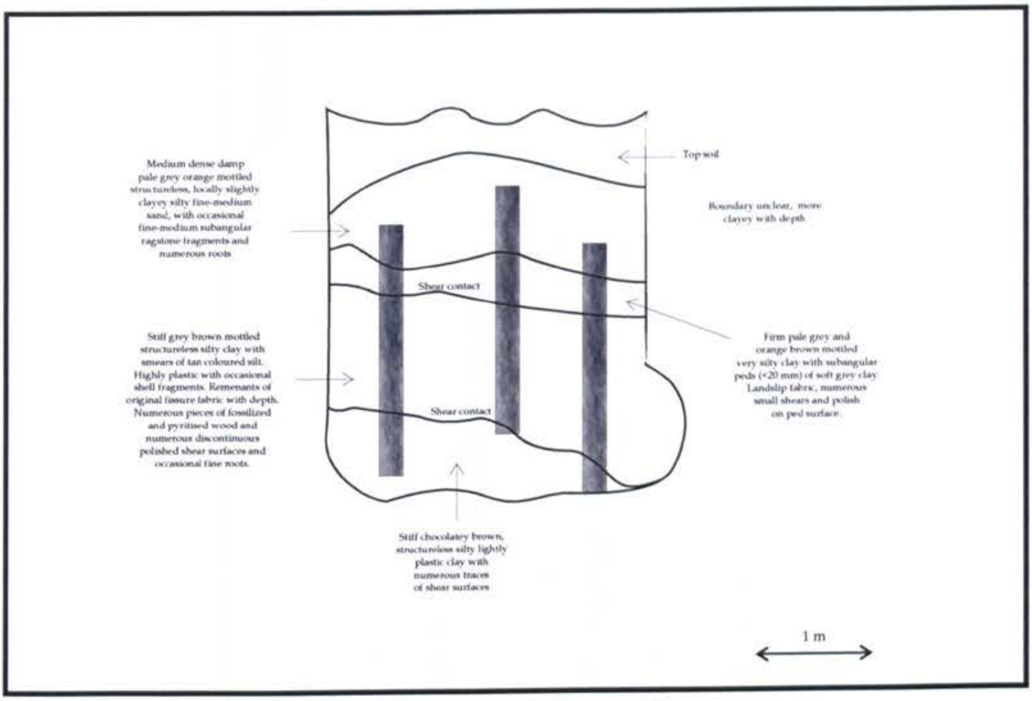


Figure 6.3 (above) and Figure 6.4 (below). Diagrams depicting western face of Trialpit 1 and eastern face of Trialpit 2, respectively.



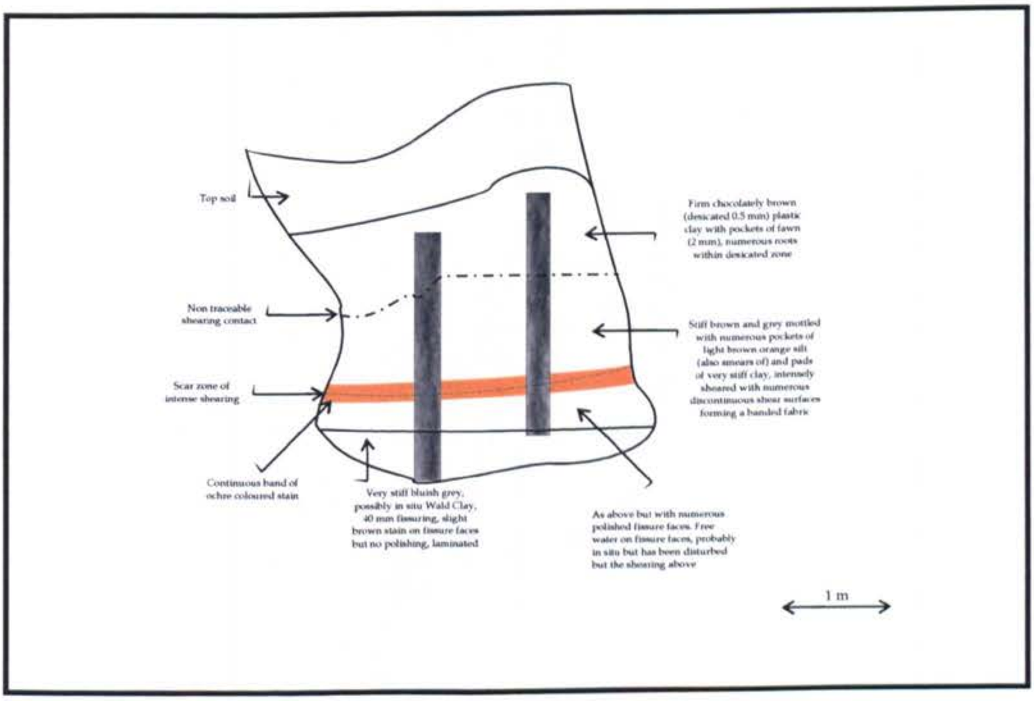


Figure 6.5 (above) and Figure 6.6 (below). Diagrams depicting the western face of Trialpit 4 and the western face of Trialpit 5, respectively.

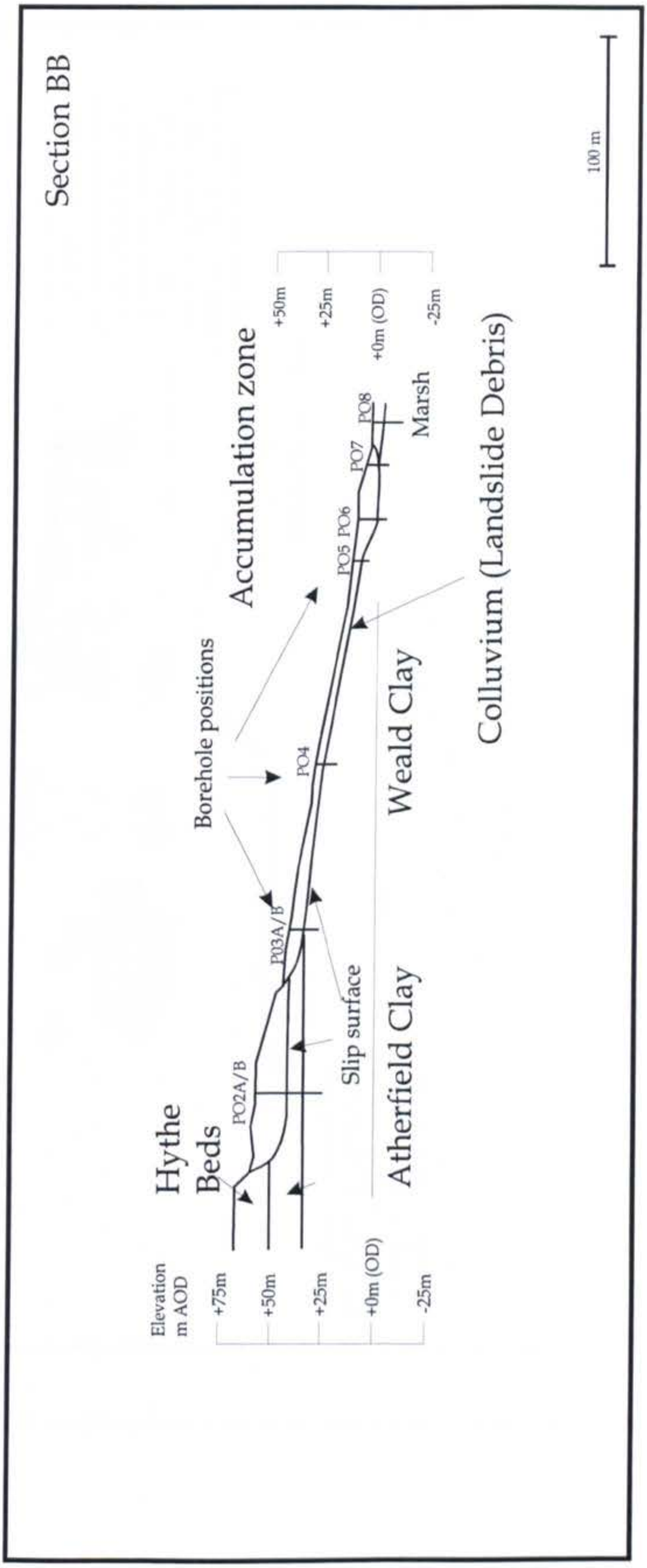


Figure 6.7 (above) and Figure 6.8 (below). Diagrams depicting eastern face of Trialpit 6 and western face of Trialpit 7.

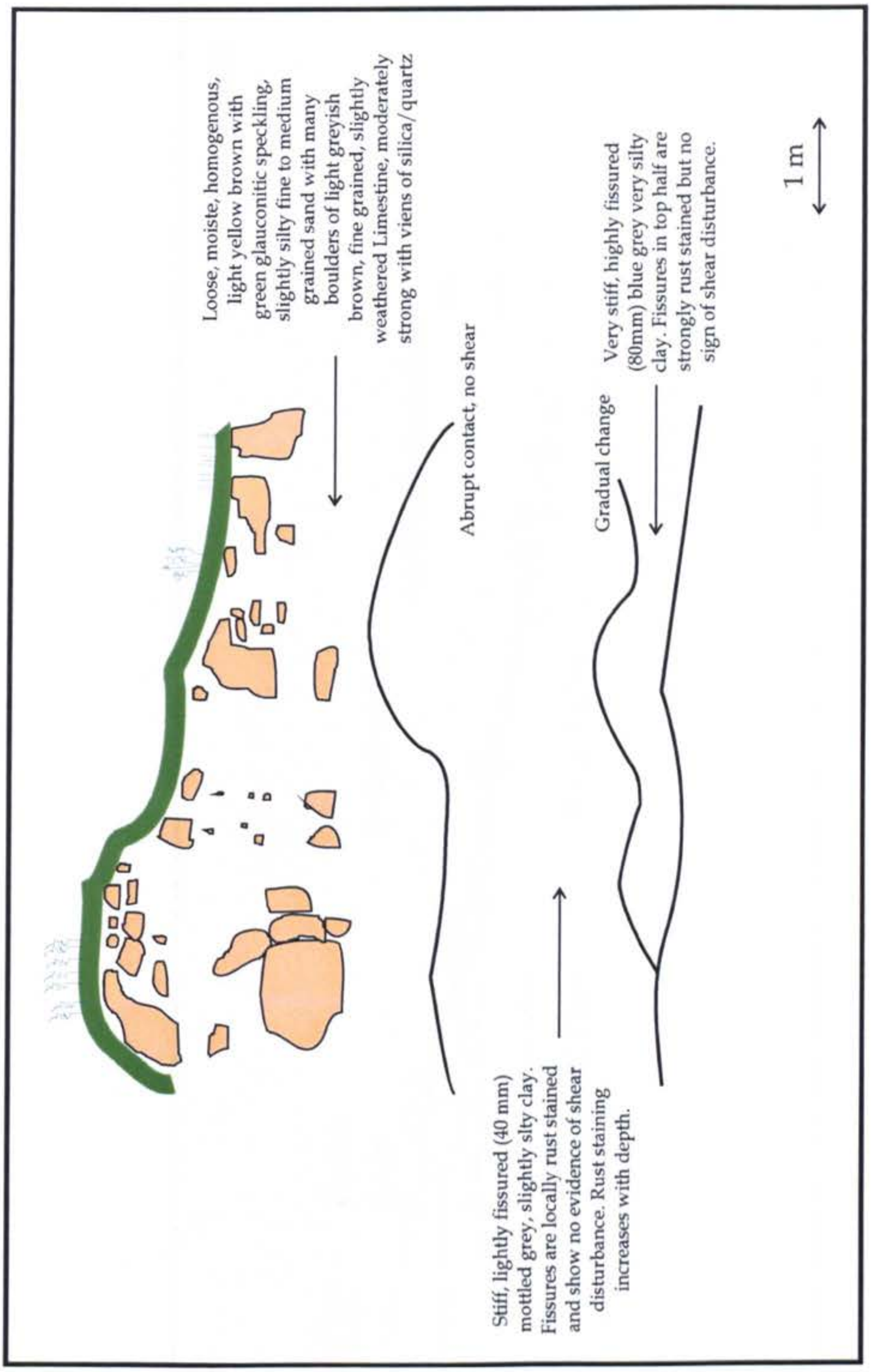


Figure 6.9. Diagram depicting the cleared face of scarp east of western mudslide.

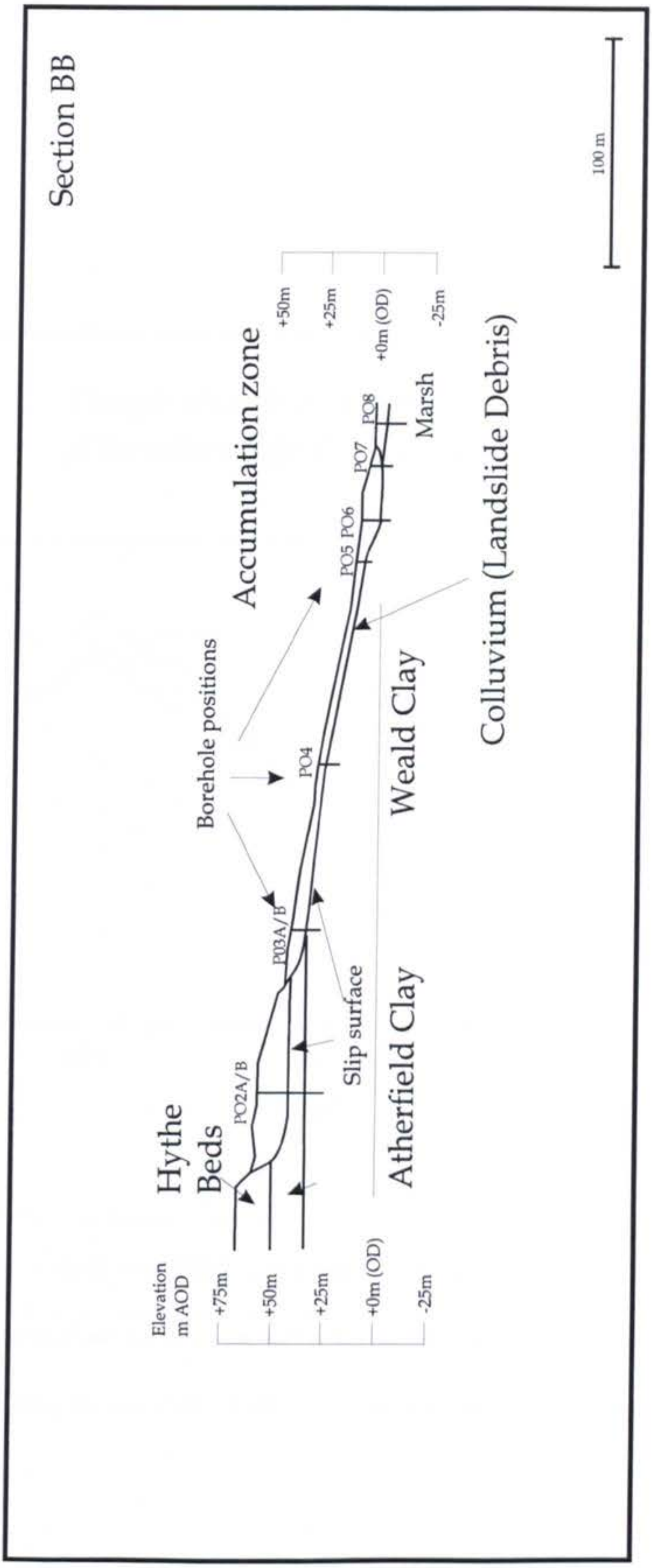


Figure 6.10. Section BB through The Roughs

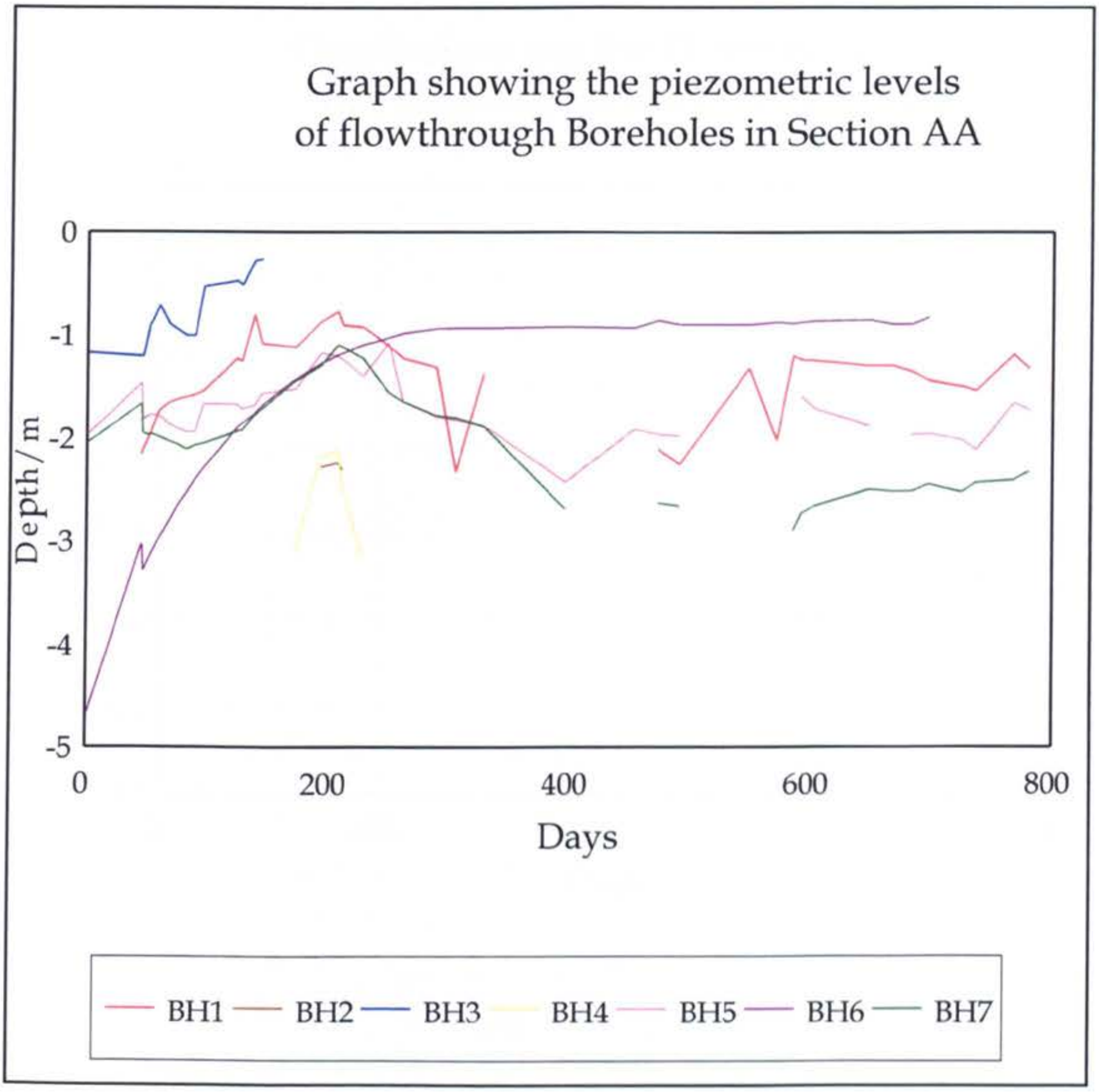


Figure 6.11. Graph showing the piezometric levels of flowthrough boreholes in Section AA.

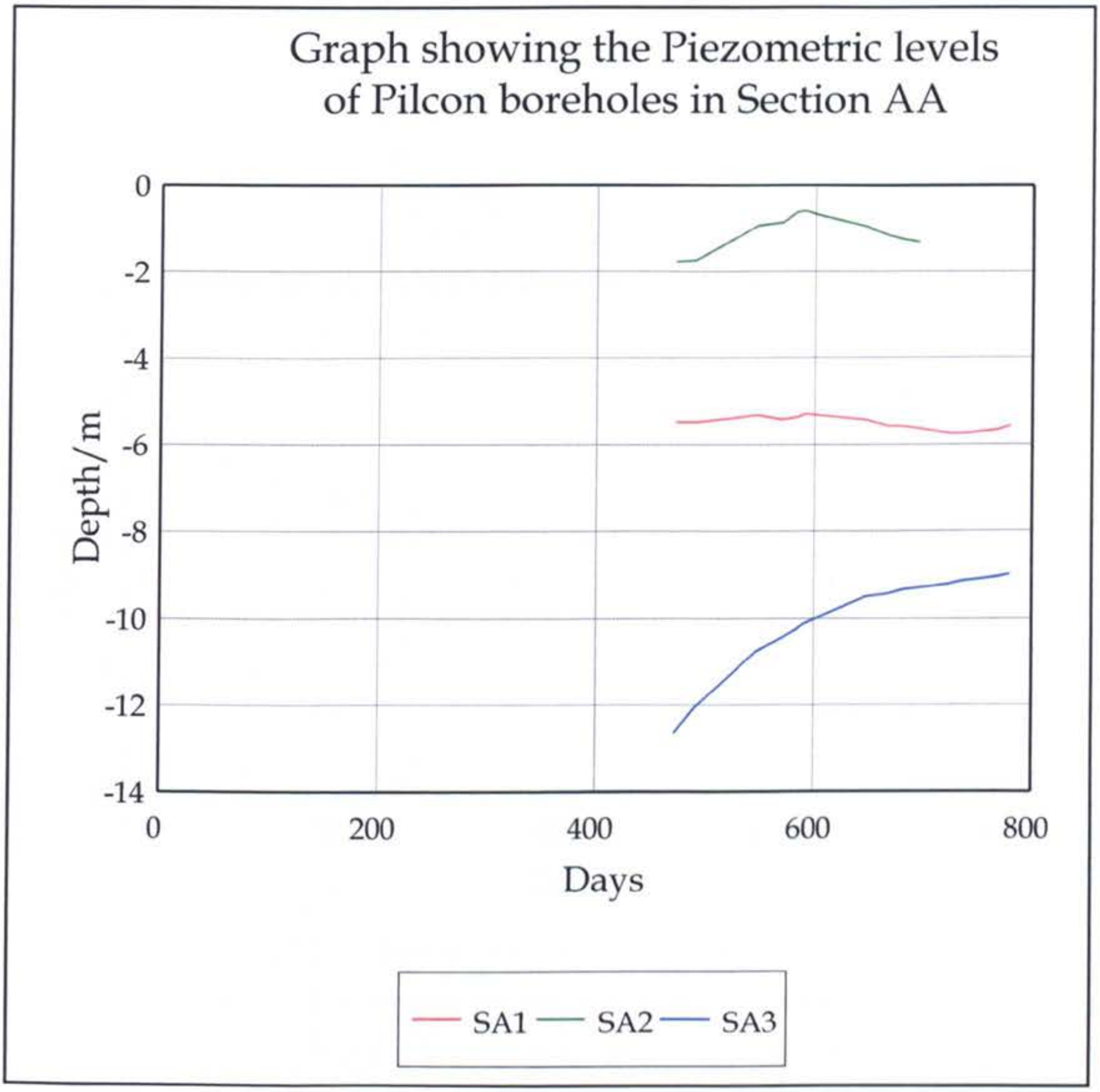


Figure 6.12.Graph showing the piezometric levels of pilcon boreholes in Section AA.

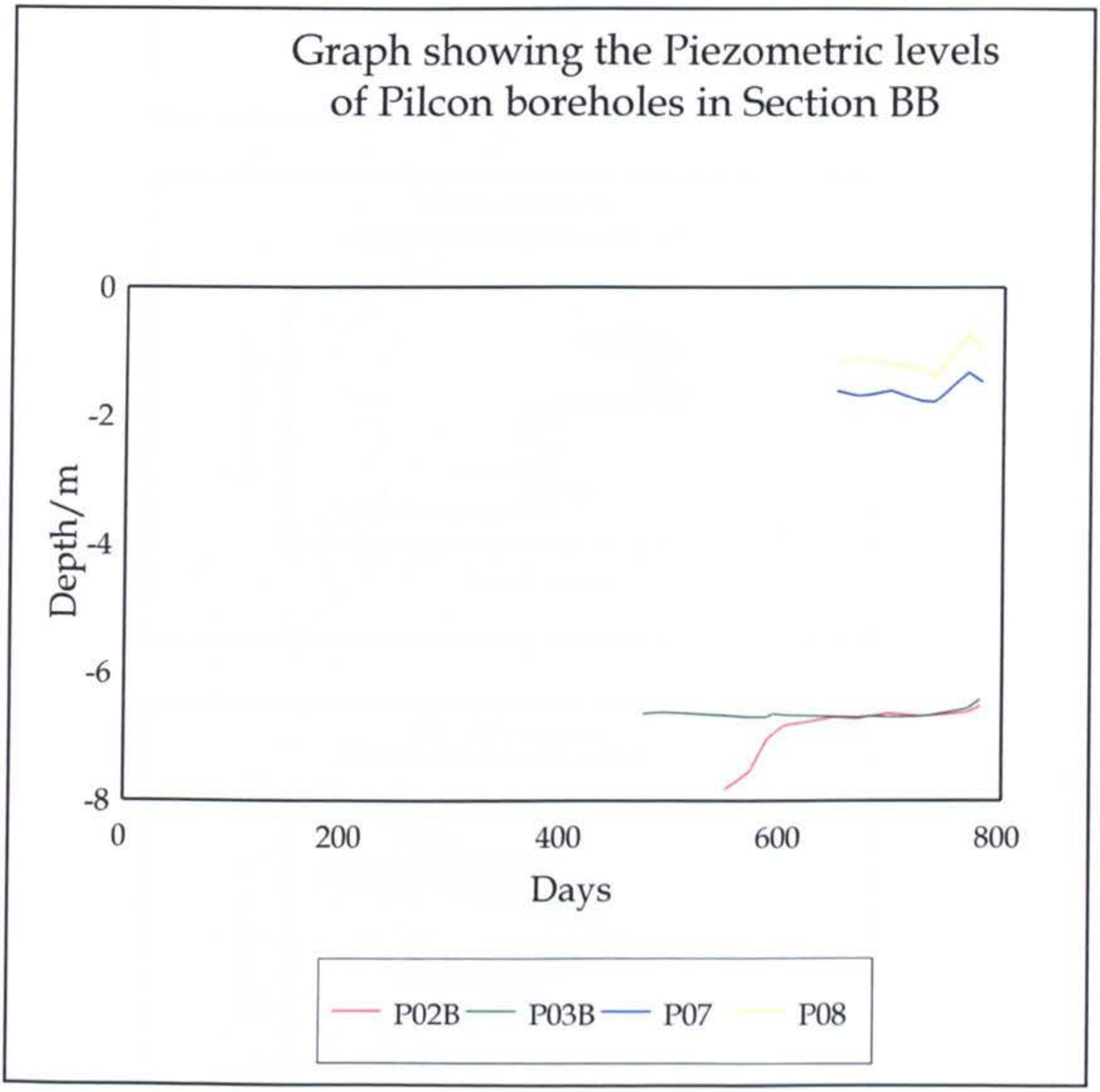
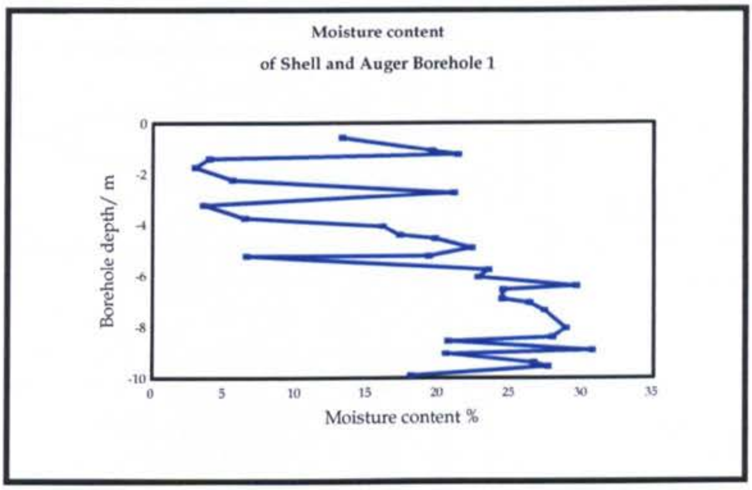
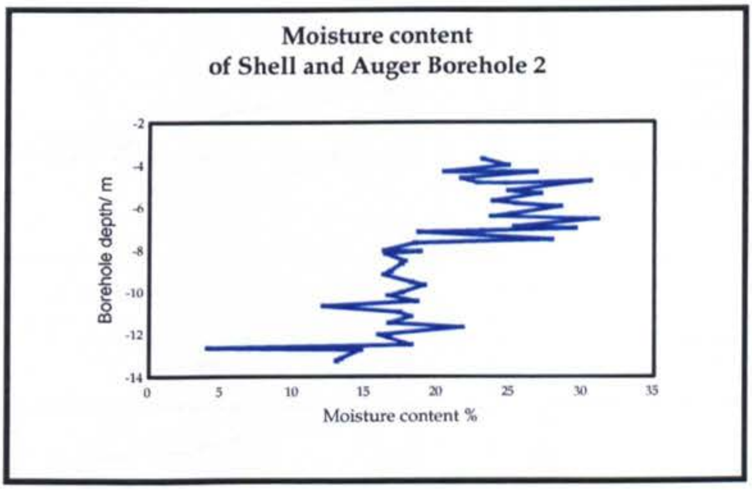


Figure 6.13.Graph showing the piezometric levels of pilcon boreholes in Section BB.





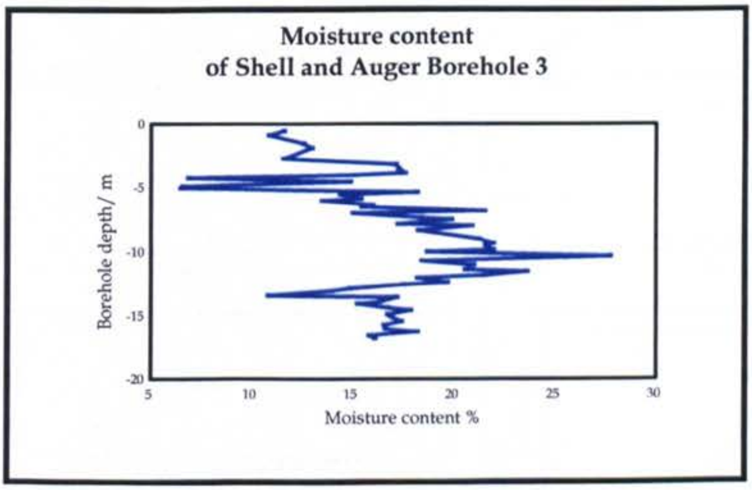


Figure 6.14. Graphs showing the variation of moisture content with depth for the pilcon boreholes in Section AA.

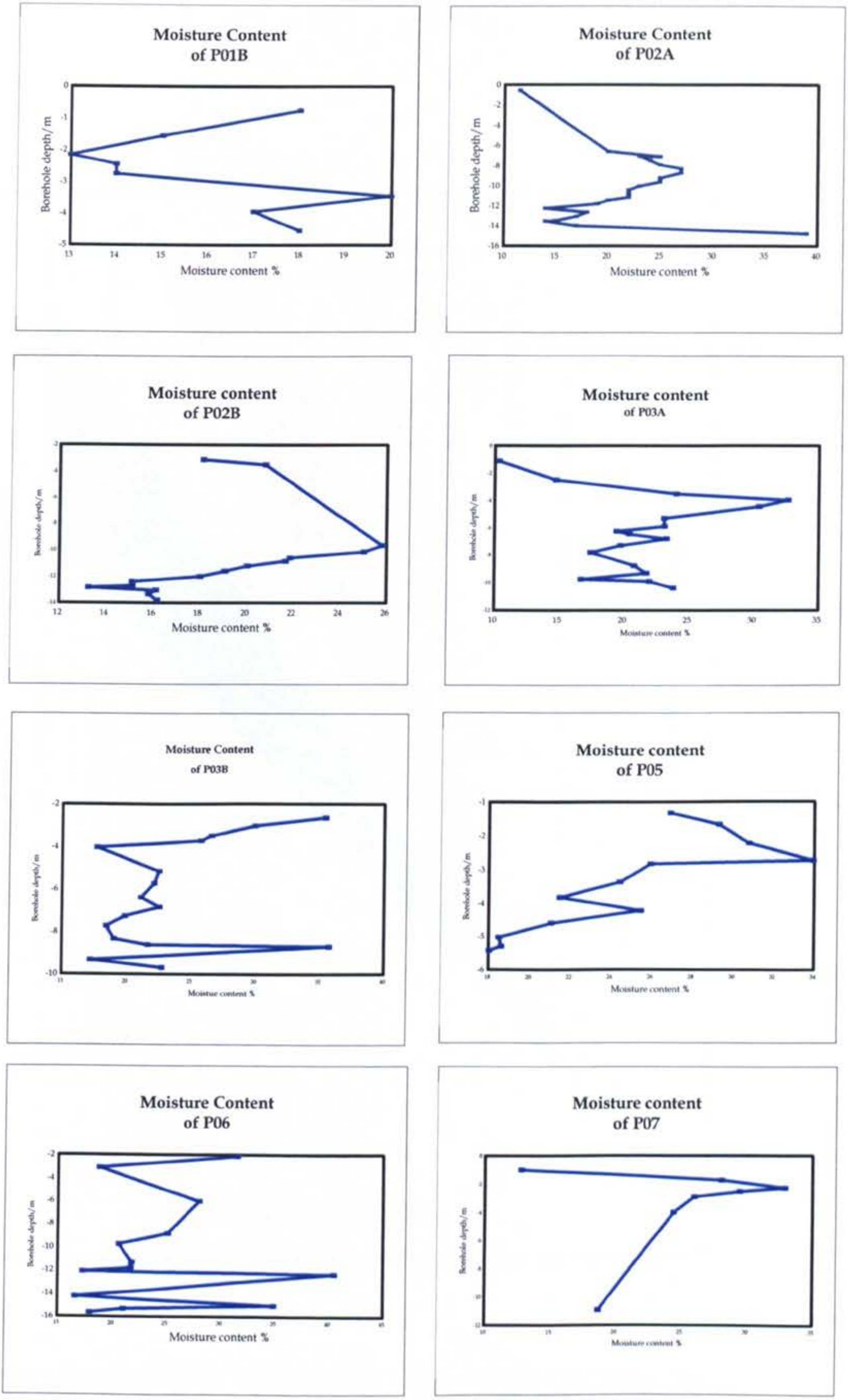


Figure 6.15. Graphs showing the variation of moisture content with depth for the pilcon boreholes in Section BB.

Figure 6.16. Results of the Ring Shear tests.(After Bromhead, Hopper and Ibsen, 1998)

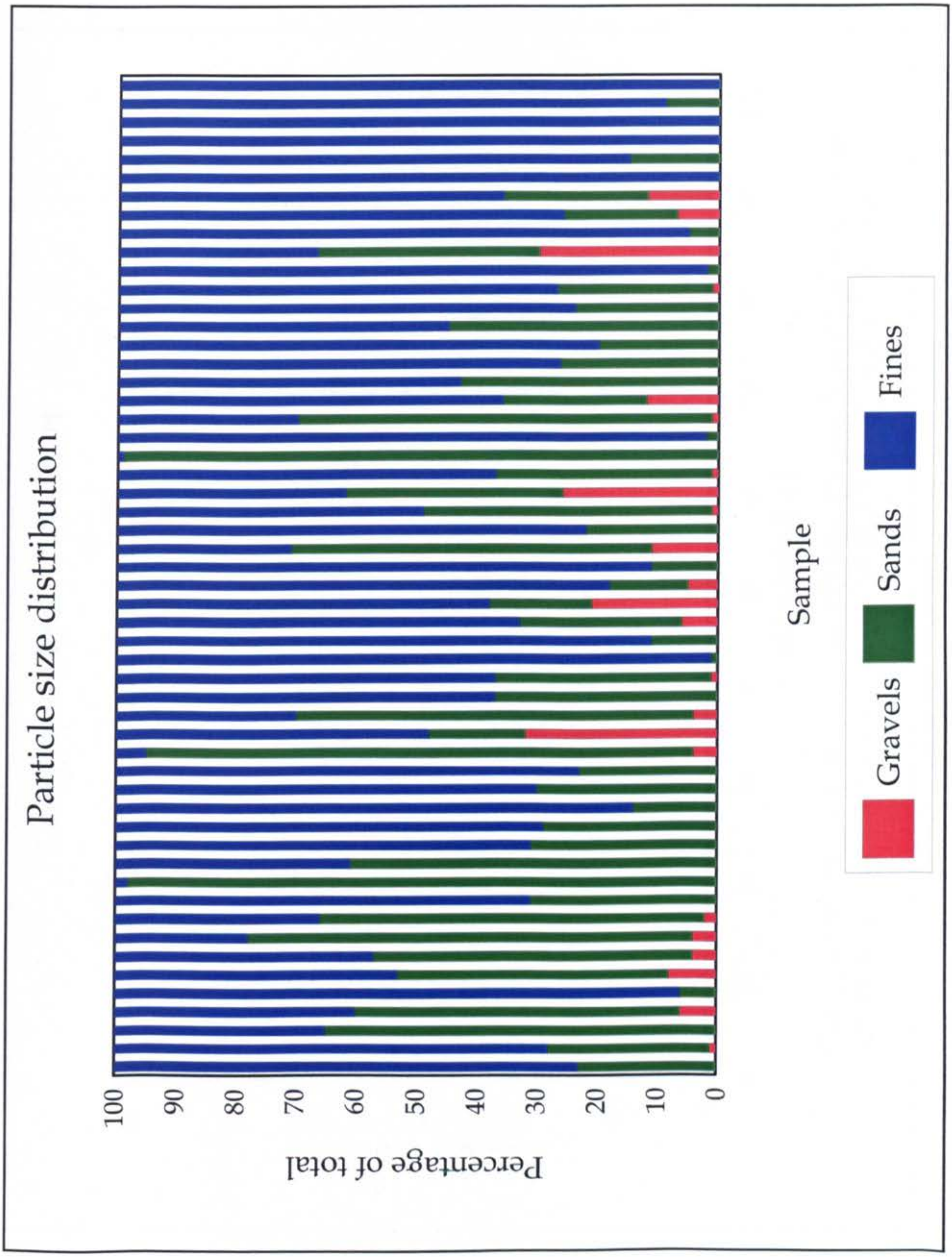


Figure 6.17. Bar chart showing the particle size distribution in samples from The Roughs.

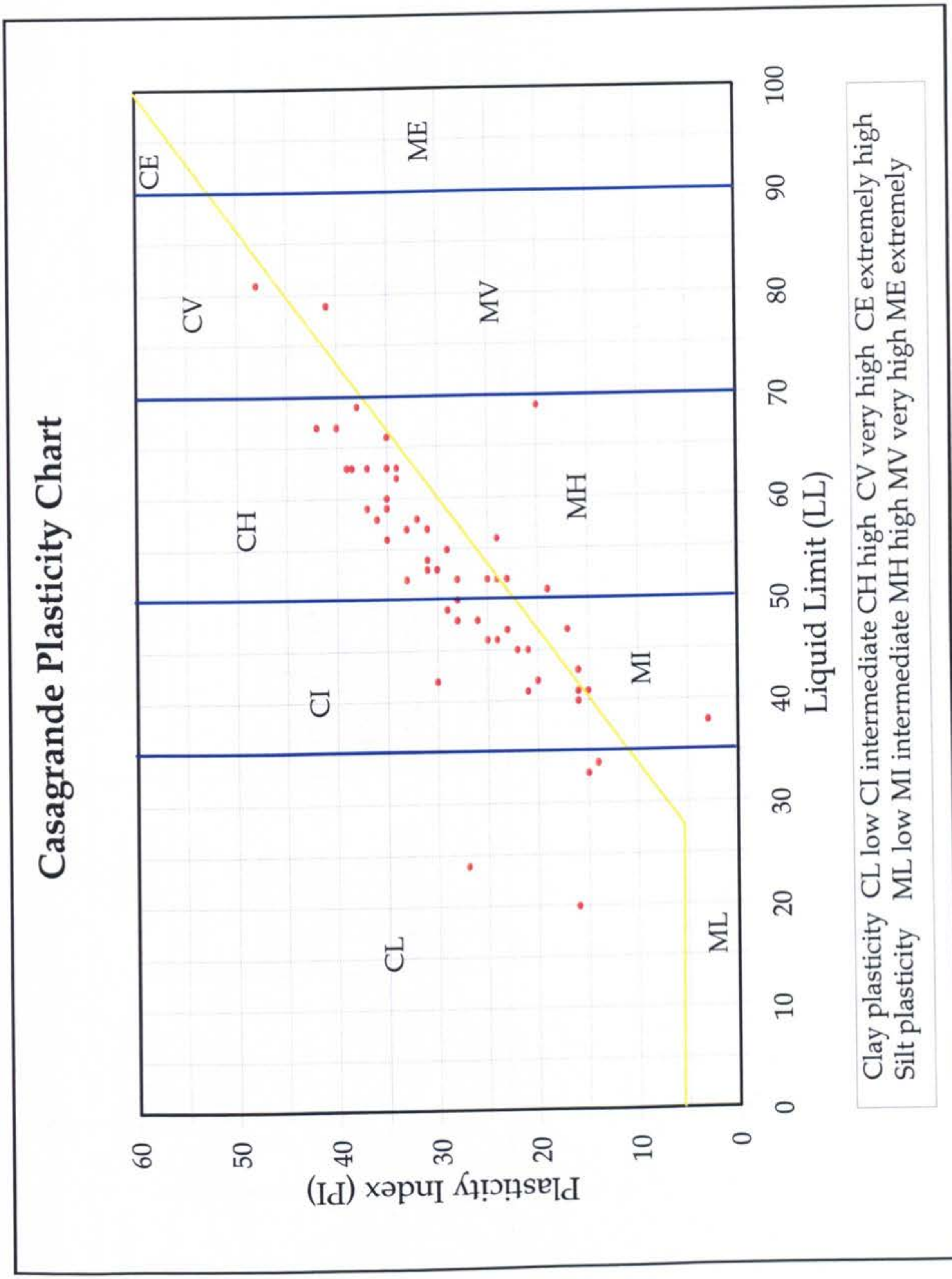


Figure 6.18. Cassagrande plasticity chart.

Table 6.1. Summary of the boreholes and trialpits investigated during the three fieldwork campaigns .

Campaign	Flow-through boreholes	Pilcon boreholes	Minuteman boreholes	Skidster boreholes	Trialpits
First	BH1, BH2, BH3, BH4, BH5, BH6, BH7	SA1, SA2, SA3			TP1, TP2, TP4, TP5 TP6 TP7
Second		PO1A, P01B, PO2A, PO2B, PO3A, PO3B	MM1, MM2, MM3, MM4, MM5, MM5A, MM6, MM7, MM8, MM9, MM10, MM11, MM12, MM13, MMA	SK01, SK02, SK03, SK001, SK002, SK003, SK004, SK005	TP1, TP2, TP3
Third		PO5, PO6, PO7, PO8	MM01, MM02, MM03, MM04, MM05, MM05A MM06, MM06A, MM07, MM08, MM10, MM11		TP01, TP01A, TP02, TP03, TP04, TP05, TP06, TP07, TP08, TP/P04, TP/P05A, TP/P05B, TP/P06, TP/P07

Table 6.2. Royal School of Military Engineering sample register for second campaign.

Sample	Borehole	Depth/m	Location	Description
QB 001	SK01/03	2.5-2.8	Marsh	Very soft light grey homogeneous clay
QB 002	SK02/01	0.5-0.8	Marsh	Firm light brown clay
QB 003	SK02/02	0.0-1.35	Marsh	Firm light brown homogeneous clay
QB 004	SK01/01	0.6-1.35	Marsh	Very soft light grey homogeneous clay
QB 005	SK01/02	1.35-2.10	Marsh	Very soft light grey homogeneous clay
QB 006	SK01/04	2.10-2.85	Marsh	Very soft light grey homogeneous clay
QB 007	P01A/00	0.2-0.5	Top terrace	Light grey sand
QB 008	SK03/02	1.35-2.1	Marsh	Stiff light brown homogeneous clay
QB 009	SK03/01	1.1-1.3	Marsh	Stiff light brown homogeneous clay
QB 010	SK03/03	2.1-2.85	Marsh	Stiff light brown homogeneous clay
QB 011	MM001	1.0-1.38	Track 0+00	Bluish brown gravelly clay
QB 012	MM002	3.5-4.0	Track 0+75	Very soft greyish clay
QB 013	P01A/01	0.7-0.85	Top terrace	Very soft greyish clay
QB 014	MM2/002	3.0-3.38	Track 0+75	Firm light grey clay
QB 015	MM2/001	2.0-2.38	Track 0+75	Firm yellowish brown slightly sandy clay
QB 016	MM2/003	4.4-6.0	Track 0+75	Firm light greenish grey clay
QB 017	MM3/001	1.0-1.38	Track 1 + 25	Firm light brown gravelly clay
QB 018	SK004/01	0.55-1.3	Marsh	
QB 019	P01B/06	2.4-2.5	Top terrace	Dark brown clayey sand
QB 020	P01B/04	0.3-1.3	Top terrace	Loose brown clayey sand
QB 021	P01B/02	0.0-1.3	Top terrace	Dark brown clayey sand
QB 022	P01B/05	1.9-2.4	Top terrace	Dark brown clayey sand
QB 023	MM3/004	4.0-4.35	Track 1 + 25	Firm light brown homogenous clay
QB 024	MM3/003	3.0-3.38	Track 1 + 25	Firm light brown homogenous clay
QB 025	MM3/002	2.0-2.38	Track 1 + 25	Firm light brown homogenous clay
QB 026	P01B/03	1.3-1.8	Top terrace	Dark brown clayey sand
QB 027	SK001/01	1.35-2.10	1st Pilcon site	
QB 028	SK001/02	2.1-2.85	1st Pilcon site	
QB 029	P01B/07	2.5-3.0	Top terrace	Dark brown clayey sand
QB 030	P01B/08	3.2-3.7	Top terrace	Dark brown clayey sand
QB 031	P01B/09	3.7-4.2	Top terrace	Orangish brown clayey sand
QB 032	P01B/10	4.3-4.8	Top terrace	Orangish brown clayey sand
QB 035	MM3/006	6.0-7.0	Track 1 + 25	Firm light greenish grey clay of upper plasticity
QB 036	MM3/05	5.0-6.0	Track 1 + 25	Firm light brown clay of upper plasticity

Sample	Borehole	Depth/m	Location	Description
QB 037	P01B/01	0.3-0.8	Top terrace	Loose dark brown sand containing some gravel
QB 038	MM3/007	7.0-8.0	Track 1 + 25	Firm mottled green/grey clay
QB 039	MM3/008	8.0-9.0	Track 1 + 25	Firm greenish grey clay of upper plasticity
QB 040	SK002/001	0.3-0.8	2nd Pilcon site	Loose whitish brown heterogeneous silty sand
QB 041	P02A/02	0.2-0.9	2nd terrace from top	Firm light brown clayey sand
QB 042	P02A/01	0.2-0.7	2nd terrace from top	Loose light brown heterogenous clayey sand
QB 043	SK002/02	0.8-1.55	2nd Pilcon site	Loose whitish brown heterogeneous silty sand
QB 044	SK001/04	0.6-1.35	1st Pilcon site	
QB 045	SK001/03	2.85-3.6	1st Pilcon site	
QB 046	MM4/005	4.5-5.6	Track 1 + 75	Firm light brown clay of upper plasticity
QB 047	MM4/001	1.0-1.38	Track 1 + 75	Stiff light brown clay
QB 048	MM4/002	2.0-2.38	Track 1 + 75	Firm light brown clay
QB 049	MM4/004	4.0-4.38	Track 1 + 75	Firm light brown clay
QB 050	MM4/003	3.0-3.38	Track 1 + 75	Firm light brown clay
QB 051	MM5/002	1.0-1.38	Track 2 + 25	Soft light brown homogenous clay
QB 052	MM5/003	2.0-2.38	Track 2 + 25	Soft light brown homogenous clay
QB 053	MM5/008	5.8-6.3	Track 2 + 25	Soft light brown homogeneous clay
QB 054	MM5/004	3.7-5.4	Track 2 + 25	Firm light greyish brown sandy clay
QB 055	MM5/001	0.8-2.7	Track 2 + 25	Soft light brown homogeneous clay
QB 056	SK003/02	2.3-2.8	2nd slip	
QB 057	SK003/03	2.8-3.5	2nd slip	
QB 058	SK003/04	3.5-4.25	2nd slip	
QB 059	SK004/01	0.1-0.7	Marsh	Loose light brownish grey homogeneous very silty sand
QB 060	P02A/05	6.9-7.2	2nd terrace from top	Firm bluish grey homogenous clay
QB 061	P02A/03	6.4-6.8	2nd terrace from top	Soft mottled yellowish brown sandy clay
QB 062	P02A/04	6.9-7.3	2nd terrace from top	Firm bluish grey clay
QB 063	P02A/06	7.2-7.7	2nd terrace from top	Bluish grey clay
QB 064	P02A/07	7.7-8.1	2nd terrace from top	Bluish grey clay
QB 065	P02A/08	8.1-8.5	2nd terrace from top	Bluish grey clay
QB 066	P02A/09	8.5-8.9	2nd terrace from top	Bluish grey clay
QB 067	P02A/010	9.0-9.4	2nd terrace from top	Bluish grey clay
QB 068	SK005/001	0.65-1.35	3 rd slip	
QB 069	SK005/02	1.35-2.1	3 rd slip	
QB 070	SK005/03	2.1-2.85	3 rd slip	
QB 071	MM6/002	2.5-2.83	Track 2 + 27	Firm bluish grey homogenous clay

Sample	Borehole	Depth/m	Location	Description
QB 072	P02A/11	9.4-9.8	2nd terrace from top	Firm bluish grey homogenous clay
QB 073	P02A/12	9.8-10.25	2nd terrace from top	Very stiff dark grey homogenous clay
QB 074	P02A/13	10.25-10.6	2nd terrace from top	Very stiff dark grey homogenous clay
QB 075	P02A/14	10.6-11.0	2nd terrace from top	Very stiff dark grey homogenous clay
QB 076	P02A/15	11.0-11.3	2nd terrace from top	Very stiff dark grey homogenous clay
QB 077	P02A/16	11.3-11.6	2nd terrace from top	Firm bluish grey homogenous clay
QB 078	P02A/17	11.6-12.0	2nd terrace from top	Firm bluish grey homogenous clay
QB 079	P02A/18	12.0-12.5	2nd terrace from top	Firm bluish grey homogenous clay
QB 080	MM6/001	0.0-2.5	Track 2 + 27	Firm light brown clay
QB 081	MM6/003	3.0-4.5	Track 2 + 27	Firm bluish grey clay
QB082	SK003/001	0.2-1.5	2nd slip	Loose light brown silty sand
QB 083	SK003/003	3.2-3.55	2nd slip	Firm light greenish brown sandy silt
QB 084	P02A/19	12.45-12.85	2nd terrace from top	Firm light bluish grey homogenous clay
QB 085	SK005/004	3.6-4.35	3 rd slip	
QB 086	MM04/06	5.6-6.3	Track 1 + 75	Very stiff grey homogenous clay
QB 087	P02A/20	12.8-13.3	2nd terrace from top	Firm light brownish grey homogenous clay
QB 088	P02A/21	13.35-13.6	2nd terrace from top	Firm light brownish grey homogenous clay
QB 089	P02A/22	13.25-3.8	2nd terrace from top	Firm light brownish grey homogenous clay
QB 090	MM05A/03	5.3-5.6	Track 2 + 25	
QB 091	MM05A/02	3.6-3.9	Track 2 + 25	
QB 092	MM07/03	3.2-4.9	Track 3 + 25	
QB 093	MM07/01	1.0-1.8	Track 3 + 25	
QB 094	MM05A/01	0.2-1.0	Track 2 + 25	
QB 095	P02A/23	13.8-14.15	2nd terrace from top	Stiff light brownish grey homogenous clay
QB 096	P02A/24	14.15-14.55	2nd terrace from top	Stiff light brownish grey homogenous clay
QB 097	MM08/03 MM08/04	4.5-5.0 6.0-6.5	Track 3 + 75	Medium dense orangish brown sub-angular sub-rounded sand
QB 098	TP03/01	0.7-1.0	Track 1 + 40	Light brown heterogenous sandy clay
QB 099	P02B/04A	9.4-9.9	2nd terrace from top	Stiff grey clay
QB 100	P02B/06A	10.4-10.8	2nd terrace from top	Stiff grey fissured clay
QB 101	P02B/05A	9.9-10.4	2nd terrace from top	Stiff grey clay
QB 102	P02A/25	14.55-15.0	2nd terrace from top	Soft light greyish brown homogenous clay
QB 103	P02B/07	10.7-11.0	2nd terrace from top	Stiff grey clay
QB 104	P02B/05	8.55-8.85	2nd terrace from top	Firm bluish grey clay
QB 105	P02B/03	3.35-3.36	2nd terrace from top	Dense dark brown homogenous clayey sand

Sample	Borehole	Depth/m	Location	Description
QB 106	P02B/04	8.8-8.55	2nd terrace from top	Firm grey clay
QB 107	P02B/20	2.9-3.35	2nd terrace from top	Dense dark brown clayey clay
QB 108	P02B/06	8.85-9.25	2nd terrace from top	Firm bluish grey clay
QB 109	P02B/01	0.4-1.2	2nd terrace from top	Loose light brown heterogenous sand with occasional gravel
QB 110	P02B/14	12.9-13.2	2nd terrace from top	Firm grey clay
QB 111	P02B/16	13.6-14.0	2nd terrace from top	Firm brownish grey clay
QB 112	P02B/10	11.8-12.2	2nd terrace from top	Firm brownish grey clay
QB 113	P02B/15	13.2-13.5	2nd terrace from top	Firm brownish grey clay
QB 114	P02B/13	12.8-12.9	2nd terrace from top	Firm grey clay
QB 115	P02B/09	11.4-11.8	2nd terrace from top	Stiff brownish grey clay
QB 116	P02B/12	12.6-12.8	2nd terrace from top	Firm grey clay
QB 117	P02B/11	12.2-12.6	2nd terrace from top	Firm brownish grey clay
QB 118	P02B/08	11.0-11.4	2nd terrace from top	Stiff brownish grey clay

Table 6.3. Royal School of Military Engineering sample register for third campaign.

Sample 95/1/	Borehole	Depth/m	Location	Description
QB/001	P08/001	0.45-0.7	Marsh	Light brown homogenous clay of upper plasticity
QB/002	P08/002	1.25-1.45	Marsh	Light bluish grey homogenous clay of upper plasticity
QB/003	P08/003	1.8-2.1	Marsh	Soft dark bluish grey homogenous clay of upper plasticity
QB/004	P08/004	2.65-3.1	Marsh	Soft bluish grey heterogenous clay of upper plasticity containing small amounts of limestone
QB/005	P08/005	3.1-3.25	Marsh	Grey homogenous limestone
QB/006	MM02/01	0.1-1.0	Lower track	Stiff light yellowish brown homogenous clay
QB/007	MM02/02	3.2-3.6	Lower track	Very soft light yellowish brown homogenous clay of upper plasticity
QB/008	MM02/03	3.6-3.84	Lower track	Very soft dark blue homogenous clay of upper plasticity
QB/009	MM02/04	5.0-5.4	Lower track	Very soft light blue homogenous clay of upper plasticity
QB/010	MM02/05	5.4-5.68	Lower track	Very stiff dark greenish blue clay
QB/011	P08/006	3.9-4.05	Marsh	Very soft brownish grey heterogenous sandy clay
QB/012	P08/007	/-4.6	Marsh	Very soft brownish grey heterogenous clay of upper plasticity
QB/013	P08/008	/-4.8	Marsh	Very soft brownish grey heterogenous angular gravelly clay with much sand
QB/014	TP02/01	0.9-1.0	Lower track	Stiff light yellowish brown slightly gravelly clay of upper plasticity
QB/015	P07/001	0.5-0.6	First terrace	Soft light brown heterogenous very sandy clay of lower plasticity
QB/016	P07/002	0.6-0.77	First terrace	Soft light mottled greyish brown heterogenous very sandy clay of lower plasticity
QB/017	P07/003	0.77-1.22	First terrace	Soft light mottled heterogenous very sandy clay of lower plasticity with some sub-angular gravel
QB/018	P07/004	1.5-1.95	First terrace	Stiff light mottled greyish brown heterogenous slightly sandy clay
QB/019	P07/005	1.98-2.53	First terrace	Stiff greyish brown heterogenous slightly sandy clay of lower plasticity containing some gravel
QB/020	P07/005A	1.98-2.35	First terrace	Firm light mottled orangish grey brown heterogenous sandy clay of lower plasticity
QB/021	P07/006	2.35-2.69	First terrace	Firm light mottled orangish brown heterogenous sandy clay of lower plasticity
QB/022	TP01/01	0.2-0.4	Bottom track	Stiff light brown homogenous clay of upper plasticity
QB/023	TP01/02	0.1-0.2	Bottom track	Dark yellowish brown very stiff clay of lower plasticity
QB/024	P07/007	2.69-3.12	First terrace	Orangish grey brown heterogenous very sandy clay of lower plasticity

Sample 95/1/	Borehole	Depth/m	Location	Description
QB/025	P07/008	3.15-3.25	First terrace	Mottled yellowish grey brown clay of upper plasticity with some sand
QB/026	P07/009	3.4-3.65	First terrace	Soft orangish grey heterogenous clay of upper plasticity with some sand
QB/027	TP01/03	0.4-0.6	Bottom track	Stiff light brown homogenous clay of upper plasticity with some gravel
QB/028	P07/010	3.73-4.28	First terrace	Firm light greyish brown heterogenous clay of lower plasticity
QB/029	P07/011	4.5-4.65	First terrace	Loose dark brown heterogenous sand containing a little upper plastic clay
QB/030	MM03/02	2.7-2.94	Lower track	Medium dense light yellowish brown heterogenous very silty sand
QB/031	MM03/07	0.7-0.94	Lower track	Firm light brown very silty clay with some gravel
QB/032	P07/012	5.80	First terrace	Very stiff purplish grey heterogenous sightly sandy clay of upper plasticity with some sub-angular cobbles
QB/033	P07/013	6.40	First terrace	Very stiff purplish grey homogenous slightly sandy clay of upper plasticity
QB/034	TP04/01	0.3-0.55	Mid track	Firm light mottled orange brown heterogenous clay of lower plasticity containing some angular irregular gravel
QB/035	TP04/02	0.4-0.5	Mid track	Stiff light mottled orangish brown slightly gravelly clay containing some sand
QB/036	TP04/03	0.7-0.9	Mid track	Stiff light mottled orangish blue slightly sandy clay
QB/037	TP04/04	1.0-1.2	Mid track	Loose light mottled bluish orange slightly gravelly clay containing some sand
QB/038	TP05/01	0.2-0.4	Mid track	Stiff light yellowish brown heterogenous slightly gravelly clay
QB/040	P07/014	10.7-11.15	First terrace	Very stiff purplish blue homogenous clay of upper plasticity
QB/041	P06/001	1.0-1.1	2nd terrace	Soft dark mottled greyish brown heterogenous sandy clay containing a little gravel
QB/042	P06/002	1.2-1.6	2nd terrace	Soft light mottled orangish yellow heterogenous very sandy clay of upper plasticity
QB/043	P06/003	1.9-2.35	2nd terrace	Firm light brownish yellow slightly gravelly clay containing some sand
QB/044	P06/004	2.0-2.3	2nd terrace	Soft dark orangish yellow sandy clay containing a little gravel
QB/045	MM04/001	0.86-0.99	Mid track	Light brown homogenous clay of low plasticity
QB/046	TP07/001	1.6-1.8	PO7 site	Light orange sand
QB/047	TP07/002	1.4-1.8	PO7 site	Light mottled orangish grey brown clay of upper plasticity
QB/048	P06/005	2.85-3.35	2nd terrace	Dark orangish brown very sandy clay
QB/049	P06/006	3.0-3.2	2nd terrace	Light yellowish brown very sandy clay
QB/050	MM04/002	1.5-1.73	Mid track	Bluish grey clay of upper plasticity

Sample 95/1/	Borehole	Depth/m	Location	Description
QB/051	MM05/001	2.5-2.73	Mid track	Light brown clay of upper plasticity
QB/052	P06/008	5.0-5.2	2nd terrace	Stiff light green homogenous clay containing s gravel
QB/053	P06/009	5.8-6.25	2nd terrace	Stiff mottled greenish grey heterogenous clay of upper plasticity
QB/054	P06/010	6.3-6.5	2nd terrace	Light brownish green heterogenous very sandy clay of upper plasticity
QB/055	MM06/002	1.2-1.43	Upper track	Light orange homogenous sand
QB/056	MM06/001	0.8-1.03	Upper track	Soft dark brown very sandy clay of low plasticity
QB/057	MM06/CBR 2	0.3-0.4	Upper track	Dark brown very sandy clayey sand with some cobbles
QB/058	MM05/CBR 2	0.3-0.4	Upper track	Light brown homogenous clay of upper plasticity
QB/059	MM05/CBR 1	0.1-0.2	Upper track	Dark brown clay topsoil
QB/060	MM06/CBR 1	0.1-0.2	Upper track	Dark brown very clayey sand topsoil of low plasticity
QB/061	MM05A/001	1.6-1.83	Upper track	Soft light greyish brown clay
QB/062	P06/014	7.8-8.5	2nd terrace	Stiff dark greyish blue clay of upper plasticity
QB/063	P06/015	8.7-9.0	2nd terrace	Soft dark greenish blue slightly silty clay of upper plasticity
QB/064	P06/013	7.5-7.7	2nd terrace	Firm light greyish blue heterogenous clay of upper plasticity
QB/065	P06/016	7.8-9.0	2nd terrace	Stiff dark greyish blue clay of upper plasticity
QB/066	MM06A/001	1.6-1.8	Upper track	Light orange sand
QB/067	P05/001	1.2-1.45	Translational	Stiff light orangish grey heterogenous clay of upper plasticity
QB/069	P05/003	1.3-1.5	Translational	Stiff light brownish grey heterogenous clay of upper plasticity
QB/070	P05/004	1.45-1.9	Translational	Soft light brown homogenous clay of upper plasticity
QB/071	P05/005	2.0-2.45	Translational	Firm mottled brownish grey, inter-stratified clay of upper plasticity
QB/072	P05/006	2.5-2.95	Translational	Very stiff dark grey blue laminated clay
QB/073	TP5B/001	2.5-3.2	PO5 site	Very stiff light and dark mottled purplish brown heterogenous clay of upper plasticity
QB/075	P05/007	2.6-3.05	Translational	Firm light blue mottled grey clay of upper plasticity
QB/076	P05/009	3.15-3.6	Translational	Firm mottled grey clay of upper plasticity
QB/077	P05/010	3.6-4.05	Translational	Very stiff dark grey inter-stratified clay of upper plasticity
QB/078	P05/008	2.0-2.3	Translational	Firm brownish grey heterogenous clay of upper plasticity
QB/079	P05/011	4.0-4.4	Translational	Firm light brown grey homogenous clay of upper plasticity

Sample 95/1/	Borehole	Depth/m	Location	Description
QB/080	P05/12	4.4-4.8	Translational	Stiff light blue inter-stratified clay of upper plasticity
QB/081	TP01A/02	0.4-0.5	Bottom of track	Soft light brownish grey sandy clay containing some gravel
QB/082	TP01A/01	0.0-0.15	Bottom of track	Firm dark brown sandy clay
QB/083	TP01A/03	0.4-0.55	Bottom of track	Soft light brownish grey sandy clay
QB/084	TP1		Bottom of track	
QB/085	TP06/03	0.3-0.4	2nd terrace	Light orangish brown clay of upper plasticity
QB/086	P05/017	5.3-5.55	Translational	Stiff light grey blue homogenous clay of upper plasticity
QB/087	P05/013	4.8-5.25	Translational	Firm light greyish blue homogenous clay of upper plasticity
QB/088	P05/015	5.05-5.5	Translational	Firm light blue clay of upper plasticity
QB/089	P05/016	5.3-5.53	Translational	Firm light grey blue clay of upper plasticity
QB/090	TP06/02	0.3-0.45	2nd terrace	Firm dark brown sandy clay with some gravel
QB/091	TP06/01	0.0-0.5	2nd terrace	Firm dark brown sandy clay
QB/092	TP07/01	0.05-0.2	First terrace	Loose dark brown clayey sand with some gravel
QB/093	TP07/02	0.3-0.45	First terrace	Loose dark brown clayey sand containing some gravel and occasional cobbles
QB/094	TP07/03	0.3-0.5	First terrace	Loose light yellowish brown clayey sand containing some ravel
QB/095	P06/019	9.5-9.85	2nd terrace	
QB/096	P06/020	9.7-10.5	2nd terrace	Light grey blue homogenous clay
QB/097	P06/021	11.10-11.55	2nd terrace	Stiff light greyish blue homogenous clay
QB/098	P06/022	11.10-11.55	2nd terrace	Stiff dark greyish blue clay
QB/099	P06/023	11.6-12.0	2nd terrace	Light blue grey fissured clay of upper plasticity
QB/100	P06/024	11.8-	2nd terrace	Mudstone
QB/101	P06/025	12.0-12.45	2nd terrace	Very stiff greyish blue clay of upper plasticity
QB/102	P06/026	12.7-13.1	2nd terrace	Stiff greyish blue fissured clay
QB/103	P06/027	13.1-13.55	2nd terrace	Stiff greyish blue fissured clay
QB/104	P06/027	14.1-14.4	2nd terrace	Light greyish blue homogenous clay
QB/105	MM07/01	4.4-	Upper track	Dark brown sandy clay containing some gravel
QB/106	MM08/01	1.0-1.23	Track at scarp	Dark greenish brown clayey sand
QB/107	MM08/02	1.23-1.7	Track at scarp	Dark greenish brown clayey sand
QB/108	P06/028	14.95-15.4	2nd terrace	Light grey blue fissured clay
QB/109	P06/029	15.4-15.85	2nd terrace	Light greyish blue fissured clay of upper plasticity
QB/110	P06/030	15.7-15.9	2nd terrace	Light bluish grey laminated clay

Table 6.4. Summary of laboratory data obtained by the Royal School of Military Engineering.

Sample	MC%	LL%	PL%	PI%	Gravels	Sands	Fines
QB 001	47/48						
QB 002	33/34						
QB 003	27/43						
QB 004	32/32						
QB 005	50/54						
QB 006	42/49						
QB 007	13/13						
QB 008	42/45						
QB 009	29/29						
QB 010	43/33						
QB 011	48/59						
QB 012	48/59	28	10	18			
QB 013	12/13						
QB 014	33						
QB 015	32						
QB 016	31/32	62	28	34			
QB 017	18						
QB 019	14/14/14						
QB 020	14/15/17						
QB 021	18						
QB 022	13						
QB 023	30						
QB 024	14						
QB 025	44						
QB 026	14/16						
QB 027	17						
QB 029	14						
QB 030	20						
QB 031	17						
QB 032	18						
QB 035	28/28	56	21	35			
QB 036	20/20	47	24	17			
QB 038	25/26	56	21	35			
QB 039	32/26	52	29	56	21	35	
QB 040	9.2/9.2						

Sample	MC%	LL%	PL%	PI%	Gravels	Sands	Fines
QB 041	12/11						
QB 043	11/9.8						
QB 044	19						
QB 045	21						
QB 046	20/20	20/20	44	16			
QB 053	35/37	57	26	31			
QB 054	23/23	63	24	39	0	6	94
QB 055	24/24	24/24	55	27			
QB 056	20/24						
QB 057	25						
QB 058	23						
QB 059	33						
QB 060	23/23						
QB 061	20						
QB 062	25						
QB 063	24						
QB 064	25						
QB 065	27						
QB 066	27						
QB 067	25						
QB 072	25						
QB 073	23						
QB 074	22						
QB 075	22						
QB 076	22						
QB 077	20						
QB 078	19						
QB 079	14						
QB 080	21/22	56	32	24			
QB 081	28/28	52	19	33			
QB 084	18						
QB 086	55/59	51	32	19			
QB 087	17						
QB 088	15						
QB 089	14						
QB 090	27						

Sample	MC%	LL%	PL%	PI%	Gravels	Sands	Fines
QB 091	26						
QB 092	24/24	50	22	28			
QB 093	26/27	67	27	40			
QB 094	24	27	69	20	49	0	31
QB 095	17						
QB 097	14				0	98	2
95/1/QB/001	38	69	31	38		61	39
95/1/QB/002	39	48	22	26		31	69
95/1/QB/003	44	48	20	28		29	71
95/1/QB/006	35	60	25	35		14	86
95/1/QB/007	48	54	23	31		30	70
95/1/QB/008	31						
95/1/QB/009	69	59	22	37		23	77
95/1/QB/011	29				4	91	5
95/1/QB/014	20	50	22	28	32	16	52
95/1/QB/015	21	45	23	22	4	66	30
95/1/QB/016	21	42	22	20		37	63
95/1/QB/020	27	24	59	35	1	36	63
95/1/QB/025	27	45	24	21		1	99
95/1/QB/026	30	59	24	35		11	89
95/1/QB/027	36	49	20	29	6	27	67
95/1/QB/033	24	52	27	25	21	17	62
95/1/QB/035	33	66	31	35	5	13	82
95/1/QB/036	28	63	28	35		11	89
95/1/QB/037	32	46	22	24	11	60	29
95/1/QB/038		55	26	29		22	78
95/1/QB/041	23	40	24	16	1	48	51
95/1/QB/042	19	41	23	15	26	36	38
95/1/QB/044	23	41	25	16	1	36	63
95/1/QB/046						99	1
95/1/QB/047		52	28	24		2	98
95/1/QB/049		33	18	15	1	69	30
95/1/QB/052		61/ 65	21 /28	40 /37	12	24	64
95/1/QB/054		42	12	30		43	57
95/1/QB/058	34						
95/1/QB/059	46						

Sample	MC%	LL%	PL%	PI%	Gravels	Sands	Fines
95/1/QB/060	21						
95/1/QB/069	27	57	24	33		26.5	73.5
95/1/QB/073	34	38	35	3		20	80
95/1/QB/078	27	79	38	41		45	55
95/1/QB/081	37	63	29	34		24	76
95/1/QB/082	14						
95/1/QB/083	34						
95/1/QB/085	26	81	33	48	1	26	73
95/1/QB/086	19	47	24	23		2	98
95/1/QB/090	37						
95/1/QB/091	33/37						
95/1/QB/092	23/26						
95/1/QB/093	21/19						
95/1/QB/094	16	43	27	16	30	37	33
95/1/QB/096	36	46	21	25		5	95

Table 6.5. Summary of laboratory results obtained by Kingston University.

Sample	Depth/m	MC%	LL%	PL%	PI%	Gravels	Sands	Fines
SA1:1	-0.55	13.25						
SA1:3	-1.05	19.5						
SA1:3	-1.225	21.35						
SA1:3	-1.4	3.99						
SA1:5	-1.725	3.03						
SA1;7	-2.225	5.6						
SA1:9	-2.725	21.09						
SA1:11	-3.225	3.57						
SA1:13	-3.725	6.48						
SA1:15	-4.05	16.07						
SA1:15	-4.4	17.31						
SA1:17	-4.55	19.76						
SA1:17	-4.9	22.34						
SA1:19	-5.224	19.32						
SA1:19	-5.226	6.6						
SA1:21	-5.755	23.5						
SA1:23	-6.05	22.79						
SA1:23	-6.4	29.62						
SA1:25	-6.55	24.51						
SA1:25	-6.9	24.44						
SA1:28	-7.05	26.37						
SA1:28	-7.35	27.41						
SA1:32	-8.05	28.92						
SA1:32	-8.4	27.97						
SA1:34	-8.55	20.73						
SA1:34	-8.9	30.75						
SA1:36	-9.05	20.56						
SA1:36	-9.4	26.67						
SA1:38	-9.55	27.66						
SA1:38	-9.9	18.1						
SA2:14	-3.72	23.14						
SA2:15	-4.02	24.88						
SA2:16	-4.32	20.51						
SA2:16	-4.33	24.53						
SA2:16	-4.34	26.79						
SA2:17	-4.62	21.69						
SA2:18	-4.84	22.83				7	19	74
SA2:18	-4.86	24.74						
SA2:18	-4.75	30.58	48	20	28			
SA2:19	-5.22	24.95						
SA2:20	-5.35	27.12	41	20	21	12	24	64
SA2:20	-5.7	23.87						

Sample	Depth/m	MC%	LL%	PL%	PI%	Gravels	Sands	Fines
SA2:21	-5.82	25.5						
SA2:22	-5.95	28.5	63	26	37			100
SA2:22	-6.3	25.1						
SA2:23	-6.42	23.78						
SA2:24	-6.55	31.1	67	25	42		15	100
SA2:24	-6.9	25.38						
SA2:25	-6.99	29.51						
SA2:25	-7.1	23.13						
SA2:26	-7.15	18.74	58	26	32			100
SA2:26	-7.5	27.9						
SA2:28	-7.65	18.46	53	22	31			100
SA2:28	-8	16.32						
SA2:29	-8.05	18.79						
SA2:30	-8.15	16.42	53	22	31		9	91
SA2:30	-8.5	17.75						
SA2:32	-8.65	17.54	53	23	30			
SA2:32	-9	16.8						
SA2:34	-9.15	16.31	53	23	30			100
SA2:34	-9.5	18.22						
SA2:36	-9.65	19.09	53	23	30			
SA2:36	-10	17.56						
SA2:38	-10.15	16.64	53	23	30			
SA2:38	-10.45	18.57						
SA2:40	-10.65	12.14						
SA2:40	10.95	17.41						
SA2:42	-11.15	18.15						
SA2:42	-11.45	16.71						
SA2:44	-11.65	21.72	52	24	28			
SA2:44	-12	15.96						
SA2:46	-12.15	16.71	53	23	30			
SA2:46	-12.5	18.18						
SA2:48	-12.65	4.13	58	22	36			
SA2:48	-12.75	14.7						
SA2:50	-13.15	13.35	34	20	14			
SA2:50	-13.25	13.06						
SA3:1	-0.55	11.6						
SA3:1	-0.9	10.9						
SA3:5	-1.55	12.59						
SA3:5	-1.9	13.01						
SA3:9	-2.725	11.6						
SA3:11	-3.225	17.18						
SA3:13	-3.55	17.31						
SA3:13	-3.9	17.62						
SA3:15	-4.05	15.15						

Sample	Depth/m	MC%	LL%	PL%	PI%	Gravels	Sands	Fines
SA3:15	-4.25	6.9						
SA3:17	-4.55	14.92						
SA3:17	-4.9	6.63						
SA3:19	-5.05	6.544						
SA3:19	-5.4	18.21						
SA3:22	-5.55	14.44						
SA3:22	-5.9	15.43						
SA3:24	-6.05	13.52						
SA3:24	-6.4	16.01						
SA3:26	-6.55	15.48						
SA3:26	-6.9	21.57						
SA3:28	-7.05	15.07						
SA3:28	-7.4	18.42						
SA3:30	-7.55	19.91						
SA3:30	-7.9	17.3						
SA3:32	-8.05	20.94						
SA3:32	-8.4	18.28						
SA3:36	-9.05	21.25						
SA3:36	-9.4	21.99						
SA3:38	-9.55	21.62						
SA3:38	-9.9	21.98						
SA3:40	-10.05	18.78						
SA3:40	-10.4	27.77						
SA3:42	-10.7225	18.47						
SA3:44	-11.05	21.05						
SA3:44	-11.4	20.64						
SA3:46	-11.55	23.66						
SA3:46	-11.85	21.56						
SA3:48	-12.05	18.24						
SA3:48	-12.4	19.73						
SA3:50	-12.55	17.61						
SA3:50	-12.85	14.92						
SA3:52	-13.05	14						
SA3:52	-13.4	10.87						
SA3:54	-13.55	17.24						
SA3:54	-13.9	16.38						
SA3:56	-14.05	15.31						
SA3:56	-14.35	16.78						
SA3:58	-14.55	17.89						
SA3:58	-14.85	16.8						
SA3:60	-15.05	16.96						
SA3:60	-15.4	17.45						
SA3:62	-15.7	16.61						
SA3:64	-16.05	16.7						

Sample	Depth/m	MC%	LL%	PL%	PI%	Gravels	Sands	Fines
SA3:64	-16.25	18.23						
SA3:66	-16.55	15.86						
SA3:66	-16.8	16.15						
SA2B:2	-3.125	18.15						
SA2B:3	-3.5	20.82						
SA2B:4	-9.65	25.84						
P02B:5	-10.15	25.07						
P02B:6	-10.6	21.88						
P02B:7	-10.85	21.69						
P02B:8	-11.2	20.06						
P02B:9	-11.6	19.09						
P02B:10	-12	18.04						
P02B:11	-12.4	15.13						
P02B:12	-12.7	15.15						
P02B:13	-12.85	13.28						
P02B:14	-13.05	16.15						
P02B:15	-13.35	15.85						
P02B:16	-13.8	16.21						
P03A:1	-1.15	10.36						
P03A:2	-2.5	14.74						
P03A:3	-3.525	24.05						
P03A:4	-3.975	32.68						
P03A:5	-4.45	30.43						
P03A:6	-5.3	23.1						
P03A:7	-5.9	23.16						
P03A:7	-6.25	19.44						
P03A:8	-6.45	20.42						
P03A:9	-6.8	23.31						
P03A:10	-7.3	19.77						
P03A:11	-7.8	17.46						
P03A:13	-8.775	20.81						
P03A:14	-9.325	21.75						
P03A:15	-9.75	16.7						
P03A:15	-9.9	21.99						
P03A:17	-10.4	23.84						
P03B:1	-2.65	35.48						
P03B:2	-3.025	29.99						
P03B:3	-3.475	26.6						
P03B:4	-3.725	25.8						
P03B:5	-4.025	17.69						
P03B:6	-5.15	22.52						
P03B:7	-5.725	22.16						
P03B:8	-6.4	21.17						
P03B:9	-6.85	22.58						

Sample	Depth/m	MC%	LL%	PL%	PI%	Gravels	Sands	Fines
P03B:10	-7.275	19.9						
P03B:11	-7.725	18.49						
P03B:12	-8.35	19.07						
P03B:13	-8.625	21.68						
P03B:13	-8.7	35.8						
P03B:14	-9.325	17.24						
P03B:15	-9.7	22.79						
P05:1	-1.325	26.91						
P05:4	-1.675	29.32						
P05:5	-2.225	30.82						
P05:6	-2.725	33.96						
P05:7	-2.825	25.97						
P05:9	-3.375	24.46						
P05:10	-3.825	21.5						
P05:11	-4.23	25.48						
P05:12	-4.6	21.09						
P05:13	-5.025	18.51						
P05:15	-5.3	18.63						
P05:16	-5.425	18.04						
P06:3	-2.125	31.55						
P06:5	-3.1	18.84						
P06:9	-6.025	28.04						
P06:5	-8.85	25.17						
P06:9	-9.725	20.66						
P06:22	-11.325	21.86						
P06:23	-11.8	21.75						
P06:25	-12.1	17.33						
P06:25	-12.35	40.54						
P06:27	-14.25	16.64						
P06:28	-15.05	34.94						
P06:28	-15.3	21.06						
P06:29	-15.625	17.97						
P07:3	-0.995	12.8						
P07:4	-1.725	28.09						
P07:5	-2.273	32.93						
P07:6	-2.52	29.48						
P07:7	-2.905	26.02						
P07:10	-4.015	24.45						
P07:14	-10.925	18.72						

Table 6.6. Summary of borehole logs.

Borehole number	Colluvium/ Top Soil	Hythe Beds	Atherfield Clay	Weald Clay	Slip Surfaces	Average Piezometric Level
BH1	0-2.7			2.7		-1.5
BH2	0.2.5			2.5		
BH3	0-2.7					-0.5
BH4	0-4.0					
BH5	0-3.05					-2
BH6	0-5.5			5.5		-1
BH7	0-2.3			2.3		-2.5
SA1	0-8			8		-5
SA2	0-8.7		8.7-11.5	11.5		-1
SA3	0-7.3		7.3-7.5	7.5		-9
P01B	0-4.8					
P02A	0-10.25			10.25	11.5, 12.7, 12.9, 13.1, 13.2, 14.0, 14.1, 14.3	
P02B	0-10.4			10.4		-7
P03A	0-5			5		
P03B	0-5			5		-7
P05*	0-4			4		
P05	0-4.4			4.4		
P06*	0-9			9		
P06	0-9			9		
P07*	0-7			7		-1.5
P07	0-3.75			3.75		-1.5

*Logged by the RSME (drilling logs only).

Key to position of boreholes

Rotational zone	Degradation zone	Accumulation zone	Marsh

Plate 6.1 (Upper). Example of a 'slickensided' shear surface.
Plate 6.2 (Lower). Trial pit.

Plate 6.3 (Upper). Trial pit.
Plate 6.4 (Lower). The rear scarp in the Hythe Beds.

Plate 6.5. The pilcon cable percussion rig.

Problems of slow moving landslides

7.1 Introduction

Although slow, compared with many landslides outside of the UK, such as avalanches and rock falls in mountainous regions, the movements on The Roughs were considered to be large displacement events, which took place on a short timescale, bearing in mind that the escarpment is considered to have achieved a state of long term equilibrium, and thus to be "stable". Since the speed of a landslide is somewhat relative, employing a universal system of defining landslides by their velocities is, therefore, necessary. The landslide velocity scale that the author choses to adopt for this work is that proposed by **Cruden & Varnes** (1996) and is shown in *Table 5.1*. Landslides in the *Very Slow* to *Extremely Slow* ranges are examined primarily, here. These slow moving landslides are sometimes termed "creep" *(Section 5.5)*.

7.1.1 "Rapid" motion landslides in the UK

First-time slide failures in the generally low, gently sloping, hillslopes of the southern and central parts of the UK, have a record for comparatively slow movement, although the magnitude of the movements might be an appreciable fraction of the slope height. **Potts** *et al., 1997* show how dilatancy in stiff clays leads to pore pressure reductions as progressive failure is approached, and during sliding, decreases in total stress accompanying the gross geometric changes in the slope, lead to further pore pressure decreases (**Cooper** *et al,* 1998). So that in total stress terms, the shear strength of the slide surface may not be so dramatically reduced as the drained brittleness implies. These factors decrease the rate of movement more than the magnitude, so that first-time slides in stiff clays occur in hours or days, rather than minutes or seconds, so that inertial effects are small, and overshooting the point of equilibrium under residual strength, if any, is minimal. These landslides generally exhibit *moderate* velocity *(Table 5.1)* or exceptionally just inside the *fast* class.

7.1.2 Slow-moving landslides in the UK

Most landslides in Britain, such as those on The Roughs, are the reactivation of earlier movements and thus they move on pre-existing, or largely pre-existing shear surfaces. Such shear surfaces usually have negligible brittleness, especially in comparison to the loss of strength with strain from the peak strength to the residual strength. As a corollary to the belief that first-time movements are quicker and have large displacement, it is widely believed that slides on pre-existing shears move slowly and very small distances, primarily as a consequence of the essential lack of brittleness exhibited by a slip surface.

7.2 Slope stability analysis and modelling

One of the primary divisions of the techniques for slope stability modelling is between those models which may be loosely termed "static methods", in contrast to models which might be termed "dynamic methods".

In the static methods, equilibrium along a potential (or actual) slip surface is considered. This assessment may be made on one or more typical sections through a landslide, or by considering its shape in three dimensions. The principles are, however, the same. The forces that tend to cause instability are evaluated, often by dividing the landslide mass into vertical slices, and computing the forces in each slice (**Petterson**, 1955). A simple sum across all the slices is sufficient for this purpose and the nett driving force is D. Similarly the maximum available forces that could be mobilized from the strength of the soil or rock mass are also calculated and summed (or sometimes *integrated, e.g.* **Morgenstern & Price**, 1965) along the potential slip surface to produce the maximum available resistance, R. These two sums can produce a ratio, which is termed the factor of safety, F (*Section 5.9).*

$$F=\frac{R}{D} \qquad \text{Eq. 7.1}$$

Where it is more appropriate to do so, moments rather than forces are used (**Bishop**, 1955). Various refinements to the general procedure exist: these are concerned with modelling pore water pressures in the slide mass; various formulations for the shear strength; seismic forces; ground anchor forces and other factors. Not least of these other factors is the nature of the interaction which is assumed to occur between the individual slices, which can make some difference to the results.

Continuum methods are sometimes used instead of limit equilibrium methods. The analyst then has a choice between using this refined stress analysis as the basis for computing the D/R ratio to provide an improved, but otherwise standard, factor of safety, or he can track deformations at one or more points on the slope section while factoring the shear strengths progressively, and thus from a change in deformation behaviour, deduce the factor of safety.

Although limit equilibrium methods do not explicitly consider strains, some deformations in a slope can be expected as the factor of safety diminishes. Some cases which subsequently progressed to collapse (*e.g.* the engineered slope failures at Selborne (**Cooper *et al.***, 1998) or the Carsington Dam failure (**Skempton & Coats**, 1986), suggest that unmistakable signs of slope distress might be recorded in instruments or seen on the surface. An appraisal of such cases suggests that this begins to occur at Factors of Safety of the order of 1.2. It is only when the factor of safety drops below one that the equilibrium between the *actual* resistance (*i.e.* that portion of the maximum available resistance which is *actually* mobilized) and the driving forces is disturbed. Dynamic models are based on the equations of motion, although alternative energy-based models are equally applicable. The *ratio* of resistance R to driving force D in a moving slide becomes less important than the *difference* between them, which is the nett force on the slide mass $(D - R)$, which leads to acceleration (*Eq. 7.2*). Where the time-history of the nett force can be computed, the velocity and displacements of the slide mass follow from the mathematics. Integrating once with respect to time produces the velocity profile (*Eq. 7.3*), and twice (*Eq. 7.4*), the distance of travel.

7.3 Mechanics of movement - a simplified treatment

For a block slide, or other failure along a planar slip surface, which can be approximately modelled by the infinite slope method (**Skempton & Delory**, 1957 after **Haefeli**, 1948), the meaning of D and R may be readily visualised (*Figure 7.1 (upper)*). For a slide with a rotational character, *e.g.* a slip circle (**Petterson**, 1955, **Bishop**, 1955), a similar analysis could be proposed in terms of moments, rather than forces (*Figure 7.1 (lower)*). However, the resultant of all the resisting forces, vectorially summed over all the slices, together with the resultants of all the normal forces and the weight, form a polygon of forces, which reduce even a rotational landslide to the equivalent "block" slide. This process is implicit in the Newmark-Sarma approach.

This nett driving force causes acceleration, a, which may vary with time (*Figure 7.2*).

$$a=\frac{(D-R)g}{W} \qquad \text{Eq. 7.2}$$

In order to proceed, we must know the way in which a, and hence $D - R$, varies with time. For example, consider the resistance R remaining constant while the driving force is increased instantaneously, such as a large load placed rapidly on the head of the slide. With a translational landslide with an extensive track to be travelled over, the force difference (D - R) would stay constant, so that the mass could accelerate. Eventually, however, the resistance must rise, usually as a result of gross geometric changes when the end of the available track is reached. This change in resistance is more marked in a rotational landslide, or one with a rising toe. Various time-histories for the acceleration could therefore be envisaged.

Similarly, the driving forces may stay constant while the resistance alters. One could envisage this being the result of an increase in the level of the water table, followed by a decrease. Seasonal (or shorter-term) variation in the ground water table elevation may produce nett force imbalances which take a waveform, which can be approximated to (as in the Newmark-Sarma approach) by sine, square or triangular waveforms. No doubt other, more complex, interactions are possible.

$$v=\frac{(D-R)gT}{W} \qquad \text{Eq.7.3}$$

$$s=\frac{(D-R)gT^2}{2W} \qquad \text{Eq. 7.4}$$

However, if one imagines that the nett force imbalance is instantaneously applied, remains constant for a time *(T)*, and then reduces, resulting in a square waveform, the integrations become trivial. Thus, integrating the acceleration, *a*, once against time, would yield a velocity, and twice, a distance moved. These integrations are trivial for the square wave pulse, and up to time, *T*, give:Deceleration then occurs because the sign of *D - R* changes. To describe the motion of a slide from rest through a state of movement and back to rest would require an extra term for *v* and *s* in the above expressions. For the purposes of developing this argument, however, it is sufficient to operate with the velocity and distance at *T*, when the magnitude of the forcing function begins to decrease, noting that unless the sign of *(D - R)* changes these may not be peak velocity *or* displacement. It is inescapable that the magnitude of peak velocity is a function of the parameters *(D - R)/W* and *T*, and that in addition, displacements are a function of *(D - R)/W* and T^2. It is not of any further benefit to explore the mechanics any deeper, since a number of striking results flow from even this level of simplification.

In the case of a seismic pulse, *T* is likely to be only a small fraction of a second. Therefore, almost regardless of the magnitude of *(D - R)*, the velocity must be small and the displacement even smaller. In stark contrast, most other destabilising events have an appreciable duration. For example, the sine pulse could represent the effect on the resistance of a seasonal rise and fall in the ground water table, and the positive *(D - R)* might last for months.

Even a cursory trial calculation with *Equations 7.3* and *7.4* will show appreciable velocity and displacement. Hence, we should not be surprised at rapid or large runout movements: they only signify a nett driving force applied for a finite period. For example, in a system of *kN*, *m* and *sec*, a nett force of 1% of the slide weight acting for a month (which is approximately 2.6 *million* seconds) will impart an improbably large velocity to the slide mass. The same result occurs when

0.001% of the slide weight forms the nett driving force *(i.e. 1.0>F>0.99999 approximately)*.

Where the destabilising pulse is the result of, for example, a winter water level rise, the time period is very long, and large deformations as well as velocities are inevitably calculated. It is unlikely that this corresponds to reality, since many slide reactivations are slow and of small magnitude. Even the rise in groundwater level resulting from a week of wet weather culminating in a major deluge is likely to cause a state of $F<1$ for some hours, during which time even minuscule accelerations would mount up to appreciable velocities and displacements.

With fast slides, where the computation of fast displacements and large run-out is the intention, the problems which arise in modelling come from factors such as restructuring of the slide zone (soil/rock mechanics), gross geometric changes in the slide mass (morphology), changes in its internal structure (rock mechanics), and the nature and geometry of the track it passes over (geomorphology). All of these are effects not likely to be present in a slow landslide, except, perhaps, changes in the shear surface.

In modelling the movement of landslides it has been conventional to invoke a rate-dependent or "viscous" component to shear strength. This corresponds, in an energy-based model, to the factors which dissipate energy in the shear surface or within the sliding mass. In effect, to use the rate effect in shear strength to slow a landslide, one must have rate parameters which increase R to such an extent that no appreciable velocity or displacement can result. This, in effect, requires that $(D - R)$ is always virtually zero.

Strain rate effects are, of course, well-known in soil and rock mechanics, but they are experienced at the higher rates of strain in the laboratory. **Tika** *et. al.*, 1996 quote ring shear tests at variable speeds which reveal losses, as well as gains of strength as the rate of shear increases. In any case, they concede that rate effects are of little significance for low rates of shear, and rate dependency in the shear strength comes into play primarily at high rates of shear. This is useful for modelling controls on rapid motion landslides, but useless when slow or creeping landslide movements are considered. The threshold for a change in rate effect from insignificant to significant in their tests occurs at the same rate in all their tests, regardless of material used. Such an effect is more likely to be the result of apparatus design than to some universal aspect of soil behaviour.

Further work on strain rate effects has been undertaken at Kingston since the completion of the field and laboratory programme related to the Author's research. The following summary (**Bromhead**, pers. Comm., 2000) describes the work and its results.

Most of the concern in respect of errors in the measurement of residual strength in the Bromhead ring shear machine has focussed on frictions, which may cause the residual strength to be overestimated. Some of this friction is connected with the centring pin, other frictions relate to the friction between the load hanger and the top platen. Intrinsically, these frictions are small, but may amount in total to a few percent of the shear measured, thus limiting in absolute terms the accuracy of residual strength measurement. Other errors, of both algebraic signs, may be caused by failure to strain the sample far enough, and other effects, although such errors can be minimised by simply running the test long enough and correctly interpreting the results.

A further unquantifiable error comes from the work done in continuously remoulding the sample to reform its shear surface as material is extruded from the gap. The design of the small (Bromhead) ring shear depends on this gap being sufficiently small to minimise soil extrusion, but sufficiently large to prevent binding occurring with the small amount of play which exists in the centring arrangement. Soil extrusion can be observed as the test proceeds, with a fine deposit visible around the outer gap (the inner gap is concealed from view). It is also evident in the continuing settlement of the loading platen from which the volume extruded can be determined.

Lest it be thought that this is solely a defect of the small ring shear test, consider too the IC/NGI apparatus. This is a machine which shears not at the sample surface (or close to it) but through the approximate mid plane of the sample. A method is specifically provided for keeping the confining rings in close contact during shear, and for opening them when it is required to perform a test free of confining ring friction. The load required to open the gap is determined by a load ring, which may be subtracted from the total normal applied stress, although this is an undrained unloading, and the sample may need to come back into pore water pressure equilibrium with the gap open before the disturbing effect is lost. In practice, the tests nowadays appear to be performed with the gaps held permanently open. Accordingly, there is a continual loss of material from the specimen.

The apparent residual shear strength in the test is:

apparent residual shear strength	=	true residual shear strength	+	friction	+/-	test procedure errors	+	rate of work done by extrusion

It may not be possible to precisely quantify the work done on soil extrusion in absolute terms, but if the work is proportional to the volume extruded, and the rate of doing work to the rate of volume extrusion, then we find some correlation between the change in apparent residual shear strength and the strain rate to take a similar form to that actually observed in variable rate tests.

A further effect is of some significance. If the sample is sheared quickly enough, the extrusion changes from being drained to being an undrained phenomenon. By its design, the IC/NGI device exhibits what may be a transitional behaviour at smaller strain rates than the Bromhead device, where the drainage path is shorter. This accounts for the observation that for lower strain rates there is always some increase of apparent shearing resistance, but as the strain rate increases, the rate effect may be come negative: a material whose undrained shear strength is less than its drained strength, perhaps.

Tests done at Kingston reveal negligible rate effects in specimens of several natural clays (London Clay, Gault) tested in the ring shear machine up to the *Fast* rate of landslide movement, **Cruden & Varnes** (1996). Beyond that, rate effects are measured, but they are accompanied by massive extrusion of soil, limiting the maximum strain rate to that which permits a constant shear strength to be determined before the sample container is exhausted. The threshold strain rate at which a change in behaviour occurs is different to that observed by Tika et al.

This is incompatible with all rate-dependency models for slow landslide movement

The norm, even for pre-existing landslides with slip surfaces of low or negligible brittleness but subject to destabilizing forces at F<1 for an appreciable time *should* be large movements at fast rates. This is not unknown.

7.4 Slides on pre-existing slip surfaces with slow motion and small movements

In the following sections, a variety of cases where the movements are slow or small are discussed. They are divided into sections in which the possible mechanics are covered.

7.4.1 Quasi-negligible nett driving force

Many large coastal landslide systems appear to move very slowly, but almost continuously, except in a small number of instances where the movements have been large: most if not all of which have been associated with elements of first-time failure. The primary factor that is common to all of them which could be causing progressive destabilisation is the year in year out slow, steady denudation of the shore, which forms the toe zone of the landslides.

If the rates of movement are considered, they will be seen to be extremely slow. At St. Catherine's Point, Isle of Wight, successive surveys have observed (**Bromhead *et al.*,** 1988) movements at a rate of 25 mm per annum, approximately 0.5 mm per week. Other parts of the landslide complex known as the Undercliff, of which the St. Catherine's Point landslides form a westerly part, move at rates of up to 50 mm per month (**Woodruff, *pers. comm.*,** 1997). The landslide toes break out offshore, and it will be readily appreciated that if the foreshore were to be denuded at an average rate of, for example, 0.1 mm per day (0.05 mm per tide), then all the slide mass is doing is moving into a new position of stability (*Figure 7.3*). In this case, the movements probably occur tide by tide, and so are not resolved by the deflection measuring system and are occurring in response to a nett destabilising force $(D - R) / W$ which is effectively negligible.

An intriguing result comes from attempts to model the influence of minuscule toe scours using limit equilibrium. Using iteration with some system of equations to compute the Factor of Safety is conventional. When the difference between consecutive estimates of the Factor of Safety falls below some finite level, say $DF<0.0001$, then convergence is assumed, and the resulting Factor of Safety is reported. The method is clearly useless at computing changes in the Factor of Safety, DF, of less than the iteration tolerance, and is likely to be poor within a range of several times the iteration tolerance. The work-around is to compute DF for an appreciable destabilizing event (e.g. toe scour) and to determine the actual DF *pro-rata* to the volumes involved.

7.4.2 Final stages of a large (perhaps prehistoric) landslide or premonitory movements of a new one

A landslide starting from a state of rest, and finishing its motion at rest, must have periods at the start and finish of motion where the speed of movement is small. Most landslides give clear premonitory signs, such as the opening of tension cracks at the slide head, and other ground surface deformations. They also indicate a wide spread of internal strain by the release of acoustic energy (**Dixon *et al.,*** 1996). It is possible to mistake the onset of a first time failure for slow movement of a pre-existing slide if the magnitude of the slide event is such that even the pre-failure ground deformations give rise to characteristic features of landslide morphology.

Similarly, in the latter stages of landslide movement, the rates of deformation must slow appreciably before the slide mass comes to rest. However, since a large first-time failure accompanied by significant displacement at relatively high rates of movement is likely to "overshoot", it is far more probable that the final stages of movement come to an abrupt halt. Experience of the failures at the Carsington Dam and in the Selborne Controlled Slope Failure Experiment (**Cooper *et al.,*** 1998), suggest that the general level of total stress along the slip surface decreases during the slide, and as a result, a decrease in pore pressure also occurs, which has an additional braking effect on the slide.

7.4.3 Slopes moving under oscillating ground water conditions insufficient to provoke overall failure

In a slab-type landslide (*Figure 7.4*) the water levels rise locally. If this local area was the entire landslide, then it would certainly move (*Figure 7.5*). However, it is prevented from moving by the restraint of the surrounding ground. If the area were large enough, or the restraint small enough, then this part of the mass might "break out" of the overall slide. Such behaviour is not unknown, with small parts of a widespread solifluction sheet, for example, being locally reactivated (*e.g.* **Chandler, 1970**). The toe breakout zone of a particularly shallow slab type landslide may be visible as a ripple in the ground surface. Where the local destabilisation is insufficient to create a separate slip movement, the surrounding ground will be in tension (upslope), compression (downslope) and in shear to either side. The changed stress conditions cause deformations, a proportion of which are irreversible when the locally elevated ground water levels revert to their "normal" conditions. A groundwater body advancing through a landslide would cause a ripple of movement through the slide mass.

Taken over sufficient individual events (each of which could be single rainstorm events at the lowest end of the scale, or complete wet seasons at the other), the cumulative displacements of the slide mass would equate to slow, and possibly intermittent movement downslope. The essence of this is a general, irreversible, slow and small-magnitude downslope movement under conditions of a factor of safety greater than one. A more detailed treatment of this is given by **Piccarelli *et al.*, 1995**.

Picarrelli's model is applied, primarily to elongate or lobate mudslides, and correctly models the differential movements which can propagate downslope. Conceptually, it is even better with the lateral freedom of sheet or slab-type slides. *Figure 7.6* shows (top) the conventional assumptions of the infinite slope method (**Haefeli, 1948; Skempton & Delory, 1957**). Every real failure of a slab, sheet or lobate type must have a toe breakout zone, and a head zone (centre). The inclinations of slip surfaces in these two zones should correspond approximately to the passive and active wedges of earth-pressure theory. For an extensively sheared mass, sufficient numbers of pre-existing shears may be present for the slide surfaces in these wedges to function at residual strength. In some cases, the active and passive zones may be in soil that is not sufficiently sheared, and alternate conditions may apply. Failing such a composite failure surface is clearly more difficult than to fail the "infinite slope", as the additional resistance in the toe zone (passive wedge) clearly exceeds the additional thrust from the head zone (active wedge). This "finite" slope method is discussed further in *Chapter 9*.

Some simple computations have been carried out by way of demonstration (*Figure 7.7*). Assuming residual strength throughout, the effects of toe and head zones on an infinite slope mechanism have been calculated for a typical combination of parameters. One might do the same calculations for a slide of constant depth, but variable length. The influence of the toe zone in particular is greatest as the length to depth (L/d) ratio shortens. There is not a critical length, and the influence of toe breakout resistance diminishes as L/d lengthens. A typical reason why a finite length of slide debris might be reactivated is shown in *Figure 7. 6* (bottom). Here, irregularities in the ground surface (possibly the results of past slide activity) act as the focus for infiltration, locally raising ground water tables. A toe breakout zone is indicated in one such depression, since not only are the pore water pressures elevated, but also the passive resistance is critically dependent on soil depth.

After a failure on the composite slip surface shown in *Figure 7. 6* (bottom) or *Figure 7.5*, the geometry of the toe zone is locally changed appreciably. This has the effect of discontinuing movement on that slip, but changes the conditions so that another part of the slide mass may then become critical.

As well as irregular rises in ground water tables, the concept may be applied to the infiltration of water irregularly from the surface into a desiccated debris mantle, similarly indicating the propensity of local areas to become destabilized.

It is a consequence of the Picarrelli model that every incident which gives rise to a local reduction in stability insufficient to permit breakout causes strain in the slip mass which accumulates to give slow movement. The Author considers that this model describes the slow movements and occasional failures of slab type landslides in Britain better than a rate-dependent model.

7.5 Implications under changing climatic conditions

The various static models all imply a close relationship between the onset of movement, and rising groundwater levels. Effects such as those of soil chemistry may alter the basic soil shear strength parameters, but not to the extent which significant rises in groundwater level may make, especially if the loss of soil suction in the unsaturated zone is involved.

Dynamic models predict different behaviours as groundwater levels continue to rise after the initial reactivation. Rheological or viscosity models predict progressive increases in the rates and magnitudes of movements, with progressively larger proportions of the pre-existing slide populations becoming involved. As a result, the implication for the performance of low angled slopes in the UK containing pre-existing slip surfaces, if this model is applied, is that there would be a tendency for the number, size and magnitude of movements progressively to increase as precipitation increased. Without an extension to the theory, there does not appear to be a threshold at which behaviour changes significantly.

However, application of the Picarelli model implies that the slow movements occur at Factors of Safety in excess of one. Further worsening of groundwater levels would permit the $F=1$ case to be passed. This is a genuine threshold, beyond which the character of slide movements alters dramatically, and is in line with many experiences in Britain. Shallow mantles of slide debris on low-angled slopes show a propensity to react to comparatively short term intense precipitation events, although the preparation of such slopes by longer term heavier-than-average precipitation can be important. A rise in frequency of this type of movement, would occur both with a shift to more seasonal rainfall patterns as well as an overall increase in precipitation.

The case where $(D - R)$ is negligible is influenced by rising groundwater, dependent on the scale of the slide. Slide masses usually have to be large for the negligible $(D - R)$ condition to apply, and are less sensitive to rising groundwater levels under relatively small changes in precipitation. Again, however, a progressive increase in the rates and magnitudes of movement, with an increase in the proportion of the slide population involved, is likely to be the result of worsening groundwater levels. This category of movement is likely to be more affected by overall increases in precipitation than by a change in distribution of rainfall throughout the year, although increased storminess, with possible effects of beach scour in the case of coastal landslides, may not be neglected.

7.6 Summary

The project commenced with the explicit understanding that the mechanics of the landslides needed to be explained, because the norm for slopes containing pre-existing shears was small creep type movement, and the movements of the Roughs were unusual both in terms of rate and magnitude. As the project unfolded, it because evident that the Roughs movements were not unusual, nor indeed, were they particularly large in magnitude or rate. The principal error was the assumption of what constituted the norm.

Consideration of first-time movements shows that in most failures involving stiff clays, various braking effects are present. These involve dilatancy, and gross geometric changes, some of which involve undrained pore pressure effects. At the scale of failure possible in the UK, such failures are likely to be comparatively sedate if they involve only sliding.

There is nothing about slopes containing pre-existing shears that compels their movements to be exclusively of the creep kind. Indeed, Picarelli's model (and others, discussed above) suggests that such movements are the result of differential ground strains at overall factors of safety greater than one. For the Roughs, a factor of safety of one was approached. Changes in the landslide morphology (largely through increased toe resistance as the slide system moved) acted as a brake.

In any case, the onset of movement is best analysed by a static method, since the deformations and deformation rates to that point are negligible.

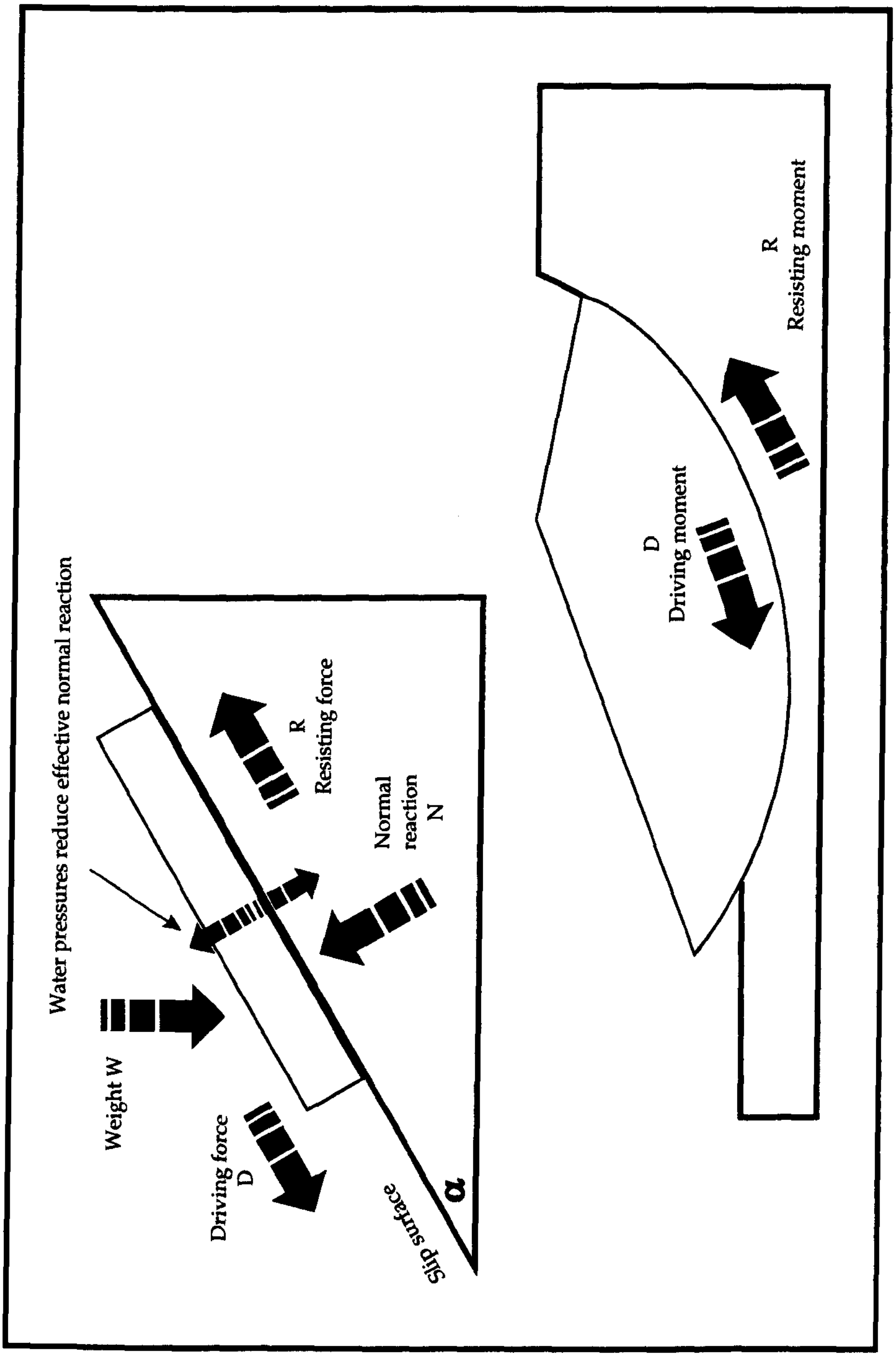


Figure 7.1. Summary of forces acting on a simplified landslide.

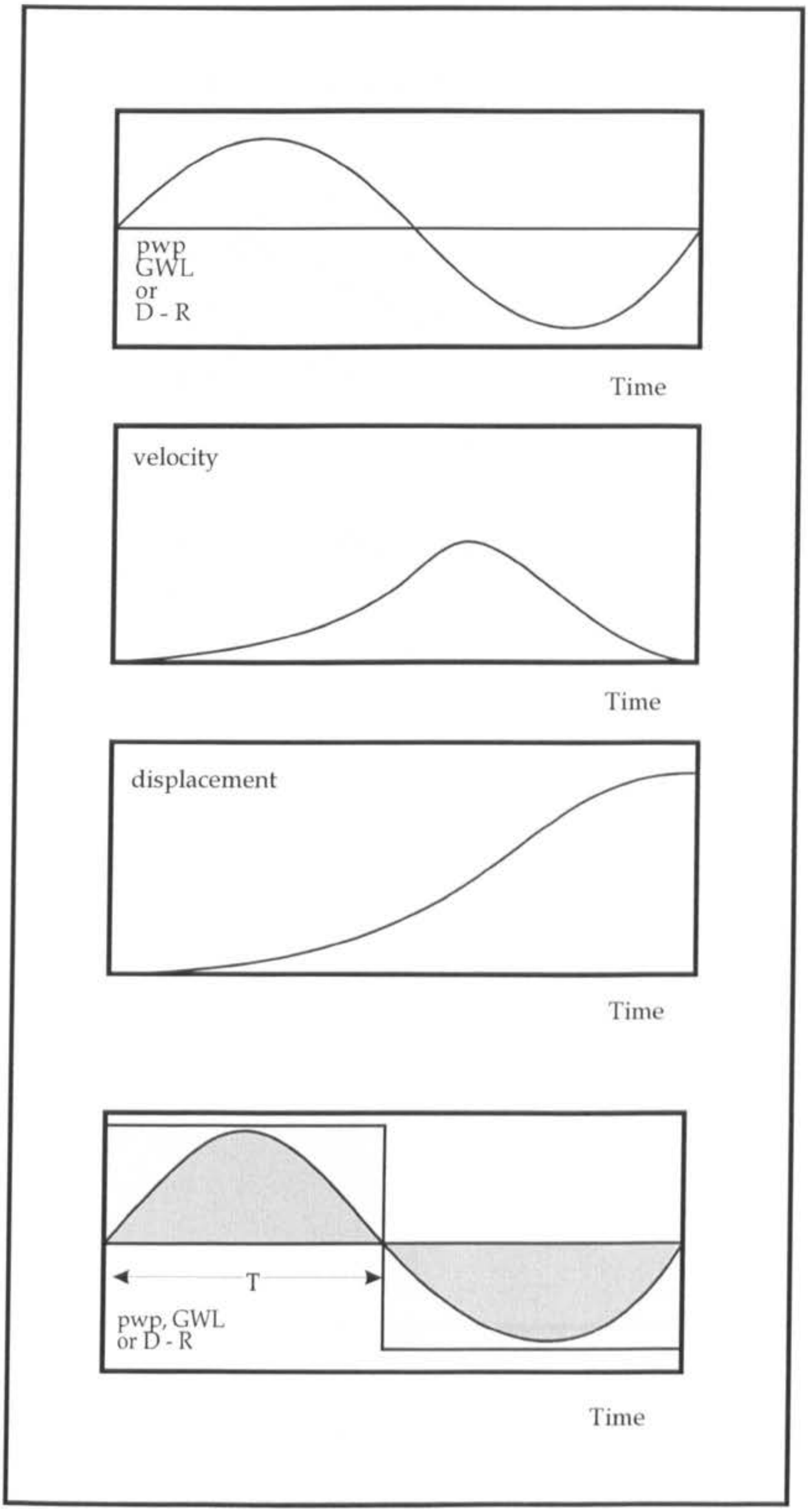


Figure 7.2. Diagrammatic representation of velocity and travel profiles as a function of the time for which a destabilized situation exists.

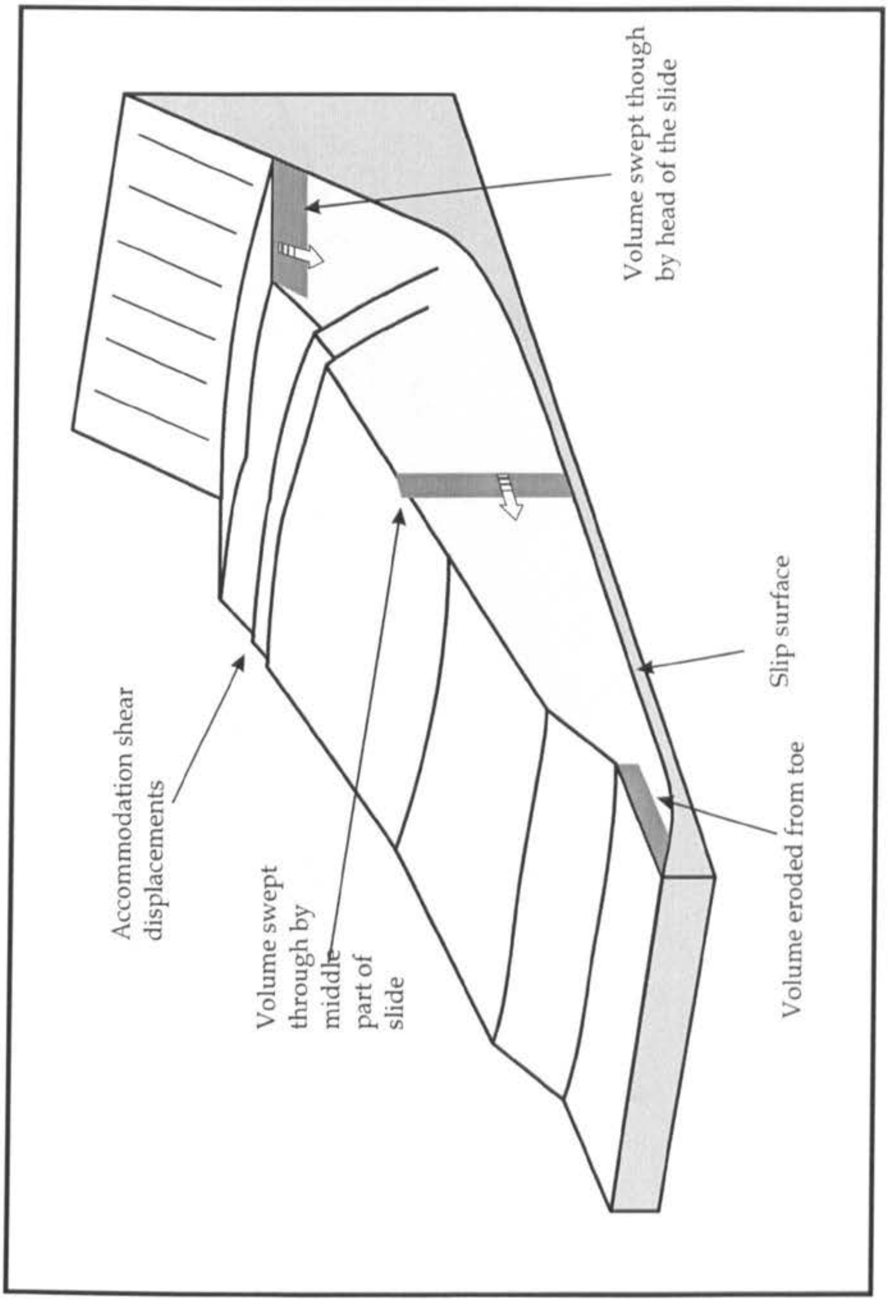


Figure 7.3. Coastal landslide subject to foreshore toe erosion. The movements of different parts of the landslide following a removal of a small part of the toe may be in various directions, and of varying amounts, but they return the slide to perfect equilibrium.

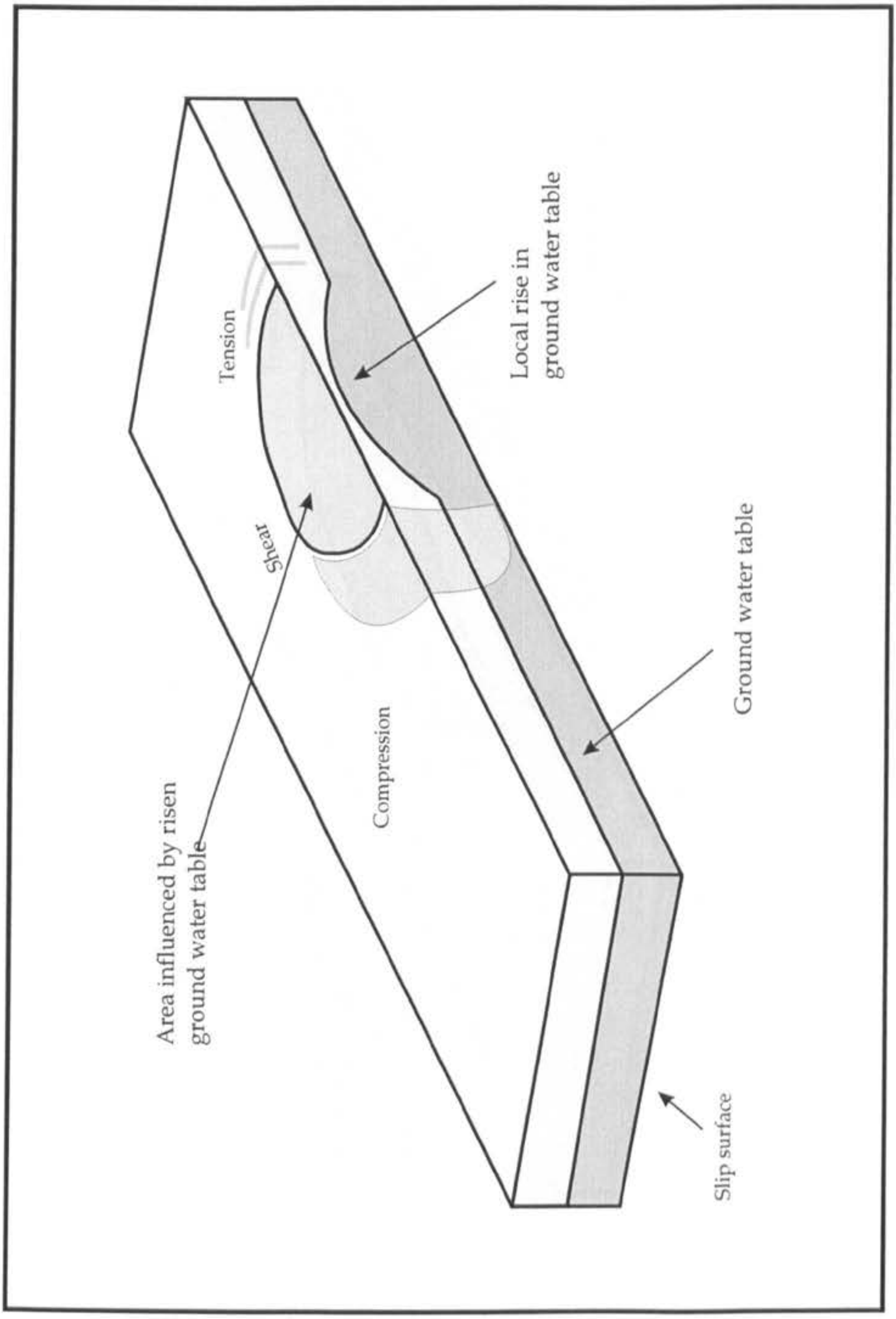


Figure 7.4. In a slab-type landslide mass, a finite area may have a local Factor of Safety less than 1, but be unable to "break out", and as a result, causes deflections in its locality.

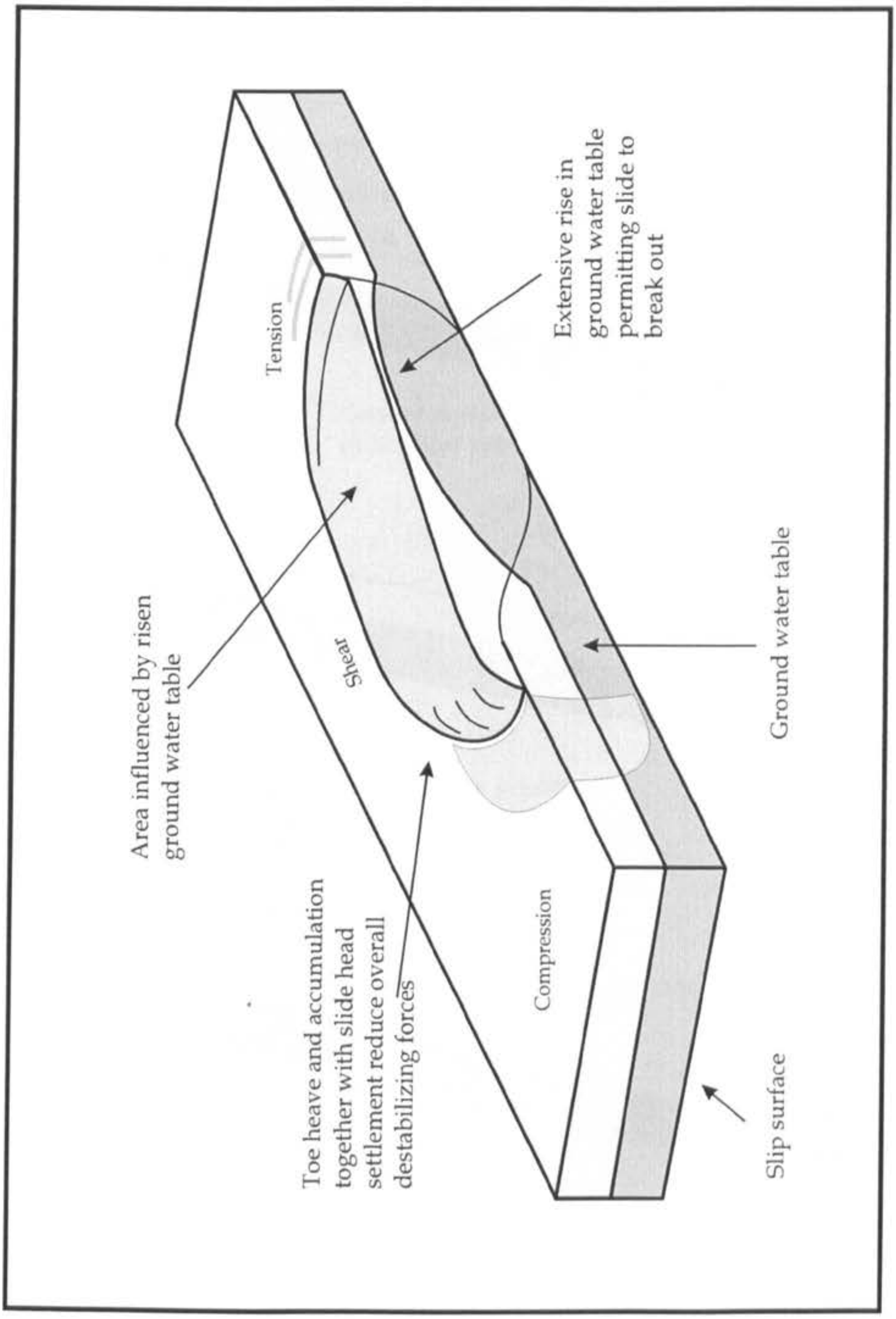


Figure 7.5. In this slab-type landslide mass, a large enough area may have a local Factor of Safety less than 1, to be able to "break out", and as a result, a "new" landslide is seen.

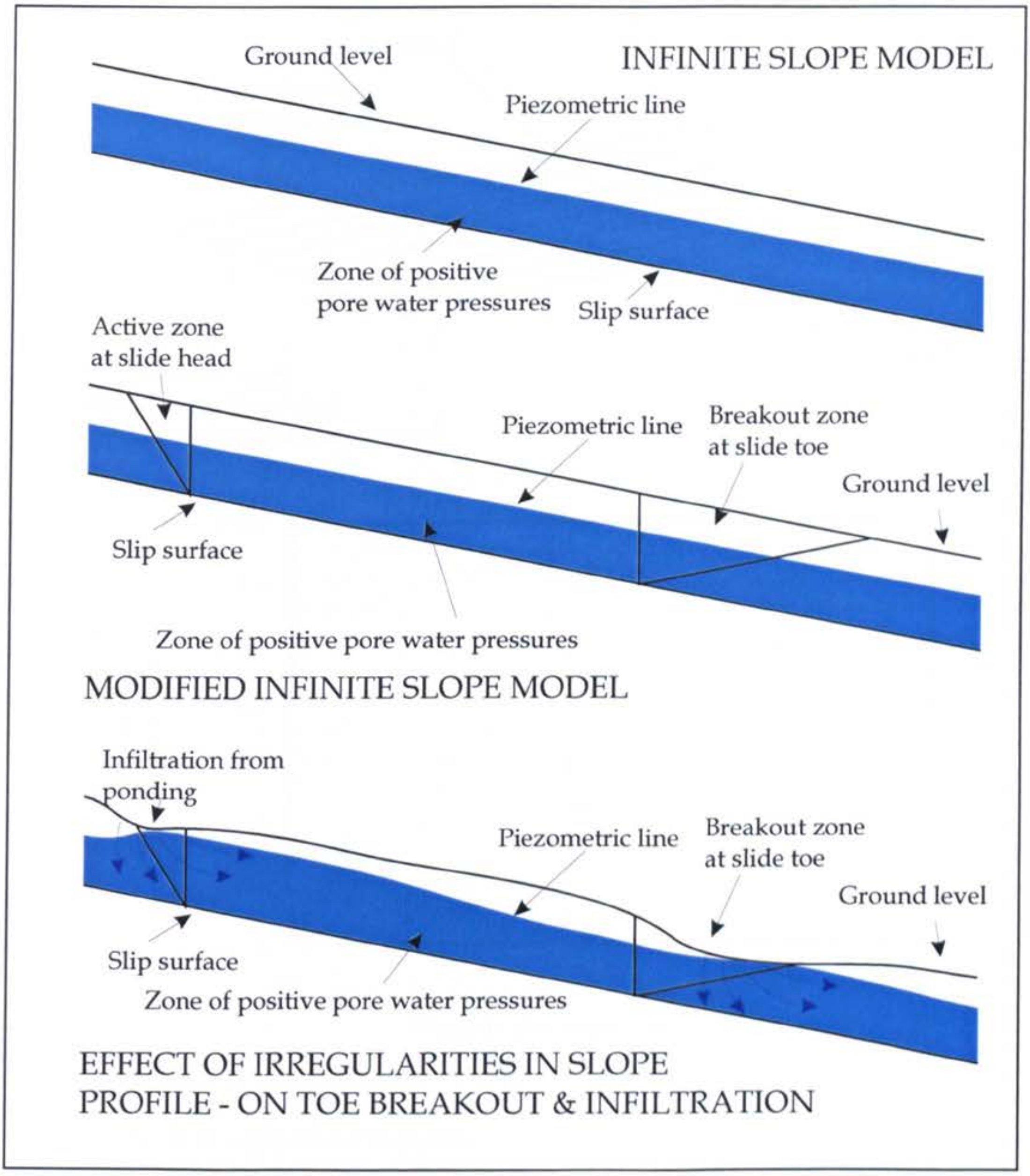


Figure 7.6. Haefeli's (1948) Infinite Slope model (top) assumes that conditions are identical throughout the length of the slope. It expressly neglects (centre) the head wedge (actually has only small impact) and the toe break-out wedge (significant). The occurrence from time to time of small movements creates slope profile irregularities which account, in part, for non-uniform ground water tables during and immediately after rainfall (lower).

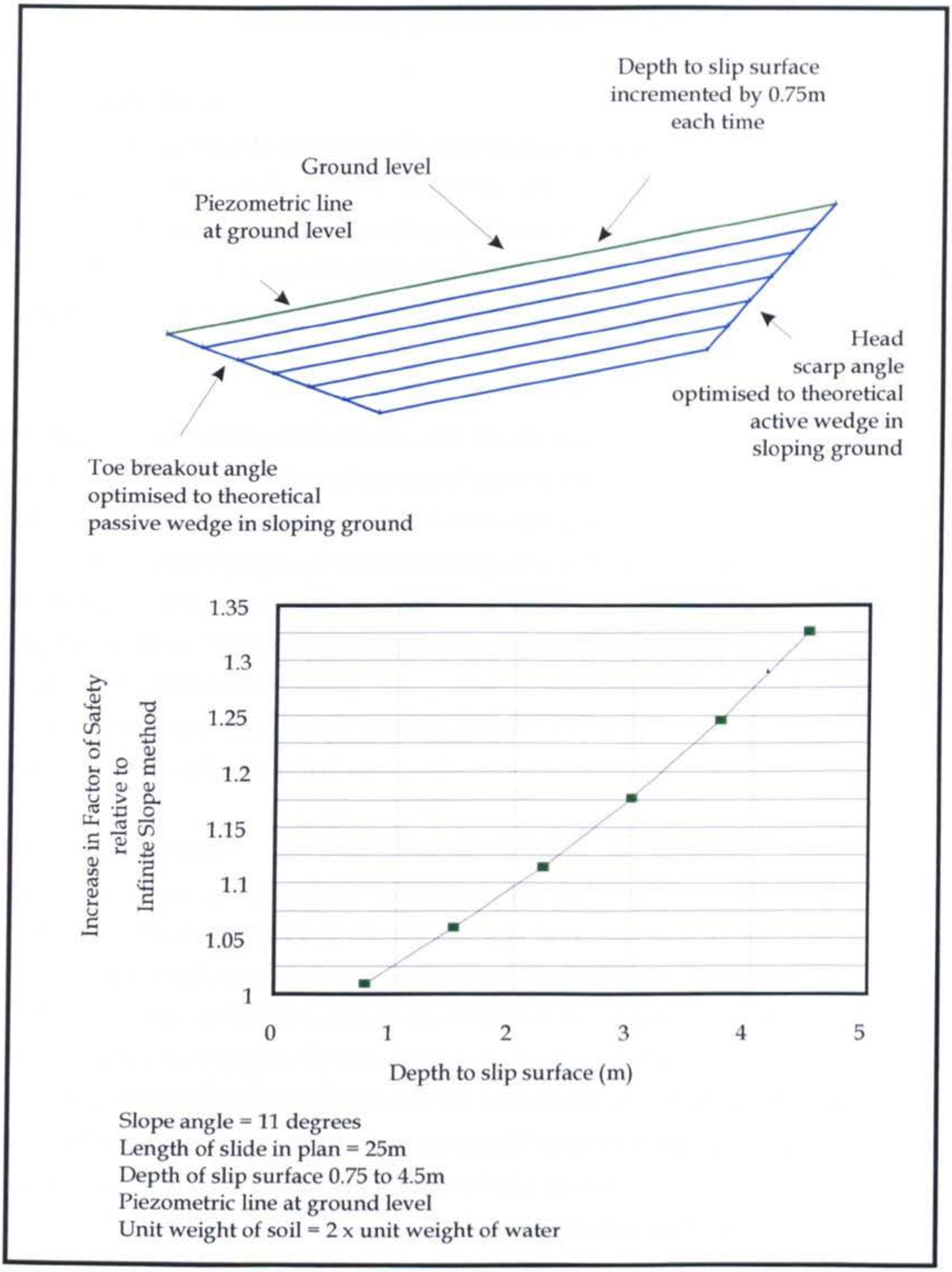


Figure 7.7. Typical case of effect of toe breakout wedge on the stability of what is otherwise the "Infinite Slope" case.

Climate analysis and slope hydrogeology

8.1 Introduction

Climate, or the prevailing weather conditions of an area, is one of the major contributory factors to landsliding, second only, probably, to gravity. Statistics, it has been pointed out, can prove anything, but there is an unarguable correlation between climate and the onset of landsliding, as is demonstrated by the European Community sponsored work completed by **Casale et al.**, 1994, **Dikau et al.**, 1996 and **Ibsen**, 1994. It has been shown that landslides do, in fact, occur after wet year series.

Work published by **Ibsen and Brunsden**, 1996, developed a model to examine the effect of rainfall on an existing landslide complex, on the Isle of Wight. Rainfall records, which were obtained from the nearest weather station, were converted into *effective precipitation* by subtracting the evapotranspiration. Running averages over increasing periods of time were then plotted. Eventually, peaks appeared in these running averages at the time of known land movements. **Bromhead, Hopper and Ibsen**, 1998 then went on to examine the rainfall records from the area of The Roughs in a similar manner, but developing the evapotranspirational model dramatically. Yearly rainfall accumulations were also plotted. Rainfall records from both Cherry Gardens, Folkestone (Appendix 8.1) and Sandling Park were used.

8.2 Hydrogeology

Seepage erosion or "spring sapping" occurs when flowing water removes the "fines" from a soil mass. This will eventually cause the soil structure to collapse. One of the most susceptible combinations of geological units to seepage erosion, is the presence of a permeable layer, such as sand, over an impermeable one, such as clay. Effectively, this acts as storage for water, providing the piezometric conditions for the area. If water is added, as rainfall, faster than it can escape, as springs etc., then the piezometric levels rise. This is the situation on The Roughs as well as in many areas of lowland Britain. *Figure 8.1* depicts a typical hillslope hydrological cycle, showing how water is stored in aquifers. The strength of a soil at a given time is determined by its effective stress, which is itself determined by the water pressure in the ground. It is therefore fundamental in determining the stability of a slope, to first understand the hydrological cycle and the piezometric levels present in that slope.

The lithology of the slope determines how the water behaves in the ground. Water tables (the level in the ground beneath which all voids are water-filled) are formed in slopes, such as The Roughs, where an impermeable layer or aquiclude, such as Atherfield Clay is underlying a permeable layer or aquifer, such as the Hythe Beds. Where the impermeable layer outcrops on an escarpment, a springline may be formed, as an aquifer is discharged as shown in *Figure 8.1*. This may indicate high pore water pressures which are detrimental to the slope stability.

Pore pressures and water forces affect the stability of slopes in three ways.

- Tension cracks
- Submerged forces
- Pore pressures on slip surfaces

After a particularly wet period, the aquifer may be at or near capacity. It will also be continually discharging, but if a heavy storm or a few days of continuous rain occurs, then the aquifer will be filling faster than it can empty and the piezometric levels will quickly rise. When these levels reach a slip surface then it is possible that a landslide may occur. Obviously, a shallow landslide will be more easily triggered than a deep-seated one as less rain is needed to increase the water table to a critical level. This explains why The Roughs reacts more quickly to rainfall than Folkestone Warren or Ventnor.

8.2.1 Human intervention of the hydrological cycle

Figure 8.1 shows some ways in which human activity affects the hillslope hydrological cycle: irrigation, wells and wastewater disposal. Add to this list: drainage tunnels, roads and paved areas (more run-off, less infiltration), crop and tree-planting (and felling), reservoir building...only the hydrological cycles in the remotest areas in the UK are apparently unaffected by human activity. According to commonly accepted beliefs, pollution and greenhouse gases pumped into the atmosphere are affecting the climate globally, which would imply that nowhere on this planet is untouched. Not all of the changes are detrimental to slope stability though, as demonstrated by the drainage works at Sandgate.

8.2.2 Post-movement hydrological changes

Once a slope has moved, the hydrology of the system changes. This is due to a variety of reasons, such as:

- land drainage systems may be damaged
- the position of natural stream lines may be effected
- the composition of the soil may be altered-the alignment of clay platelets etc.

- soil particle packing and porosity may be changed
- damage to plant and tree roots
- alteration of surface profile
- changes in inclination and roughness of soil surface
- ponding

Thus, after movement, the slope will not react in the same way to triggers such as rainfall.

- leaking drains may concentrate water in one area
- clay once at peak strength may now be at residual strength
- sheared loosely pack soil may absorb more water
- soil structure not held together with roots
- more surface area to absorb rainfall
- new mudslide caused by water from ponds

Knowing how the newly formed slope will behave is, therefore, difficult.

8.2.3 Comparison of evapotranspiration equations

According to Thornthwaite in 1948, potential evapotranspiration *"is the rate at which evapotranspiration would occur from a large area completely and uniformly covered with growing vegetation which has access to an unlimited supply of water and without advection or heat effect."* This was redefined by Penman in 1956 as *"the amount of water transpired by a short green crop, completely shading the ground, of uniform height and never short of water."*

Lawrence, 1994, gives the methods used in hydrologic studies to classify potential evapotranspiration which can be classified into four groups:

- temperature-based, use only temperature and occasionally day length
- radiation based, net radiation and air temperature
- combination, net radiation, air temperature, wind speed and relative humidity
- pan, pan (free-water surface) evaporation with or without modifications

The method used by **Bromhead et al, 1998,** the Thornthwaite method, is temperature based. This has the advantage of simplicity and availability of data but makes no allowance of, for example:

- overland water flow

- land drainage
- vegetation
- air-flow (significant on The Roughs which are affected by coastal breezes)
- water being drained onto the land from roads and houses etc.

The resulting index is therefore not considered as efficient as the Penman formula, which is expressed in terms of sunshine duration, mean air temperature, mean air humidity and mean wind speed. In fact the modified Penman equation, the Penman-Monteith equation, is regarded as the best estimator of evapotranspiration for all vegetated surfaces and takes into account canopy conductance. Unfortunately, most of these components have only recently been monitored and, therefore, do not extend over a significant time for this study. However, this is a study of landslides and not hydrology and the figures resulting from the use of the Thornthwaite series are perfectly adequate for this study.

8.3 Cyclic variations in the factor of safety

The diagram *Figure 8.2* shows the concept of the Factor of Safety of a slope varying seasonally, with one complete oscillation per year. It will be seen later that this is an oversimplification. In the wettest season of the year, normally late winter /early spring, the Factor of Safety reaches its lowest value, and in the driest season, normally late summer/early autumn, it reaches its highest value.

Consider first what the possible range of safety factors is. For the infinite slope model as shown in *Figure 9.1*, the equation for the Factor of Safety is:

$$F = \frac{c' + (\gamma z - \gamma_w h_w)\cos^2\alpha \tan\varphi'}{\gamma z \sin\alpha \cos\alpha} \qquad \text{Eq. 8.1}$$

Simplifying the above equation to a case where $c'=0$, it is found that:

$$F = \frac{(\gamma z - \gamma_w h_w)\tan\varphi'}{\gamma z \tan\alpha} \qquad \text{Eq. 8.2}$$

Where c'= soil cohesion

F= factor of safety

h_w=ground water level above slip surface

z= depth of slip surface

α= slope angle

λ_w= unit weight of water

λ= unit weight of soil

φ'=friction angle

During the wettest season the groundwater level might be at ground level (h_w=z) as a worst possible case and substituting this into *Equation 8.2* gives:

$$F = \frac{(\gamma - \gamma_w)}{\gamma} \frac{\tan\varphi'}{\tan\alpha} \qquad \text{Eq. 8.3}$$

For the majority of clay soils, the bulk unit weight of soil is approximately twice that of water, so that *Equation 8.3* reduces approximately to:

$$F = \frac{1}{2} \frac{\tan\varphi'}{\tan\alpha} \qquad \text{Eq. 8.4}$$

During the summer the slope might dry out (h_w=0) as a best case. This reduces *Equation 8.2* to:

$$F = \frac{\tan\varphi'}{\tan\alpha} \qquad \text{Eq. 8.5}$$

Hence, the Factor of safety in the best case is twice that for the worst case. This makes the range of F become $A \leq F \leq 2A$. A somewhat larger range is possible if the groundwater conditions become artesian due to fissure flow (or some other mechanisms in the slope) or if the soil properties change during complete desiccation (improbable at depths of c. 4 m in the UK).

As *Figure 8.2* is drawn, there is an approximate periodicity, or a return period, of 5 years. This gives a periodic low Factor of Safety which is just low enough to cause some small ground movements, but not to permit wholescale sliding. It is inconceivable that this condition would return exactly on each 5th year, but the diagram is a concept sketch. Within the first 5 year cycle, a run of wet years is shown. This run makes the end-of-year low point become progressively lower. A converse situation would apply with a run of dry years. Within the run, the fluctuation is held constant. There is, however, the prospect of seasonal changes in rainfall, shown in the second 5 year cycle as a wet summer and a wet winter.

The natural fluctuation of weather has a rational basis, being driven, inter alia by the 11.5-year sunspot cycle, and other rhythmic variations such as el Niño in some parts of the world.

To model reality, putting in fluctuations of Factor of Safety on a less than yearly cycle would be necessary: To show a longer timescale, in which the Factor of Safety permits significant movement reached as either a result of a very short-period extreme climatic event (or short-term run of events) or as the culmination of the periodic oscillation of not particularly unusual events. Experience suggests that the combination of the two causes failure to occur.

8.4 Statistical rainfall investigation for The Roughs

For the statistical examination of The Roughs landslides, on which the following section is largely based, **Bromhead et al,** 1998, first investigated the incidence of landsliding in the coastal region from Hythe to St. Margaret's Bay. The records were probably incomplete, the figures being much lower than expected, at one to six incidents per decade. The information was not comprehensive enough for a statistical approach and therefore emphasis was switched to the comprehensive climactical series available, since this approach had previously been so successful on the Isle of Wight, **Ibsen and Brunsden,** 1996.

8.4.1 Rainfall series

Appendix 8.1 gives the yearly rainfall figures, from 1868 to 1996, recorded at Cherry Garden, Folkestone, Kent, which is approximately 8 km from The Roughs. In conjunction with the long running temperature series from the National Central Meteorological Office, these figures were used to calculate the effective rainfall series which is the rain available to an area. The Thornthwaite potential evapo-transpiration method was used to convert these figures into an effective rainfall series. This is a simplified formula based on only the temperature in °C, to calculate the potential water loss from a surface and does not take into account vegetation. The formula used to calculate the potential evapotranspiration is:

$$PE = 1.6\left(\frac{10T_a}{I}\right)^2 \qquad \text{Eq. 8.6}$$

T_a = the mean monthly air temperature (°C)

I = the annual heat index which is equal to *Equation 8.7*:

$$I = S_{i=1}^{12}\left(\frac{T_{ai}}{5}\right)^{1.5} \qquad \text{Eq.8.7}$$

and where a = a complex function of I given in *Equation 8.8*:

$$a = 0.49 + 0.0179\,I + 0.0000771\,I^2 + 0.000000675\,I^3 \qquad \text{Eq. 8.8}$$

Subtracting these figures from the Folkestone annual rainfall series produces a moisture balance index or effective rainfall index.

Figure 8.3(a) shows this calculated effective rainfall series plotted with the regression line indicated. This shows that the effective rainfall, is increasing over time. Deducing anything from these figures as plotted is difficult, so the method of moving averages was used to identify any patterns in the rainfall series. Symmetric moving averages were calculated over periods of 3, 5, 7, 9 and 11 years. The most successful of these was the 9 year series, shown in *Figure 8.3(b)*, which showed an apparent 12-20 year wet weather cycle. At Folkestone, there were large land movements in 1877, 1896 and 1915 which correspond extremely well with the peaks in the 9 year series.

*Figure 8.4*a shows the cumulative years with above average rainfall and *Figure 8.4b*, the cumulative effective rainfall departure from the mean using figures from Folkestone. The two wettest periods are shown to be 1910-14 and the 1920's.

8.4.2 Monthly rainfall analysis

Rainfall records from Sandling Park, which is only about 2.5 km from The Roughs, also exist from 1900 but are only complete from 1967 onwards. These records were used to calculate and plot accumulative rainfall series for 4, 6 and twelve months shown in *Figure 8.5*. The reactivation of The Roughs in 1988 is marked with a clear peak on the graph, as is the development of the eastern mudslide toe in 1994/5. This strongly suggests that movements on The Roughs are triggered by extended periods of rain rather than a few isolated days. Also visible are peaks in 1976, 1978 and 1983/84.

8.4.3 Yearly rainfall analysis

The rainfall figures from Sandling Park were used to plot graphs of rainfall accumulations over periods ranging from 1 year to 15 years, *Figure 8.6*. The most obvious peak, even on the 15 year accumulation plot, is that in 1987. This preceded the reactivation early in 1988. As with the monthly analysis, there is also a peak in 1994 before the development of the eastern mudslide toe.

Moving averages, up to an 11 year cycle, were also plotted, *Figure 8.7*. The nine year plot gave the clearest picture, having peaks every 5 to 6 years. At Folkestone, the nine year plot was also the most appropriate but showed peaks, or wet weather periods, every 12-20 years. The rainfall figures at Folkestone, however, are complete for a much longer period.

8.5 Brief review of literature

Anderson, 1990, used a slightly different approach to find what effect, if any, heavy rainfall had on the landslide reactivation in 1988. He used weekly rainfall data from the Meteorological Office for the years 1986, 1987 and 1988 and daily data from Station 305733 at Dymchurch. He then considered the influences of the soil moisture deficit (SMD) and effective rainfall (EF) on groundwater conditions. When the underlying soil is at field capacity, the formula used by Anderson to calculate the effective rainfall is:

$$EF = AR - PE$$

where AR represents the actual rainfall and PE represents the potential evapotranspiration.

Before the first instability, on the 15th January, 1988, there was a thirteen day period of substantial rainfall. For 70% for this time, the soil was at field capacity which resulted in an accumulative effective rainfall of 54.4 mm over the wet period. A ten day period of rain preceded the second stage of the instability. The accumulative effective rainfall of 68.2 mm and the soil was at field capacity for 90% of this time.

He argues that the Stage II slide (as depicted in *Figure 3.11*) did not take place a couple of days after the maximum effective rainfall (January the 27th) during that winter. This was partly because of the seasonal build up of pore-water pressures which negated the storm response, but mainly because the Stage I slide unloaded the front of the rotated blocks over a period of time. Only by the beginning of February were blocks destabilised enough for the storm-triggered Stage II slide to occur.

Bowdler, 1972, examines the occurrence of days of abnormally high rainfall in conjunction with spring tides in his examination of the landslides at Sandgate. This happened on three occasions, in October, 1939, October, 1949 and October, 1966. He questions why, the wet period in 1966 resulted in a landslide but there were no reports of ground movements in 1939 or 1949. He also includes a monthly rainfall chart covering the years 1967-1971, given in *Appendix 8.2*. This chart shows that the total rainfall in the consecutive years 1967-1970, inclusive, was a lot higher than average and some moment might be expected in the winter of 1970 because of this. Bowdler does, in fact, report increased movement in the winter of 1970/71.

In his examination of the Encombe ground movements, **Palmer** (1991) looks at the relationship between 6 and 12 monthly rainfall accumulations and the number of recorded ground movements. He notes that rainfall of 400 mm and 800 mm a year for 6 and 12 monthly accumulations respectively, appears to correlate with principal movements in the winter and spring of that year. High rainfall and the resulting land movements occur in 1961, 1967, 1975, 1977 and to a lesser extent 1983 and 1988. It is stressed that although the rainfall accumulations preceding the landslide of 1893 were high (510 mm and 875 mm respectively), these levels have been exceeded since and no sudden movements have occurred. He therefore attributes the landslide of 1893 not only to rainfall, but to several other factors such as beach levels and tidal variation. Obviously, the remedial measures installed since the landslide have also contributed to the relative stability of the area.

8.6 Discussion

It is interesting to compare the dates of land movements both at Hythe (1976, 1978 and 1983/84) and Sandgate (1961, 1967, 1975, 1977, 1983 and 1988). There is an obvious correlation between the two which adds weight to the 'landslides are triggered by an extended period of wet weather' theory.

The question has to be asked why there were no incidents in Hythe in the 1960's. Maybe there were, but they remained unnoticed and/or unreported. One other possibility is that the slight variation in geology and geomorphology between Hythe and Sandgate accounts for the sites varying response to wet weather. The third affirms both Palmer's and Bowdler's theory about the relevance of the tide in Sandgate, since both Hythe and Sandgate are subjected to the same amount of rainfall, but only Sandgate is affected to any degree, by the tide.

There are a series of peaks, in the 9 year moving average (*Figure 8.3*), at Folkestone, which did not precede a large movement. It may be that slopes can cope with an extended period of weather if they have time to discharge. However, when a storm event follows the period of wet weather, then the extra water 'tips the balance', as the slope cannot discharge at a fast enough rate and a landslide is triggered. It is possible that even extended periods of wet weather will not trigger a landslide if it is not followed by a shorter period of extreme rainfall.

Ibsen and Brunsden, 1996 have performed a similar analysis to that done on The Roughs, by **Bromhead et al,** 1998, at Ventnor on the Isle of Wight. Here, it was also found that the nine year moving average best identified the cyclic patterns. The cycles were not as pronounce as for The Roughs and the wet-weather cycle difficult to determine. Among the periods of increased moisture balance highlighted were 1875-83, 1923-39, 1948-71 and 1976-83. These correspond exactly to the periods of increased landslide activity shown in the annual landslide index for Ventnor. *Figure 8.8* shows the number of years with above average rainfall and the commutative effective rainfall departure from the mean for Ventnor. It is interesting to compare these graphs with those for The Roughs, *Figure 8.4*. Some similarities are immediately apparent. Looking at the top graph in each case, it can be seen that there was a very wet period in the 1920's in both areas. The general shapes of the curve in the two lower graphs are similar, rising from circa 1920. Overall, however, there have been more extended wet periods on the Isle of Wight than on The Roughs and the number of land movements reflects this fact. For the big, deep, slips on the Isle of Wight, longer periods of significant rainfall we involved than for the South Kent area.

To examine the relationship between annual rainfall and annual moisture balance, with the incidence of landsliding, further, **Ibsen and Brunsden,** 1996, looked at the number of years that each rainfall class and moisture balance class occurred. They then calculated the percentage of these years in which a landslide developed. Bar charts have been compiled to illustrate this data and are shown in *Figure 8.9*. Both charts show an upward trend indicating the link between increased annual rainfall and annual moisture balance and landsliding.

Ibsen and Brunsden, 1996, calculate, using figures from the escape model of the Climate Research Unit at East Anglia, that the coastal landslide frequency will rise by 5 - 10 events per year above the present mean.

8.7 Conclusions

It cannot be known whether the general trend of increasing effective rainfall noted in weather records will continue or if it will be reversed in the near future. Obviously, if the precipitation levels do increase, as predicted, the return period for land movements on The Roughs would be reduced. More worryingly, this also applies to numerous other built up areas on 'stable' landslides locally and throughout the South of England.

As a general principal it can be said that increased rain leads to increased landslide activity. The larger the landside, the longer the wet-weather period needed to begin it. As discussed in *Section 8.2*, a particularly heavy downpour towards the end of long run of wet weather can have a significant effect.

Although correlations are good in geographical terms, no-one has yet produced a good model for a single site. To complicate matters further, few of the landslide areas are free of the effect of human activity, so there is a difficulty in correlating the 19th Century and 20th Century (especially the later 20th Century) rainfall and subsequent events.

One flaw in statistical approaches, is that a wet year may initiate a major landslide resulting in a relatively stable slope. Sequential wet years may be insufficient to reactivate the slope, statistically showing that wet years do not cause landslides. Long term studies are often necessary to calculate the return period of landslides.

The Roughs landslide complex itself can be seen to, even using these limited statistical techniques, be very responsive to climatic factors during the recent past. Since, each recent event may be coupled with a wet weather period of at least 12 months. The system of landslides at The Roughs appears likely to have been put into a less stable state by the rainfall in 1988, and the later mudslide at the east end of the slide complex owes its formation to changed hydrology of the slide and the rainfall leading up to it. According to **Bromhead et al**, 1998, almost 75% of the landslide events recorded in the geographical area surrounding The Roughs in South-East England correlate with a year or years of above average effective rainfall. This is corroborated by **Bowdler**, 1972 and **Palmer**, 1991 in their studies of the land movements in Sandgate, as demonstrated in *Section* 8.5.

8.8 Summary

This chapter introduces the principals of hydrogeology, emphasising its importance to slope stability. Also discussed is the effect of human interference on the hydrological cycle. The concept of cyclic variations in the factor of safety of a slope is then considered.

The coincidence of land movements and wet weather cycles in is looked at in three locations: Ventnor, Folkestone and The Roughs. In each case, the connection between the two events is clear. However, some wet weather sequences were found not to result in land movements and this was attributed to one of two reasons:

:

i. The wet weather sequence was not followed by a 'storm' event.

ii. The landslide had recently moved and was in a stable state.

Since the wet weather sequences and landslides are inextricably linked, and the statistics show that the climate is becoming wetter, then the incidences of landsliding will also increase.

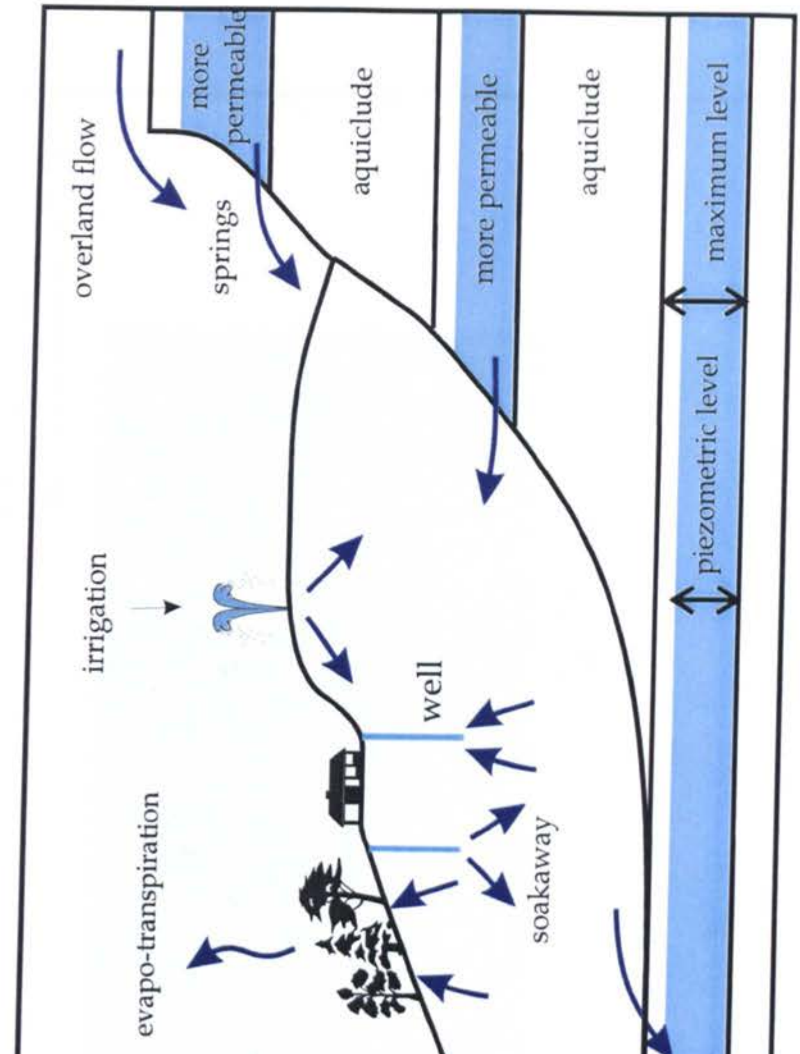


Figure 8.1. Diagram of typical hillslope hydrological cycle.

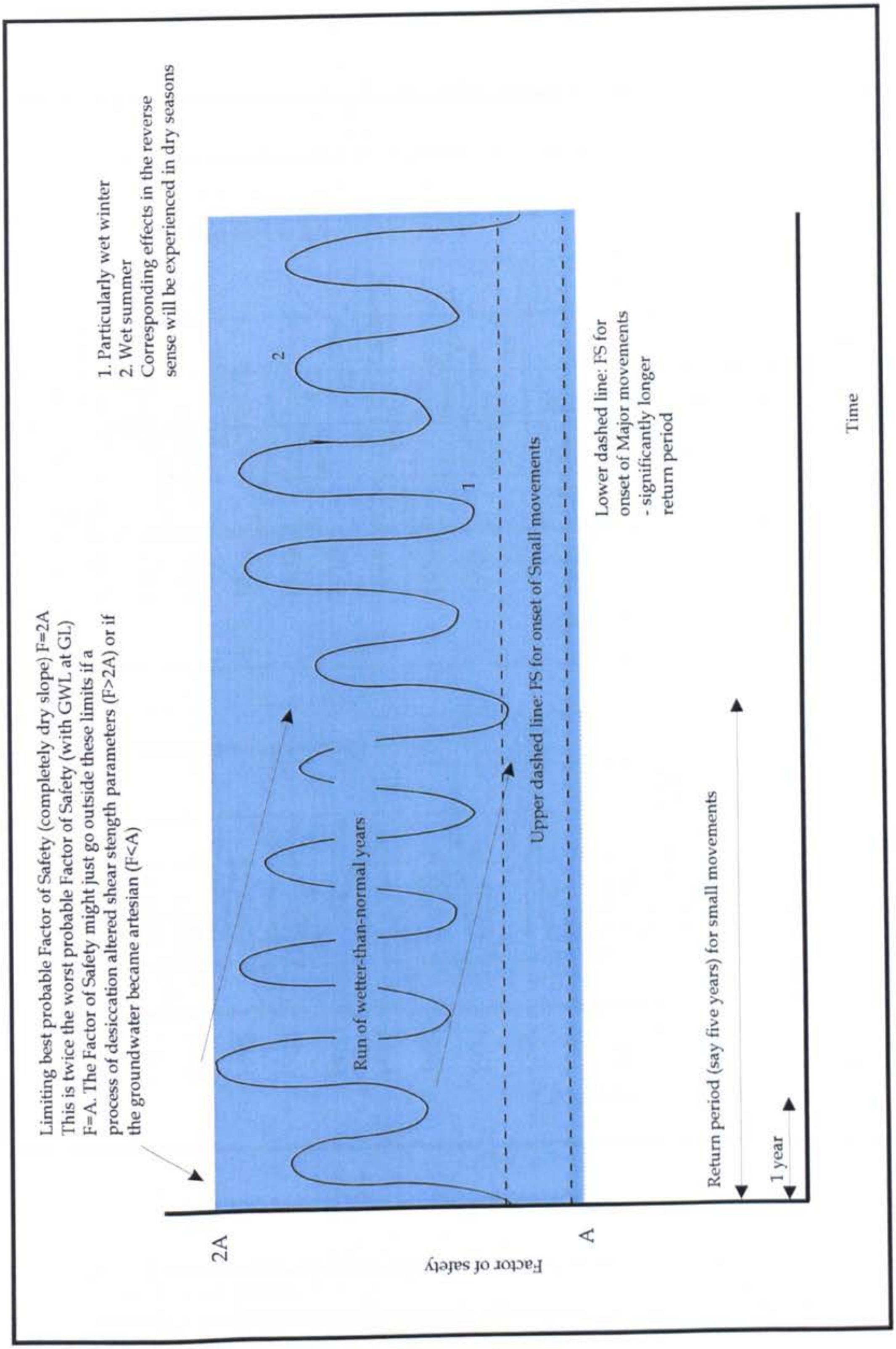


Figure 8.2. The seasonal cyclic variation of the Factor of Safety.

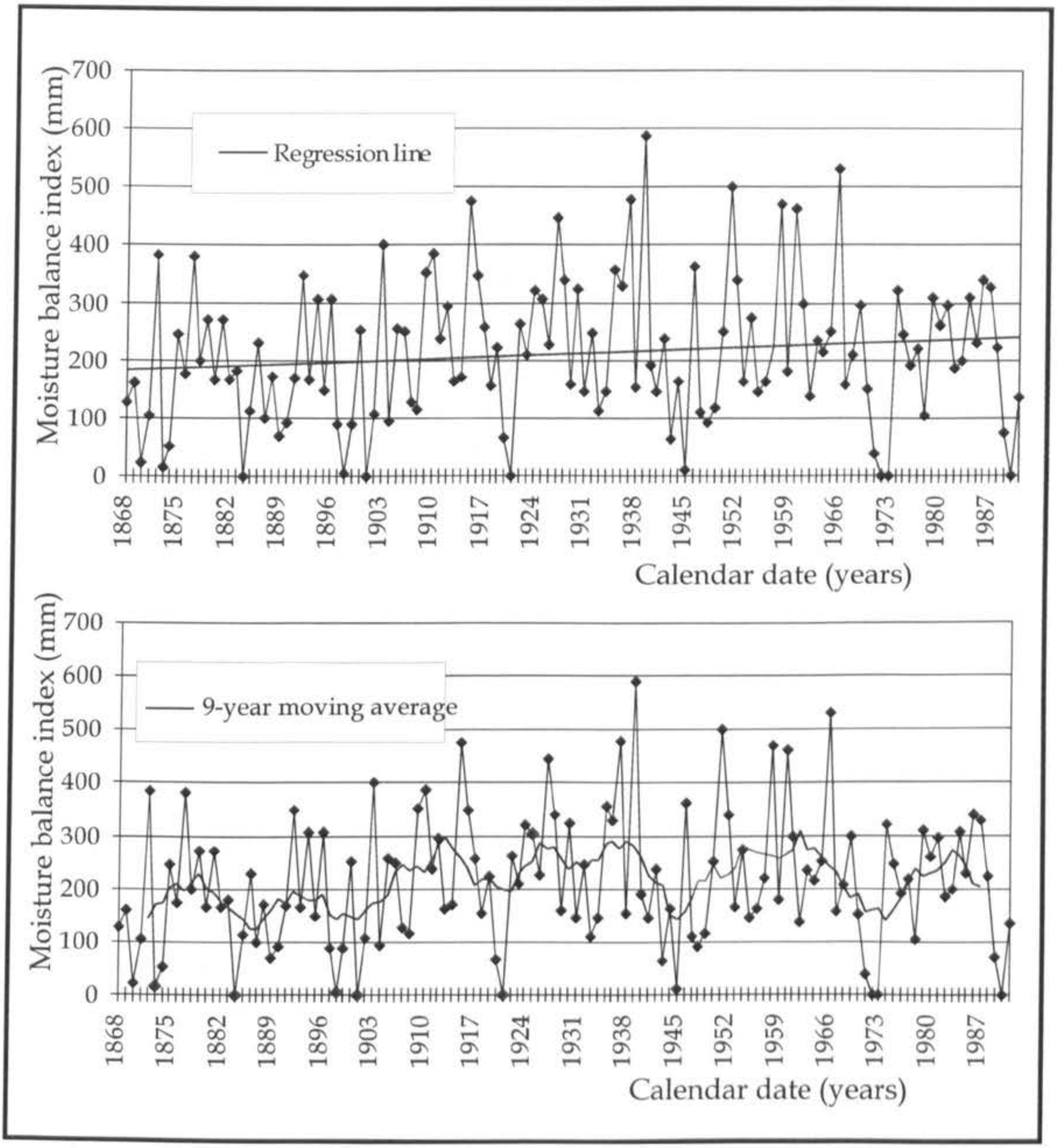


Figure 8.3. Moisture Balance Index against Time for Folkestone, UK, (1868-1991)
(a) (Top) Regression Line
(b) (Bottom) Nine-year moving average (After Bromhead, Hopper and Ibsen, 1998)

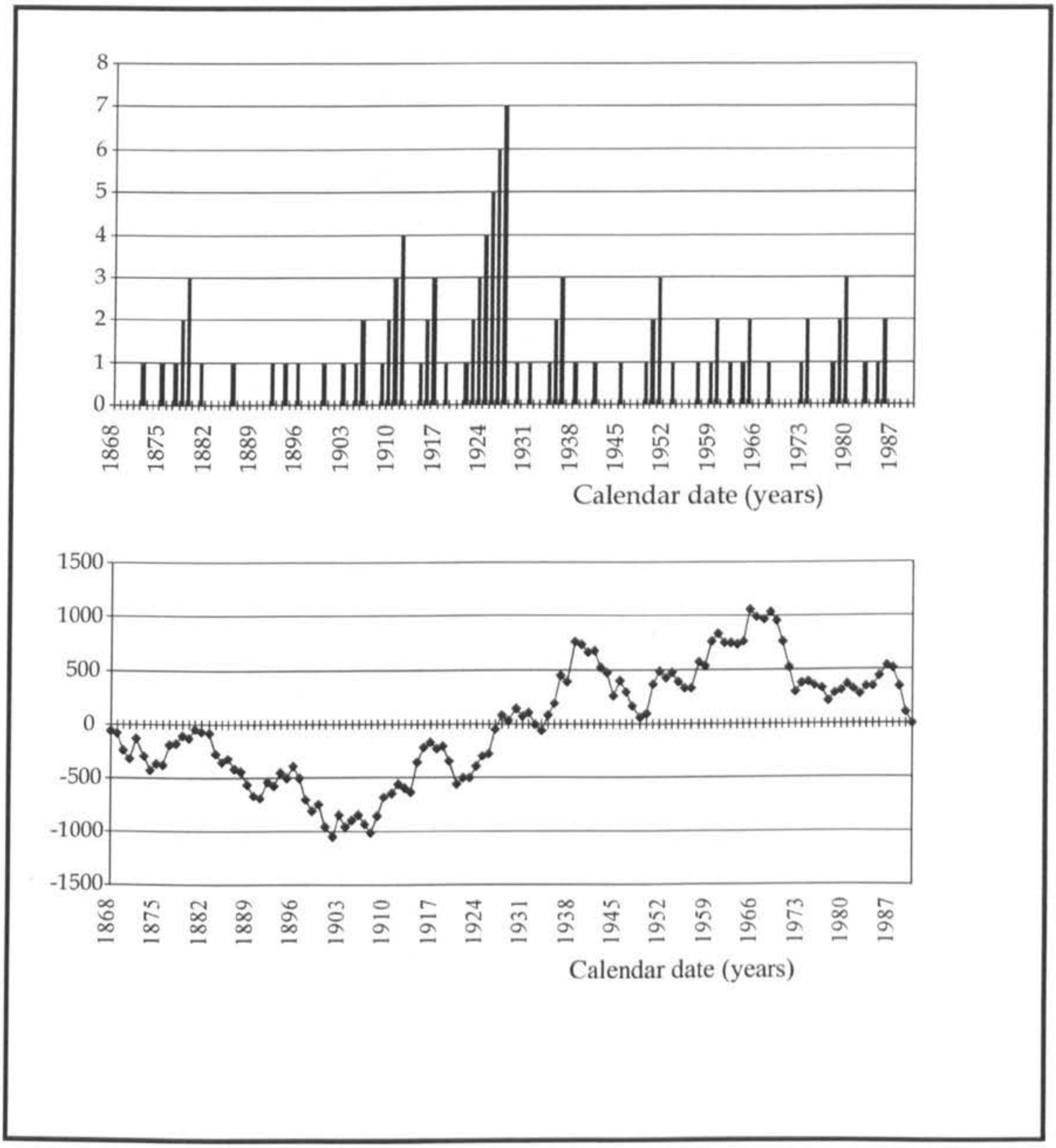


Figure 8.4. Moisture Balance index against time for Folkestone, UK, (1868-1991)
a) (Top) Number of years with above average rainfall
b) (Bottom) Cumulative affective rainfall from the mean. (After Bromhead, Hopper and Ibsen, 1998).

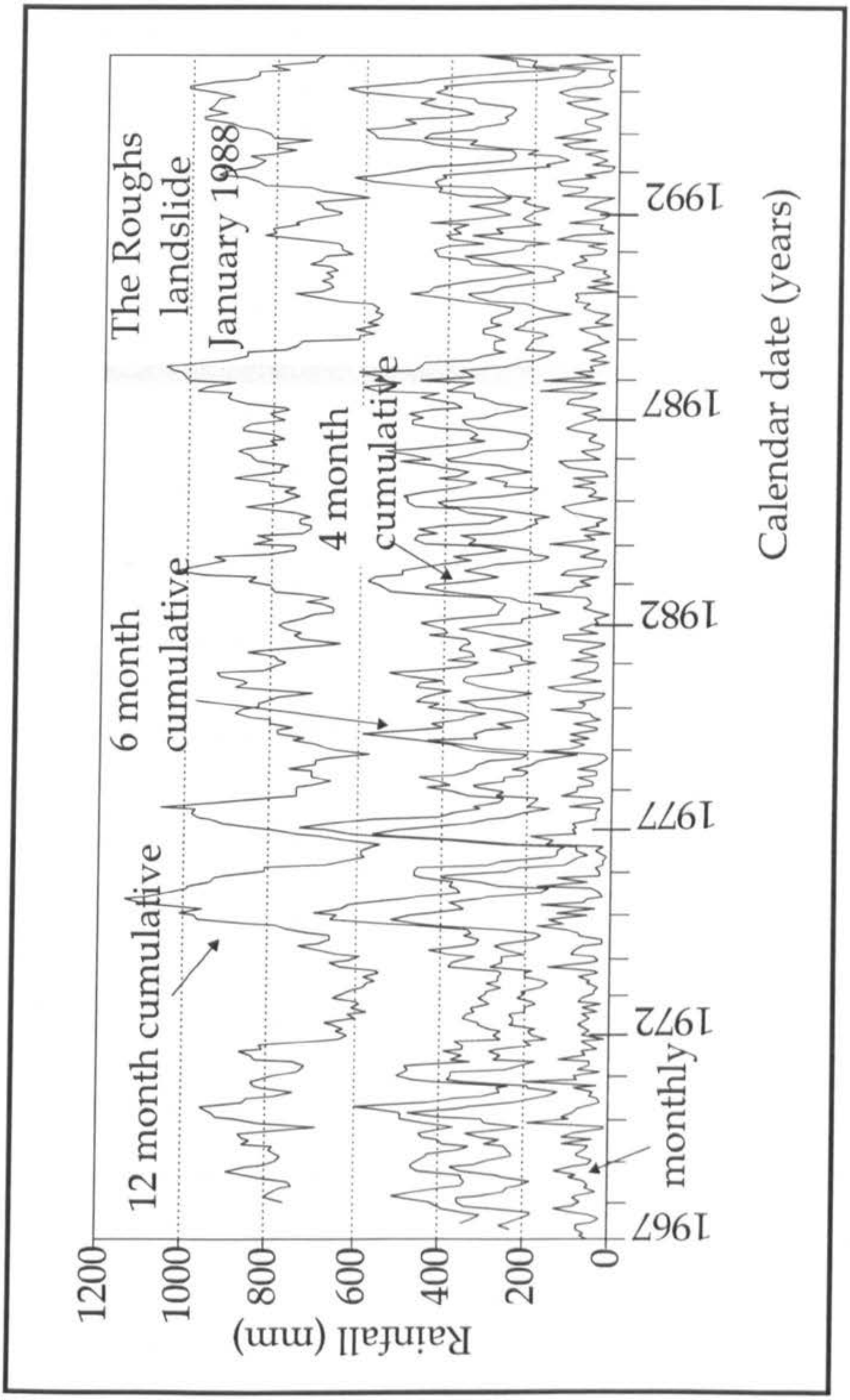
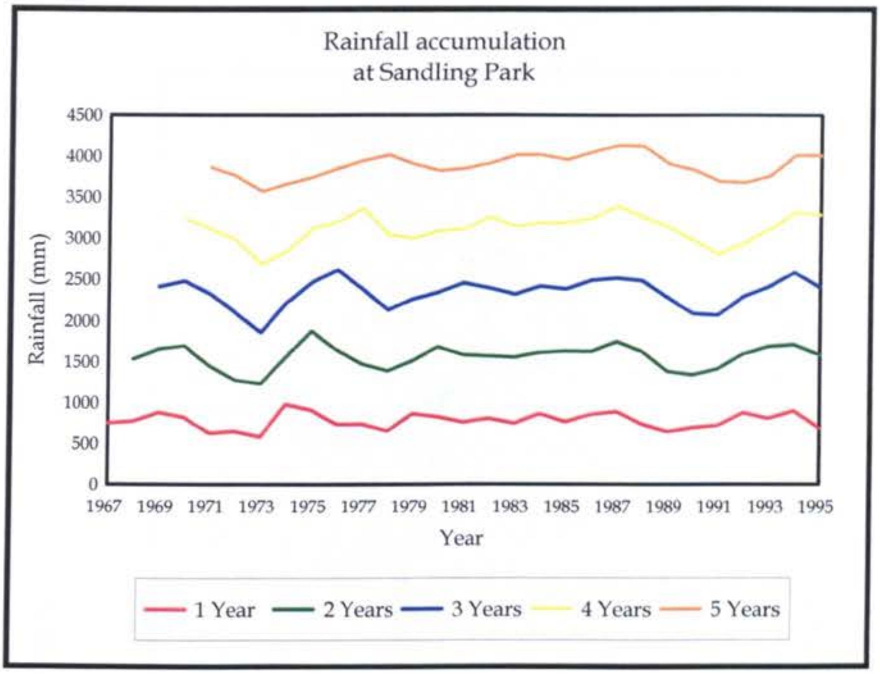


Figure 8.5. Monthly rainfall and cumulative rainfall for 4, 6 and 12 month periods at Sandling Park, (nearest weather station to The Roughs) 1967-1997. (After Bromhead, Hopper and Ibsen, 1998).



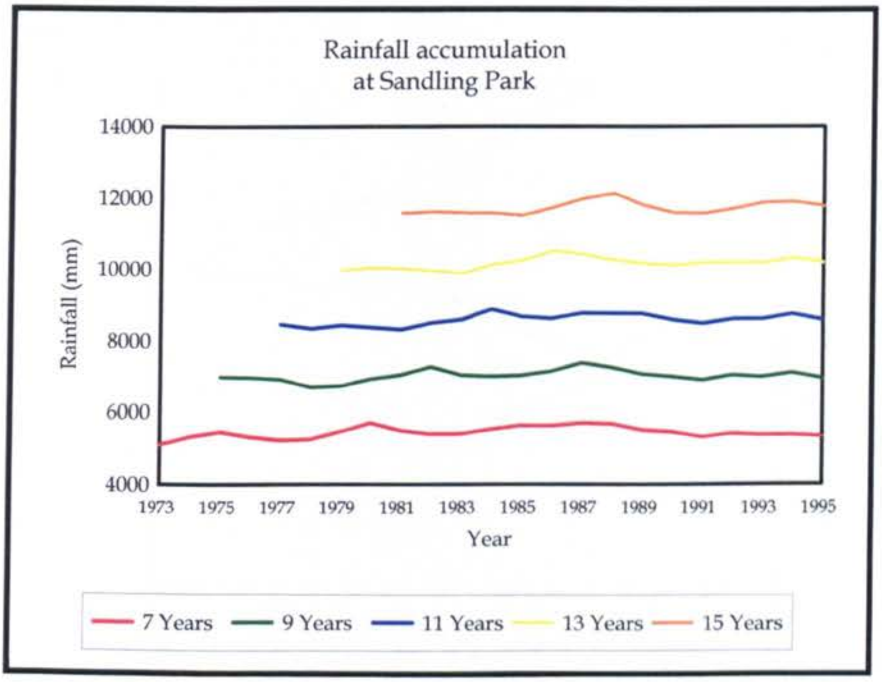


Figure 8.6. Accumulation of rainfall, over one to eleven years, at Sandling Park.

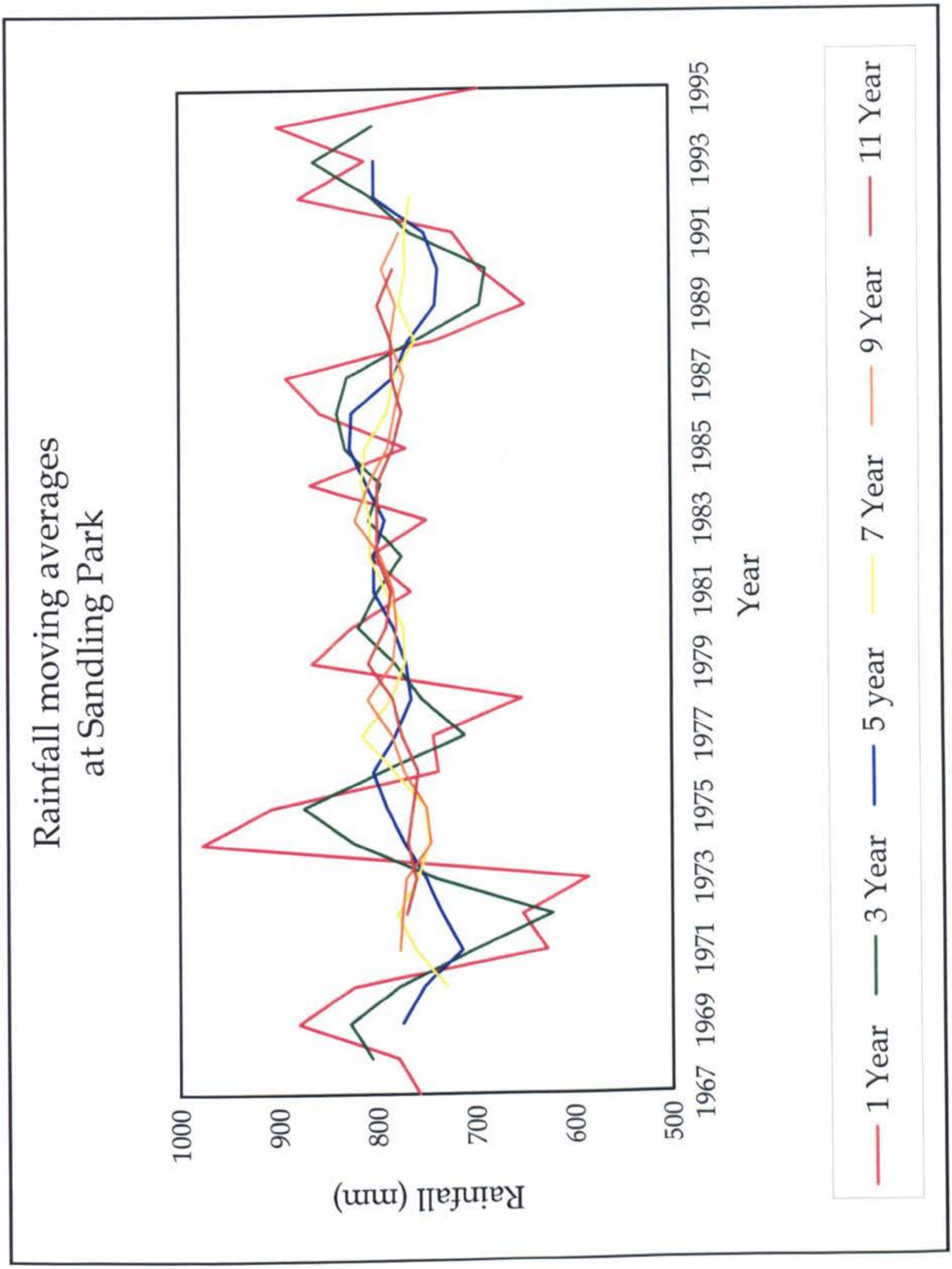


Figure 8.7. Rainfall moving averages, over one to eleven years, at Sandling Park.

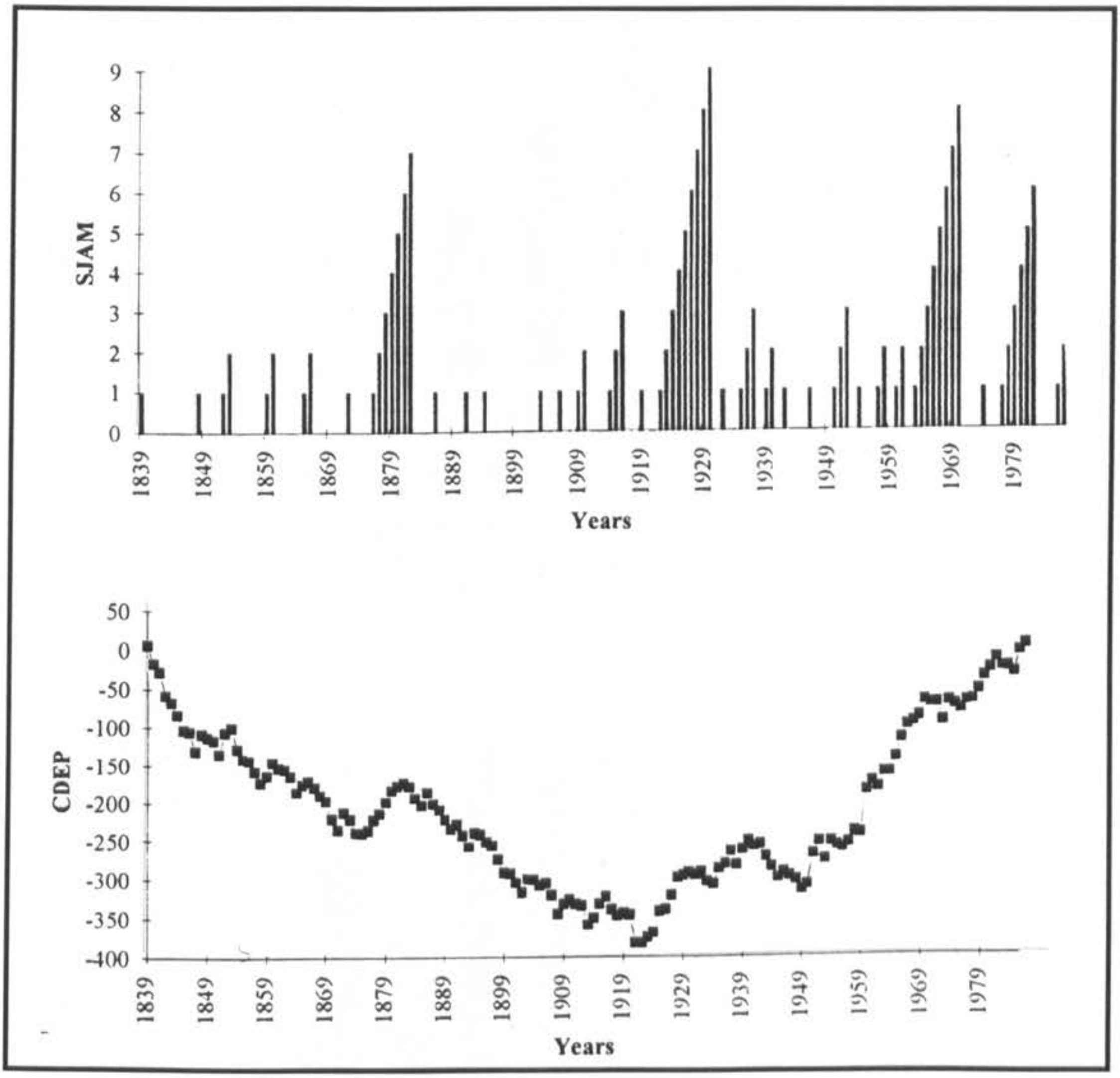
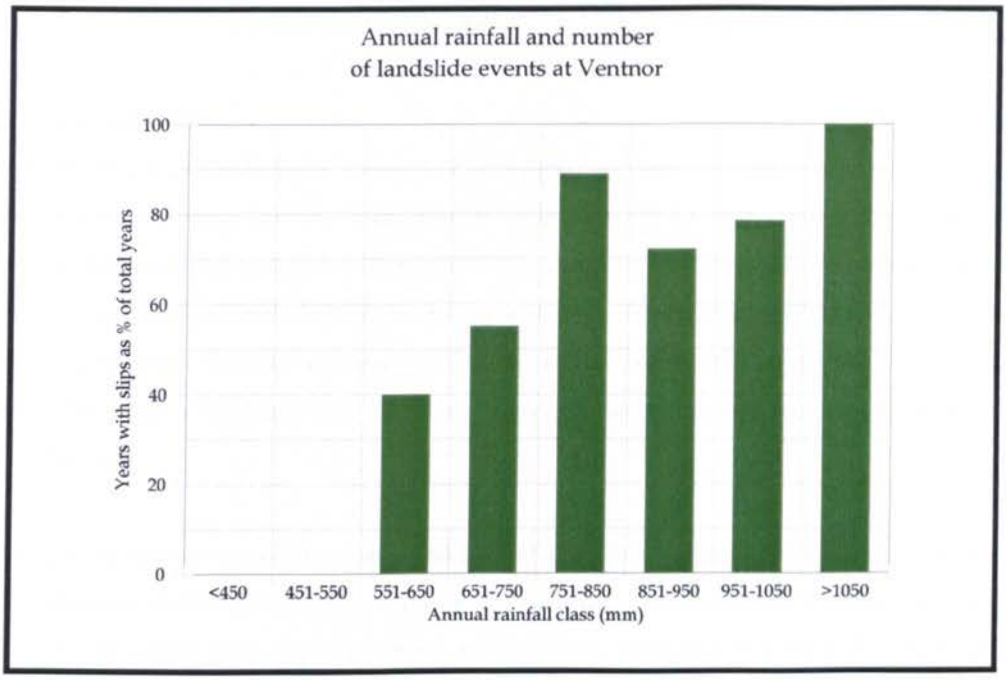


Figure 8.8 Moisture Balance Index against time for Ventnor, Isle of Wight. (1839-1987)
(Top) Number of years with above average rainfall.
(Bottom) Cumulative effective rainfall from the mean. (Ibsen and Brunsden, 1996).



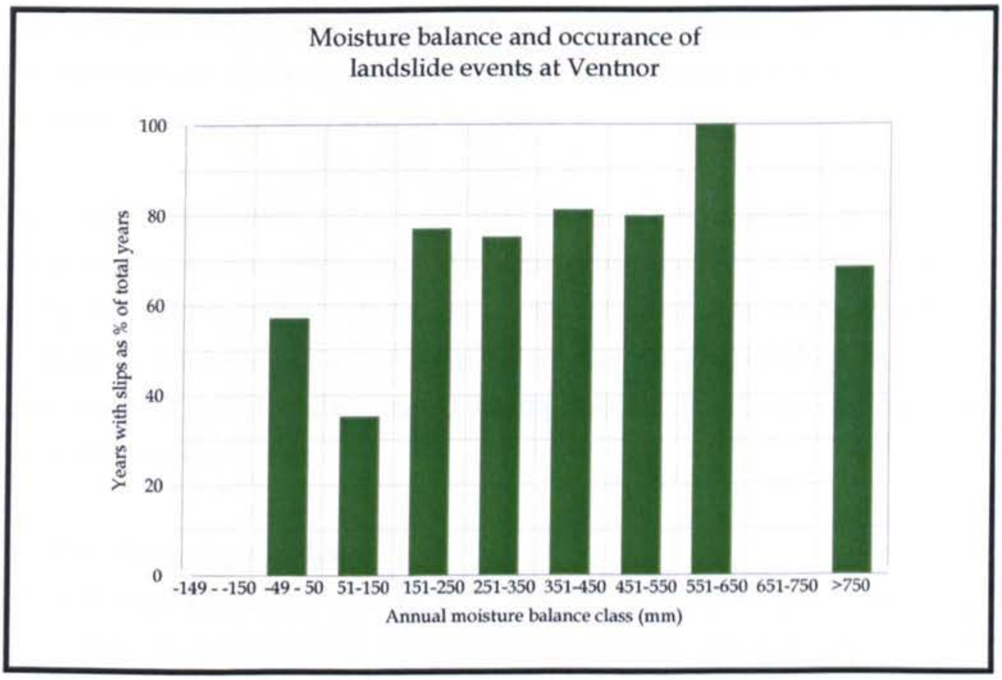


Figure 8.9. Ventnor, Isle of Wight.
(a) (Top). Number of years with landsliding events, as a percentage of total years within each annual rainfall class.
(b) (Bottom). Number of years with landsliding events, as a percentage of total years within each annual moisture balance class.

Analysis of the stability of The Roughs

9.1 Introduction

The slope system at the Roughs consists of an upper, perched, bedding-controlled compound landslide and a central, shallow translational slide element moving sub-parallel to the slope surface. Debris from this system has accumulated in a wedge occupying the lower slopes, but since this did not move appreciably in 1988, it is not considered part of the main slide system. The lateral extent of both slide elements is finite. Mudslides, emanating from the vicinity of the junction between the upper and central slide zones, run down the slope, in cases reaching the marsh at the foot of the slope.

The clear separation between the upper and central slide zones, marked by a scarp, indicates that they can be analysed independently. A computer code using the **Morgenstern-Price** (1965) method is used for the upper slide, and the central, translational, slide zone is analysed using **Haefeli's** (1948) infinite slope method, and a new method which makes allowance for toe breakout resistance and side shear. (The Morgenstern-Price method fails to converge with purely translational slides: simpler methods do not work well with curved deep slide surfaces due to insufficient consideration of internal stresses in the slide mass).

Having separated the central translational slide and the upper rotational slide, it is then an iterative process to ascertain which came first. Did the translational/accumulation zone move first, unloading the toe of the rotational slide, or did the rotational slide add head loading to the translational/accumulation zone, initiating movement? General observations, eye-witness accounts, fieldwork and laboratory data and stability analyses are all used to determine the most likely order and reason of events.

9.2 The infinite slope analysis

More than 50 years ago, Haefeli developed a method of analysing translational landslides with parallel seepage. The basic model for Haefeli's analysis, **Haefeli, 1948**, is shown in *Figure 9.1* and the equation he developed can be written as:

$$F = \frac{c' + (\gamma z - \gamma_w h_w)\cos^2\alpha \tan\varphi'}{\gamma z \sin\alpha \cos\alpha} \qquad \text{Eq. 9.1}$$

Where c′= soil cohesion

F= factor of safety

h_w=ground water level above slip surface

z= mean depth of slip surface

α= slope angle

λ_w= unit weight of water

λ= unit weight of soil

φ'=friction angle

Parallel seepage can occur when the increased density of an unweathered layer, below a weathered one, causes water to flow along the interface of the two layers. This parallel seepage is a perfect example for which the infinite slope method would be appropriate. Landslides which follow the contours of the slope can also be analysed using this method. Some slope failures by sliding or flowing are also suitable and some less so. The reasons for unsuitability are discussed in *Section 9.2.1.*

In 1957, **Skempton & DeLory** applied Haefeli's equation to the analysis of natural slopes, a process completed by **Hutchinson** (1969 - Oslo conference) who added the concept of residual strength to the mix. However, on a site-specific basis, the infinite slope method falls down. The reasons for this are explored below.

9.2.1 Limitations of the *infinite* slope analysis

The major limitations to the *infinite* slope analysis are:

- slip surface and ground surface morphology
- sides and side shear
- end effects

Haefeli's infinite slope method is not suitable for deep seated or rotational slides. The effect of side shear resistance is of more relevance to slides with indistinct boundaries, such as channelised or lobate slides, than for those with distinct edges such as slab slides.

9.3 The finite slope analysis

To compensate for the limitations of Haefeli's *infinite* slope method, a refined equation, including both sides and ends, has been developed by **Bromhead and Huggins** (pers comm.) which will be termed the *finite* slope method. This is represented diagrammatically in *Figure 9.2*

To summarise, the forces causing instability are:

$$D = W_a \sin\alpha_a + W_s \sin\alpha_s - W_p \sin\alpha_p \qquad \text{Eq. 9.2}$$

and the forces which resist instability, collectively termed R, are as follows:

$$R = 2S_p + 2S_s + 2S_a + P_p + P_s + P_a \qquad \text{Eq. 9.3}$$

These symbols are defined in *Table 9.1.*

9.4 Analysis using the infinite and"modified" infinite (or finite) slope method

The most suitable method for analysing translational slides is the infinite slope method, or in this case both the infinite and finite slope methods. An analysis of the translational/accumulation zone of the landslide was carried out using the programme *Haefeli* (Bromhead, pers comm.). As with any analysis of slopes, this involves a certain amount of "guesstimation". The input data required for the programme is as follows:

The angle of shear resistance	φ
Slope angle	β
Wall friction angle (0 or φ)	δ
Depth of slip surface at toe (m)	DT
Depth of slip surface at head (m)	DH
Plan length including wedges(m)	L
Plan breadth of central section (m)	B
Pore water pressure at toe (m)	PT
Pore water pressure at head (m)	PH

Two sections of the slope have been constructed, *Section AA, Figure 6.2* and Section BB, *Figure 6.10.* Of the two, Section BB is thought to be the most accurate and is used as the model for the stability analysis. The angle of shear resistance to be used is taken from the results of the ring shear tests, *Section 6.7.3, Figure 6.16.* The colluvium in the translational/accumulation zone is mainly a mixture of Hythe Beds (φ=30°) and Atherfield Clay (φ=10°). Since the Hythe Beds are broken up, and hence more plastic, the starting point used is φ=15°. Due to natural deviation, the slope angle varies along the length of the slope but is overall about 11°. Borehole P07 is at the base of the slope in Section BB. It was assumed that the slip surface was along the top of the Weald Clay. As can be seen in Figure 6.10, the colluvium at this point is deeper than further up the slope, due to the presence of the abandoned cliff. Therefore, the slip surface, although not identified in the logs, is assumed to be at a depth of about 7 m. Borehole P03B is at the head of the degradation zone. The slip surface there was taken to be at a depth of 5 m.

The length of the zone was taken as 300 m and the breadth as 500 m. The piezometric level at the base of the slope was taken as 1.5 m and at the head of the slope to be 7 m. (NB These figures are taken as distance below ground level whereas those used for the data input are given as height above the slip surface.)

It is an interesting exercise to examine the effects on the stability of the slope by varying certain parameters. The programme input data is given in *Table 9.2*, the series being examined is highlighted.

The programme first uses this data to calculate the average factor of safety for the slope using the infinite slope method as a comparison. It then gives a series of results using the finite slope method, under dry, hydrostatic and specified conditions. The factors of safety are given with and without the inclusion of the effect of side shear. A list of the entire range of output data is given in *Table 9.3* along with the actual figures calculated.

9.4.1 Comparison of the translational and accumulation zones

Firstly, the translational zone and the accumulation zone were analysed separately to determine their relative stabilities. It is impossible to distinguish exactly the point at which the degradation zone ends and the accumulation zone starts. Therefore, it was assumed that they were of equal length, 150 m. Borehole P05 was the closest to the midpoint of the slope. The depth of the shear surface here was at 4 m, according to both the inclinometer readings and the borehole logs. No piezometric readings were taken here but the comparable reading from Section AA was 1.5 m. This was used as the toe of the degradation zone and the head of the accumulation zone. *Figure 9.3* shows the model considered in this analysis. The data set used for the translational zone is given as Case no. 1 and that for the accumulation zone as Case no. 2. For the given conditions and using the finite slope method, the factor of safety, for the degradation zone is 1.34 (1.34) and for the accumulation zone, 0.92 (0.88), using the finite (infinite) analysis. The slope was then considered as a whole in Case no. 3 and the factor of safety was found to be 1.21 (1.18).

9.4.2 Effect of varying the slip surface depth

The first parameter to be looked at was the depth of the slip surface, since this was the least clearly defined. Case no. 3 as detailed in *Table 9.2*, used the values outlined above in *Section 9.4.1* and was calculated to be stable, with a factor of safety of 1.21 (1.18). This indicated that the initial assumptions were probably about right. To see by how much the position of the slip surface had to be raised, for the accumulation zone to be stable, two variations were considered (Case nos. 4-5). The analysis shows that the shear surface would have to be raised by 1.5 m to reach a factor of safety of 1 if the finite slope method is used and by 2.1 m for the infinite slope method. These figures are shown graphically in *Figure 9.4.*

9.4.3 Effect of varying the pore water pressures

This is probably the most important parameter to examine. Considering the complete slope and using the data specified, the factor of safety under dry conditions for finite (infinite) method is 1.21 (1.18). Case nos. 6-8 show the effect of varying the piezometric levels.

To determine the height of water needed to instigate unstable conditions (ie a factory of safety of one) the programme was run with a further data series. *Figure 9.5* shows graphically how the factor of safety varies with the water levels at the head of the slope. The piezometer levels, at the possible onset of instability, are 5.18 m and 5.35 m for the finite and infinite calculations, respectively. This rise in piezometric levels would result in ground water levels at the toe. During the period of monitoring the piezometric levels on the site, not one of the readings ever rose to critical levels.

Assuming that the voids ratio of the colluvium is about 30%, 0.5 m of water would be necessary to raise the piezometric levels by 1.65 m to attain critical ground water conditions. Although there was heavy rainfall immediately preceding the first movements, it totalled only about a fifth of this amount. The ground water levels required for instability could only take place with an input of water other than direct rainfall.

9.4.4 Effect of varying the length

Case nos. 9-11 have varying input values for the length of the slope. By definition, this has no effect on the values produced by the infinite slope method. It also has negligible effect on the factor of safety calculated by the finite slope method until the length reduces to 75 m. After this point, the effect of the toe breakout increases the factor of safety hugely as shown is *Figure 9.6.* In the range of the slope length of The Roughs, there is very little variation of factor of safety with large increases or decreases in length.

9.4.5 Effect of varying the width

As with the previous investigation, variation of the slope width (Case nos. 12-14) has no effect on the results of the infinite slope method for obvious reasons. *Figure 9.7* shows how the factor of safety varies with the slope width. There is little variation in factor of safety when large widths are considered . As the width decreases, the edge effects have greater significance and the factor of safety rises sharply.

9.4.6 Effect of varying φ′

The angle of shear resistance is one of the more critical parameters, since in theory φ could vary from 10° to 30°. If an area is considered which is predominantly Atherfield Clay, then it can be presumed that its angle of shear resistance could be as low as 10°. In Case nos. 15-17, the angle of shear resistance is varied to cover the range of angles of shear resistance possible, in the colluvium on The Roughs. For a reduction in the angle of shear resistance of only 3°, the factor of safety is reduced to 0.96 (0.93), for the finite (infinite) method. To discover the critical angle of shear resistance, a graph was plotted of the factors of safety against φ, *Figure 9.8*, and were found to be 12.5° (12.9°).

9.4.7 Effect of varying β.

The slope of The Roughs (β) is not smooth and locally shows a lot of variation. Case nos. 18-20 examine the effect of small changes to the slope angle. An increase of 3° or 27 % in the slope angle results in the factor of safety, using the finite (infinite) calculation, reducing to 0.93 (0.92). To determine the critical angle at which the factor of safety has a value of one, a graph was plotted of factor of safety against slope angle, *Figure 9.9*. The critical angles were found to be 13.2° (13°). These angles are not very different to the overall slope angle which leads to the presumption that a proportion of slope is in an unstable condition.

9.4.8 Conclusions from the finite and infinite slope analyses

One of the most noticeable results from this analysis, is the fact that the translational zone is stable whereas the accumulation zone is not. If this really were the case, then the whole system would be active. The accumulation zone did move and is, in fact, still moving, as is evident from the large tension cracks covering the toe. This movement must be filtering up through the whole landslide system, although the evidence is less clear on the surface. However, the inclinometers in both the degradation zone and the rotational zone prove that there are movements in these locations too. It would seem likely that the piezometric levels at the toe of the accumulation zone would be low, as the water under-drained into the beach deposits in the marsh. One possible explanation , is that the head of the degradation zone contains larger pieces of Hythe beds debris. As this migrates down the slope, it gets more broken up and mixed with clay colluvium, which decreases its permeability. The high levels of water from the wet weather period before the slides would have raised the piezometric levels at the head of the degradation zone. It seems unlikely that reactivation of the landslides was caused by large movements in the accumulation zone. Eye-witness accounts report that the movements started at the base of the rotated blocks.

The Roughs can not be considered as having a smooth homogenous slope. The colluvium on the degradation/accumulation zones is a mixture of Hythe Beds, Atherfield Clay and in places, Weald Clay. If a steep area of predominantly Atherfield Clay is considered then the figures would suggest that the factor of safety of that area is below one. Conversely, a shallow area of Hythe Beds would be very stable. Although the slope is locally unstable, the thrust required in these areas to 'break away' from the hold of the surrounding stable areas, is large enough to prevent failure. At any given time, a proportion of The Roughs is in a partially unstable, condition and undergoing the process of creep. When the ground water levels rise then this proportion will increase and eventually instigate total failure.

Within the range of parameters on The Roughs, the slope angle and the angle of shear resistance are the crucial parameters.

It is interesting to note the variation between the factor of safety calculated by the infinite slope method and that calculated by the finite slope method. In four of the graphs, *Figures 9.6-9.9*, the two values converge as the factor of safety decreases and the value given by the infinite slope method is the lower of the two. For the range of figures considered in this exercise, there is negligible difference in the two values, although this is not true for all scenarios. The largest variations in the two values are seen when the side shear is of relevance, such as when a very narrow slide is considered.

9.5 Morgenstern Price analysis

Stability analyses using the Morgenstern Price method (**Morgenstern and Price, 1965 and 1967**) were performed on the rear, rotational slips of section BB. Information from boreholes logs, moisture contents and inclinometer readings was used to estimate the positions of the slip surface. It was presumed that the soil properties for the Hythe Beds and Atherfield Clay, respectively were:

cohesion	0 kNm^{-3}	0 kNm^{-3}
angle of shear resistance	30°	10°
unit weight	20 kNm^{-3}	20 kNm^{-3}

Figure 9.10 shows the dimensions used for the first case and the initial data set processed. Two more sets were analysed, the second and third cases, with higher piezometric levels. These three cases, as inputted into "Morgen" were automatically plotted out as a validation of data input, and are shown in *Figure 9.11*.

Results from programme "Morgen" take the form of values for average effective stress, average shear stress and average R_u for each specified strata as well as average values for the whole system. Two values for the factors of safety are given: that for the standard analysis and that for the result after the third iteration by the Janbu method. These results are shown in *Table 9.4*, stratum 1 being Hythe Beds and stratum 2, Atherfield Clay.

The results show, as expected, that the three cases become progressively unstable as the piezometric levels rise. What was not anticipated was that the first case would have a factor of safety below one. It is presumed that the effect of the toe loading increases this value to above one. Of more relevance to the stability analysis of the 1988 landslide on The Roughs is the analysis of the rotational zone as it was then. This is not known but can be conjectured. Assuming that the last rotational slip was of a similar magnitude and form to the current one, and that the whole system has naturally degraded back since then, then the slip surface present, at the time of the reactivation, can be assumed to be that shown in *Figure 9.12*. The factors of safety from this analysis are 0.97 (Morgenstern and Price) and 1.00 (Janbu).

9.5.1 Conclusions from the Morgenstern and Price analysis

It would seem that, if the model of the rotational zone is correct, that it is unstable. It is probable that the model is not entirely accurate. It is difficult to be exact as to the shape of the slip surface and there are possibly perched water tables confusing the issue. Although side shear has been considered in the analysis of the translational and accumulation zones, which are comparatively shallow, the Morgenstern Price analysis is only two-dimensional. It is in these deep seated landslides where the side shear has significant effect. It is not beyond the bounds of possibility, that it would have enough effect to raise the factor of safety well above one and probably, also above the factor of safety of the translational zone. The morphology of the rotational slide would suggest relatively easy drainage both into the head of the translational zone, along the slip surfaces and eastward along the regional dip. This would also contribute to its stability.

9.6 The mudslides

The two mudslides stretch the full height of the slope, and are the most active bit of the slide complex, but also have the largest proportion of side shear. The question has to be asked, why are they so active? In all probability, they have a supply of water which can keep the groundwater level up to or near ground level. There is no apparent source of debris at the head of either mudslide to provide loading and undrained pore water pressures.

It is probable that the western and eastern mudslides were created by different mechanisms, although both resulted from the 1988 movements. The first to develop fully was that to the west of the landslide. It is probable that this mudslide was a direct consequence of leaking drainage pipes which were installed as part of the acoustic research centre development, *Section 3.6.2*, and subsequently damaged in the landslide.

The eastern mudslide developed slightly later and was seen not to be fully developed in the 1990 aerial photographs. It was probably due to the massive hydro-geological changes which resulted from the landside. A typical rotational slide is characterised by the back-tilted slump at its head. At The Roughs, this hole eventually filled up with Hythe Beds debris. This acts as a massive aquifer, storing rainwater. The water is then channelled down the slope, favouring the eastern edge due to the 1° dip to the east which exists on The Roughs.

What are the reasons why the central translational slide zone isn't equally as active? It differs from the mudslides in that it has proportionately less side shear and doesn't extend down the length of the slope to the toe of the escarpment, running out in the upper part of the accumulation zone. However, it has the same gradient as the mudslides, and approximately the same properties. The obvious answer must be that the ground water level doesn't get so high. It is probably because rain is less efficient than a concentrated source fed by a larger catchment, at filling up the voids in the soil.

9.7 Sequence of events

Section 3.7 describes Jill Edison's account of the events on The Roughs in 1988 and the deciphering of these events by **Anderson, 1990**. Their opinion was that a primary translational slide formed, possibly due to an associated streamline, which caused movement in the zone above.

One of the difficulties of determining the sequence of events on The Roughs is the time elapsed between them and the start of the investigation. Therefore an eye-witness account can not be ignored. This has to be part of the basis for the interpretation of the sequence of events in 1988, taking into account what the stability analyses have revealed.

Each section of Jill Eddison's account of the stages of the landslide must be looked at in turn. On

January the 15th, 1988 she reports the initial stages of landslide I, see *Figure 3.11*. It was already part rotational and part translational. Anderson adds that the landslide started below a streamline emerging from the base of the rotated blocks. Apparently, high water levels had led to seepage erosion at the base of zone of rotated blocks. Two gullies were formed at the head of the slope, the debris from which formed a mudflow which migrated down the slope. Inclinometer evidence shows that the mudslide overlay a true translational slab type landslide.

The statistical analysis of rainfall data in *Chapter 8*, shows a wet weather period before the events of 1988. The Author suggests that the hydrology of the site was affected by this extra water and that new streamlines were formed or existing ones increased in volume. The finite analysis indicates that the piezometric levels would have to rise 0.5 m to instigate instability. However it is easy to believe, since it had not undergone any significant movements in many years, that The Roughs in 1988 was in a much more unstable condition than it is presently. The amount of water needed to reactivate the translational zone would have been significantly less than the analysis has shown. Water from the streamline, in addition to the high piezometric levels resulting from the period of wet weather, raised the water levels at the head of the translational zone, sufficiently to reach critical conditions. This movement permeated down the slope as the unstable material added head loading to the material further down the slope. This, very quickly unloaded the toe of the rotational slide, since there was already evidence of the rotational slide, in the form of a 4.6 m scarp, when Jill Eddison first investigated it. The stability analysis of the rotational zone would imply that this would move first, but again, the slope profile was not the same as it is now.

According to Jill Eddison's account, this process continued and the initial stages of the rotational slump could be seen on 6th February. A week of rain culminated in very heavy rain on the 5th of February. Another wet week followed and the remainder of February was quite dry. On the 15th of February she noticed the first signs of landslide II and landslide III. Each are associated with a springline. Landslide II is also described as a mudslide. So it is likely that it had the same mechanism as the first one.

Anderson, 1990, emphasises the number of days that the site was at field capacity as the cause of the movements. The Author is not disputing that the field capacity is of relevance, but does not think that it is the sole contributory factor. The obvious flaw is that the whole site was subjected to the same conditions, but only a limited area moved. The suggestion is that the wet weather period changed the hydrology of the slope, creating or increasing the number of springlines. These springlines added a large volume of water to the head of the translational zone, activating it. This unloaded the toe of the rotational zone triggering the rotational slide. Why the springlines where formed where they were was probably due to the scars of ancient landslides.

9.8 Summary

The interpretation of borehole logs, inclinometer readings and piezometer readings have resulted in the construction of two geological sections through the Lower Greensand Escarpment on The Roughs. One of the sections, BB, was thought to be the more realistic of the two due to the deeper boreholes on that section. The ground profile showed on the survey of this section, especially at the top of the slope, did not appear to be entirely natural. Possibly this is due to landscaping, done in conjunction with the construction of the Acoustic Research Centre.

The analyses carried out on the rotational and the translational slides show the stability of The Roughs at the time of the investigation. This is not the same as the situation which existed before the reactivation in 1988. There is evidence of former rotational slides but these landforms have degraded over the years. A large scale collapse of the rear cliff of Hythe Beds did not occur during the recent movements. However, the presence of Hythe Beds in the landslide colluvium indicates that larger events have happened in the past. The exact situation on The Roughs before the reactivation is not known. This makes a stability analysis for the conditions at the time somewhat difficult. However, a system was proposed which may have been similar to that present at the time.

The clear starting point when trying to fathom out the sequence of events on The Roughs was to go back to the eye-witness account. This was very specific as to where and when the landslides started and no reason could be found to dispute it. It was concluded that the wet weather period preceding the reactivation, caused a change in hydrology of the slope. This created springlines at the base of the zone of rotated blocks. These springlines raised the water levels at the head of the degradation zone to critical levels and caused the onset of landsliding. The rotational slides followed soon afterwards.

There is evidence that the landslide system is still moving, although not as dramatically as in 1988. Whether this is due to the massive changes in the morphology of the slope, or to the general trend in increased rainfall, or indeed a combination of the two, is a matter for further research.

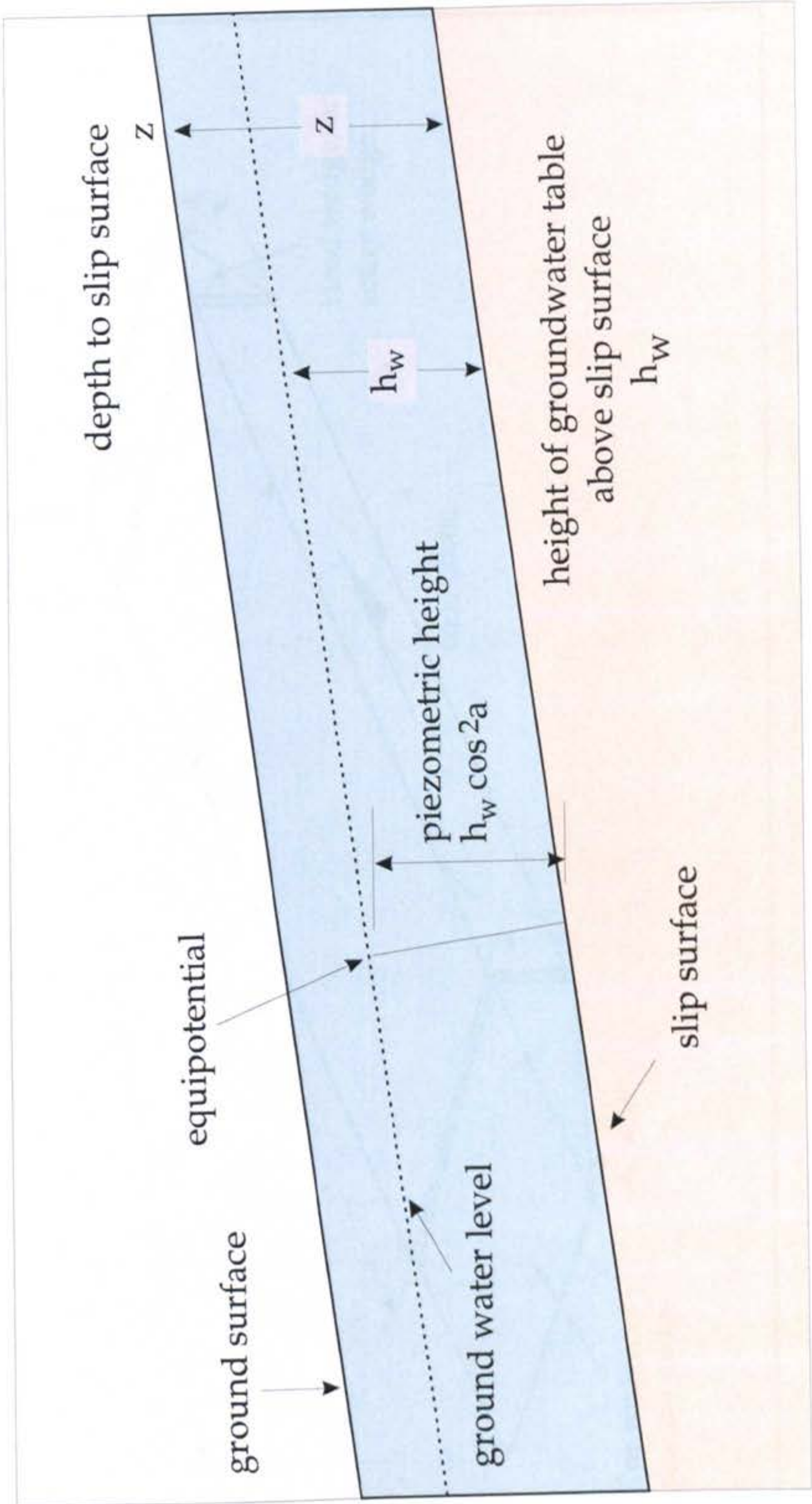


Figure 9.1. Basic dimensions of Haefeli's Infinite Slope method.

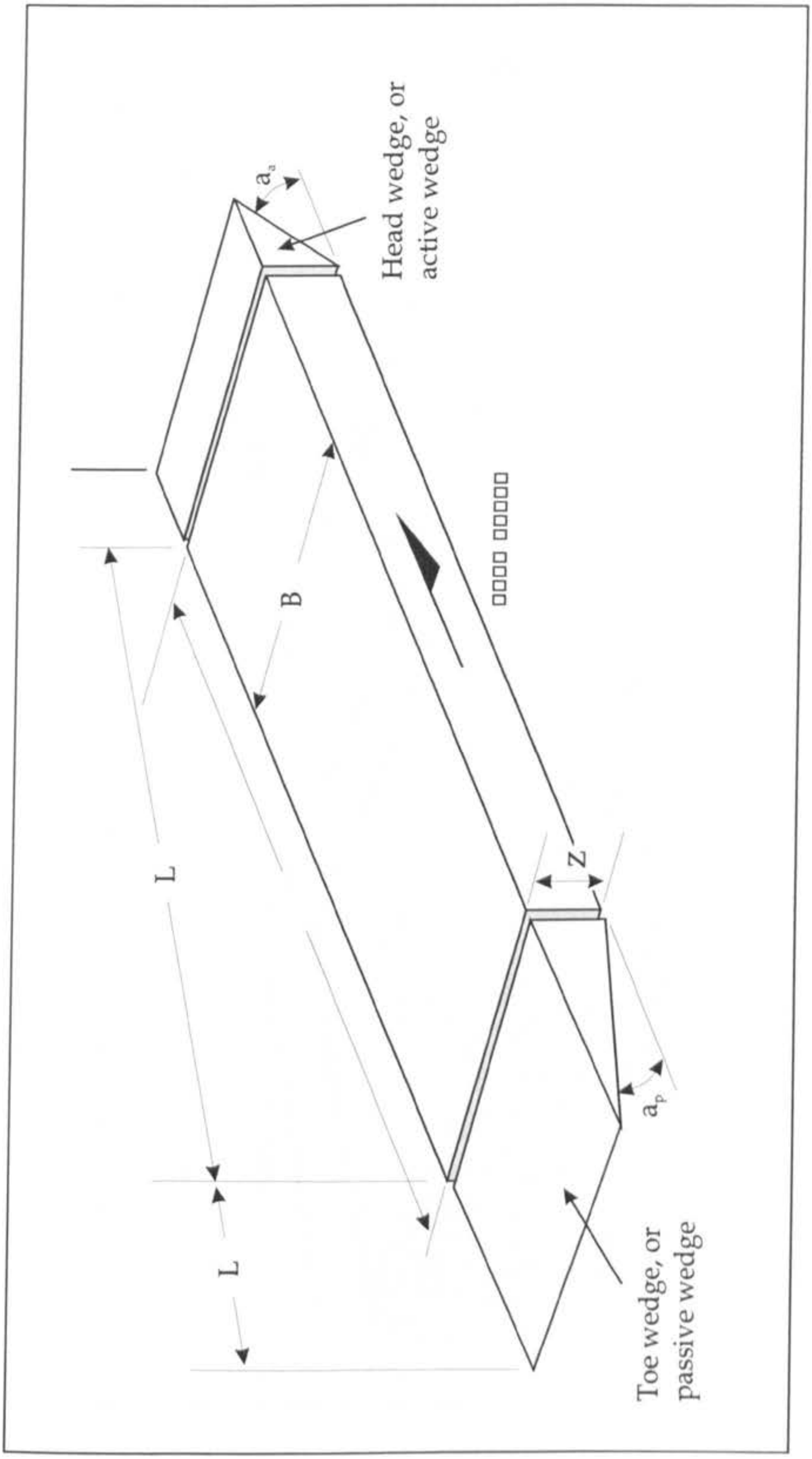


Figure 9.2. Basic dimensions of the finite slope method showing side shear and head and toe wedges.

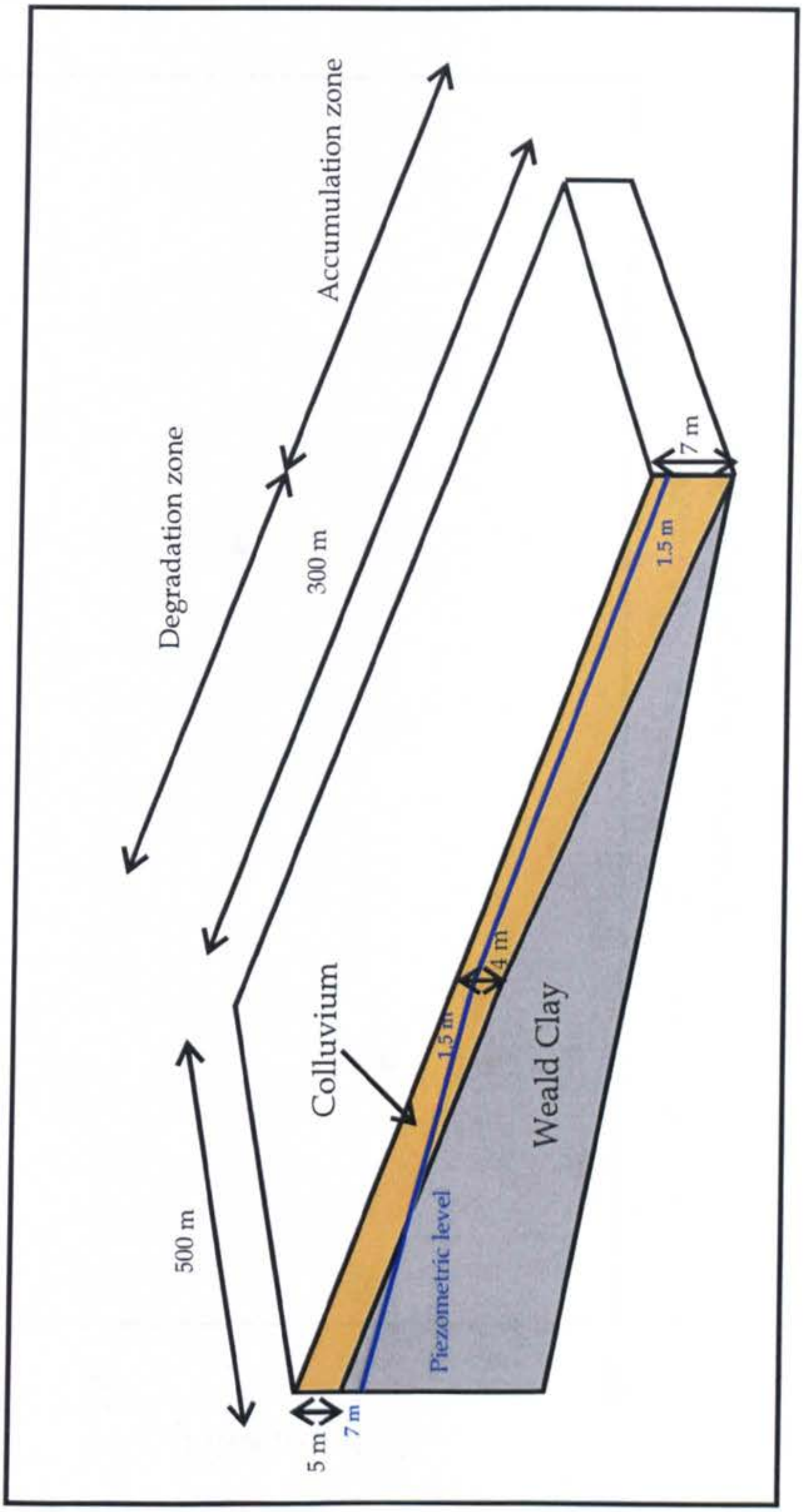


Figure 9.3. Model of the accumulation and degradation zone, showing piezometer levels, used in the finite and infinite slope analysis.

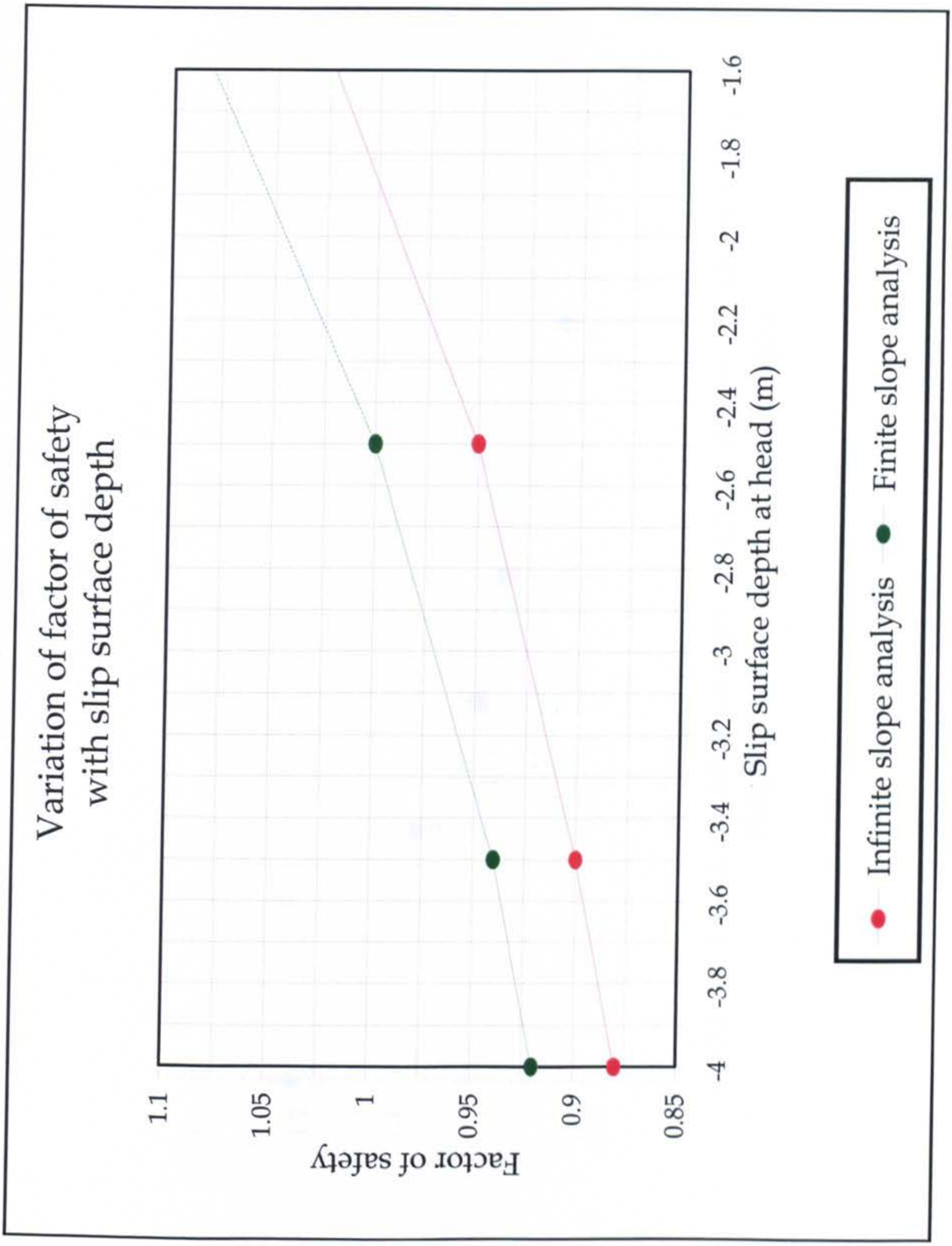


Figure 9.4. Graph showing the relationship between the factor of safety and the depth of the slip surface in the accumulation zone.

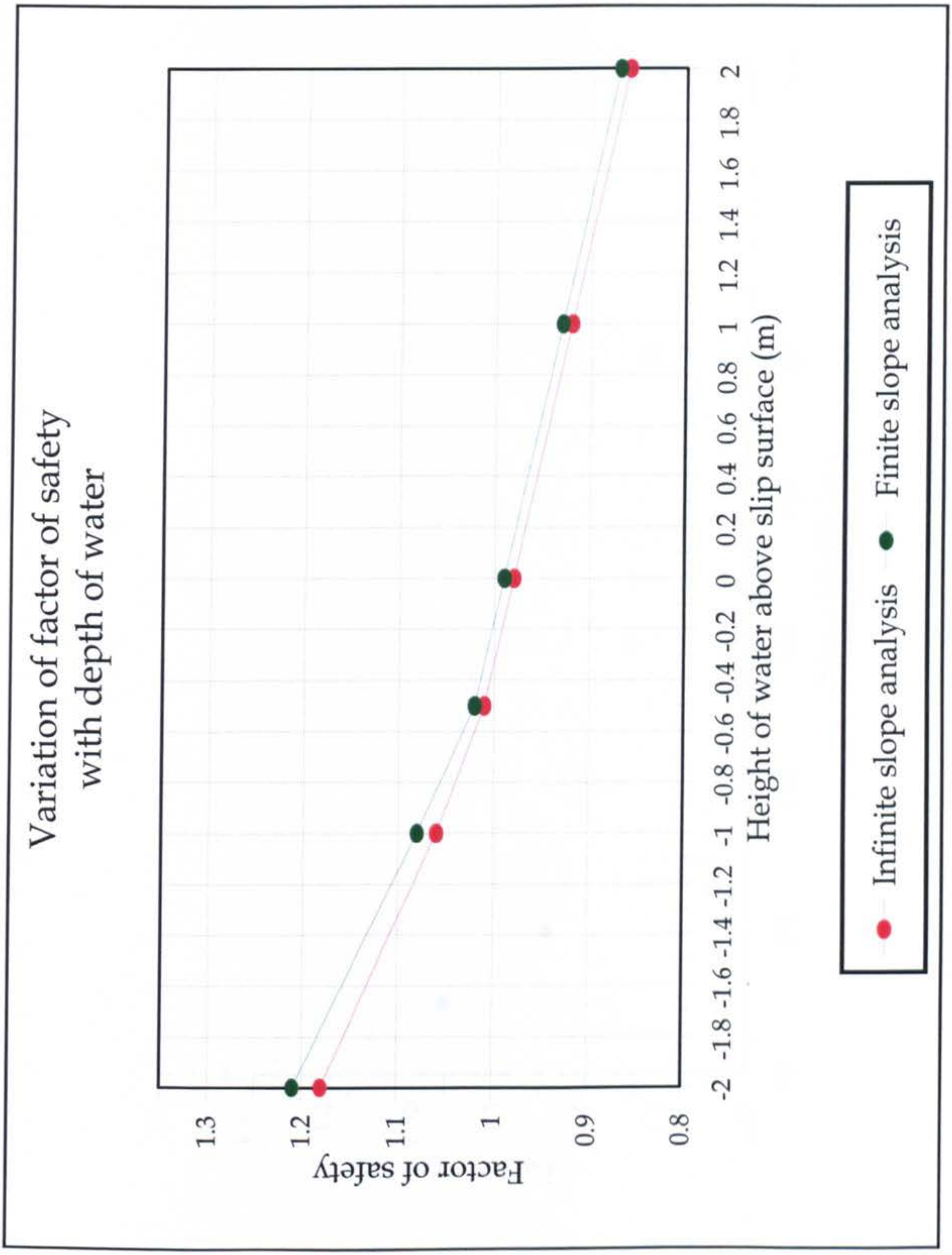


Figure 9.5. Graph showing the relationship between factor of safety and water level at the head of the translational zone.

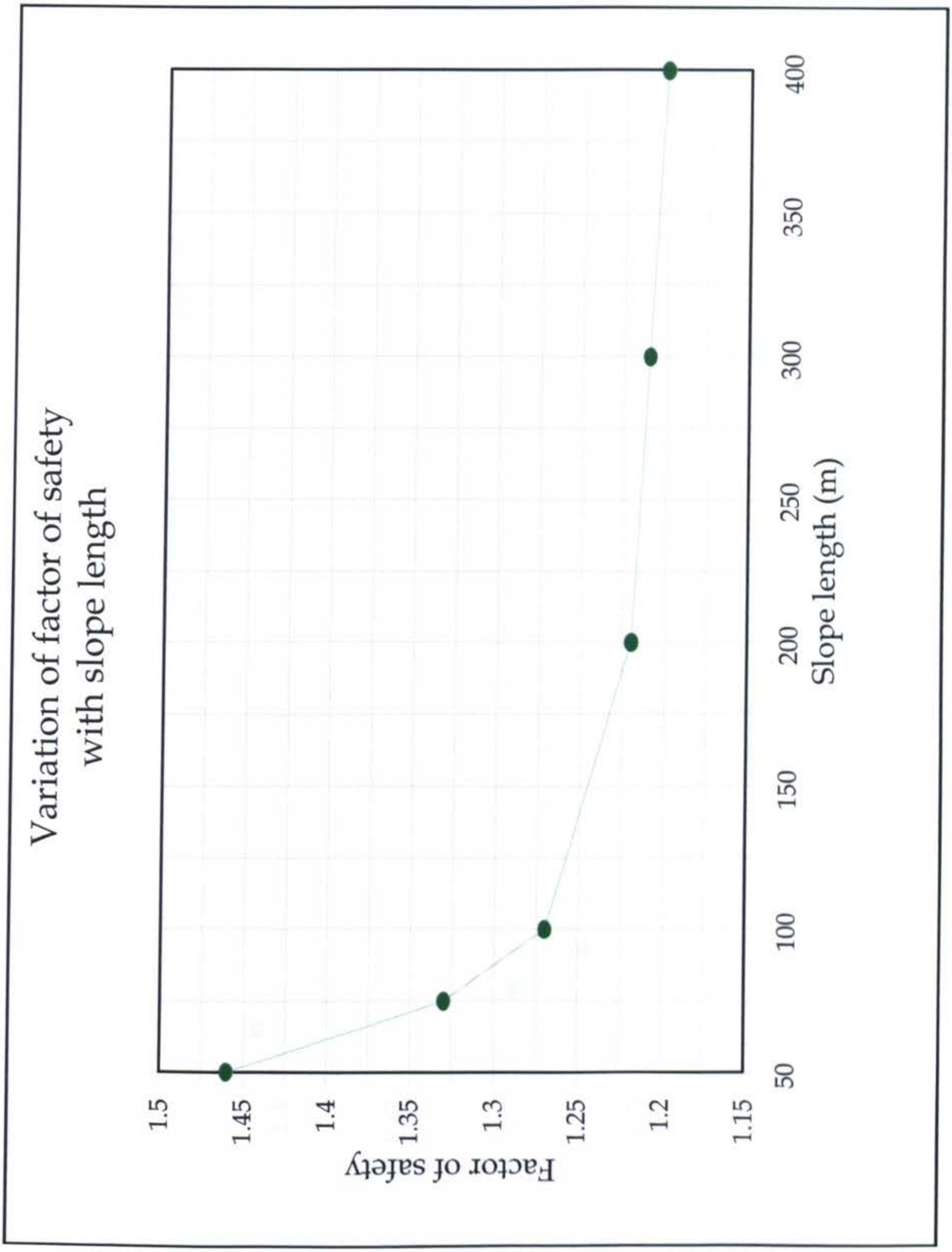


Figure 9.6. Graph showing the relationship between factor of safety and length of the degradation zone.

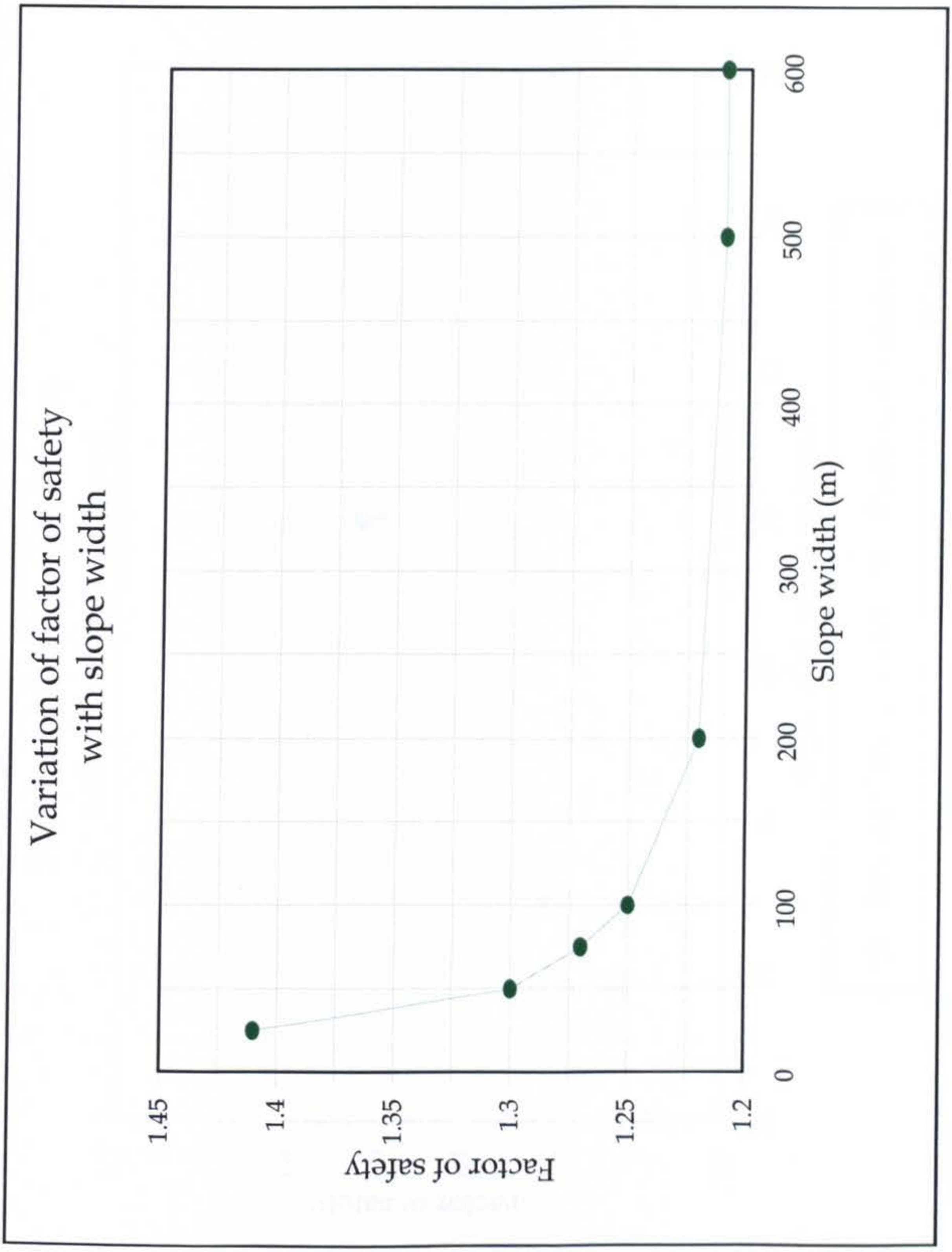


Figure 9.7. Graph showing the relationship between factor of safety and the width of the translational zone.

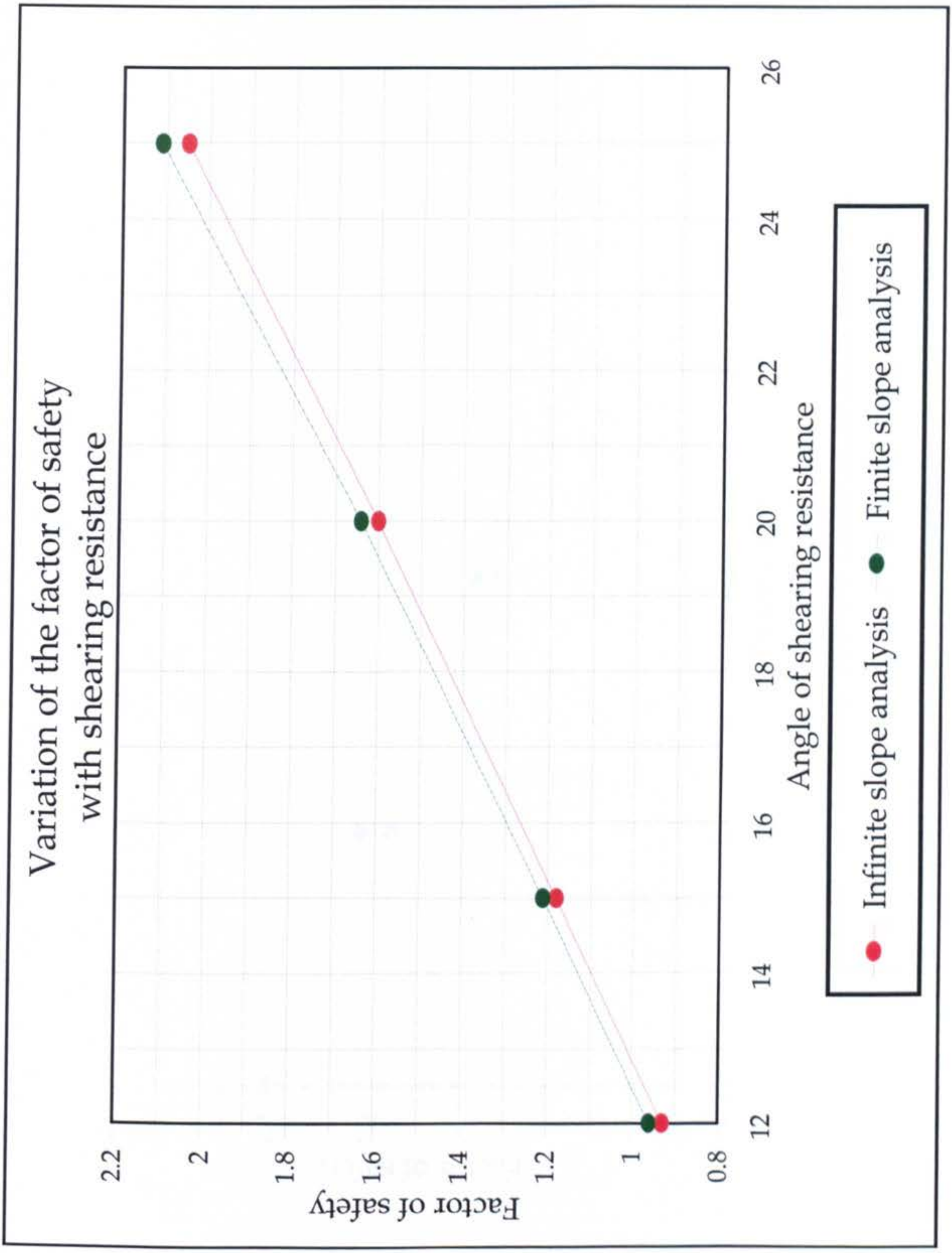


Figure 9.8. Graph showing the relationship between factor of safety and the angle of shear resistance.

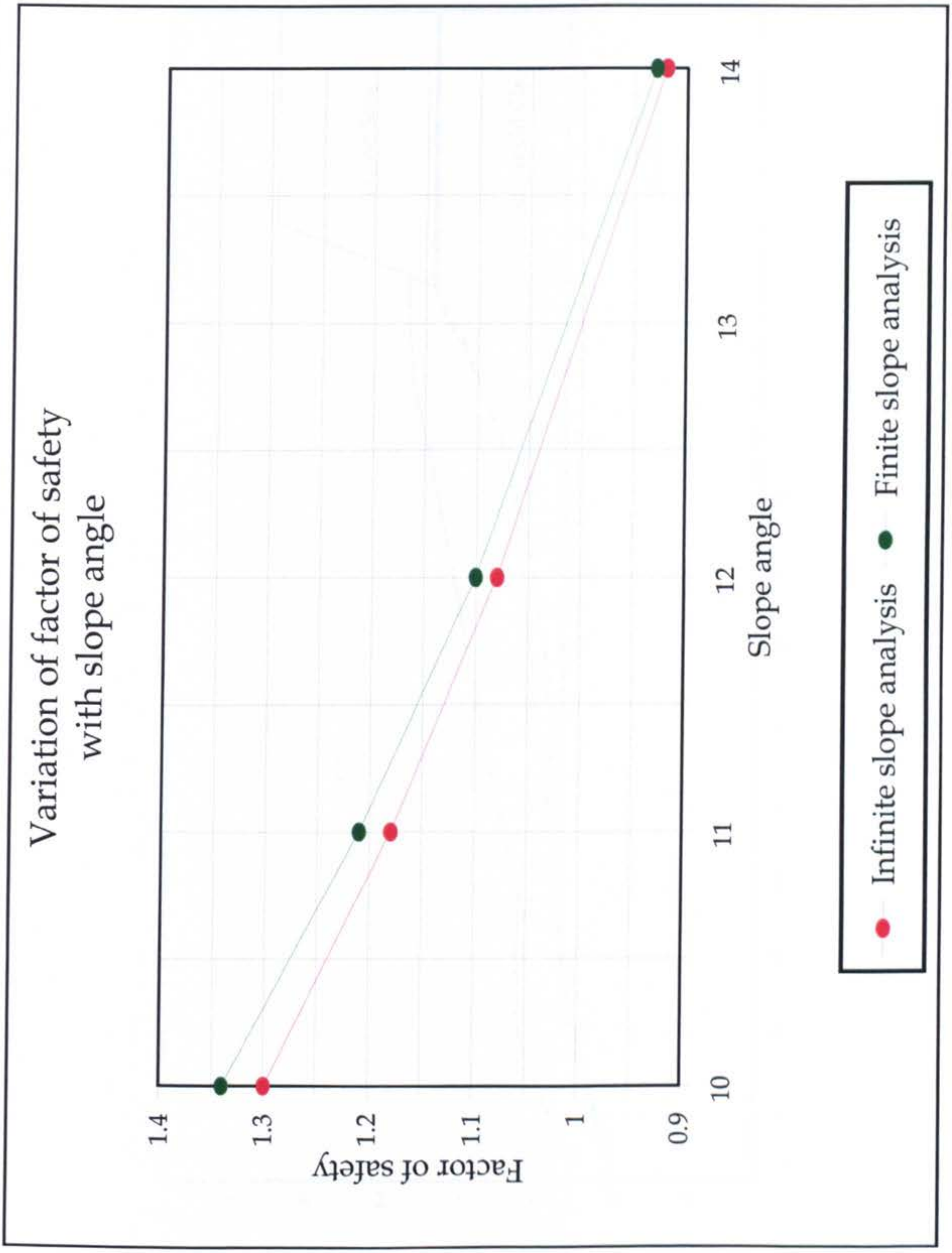


Figure 9.9. Graph showing the relationship between factor of safety and the slope angle.

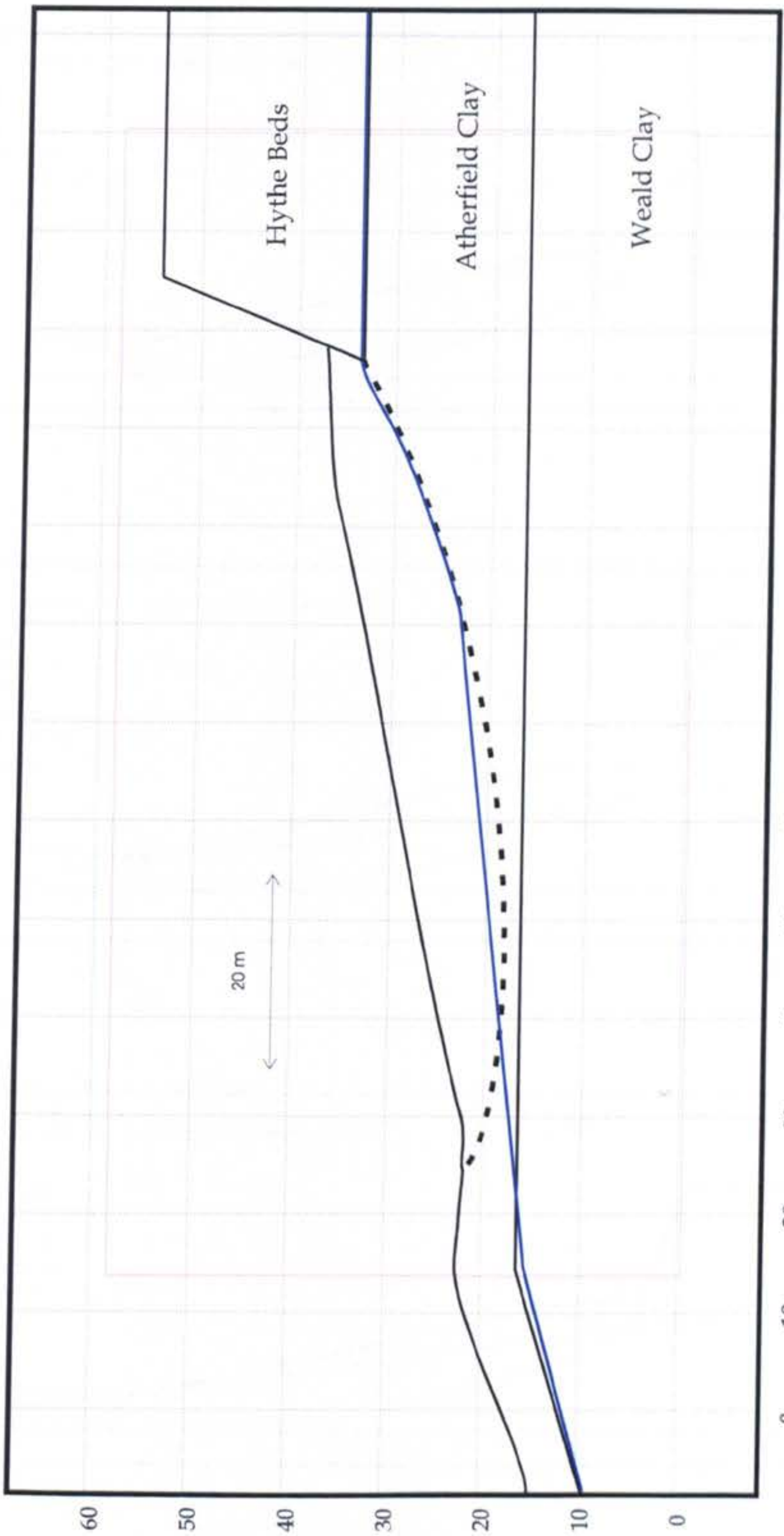
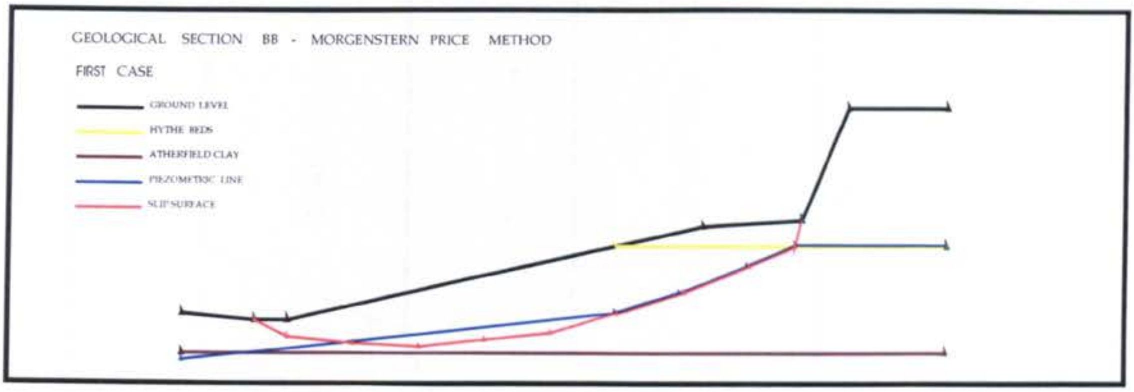
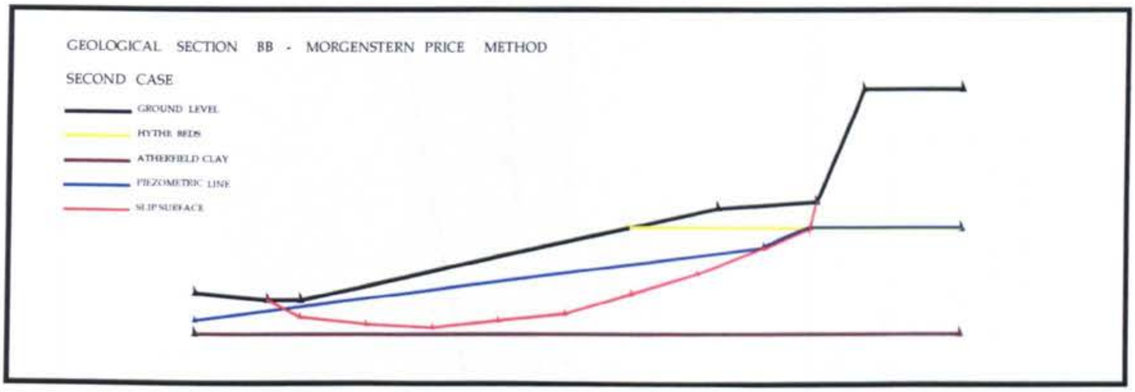


Figure 9.10.Present day situation on The Roughs, used as the model for the Morgenstern and Price analysis of the rotational zone.





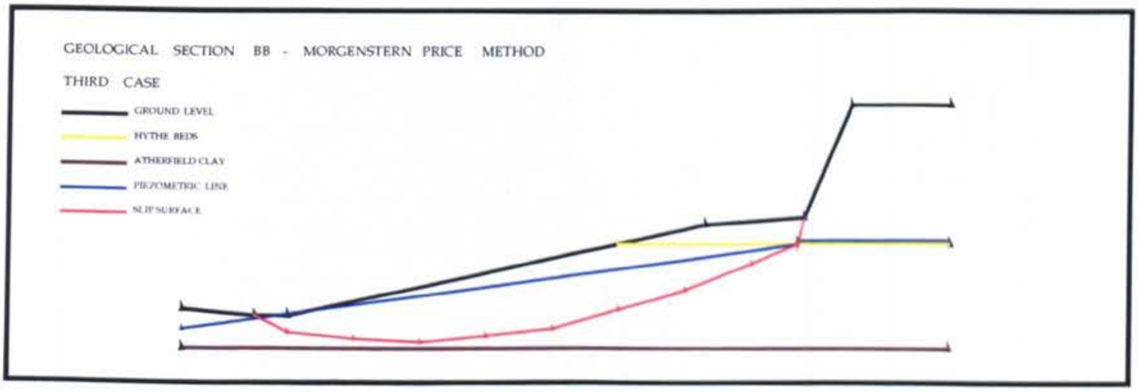


Figure 9.11. The first three cases analysed by the Morgenstern and Price method.

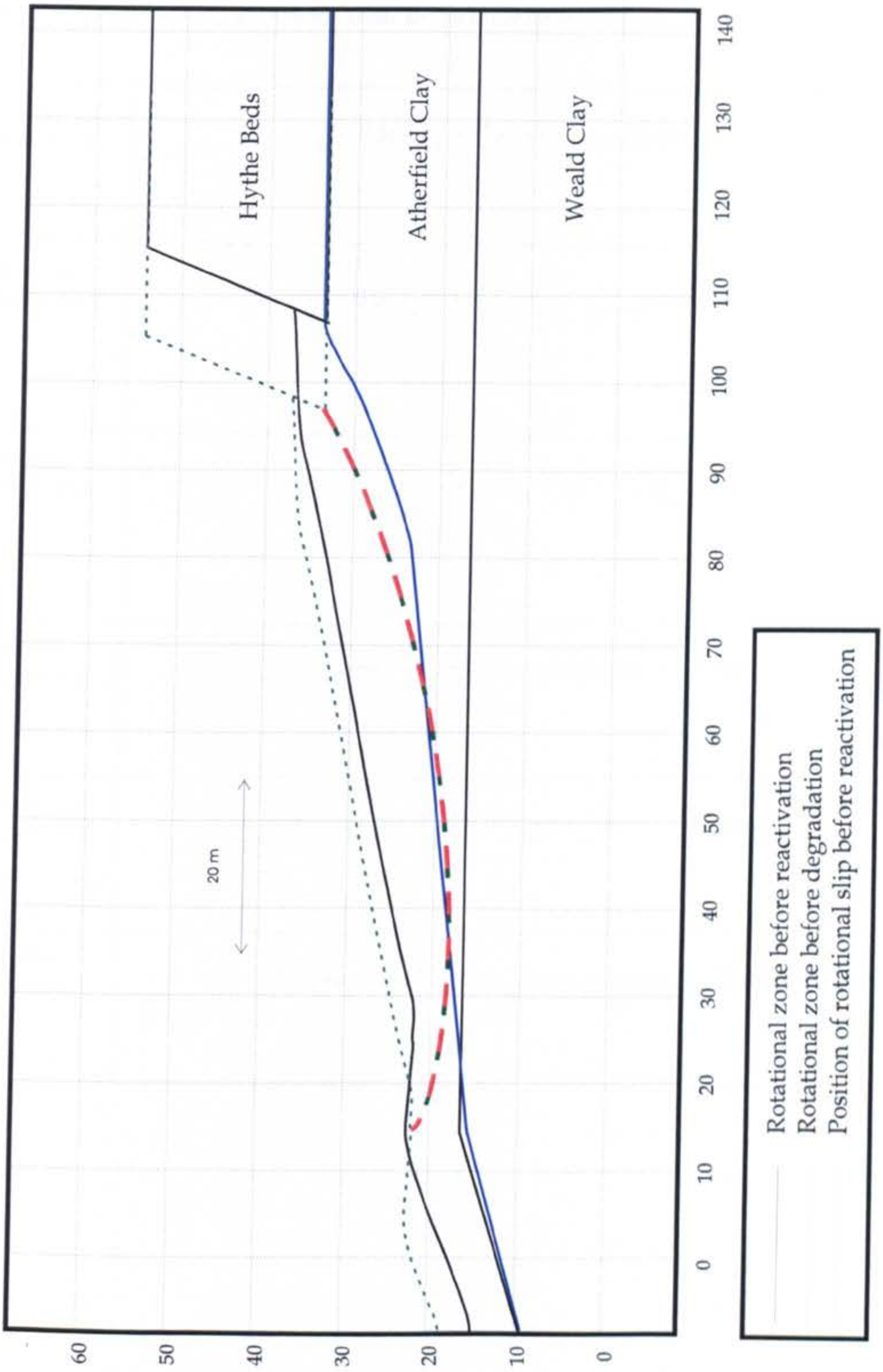


Figure 9.12.Assumed situation on The Roughs before the 1988 reactivation.

Table 9.1. Definitions of equations for shear in the finite slope method.

Side shear active block	$S_a = 0.5c'L_a z_a + \frac{1}{3}K_o(\gamma z_a - \gamma_w h_{wa}) \tan\varphi' L_a z_a$
Side shear passive block	$S_p = 0.5c'L_p z_p + \frac{1}{3}K_o(\gamma z_p - \gamma_w h_{wp}) \tan\varphi' L_p z_p$
Side shear central block	$S_s = 0.5c'L_a(z_a + z_p) + \frac{1}{4}K_o(\gamma(z_a + z_p) - \gamma_w(h_{wa} - h_{wp}))\tan\varphi' L_a z_a$
Base shear active block	$P_a = (W_a\cos\alpha_a - 0.5(h_{wa}B)\gamma_w\sec\alpha_a)\tan\varphi' + c'L_a\sec\alpha_a\ B$
Base shear passive block	$P_p = (W_p\cos\alpha_p - 0.5(h_{wp}B)\gamma_w\sec\alpha_p)\tan\varphi' + c'L_p\sec\alpha_p\ B$
Base shear central block	$P_s = (W_s\cos\alpha_s - 0.5(h_{wa} + h_{wp})B\gamma_w\sec\alpha_s)\tan\varphi' + c'L_s\sec\alpha_a\ B$

Table 9.2. Programme input data for the infinite and finite slope analyses.

Case no.	φ	β	δ	DT	DH	L	B	PT	PH
1	15	11	11	4	5	150	500	2.5	-2
2	15	11	11	7	4	150	500	5.5	2.5
3	15	11	11	7	5	300	500	5.5	-2
4	15	11	11	6.5	3.5	150	500	5	2
5	15	11	11	5.5	2.5	150	500	4	1
6	15	11	11	7	5	300	500	6.5	-1
7	15	11	11	7	5	300	500	7	-0.5
8	15	11	11	7	5	300	500	7	0
9	15	11	11	7	5	400	500	5.5	-2
10	15	11	11	7	5	200	500	5.5	-2
11	15	11	11	7	5	50	500	5.5	-2
12	15	11	11	7	5	300	600	5.5	-2
13	15	11	11	7	5	300	200	5.5	-2
14	15	11	11	7	5	300	50	5.5	-2
15	12	11	11	7	5	300	500	5.5	-2
16	20	11	11	7	5	300	500	5.5	-2
17	25	11	11	7	5	300	500	5.5	-2
18	15	10	10	7	5	300	500	5.5	-2
19	15	12	12	7	5	300	500	5.5	-2
20	15	14	14	7	5	300	500	5.5	

Table 9.3. Results from the programme Haefeli.

Output	1	2	3	4	5	6	7	8
AWA	34.25	34.25	34.25	34.25	34.25	34.25	34.25	34.25
PWA	8.25	8.25	8.25	8.25	8.25	8.25	8.25	8.25
MD	4.5	5.5	6	5	4	6	6	6
AW	9.25	11.3	12.33	10.28	8.22	12.33	12.33	12.33
PW	13.26	16.2	17.68	14.73	11.79	17.68	17.68	17.68
ML	22.51	27.51	30.01	25.01	20.01	30.01	30.01	30.01
PL	127.49	122.49	269.99	124.99	129.99	269.99	269.99	269.99
F_{dry}	1.38	1.38	1.38	1.38	1.38	1.38	1.38	1.38
F_{gwl}	0.69	0.69	0.69	0.69	0.69	0.69	0.69	0.69
F_{ave}	1.34	0.88	1.18	0.90	0.95	1.06	1.01	0.98
W_{aw}	231221	226082	308294	179838	102764	308294	308294	308294
DF_{aw}	130139	127247	173518	101219	57839	173518	173518	173518
W_{mb}	5737178	6737019	16199430	6249603	5199746	16199430	16199430	16199430
DF_{mb}	1094705	1285484	3090997	1192480	992158	3090997	3090997	3090997
W_{pw}	265166	567161	618721	478772	324092	618721	618721	618721
DF_{pw}	-38058	-81403	-88803	-68717	-46516	-88803	-88803	-88803
F_{dry}	1.37	1.48	1.41	1.48	1.47	1.41	1.41	1.41
RF_{dry}	1630554	1972482	4493224	1810598	1476371	4493224	4493224	4493224
Fss_{dry}	1.38	1.49	1.43	1.49	1.48	1.43	1.43	1.43
$RFss_{dry}$	1641396	1988490	453386	1824001	1485172	4533386	4533386	4533386
MB_{dry}	91.94	89.11	93.99	90.12	92.09	93.99	93.99	93.99
TB_{dry}	4.28	7.56	3.62	6.96	5.79	3.62	3.62	3.62
SS_{dry}	0.66	0.81	0.89	0.73	0.59	0.89	0.89	0.89
F_{hyd}	0.65	0.71	0.68	0.71	0.70	0.68	0.68	0.68
RF_{hyd}	774157	939574	2148561	863675	706167	2148561	2148561	2148561
Fss_{hyd}	0.66	0.71	0.68	0.71	0.71	0.68	0.68	0.68
$RFss_{hyd}$	779577	947578	2168642	870376	710567	2168642	2168642	2168642
MB_{hyd}	93.13	89.97	94.53	90.86	92.6	94.53	94.53	94.53
TB_{spec}	4.41	7.77	3.7	7.14	5.92	3.70	3.70	3.70
SS_{hyd}	0.70	0.84	0.93	0.77	0.62	0.93	0.93	0.93
F_{spec}	1.33	0.92	1.2	0.94	0.99	1.07	1.01	0.98
RF_{spec}	1579608	1220537	3802545	1147013	994254	3412097	3216872	3119753
Fss_{spec}	1.34	0.92	1.21	0.94	1	1.08	1.02	0.99
$RFss_{spec}$	1590138	1230577	3836663	1155596	1000211	3442879	3245986	3148038
MB_{spec}	92.16	89.66	94.25	90.58	92.39	94.33	94.37	94.38
TB_{spec}	3.01	7.32	2.56	6.67	5.4	2.51	2.47	2.55
SS_{spec}	0.66	0.82	0.89	0.74	0.60	0.89	0.90	0.90

Output	9	10	11	12	13	14	15	16
AWA	34.25	34.25	34.25	34.25	34.25	34.25	23.2	43.55
PWA	8.25	8.25	8.25	8.25	8.25	8.25	0.20	12.55
MD	6	6	6	6	6	6	6	6
AW	12.33	12.33	12.33	12.33	12.33	12.33	25.61	7.94
PW	17.68	17.68	17.68	17.68	17.68	17.68	30.32	14.39
ML	30.01	30.01	30.01	30.01	30.01	30.01	55.93	22.33
PL	369.99	169.9	19.99	269.99	269.99	269.99	244.07	277.67
F_{dry}	1.38	1.38	1.38	1.38	1.38	1.38	1.09	1.87
F_{gwl}	0.69	0.69	0.69	0.69	0.69	0.69	0.55	0.94
F_{are}	1.18	1.18	1.18	1.18	1.18	1.18	0.93	1.60
W_{aw}	308294	308294	308294	369953	123317	30829	640365	198391
DF_{aw}	173518	173518	173518	208222	69407	17351	252277	136676
W_{mb}	22199430	10199430	1199428	19439310	6479771	1619943	14643920	16660340
DF_{mb}	4235850	1946143	228861	3709196	1236399	309099	2794191	3178944
W_{pw}	618721	618721	618721	742465	247488	61872	1061203	503717
DF_{pw}	-88803	-88803	-88803	-106564	-35521	-8880	-3723	-109411
F_{dry}	1.41	1.44	1.75	1.41	1.41	1.41	1.12	1.93
RF_{dry}	6071381	2915067	547831	5391869	1797290	449322	3406143	6183759
Fss_{dry}	1.42	1.45	1.76	1.43	1.45	1.54	1.13	1.94
$RFss_{dry}$	6125842	2940929	552244	5432030	1837451	489483	3438115	6233081
MB_{dry}	95.32	91.22	57.13	94.13	92.76	87.05	88.87	95.5
TB_{dry}	2.68	5.58	29.71	3.62	3.57	3.35	6.56	2.87
SS_{dry}	0.89	0.88	0.80	0.74	2.19	8.20	0.93	0.79
F_{hyd}	0.67	0.68	0.80	0.68	0.68	0.68	0.54	0.92
RF_{hyd}	2907825	1389297	25040	2578273	859424	214856	1633855	2951355
Fss_{hyd}	0.68	0.69	0.81	0.68	0.69	0.74	0.54	0.93
$RFss_{hyd}$	2935056	1402228	252607	2598354	879505	234936	1649841	2976016
MB_{hyd}	95.71	92.04	60.09	94.67	93.23	87.26	89.1	96.23
TB_{spec}	2.74	5.73	31.79	3.71	3.65	3.42	6.84	2.86
SS_{hyd}	0.93	0.92	0.87	0.77	2.28	8.55	0.97	0.83
F_{spec}	1.19	1.21	1.45	1.20	1.20	1.20	0.95	1.63
RF_{spec}	5141859	2463232	454261	4563054	1521018	380254	2884720	5229030
Fss_{spec}	1.20	1.22	1.46	1.21	1.22	1.30	0.96	1.64
$RFss_{spec}$	5188191	2485136	457845	4597173	1555137	414372	2911862	5270970
MB_{spec}	95.51	91.61	58.48	94.39	93.01	87.26	89.05	95.84
TB_{spec}	1.89	3.95	21.46	2.56	2.53	2.37	4.7	2
SS_{spec}	0.89	0.88	0.78	0.74	2.19	8.23	0.93	0.79

Output	17	18	19	20				
AWA	49.58	36.43	31.78	24.91				
PWA	13.58	11.43	4.78	-4.09				
MD	6	6	6	6				
AW	6.12	10.68	14.75	27.90				
PW	13.76	15.85	20.26	33.74				
ML	19.89	26.53	35.01	61.65				
PL	280.11	273.47	264.99	238.35				
F_{dry}	2.4	1.52	1.26	1.07				
F_{gwl}	1.2	0.76	0.63	0.54				
F_{ave}	2.05	1.30	1.08	0.92				
W_{aw}	153093	267015	368642	697522				
DF_{aw}	116552	158567	194128	293780				
W_{mb}	16806780	16408090	15899510	14301250				
DF_{mb}	3206884	2849236	3305694	3459786				
W_{pw}	481716	554790	709187	1181072				
DF_{pw}	'-113110	-109950	-59051	84259				
F_{dry}	2.48	1.56	1.29	1.10				
RF_{dry}	7957774	4533014	444052	4203358				
Fss_{dry}	2.50	1.58	1.3	1.1				
$RFss_{dry}$	8013559	4573500	4480196	4240553				
MB_{dry}	96.00	94.67	93.01	87.68				
TB_{dry}	2.72	3.19	4.23	7.44				
SS_{dry}	0.70	0.89	0.89	0.88				
F_{hyd}	1.18	0.75	0.61	0.51				
RF_{hyd}	3795271	2180540	2109342	1967024				
Fss_{hyd}	1.19	0.76	0.62	0.52				
$RFss_{hyd}$	3823164	2200783	2129189	1985622				
MB_{hyd}	96.81	95.31	93.44	87.81				
TB_{spec}	2.69	3.18	4.42	7.91				
SS_{hyd}	0.73	0.92	0.93	0.94				
F_{spec}	2.09	1.33	1.09	0.92				
RF_{spec}	6724703	3840164	3753659	3543985				
Fss_{spec}	2.11	1.34	1.10	0.93				
$RFss_{spec}$	6772028	3874568	3787368	3575520				
MB_{spec}	96.41	94.94	93.26	87.88				
TB_{spec}	1.88	2.22	3.02	5.34				
SS_{spec}	0.70	0.89	0.89	0.88				

The infinite slope method
Output data

Active wedge angle	AWA
Passive wedge angle	PWA
Mean depth	MD
Active width	AW
Passive width	PW
Minimum length	ML
Plan length of central section	PL

For comparison with infinite slope method

Factor of safety dry	F_{dry}
Factor of safety at ground water level	F_{gwl}
Average factor of safety	F_{ave}

Driving force

Weight of active wedge (kN)	W_{aw}	Driving force (active wedge) (kN)	Df_{aw}
Weight of main body (kN)	W_{mb}	Driving force (main body) (kN)	Df_{mb}
Weight of passive wedge (kN)	W_{pw}	Driving force (passive wedge) (kN)	Df_{pw}

Finite slope analysis dry (zero pore water pressure)

Factor of safety with no side shear	F_{dry}
Resisting force with no side shear (kN)	RF_{dry}
Factor of safety with side shear	Fss_{dry}
Resisting force with side shear (kN)	$RFss_{dry}$

Proportions

Main body %	MB_{dry}
Toe breakout %	TB_{dry}
Side shear %	Ss_{dry}

Finite slope analysis with hydrostatic conditions(water at ground level)

Factor of safety with no side shear	F_{hyd}
Resisting force with no side shear (kN)	RF_{hyd}
Factor of safety with side shear	Fss_{hyd}
Resisting force with side shear (kN)	$RFss_{hyd}$

Proportions

Main body %	MB_{hyd}
Toe breakout %	TB_{hyd}
Side shear %	SS_{hyd}

Finite slope analysis as specified

Factor of safety with no side shear	F_{spec}
Resisting force with no side shear (kN)	RF_{spec}
Factor of safety with side shear	Fss_{spec}
Resisting force with side shear (kN)	$RFss_{spec}$

Proportions

Main body %	MB_{spec}
Toe breakout %	TB_{spec}
Side shear %	SS_{spec}

Table 9.4. Results of the Morgenstern and Price analysis.

Case	Average effective stress			Average shear stress			Average R_u			Factor of safety Morgenstern and Price	Factor of safety Janbu
	Stratum 1	Stratum 2	Whole slip	Stratum 1	Stratum 2	Whole slip	Stratum 1	Stratum 2	Whole slip		
First	9.84	141.64	135.55	5.98	26.29	25.35	.0000	.0368	.0351	.9112	.9204
Second	8.63	113.26	108.42	6.55	26.24	25.33	.0000	.2163	.2063	.7250	.7395
Third	7.87	98.42	94.23	6.86	26.21	25.32	.0000	.3286	.3134	.6273	.6433
Fourth	.00	111.72	111.72	.00	18.90	18.90	.0000	.0121	.0121	.9717	1.0045

Summary and conclusions

10.1 List of Points

This research project set out to answer a number of questions relating to a recently reactivated landslide on the Lower Greensand Escarpment of South Kent. The information collected and work undertaken, can be divided in a number of separate categories.

i. Geology and structure
ii. Description and history of the escarpment
iii. Relationship between landslide type and geology
iv. Fieldwork study
v. Landslide activity
vi. Climatic influence on activity
vii. Rates of movement and movement models
viii. Shear strength and back-analysis
ix. New insights into slope movement at Stutfall Castle
x. Suggestions for future research

No research is carried out in isolation, and this project is no exception. Indeed, because of its duration, some of the derivative work has already come to fruition and been published, and a paper (**Bromhead, Hopper and Ibsen, 1998**) has been published, which describes some major parts of the study. **Hutchinson** (2000) argues that knowledge is of two kinds: qualitative and quantitative. In engineering, the former is more highly valued, although society at large probably holds the opposite view. The findings of this research project largely add to qualitative knowledge, often confirming beliefs or demonstrating that one of a number of hypotheses is more likely to be correct.

10.2 Geology and structure

The geology and structure of the Wealden District was examined as a whole. It was established by walkover survey and visual inspection that geology at The Roughs site is conformable with the overall structure of the Wealden Dome. The outcrop of the Lower Greensand runs in an ellipse around the edge of the Wealden District and consists of, in ascending order, Atherfield Clay, Hythe Beds, Bargate Beds, Sandgate Beds and Folkestone Beds. It was determined that only the first two of these strata overlay the Weald Clay on The Roughs although there are outcrops of Sandgate Beds in the vicinity.

The Lower Greensand Escarpment from Aldington to Hythe, known as the Lympne Escarpment was studied further. It was confirmed that the current morphology of The Roughs is due to the protection from erosion afforded by the formation of Romney Marsh in recent geological history. The Roughs were once under attack from the sea and an abandoned cliff was formed at the base of the slope as the sea levels rose and fell during the Devensian Glaciation. As Romney marsh developed, from the west of the escarpment towards the east, the slope was able to undergo a process of free degradation. Therefore, the slope is more highly degraded towards the west of the escarpment. The variation of the geomorphological units along the escarpment were examined. Most of the features could be attributed to the difference in the number of years of degradation. For example, the western end of the slope is shallower, has a thicker accumulation zone, less evidence of landslide features and is more stable than the eastern end.

10.3 Description and history of the Lower Greensand Escarpment

At first sight, The Roughs and, in fact most of the Lympne Escarpment, do not seem to hold any importance and landslide damage would apparently not be of any great significance. After research into the history of the area and its current standing, it can be seen that the first impressions were wrong and the whole area is indeed of 'high societal value'.

Hythe is one of the Cinque Ports and has been since 1278. Shepway Cross, on the brow of the Lympne escarpment, marks the meeting point of the Cinque Port dignitaries. Also of historical significance are the Martello Towers, the Royal Military Canal and the Royal Military Road which date back to the early 19th Century.

There are several small towns and villages, situated on or near the escarpment, which are at risk of damage if (or when) subsequent landslides occur. The Lympne Escarpment itself is a Site of Special Scientific Interest. The Roughs itself is used for grazing and occasionally military exercises. It was also the site of an Acoustic Research Centre between the Wars and one concrete listening mirror still survives. The construction of the centre involved a lot of landscaping of the slope and the construction of drains. Both of these features have had a significant effect on The Roughs and were probably central to the movements of 1988.

10.4 Relationship between landslide type and geology

To understand the nature of the land movements on The Roughs, a study of local landslides both active and relict was undertaken. There are two very well-known landslides or landslide areas not far from The Roughs. The first of these is the Encombe landslip, Sandgate ,which took place in 1827, which is well-documented and well-researched. At Sandgate, the critical bed is the Atherfield

Clay, the base of which is below sea level. The consensus of opinion is that the landslip was triggered by heavy rains, possibly in conjunction with high tides. Remedial measures have prevented any reoccurrences. Folkestone Warren is the site of many landslips and rock falls and is also well-researched. In this case, the critical bed is the Gault, accompanied with occasional rock fall in the Chalk. These landslides are also thought to be water driven, in conjunction with tidal erosion.

One of the most interesting landslides as far as research into The Roughs goes, is that at Lympne. In 1728 a large landslide took place at French house which was probably responsible for further destroying the ruins of the Roman Fort Lemanis. A combined archaeological and geotechnical investigation took place there. This suggested that a perched rotational slides took place in the Atherfield Clay, with a translational slide below in the Weald.

Two main types of slide in the area were identified. The first type, such as the Encombe Landslide or the slips at Folkestone, has a basal shear surface at or below sea level. The other, such as French House, is 'perched' with a basal shear surface at elevation. Primarily, the local dip towards the north-east controls the type of landslide since the critical beds are at or below sea level towards the east and above sea level to the west.

10.5 Fieldwork study

Fieldwork constituted a major part of the research into the landslides on The Roughs. With help from the Royal School of Military Engineers as part of the Joining Forces for the Environment scheme, and a contribution from industrial collaborators Soils Ltd., an extensive programme of fieldwork was undertaken. This involved a series of boreholes and trialpits on two cross-sections and subsequent logging and testing of samples.

Two geological cross-sections through the slope, AA and BB were then constructed. Section BB is the model for the stability analysis, although it is a post-slide section when a pre-slide section is actually needed to calculate the factor of safety of The Roughs at the beginning of 1988. The findings were, as expected, a perched rotational slide in the Atherfield Clay and a translational slide in the Weald Clay. The only position on the slope where all the strata exist in situ is at the rear scarp. It was difficult to determine the exact position of the shear surfaces due to the amount of material which had moved 'en masse' and appeared to be in situ. Finally, a combination of inclinometer readings, piezometer readings and sample moisture contents were used to determine the positions.

10. 6 Rates of movement and movement models

The commonly held view, that landslides on pre-existing shears are of a slow "creep" nature was re-examined. These movements are, in fact, explained by Picarelli's model of creep type movements, on a slope with an overall factor of safety well above one. This model suggests that some areas on a slope may be have a low angle of shear resistance, a steeper slope angle or have a localised high water table and, therefore, have a factor of safety below one. This describes the current situation on The Roughs, which is still moving slowly. However, it does not explain the large and fast movements in 1988. When the equations of force and motion were looked at, in the context of landslide movement, it was seen that there was no reason why slides, on pre-existing shears, should not move far and fast. The movements on The Roughs were large because of the long time period during which the ground water levels were high enough to bring the factor of safety to below one. Changes in morphology acted as a brake to the movements.

10.7 Landslide activity

Research showed that the incidence of landsliding in the UK has been greater than at the present time. During the Devensian, periglacial landslides occurred even on very shallow slopes. A survey by the Department of the Environment, shows that by far the majority of landslides are of unknown age. This would imply that they are in either the relict (100-1000 years old) or fossil (historic or prehistoric) categories, since, even recent (100-1000 years old) slides would be remembered or documented in some way. Of those which have been dated, coastal landslides are predominantly either active (within the last five years) or recent and inland slides, recent or relict.

10.8 Climatic influence on activity

It is commonly believed that there is a strong connection between the onset of landsliding and heavy rainfall. The question is, how much and for how long, does it have to rain, to trigger land movement.

Firstly, the concept of the seasonal variation of factor of safety was discussed, showing how a wet weather period could destabilise a landslide. To investigate this further, three sets of rainfall data were analysed using a variety of statistical methods. Primarily, extended wet weather periods preceding land movements were looked for. It was found that at Ventnor, Sandgate, Folkestone and The Roughs, there was a firm correlation between the two. The larger, deep-set landslides need longer periods of wet weather to move. Another point was brought to light was that not all wet weather periods precede a landslide. It was suggested that either the wet weather period would have to be followed by a shorter intense period of rain, for the landside to be triggered or that a recently activated landslide might be too stable to move.

10.9 Shear strength and back-analysis

The better of the two cross-sections, section BB, was used as a model to analyse the stability of the slope. For analysis, the slope was split into two sections: the translational slide zone and the rotational slide zone.

Both the *infinite* and *finite* slope methods were used for the analysis of the translational section. It was found that the results for the two were similar, unless very narrow or very short slopes were considered. In these cases, the factor of safety was raised considerably by side shear and end effects, respectively. If the zone was considered as a whole, then, using the finite slope method, the factor of safety was 1.21. When the accumulation zone and the degradation zone were analysed separately, then their factors of safety were found to be 0.92 and 1.34. It was found that the slope angel and the angle of shear resistance were the critical parameters in the stability analyses.

The Morgenstern and Price method was used for the analysis of the rotational zone.

It has to be born in mind, that Section BB is a representation of the situation of the slope after the reactivation took place. The conditions before the land movements of 1988 were very different and the stability analysis of that slope would have yielded very different results. Therefore, when the sequence of events, during the period of landsliding, was deciphered, the eye-witness account of Jill Edison was of primary importance. Using this information, in conjunction with the rainfall analysis and the stability analysis, it was put forward that the sequence of events were as follows.

I. Wet weather period changed the hydrology of the slope and new springlines were created. These springlines emerged form the base of the zone of rotated blocks, at the head of the degradation zone.

II. The water levels in the degradation zone, already at field capacity, were raised to critical levels.

III. A combination of mudslides and translation sliding ensued.

IV. The movement of the degradation zone unloaded the toe of the rotational zone, activating the rotational slides.

10.10 New insights into slope movement at Stutfall Castle

Prior to undertaking the investigations of the project at Lympne, a great deal was known, or could be inferred, about the site. The geological sequence and structure was understood from the Geological Survey's mapping. The investigations at Lympne showed that the slope was formed from different slide elements. It was fully appreciated that even if there was an upper, compound slide at Lympne the details of it would be obscured by Roman and later quarrying. The volumes of masonry and concrete in the Roman fort and medieval Lympne Castle, built from a minor constituent of the Hythe Beds, required massive excavation both into the rear scarp of the landslides and into the landslide bodies in the upper slide complex, and would have generated large volumes of discarded sand. It is not surprising that the details of the upper slide complex eluded the team working at Lympne.

With a fuller understanding of the nature of the landslide activity in The Roughs, it was possible to re-interpret the damage to Lympne Castle. In their 1985 paper, **Hutchinson et al.** had begun to appreciate that the east wall damage was due to a mudslide reaching (as does the major mudslide at the Roughs) down to the marsh level. At Lympne, the upper slide was identified because of the pond (now artificial, but developed from a smaller natural pond), together with the side shears (particularly well developed at its eastern end, and comparable with the west end of the Roughs slide). However, damage to the north west wall was attributed to movement of the accumulation zone.

The work at The Roughs made it necessary to re-examine the record at Lympne. It is now believed that the north west wall damage occurred in 1725, during what is known as the "French House landslip" (**Topley, 1893**). Evidence for this comes from the 1722 Stukeley engraving (*Figure 4.14*) which shows the north west wall still standing and the understanding of the nature of The Roughs landslide movements.

Present day geomorphological expression of the extent of the French House landslip is made more difficult by the construction of sewage filter beds on the upper slip platform (now removed and landscaped) for Lympne Airfield, together with difficulties in accessing the site, now a part of the Port Lympne animal reserve, and the above insights could only be gained after a study of The Roughs.

10.11 Suggestions for future research

Ideally, but impractically, geotechnical investigations should be carried out on the Lympne Escarpment before the onset of landsliding. Monitoring of inclinometers and piezometers before, during and after sliding would give a much clearer picture of the events. The question has to be, which part of the site to investigate and there can be no guaranteed correct answer. Probably, subsequent slides will take place between the research site on The Roughs and the town of Hythe, beneath a spring. If the incidence of landsliding is indeed rising, due to the trend of increased rainfall, then there may be an increased chance of choosing an appropriate site. More easily, the areas in which activity is most likely could be accurately surveyed and photographed.

It is clear that wet weather periods followed, in some case, by a storm effect, is the reactivation trigger of many landslides. However, the exact mechanism behind this is not clear and would be another topic to investigate further.

Neither the analyses of the translational/accumulation zone and the rotational zone took into account the oblique dip of the strata on The Roughs. The effect of side shear was not included in the Morgenstern Price analysis. Both of these could be omissions could be amended in the future.

10.12 The answers

A number of questions were posed in the introduction. It was the aim of this research to answer these questions.

What, exactly triggered The Roughs landslides?

The landslides were triggered by an extended period of wet weather, which had caused a change in hydrology on the site.

What was the mechanisms of these landslides?

New or increased-flow springlines, emerging from the base of the zone of rotated blocks, scoured the base of the rotational slide. The increased volume of water at the head of the translational slide, which was already at field capacity, destabilised it and caused mud-slides and translational slides to propagate down the slope. This activated the rotational slides as their toe-loading was removed.

Why did they move relatively 'fast' and 'far'?

The factor of safety on the landslide was reduced to below one by a rise in the ground water level. These condition continued for a period of weeks, until a change in morphology of the slope, mostly in the form of increased toe-resistance, finally brought the movements to a halt.

What significance does the reactivation of The Roughs have on other 'stable' landslides?

The wet weather period which triggered the reactivation of The Roughs is thought to be part of a change in the climate, which will cause increasing rainfall in the near future. This ultimately, will have an effect on many dormant landslides and will activate many new ones.

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Appendix 6.2: Borehole log for Shell and Auger boreholes

SA1 Laboratory Log		
Sample	Description	Depth/m
U1	Mottled light grey and light brown TOP SOIL with roots and fine gravel to large boulders of LIMESTONE	0.0-1.0
U3	Soft to firm orange and brown mottled sandy clay with small black patches [COLLUVIUM]	1.0-1.45
U5 U7	Light yellowish brown medium grained sand [COLLUVIUM]	1.45-2.7
U9	Soft to firm grey mottled orangish brown sandy structure-less clay	2.7-3.0
U11 U13	Light yellowish brown medium grained sand [COLLUVIUM]	3.0-4.3
U15	Soft to firm light grey sandy clay with bands of iron staining	4.3-4.5
U17	Mixed layers of landslipped [COLLUVIUM]	4.5-4.7
U17	Soft to firm light grey mottled light yellow clay [COLLUVIUM]	4.7-5.0
U19	Soft brownish grey mottled black clay surrounded by light yellowish and orangish brown mottled medium grained sand [COLLUVIUM]	5.0-5.45
U21	Light yellowish brown and orangish brown mottled medium grained sand [COLLUVIUM]	5.45-5.75
U21	Stiff brownish grey mottled grey and light yellowish brown clay [COLLUVIUM]	5.75-6.0
U23	Firm light brownish grey mottled grey clay [COLLUVIUM]	6.0-6.5
U25	Firm grey mottled light grey and yellowish brown clay [COLLUVIUM]	6.5-7.0
U28	Firm grey mottled dark grey and yellowish brown clay [COLLUVIUM]	7.0-7.45
U30	Firm light grey mottled yellowish brown clay [COLLUVIUM]	7.45-7.9
U32	Firm greyish brown mottled light grey and orangish brown clay [WEALD]	7.9-8.45
U34	Firm to stiff greyish brown mottled grey and yellowish brown clay with structure and fissuring [WEALD]	8.45-9.0
U36 U38	Stiff grey mottled yellowish brown and white clay [WEALD]	9.0-10.0
U40	Stiff grey mottled light yellowish brown clay becoming silty with depth [WEALD]	10.0-10.4

Appendix 6.2: Borehole log for Shell and Auger boreholes

SA2 Laboratory Log		
Sample	Description	Depth/m
U1	Mid brown silty clay with Sandstone boulders [COLLUVIUM]	0.0-0.7
U3 U5 U7 U8 U11 U14	Light yellow soft clayey sandy silt [COLLUVIUM]	0.7-3.95
U16	Soft light greenish brown mottled orange and grey slightly sandy clay [COLLUVIUM]	3.95-4.3
U16	Stiff light greenish brown mottled orange and grey slightly sandy clay [COLLUVIUM]	4.3-4.65
U18	Very soft light greenish brown mottled orange and grey sandy clay [COLLUVIUM]	4.65-4.9
U18	Firm light greenish brown mottled orange and grey sandy clay [COLLUVIUM]	4.9-5.2
U20	Very soft light greenish brown mottled orange and grey sandy clay [COLLUVIUM]	5.2-5.55
U20	Firm light greenish brown mottled orange and grey sandy clay [COLLUVIUM]	5.55-5.9
U22	Firm to stiff light greenish brown mottled orange and grey sandy clay [COLLUVIUM]	5.9-6.4
U24	Soft to firm light greenish brown sandy clay [COLLUVIUM]	6.4-7.0
U26 U28	Very stiff dark grey fissured clay with occasional rock and shell fragments [COLLUVIUM]	7.0-8.0
U30	Very stiff brownish grey fissured clay [COLLUVIUM] with occasional shell fragments.	8.0-8.3
U30	Soft to firm light greenish brown mottled grey sandy clay [COLLUVIUM]	8.3-8.6
U32	Very stiff greyish brown clay with horizontal bedding and extremely closely spaced fissures [ATHERFIELD]	8.6-9.1
U34	Very stiff brownish grey horizontally bedded very closely fissured clay [ATHERFIELD]	9.1-9.6
U36	Very stiff brown mottled grey thinly laminated clay [ATHERFIELD]	9.6-10.0
U38	V. stiff brown mottled grey v. closely spaced laminated clay with occasional calcareous claystone nodules [ATHERFIELD]	10.0-10.5
U40	Very stiff brown and orange mottled thinly laminated clay [ATHERFIELD]	10.5-10.9
U42	Very stiff grey mottled orangish brown thinly sub-horizontally laminated fissured clay [ATHERFIELD]	10.9-11.4

Appendix 6.2: Borehole log for Shell and Auger boreholes

SA2 Laboratory Log		
Sample	Description	Depth/m
U44	Very stiff grey mottled orangish brown and white sub-horizontally laminated clay with silt partings [WEALD]	11.4-12.0
U46	Very stiff grey mottled orangish brown and white horizontally closely laminated clay [WEALD]	12.0-12.6
U48	V. stiff dark grey horizontally laminated clay speckled with calciferous deposits [WEALD]	12.6-12.9
U50	Very stiff dark and light grey mottled horizontally laminated clay and silt [WEALD]	12.9-13.4

Appendix 6.2: Borehole log for Shell and Auger boreholes

SA3 Laboratory Log		
Sample	Description	Depth/m
	Very stiff dark brown clayey sandy TOP SOIL with sub-angular LIMESTONE gravel	0.0-0.45
U1	Very stiff light greenish brown sandy clayey TOP SOIL with pebbles and sub-angular boulders with fine roots	0.45-1.0
U3	Very stiff light greenish brown clay with sub-angular gravel and cobbles of [LIMESTONE] of light grey and light [COLLUVIUM]	1.0-1.6
U5	Very stiff light brown sandy clay becoming greenish grey with depth [COLLUVIUM]	1.6-1.9
U7	Very stiff greenish grey sandy clay with vertical bedding [COLLUVIUM]	1.9-2.2
U7	Very stiff light yellowish brown very clayey sand [COLLUVIUM]	2.2-2.45
U9	Light yellowish brown clayey medium grained sand [COLLUVIUM]	2.45-2.95
U11	Light yellowish brown mottled light grey clayey sand with iron stained marbling [COLLUVIUM]	2.95-3.4
U13	Stiff light yellowish brown mottled black and light grey clayey sand [COLLUVIUM]	3.4-3.7
U13	Stiff light yellowish brown clayey sand [COLLUVIUM]	3.7-3.9
U15	Very stiff light yellowish brown SANDSTONE [COLLUVIUM]	3.9-4.25
U17	Stiff light yellowish brown clayey sand [COLLUVIUM]	4.25-4.6
U17	Very stiff light yellowish brown SANDSTONE [COLLUVIUM]	4.6-4.9
U19 U22	Very stiff light yellowish brown mottled grey clayey sand [COLLUVIUM]	4.9-5.8
U22 U24	Stiff light yellowish brown mottled light orangish brown and grey sandy clay [COLLUVIUM]	5.8-6.3
U24 U26	Stiff light yellowish brown and grey mottled sandy clay [COLLUVIUM]	6.3-6.6
U26	Stiff to very stiff brownish grey mottled orangish brown structure-less clay [COLLUVIUM]	6.6-7.0
U28	Weekly cemented light yellowish brown mottled orangish brown sandstone [COLLUVIUM]	7.0-7.3
U28	Very stiff brownish grey and brown mottled clay [ATHERFIELD]	7.3-7.5
U30	Very stiff grey clay [WEALD]	7.5-8.0
U32	Very stiff grey and brownish grey clay [WEALD]	8.0-8.5

Appendix 6.2: Borehole log for Shell and Auger boreholes

SA3 Laboratory Log		
Sample	Description	Depth/m
U34	Very stiff grey and light grey mottled sub-horizontally laminated fissured (horizontally and vertically) very silty clay [WEALD]	8.5-8.95
U36 U38	Stiff grey and brownish grey mottled orangish brown clay [WEALD]	8.95-9.9
U40 U42	Firm to stiff grey and brownish grey mottled orangish brown thinly laminated clay [WEALD]	9.9-10.9
U44	Very stiff grey mottled orangish brown clay [WEALD]	10.9-11.4
U48	Very stiff fissured clay [WEALD]	11.4-11.9
U48	Very stiff greyish brown fissured clay with shell and fossil fragments [WEALD]	11.9-12.4
U50	Very stiff grey sub-horizontally laminated clay with layers of yellowish grey silt [WEALD]	12.4-12.9
U52	Very stiff grey clay with partings of light yellowish grey silt [WEALD]	12.9-13.4
U54	Very stiff greyish brown sub-horizontally laminated clay with occasional shell fragments [WEALD]	13.4-14.0
U56 U58	Very stiff greyish brown sub-horizontally laminated clay with occasional white calciferous patches [WEALD]	14.0-14.9
U60 U62 U64 U66	Very stiff brown sub-horizontally laminated fissured clay with trace of sulphur (?) at depth [WEALD]	14.9-16.85

Appendix 6.2: Borehole log for Shell and Auger boreholes

PO1B Laboratory Log		
Sample	Description	Depth/m
U2	Firm greenish brown clayey sand [TOP SOIL] with white silt partings with sub-angular coarse gravel [LIMESTONE] in top 150 m [COLLUVIUM]	0.0-1.3
U3	Firm brownish-green medium grained clayey sand [COLLUVIUM]	1.3-1.55
U3	Firm brown sandy clay with white silt partings, occasional sub-angular medium to fine gravel [LIMESTONE] and [IRONSTONE?] [COLLUVIUM]	1.55-1.8
U5	Green silty sand [COLLUVIUM]	1.8-2.4
U7	Green mottled orangish-brown and purplish-brown medium grained sand with area of sub-angular reddish-purple coarse gravel [MUDSTONE][COLLUVIUM]	2.4-3.1
U8	Firm green mottled orangish brown becoming greenish- brown with depth clayey sand with yellowish-brown sand at bottom [CALCITE] layer (5 mm) across bottom in [SANDSTONE] layer [COLLUVIUM]	3.1-3.7
U9	Firm mottled green and brown clayey sand [HYTHE BEDS] with iron staining [COLLUVIUM]	3.7-4.25
U10	Firm brownish green and grey mottled clayey sand with occasional shell fragments and white silt partings, with iron staining and flints [COLLUVIUM]	4.25-4.8

Appendix 6.2: Borehole log for Shell and Auger boreholes

P02A Laboratory Log		
Sample	Description	Depth/m
U1	Firm brown clayey sand [TOPSOIL] with occasional roots and sub-angular cobbles [LIMESTONE][COLLUVIUM]	0.0-0.7
U4	Very stiff grey clay with traces of at top [COLLUVIUM]	6.8-7.0
U4	Very stiff grey clay [COLLUVIUM]	7.0-7.2
U6 U7 U8	Firm to stiff grey fissured clay with sub-angular medium to coarse gravel [LIMESTONE] inter-layered with yellowish-brown clayey sand (20 -50 mm) [COLLUVIUM]	7.2-8.5
U9	Stiff grey fissured clay [COLLUVIUM]	8.5-8.9
U10	Stiff grey fissured clay inter-layered with yellowish-brown sandy clay [COLLUVIUM]	8.9-9.4
U11 U12	Very stiff grey fissured clay with [LIMESTONE] cobbles inter-layered with yellowish-brown sand [COLLUVIUM]	9.4-10.25
U13 U14 U15 U16	Very stiff grey sub-horizontally fissured clay with occasional silt partings Slip surface: 11.5-6 m, slightly polished, 15-20 ° [WEALD]	10.25-11.6
U17 U18 U19 U20	Very stiff grey fissured clay with occasional silt partings (to very silty) and occasional shell fragments (some sheared) and slip surfaces [WEALD] Slip surface: 12.74 m, polished, gently striated Slip surface: 12.785 m Slip surfaces: 12.95-6 several sub-horizontally slightly polished striated Slip surfaces: a, b, c	11.6-13.3
U21	Very stiff grey fissured silty clay [WEALD]	13.3-13.6
U22 U23	Very stiff brownish-grey fissured clay with occasional shell fragments [WEALD]	13.6-13.8
U23 U24	Very stiff grey fissured clay with polished striated slip surfaces and numerous fossilized shells [WEALD]	13.8-14.55
REMARKS: a: 13.07 m perp. to c, b: 13.15 m, undulating slightly polished striated, 30°, c: 13.25 m, slightly polished and striated, 20°,		

Appendix 6.2: Borehole log for Shell and Auger boreholes

P02B Laboratory Log		
Sample	Description	Depth/m
2	Firm olive green mottled brown medium grained clayey sand [COLLUVIUM]	2.9-3.35
3	Firm olive green mottled brown and yellowish brown medium grained sandy clay [COLLUVIUM]	3.35-3.65
4A	Stiff grey mottled reddish brown and yellowish brown fissured clay [COLLUVIUM]	9.4-9.9
5	Very stiff grey mottled orangish brown and bluish grey fissured clay [COLLUVIUM]	9.9-10.4
6	Very stiff grey mottled orangish brown horizontally fissured clay [WEALD]	10.4-10.75
7	Stiff grey mottled brownish grey and orangish brown sub-horizontally laminated fissured clay [WEALD]	10.75-11.0
8	Brownish grey clay with yellowish brown medium grained sand with LIMESTONE cobbles [WEALD]	11.0-11.2
9	Very stiff brownish grey mottled orangish brown fissured clay with occasional shell fragments	11.2-11.4
10	Very stiff brownish grey becoming grey with depth mottled yellowish brown and orangish brown clay with occasional roots [WEALD]	11.4-11.8
11	Very stiff grey mottled light grey sub-horizontally laminated silty clay with areas of silt and occasional shell fragments [WEALD]	12.2-12.6
12	Very stiff grey mottled light grey sub-horizontally laminated silty clay with occasional shell fragment and patched of grey silt [WEALD]	12.6-12.8
13	Very stiff grey mottled light grey and brown silty clay with occasional shell fragments	12.8-12.9
14	Very firm grey mottled orangish brown laminated clay with occasional shell fragments [WEALD]	12.9-13.2
15	Very stiff mottled grey and brownish grey sub horizontally laminated clay [WEALD]	13.2-13.55
16	Very stiff grey sub-horizontally laminated clay [WEALD]	13.55-14.0

Appendix 6.2: Borehole log for Shell and Auger boreholes

P03A Laboratory Log		
Sample	Description	Depth/m
	TOP SOIL	0.0-0.5
1	Stiff light orangish brown mottled light grey medium grained sand [COLLUVIUM]	0.5-1.3
	Soft light grey and yellowish brown sandy clay with top soil roots and cobbles [COLLUVIUM]	bulk
3	Stiff grey mottled yellowish brown clay [COLLUVIUM]	3.3-3.75
4	Soft mottled brownish grey and greyish brown clay [COLLUVIUM]	3.75-4.2
5	Firm to stiff mottled grey and yellowish grey clay with occasional iron staining [COLLUVIUM]	4.2-4.7
6	Firm grey clay with yellowish brown medium grained sand [WEALD]	5.1-5.5
7	Stiff brown and light brown sub horizontally laminated clay with layers of very stiff orangish brown sandy clay [WEALD]	5.8-6.35
8	Very stiff grey sub-horizontally laminated clay with light grey and light yellowish brown silt layers [WEALD]	6.35-6.55
9	Very stiff brownish grey clay with frequent white silt horizontal layers [WEALD]	6.55-7.05
10	Very stiff dark grey clay with horizontal white silt layers [WEALD]	7.05-7.55
11	Very stiff grey mottled brownish grey clay with horizontal white silt layers (with two layers of orangish brown sand at top)[WEALD]	7.55-8.05
12	Very stiff dark grey mottled orangish brown and white sub-horizontally clay [WEALD]	8.55-9
14	Very stiff dark grey mottled orangish brown horizontally laminated clay with frequent white silt layers [WEALD]	9-9.65
18	Very stiff light grey [WEALD]	9.65-10
17	Very stiff dark grey sub-horizontally laminated clay with silt layers [WEALD]	10-10.5

Appendix 6.2: Borehole log for Shell and Auger boreholes

P03B Laboratory Description		
Sample	Description	Depth/m
1	Soft to firm grey mottled yellowish grey and orangish brown clay [COLLUVIUM]	2.5-2.8
2	Soft to firm grey mottled yellowish grey clay [COLLUVIUM]	2.8-3.35
3	Firm brownish grey clay [COLLUVIUM]	3.35-3.6
4	Firm mottled grey and brownish grey clay with occasional shell fragments and cobble-sized lumps of mudstone [COLLUVIUM]	3.6-3.85
5	Firm grey mottled yellowish grey and orangish brown sandy clay [COLLUVIUM]	3.85-4.2
6	Firm greyish brown sub-horizontally laminated clay with occasional shell fragments [WEALD]	5-5.3
7	Stiff to very stiff brown sub-horizontally laminated clay [WEALD]	5.45-6
8	Very stiff grey mottled orangish brown sub-horizontally laminated clay with white horizontal silt layers [WEALD]	6.2-6.6
9	Very stiff grey sub-horizontally laminated clay with white silt and layers of orangish brown fine sand [WEALD]	6.7-7
10	Very stiff mottled brown, brownish grey and grey sub-horizontally laminated clay with white silt partings [WEALD]	7.1-7.45
11	Very stiff dark grey horizontally laminated clay with occasional light yellowish brown silt partings [WEALD]	7.45-8
12	Very stiff grey sub-horizontally laminated clay with layers of orangish brown clay and white silt [WEALD]	8.2-8.5
13	Stiff dark grey mottled brownish grey sub-horizontally laminated clay with traces of silt [WEALD]	8.5-8.75
14	Very stiff light grey sub-horizontally laminated clay [WEALD]	9.2-9.45
15	Very stiff grey mottled brown sub-horizontally laminated clay with traces of white silt [WEALD]	9.5-9.9

Appendix 6.2: Borehole log for Shell and Auger boreholes

P05 Laboratory Description		
Sample	Description	Depth/m
1	Soft to firm mottled grey and yellowish brown clay showing some sub-horizontal structure [COLLUVIUM]	1.2-1.45
4	Soft to firm mottled grey brown and orange clay showing some sub-horizontal structure [COLLUVIUM]	1.45-1.9
5	Stiff to firm brownish grey and greyish brown mottled orangish brown clay [COLLUVIUM]	2-2.45
16	Stiff mottled grey brown and orangish brown sub-horizontally laminated clay [COLLUVIUM]	2.5-2.95
7	Very stiff grey mottled light brown sub-horizontally laminated clay [COLLUVIUM]	2.6-3.05
9	Very stiff brownish grey and greyish brown mottled brown [COLLUVIUM]	3.15-3.6
10	Very stiff dark grey mottled brown clay [COLLUVIUM]	3.6-4.05
11	Very stiff dark grey mottled brown sub-horizontally laminated clay with traces of silt	4-4.46
12	Very stiff dark grey mottled brown sub-horizontally laminated fissured clay with traces of white silt [COLLUVIUM]	4.4-4.8
13	Very firm sub-horizontally laminated clay with traces of white silt [WEALD]	4.8-5.25
15	Very stiff light brown mottled white sub-horizontally laminated clay [WEALD]	5.05-5.55
16	Very stiff light bluish grey sub-horizontally laminated clay with traces of white silt [WEALD]	5.3-5.55

Appendix 6.2: Borehole log for Shell and Auger boreholes

P05 Drilling Log		
Sample	Description	Depth/m
	Stiff dark brown sandy clay TOPSOIL [COLLUVIUM]	0.0-0.45
	Stiff dark brown fissured clay [COLLUVIUM]	0.45-0.9
U1	Stiff light orangish-brown heterogenous clay of upper plasticity [COLLUVIUM]	0.9-1.4
U4	Soft light brown heterogenous clay of upper plasticity with orange sandy gravel beds [COLLUVIUM]	1.4-2.0
U5	Firm mottled brown-grey inter-stratified clay of upper plasticity [COLLUVIUM]	2.0-2.5
U6 U7	Firm blue mottled grey clay of upper plasticity [WEALD]	2.5-3.0
U9	Firm mottled brownish-grey inter-stratified clay of upper plasticity [WEALD]	3.0-3.6
U10	Very stiff dark grey inter-stratified clay of upper plasticity [WEALD]	3.6-4.0
U11 U12 U13 U15 U16	Firm light greyish-blue clay of upper plasticity [WEALD]	4.0-5.55

Appendix 6.2: Borehole log for Shell and Auger boreholes

P06 Laboratory Description		
Sample	Description	Depth/m
3	Soft to firm mottled grey and yellowish brown medium grained sandy clay [COLLUVIUM]	1.9-2.35
5	Firm brown mottled brownish grey and orangish brown medium grained sandy clay [COLLUVIUM]	2.85-3.35
9	Firm mottled grey and yellowish brown sandy clay [COLLUVIUM]	5.8-6.25
15	Firm to stiff dark grey mottled yellowish brown and orangish brown clay [COLLUVIUM]	8.7-9.0
19	Very stiff grey locally mottled brown sub-horizontally laminated clay [WEALD]	9.5-9.95
22	Very stiff bluish grey sub-horizontally laminated clay [WEALD]	11.1-11.55
23	Very stiff grey mottled orangish brown clay with traces of silt [WEALD]	11.6-12
25	Very stiff greyish brown and bluish grey sub-horizontally laminated silty clay [WEALD]	12-12.45
27	Very stiff grey and brownish grey mottled orangish brown fissured clay with silt partings [WEALD]	14.1-14.4
28	Very soft grey mottled brown clay [WEALD]	14.95-15.2
28	Very stiff grey silty clay [WEALD]	15.2-15.4
29	Very stiff blueish brown grey mottled light grey silty clay [WEALD]	15.4-15.85

Appendix 6.2: Borehole log for Shell and Auger boreholes

P06 Drilling Log		
Sample	Description	Depth/m
	Soft dark brown sandy clay TOP SOIL containing organics	0.0-0.6
	Soft dark mottled greyish brown sandy clay containing a little gravel [COLLUVIUM]	0.6-1.2
U3	Firm light brownish-yellow slightly gravelly clay containing some sand [COLLUVIUM]	1.2-2.4
U5	Firm dark orangish-brown very sandy clay [COLLUVIUM]	2.4-3.25
	Light greyish-blue slightly sandy clay of upper plasticity [COLLUVIUM]	3.25-4.0
	Light greyish blue clay of upper plasticity containing some gravel [COLLUVIUM]	4.0-4.4
	Light greyish brown clay of upper plasticity containing some gravel [COLLUVIUM]	4.4-4.9
U9	Stiff mottled greenish-grey heterogenous clay of upper plasticity [COLLUVIUM]	4.9-6.2
	Stiff light green homogenous clay of upper plasticity [COLLUVIUM]	6.2-6.7
	Stiff greyish-blue homogenous clay of upper plasticity [COLLUVIUM]	6.7-8.5
U15	Stiff dark greenish-blue slightly silty clay of upper plasticity [WEALD]	8.5-9.0
U19 U20 U21 U22 U23	Very stiff dark greyish-blue fissured clay [WEALD]	9.0-12.0
U25 U26	Very stiff light blueish-grey fissured clay [WEALD]	12.0-14.0
U27 U28 U29 U30	Very stiff light greyish-blue fissured clay of upper plasticity [WEALD]	14.0-15.9

Appendix 6.2: Borehole log for Shell and Auger boreholes

P07 Laboratory Log		
Sample	Description	Depth/m
	Yellowish brown mottled brownish grey medium grey sand [COLLUVIUM]	
	Firm greyish brown mottled yellowish brown and reddish brown clay with occasional shell fragments and LIMESTONE cobbles and gravel [COLLUVIUM]	
3	Soft brown mottled yellowish brown and grey sandy clay [COLLUVIUM]	1.5-1.95
4	Firm grey and brown mottled orangish brown clay [COLLUVIUM]	1.98-2.53
5	Soft to firm mottled brown, yellowish brown and grey clay [COLLUVIUM]	2.35-2.69
7	Firm mottled grey yellowish brown and brown clay [COLLUVIUM]	2.69-3.12
10	Very stiff grey sub-horizontally laminated clay with silt partings [COLLUVIUM]	3.75-4.28
14	Firm mottled dark grey yellowish brown and grey clay [WEALD]	10.7-11.15

Appendix 6.2: Borehole log for Shell and Auger boreholes

P07 Drilling Log		
Sample	Description	Depth/m
	TOP SOIL	0.0-0.2
	Light brown clayey sand of low plasticity [COLLUVIUM]	0.2-0.7
U3	Soft light mottled greyish-brown very sandy clay with sub-angular gravel [COLLUVIUM]	0.7-12
U4	Stiff light mottled greyish-brown slightly sandy clay with limestone gravel [COLLUVIUM]	1.2-2.0
U5	Stiff greyish brown slightly sandy clay containing some gravel [COLLUVIUM]	2.0-2.4
U6	Firm light mottled orangish-grey brown sandy clay [COLLUVIUM]	2.4-2.75
U7 U9	Firm orangish-grey brown very sandy clay [COLLUVIUM]	2.75-3.7
U10	Firm light orangish-brown clay [COLLUVIUM]	3.7-4.6
	Dark brown clayey sand [COLLUVIUM]	4.6-5.1
	Purplish-grey marsh clays [COLLUVIUM]	5.1-5.9
	Purplish-grey slightly sandy clay [COLLUVIUM]	5.9-6.7
	Purplish-grey clay with sub-angular to rounded cobbles of limestone [COLLUVIUM]	6.7-7.0
U13	Stiff grey heterogenous clay of high plasticity [WEALD]	7.0-9.0
U14	Very stiff purplish-blue clay of high plasticity [WEALD]	9.0-11.15

Appendix 6.3: Minuteman sample descriptions

MM1

Description	d/m	No.
	1 m	
	2 m	
	3 m	
Very soft greyish clay	4 m	012
	5 m	
	6 m	
	7 m	
	8 m	
	9 m	

MM2

Description	d/m	No.
	1 m	
	2 m	
	3 m	015
	4 m	014
Soft light greenish grey clay	5 m	
	6 m	016
	7 m	
	8 m	
	9 m	

MM3

Description	d/m	No.
	1 m	
	2 m	017
	3 m	025
	4 m	024
	5 m	023
Firm light brown clay of upper plasticity	6 m	036
Firm mottled greenish-grey of upper plasticity	7 m	035
Firm mottled greenish-grey of upper plasticity	8 m	038
Firm greenish-grey of upper plasticity	9 m	039

MM4

Description	d/m	No.
	1 m	
	2 m	
	3 m	
	4 m	
Firm light brown clay of upper plasticity	5 m	046
	6 m	
Very soft grey homogenous clay	7 m	086
	8 m	
	9 m	

MM5

Description	d/m	No.
	1 m	
Soft light brown homogeneous clay of upper plasticity	2 m	055
	3 m	
	4 m	
Firm light greyish-brown sandy clay	5 m	054
Soft light brown homogenous clay	6 m	053
	7 m	
	8 m	
	9 m	

MM5A

Description	d/m	No.
Soft light brown slightly gravelly clay of upper plasticity	1 m	094
	2 m	
	3 m	
	4 m	091
	5 m	
	6 m	090
	7 m	
	8 m	
	9 m	

MM6

Description	d/m	No.
A blueish- grey clay of upper plasticity	1 m	080
	2 m	
	3 m	
Light brown homogenous clay	4 m	081
	5 m	
	6 m	
	7 m	
	8 m	
	9 m	

MM7

Description	d/m	No.
	1 m	
Soft light brown clay of upper plasticity	2 m	093
	3 m	
Stiff dark greyish-brown clay	4 m	092
	5 m	
	6 m	
	7 m	
	8 m	
	9 m	

MM8

Description	d/m	No.
	1 m	
	2 m	
	3 m	
	4 m	
	5 m	
Medium dense brown sub-angular to sub-rounded sand	6 m	097
	7 m	
	8 m	
	9 m	

Appendix 6.4: Skidster borehole logs

SK01		
Sample	Description	Depth/m
4	Light grey silt (QB 004)	0.6-1.35
5	Light brown clay of high plasticity (QB 005)	1.35-2.1
6	Light brown clay of high plasticity (QB 006)	2.1-2.85
1	Light grey silt (QB 001)	2.85-3.1

SK02		
Sample	Description	Depth/m
2	Firm light brown clay (QB 002)	0.5-0.8
3	Firm light brown homogenous clay (QB 003)	0.8-1.35

SK03		
Sample	Description	Depth/m
9	Light brown clay of high plasticity (QB 009)	1.1-1.3
8	Stiff light brown homogenous clay of high plasticity (QB 008)	1.3-2.1
10	Light brown homogenous clay (QB 010)	2.1-2.85

SK001		
Sample	Description	Depth/m
1	Firm, becoming soft with depth, brown mottled orangish-brown clay with occasional roots	0.6-1.35
2	Very soft to soft greyish-brown mottled orangish-brown (mottled black for lower 0.15 m) clay with occasional peds of firm orangish-brown clay	1.35-2.2
4	Very soft greyish-brown mottled orangish brown (mottled black for lower 0.1 m) clay with occasional areas (< 10 x 5 x 5 mm) medium grained orangish-brown sand [HYTHE BEDS]	2.2-2.95

SK002		
Sample	Description	Depth/m
1 2	(QB 040)(QB 043) Loose whitish-brown heterogenous silty sand with a little gravel	0.3-1.55

Appendix 6.4: Skidster borehole logs

SK003		
Sample	Description	Depth/m
2 3	Soft yellowish brown mottled clayey sand [HYTHE BEDS]	2.3-3.5
4	Soft to firm yellowish-brown mottled grey clayey sand [HYTHE BEDS] becoming more grey with depth	3.5-4.2

SK004		
Sample	Description	Depth/m
	[TOP SOIL]	0.0-0.55
1	Soft greyish-brown mottled orangish-brown sandy clay with occasional shell fragments and roots	0.55-1.3

SK005		
Sample	Description	Depth/m
	[TOP SOIL] with occasional roots	0-0.65
1	Soft to firm brownish-grey mottled yellowish-brown clay [HYTHE BEDS] becoming grey	0.65-1.35
2	Firm grey mottled yellowish-brown sandy clay [HYTHE BEDS]	1.35-1.6
2	Soft to firm grey clay	1.6-1.85
2	Firm to stiff dark grey clay with occasional carbon and sulphur patches*	1.85-2.1
3	Firm grey, becoming brown for lower third, sub-horizontally laminated clay [ATHERFIELD] with carbonised wood and occasional shell fragments	2.1-2.85
REMARKS: *with evidence of structure: sample showed splits and gaps on extrusion and signs of landslip		

Appendix 8.1. Total rainfall (mm) per year at Cherry Gardens, Folkestone, Kent.

Year	Rainfall	Year	Rainfall	Year	Rainfall
1868	706.9	1908	667.5	1951	1051.2
1869	723.2	1909	887.8	1952	883.6
1870	558.5	1910	938.7	1953	732.8
1871	647.2	1911	808.1	1954	827.9
1872	952.5	1912	856.5	1955	690.9
1873	563	1916	898.5	1956	702.2
1874	601.3	1917	778.2	1957	796.1
1875	802.2	1918	717.5	1958	1026.2
1876	733.1	1919	750.4	1959	763.4
1877	937.7	1920	634.6	1960	1027.2
1878	744.4	1921	405.1	1964	772.8
1879	766	1922	805.8	1965	796.1
1880	711.3	1923	760.1	1966	1092.4
1881	805	1924	877.9	1967	722.5
1882	728.1	1925	854.1	1968	759.9
1883	728.5	1926	793.2	1969	838.9
1884	552.7	1927	998.3	1970	707.2
1885	651.5	1928	904.4	1971	604.6
1886	759.4	1929	698.6	1972	547.8
1887	617.6	1930	880.5	1973	507.6
1888	699.7	1931	694.6	1974	890.4
1889	611.6	1932	803.5	1975	815.6
1890	629.3	1933	671.6	1976	759.1
1891	697.9	1934	715.3	1977	780.5
1892	868.2	1935	920.2	1978	662.2
1893	733.9	1936	881.5	1979	844.4
1894	866.4	1937	1035.6	1980	817.4
1895	676.7	1938	732.7	1981	844.2
1896	861.4	1939	1152.6	1982	751.4
1897	645	1940	734.8	1983	768.1
1898	582.3	1941	688.6	1984	870.2
1899	648.7	1942	775.6	1985	767
1900	811.4	1943	637.7	1986	879.3
1901	529.2	1944	724.1	1987	872.3
1902	652.2	1945	587.8	1988	793.3
1903	960.3	1946	920.1	1989	656
1904	639.3	1947	663.9	1990	578.5
1905	806.5	1948	667	1991	690
1906	805	1949	703.1	1992	929.4
1907	674.2	1950	805.1		

Appendix 8.2. Monthly rainfall figures (mm) from Folkestone, Meteorological Station 03305094 (After Bowdler, 1972)

Month	Average rainfall*	1967	1968	1969	1970	1971
January	65.5	37.3	65.5	57.6	103.0	65.4
February	19.1	50.5	63.5	83.6	60.9	17.5
March	55.1	39.6	29.2	60.4	49.4	59.4
April	40.4	67.1	51.1	40.5	50.9	39.4
May	31.5	64.3	39.6	96.4	18.5	37.1
June	4.82	39.1	41.1	49.7	14.9	113.1
July	46.2	27.2	99.3	111.3	39.7	88.1
August	93.7	38.6	62.2	63.3	40.0	51.6
September	49.0	59.4	131.1	9.9	84.1	17.9
October	65.5	116.6	69.3	11.2	34.1	43.3
November	104.6	76.5	41.7	179.8	168.6	71.5
December	68.3	56.9	39.1	85.6	58.0	15.2
Total	**643.9**	**673.1**	**732.7**	**849.3**	**722.1**	**619.5**

* For the period 1916-1950