Residual Shear Strength with Special Reference to Rate Effects

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This thesis is dedicated to the memory of Donald Pottinger Jacobs (1910–2003)

Abstract

This thesis sets out to investigate and resolve a number of technical issues relating to the residual shear strength of clay soils. For this research the principal objectives concerned strain rate effects using the Bromhead Ring Shear and associated other factors their effects on residual shear strength, replication of the stress path and the homogeneity of clay. The principal areas of investigation are concerned with effects of varying rates of strain on residual shear strength, influence of "stress path" in determining residual shear strength and innate variability of residual strength in natural strata. Finally the principal conclusions were that the Ring Shear was excellent for British Landslides but at high strain rates there were problems with pore water movement and extrusion which are artefacts of the design. Stepless application of normal stress reduces these effects as it removes extreme stress changes. Clay strata are non homogeneous hence the requirement for continuous characterization tests.

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To anyone I missed sorry
Thanks

Karen

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1 Introduction

Shear strength of a soil is the underlying criteria for a variety of geotechnical problems from retaining walls (Clayton, Milititsky and Woods, 1993), to slope stability (Bromhead,1986), and even to foundations (Tomlinson, 1995). In particular engineers are concerned with failed soils when investigating both construction failures and landslides. The term landslide not only describes an "event" but is also used to describe the remnant material. Old landslides are not necessarily perfectly stable and stationary; they may continue to move as they are affected by rainfall, toe erosion and seismicity. Therefore an understanding of ruptured and failed soils is essential for dealing with these issues. Beneath the moving part of the landslide is what is commonly known as a slip surface. Properties of the slip surface depend on the material in which and on which it was formed and to a certain extent the processes which occurred during its formation. This thesis sets out to investigate a variety of these effects using natural materials and therefore is directly applicable to engineering problems as well as to the mechanics.

Therefore this thesis sets out to investigate and to resolve a number of technical issues relating to the residual shear strength of clay soils. The residual strength in this context is the shear strength which the soil exhibits after being subject to large relative shear displacements such as it would experience in a landslide or during tectonic deformation (Skempton, 1985). This is a mature subject of research, and many of the methodologies used are well-established, so that they need little introduction. For example, residual shear strength is determined in a ring shear device conforming to the "small ring shear" of BS1377 :(1990) test 6 (see also Bromhead, 1979), and shearing procedures conform largely to the procedures described by Bromhead (1986, 1992, Chap 4) and by Harris & Watson (1997).

The principal areas of investigation are concerned with the following:

- Effects of varying rates of strain on residual shear strength (Chapter 5),

- Influence of "stress path" in determining residual shear strength (Chapter 6).

- Innate variability of residual strength in natural strata (Chapter 7)

1.1 Essential terms for the thesis:

Throughout this thesis, technical terms are explained on first occurrence, either explicitly, or by context. However, a number of elementary terms merit definition, or explanation, at the outset.

The **shear strength** of a soil is defined as the maximum shear stress that the soil can carry. There are different states of maximum shear stress dependent on whether the soil is pre-failure or post; the **peak shear strength**, which is the shear strength of an undisturbed soil, or a soil which has not been previously sheared up to and/or beyond the point of rupture, and **residual shear strength** (sometimes termed **ultimate** shear strength) which is the strength exhibited by the soil after it has passed the point of rupture, and where continued shear deformations have occurred to the point where the shear strength of the ruptured soil has reached a constant value (Skempton, 1985).

Clearly, peak shear strength is exhibited at comparatively small shear strains in the soil mass (Figure 1.1), but residual strength is a phenomenon of large strains. Indeed, shear strain in the conventional small-strain representation ceases to have real meaning as relative shear deformation increases, and the concept of relative shear displacement of the opposite sides of the rupture zone is probably more meaningful than that of "engineer's shear strain". However, the phrase "relative shear displacement" is ponderous, and it is convenient and conventional to continue to describe it loosely as "strain", and that convention is followed here. It will be evident when this is done, as the units of relative displacement "strain" are millimetres rather than being dimensionless, as is in true strain.

This loose definition of "strain" also permits definition of a temporal **strain rate** in mm/minute. The phrase **rate effect** is used to describe any resulting apparent change in residual shear strength when the strain rate is changed.

With increasing moisture content, soils change their behaviour in ways that are evident when a hand specimen is remoulded. At low moisture contents, they behave as very friable, brittle, solids, and they become more malleable and plastic as the moisture content increases, up to the point where they exhibit certain fluid properties. Two important limits within this range are defined: the **liquid limit** (ω_L) and the **plastic limit** (ω_P), and these are respectively the moisture contents at firstly the point of transition between plastic solid and viscous liquid behaviour, and secondly the point of transition between friable solid and plastic solid. Since these transitions are gradual, and not abrupt, the determination of the limiting moisture contents relies on following a set procedure, that is, in the UK, the procedure laid down in BS1377:(1990) tests 3.2, 4.3, 4.5 and 5.3. As a further consequence of the arbitrariness of the determination of the limits, it is clear that soils exhibit aspects of both behaviours at a particular limit.

The **Plasticity Index** (I_P) is simply the numerical difference between the liquid and plastic limits, and so defines a range of moisture content in which the soil exhibits ready malleability in the hand specimen. The conventional geotechnical definition of the **Moisture content** (ω) as the ratio of the mass of water to the mass of solids within the soil and expressed to the nearest whole number percentage is used here.

The **slip surface** (also known, among other terms, as **sliding surface, shear surface, basal shear**) is the thin zone across which differential movement takes place, and in which residual strength is developed. Examples of slip surfaces developed in the field are found in this thesis as Figures 1.2 and 1.3. The slip surface comprises not simply the plane of separation, but also the thin layers of remoulded clay (**gouge**) which adhere to the adjacent soil, and which help give the slip surface its essential character (see Section 2.1) and Figure 1.3. The alternative names for the slip surface are used interchangeably throughout the thesis in order to provide variety: the Author has no specific preference for one form over another. A shear zone is a zone of soil in which two or more slip surfaces are developed. In a shear zone, the individual slip surfaces may be sub-parallel, or anastamosing.

The stress state on a slip surface involves a combination of direct stress and shear stress, together with water pressures. The direct stress minus the water pressure is termed the **effective stress**.

The water pressures in a soil may be in equilibrium with the hydraulic boundary conditions, and this corresponds to a **drained state**. The drained state does not necessarily imply zero pore water pressures, except that in a small ring shear device, drainage is achieved with a small water bath providing control over pore water pressures, and in this case, pore water pressures are sensibly zero. The corresponding extreme state, where the soil contains the entire pore water pressure change resulting from the application of further direct total stresses or shear stresses, is termed the **undrained state**. Equilibration of the unbalanced pore water pressures from the undrained state towards the drained state is known as **consolidation** or **equilibration** in geotechnical terms.

Shear strengths (whether peak or residual) are functions of effective stress, not of direct stress, although in the direct shear test (shear box and ring shear), water pressures are normally negligible, and the total direct stress and the effective stress are identical. The ring shear test is classed as a total stress test in BS 1377 as it has no facility to measure pore pressures, but it allows free movement of water and therefore is a drained test which, according to Bishop and Bjerrum (1960), makes it an effective stress test, not total stress test. This only applies at slow rates because at fast rates the shear zone is eroded too quickly to be drained.

Where the shear strength is linearly related to the effective stress (see Figure 1.4), the constants of proportionality are referred to as **cohesion** (symbol c) - which is the shear strength at zero effective stress - and "friction" (symbol Φ), where the increases in strength per unit increase in effective stress is given by the tangent of Φ. Strictly speaking, this angle is the **(residual) angle of shearing resistance**, but in common parlance, the word "friction" is convenient and simple, and will be used here. The parameters "c" and "Φ" are collectively known as shear strength parameters, and the subscript "u" is used to denote that they relate to undrained conditions, the subscript "r" denotes residual shear strength, and the superscript ' signifies drained conditions. The shear strength parameter Φ need not be constant, if the shear strength is non-linearly related to effective stress. In this case residual shear strength is likely to be convex upwards (Figure 1.5).

1.2 Aspects for Investigation

The thesis in the following chapters has been organized as follows:

Chapter 2 sets the research results of this investigation in their proper historical context, that of the development of ideas about soils' shear strength, with particular attention to residual shear strength. A review of the ring shear test technique appears redundant in the light of the thorough account by Bishop et al. (1971) and the later accounts which bring the subject up to date, notably by Lupini (1981), Parathiras (1994) and Taylor (1998).

The residual strength of clay soils is not a constant property, and it depends on a number of factors. These are discussed, with reference to other published work, in Chapter 3. Clearly, test procedure has a bearing on the results that are obtained, and the test procedures used in this research, and used or recommended by other workers, are technically discussed in Chapter 4.

The first body of original results is presented in Chapter 5. This dataset examines rate effects in residual shear strength, and the procedures and results are compared to the published work of Tika (1989), Lemos (1986) and others. Chapter 5 concludes that the position established by those Authors is erroneous, and that the so-called rate effects which they describe are simply and primarily the result of apparatus design and test procedure. A theory to account for this is presented and put to the test. It has been concluded by this author that influence of machine design, soil loss during shear and pore water movement account for these rate effects. The background behind this is discussed in Section 1.3.

Chapter 6 describes the construction and operation of a step-less normal load system for the ring shear apparatus which is air-pressure controlled, and which follows the entire residual strength envelope for a given soil (over a pre-defined stress range). This provides greater insight into the shape of such envelopes compared to the orthodox procedure of probing the envelope at a sequence of normal stresses. The roots of this procedure lie in the understanding of the evolution of landslides subject to toe erosion. A discussion of this topic sets the work in context. The background behind this is discussed in Section 1.4.

Chapter 7 builds upon the concept published by Bromhead et al. (1999) that even a carefully-contrived test programme will show considerable variability in the values of residual strength determined in a sequence of tests made on specimens from a small sample taken from a single location at a particular site. During this, variability between clay members of a given soil succession is explored. Samples were taken from three locations at various depths to determine the homogeneity of the layers. The background behind this is discussed in Section 1.5.

Chapters 8 summarises the work as a whole, and points to resulting technical issues regarding the residual strength of soil that require further attention.

1.3 Rate effects in Residual Shear Strength Testing

The consequences of the different rates of strain have been extensively researched since the 1960's and the results have been widely reported, Lupini (1980), Lemos (1986), Tika (1989) and Taylor (1998) are among those who have published on this topic. Existence of these effects is not questioned but their nature and cause is the main investigation of this research. This thesis sets out to investigate whether rate effects are due to strain rate, the apparatus design or something else.

Since this research is based around the use of natural soils, with a minimum of processing, refined soils such as industrial grade bentonite or kaolinite have not been used. Two natural clays of comparatively high plasticity, and one of very high plasticity, sampled from locations where they are actively involved in land-slipping, have been used in that part of the research which relates to strain rate effects.

The first of the high plastic clays comes from an outcrop of slipped Gault Clay (lower Cretaceous) in a mudslide at Watershoot Bay, St Catherine's Point, Isle of Wight (OS Grid Ref: SZ 504 754). This mudslide has been active throughout the past decade. In colour, this soil is dark grey to black and is a firm clay of high plasticity above the A line, as classified by the plasticity chart according to BS5930:1981 (Craig, 1997).

The very plastic clay was sampled from a basal shear at the foot of the coastal cliffs at Warden Point, Isle of Sheppey, Kent (OS Grid Ref TR 015 730). This was also the location for the samples used by Bromhead et al. (1999) and it is believed that the materials used in that investigation and this one are identical. This soil is dark grey brown initially but it oxidizes to a light tan colour when exposed to the atmosphere due to its high iron content. It is classified as very stiff clay of very high plasticity according to its plot above the A line on the plasticity chart, as before.

Locations for these sampling sites are put into context on the geological map of SE England, Figure 1.5.

The samples of the second high plasticity clay were obtained during a trenching investigation of a highway cutting failure in tills at Dromore, Co. Banbridge, Northern Ireland (OS Grid Ref: NW 307 099), and were supplied by Professor D. Hughes of Queen's University, Belfast. This soil is brown, classified as very stiff clay of high plasticity according to its plot just above the A line on the plasticity chart, almost touching the line, BS5930:1981 (Craig,1997). They are samples taken from the basal shear of the slide, determined during the investigation to be a thin layer of laminated clay separating an upper till. Broadly, the geological context for this site is shown on Figure 1.6.

Results of these investigations are discussed in detail in Chapter 5 and the detailed tables of results are contained within Appendix D.

1.4 Stress Path Analysis

Field observations of coastal landslides show that they are wasted away by toe erosion and secondary mudslides (e.g. Bromhead, 1978; Dixon & Bromhead, 2002). Back analysis of various stages in this process indicates that the mean stress state in each landslide reduces such that the locus of the point (σ_{av}', τ_{av})

follows the residual strength envelope towards the origin. This process and the resulting plot can be termed a “landslide stress path”.

In order to follow equivalent stress paths in the laboratory, a step-less system of normal load application was devised. This was designed in detail by Mr G. Pelling and provided for this research by Mr K.R. Matthews, respectfully Chief Designer and Managing Director of Wykeham Farrance International of Slough (UK). Their input is gratefully acknowledged. (Both have since retired.)

The use of step-less normal control ring shear apparatus is investigated and the ability to follow the stress path is detailed within Chapter 6.

1.5 Variability within a sample

The literature on geotechnical engineering often lumps together quite varied materials with “catch-all” terms, such as “the London Clay” or “the Lias” etc. Against this background, this section of research sets out to discover the degree of variability that might be encountered in an example deposit, the Gault of the Ventnor Undercliff, Isle of Wight and generally similar clay strata in the underlying Sandrock (Lower Greensand).

Samples for this research were provided from boreholes in Ventnor, and were made available by Consulting Engineers Halcrow and High Point Rendel, together with South Wight Borough Council, and other materials obtained by surface sampling.

The study complements similar studies of statistical variability of test results from a single location for example Bromhead et al. (1999) investigations at

Warden Point, Sheppey. Findings from these investigations are detailed within Chapter 7.

1.6 Objectives

The principal objectives are to investigate the following:

- ➤ Increasing the strain rate on a Bromhead ring shear
  - Effect on the resultant residual shear strength.

- ➤ Factors affecting residual strength
  - apparatus design
  - clay plasticity
  - strain rate variance mid test

- ➤ How rate of strain affects
  - post test moisture content
  - displacement

- ➤ Normal stress control
  - Investigate replicating the stress path using a gradually applied normal stress.

- ➤ Sample variability
  - How appropriate is the tendency to assume homogeneity, similarity and isotropic behaviour within the same sample.
  - Case study into the area west of Ventnor, Isle of Wight.

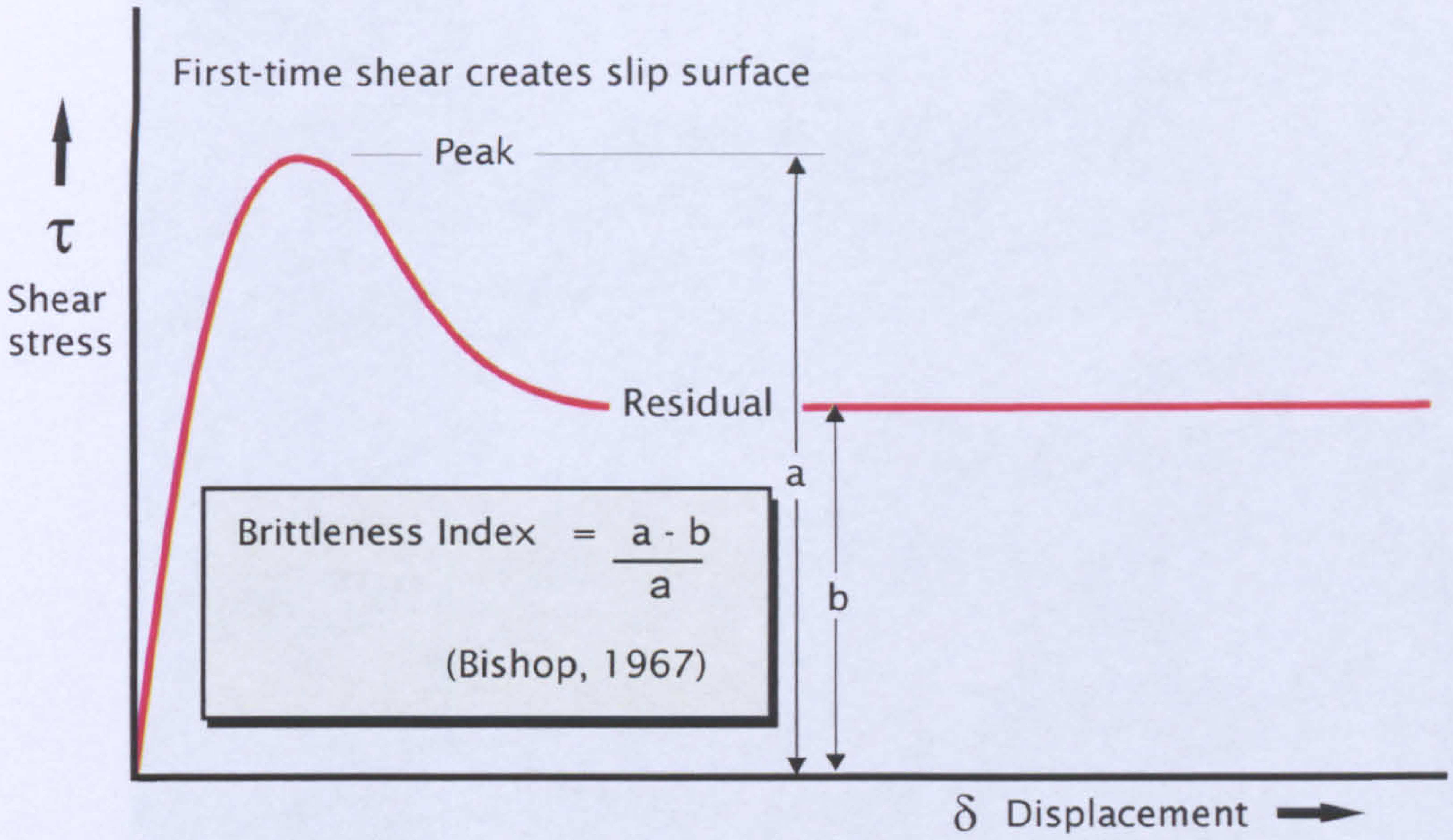


Figure 1.1 An idealised stress - strain plot showing the mobilization of peak and residual strength.
(Inset) Definition of "brittleness index".

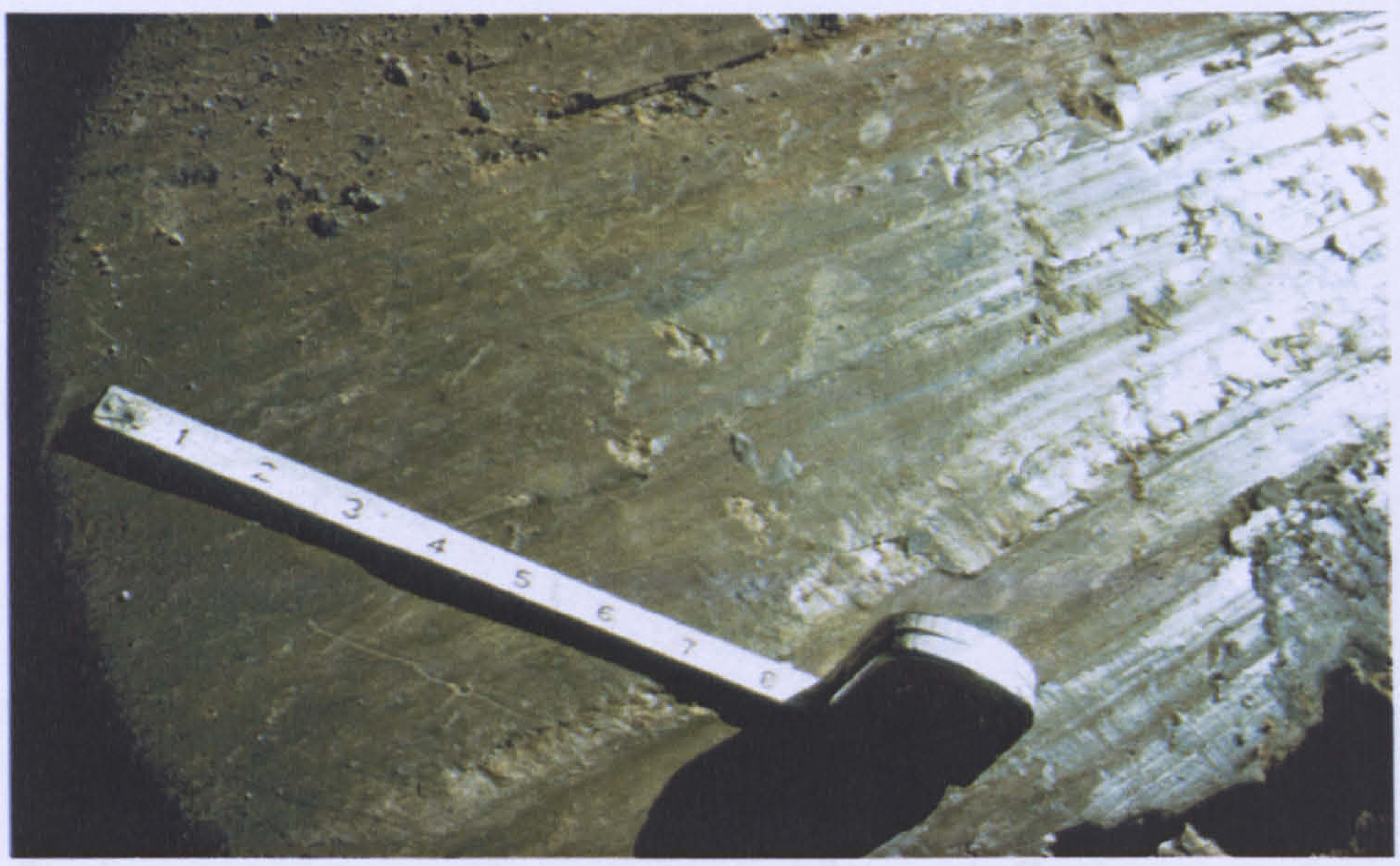


Figure 1.2 An example slip surface (from a shallow landslide at Sevenoaks).
This shows the general polish, the striations in the general direction of shear, and that there are a series of sub parallel shear surfaces forming a shear zone. (Photo J. N. Hutchinson).

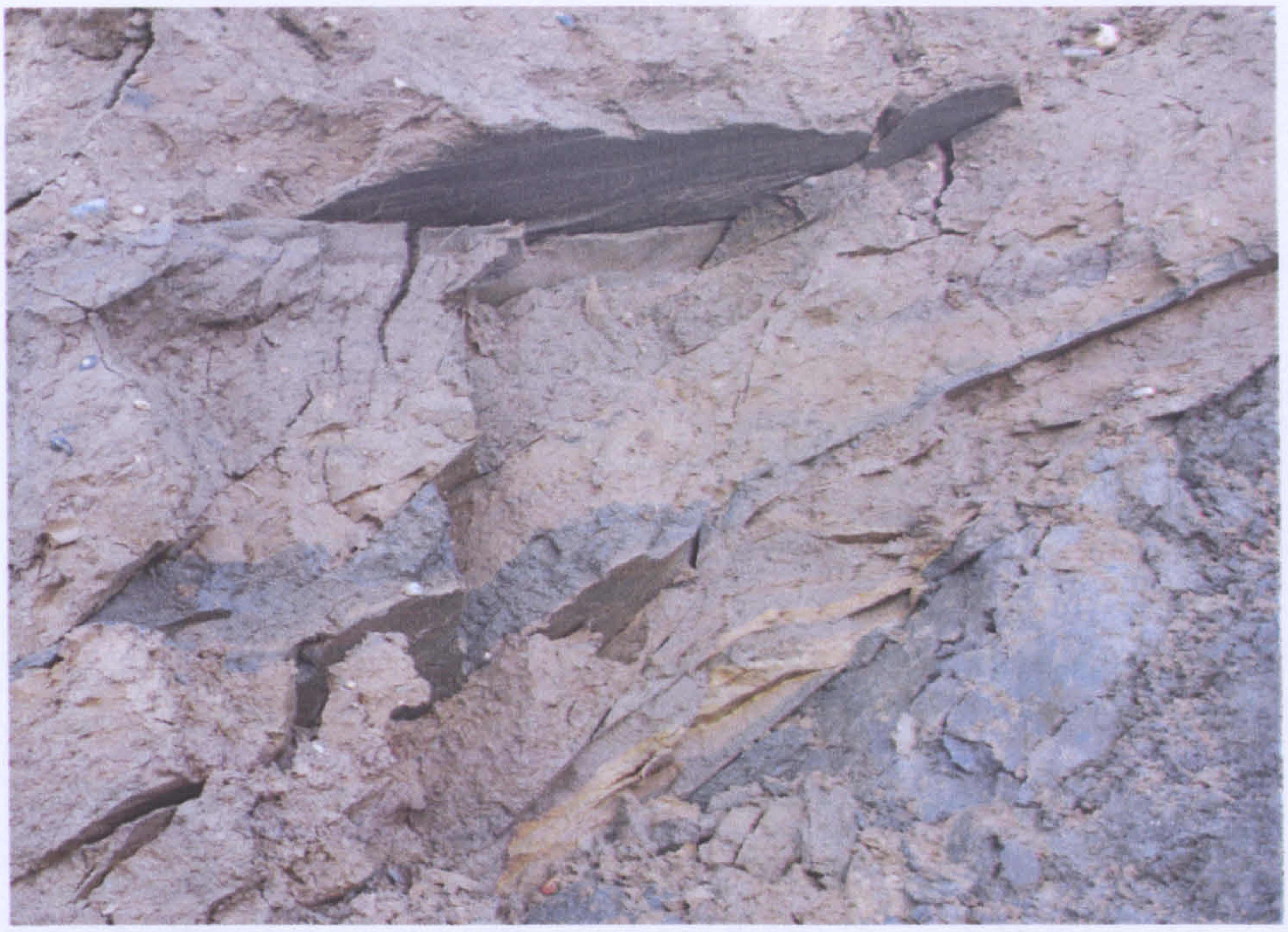


Figure 1.3 Example shear zone from a mudslide at Warden Point, Sheppey, Kent. This photograph shows the striations, the thin layer of remoulded clay "gouge" adjacent to the shear surface, and the colour variation which results from the incorporation of clay from differently weathered zones in the cliff. (Photo E. N. Bromhead).

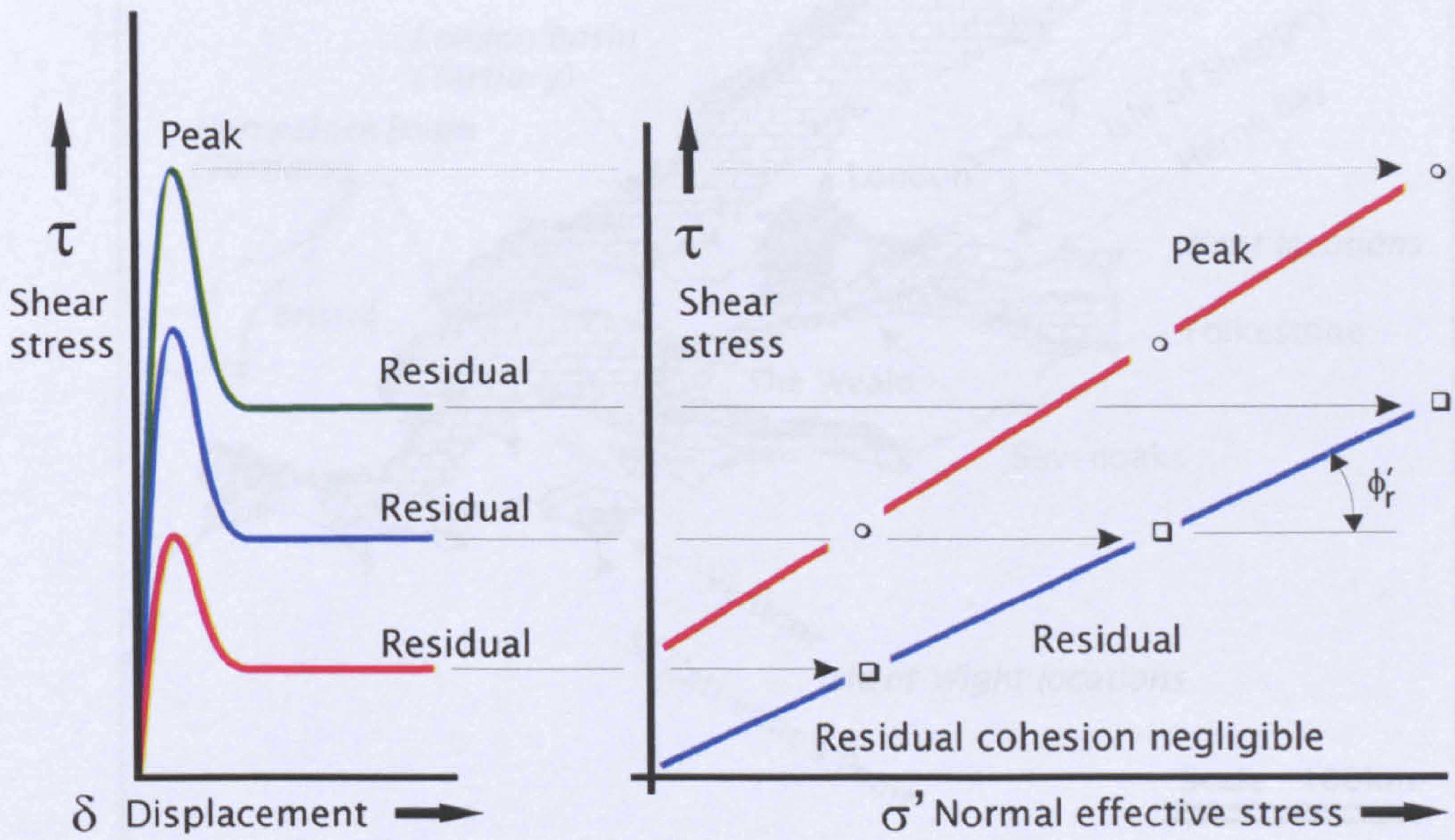


Figure 1.4 Development of a residual shear strength envelope from a series of individual tests on soil specimens. Peak strength equivalent is also shown, but for one point only. The diagram is idealised.

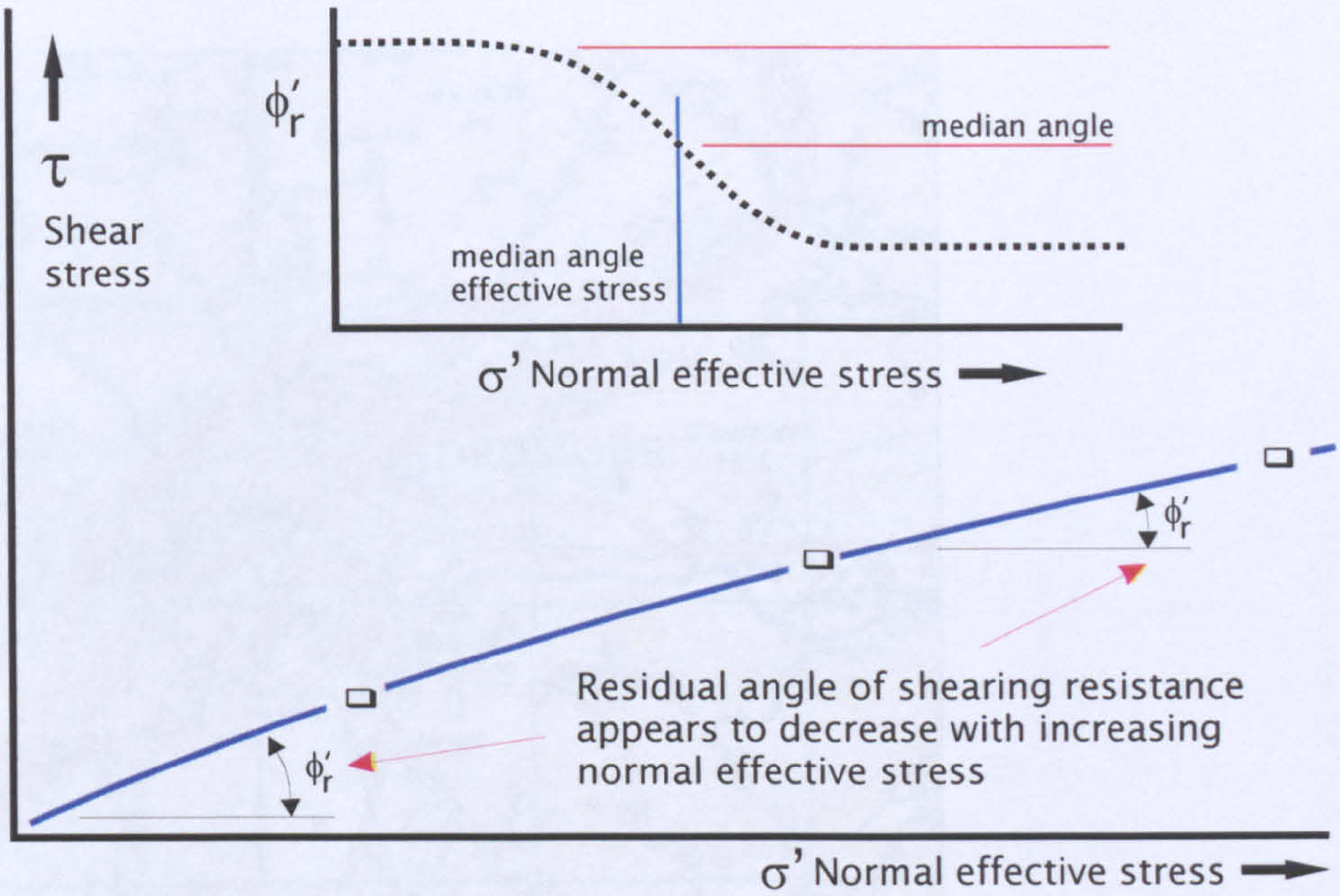


Figure 1.5 Curved residual shear strength envelope, (inset) showing Maksimovic's median angle effective stress.

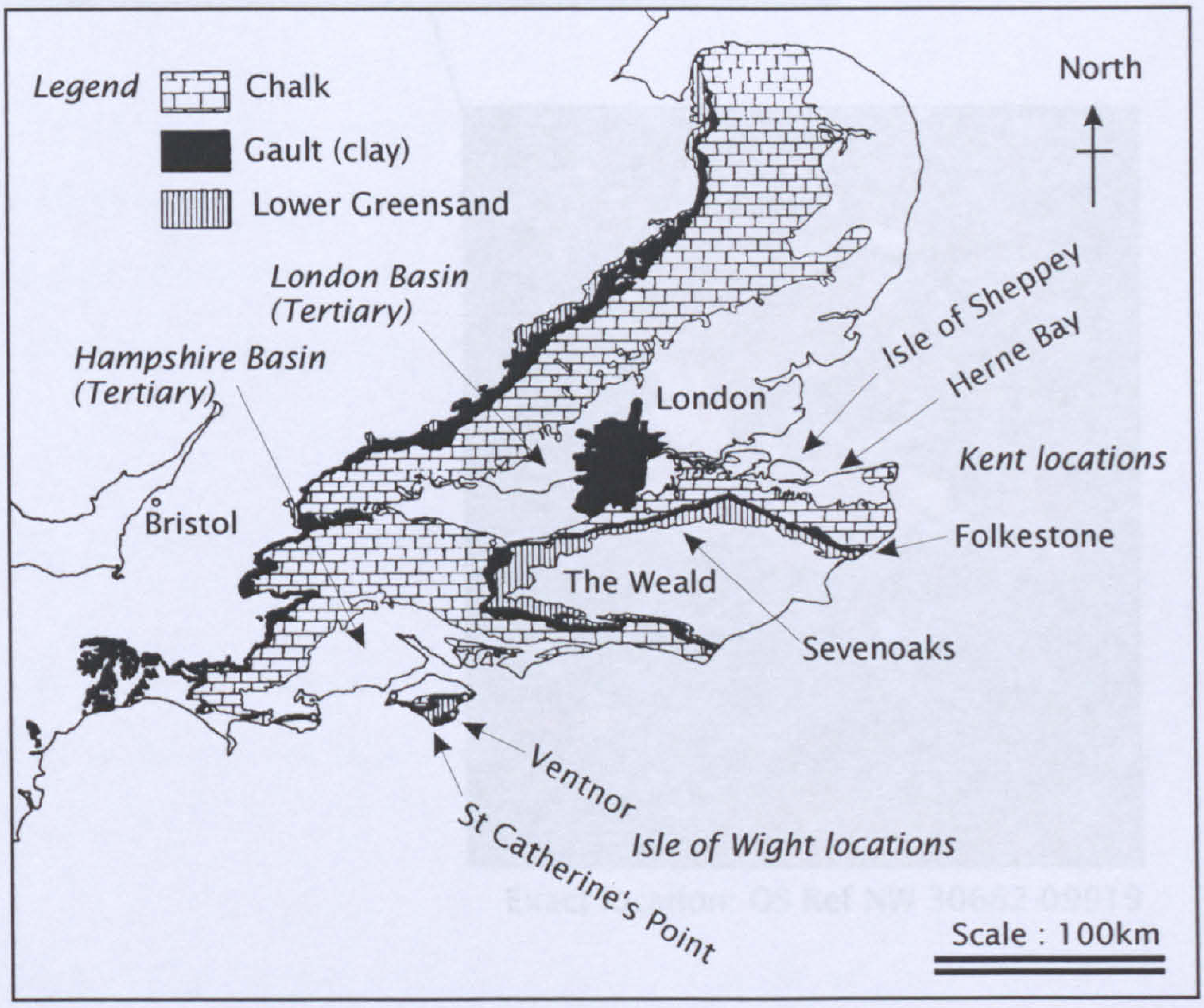
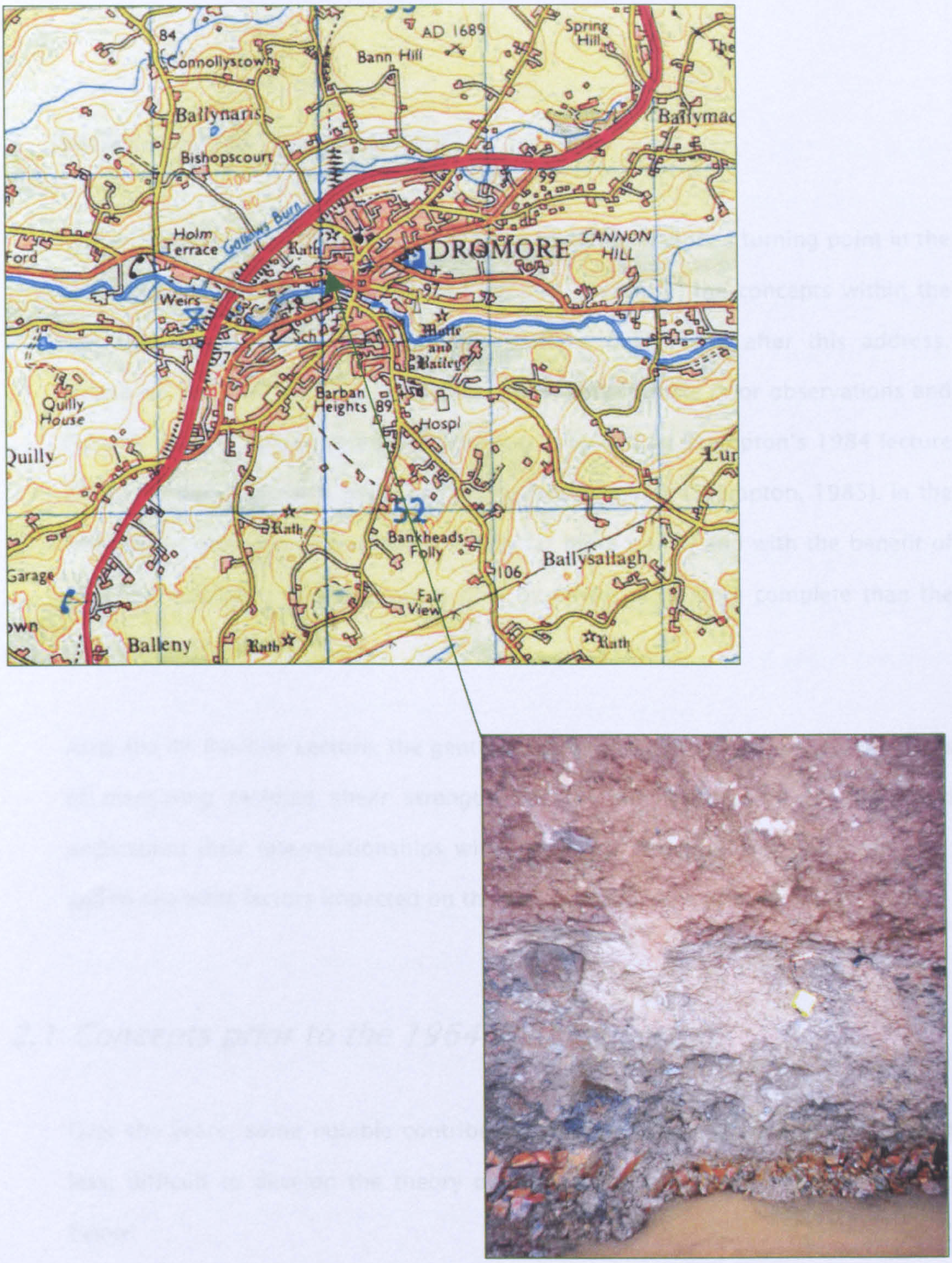


Figure 1.6 Location of sampling sites in SE England. Note that reference is also made to soils from Herne Bay and Folkestone in the text, although samples from those locations are not tested in this research.



Exact location: OS Ref NW 30662 09919

Figure 1.7 Location of sampling sites at Dromore, N. Ireland.

2 Background and context

The 4th Rankine Lecture in 1964 (Skempton,1964) represents a turning point in the understanding of residual shear strength and, therefore, the concepts within the literature can be subdivided into two periods, before and after this address. Skempton provided the unifying theory, which sets cognate prior observations and theories in their proper context. Clarity was provided by Skempton's 1984 lecture 20 years later, and again published in the following year (Skempton, 1985). In the later paper, Skempton explains his theory far more clearly and with the benefit of 20 years additional research, some of it by others, it is more complete than the earlier lecture and paper.

After the 4th Rankine Lecture, the geotechnical profession sought to find new ways of measuring residual shear strength, to find slip surfaces in the field and understand their interrelationships with geological structure and past processes; and to see what factors impacted on the properties of slip surfaces.

2.1 Concepts prior to the 1964 Rankine Lecture.

Over the years, some notable contributions to soil mechanics made it more, not less, difficult to develop the theory of residual shear strength. These are listed below:

a. Clays behave purely cohesively (Bell, 1915). As we now understand matters, the cohesive behaviour is a pore pressure effect, and manifests itself only under undrained conditions. This cohesive strength depends on water content: more water, less cohesion (strength).

b. Stiff clays, which are purely cohesive under undrained conditions, soften with time. This softening, which is the result of progressive fissure opening and moisture ingress, reduces the soil cohesion, and this is the cause of delayed failure in stiff clays (Tezaghi & Peck, 1943; Skempton, 1948). Such an explanation leads us to conceive of the strength of clays purely as undrained strength, and is unhelpful in understanding the importance of slip surface formation.

c. Residual strength is the drained equivalent of the ultimate end point in a softening process. The text of the Rankine Lecture implies this, which is a retrograde step from Skempton & Delory's (1957) end point being the "fully softened", remoulded or critical state strength.

The search for an ultimate end point in a softening process probably underlies all the research in and around the 1930's using rotary shear machines (interallia Gruner & Haefeli, 1934; Cooling & Smith, 1935; Hvorslev, 1936, 1937& 1939), although prior to the widespread dissemination of Terzaghi's principle of effective stress, these researchers could not know that the test needed to be drained. Indeed, Gibson & Henkel first published their theoretical treatise on rates of strain for quasi full drainage in a test much later (Gibson & Henkel, 1954). However, such is the nature of a ring shear test that eventually, full drainage must occur, whatever is the rate of shear (caveats to this are noted later).

2.2 *Philosophical ideas behind the 4th Rankine Lecture*

The concepts drawn together by Skempton to form the theory of residual strength include some ideas that may trace their ancestry back to the very beginnings of soil mechanics as a science. There are essentially six of these concepts.

i. Soil without exception including clays behaves essentially as a frictional material in drained terms (Coulomb, 1776; Rankine, 1856). The effective cohesion present is there occurs primarily as a result of overconsolidation and stress history, with sometimes an input from cementitious bonds. This property is a fragile and unreliable component of shear strength, and is lost entirely when a slip surface is formed and therefore can be disregarded.

ii. Soil strength follows the principle of effective stress, so that pore water pressures have an effect on shear strength. This is hinted at by Gregory (1844), and is stated by Terzaghi in 1923, although more fully and explicitly in 1936 (Jardine et al., 2004). The demonstration of this came later, as summarised (for example) by Bishop & Henkel (1957) who describe in detail the apparatus in which their tests were made.

iii. Pore pressures in soil depend in part on seepage (Casagrande, 1932; Cedergren, 1962) and in part on undrained stress change (Terzaghi & Frohlich, 1929; Bishop & Bjerrum, 1960), so that they are almost inevitably non-hydrostatic. Undrained stress changes may dissipate with time (see vi).

iv. The stress - strain behaviour of clay soils exhibits brittleness, i.e. after a peak capacity is reached, the capacity to carry shear stress diminishes with further deformation, until a residual or ultimate carrying capacity is reached (Figure 1.1). This is shown clearly by Haefeli (1951).

v. Since residual strength is largely a property of clay soils, the residual condition coincides with the formation of continuous, slickensided "slip surfaces" in both laboratory specimens and the field, that can be identified visually (Gregory, 1844; Collin, 1856; Henkel, 1956 & 1957; Cotton & Te Punga, 1955 ; Te Punga, 1957) in both earthworks failures (cut slopes) and in natural slopes.

vi. Equilibration of undrained suctions or depressed pore water pressures - in essence, a time-dependent change in seepage - is the primary cause of delayed failure in slopes (Bishop & Bjerrum, 1960). This is not articulated in the Rankine Lecture, but taken in conjunction with the material therein and explains the well-known and widely experienced phenomenon of delayed failure of cut slopes in clay. After failure, a slip surface is found to have been formed.

2.3 Sources of slip surfaces

Although slip surfaces were observed and described in natural slopes by Cotton & Te Punga, 1955(Hutchinson & Fookes, 2004) and in a variety of cut slope failures (interallia Gregory, 1844 and Collin, 1846), after the Rankine Lecture, engineering geologists and geotechnical engineers made it their business to look for them. Hutchinson (1970) found basal shears in a mudslide (then termed a mudflow) at Beltinge, Herne Bay. The London Clay studies of Bromhead (1978), and Dixon & Bromhead (2002) rely on identifying slip surfaces in the field. So too do the extensive studies in the Lias of the English Midlands by Chandler (1974); in the Gault of the Isle of Wight (summarised in Hutchinson & Bromhead, 2002), in the failed slope at Selborne (Cooper et al., 1999) and in many other cases.

Skempton (1985) also describes the finding of shears in the tectonically distorted Siwalik clays at the Mangla Dam project in Pakistan. Detailed drawings of the shear zone show the anastomosing multiple slip surfaces caused by thrust faulting.

2.4 Back Analysis

The area of numerate soil mechanics began in the mid-twentieth century. Vaughan et al (2004) in their keynote paper at the Skempton Conference credit Hvorslev (1960) as the founder of this useful tool although the factor of safety was developed by Taylor (1948). From the initial beginnings of slip analysis, which assumes the whole soil domain is in failure and not just a continuous rupture surface (Vaughan et al, 2004) it also failed with the lack of ability to consider pore water pressure effects. Limit analysis is the next iteration of numerical analysis in soils. Although better than slip analysis, it does not facilitate the input of pore water pressure as an independent variable, nor the ability to analyse drained failures. Vaughan et al noted it was used in practice with its application in the analysis of bearing capacity and retaining walls but not in slope stability.

Slope stability analysis proceeds from knowledge of soil properties, the ground water conditions and the slope geometry to provide an assessment of stability under that combination of conditions and parameters. The most widely used index for that stability assessment is the Factor of Safety, F, although Probability of Failure P_f is increasingly commonly used as an adjunct to slope stability analysis where probabilistic assessments can be made of the inputs. Reliability (Hicks et al., 2002) is the Probability of Not Failing, or $1 - P_f$ which is likely to be a more helpful concept for engineers. Risk assessment, on the other hand, which computes the consequences of a failure, must still be operated with P_f. Alternatively, the critical seismic coefficient (Sarma, 1973; Sarma & Bhave, 1974) is the horizontal acceleration, $k_c g$ that renders F=1.

The origins of modern slope stability analysis are traceable to the 18th century, and to Coulomb and others considering the sliding of soil wedges. In the early 20th century, curved failure surfaces were considered (Petterson, 1955), most notably the circular arc surface used to analyse the Gothenburg jetty failure. This led, inexorably to the method of slices for simplifying the calculations. Subsequently, methods for dealing with more general curved shapes for slip surfaces, both in 2 dimensions and 3 dimensions, have been developed (Maksimovic,1996). Occasionally, insights are gained by considering simpler failure surfaces, e.g. plane failure (Haefeli, 1948; Skempton & Delory, 1957).

Most stability analyses are based on slip surface shapes that are simple; approximately kinematically feasible, may be based on naturally occurring shapes, and thus represent (approximately) realistic mechanisms. The stress analysis is simplified and approximate, but it is in equilibrium. It is not therefore absolutely essential to provide simultaneous approximate lower and upper bound solutions. In effect, this requirement is satisfied by only considering the realistic mechanisms. Errors relate much more to the choice of shear strength parameters, pore water pressures, ephemeral loadings, and inadequacies associated with the search for a critical mechanism, rather than to defects in the computation of the factor of safety, once those inputs are largely correct.

Back Analysis (Chandler, 1977) operates to determine one of the conventional inputs (soil properties or ground water conditions) from knowledge of the state of stability, most usually, from the occurrence of failure, at which point F=1. Chandler's paper does not introduce the concept, but codifies and popularises it.

Back analysis was used, for example, by Hutchinson (1969) and named explicitly; it was used long before that, for example by Skempton (1948) among others.

Slope stability analysis uses the final of the three. Limit equilibrium analysis involves vertical sections of a trial rupture surface, where statics can be used to determine forces acting on the slices. This analysis assumes identical fractions of shear strength are mobilised at the base of all the vertical sections, an advantage being this fraction acts as a safety factor. Details of the various methods are given by Bromhead (1986) and some of the advances are given by Vaughan et al (2004) and Chowdhury et al (2004).

Basically, the analysis relies on one implicit assumption, (Vaughan et al, 2004) that the method of failure actually occurs, due to movement of a series of blocks and only when sufficient parts have failed to produce a continuous rupture surface - one of the basic failures of slip analysis. Various methods exist, Bishop (1955), Morgenstern & Price (1965), Janbu (1973), Sarma (1973), Spencer (1973) and Maksimovic (1979) are all noted by Vaughan et al (2004). Bromhead (1986) noted that most authors had more than one attempt at development of this analysis.

Case studies of previous landslides are a major source of information. These investigations involve slope stability analysis in reverse which enables the derivation of field shear strengths from the use of a known safety factor which was developed by Taylor in the 1940s (Taylor, 1948). Developments in limit equilibrium analysis have enabled this. Bromhead (1986) quotes a value of one as a factor of safety at the instance of failure and credits Chandler (1977) for the term back analysis. More specifically, Bromhead (1986) recommends the use of Spencer's (1967) and Morgenstern-Price "like" methods with $f(x) = 1$. With computers even in their infancy in 1986, Bromhead (1986) recognised the impact their invention would have on analysis.

Numerical stress-strain methods, to analyze progressive failures and finite element analysis to simulate the situations, have all improved the accuracy of this area of numerical analysis. Limit equilibrium analysis, in its various forms, still acts as a basis for these. The requirement of correction factor was suggested by Skempton (1985) to include the effects of side shear, which Bromhead (1986) termed as a reduction factor for shear stress. Hutchinson (1969) was noted by Bromhead (1986), as incorporating the use of lateral variation in section geometry into the analysis.

Skempton's 1964 Rankine lecture (Skempton, 1964) is the best summary of research into slope stability which incorporates the relatively new idea at that time of peak and residual strength. Later, towards the end of his career, Skempton (1985) summarized the progress that had been made post his Rankine lecture. Skempton noted the shear strength of landslides was now obtained by the determination of residual shear strength parameters and residual shear strength had become the operative strength for landslide slip surfaces, geological faults and tectonically-sheared geomaterials.

2.5 Developing test methodologies and apparatus

Residual shear strength can be determined using three different methods of these; two in the laboratory and one through a more analytical numerical route.

- Laboratory tests on a specimen with an actual slip surface. Skempton (1964, 1985) is noted for using this method.

- Slip surface formed by shearing a remoulded specimen in the laboratory. Hvorslev (1939) and Bishop et al (1971) widely used this method and designed ring shears to determine the value of residual shear strength.
- Back analysis, which uses a system of safety factors and numerical analysis to derive the value of residual shear strength. Arising from collaboration between Engineering Geology and Numerate Soil Mechanics; the best records of this area are Chandler (1977) a paper on this methodology which is extremely comprehensive and Bromhead (1986), who provided an excellent summary of methods available.

For this particular thesis the main focus is on the second method; forming slip surfaces in the laboratory by shearing, using an appropriate apparatus with some references to back analysis. No actual slip surfaces have been used in this research. The Bromhead ring shear apparatus was used in this research to form the slip surfaces. Both the triaxial and the shear box apparatuses are inappropriate for this research as neither have the same facility for horizontal displacement to form the slip surface when compared to the ring shear.

2.6 The Ring Shear concept

Subsequent development post the 1964 Rankine lecture of the ring shear concept was described in detail by Bishop et al (1971), who introduced the Imperial College/Norwegian Geotechnical Institute (IC/NGI) ring shear apparatus. Lupini (1981), Tika (1989), Parathiras (1994), Fearon (1998) and Taylor (1998) all give excellent summaries of the development of the ring shear test described as a drained test for residual strength. The test is now a test in the British Standards (BS1377). Ring shear testing developed because of the ease with which residual conditions are achieved when compared to the shear box. On the downside, it has high local concentrations of strain and the directions of principal stresses are uncertain as testing proceeds. The major difference with the shear box is that the ring shear is a controlled rate of displacement device and its principal purpose is to investigate the post peak section of the stress displacement curve.

According to La Gatta (1971) Casagrande stated in a personal communication in 1965; that to achieve residual strength required large unidirectional displacement. A ring shear apparatus would permit unlimited shear displacements and the concept of residual strength could therefore be explored. From this it was clear that there was both a decrease in strength at large displacements and that strength would level off to an ultimate or residual value.

This evolutionary process continued and the 1970's saw the introduction of not one, but four ring shears.

La Gatta (1970) developed a ring shear apparatus at Harvard University. The concept was initially introduced at a conference in 1969 by Sembenelli and Ramirez. There is also an excellent summary of its development in Parathiras' thesis (Parathiras, 1994). The Harvard apparatus used a small thin sample and resulted in fast drainage and, to reduce friction, the top platen was held in place by the load cells. Bishop et al (1971) described an annular shaped failure surface produced when an annular disc is normally loaded and twisted with the actual specimen being either solid or divided using the confining rings.

The IC/NGI apparatus has been in use since 1971. Taylor (1998) attributes the development of both the Harvard and NGI/IC ring shear equivalent to the renewed interest aroused by Professor Skempton's Rankine Lecture in 1964. Compared to its predecessors the NGI/IC has reduced levels of error, although still has major disadvantages associated with it, as it is neither easy to assemble nor simple to operate these complications include one hundred different screws all of which are designed for their own specific level of load bearing. In addition this apparatus is extremely expensive to construct and the duration of tests are dependent on the thickness of the specimen, in other words the thicker the sample the longer the duration of the test.

The major disadvantage with the IC/NGI apparatus is that the clearing distance between the upper and lower rings has to be maintained and a gap must exist when the ring shear is run initially at low speeds in order for the shear zone to form. As an open gap, this causes increased displacement of soil; after the formation of the shear zone, the gap is closed and the test rate increased, thus resulting in a change in normal stresses, reducing the accuracy of the results. However, when the gap is open, more soil is displaced, so the test is usually run with the gap open so normal stresses are not disturbed part way through the test, but, everything possible is done to minimise soil loss during the test; at the same time metal to metal friction has to be minimised. Taylor (1998) gives an excellent summary of this machine, including a detailed technical specification of this apparatus. Taylor also clearly indicates the problems which occur as a consequence of the "gap control" difficulties at fast rates of shear. Taylor himself did not see any significant problem, but, stated that it was occasionally necessary to readjust the gap which caused minor short term fluctuations in results.

The Cambridge ring shear test was developed by Mandl et al (1977). The main aim of this apparatus was to investigate the structure of shear zones in soils. Parathiras (1994) gives an excellent summary of the development in his thesis, noting that the main aim of this apparatus was to investigate the structure of shear zones in soils. The unique feature of this apparatus is the way the gap is maintained between the upper and lower rings, but can be closed via a system of springs and adjustment screws thus preventing soil extrusion. The normal load was applied using gas pressure and normal and shear stresses were measured using Cambridge load cells and photo elastic meters.

The Bromhead ring shear apparatus is the least complicated and most inexpensive available. Bromhead (1979) developed this during the 1970s at Kingston University in the UK with both the commercial and academic market in mind. The Bromhead ring shear test has a 5mm deep sample which enables the drainage to be more rapid, so reducing consolidation time which facilitates the use of faster rates of drained shearing. The apparatus also has shorter test durations than its contempories and an excellent accuracy. Errors occur due to the extruding soil causing friction between the confining rings and the top platen; side friction between the sample and confining rings; tilting of the top load platen and inaccuracies in force measurement. The Bromhead Ring Shear has been part of BSI377: Part 7 since 1990 but there have been more recent revisions, interallia step-less loading and data collection.

Parathiras (1994) details three other ring shear tests. A detailed description is given of the ring shear test developed by Nienwenhuis (1991) from the University of Utrecht, The Netherlands, and a brief description of the other two developed by Yagi et al (1991). Taylor (1998) summarized the more recent development of the ring shears in Japan by Sassa. These are therefore beyond the scope of this thesis. Both the NGI/IC and the Bromhead Ring Shears have continued to develop as research objectives have dictated.

2.7 Applications

Over time, the ring shear apparatus has had cause to adapt. Advancement of scientific knowledge has seen the requirements of shear strength testing change and the ring shear has therefore been modified to cater for the demands of modern-day soil mechanics. The introduction of geosynthetic materials caused modifications in the design due to the fact that geofabrics are different to natural soil. Tests have also been modified, so, there is less soil lost, which was always a particular problem with the NGI/IC machine.

Anayi (1990) did research into residual strength of clay at low normal stresses. This research required some modification to the Bromhead ring shear test with vanes being added to improve the accuracy of the result. These cause the shear surface to form at mid height in the sample as opposed to near the surface. This modified ring shear apparatus requires less horizontal displacements for the clay to achieve residual strength and a Teflon coating inside the sample container was used to reduce extrusion.

Stark & Eid (1993) modified the Bromhead Ring Shear to facilitate testing of geotextiles. Stark & Poppel (1994) describe how the Bromhead ring shear has been successfully modified to allow for soil/geosynthetic and geosynthetic/geosynthetic interfaces. The specimen container is enlarged from 0.5 cm deep to accommodate 2 cm deep specimen and a Lucite ring is used to facilitate securing geosynthetics to the top platen using a glue. For the different interfaces there is a different modification to be base platen. For soil/geosynthetic interfaces the bottom bronze porous stone is replaced with a "knurled" stainless steel ring which minimises drainage. For the geosynthetic/geosynthetic interfaces the bottom bronze porous stone is replaced with another Lucite ring to allow for the geosynthetic material to adhere to one surface with glue.

Parathiras (1994) noticed that the NGI/IC ring shear test was not performing as it should during prolonged stages of fast shearing or at high rates of displacement. The problem was identified as the gap control mechanism. The required solution was to develop a modification, which minimised soil loss, allowed for multistage, long duration and fast shearing tests on fine-grained soils. The final change resulted in new confining rings being introduced which, when compared to the old, result in a difference in residual strength values of between O and 2%. The modification has resulted in tests losing up to seven times less soil than with the original rings. It also allows for prolonged stages of shearing.

Iverson et al (1997) investigated till deformation using an adapted NCI/IC machine. The apparatus was adapted to accommodate large samples, so allowing gravel-size clasts to be left in the till. It had transparent outer walls with markings to allow for continuous visual monitoring, a feature similar to the Cooling & Smith machine of 1935. Specialised monitoring of the sidewalls allows the wall effects to be quantified. Local normal stress monitoring devices allow for monitoring of local concentrations of stress during the shearing process. It was concluded that this modified ring shear test was a viable means of studying the mechanical properties of glacial sediments.

Tan et al (1998) used an adapted form of the Bromhead ring shear apparatus to investigate the soil/geosynthetic interface shear strength. To achieve this, a fabricated Perspex ring replaces the bronze porous stone to allow the geo-textile to be fixed onto the top platen. It was concluded from this research that, the ring shear test is suitable to determine the peak and residual sand/geo-textile interface shear strength.

Increasingly the ring shear apparatus is being used for a greater variety of soils and geo-textiles to find their shear strengths and the technology surrounding the apparatus is also developing. Weight loading systems are being replaced by air pressure loading, making the test more automated. Devices are used to ramp the pressure and readings are taken by data-loggers. The more advanced the ring shear test is technically, the less human intervention is needed: therefore, the error shifts from human to controllable mechanical errors. The expectation is, that improvements will be achieved without compromising Hvorslev's recommendation for an easy to assemble and simple to operate test procedure that doesn't compromise accuracy.

2.8 Summary

To summarize, following Skempton's definitive statement of what slip surfaces and residual strength mean to geotechnical engineering; research has concentrated on clarifying minor issues not in providing alternative theory.

This thesis follows the same principle as the majority of research post the fourth Rankine lecture. The following chapter is a more precise literature review to the issues covered in the thesis.

3 A review of the previous research into factors which influence the value of Residual Shear Strength

Landslides move at different rates and the causes of this phenomenon have been an active area of research for many years. Controversy surrounds the mechanics of this phenomenon. A critical factor in our understanding of landslide movement rates is the behaviour of slip surfaces at different rates of strain. However, this is not the only factor that may vary the residual strength.

Recent years have seen the focus of research into the residual shear strength of soil concentrate on the effects of normal stress change, strain rate effects and variation in soil or pore fluid chemistry. The following is a review of this recent work.

3.1 Influence of normal stress on the residual strength envelope

Idealised direct shear test results show an initial peak followed, post failure of the sample, by a gradual reduction in strength to the residual state (Figure 1.1). If tests are performed at a range of normal stresses, then the corresponding peak and residual shear strengths will be higher at larger normal stresses (Figure 1.4). Ordinarily, the relationship between strength and normal effective stress is assumed to be linear, and this is likely to be an adequate approximation over a sufficiently small range. The resulting line may be described by residual shear strength parameters, cohesion and friction, and most researchers couch their results using this methodology; among those are Bishop et al. (1971), Lupini (1980) and Skempton (1964, 1985), especially, when determining residual shear strength at a particular effective stress level from the parameters of the residual shear strength envelope.

In contrast to this, there is a school of thought which argues that the relationship between normal and shear stress is not linear, but curved. This can be the result of simply too few data points defining the line, especially if it is forced to pass through the origin. Figure 3.1 demonstrates this with two data points. Each data point has three possibilities: too high (high), exact or too low (low). There are thus nine possible combinations. Only one (exact:exact) guarantees a straight line through the origin. The majority of combinations (4) appear to show a convex upwards curvature of the envelope: (high:exact), (high:high), (high:low), (exact:low); and (low:low) may also, in some circumstances, look like a convex upwards curved envelope. Remaining combinations (low:high), (low:exact) and (exact:high), are more likely to be interpreted as a 'best fit' straight line (with scatter or error) than a concave upwards curve. A curved envelope may therefore be a consequence of test errors and analyst perception. To remove this inaccuracy, more normal stress stages need to be incorporated in the testing procedure, and a better idea needs to be gained of the likely errors present in determining the position of each point in the test.

Maksimosvic (1989) attempts to justify a perfect fit method which exhibits a curved envelope as opposed to the conventional linear approach by recalculating the back analysis data from Skempton's 1985 paper using his new method. Interestingly, Skempton (1985) concluded that the envelope was non-linear but, due to variations in the "degree of curvature" using different clays, recommended the best-fit linear envelope to be used for design purposes.

Underlying this, in part, is the question as to whether or not residual cohesion actually exists. Skempton, in his 1964 Rankine Lecture (Skempton, 1964) stated that the residual cohesion is negligible. Maksimovic (2002) concluded that residual cohesion did not exist, but with a curved residual strength envelope, the 'best fit' straight line for a part of this curve would imply a cohesion intercept if projected back to zero effective stress. That would be different to a real residual cohesion, or actual residual strength at zero effective stress, which would represent the amount of attraction (or bonds) between the soil particles. Post-formation of the shear surface this attraction is disturbed, therefore, it seems rational that post rupture; the residual cohesion should be negligible. This explanation lies behind the customary assumption that residual cohesion is assumed to be zero in most landslide analyses, especially back analyses.

Continuing this theme, Maksimovic (2002) decided that linearity was dependent upon the value of stress level ratio (SLR) which is the effective normal stress divided by the "median angle normal stress", or the normal effective stress at which the secant angle of shearing resistance is midway between the values at extreme small and large values of effective stress. According to Maksimovic, if this value is greater than two, then there is a linear relationship. Maksimovic found power-type expressions to be applicable for values of SLR between 0.2 - 2.0, whereas log-type, he found to be approximately equal to 1. For soils with zero cohesion, Maksimovic used an equation that incorporated a basic angle of friction, a maximum angle difference and a median angle normal stress, to give a value of shear stress. Therefore, the only alteration to the normal equation is the value of Φ'.

Maksimovic (2002) concluded that the non-linear approach should be the preferred method, whereas Skempton (1985) stated, that the linear method was less complicated than the curved approach and gave adequate accuracy. The analysis presented later in the experimental chapters follows Skempton's recommendation.

3.2 *Ascending and descending sequences of normal load.*

In the simple example idealised test results, Figure 1.4, tests are shown as though each one was done on a fresh sample. This was merely to demonstrate the three corresponding peaks and residuals. Residual shear strengths are often done using a multistage procedure, where the slip surface is established during the first stage, and then further stages are carried out with a range of increased normal stresses, each stage being a "stress probe", which provides a single point on the residual strength envelope on a normal effective stress v. shear stress plot. Observations of the soil specimens' response to normal stress being increased or decreased during this process has shown very different results; Bromhead (2004b) shows a diagram, Figure 3.2, which reveals that the loading and unloading behaviours are very different. Uploading of the normal stress produces a stiff response. Unloading produces a much less stiff response. Chandler (1977) suggests that in a ring shear test, the energy stored in the torque measurement system is released, and the slip surface movement direction is reversed. Consideration will show that this is incorrect: relaxation of the torque measuring system (proving rings in the small ring shear), causes further movement in the ordinary direction of shear. It is possible, that, the different behaviours are the response to strain rate effects.

Landslides "spread" as they fail (Figure 3.4), and come to rest in a less steeply-inclined profile than pre-failure. The mean depth to the slip surface is also reduced in this process. The mean normal stress is thus reduced. Ignoring undrained pore pressure reductions due to unloading (Dixon & Bromhead, 2002), it is likely that mean normal effective stresses are also reduced. Slip surfaces form early in this movement, and so the gentle sweeping curve found in a test may be, in part, a factor in the sedateness of many movements (Bromhead & Clarke, 2003).

Harris & Watson (1997) describe the standard procedure in use at Kingston University for many years. In this procedure, no unloading stages are carried out before reducing the torque carried on the sample to zero, usually with the rotation drive switched off. Subsequently, normal stress unloading does not cause inadvertent movement on the slip surface; and on restarting the machine drive, the residual conditions at the lowered normal effective stress are found via a stiff curve, as though a loading, rather than unloading, path was being followed. This is illustrated in Figure 3.5. Figure 3.6 shows the small "peak" that is found when one of a variety of strength regain processes have acted (Angeli et al., 2004).

Harris & Watson (1997) can see no benefits in conducting separate consolidation stages in between shearing stages, as recommended by BS1377.

3.3 Strain Rate

Investigations into the effect of varying the strain rate on residual strength began in the nineteen-sixties. The reported results fall into several classes:

- Results that show no rate dependency
- Results where the rate dependency is very small (and usually positive)

- Results where the rate dependency is significant (and where it is commonly positive in some soils and negative in others).

Even where the rate effect is significant, it is usually only large at comparatively fast rates of shear. Figure 3.7 gives the landslide speed classes according to Cruden & Varnes (1996). Even slow ring shear tests fall in the middle of the Moderate landslide velocity class. Very rapid ring shear tests lie in landslide velocity classes where engineering remedial works are difficult to impossible.

Early investigations were carried out by Novosad (1964) on granular materials that displayed no variation in residual strength at rates ranging from 300 mm/min to 3000 mm/min using a modified Hvorslev apparatus (Taylor, 1998). Sassa (2004) confirmed, that strain rate effects on granular soils (tested using a large diameter ring shear at very fast strain rates) were non-existent.

The ring shear apparatus makes it very easy (in comparison to testing in direct shear in a shear box, or using a triaxial cell) to examine the effects of strain rate. Even at high rates of shear, the test must eventually become drained.

There are different and conflicting views on rate dependency in clays. Firstly, there are a small group of researchers who could find no evidence for a rate effect. Among those were Ramiah et al. (1970), who found no significant variation of residual strength at strain rates ranging from 0.023 to 19.2 mm/min. Garga (1970) had similar findings from tests, using the NGI/IC ring shear at strain rates between 0.0038 and 0.0076mm/min on silty Blue London clay. La Gatta (1970) later confirmed these conclusions, using the Harvard Ring shear and simultaneously found Pepper Shale & Cucaracha Shale to be rate insensitive.

Secondly, and perhaps the widest group of researchers, found that an increased strain rate resulted in an increased value of residual shear strength. Petley (1966) used the reversal shear box technique, with varied rates between 0.00004 mm/min and 0.10 mm/min using Brown London clay where a 4% increase in residual strength between the quoted range of strain rates was observed. La Gatta at Harvard (La Gatta, 1971), similarly, found that Blue London clay observed at strain rates between 0.0048 mm/min and 2.4 mm/min to exhibit an increased value of residual strength with increasing rate. At the same time, La Gatta used Bearpaw Shale for the same tests, with the identical strain rates, which resulted in variations in residual shear strength. During the seventies, there were other investigations that showed an increase in residual strength with increased strain rate. Cullen & Donald (1971) observed first, what Martins (1983) defined as threshold strength (Taylor, 1998). Likewise it was Gostelow (1974) who first observed Fast Peak strength, as defined by Martins (1983) (see Taylor, 1998). These investigations were on varying types of soil, on both ring shears and shear boxes with reversals.

During the eighties, the further characterisation of the effects was developed. Lupini (1980) postulated, that drained residual shear mode, was linked to rate effects in residual shear strength. The effect of increasing the strain rate was linked by Lupini to increasing the viscous effects which would cause an increase in interparticle friction. As residual angle Φ is a function of inter particle friction, it therefore results in a corresponding increase in residual strength. Lupini et al. (1981) then categorized effect of increasing strain rate into three different modes; sliding mode, turbulent mode, and a transitional mode in between the other two.

- Sliding mode is characterized by particle reorientation and shearing. This does not involve a change in volume.
- Turbulent mode was defined by Lupini et al. (op. cit.), as involving particle rotation and translation, as opposed to pure sliding.

Lupini and his colleagues then postulated that between these two extreme modes, there would be a transitional mode that exhibited a selection of characteristics from both mechanisms.

Lemos (1986), in the same way, discovered fast rates to have a considerable effect on the residual strength and in consideration of this finding, confirmed Lupini's theory of different shear modes. As part of his research, Lemos (1986) investigated the drop in shear strength at the faster rates and attributed this to the high void ratio shear zone that is formed with the increased shear rate. The high void shear zone is upheld by the dispersive pressure of grains, which equates to the applied normal stress, and, with this, usually an increased volume of the sample. Resulting, initially, in negative pore pressures, the high remoulding activity in the shear zone causes these negative pore pressures to dissipate, which leads to increased water content in the shear zone thus, reducing the shear resistance. Lemos (1986), attributes the initial gain in strength to a combination of pore pressure and viscous effects.

Martins (1983) defined various terms in the fast shearing process that he summarized into four rate dependent strength effects. Taylor (1998) noted Martins' suggestion that a critical rate will exist below which there is no particle disorientation with the shear zone. Lemos (1986) related the shear mode to types of effect, thus categorizing them into positive, negative and neutral effects. Tika (1989) then correlated Lupini's shear modes and Lemos' effects, with clay fraction, plasticity index and granular void ratio. Tika used this correlation as the basis of a model containing five different responses to fast shearing.

In the nineties there were further investigations not just focused at Imperial College but from at other institutions as well. Starting with those other than Imperial College, there was Bracegirdle et al. (1991), who related shear strength to the rate of shear. Bracegirdle et al. then integrated their findings with practical implications for use in the prediction of the long term behaviour of landslides with pre-existing shear surfaces and which were subject to toe erosion. These predictions were then investigated using a case study that allowed Bracegirdle et al. (1991) to simulate trigger movements. Bracegirdle et al.'s observations enabled the design of adequate drainage and thus, successfully, prevented further slipping at the particular location cited in the paper.

Parathiras (1994) categorized the effect of strain rate into two types of behaviour, each with its own sub type. Of these, type 1 is a positive rate effect, where post-threshold strength and fast residual strength have higher values than with slow-residual strength. The sub type 1A is the same with the exception of post-threshold strength that has a lower value than slow residual strength. Type 1A is the opposite of type 1 and type 2A is only different to type 1A due to the fact that, fast residual strength although initially stabilizing above slow residual strength, ultimately stabilises at a lower value.

Tika et al. (1996) then investigated the effect of fast strain rates on the shear zone and the mechanisms involved thus linking the two concepts. The conclusion of her research was that there is a possible soil model which links the strength of the pre existing shear zone with displacement and rate of displacement which produced useful practical implications in stability and prediction of sliding studies.

Taylor (1998) investigated the effect of rates of shear up to a maximum of 1000 mm/min and confirmed the existence of positive and neutral effects. Taylor defines the critical effect as being independent of soil grading and as being a determining factor as to positive and negative effects. In detail, Taylor describes negative rate behaviour as occurring when soils display a critical effect curve when the shear velocity is higher than the critical rate. Taylor then attributes fast peak strength to sample dilation, causing particle disorientation and negative pore pressures generated by the rapid dilation.

Tika and Hutchinson (1999) investigated the Vaiont landslide slip-surface using ring shear tests. Using rates above 100 mm/min, the minimum fast strength achieved according to Tika & Hutchinson was 60% below the slow residual strength, thus indicating the existence of negative rate effect. Moreover, Lemos (1999) attempted to categorize the effect of varying the strain rate and fast residual strength. Lemos postulates that, soils, which exhibit turbulent shear, will have a neutral or negative rate effect. Soils which exhibit sliding shear will have a positive or negative rate effect.

A major part of the research during the last decade or so has been into the categorization of the effect of increasing the strain rate that is summarized in Parathiras (1994) and Taylor (1998). Recently Fearon et al. (2004) have investigated the existence of negative rate effects due to strain rate and their findings indicate that, the role of apparatus set up is more important than at first indicated.

Rather paradoxically, both positive and negative rate effects can help resolve some problems in landslide mechanics. Tika & Hutchinson (1999) rely on a strong negative rate effect to account for the high speed of the Vaiont landslide, but Bracegirdle et al. (1991) find a positive rate effect useful for explaining very slow landslide movements.

Research to date has not been successful in determining whether such rate effects are genuine properties of soil, or are a result of laboratory test equipment and methods. The one constant factor is that all researchers do agree that, at very slow rates of strain, the residual strength is constant.

3.4 Enlarging slip surfaces: "basal incorporation"

Hutchinson's investigations from his 1970 paper into the mechanics and physical properties of a particular coastal mud flow or mudslide on North Kent Coast (Hutchinson, 1970), showed that the landslide appeared to scour its bed. A process which Hutchinson termed "basal incorporation". This process is further discussed by Bromhead & Clarke (2003) - see appendix A where it is reproduced. The process is difficult to explain, as the shear stresses transmitted through a slip surface at residual strength are not enough to shear out soil below the slip surface at its peak strength. Hutchinson's mechanisms include moisture transfer and abrasion by hard particles in the mudslide. Bromhead & Clarke suggest that neither theory are very convincing, since the amount of moisture transfer would need to be huge , and for instance a weak mudslide could not grip "scrapers" to the same extent. Similar processes to this occur in a glacier where the process has been well documented and understood.

Indeed, Bromhead & Clarke point out that Hutchinson's mudslide (Figure 3.8) travelled mainly along a preferred bedding-controlled basal shear and that further basal erosion was improbable. Unable to find a convincing mechanism to explain this phenomenon, possibly the only useful suggestion made by Bromhead & Clarke was the mechanism which is associated with the mechanisms of strength regain (Angeli et al., 2004). This requires that the mudslide has a period of inactivity, over at least one stage of its annual cycle. Recementation with calcite or iron compounds, during this stage, would help. Finally, strength regain on a slip surface results from void ratio changes as the effective stresses are cycled (Figures 3.8 and 3.9).

3.5 Pore Pressure Effects on Residual Strength

Berander et al. (1985), using the direct shear apparatus on Swedish clays, suggested, that the loss of residual strength at faster rates was due to a decrease in effective stress. This reduction in effective stress was attributed to by localised increases in pore pressure by Berander et al. Tika (1989) questioned the validity of Berander's findings due to the length of displacements, resultant from the direct shear box's limited facility for this.

Idriss (1985), as part of his evaluation of seismic risk in engineering practice, identified that earthquakes cause a complete reorientation of clay particles and change the pore water conditions (Parathiras, 1994). This causes a reduction of between 30% and 70% of the peak strength, dependent on the amount of displacement. Sassa et al. (1991) and Sassa (1992) concluded that there was no significant pore water pressure increase in the shear zone during fast shearing (Taylor, 1998). Sassa in 1992 postulated that, this lack of increased pressure could be the result of opposing dilating and contracting mechanisms which result from cyclic loading. Although conversely, Sassa discovered that when the normal stress was decreased and shearing velocity increased, resulting in a significant rise in pore water pressures there was a resultant negative rate effect.

There were simultaneous investigations into the effects of pore water pressures by other researchers with Sassa et al. (1991) and Sassa (1992). Yagi et al. (1991) measured pore pressures in the ring shear and discovered a boundary rate of 0.35 mm/min above which the soil strength is affected by generated pore pressure (Parathiras, 1994). The main emphasis of Yagi et al.'s research was actually to investigate how displacement varies pore pressure. From this Yagi et al. concluded that the first 80 mm of displacement produced the maximum value of pore pressure during shearing. The next 80 mm of displacement results in a maximum of 80% decrease in pore pressures. After 160mm of displacement the pore pressures were observed to continue the decline to a minimum of 10% of the maximum value as displacement increases. This minimum value usually occurs at around 400 mm displacement and any further displacement does not result in any further reduction in pore pressures. By monitoring pore water pressures, Shoaei and Sassa (1994), investigated rapid landslides using the NGI/IC ring shear to test the shear behaviour of landslides during earthquakes (Parathiras, 1994). Pumice, loess and toyourna sand were used for this simulation of the rapid landslides, which indicated the cause as being the build up of pore pressures. The seismic co-efficient of failure will also vary dependent upon whether the slope is drained or saturated. Parathiras (1994) postulated a characterisation of the causes of negative rate effects and classified them as the following:

- dilation of shear zone,
- increase in porosity of shear zone
- increase in particle rotation in one zone

All the above causes lead to an increase in pore pressure. Parathiras commented that the water availability affected the behaviour of the soil i.e. the ability for the sample to be submerged, as Fearon (2004) confirmed to be the critical factor for negative rate effects to occur. Tika et al. (1996) had previously speculated that negative rate effects were not due to a reduction in effective stress as a direct result of positive pore pressures, but, were in fact, due to the increase of porosity and water content within the shear zone.

After pursuing the same line of enquiry, Lemos (1986) sought to confirm using tests on the IC/NGI ring shear apparatus, the disturbance of the shear zone, which occurs during the fast stage in turbulent mode, leads to an increase in void ratio, which causes an increase in water content. Lemos then postulated that the decrease in shear resistance could entirely be due to the increase in water content. As a continuation of this research, Lemos investigated the changes in pore water pressure, which showed initial decreases in pore water pressure during shear. Lemos found that the post-peak strength is due to the dissipation of negative pore water pressures with associated increase in water content. Lemos attributed the minimum strength as a function of void ratio at the fast rate. Lemos confirmed the existence of positive pore water pressures by direct and indirect measurement at the end of the fast rate of shear thus demonstrating, the generation of high water content in the shear zone.

Rather controversially, Fearon et al. (2004) investigated the effects of empty and full water baths and produced some interesting results. Their research has indicated that negative rate effects only occur when there is no water in the water bath which can, therefore, be interpreted, that the effect is due to movement of pore pressures and to some degree the design of the apparatus.

Pore water pressures play an active role during the shearing process. At fast rates, the void ratio changes and, this generates a change in water content, which directly affects the pore water pressures. The higher the moisture content, the lower the strength of the soil. More moisture means more water molecules therefore void space is expanded to accommodate them, thus resulting in soil particles being pushed apart. The increased void space is therefore, mostly filled by water, which is less compressible than air, which would be compressed out whereas the water remains within the soil. The actual chemical composition of the clay or soil has an effect on its value of residual shear strength.

3.6 Soil Chemistry

Approximately four decades ago, investigations into the effect of mineralogy and pore water chemistry began. Initial investigations into the effect of plasticity, water content, rate of strain, particle friction <2μ and clay fraction on residual strength of natural clays and mixtures were performed by Kenney (1967). From these, Kenney concluded that only clay fraction had an effect on residual strength of natural soils. The mixtures produced the same conclusion. Skempton (1985) later confirmed Kenney's (1967) conclusion on the effect on residual strength of clay fraction.

Ramiah et al. (1970) investigated the effect of change of concentration of particular chemicals on residual shear strength, which they proved to be insignificant. Although, Ramiah et al. did find that cation valency and ion concentration in the pore fluid did affect the structure of the soil, and thus, caused the residual strength to vary. Ramiah et al. concluded that calcium divalent ions and high ion concentration in the pore fluid resulted in a flocculated structure, with a higher corresponding value of residual shear strength. Conversely, sodium monovalent ions and a low ion concentration produced a dispersed structure with lower residual shear strength than with calcium divalent ions. Ramiah et al. also investigated the effect on residual strength of rate of strain between 0.0223 mm/min and 10.16 mm/min. and initial water content, and, confirmed Kenney's conclusion (1967), that there was no effect on residual strength

Continuing on from earlier investigations, Kenney (1977) researched the effect of mineral composition, chemical state, relative volumes of clay mineral matrix and massive minerals on the residual strength of natural soils and mineral mixtures. Kenney demonstrated for most of the tests, residual strength was higher for an increased content of massive minerals except with the Hydrous-mica III, where the reverse occurred. During Kenney's investigations, potassium mixtures were found to have higher residual strengths than those of sodium mixtures. Kenney (1977) concluded that the residual strength of a mixture is strongly dependent on the relative contents of massive minerals, montmorillonite and on residual strength characteristics of the component minerals. Specifically, for sodium montmorillonite, Kenney found it to be dependent on the concentration of sodium chloride in the pore fluid.

Lupini (1980) concluded, residual strength is dependent on mineralogy and pore water chemistry as these affect the dominant particle shape and the inter particle friction. Lupini et al. (1981) concluded amongst other contributing factors that, pore water chemistry dictates the type of shear mode. Conversely, Lemos (1986) concluded, that turbulent shear mode was not affected by changes in pore water chemistry. Although, Lemos did conclude that Van der Waal forces were most active in lower ion concentrations which the author notes are weak bonds and therefore would result in lower residual strengths.

Moore (1991) investigated the effects of chemical composition and mineralogy on the residual strength of pure and natural clays. Moore concluded the magnitude of cohesion and the intrinsic effective stress components of shear strength are dependent on the direct ionic and hydrogen bonding between the particles. Whilst investigating the physico-chemical behaviour of clays influences on these bonds, Moore concluded that, shear strength was affected by fluctuations in pore water chemistry, weathering of clay minerals and exchange of one type of cation for another. Moore felt that greater attention should be given to the above factors when measuring residual strength. Moore concluded that, the chemical properties do not affect the intrinsic frictional component of shear strength and the limit of the effect of the strength of the chemical bonds was found to be its effect on the soil cohesion. Moore (1991) confirmed the Ramiah et al. (1970) conclusion regarding the type and concentration of cations. In addition, Moore found that calcium divalent cations resulted in higher residual strength than sodium monovalent cations.

Steward & Cripps (1998) and later, Moore & Brunsden (1996), had made modifications to the ring shear which immediately put the chemicals in direct contact with the shear surface. During both these sets of investigations, pore fluids of different chemistry had been passed through the specimen during the test. Bromhead and Harris commented that by over pressurizing the system, it causes the pore fluid to act like the air beneath a hovercraft.

Skempton (1985) investigated the effect of different clay minerals on shear mode and direction. From this, Skempton noted, that clay minerals have different angles of interparticle friction and due to the direction of shear; the residual angle approaches the angle of interparticle friction. La Gatta (1971) attributed the decrease in strength to residual in normal consolidated soils as partly due to bond breakage between particles. A 10 kPa pressure injection of fluid reduces the effective stress by approximately the same proportion as the detected effect.

The effect of pore water chemistry has been found to have a significant effect on residual shear strength, especially due to the type and concentration of cations. These contribute to the bonding within the clay which results in the variation in strength.

3.7 The effect of clay fraction

It was Skempton (1964) who during his 1964 Rankine lecture postulated the link between increasing clay fraction and decreasing residual friction and that more research was required in this area.

Skempton linked the increase in residual strength with the increase in silt content in soils comprising a mixture of clay and silt particles. Lupini et al. (1981) noted Borowicka's (1961, 1965) conclusions with regard to the effect of colloidal content on residual strength. Borowicka had concluded, that in remoulded clays with a low colloidal content, for example silty clays, then the angle of internal friction is unaffected by repeated shearing. However, Borowicka concluded that for clays with a high colloidal content, a continuous reduction in internal friction until it reached a very low final value was exhibited. This was attributed by Borowicka to the "scale-like and flake-shaped colloids" which re-orientate themselves towards the direction of the shear plane eventually forming a continuous, shiny shear surface; an explanation still found satisfactory today.

In addition, Borowicka demonstrated, via his investigations, that for clays which are more than 43% clay, part of the shear plane was shown to be polished where the two parts of the specimen adhered to each other. The final outcome of these investigations was Borowicka's observation that, with decreasing colloidal content, the "interlocking effect of the coarse-grain fraction" was shown to be more effective in determining shear strength.

Petley (1966) observed, as Skempton (1964) had postulated, a distinct tendency for Φ'_r to decrease with increasing clay fraction. This tendency was less apparent with clay fractions above 60%; and below 25%, Φ'_r was approximately equal to Φ'_p. Petley based these correlations between Φ'_r and clay fraction on Skempton's (1964) conclusions. Lupini et al. (1981) noted Chandler's (1966, 1969) conclusions that the actual percentage clay fraction is higher at the shear surface than the adjacent soil, which to Chandler, had indicated a breakdown of the soil structure during the process of shear.

There are a group of researchers, who, during the seventies came to the overall general conclusion, that clay mineralogy and particles within the clay had a major affect on residual strength. Townsend and Gilbert (1974) concluded, that Φ'_r was dependent, only, on clay fraction and clay mineralogy (Taylor, 1998). Vaughan et al. (1975) and later, Vaughan et al. (1978) were of the opinion that the governing parameter of residual strength was the proportion of clay minerals (Taylor, 1998). Kenny (1977) had concluded from his tests on clay, water and quartz mixtures that as the percentage of quartz increased, the structure changed within the matrix and the quartz then controlled the ultimate residual strength. Wesley (1977) confirmed the importance of particle shape in clays with a high residual strength (Lupini et al., 1981).

Early researchers had investigated the dependence of residual strength on clay fraction and this continued during the eighties. Skempton (1985) attributed the post peak decline of residual strength in clays with a clay fraction lower than 20%, to an increase in water content which results in the actual value of residual strength. For clays with a higher fraction of more than 40% clay, Skempton concluded, this decline was due to a combination of increased water content, particle re-orientation and the large displacements required achieving residual strength. The development of the ring shear followed from the displacement requirement. Lemos (1986) continues these investigations and concluded that residual strength was entirely controlled by clay minerals when the clay fraction is above 50%. However, Lemos noted that there was no dependence on displacement rate for soils with 0% or very small clay fraction. The dependency is apparent in moderate clay fraction soils where, a significant decrease in residual strength is exhibited for an increase in displacement rate according to Lemos (1986).

Research then began to focus on the type of shear that the soil exhibited. Lupini et al. (1981) linked the three types of shear mode with different particle compositions. For turbulent mode, the soil was shown to have a high proportion of rotund and platy particles, thus, exhibiting high values of inter-particle friction. For sliding mode, the soil was shown to consist of low strength platy particles. In between the previous two modes, a transitional mode was found by Lupini et al. to be dependent on particle packing and the porosity of rotund particles. Lupini and colleagues argued that the shearing mechanism changed as the clay content of the cohesive strength increased.

Lemos (1986) then categorised strain rate effects with the introduction of positive, neutral and negative effects through relating fast residual strength to slow residual strength. Lemos noted that, the type of effect, to an extent, was dependent on clay fraction. Positive effects were exhibited in soils with more than 50% clay fraction. Less than 5% clay fraction resulted in no change; in other words, a neutral effect. Negative effects were exhibited by soils with a 5% to a 40% clay fraction.

Tika (1989), Parathiras (1994) and Tika and Hutchinson (1999), further investigated the effect of clay fraction on residual strength; not just on type of shear mode but, how clays of different clay fractions were effected by different strain rates. Rate effects were concluded by Tika and Parathiras to become more significant, as the clay fraction increased. Soils with a moderate clay fraction were shown to exhibit a turbulent or transitional mode of shear which, resulted in a lower, fast residual strength compared to its slow residual strength. Soil with a high clay fraction was shown to exhibit a sliding shear mode which resulted in a higher, fast residual strength when compared to its slow residual strength. The fast residual strength was found by Tika and Parathiras, to increase as the rate of shear increased.

Tika (1989) included clay content, sand content and silt content within her classification of the rate effects table. Soils for the clay fraction of less than 5% were found to exhibit no change in residual strength, and therefore, were independent of strain rate and neutral effect. Soils consisting of greater than 50% clay, were concluded by Tika, to show an increase in residual strength as strain rate increased; a positive effect. The moderate fractions of clay, between 5% and 40%, exhibited a decrease in residual strength, as strain rate increased, which is a negative effect. Tika (1989) noted a positive effect was more apparent in clay soils which contain massive particles.

Tika and Hutchinson (1999) noted that as clay fraction increased, the residual friction angle decreased. From this observation, Tika and Hutchinson concluded, that the slow residual strength was dependent on clay fraction but, the fast residual strength was independent and therefore, not affected by changes in clay fraction.

3.8 Other influencing factors

Over the years, other factors have been investigated in order to determine their effect on residual strength; and which, among others, include plasticity index, apparatus design, influence of disturbed versus undisturbed samples and the actual size of the sample to be tested which are reviewed in this section.

The effect of plasticity index has been investigated over a period of nearly forty years. The conclusions of this research fall into four categories as demonstrated by Figure 3.12:

- A decreasing $\Phi'r$ with increasing plasticity index
- A rough correlation indicating a decreasing effect $\Phi'r$.
- A discontinuous relationship between plasticity index (Ip) and $\Phi'r$, where $\Phi'r$ decreases as Ip increases.
- Plasticity index has no satisfactory relationship with residual strength.

Kenney (1967) concluded there was no satisfactory link between $\Phi'r$ and I_p. However, the conclusions of Vaughan and Walbancke (1975) and Vaughan et al. (1978) propose an alternative discontinuous relationship between I_p and Φ'_r (Lupini et al., 1981). Up to 25% I_p, Vaughan et al. (1978) indicate a decreasing effect on $\Phi'r$ as I_p increases. Above 25% I_p, Φ'_r were concluded to be unaffected by any increase in I_p,

A rough correlation between Φ'_r and I_p was noted by Lupini et al. (1981), when summarizing the results of six researchers including Fleischer (1972), Voight (1973), Kanji (1974), Bucher (1975), Vaughan el al. (1978) and Seycek (1978). Lupini et al. indicated a rough correlation between I_p and Φ'_r, with an increase in I_p leading to a decrease in Φ'_r. Lupini et al. concluded that correlations between residual strength and index properties cannot be generally applied.

Seycek (1978) analysed the effect of I_p on the residual angle of internal friction (tg or tan Φ'_r) and concluded that a more specific trend when I_p was plotted on a log scale. Then, a definite trend of decreasing tg Φ'_r with increasing I_p was apparent. Seycek concluded that this was the best correlation. Interparticle friction, tg Φ'_r increases as Φ'_r increases therefore the same correlation must exist between Φ'_r and I_p. Lupini et al. (1981) cited Bucher (1975) to have found a general correlation between decreasing Φ'_r and increasing I_p.

Conversely Wedage (1997) and Wedage et al. (1998) concluded that there was a rough correlation between percentage increases in strength per log cycle increase in shear rate which was found to increase with increasing plasticity index. Both Wedage and Wedage et al. noted that this relationship was general and had a fairly wide scatter. Tika and Hutchinson (1999) concluded, the final category with decreasing Φ'_r with increasing I_p for the case of slow residual strength, fast residual strength was found to be independent of I_p.

Investigations into the effect of plasticity index (I_p) on the shear mode have been carried out over the last two decades. Tika (1989) included I_p as part of her classification of shear modes:

- Type 1 exhibited turbulent mode with Ip of less than 9%, usually sands or silty sands.
- Type 2 exhibited are turbulent or transitional mode, with Ip of between 9% and 21% usually a sandy silt.
- Type 3 exhibited a turbulent or sliding mode with an Ip between 24% and 26%, usually silty clay.
- Type 4 had two variations with slightly different Ip ranges - one for silty clays the other for low plasticity clays. Silty clays exhibited a transitional or sliding shear mode and had an Ip between 26% and 36%. Low plasticity clays exhibited turbulent or transitional shear mode with an Ip between 10% and 21%.
- Type 5 usually high plasticity clays with Ip between 36% and 51% which exhibit sliding shear mode.

Parathiras (1994) and Tika (1989) both correlate the rate effects with plasticity index. Tika (1989) concluded that an I_p between 10% and 37% resulted in a negative effect. Parathiras (1994) concluded that if I_p was greater than 12% then, the effect would be positive. A negative effect would occur for an I_p equal or approaching 0%.

The effect of machine design has long been a background issue in measurement of residual strength. With each development, controversy arose as to the accuracy of the result. Although prior to the development of a commercial and economic ring shear device, the use of shear box tests with reversals were widely used. Lupini et al. (1981) noted the observations of Bishop et al. (1971) and Vaughan (1971) of higher values of residual strength; from the reverse shear box than from the ring shear. These values can be attributed to insufficient displacement, which results in the residual strength not being achieved.

Yagi et al. (1991) noted that back analysis produced lower values of residual strength (Taylor, 1998). This, Yagi et al. concluded, was due to the development of "slickensides" which are routinely produced in the laboratory but are not always present in "real" landslides and are entirely a result of an apparatus design effect. Given the readiness with which slip surfaces are formed in the field, this must count as an extraordinary observation.

Anayi et al. (1989) adapted a Bromhead ring shear by introducing vanes within the sample and increasing the depth of sample from 5 mm to 10 mm. The effect of an imbalance of readings on the torque proving rings had, Anayi et al. claimed, to have resulted in friction on the central pivot and tilting of the loading platen which, he claimed, resulted in an undulation of the failure surface. This resulted in the introduction by Anayi et al. of vanes to stabilise the situation. Taylor (1998) reported the same difficulties with the imbalance on the NGI/IC ring shear, but, concluded that this did not have an effect on the final strength. The increased sample depth, Anayi et al. suggested, was to be applied in two layers, each of a depth of 5 mm. This, Anayi et al., concluded, would encourage the planar shear surface to occur mid sample instead of Bromhead (1979) initial design of just beneath the top platen. Taylor (1998) postulated that Anayi et al.'s 1989 alteration would increase the side friction which would have a significant effect on the value of residual strength.

It is self-evident that friction on the centre pin does affect the torque in the ring shear test, the question is: is it significant? An analysis is presented in Figure 3.13 which indicates, that the amount of friction is dependent upon the amount of grease applied and the maximum value of error is approximately 2%.

Parathiras (1994) designed new confining rings for the NGI/IC ring shear; to reduce soil extrusion, which, he concluded, resulted in greater accuracy when determining a value of residual strength. Bromhead (2004b) suggested that most rate effects were simply a function of apparatus and test procedure, and not fundamental soil behaviour". This, he attributes to soil extrusion due to the small distance to the "free edge" in the ring shear which can be as small as 7.5 mm in direct comparison to a distance of hundreds of metres in the real world in actual landslides. Bromhead noted that the ring shear design (area: perimeter ratio) is actually similar to that of an elongated mud slide. Therefore, he commented, it is not surprising for lateral ridges or levees to develop for the type of slide, by processes of lateral extrusion, similar to those in the ring shear; although in the ring shear the process is assisted by the rotary motion. Furthermore, Fearon (2004) noted the development of an irregular shear surface during testing that resulted in a negative effect. Conversely Lemos (1986) concluded that the design of the apparatus had no effect on the residual strength despite the major soil loss during testing.

Sassa (2004), working on seismic research in Japan, has developed a ring shear apparatus which is capable of facilitating an undrained test. Under these conditions, Sassa has concluded that subject to rapid movement, in other words, earthquake conditions, a sliding surface liquid faction could be initiated. Therefore, if due to a rock fall or a minor slip at the top of the slope, the resulting disturbance of the equilibrium, which occurs after the process of drying-out, could well be the cause of basal incorporation.

Other effects that have been investigated are the effect of pre-disturbing the sample and the size of the actual specimen. Olsen (2002) investigated the effects of ploughing on the strength. From this, Olsen (2002) demonstrated that disturbing the sample had no effect on residual strength by using ploughed and unploughed soil. Olsen did observe that unploughed soil had a lower value for cohesion 9kPa, but a higher value for the angle of friction 46°, than ploughed soil, where the opposite was observed. Cohesion for ploughed soil was higher 22kPa, but the angle of friction 40° was smaller compared to unploughed soil therefore, the effect cancelled itself out and the same result was achieved for residual strength. Garga (1987) investigated the effect of the size of the actual sample using basalt. Angle of friction, Φ', was observed to be unaffected whereas cohesion, c', was affected, which resulted in a reduction in strength as sample size increased, due to the reduction in cohesion.

3.9 Summary

Significantly, the effect of machine design has been by passed by numerous researchers. The effects of strain rate, pore water pressure, soil chemistry, clay fraction have been extensively researched, although there is no definite undisputed cause which has been confirmed. Fearon et al. (2004) have by their findings effectively pushed the effects of pore water pressure and machine design to the fore front of research. Positive and negative rate effects are useful as they give a reason for why landslides behave the way they do, but whether this is the actual cause, is still a controversial research topic. The findings of this research are detailed within chapter 5 and the experimental procedure used is described in chapter 4.

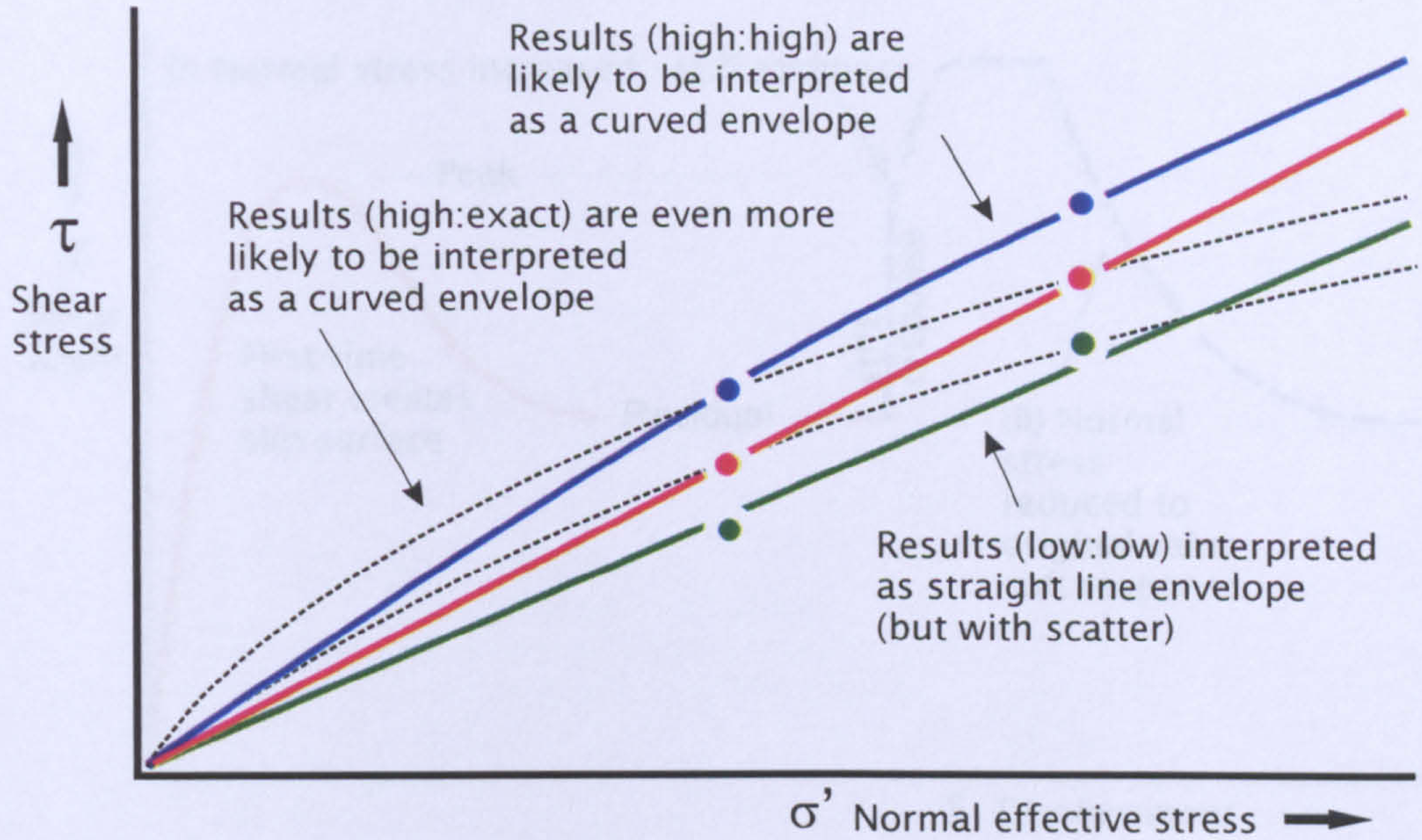


Figure 3.1 There is a tendency to interpret results to fit the experimenter's prejudice. When combined with a notion that the residual strength envelope may be curved (convex upwards), this case is easily found! The majority of combinations of 2 point results and a curve through the origin indicate a curved envelope (see text), and indeed, the fit even looks better (above).

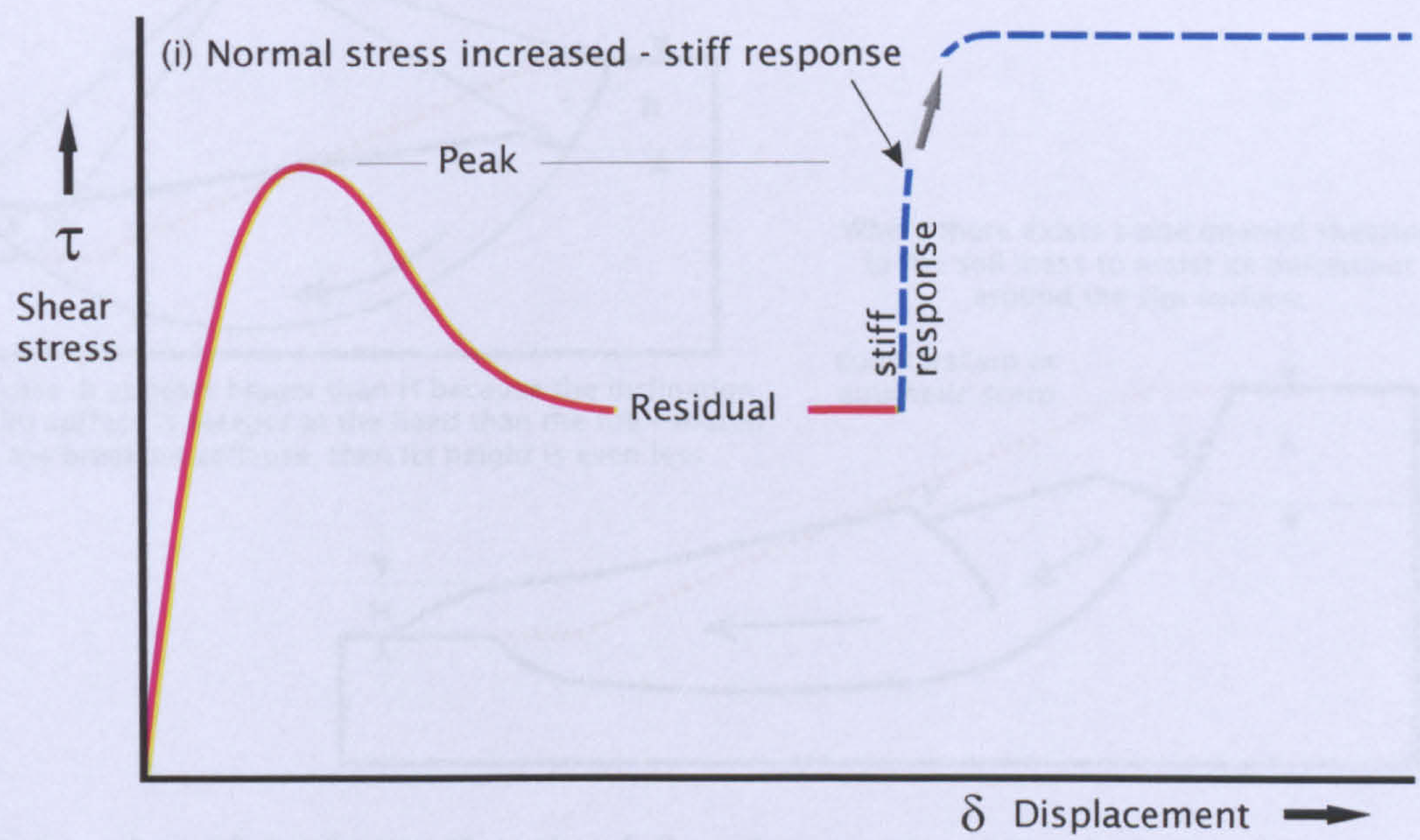


Figure 3.2 After establishing residual conditions on a slip surface, the normal stresses are increased. The load carrying capacity of the specimen increases rapidly ("stiff behaviour"). Several additional stages may be carried out. Practice at Kingston is usually to determine at least 6 points.

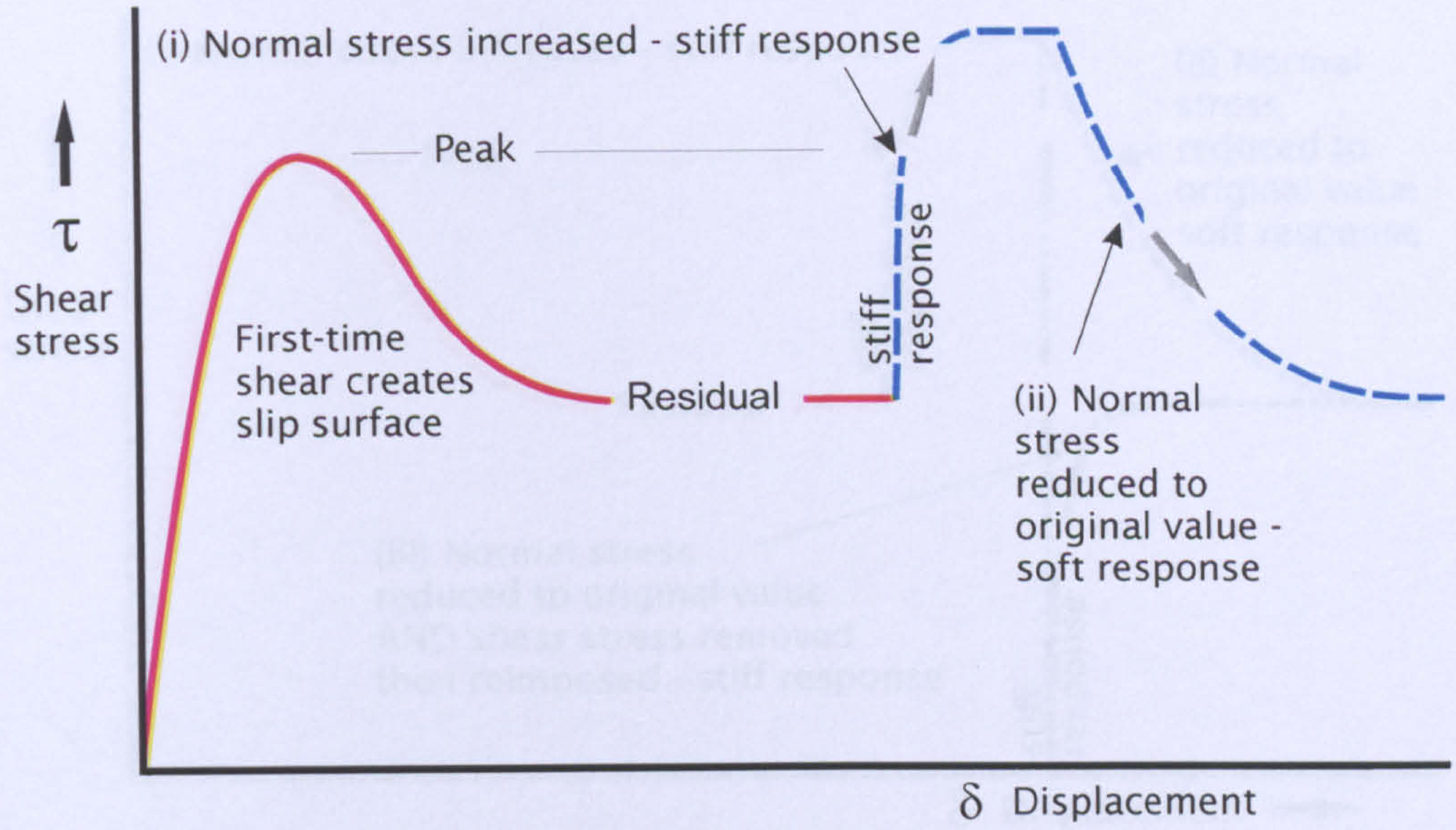


Figure 3.3 The behaviour of the specimen in a ring shear test is different when the normal stresses are reduced. This is a significantly less stiff behaviour.

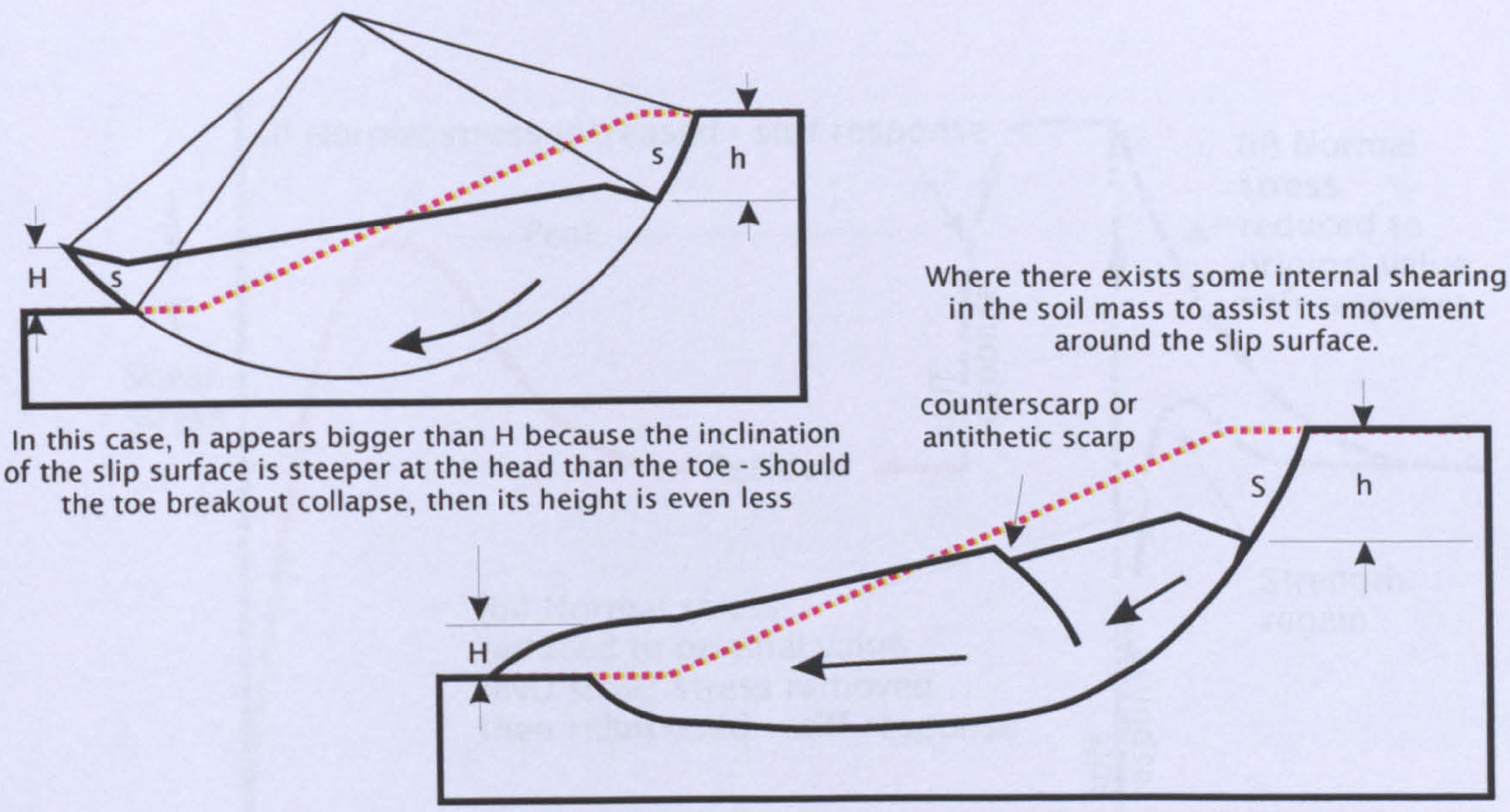


Figure 3.4 Landslides "spread" as they fail, and come to rest in a less steeply-inclined profile than pre-failure. The mean depth to the slip surface is also reduced in this process. The mean normal stress is thus reduced. Ignoring undrained pore pressure reductions due to unloading, it is likely that mean normal effective stresses are also reduced. Slip surfaces form early in this movement (After Ibsen & Bromhead, 1999).

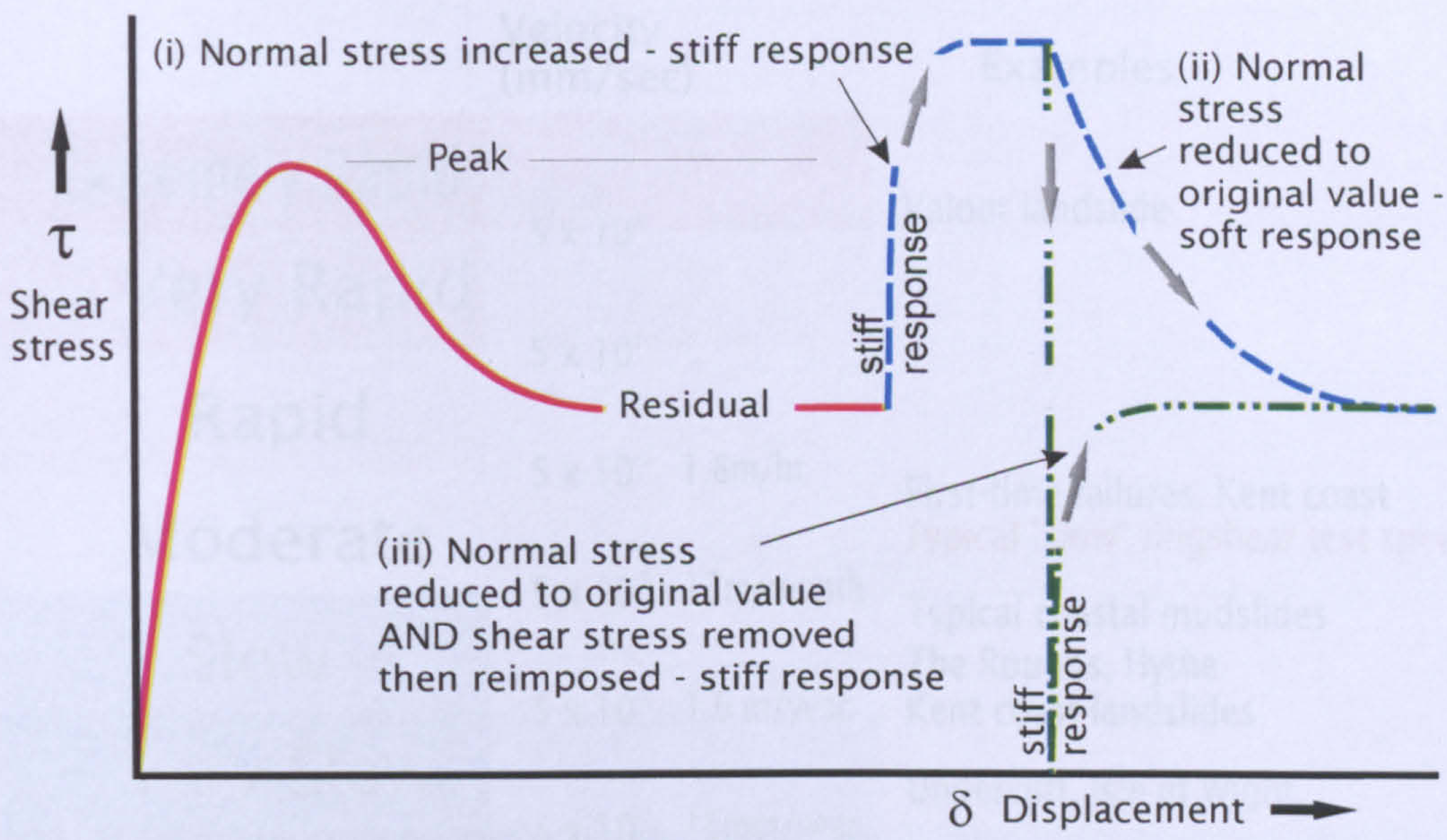


Figure 3.5 Harris & Watson (1997) recommend a procedure that obviates the slow response when performing an unloading stage. This relies on reducing the torque to zero before unloading the normal stress.

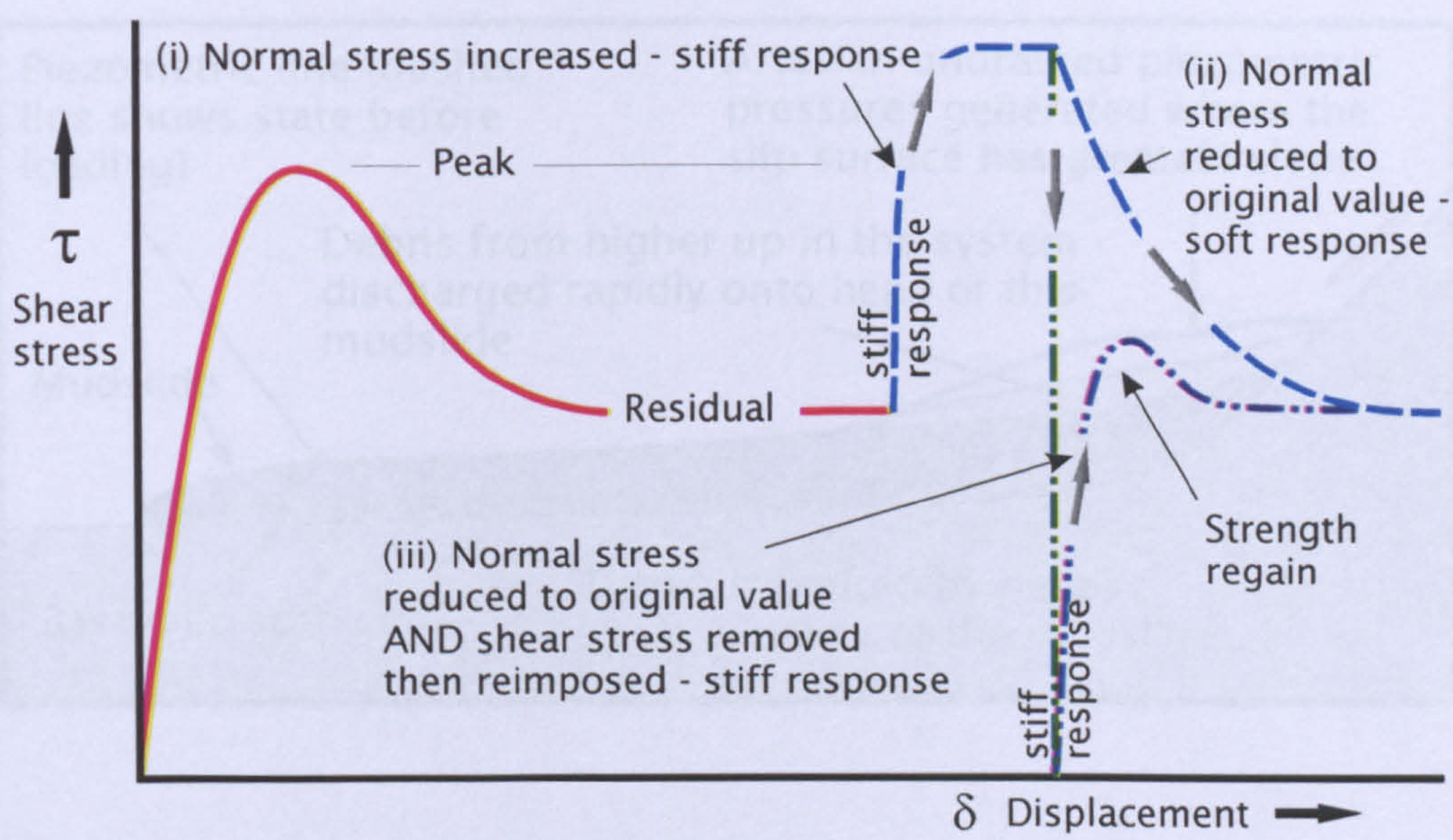


Figure 3.6 In the event that the slip surface is unloaded for any length of time, a variety of strength regain mechanisms may come into play (Angeli et al., 2004) producing a peak in the stress strain plot.

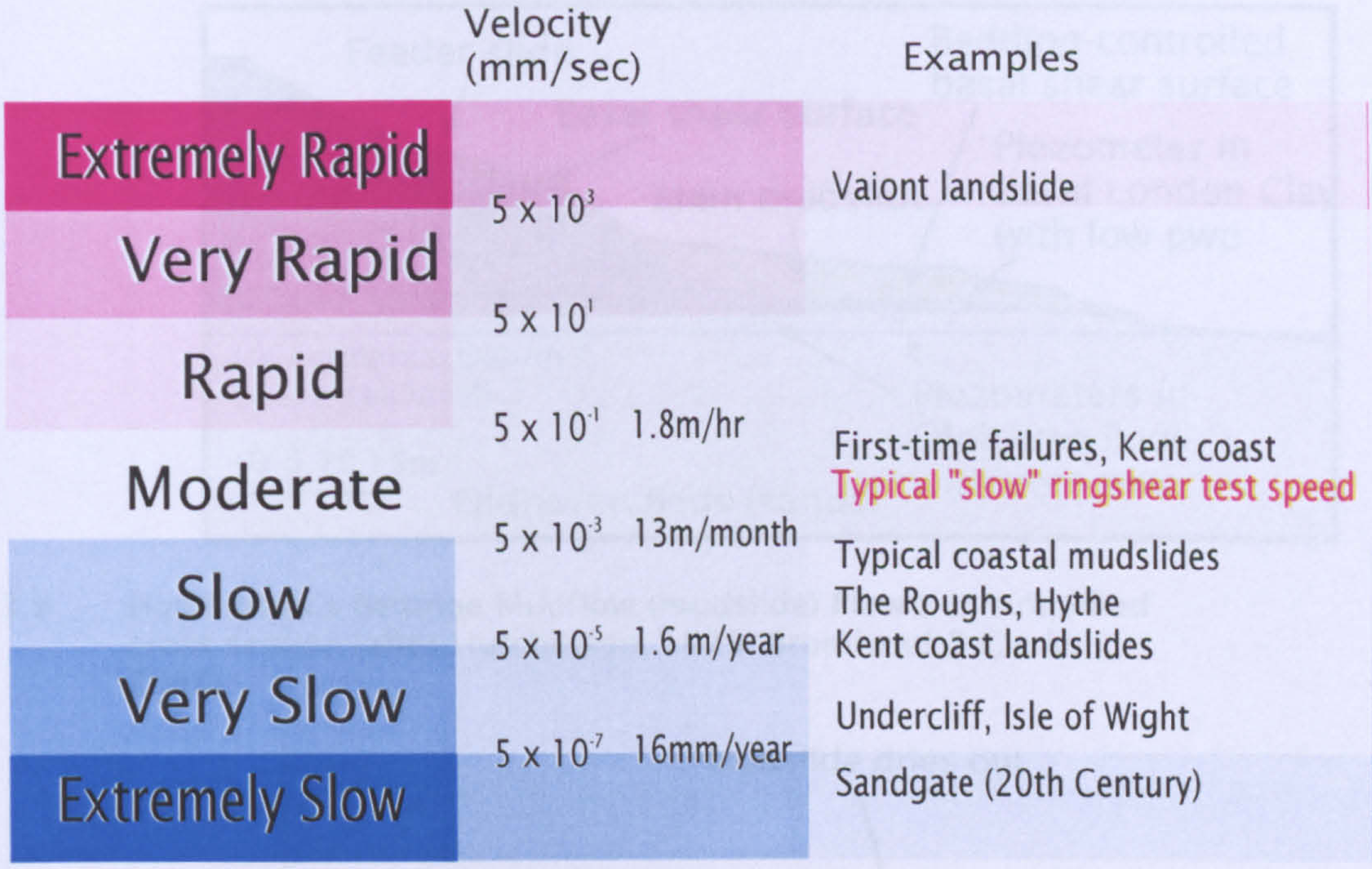


Figure 3.7 Landslide Velocity Classes (Cruden & Varnes, 1996).

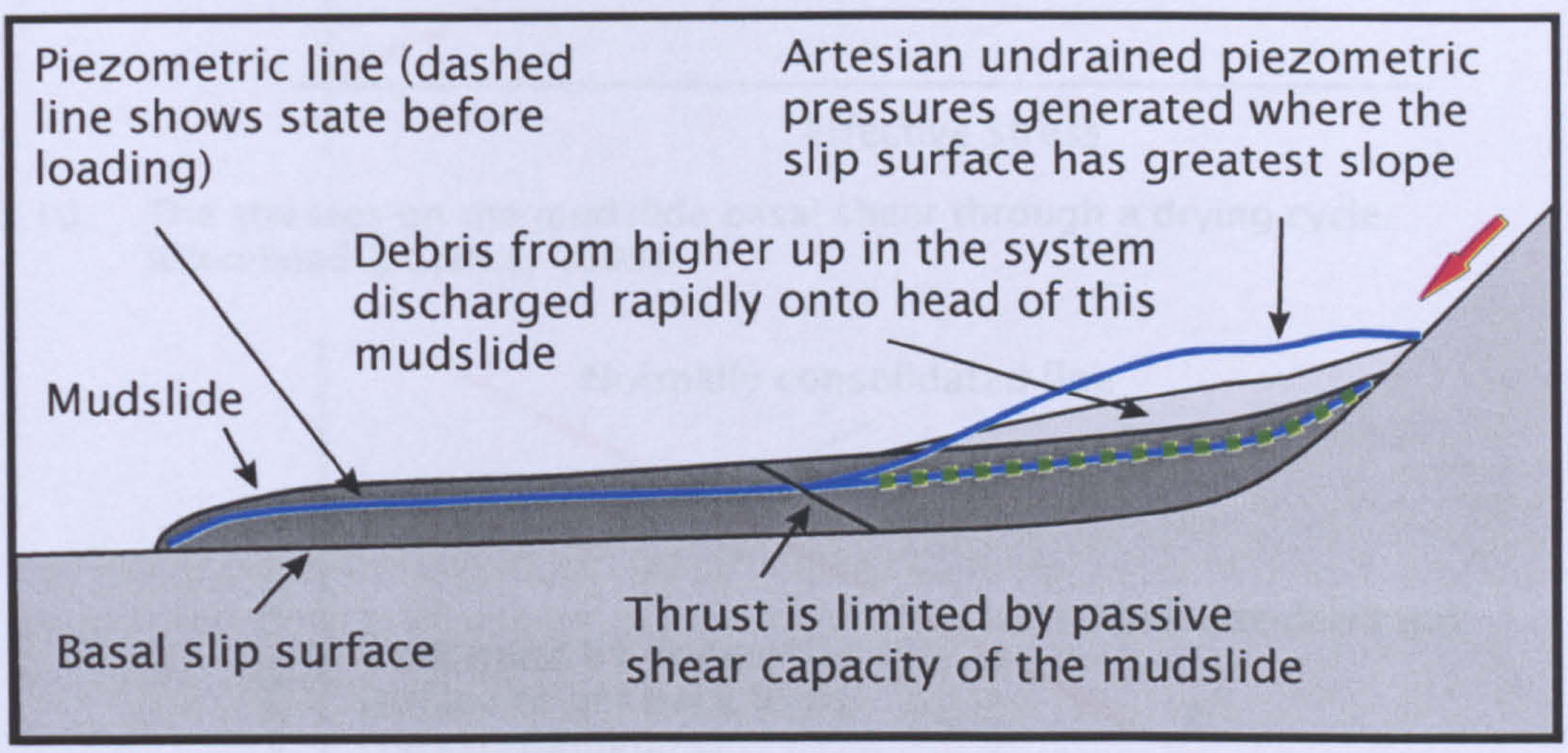


Figure 3.8 Undrained loading mechanism (Hutchinson, 1970)

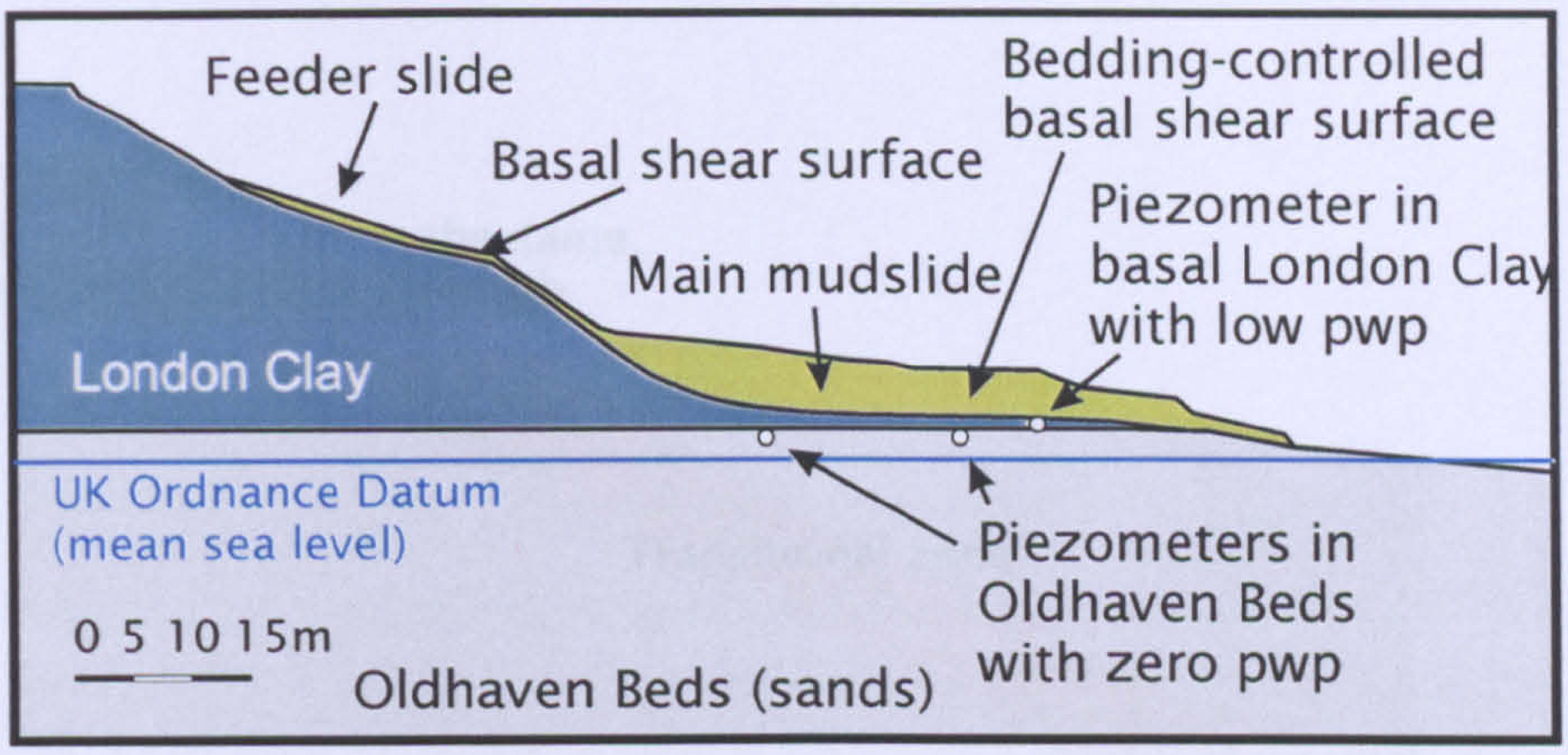


Figure 3.9 Hutchinson's Beltinge Mudflow (mudslide) Flow II in simplified cross section (after Hutchinson, 1970; Bromhead & Clarke, 2003)

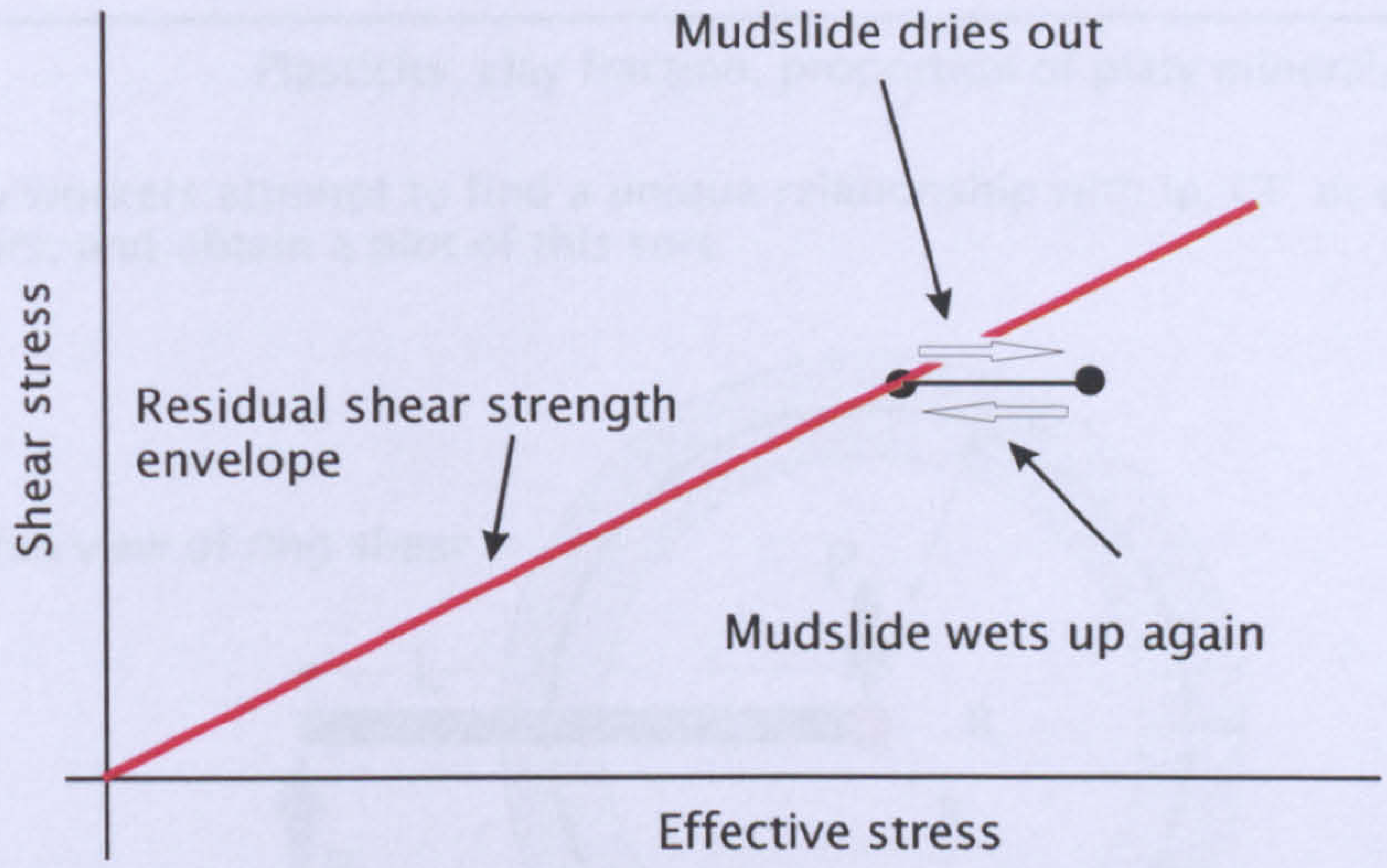


Figure 3.10 The stresses on the mudslide basal shear through a drying cycle. (Bromhead & Clarke, 2003).

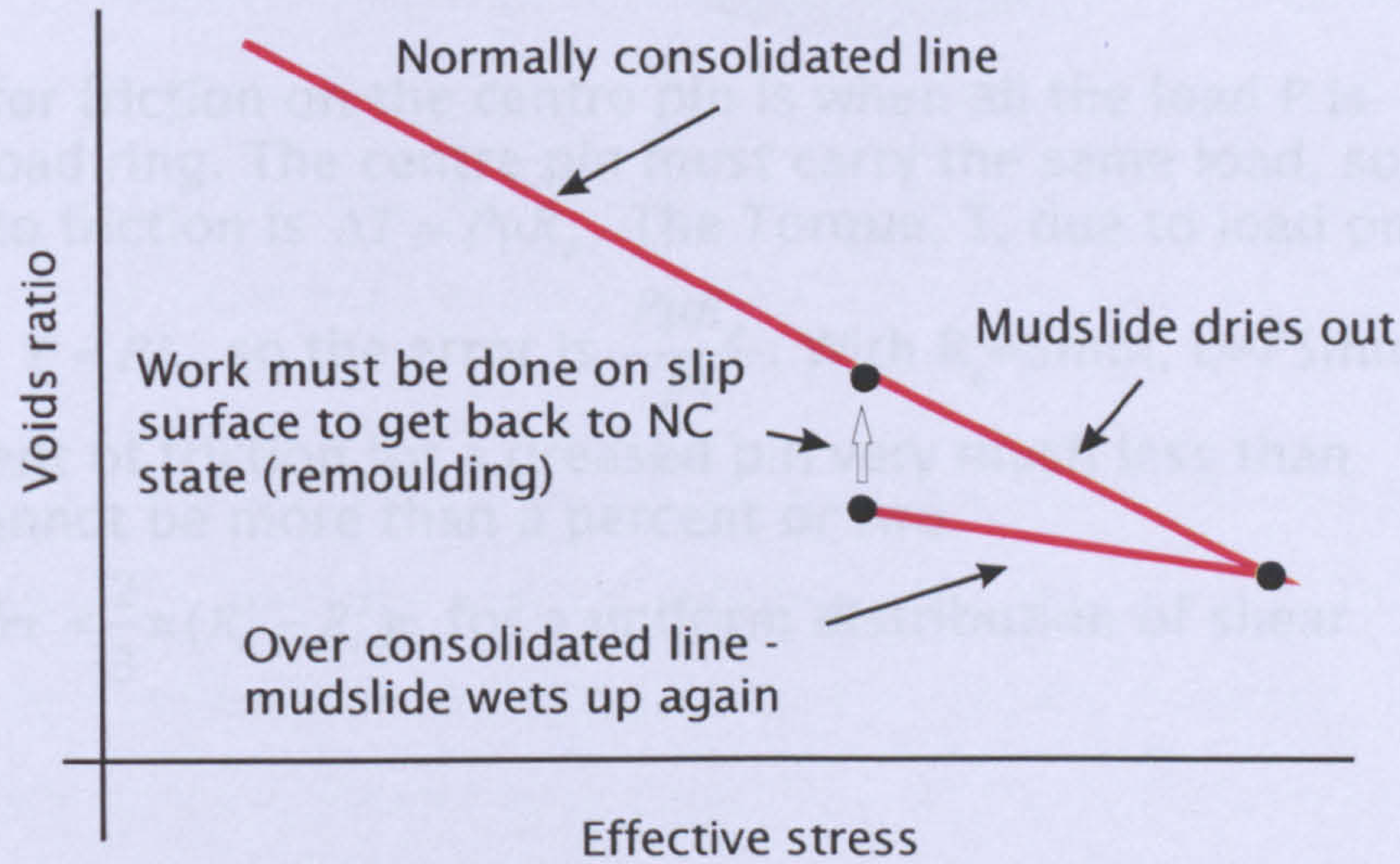


Figure 3.11 The voids ratio changes in a mudslide basal shear through a drying cycle (Bromhead & Clarke, 2003).

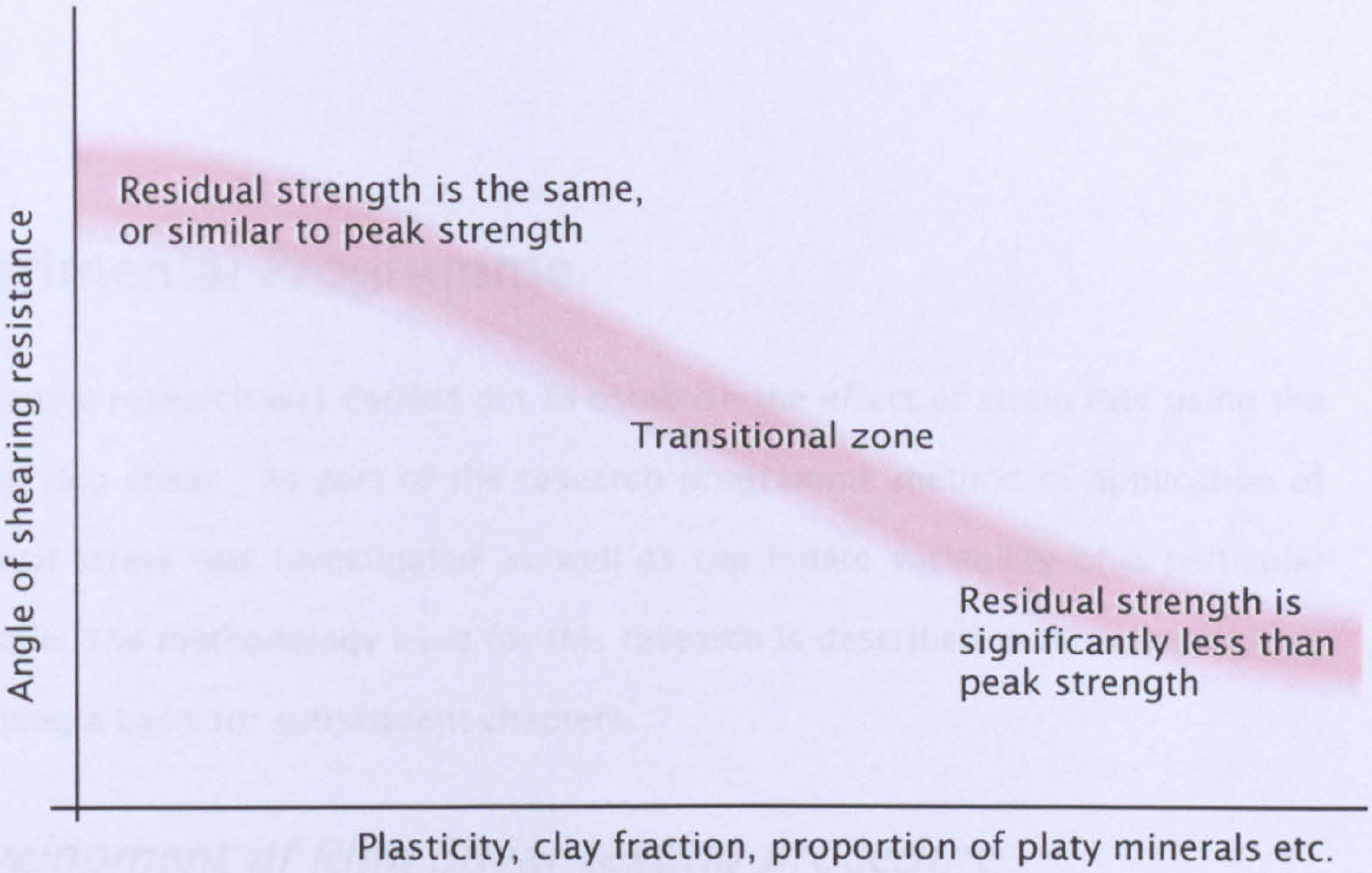
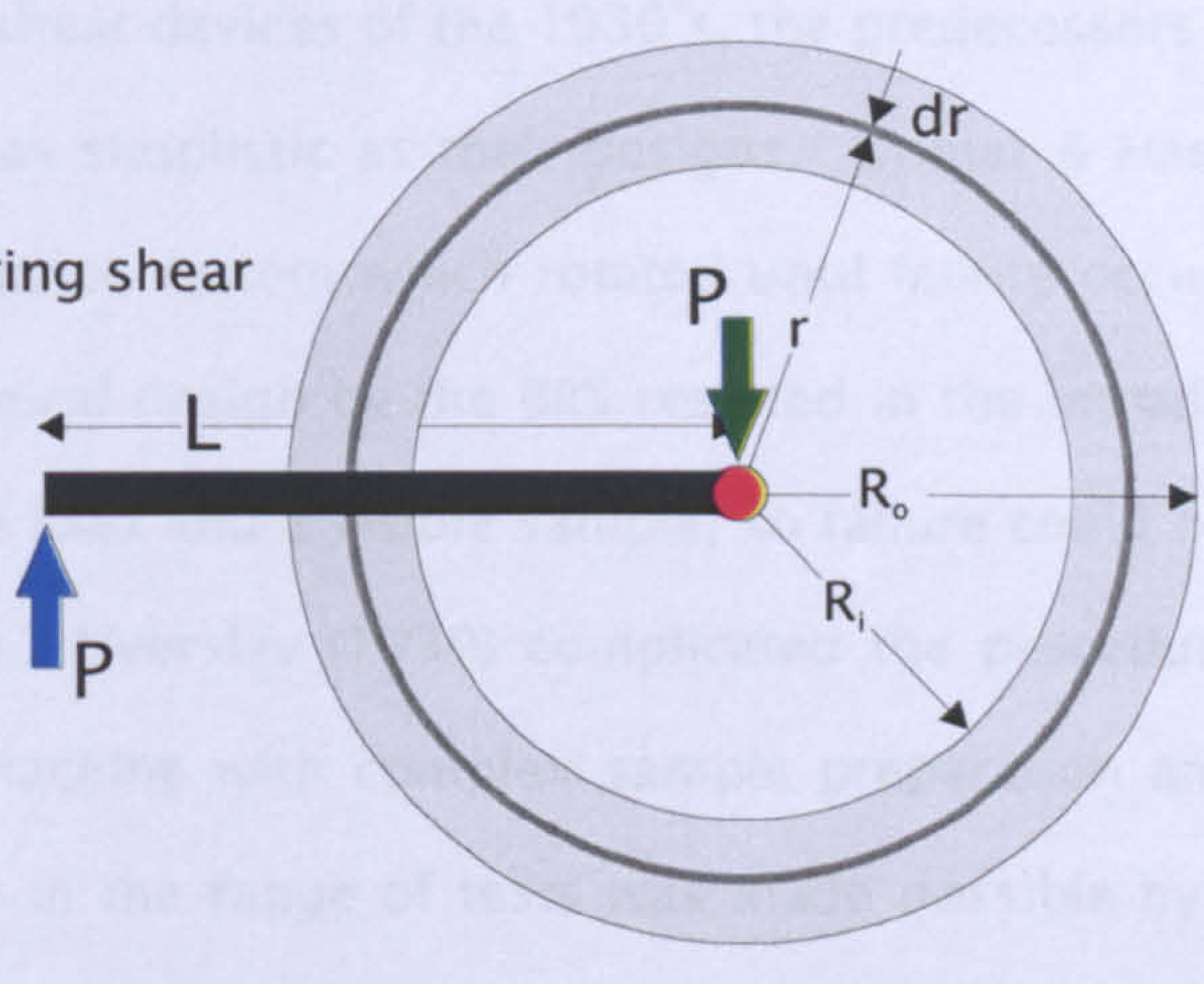


Figure 3.12 Many workers attempt to find a unique relationship with Ip, CF, or other factors, and obtain a plot of this sort.



The worst case for friction on the centre pin is when all the load P is carried on one load ring. The centre pin must carry the same load, so the torque due to friction is $\Delta T = P\mu R_p$. The Torque, T, due to load on the cross arm is $T = PL$, so the error is $\frac{P\mu R_p}{PL}$. With R_p=5mm, L=75mm, and the coefficient of friction for a greased pin very much less than 0.3, the error cannot be more than a percent or two.

Also, $T = \int_{R_i}^{R_o} 2\pi r^2 dr\tau = \frac{2}{3}\pi(R_o^3 - R_i^3)\tau$ for a uniform distribution of shear stress.

Figure 3.13 Centre pin friction error.

4 Experimental Programme

Extensive research was carried out to establish the effect of strain rate using the small ring shear. As part of the research programme method of application of normal stress was investigated as well as the innate variability of a particular sample. The methodology used for this research is described in this chapter thus forming a basis for subsequent chapters.

4.1 Development of Ring Shear testing procedure

Procedures adapt as the machines get more complex and the ring shear is no different. The torsional shear devices of the 1930's, the predecessors to the ring shear, used procedures as simplistic as their designs. Gruner & Haefeli (1934) used a basic counter balance system which rotated until failure occurred within the soil. Increased technical design by the BRS resulted in the introduction of a weighbridge to apply the load and a visible sample, so failure could be observed (Cooling & Smith, 1935). Hvorslev (1939) complicated the procedure with the introduction of a new machine with complex sample preparation and machine calibration. An increase in the range of tests was made possible by Hvorslev's introduction of fixed, free and restrained rings, but, he noted the effects of soil extrusion via the gap between the confining rings. The increased complexity prompted Hvorslev (1939) to note the requirement of a simpler test procedure.

The seventies saw the introduction of four new ring shears each complete with its own procedure - three for clay and one for rocks (Mandl et al 1977). Introduced by Sembenelli & Ramirez (1969), the Harvard Apparatus and procedure were eventually detailed by La Gatta (1970, 1971). La Gatta notes the complex assembly procedure but notes its advantages of the ability to use undisturbed and remoulded specimens, pre-cut surfaces ensure that failure was at mid height within the specimen and ten stages of consolidation. Bishop et al (1971) designed the NGI/IC ring shear, a complex apparatus with over a hundred screws. The clearing distance between the upper and lower rings has to be maintained and a gap must exist when the ring shear is run initially at low speeds in order for the shear zone to form. As an open gap this causes increased displacement of soil. After the formation of the shear zone, the gap is closed and the test rate increased, thus resulting in a change in normal stresses, reducing the accuracy of the results. However, when the gap is open, more soil is displaced, so, the test is usually run with the gap open so normal stresses are not disturbed part way through the test. Everything possible is done to minimise soil loss during the test. Taylor (1998) gives an excellent summary of this machine, including a detailed technical specification of this apparatus. Taylor also clearly indicates the problems that occur as a consequence of the "gap control" difficulties at fast rates of shear. Taylor himself did not see any significant problem, but, stated that it was occasionally necessary to re-adjust the gap that caused minor short-term fluctuations in results. Metal to metal friction has to be minimised. Procedural steps involve apparatus assembly, stabilization under constant load, opening gap between confining rings, setting the strain rate to allow for excess pore water to dissipate and rotation to failure, soil extrusion being the major disadvantage. Kingston's contribution, the Bromhead ring shear, although relatively inexpensive, this simplistic design is an effective test. Apparatus preparation is minimal, with just "two finger tight" nuts (Bromhead, 1979).

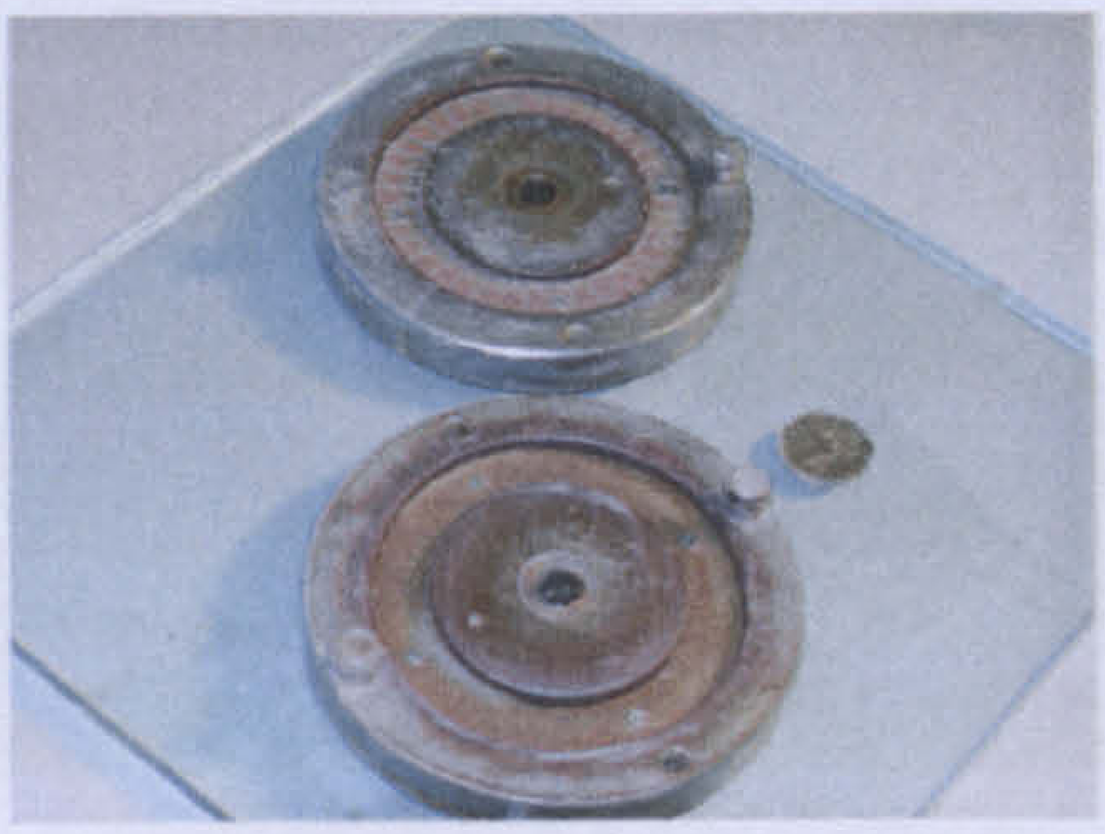
Harris & Watson (1997) introduced the Kingston procedure; less cumbersome with limited settlement and reductions in both test duration and wall friction. It showed advantages over the old BS methods. To validate their conclusions

Harris & Watson compared their values on the ring shear with back analysis, resulting in identical values of residual strength. Overall, the accuracy is to a quarter of a degree for the effective residual friction angle ϕ. These conclusions were later confirmed by Bromhead et al (1999), citing no difference in values of residual shear strength achieved on the ring shear to back analysis. Hvorslev (1939) stated the need for a simple, but effective test, which is exactly what the Bromhead Ring Shear is.

4.2 *Ring shear test - manual staged loading*

Test carried out to Kingston University method as per Harris & Watson (1997) technical note and BS 1337: Part 7 test 6 which is shown below in stages of the procedure:

- Stage 1



Make sure lower platen is clean. (On the left is the two types of internal plate used)

- Stage 2

Take clay and kneed it well into the lower ring

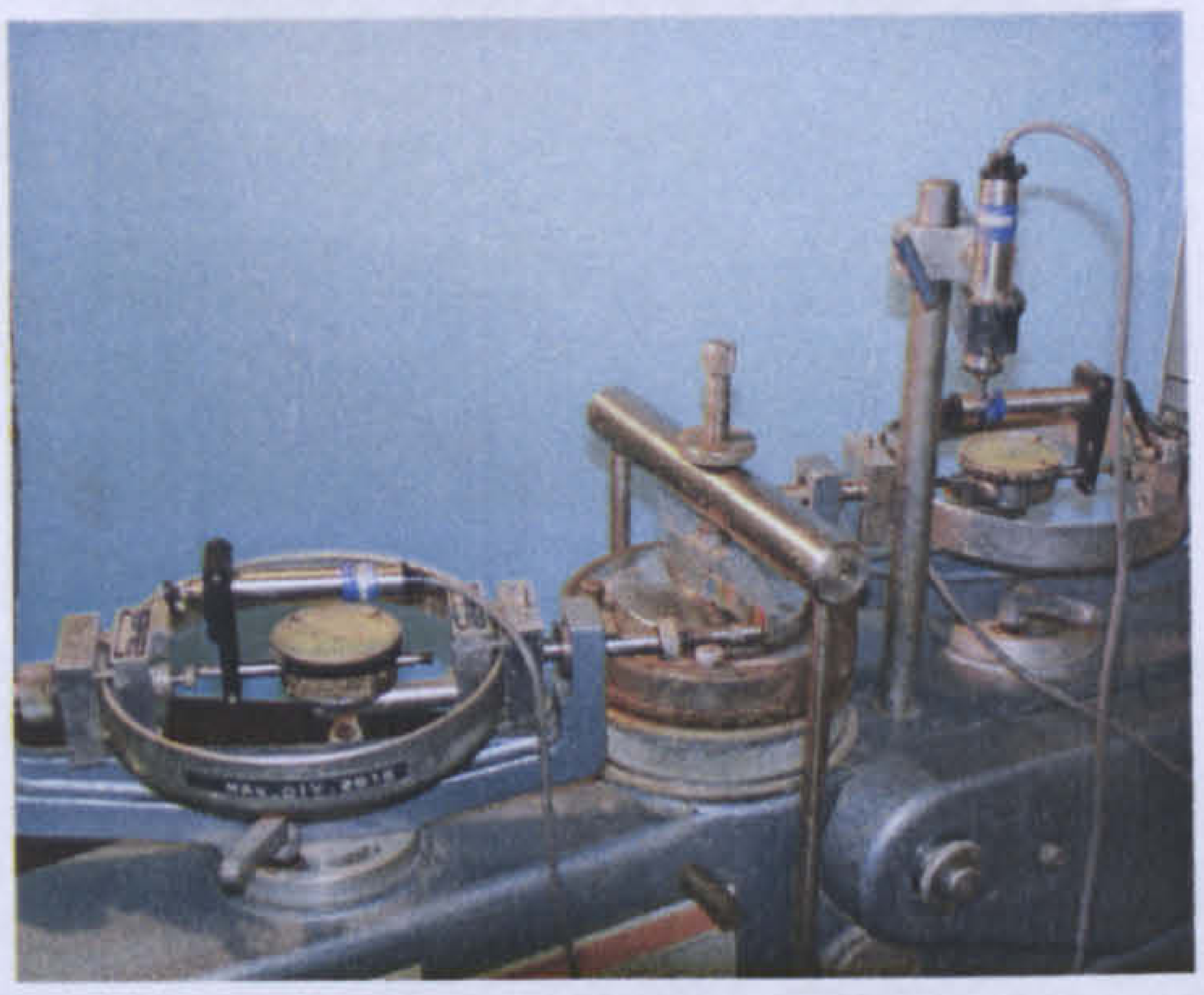
➤ Stage 3

Trim excess clay off with a palette knife

➤ Stage 4

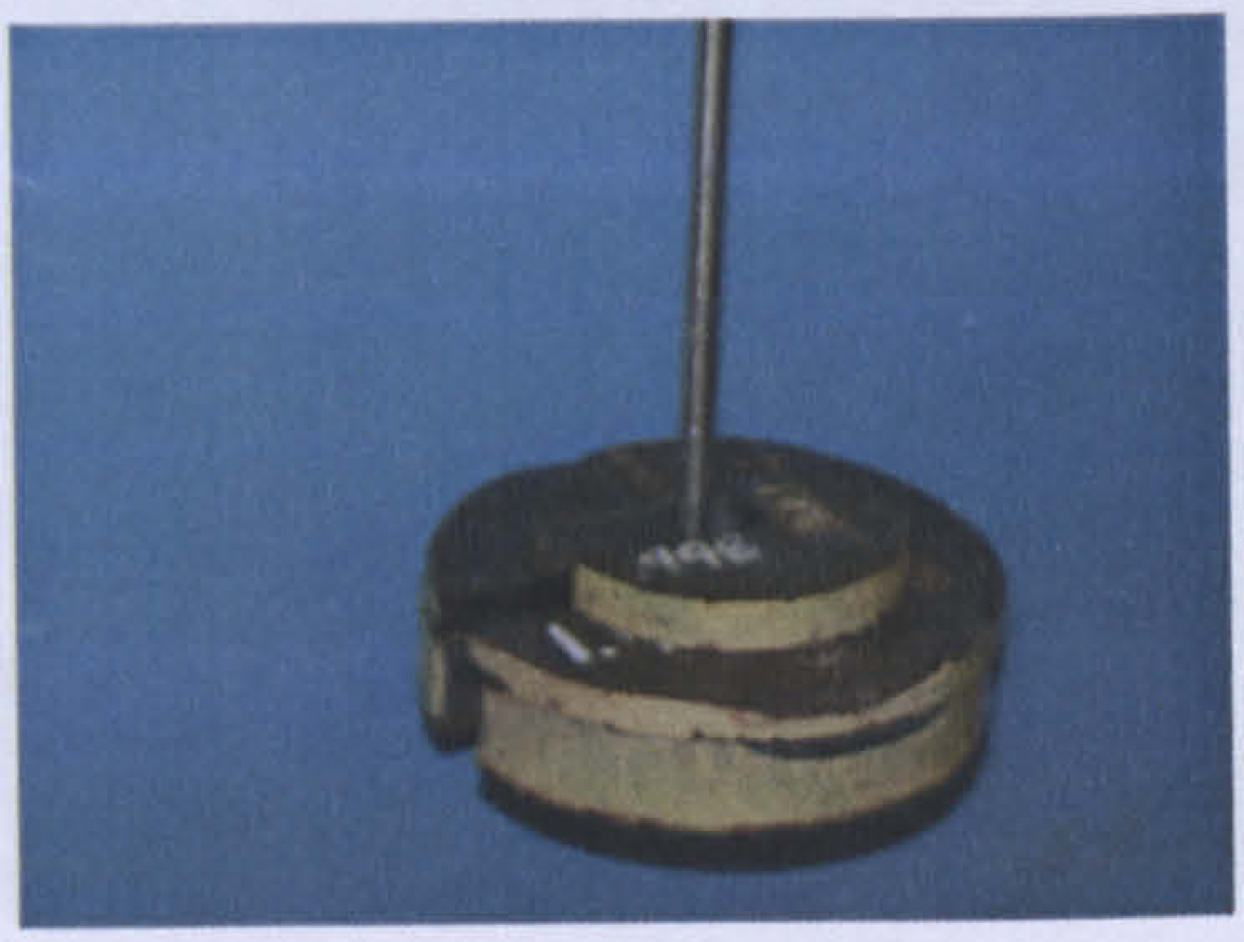
Place lower ring into the empty water bath then place upper ring on top of the lower ring.

➤ Stage 5



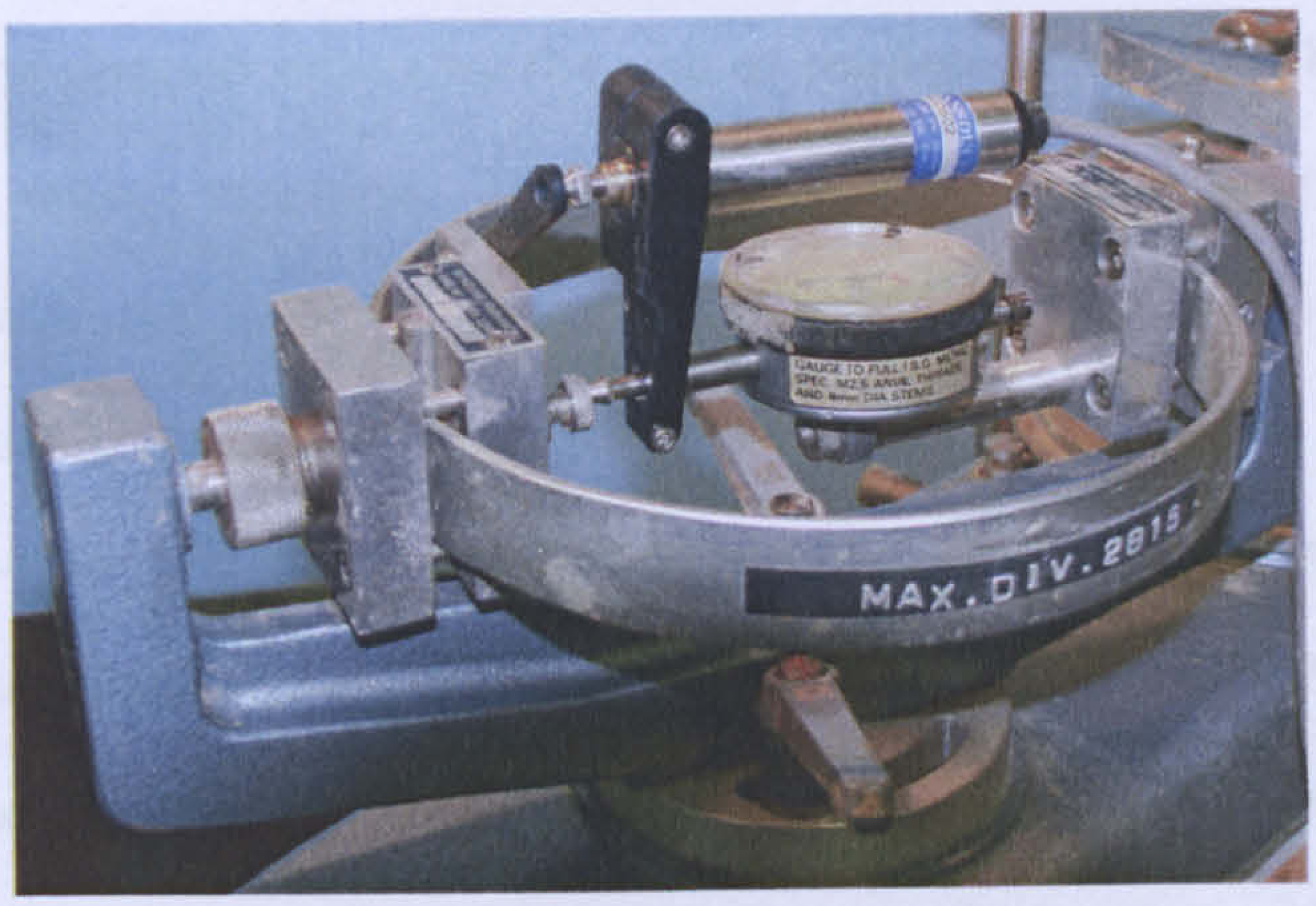
Turn until arm is normal to proving rings then lift weight hanger into position.

➤ Stage 6



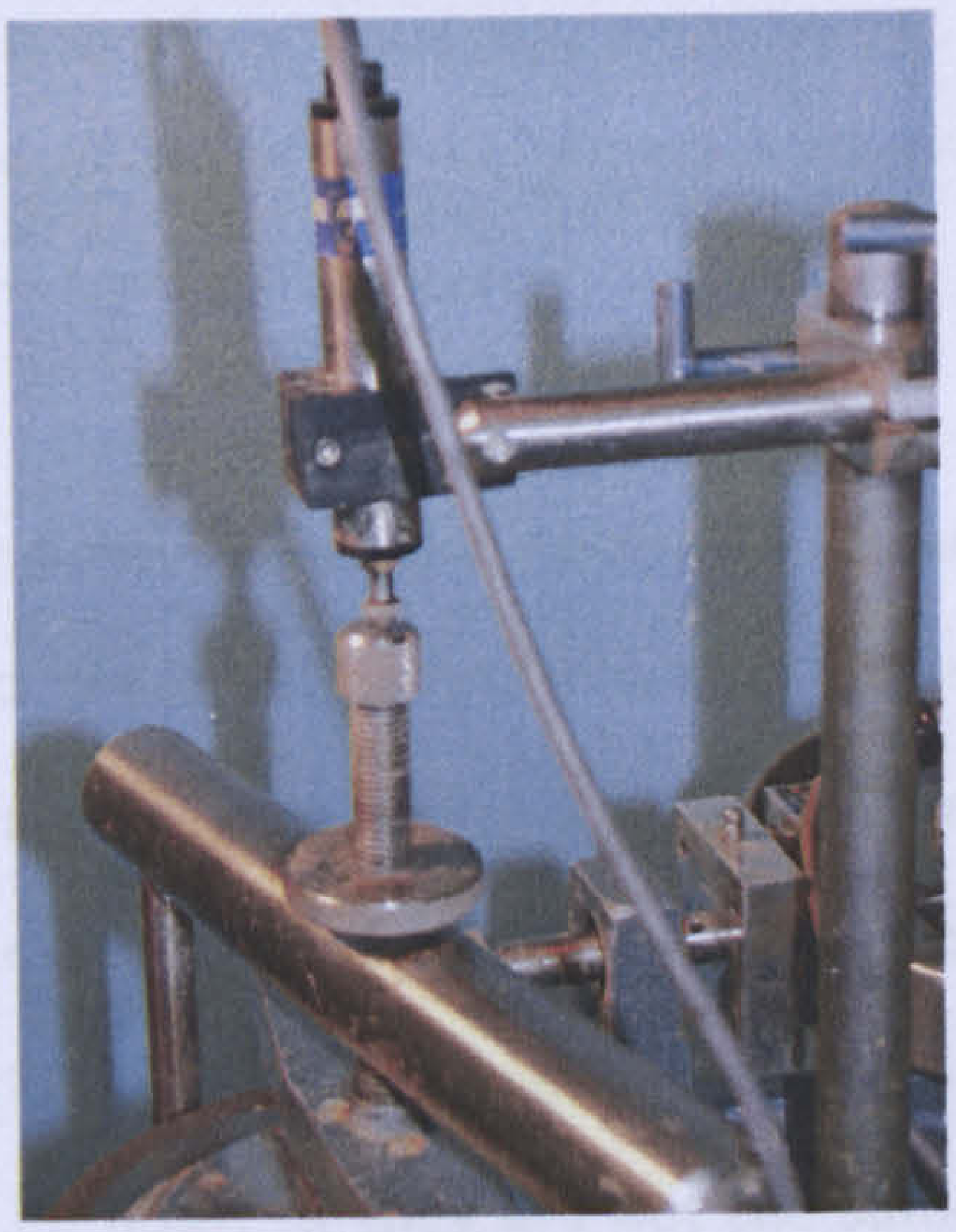
Place the first load on weight hanger and check that the hanger is as high up as possible.

➤ Stage 7



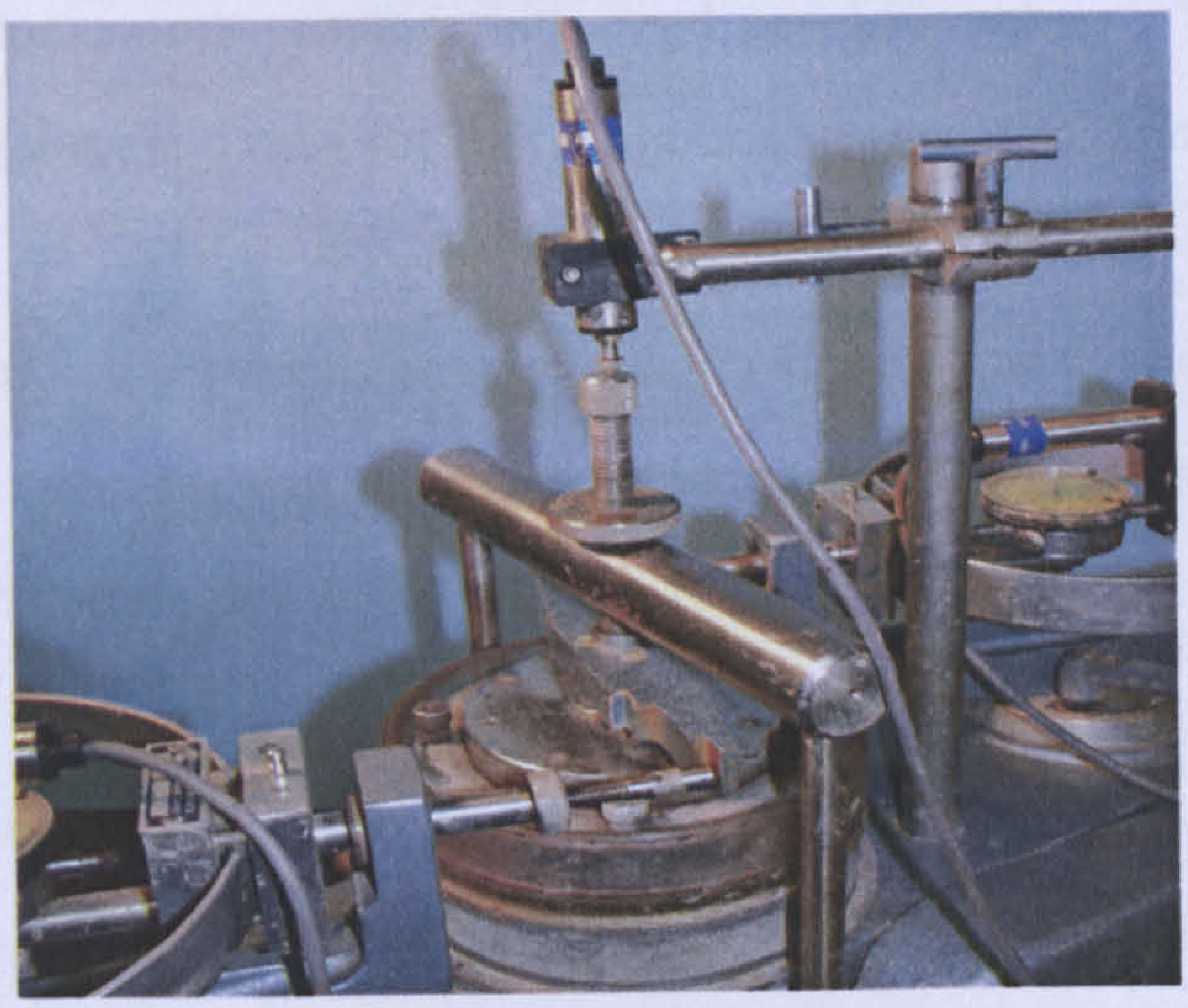
Zero proving rings once they are touching the torque arm.

➤ Stage 8



Then set up vertical displacement gauge and zero it.

➤ Stage 9



Fill water bath. Then check all dial gauges read zero.

Load can be applied either by the use of manual weights or by using compressed air on the newest machine. The older machines have weight hangers to the right of the apparatus which give a 10:1 lever arm on the loading beam. The compressed air regulated using a manual valve, is applied via a cylinder at the base of the apparatus which pulls the loading beam down onto the sample. The pressure applied has an equivalent loading of 234 KN/m2 per 1 bar pressure which is equivalent of 9.3kg. This is applied up to a maximum of 1170KN/m2.

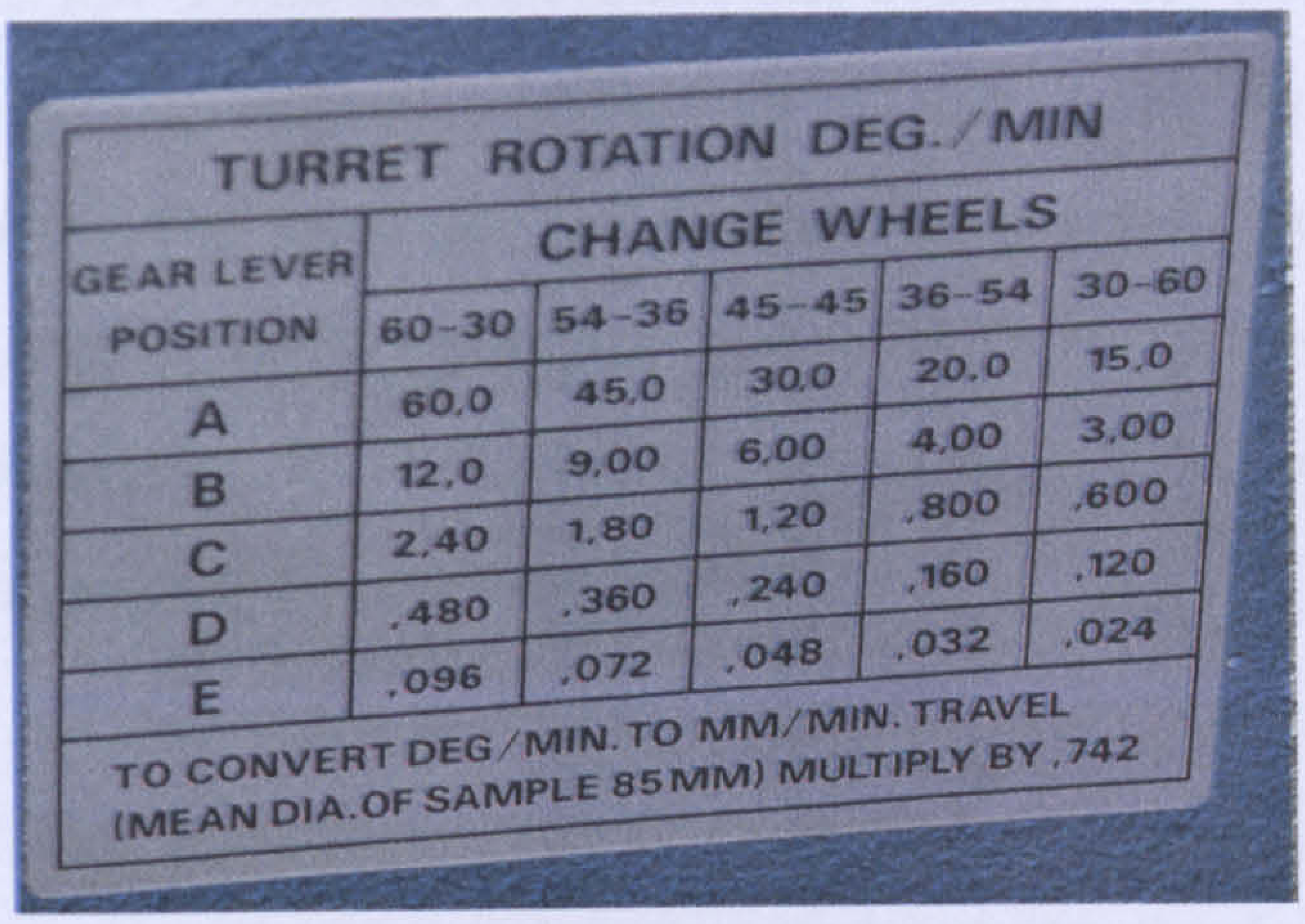
From the results, a graph of normal stress v residual shear stress is plotted to derive the shear strength parameters.

Start test and note time. When the ring shear is set at slower speeds (C or E), it is left overnight to settle. If at faster speed (A), leave for a minimum of three hours to settle or until readings stay the same. Then, take a reading of all three gauges. Leave for half an hour and then take another reading. If the proving ring gauges read the same figure again, more load is added to the weight hanger and leave for half an hour. If they are not the same, leave it for another half an hour and repeat until proving rings read constant values. Then, take a reading of all three gauges. Leave for half an hour and then take another reading.

If the proving ring gauges read the same figure again, add load to the weight hanger and leave for half an hour. Repeat a total of five load stages at least, so that a graph can be plotted to find the residual strength.

After the last set of readings for the last load stage has been taken, unload weight hanger until only the initial load remains. This needs to be left for a further two hours or until the readings stabilize.

This test is carried out at variable rates as shown the photos below:

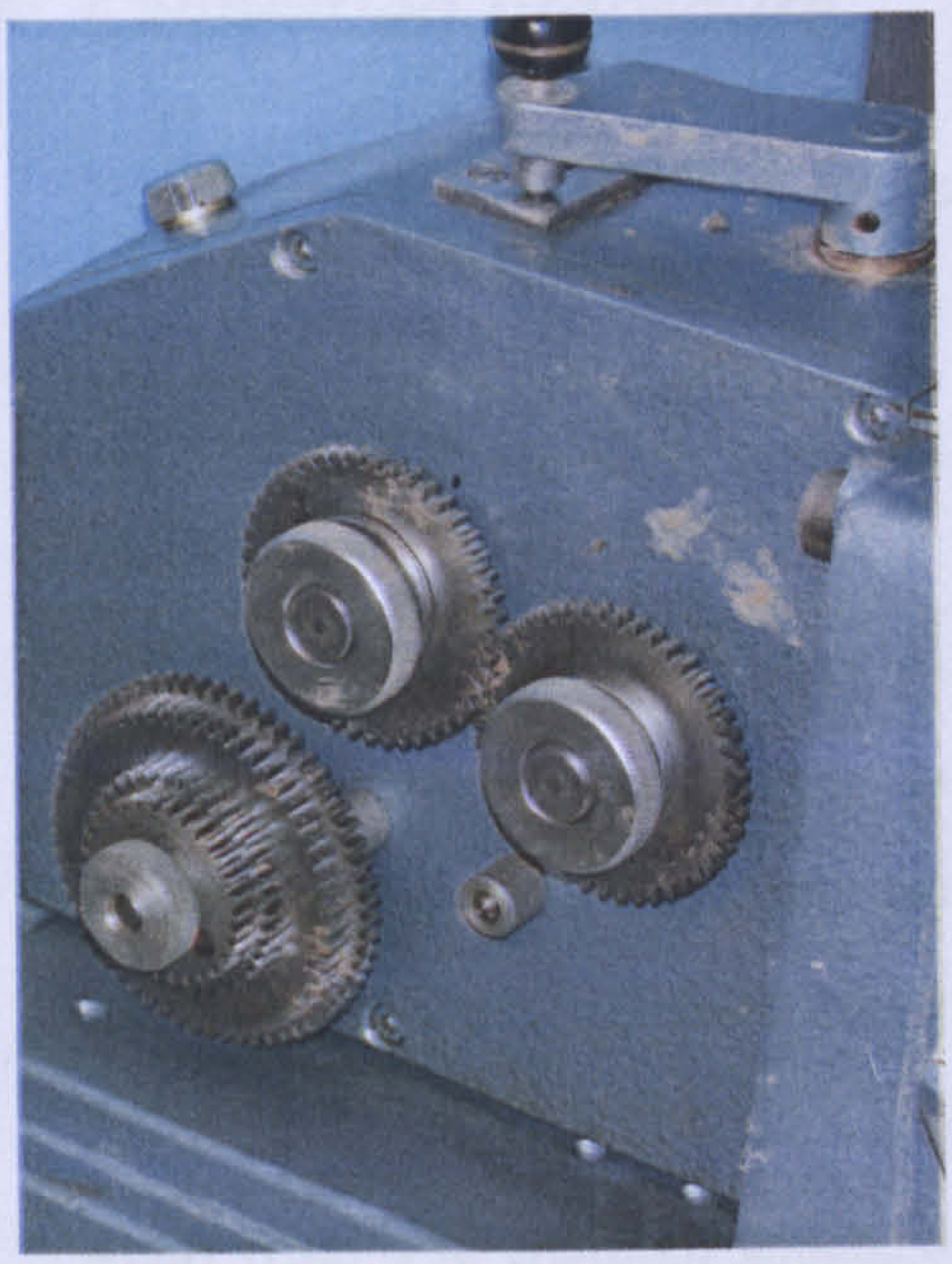


TURRET ROTATION DEG./MIN

GEAR LEVER POSITION	CHANGE WHEELS 60-30	54-36	45-45	36-54	30-60
A	60.0	45.0	30.0	20.0	15.0
B	12.0	9.00	6.00	4.00	3.00
C	2.40	1.80	1.20	.800	.600
D	.480	.360	.240	.160	.120
E	.096	.072	.048	.032	.024

TO CONVERT DEG/MIN. TO MM/MIN. TRAVEL
(MEAN DIA. OF SAMPLE 85MM) MULTIPLY BY .742

Values in the photo above are in degrees per minute, and are converted to millimetres per minute by multiplying by 0.742. These values relate to the gear lever which has five different positions, A to E and combined with a five different arrangements of cog wheels facilitate a 25 speed gear box.



The gear cog wheels are in five different pairs and slot together as shown on the photo to the left with a gear handle above which dictates the range of speeds.

Post Test Soil Analysis

Pre test, the soil is weighed before the test begins. Post test, the soil is taken out and weighed to determine percentage soil loss during the test. A sample of the post test soil is taken out and weighed to determine percentage soil loss during the test. A sample of the post test soil is taken and the moisture content determined by using BS 1377: Part 2: 1999: 3.2. Prior to one test the sample moisture content has been pre determined.

Data Analysis

The dial gauge readings are then inputted into an Excel spreadsheet where residual shear strength displacement rate and, if required, rate of soil loss, can be determined. The coefficients are derived via plots of normal stress v. shear stress.

4.3 Calibration procedure

To improve the accuracy of results, the tests are data logged. In order for the data loggers to produce sensible data, the linear transducers all have to be calibrated first.

The data logger has an associated software package which allows for a range to be set for each channel. One channel is linked to one linear transducer. With each transducer they were set a range of 5000 using the software. Then, each of the linear transducers were put through the software's calibration procedure. For this, the 4.0 feeler gauge was inserted prior to the channel being zeroed. Apart from one channel, which, due to the physical set up was zeroed at 2.8. The feeler gauge was then removed, to allow the software package to automatically find the range for the particular channel corresponding to the particular linear transducer. Once the range has been found the computer informs that the calibration is complete but, sometimes this procedure needs to be run more than once.

Then once all channels and linear transducers have been calibrated, feeler gauges are used to investigate for a calibration factor. Feeler gauges of 0.25, 0.5, 0.75, 1.0, 2.0, 2.8 and 4.0 were used to derive the corresponding factor for each transducer.

As a check, the dial gauges were also displaced, using the same technique.

4.4 Data logger

The linear transducers are set up and plugged into the rear of the data logger using one 5 dinn plug.

The data logger is attached via a USB cable to a computer with a minimum spec of Pentium 2 and Windows XP. Software is installed on the computer to allow the data to be downloaded from the data logger to the computer memory.

The software allows for the channels to be read simultaneously and recorded at set intervals through a process of setting up tests. These tests can be set up by choosing the channels to be recorded. Channels can either be zeroed or not zeroed at the beginning of the test being run. The time interval for recorded readings can also be set for all the channels to run for the entire duration of the test.

Once the test has been set up and channels have been calibrated, the test can be run and the procedure of calibration and test setup need not be done every time; providing the physical setup, the number of channels and the time interval stays the same. All that needs to be done is a test run which starts the data logger after thirty seconds and continues to data log the readings at set intervals until the test is stopped. During the test, the data logged can be viewed as a graph per channel in run mode. This allows the data logged to be monitored.

Once the test has finished, the data logger is stopped by instructing the programme to "stop test". The test can then be downloaded by one of two methods either by, downloading the test as a whole which downloads the memories of the selected channels loggers as part of the test setup or individual channels can be downloaded individually. The files are then be saved as a comma separable variable file by putting the name of the file then a .csv selecting all files.

e.g. Name 1.csv

The .csv file can then be transferred into an excel file. This allows greater manipulation of the data.

4.5 Ring Shear test - Ramped pressure loading -Step-less loading

The apparatus is set up in exactly the same way as 4.2. The sample is prepared and placed into the lower ring and complete with the upper ring is placed in the water bath. The newest apparatus is the only one capable of running a stage less test as it is the only machine which is loaded using compressed air instead of weights.

For this test, instead of a manual air regulator valve, an automatic pressure controller is used. When programmed this automatically can either ramp or stage load the sample.

To set up the automatic pressure controller (APC) the air line from the compressor has to be attached to the air inlet supply valve. This has to be at least 200KPa greater than target pressure but a maximum of 1400 kPa. The air outlet pipe has to be attached to the air cylinder on the ring shear apparatus. The pressure transducer needs to be attached to the automatic pressure controller. The computer is connected by an RS232 in and out socket to the APC.

When the automatic pressure controller is switched on to configure and calibrate the APC, a pass code is required. This protects against misuse or unauthorized operations.

To configure and calibrate the APC, first the pressure channel has to be selected. This does require pass code access. The pressure transducer has to be connected via the DIN socket on the back of the APC. The APC needs to warm up for at least thirty minutes before calibration can commence.

The first stage in the calibration is to select the full scale pressure value. This is the maximum pressure at which the APC will be operating. The pressure transducer has to be calibrated between 0 and the FSD (full scale deflection).

There is, at this stage, an option to clear calibration. Then, the calibration can begin. Then, zero pressure is applied, (air pressure valve switched off) when the display is stable, then, the zero option is selected. The transducer is then pressurized to the FSD value and when the display is stable the span option is then selected to complete the calibration procedure.

Calibration values are indicated against the zero and span options. These values are noted for future. In the event of the calibration being lost, then the zero and span values can be entered via the Engineer option. This means that the calibration procedure does not need to be carried out again.

The Engineer function does require pass code entry which is different to the pass code used for calibration and configuration. This function is used by Installation Engineers to set up the instrument. The "init" option is selected to set default values. This option does not change or clear the calibration values. The system units can also be selected as part of this function. There is a choice between metric and English units. Metric is the default. In the event of the calibration being lost the zero and span values can also be inputted using this function.

When the calibration and configuration has been completed the control functions can be selected. The "Run" function allows the target pressure to be set. The "Target Pressure" can be anything up to a maximum of 1000KPa. The Stop function will terminate the pressure control. In run mode, the pressure value is shown as white characters on a black background, when the stop button is pressed the display pressure reverts to black characters on a normal white background. Note, that in the stop mode, the pressure can fluctuate. As part of the control functions, there is an option to increase one pressure linearly or as it is named "RAMP". To Ramp the pressure, a rate of increase has to be entered and a target value selected. This has to be at least 200KPa less than the FSD. The direction of Ramp is indicated by an animated middle bar on the display. On reaching the target pressure, this is maintained until the Stop option is selected.

The way the pressure is controlled automatically means that the test can operate independently, providing linear transducers are attached to the ring shear to record the displacements onto a data logger. This reduces the amount of time that the ring shear requires supervision and allows for the test to be left to run. This is very useful since, the ring shear is a long test requiring several hours of shearing to achieve a plot of the Mohr Coulomb failure envelope to derive the coefficient of friction.

4.6 Classification Index Tests

Moisture content tested in accordance with BS 1377: Part 2: 1990: 3.2. The only aspect which may vary slightly is the weight of the sample. Plastic index tests conducted in accordance with BS 1377: Part 2: 1990: 5.3. Liquid Limit tests were carried out using BS 1377 : Part 2 : 1990 : 4.3 on all but one soil sample from St Catherine's Point, IOW was tested using BS 1377 : Part 2 : 1990 : 4.5. This was due to a shortage of the soil sample remaining to be tested.

5 Investigations into Strain Rate Effects in Ring Shear Testing

Most landslides in the UK move at slow to moderate rates according to the Cruden & Varnes Scale (Chapter 3; Cruden & Varnes, 1996) which can be replicated using very slow strain rates in the laboratory using a ring shear apparatus. Principally these investigations into strain rate are concerned with the effect on the achieved value of residual shear strength. At these slow rates it is widely felt that there are negligible rate effects on residual shear strength (Fearon et al., 2004), and as a result most ring shear test research is done at moderate to fast strain rates which apply to fast moving landslides.

Since Skempton's Rankine Lecture in 1964 these alleged rate effects have been one of the key areas which have been under investigation; what causes a landslide to be either slow or fast. Rate effects are used to explain the distance a landslide travels before stopping. These are usually classified as negative effects for fast moving landslides and positive for slow.

Previous research, in the laboratory has seen the effect of soil extrusion is overlooked and even dismissed (Lemos, 1999). All ring shears to some degree extrude soil and the faster the strain rate the greater the quantity of soil extruded. Some work has been done on the prevention of soil loss; Parathiras (1994) designed confining rings for the NGI/IC ring shear which improves the situation but does not eliminate it. Therefore as it is not possible to prevent, the only option is compare more with less soil loss to quantify the effects.

Fearon et al. (2004) from their recent investigations found that negative effects only occurred when there was no water in the water bath, in other words when the test was effectively undrained. Interestingly this would indicate that negative effects were due to the test being undrained which occurs at fast rates due to a defect of the apparatus design.

Accepting the limitations of the Bromhead Ring Shear apparatus this chapter describes the investigations into this area along with strain rate, discontinuous strain rate, soil extrusion, machine set up and the movement of water.

5.1 Background

Investigations into the existence of rate effects can be attributed to the causes behind slow and fast landslides. There have been some suggestions that movement of a landslide is constant over time (Picarelli et al, 1995 and Nieuwehuis, 1991).This would infer that the longer the state of imbalance occurs the more the landslide enlarges itself and therefore the greater the run out. Mathematically this can be proved inaccurate (Angeli etal, 1996 and Van Beek etal, 1996) hence the assumption, that there must be some sort of rate effect; thus providing an explanation for the discrepancies between theoretical and actual run-out distances. However it could be proved that even during a particularly unsettled period, movement rate is neither constant nor continuous.

Previous research into increases in strain rate and its effects are described in Chapter 3. This chapter details a series of new investigations into rate effects which were performed on two Bromhead ring shear machines, with two different designs of internal plate to observe whether there were any significant differences in the results using identical clay samples. The objective was to determine whether increasing the strain rate produced an increase or decrease in residual shear strength. On the Bromhead ring shear apparatus there is no gap between the upper and lower platens unlike the NGI/IC ring shear apparatus. For the NGI/IC its actual design means that during testing the gaps have to be left open or the normal stress has to be changed when the gap is opened after consolidation so to minimise metal to metal friction. Obviously this results in a large amount of extrusion at fast strain rates. Theoretically on the Bromhead ring shear there should not be large amounts of soil extrusion even at fast strain rates as there is only a minimal gap between the upper and lower platens. Different clays with varied plasticity indexes (Ip) were used to determine:-

- the effect, if any, Ip had on the variation of strain rate effects on residual strength
- post test moisture content at variable strain rates
- the effect of soil extrusion due to the rotation of the top platen displacing the soil
- the changing of the cross sectional area

Determining the post test moisture content ascertained whether the increased strain rate of the test resulted in an increase in the percentage of water compared to the actual clay content. The use of these two Bromhead ring shears also gave the opportunity of investigating the effects of the two different internal plates.

In the Bromhead ring shear apparatus a shear surface is induced beneath the top platen therefore equally under investigation was to ascertain whether the texture of this surface affects the residual strength. Variation of strain rate on the three clays mid-test was also investigated. For this, the strain rate was increased and decreased mid test to assess the effect on the value of residual strength. This was investigated using three different types of clay.

Previous research, into rate effects on residual shear strength which has been undertaken, have been investigated using the NGI/IC ring shear apparatus by among others interallia Lupini, 1980; Lemos, 1986; Tika, 1989 and Taylor, 1998. The main focus of this research has been to investigate whether the same effects are observed using the Bromhead ring shear. Palmer (1999) investigated the effects of ploughing speed and showed that the cutting force would increase with the speed of cutting which would cause the soil to deform, resulting in a reduction in pore pressures, which would increase effective stress and therefore increase resistance to shear. In the ring shear the increased strain rate would increase the force applied to the clay, resulting in more soil being extruded and compressed. Forcing the water out as well as the soil would lead to a reduction in pore pressures which reduces effective stress and increases the resistance to shear, hence Lemos (1986) positive effects.

As Harris and Watson (1997) postulated, the correct rate of strain ensures the pore pressures are fully dissipated during the test and complete consolidation occurs. Reducing the strain rate increases the measured torque result from the generation of increased water pressures. For high rates of strain the inferred strength would increase due to the disruption of the soil fabric. This chapter sets out to detail the results and findings of these new investigations.

5.2 Samples used in investigations

The samples for the investigations described in this Chapter came from three different locations as explained in detail in Chapter 1.

- St. Catherine's Point, Isle of Wight, which is blue grey gault clay with a high plasticity index. The specific location is Watershoot Bay which is just east of Rocken End, the most southerly point on the Isle of Wight just above high tide at the toe of the mudslide.
- Dromore, Northern Ireland the sample is orange brown stiff clay with a high plasticity index from a failure on the side of the B2 (NI minor road) just west of the A1(NI major road).
- Warden Point, Isle of Sheppey, the sample is light tan brown stiff clay with a very high plasticity index. It was taken from the toe of the landslide just above the high tide line.

5.3 Strain rate

Investigations into the effects of rate of strain were performed on two Bromhead ring shears using the procedure described within chapter four. Moderate rates of strain were used to ascertain the rate dependency of the shear strength parameter Φ'_r and shear stress within the capabilities of the apparatus. Overall per sample, one test per machine was performed using one of the fifteen rates of strain used which ranged between 0.018 mm/min and 44.52 mm/min. This was the case with all but the sample from Warden Point, Isle of Sheppey where a smaller range of tests were conducted. Previous research cited within chapter three describes a threshold of rate dependency of approximately 10 mm/min as noted by Bromhead (pers comm.).

Figure 5.1 indicates that overall as strain rate increases so does Φ'_r thus implying a positive effect. However more detailed analysis reveals that the exact correlations are more variable.

The trends for the two samples from St. Catherine's Point, highly plastic clay, predictably follow an almost identical trend with the exception of the strain rate of 0.018 mm/min where the second sample (shown as squares in Figure 5.1) has an unusually high value of 13° for this type of clay compared to the others within the low speed strain rates and therefore obscures the trend. At 8° the value for the first sample (shown as diamonds in Figure 5.1) is within the expected range. For the next three rates an increase in Φ'_r is observed, as shear strength parameter gains three degrees. The last of the very slow strain rates of less than 0.1 mm/min, results in a decrease of five degrees compared to the previous strain rate. Mid range speed tests span the 1 mm/min rate and for the St Catherine's Point samples resulted in three of the five strain rates having a Φ'_r of 12°. Although at mid range of this set of strain rates, 0.8904 produced an increase in Φ'_r to around 14°, conversely the outcome of the highest rate in this set 1.78 mm/min was a decrease to 9°. Analysis of the faster rates for the St. Catherine's Point samples displayed a trend of a positive rate effect, in other words Φ'_r increased with strain rate increase, excluding the second fastest test, which resulted in a lower value than the fourth fastest at 12° for 33.39 mm/min.

In general the trend for the St. Catherine's Point samples can be taken as increasing Φ'_r with increasing strain rate for both samples. Although values of Φ'_r span a 7° range for the first sample and an 10° range for the second sample from St. Catherine's Point for the entire range of strain rates. Specifically the maximum range is for the lower speeds using the second sample and they have a variation of 10° for the low speed strain rates .The range for the mid and high speeds was 3° and 4° for the first sample and 6° and 7° for the second sample respectively. The first sample showed less divergence with the variation in strain rate although the second had a more consistent range size over all three ranges of strain rates.

A smaller range of strain rates were used for the sample from Warden Point, (shown as circles in Figure 5.1) which is classed as very high plasticity clay. In general, the trend demonstrated for this sample was an increase in the value of Φ'_r as the strain rate escalated as shown in Figure 5.1. Conversely slow rates of strain rate of less than 0.1 mm/min are shown to exhibit a decrease in Φ'_r of 3.5°when accompanied by an increase in strain rate of 0.02 mm/min. From the low point at 0.036 mm/min which is ϕ'_r equal to 11.6°, the trend then reverts to increasing Φ'_r with rising strain rate as shown in Figure 5.1. In detail this gives a ϕ'_r of 13.2° for 0.8904 mm/min; 14.8° for 22.26 mm/min and 17.6° for 44.52 mm/min. Variance within the results for this sample was small with a range of 6° Φ over a range of 0.018 to 44.52 mm/min, the maximum range of strain rates for the Bromhead Ring Shear. The Warden Point sample therefore resulted in a positive effect for rate dependence.

Dromore, Northern Ireland, a high plasticity clay, was noted to exhibit a fairly linear trend as shown in Figure 5.1 (triangles) over the majority of strain rates which resulted in values of Φ'_r between 16° and 18°. The anomalies occur at the strain rates of 0.5936 mm/min which was 14.8°, and 0.4452 mm/min at 15.77° as the exceptions to the range. Below 0.1 mm/min, Φ'_r fluctuated between 16.85° and 18.01°, initially a decrease of 1°, then across the three mid range speeds between 0.027 and 0.053 mm/min ϕ'_r had a range of 0.31° with a marginal increase at the mid strain rate of 0.17° at 0.036 mm/min. An increase of almost 1° followed at the strain rate of 0.071 mm/min. Strain rates between 0.4452 and 1.78 mm/min produced the two anomalies of the trend and the corollary being that this is a slight depression in the otherwise almost linear trend. Between 0.8904 mm/min and 1.78 mm/min Φ'_r continued to increase with strain rate by 0.18°. All of the rates above 10 mm/min result in Φ'_r of approximately 17°, conversely the one rate below 10 mm/min in the higher range is 7.42 mm/min which produces an 18.01° Φ'_r. Overall the variance is insignificant at less than 1° at these rates. Mid range speeds consequently had the highest variance at just less than three degrees.

Investigating the variation in shear stress for a particular normal stress over the entire range of strain rates demonstrated the correlations between shear stress and strain rate. For the same four samples as previously described, most shear stress values fell within a 99% confidence boundary at 198 kPa normal stress. The divergence within the sample was dependent on the individual specimen as all four resulted in slightly different correlations as shown in Figure 5.2. St. Catherine's Point sample one had almost half the variance at 9.6 kPa than sample two at 15.9 kPa which are two samples from virtually the exact same location within half a metre of each other. Dromore displayed a smaller variance than St. Catherine's Point's second sample at 14.2 kPa. Warden Point, Isle of Sheppey, produced the largest variance at 19.6 kPa which was the smallest range of tests producing the largest variation. Clay being a non homogenous material could be the cause for this variation.

Statistical analysis via all "R" squared test on the linearity of the envelope as shown in Figure 5.3, indicated that the majority of tests had resulted in either perfectly linear or almost linear stress envelopes. Of the points below 9.98, the majority were for Dromore but there was no fixed trend with strain rate. Conversely "R" squared analysis at specific normal stresses as shown in Figure 5.4 and 5.5 shows a definite trend of decreasing linear correlation between shear stress and strain rate, indicating that with higher normal stresses there was a greater percentage of divergence in shear stress across the range of strain rates. Except for Dromore, which initially exhibited a 0.2 increase in "R" squared for 50 kPa then stabilized until 200 kPa and decreased by 0.2 for 250 kPa, St. Catherine's and Sheppey all exhibit the general trend.

Overall fifteen tests each at a different strain rate were carried out on each machine for samples from both St Catherine's Point and Dromore. The sample from Warden Point was used for five tests in all each of these tests were carried out at a different strain rate on each machine. St Catherine's results indicate a slight positive trend but limited rate dependence due to the variation within the trend; Warden Point results indicated a positive trend with rate dependence but Dromore although there was a slight negative trend within the ranges was virtually rate independent.

5.4 Discontinuous Strain Rate

Procedures for ring shear tests have traditionally involved starting at a low speed then increasing the rate once a shear surface has been formed (Fearon, 2004) As part of this research the question was raised as to whether this procedure resulted in a different value of residual strength. To determine this, the procedure described in Chapter 4, was adapted to enable a rate change once a shear surface had been formed, then allowed time to settle before the original procedure was resumed. A reduced range of tests was used, with only the following utilized, 0.018, 0.036, 0.8904, 22.26 and 44.52 mm/min. Validity was ensured by using three different clay samples on two different machines. In total, twelve tests where the strain rate was changed mid-test post the consolidation stage per sample per machine; plus the five control tests where the strain rate remained at a constant rate throughout the test.

Figure 5.6 demonstrates with the use of St. Catherine's Point clay, which by increasing the rate from below 0.1 mm/min to above 10 mm/min resulted in a reduction in residual strength. Mid range strain rates have been either decreased from rates above 10 mm/min or increased from below 1 mm/min, resulted in a larger value of residual strength. Starting at strain rates above 10 mm/min and reducing them to below 0.1 mm/min exhibited a larger value of residual strength, residual strength being represented by the effective drained residual shear strength parameter, Φ'_r .

Investigations into the influence of the internal plate design are demonstrated in Figure 5.7. This resulted in similar trends to Figure 5.6 but it is noted that there is a disparity within the response of the clay between the different machines. Both would normally exhibit a positive effect. Altering the strain rate mid test at the point of loading but allowing it to settle prior to continuing with the process will produce at higher rates a negative effect. This is more evident on machine 2 (grooved internal plate design) but at mid range machine 1(rough internal plate design) shows the greatest impact. Greater disparity is also exhibited between the machines which imply that the machine design does have an affect on the final residual strength. Reducing the rate from above 10 mm/min to below 0.1 mm/min on both machines produces a positive effect.

The role of clay plasticity was also investigated. Figure 5.8 reveals similar trends for the high plasticity clay at St. Catherine's Point (blue symbols) and very high plasticity clay at Warden Point (green symbols). These trends consist of positive effects for a reduction in strain rate and negative effects for an increase in strain rate. Detailed analysis of St. Catherine's Point clay reveals that by reducing the strain rate the result was either a positive or neutral effect. Low increasing high strain rates produced lower values than non varied rates resulting in negative effects. Specifically the largest increase in Φ'_r was when the fastest speed was reduced to the slowest resulting in, an increase of 9.2°. Analogously, when the slowest rate was increased to the fastest rate, a reduction of 10.42° in Φ'_r was achieved. These values showed less disparity when compared to their original rate's value of Φ'_r with differences of 0.3° and 1.53°, indicating that it is the original rate which has the greatest influence on the final strength. Warden Point, Sheppey sample exhibited the same correlation as the St. Catherine's Point sample. The largest increase was 6.62° which was between the fastest and the slowest rate, a decrease of 44.5022 mm/min. By increasing the rate, the largest decrease occurred when the rate was accelerated by 22.22438 mm/min which reduced Φ'_r by 6.4°.

Dromore (red symbols) has a different correlation where the same general trend occurs but in reverse to the previous two (Figure 5.8). On closer examination those that ultimately end up as 0.8904 mm/min instead of the rate overall can be taken as those for which this is an increase resulting in a negative effect and for the opposite where the rate is decreased, a positive effect occurs. Two of the rate changes are not typical of the trend, the rate decrease from 44.52 mm/min to 0.8904 mm/min which results in an increase of 0.8°. 0.036 mm/min to 0.8904 mm/min also does not follow the trend exhibiting a decrease of 0.9°. Rates from the two extremes up and down from 0.8904 all follow the reverse of the trend excluding 0.036 mm/min to 22.26 mm/min which produced a decrease of 2.9°. Dromore exhibited little variation when the rate was constant throughout the test. Increasing the rate produced an increase in Φ'_r to a maximum of 2.68° which occurred between 0.018 mm/min and 0.8904 mm/min and the largest decrease occurred when 22.26 mm/min was reduced to 0.8904 mm/min which resulted in a 7.5° reduction in Φ'_r.

In conclusion, high plasticity clays exhibit positive and negative effects dependent on whether rate is decreased or increased from the original strain rate. Some high plasticity clays exhibit a slight reverse trend to the high plasticity clays but display a larger percentage of divergence within the results. This could be due to the silt content as the clay from Dromore is a boundary layer between two till deposits. The overall conclusion is changing the rate mid test does affect the final result.

5.5 Post-Test Moisture Content

For the ring shear test the sample is enclosed within an upper and lower platen. Then the platens, as a whole, are submerged in a water bath. The effect of rotation and soil extrusion changes the ratio of soil to water. In addition, the rotation is a "cake mixture" effect, the faster the clay is rotated, the better the clay and water combine. Clay was analysed pre and post test according to the moisture content procedure cited in Section 4.6. Using the second of the samples from St Catherine's Point for each strain rate test on each machine the moisture content of the sample was monitored. To aid the analytical process, analysis of the plastic and liquid limits were also carried out to identify these boundaries.

Generally, as demonstrated by Figure 5.9, post-test moisture content has a tendency to increase with strain rate. The clay tested was from St. Catherine's Point, Isle of Wight and was at just above the plastic limit. All strain rates as seen from Figure 5.9, exhibit an increase in moisture content post test. Between the two Bromhead ring shears there was also some disparity between the different samples for the identical strain rates.

Low speeds of under 0.1 mm/min produced a slight trend of increasing with increasing strain rate. Initially there is an increase of 9% from the sample moisture content at the lowest rate of 0.018 mm/min. At this point, the difference between the two machines was 1.2%. By increasing the rate by 0.009 mm/min to 0.027 mm/min, the moisture content post test increases by between 6.63% and 8.4% depending on the machine used. Actually, the moisture contents post test at this rate display a variance of 0.6. For the middle of the range at 0.036 mm/min, there is a decrease of 3.3% on one machine compared to a 4% increase on the other machine resulting in an overall average increase of 0.4%. There is still a minimum of a 14% increase compared to the sample moisture content. Variation between the different machines results widened further at 0.053 mm/min with a 12.3% gap between the two post test moisture contents, both of which are a smaller percentage to the ones at 0.036 mm/min. Furthermore the next increase in rate to 0.072 results in a reduction in variation between machines to 6.71% which is due to the machine with the lowest post test moisture content increasing and the other one reducing by 2% compared to the previous strain rate. Overall, the trend at low strain rates fluctuates but is still a minimum of 8% gain in moisture content during the test compared to the sample moisture content. This rises to a maximum 20% increase with an average of 8.5% reduction in soil mass.

The next block of tests was carried out around a strain rate of I mm/min. On average they produce a slight increase in moisture content compared to the low speeds of below 0.1 mm/min. At 0.4452 mm/min there is a 25-26% increase in moisture content compared to sample moisture content which effectively doubles the amount of water within the soil. This is an increase of 8.3-15.4% compared to the highest of the strain rates. Variance between the two machines then increases to 15.7% when the strain rate is increased to 0.5936 mm/min causing a 2.4% to 18.4% decrease in moisture content post test compared to 0.4452 mm/min. Comparatively a smaller decrease is produced by 0.8904 mm/min that is to 0.5936 mm/min although the variance between the machines does increase from 15.7% at 0.5936 mm/min to 16.52% at 0.8904 mm/min. At 1.34 mm/min smaller post test moisture content is achieved with a range between 35.05 and 35.5 therefore only a variance of 0.45%, a massive reduction when compared to the same at the strain rate of 0.8904 mm/min. An increase in post test moisture content is then attained with the increase in strain rate to 1.78 mm/min. However this is not as large percentage moisture content as 0.4452 mm/min produces but is an increase of 8.49% - 11.8% compared to 1.34 mm/min. Comparatively there is variance increase at 3.76% between the two machines - it is greater than the variance at 1.34 mm/min. In general, there is a slight trend of increase with strain rate this is associated with an average 11.9% in soil loss. Maximum post test moisture content for this range of speeds occurred at 0.4452 mm/min with a moisture content of 50.9% compared to the minimum which occurred at 0.8904 mm/min and was only 30.8% which in contrast to the initial sample moisture content is only a 5.9% increase.

At between 7.42 mm/min and 44.52 mm/min, four out of the five rates are above Imperial College's threshold of effect at 10 mm/min rate. Most of the rates within this range have post test moisture contents on one or both machines which are above the liquid limit. Specifically they are on the verge of being fluid and therefore would have different strength properties. Overall the post test moisture content follows a basic trend of increasing with strain rate. Even the lowest of the range rates at 7.42 mm/min has an increase of between 8.76% and 13.7% when compared to a rate of 1.78 mm/min. Differences between the machines give rise to a disparity of 8.7% in the moisture content post test. When the rate is increased to 14.84 mm/min, the post test moisture content is between 65.33 - 65.6 giving a variance of 0.27% between the two machines. That's an increase of 5% when compared to the liquid limit. Divergence within the range occurs at 22.26 mm/min where the post test moisture content is between 48.8 and 51.63% with a variance of 2.83% in the moisture contents of the two machines. For this range, the minimum value of moisture content post test at 48.8% occurs at 22.26 mm/min. At 33.39 mm/min the range of moisture contents gives the highest average at 69.95% post test for this range of tests. Comparing the two machines, there is a 4.7% variance between them both of which are above the liquid limit by 7.1% and 11.8% respectively. Greater variation between the machines occurs at 44.52 mm/min, where the variance is large at 26.9%. Despite this, the maximum value for post test moisture content is achieved at this rate, 77.3% is a 52.4% increase on the pre test moisture content and 16.8% above the liquid limit. Further more, combined with an average 12.3% loss of the original sample, the remaining clay will be extremely workable.

Regardless of the fluctuations the overall trend is of a slow increasing post test moisture content up to a point but this trend is more evident and rapid at above 7 mm/min as shown by Figure 5.9. However, overall this must affect the final value for residual strength.

5.6 Effect of plasticity index on moisture content post test

To further validate the work from Section 5.5, a smaller range of tests were rerun using more samples from different locations. Each sample was tested on one machine at four different rates and the moisture content calculated. Overall, both , high plasticity clay from Northern Ireland and Warden Point, a very high plasticity clay from Isle of Sheppey, exhibited a trend of increasing moisture content post test as demonstrated in Figures 5.10 for Warden Point and 5.11 for Dromore. Tested at close to the plastic limit of the individual clay, the results were plotted on different graphs together with the results from the St. Catherine's samples using the same machine to provide a comparison.

Warden Point, Sheppey, displays an almost linear relationship between post test moisture content and strain rate at 0.85 "R" squared. At above the sample moisture content by 9% at 0.036 mm/min, Sheppey exhibits a smaller increase in moisture content than St. Catherine's at the same rate on the same machine by 11%. This trend continues with an increase of 30% at 0.8904 mm/min which is a larger increase than St. Catherine's by 7.3%. Liquid limit for Sheppey is high at 86%. Despite this at 33.39 mm/min and 44.52 mm/min, the post test moisture content exceeds this percentage. For 33.39 mm/min there is an increase compared to the sample moisture content of 51.7% which compared to St. Catherine's produces a 4.3% higher increase in moisture content. At this rate the liquid limit is exceeded by 7.6%. The highest rate of 44.52 mm/min produces a 57.4% increase in moisture content after the test compared to the sample moisture content, which is actually 13.3% above the liquid limit. Comparatively the maximum strain rate with Sheppey has less of an impact than St. Catherine's which experiences a 16.8% increase under the same conditions. Even though the plasticity indexes vary only by 9%, there are slight variations in the response to the increase in strain rate in terms of post test moisture content. Nevertheless, they both exhibit a trend of increasing post test moisture content as strain rate increases as shown in Figure 5.10. Specifically they both exhibit higher than liquid limit moisture contents for the rates above 7 mm/min.

Dromore is high plasticity clay with a plasticity index of 28.5% although it has a lower index value compared to 40.4% for St. Catherine's Point, Isle of Wight which is also a high plasticity clay. At 0.036 mm/min and 0.8904 mm/min there is limited change compared to the sample moisture content at 1.5% and 1.8% increase respectively. A disparity of increase from sample moisture content is found between St. Catherine's Point clay and Dromore under identical conditions at 0.036 mm/min and 0.8904 mm/min is shown by a difference in increase of 11.8% and 4.1% respectively. However the increase in post test moisture content when compared to the increase in strain rate produces an "R" squared value of 0.999869 that is almost 1.0 therefore a virtually perfect linear relationship The highest rates produce lower percentage increases than Sheppey and St. Catherine's nonetheless still surpass the liquid limit. By increasing the rate to 33.39 mm/min the moisture content post test was 25.5% higher than the sample moisture content and exceeded the liquid limit by 1.1%. For the highest rate at 44.52 mm/min the moisture content surpasses the liquid limit by 3.1% and consequently the sample moisture content by 27.5%. Despite the lower plasticity index, Dromore follows an analogous trend to St. Catherine's Point as demonstrated by Figure 5.11.

In conclusion, regardless of the plasticity index, the clays tested all exceeded the liquid limit at high rates. This would obviously have an impact on the residual strength and therefore result in unexpected answers.

5.7 Vertical Displacement

Whilst the machine rotates, soil is extruded concurrently with the increase in normal stress and subsequent compression of the sample. Extrusion occurs even in the Bromhead Ring Shear with the negligible gap between the upper and lower platens. Acceleration in strain rate are accompanied by an increase of 3.8% soil extrusion across the tested range as shown in Figure 5.9 with the largest disparity between rates below 0.1 mm/min and those in the region of 1 mm/min. How this affects the vertical movement is detailed within this section.

The greatest vertical deflection typically occurs at the maximum normal stress. Generally for this, there is a trend which is highlighted in Figure 5.12 to demonstrate the increases in maximum vertical movement as the strain rate is accelerated. Closer examination of the graph reveals that the values fluctuate and different clay samples follow different patterns. St. Catherine's Point samples follow similar trends with a fairly dispersed correlation but where each block of strain rates exhibits an overall increase compared to the previous block. Maximum vertical movements vary from 0.27 to 2.19 mm at low speeds, 0.59 - 3.068 at mid range and 0.532 - 4.88 mm at the highest speeds over for the two samples using two Bromhead Ring Shear apparatuses referred to as machine 1 and machine 2. Besides the various anomalies which are detailed below, St. Catherine's Point's first sample has a tendency to have lower maximum vertical movements compared to the second sample from the same area.

Rates below 0.1 mm/min produced similar values of displacement for both machines; except for at 0.036 mm/min for sample one where they differ by 1 mm and 0.071 mm on the same sample which vary by 0.9 mm. For both samples at 0.053 mm/min there is a decrease as opposed to an increase in vertical movement by between 0.3 - 0.38 mm. Mid range rates at around 1 mm/min produced more dispersed results. Both machines for the two samples from St. Catherine's Point show trends akin to one another but exhibit a divergence between machines of 0.5 - 1.0 mm vertical movement. Out of trend results occurred on the second machine for the first sample at 1.34 mm/min which produced a 0.2 mm reduction compared to the previous strain rate at 0.8904 mm/min and four with the second sample at 0.8904, 1.34 and 1.78 mm/min where one machine 1 exhibited movements that were approximately 50% of the rest of the range's trend of values. At 1.78 mm/min machine 2 also produced a reduction of 1 mm compared to the previous strain rate with the second St. Catherine's sample.

Faster speeds resulted in greater divergence between the samples and between the results from the different machines. An overall trend of increasing maximum vertical movement with accelerating strain rate was more evident but between the machines the movement varied by as much as 2.5 mm. An anomaly occurred at the maximum speed with one test as it was approximately 3 mm below the others for same rate. Sample one from the St. Catherine's Point did exhibit a trend of increasing maximum vertical displacement up to 14.84 mm/min and then gradually decreased apart from the anomaly which was a rapid decline. The second sample from the same location showed a basic trend of increasing vertical movement with faster strain rates at the higher speeds except for machine one at 44.52 mm/min which produced a decrease of 0.5 mm compared to the previous strain rate.

Dromore, Northern Ireland displayed divergent results at lower speeds, fairly stable values at mid range and contrasting results at the highest strain rates. Overall the trend is interpretable as increasing maximum vertical movement with strain rate. Lower rates resulted in a variance between the machines between 0 and 1 mm and an overall range between 0.42 and 1.49 mm at low speeds, 1.17 and 2.22 mm at mid range and 1.71 and 4.98 mm at high rates. Values fluctuated at lower speeds and displayed a trend which is an alternating decrease/increase pattern. Mid range speeds gave higher vertical displacements and greater divergence between the machines of between 0.3 and 1.05 mm. Similar to the lower speeds an alternating pattern is observed but the reverse to the lower speeds. At high speeds the values are not the only divergence between machines, the trends are also dissimilar. Machine 1 exhibits increasing vertical movement with strain rate excluding 22.26 mm/min and 33.39 mm/min which produce out of sequence results. At 22.26 mm/min the movement is 2.5 mm less than at 14.85 mm/min. 33.39 mm/min produces an increase towards the trend but is 1.1 mm lower than at 44.52 mm/min. Conversely the second machine decreases and then increases in vertical movement with strain rate so it is the higher value of 12.68 at 7.42 which is out of sequence. Values vary between 1.71 mm to 4.98 mm in vertical movement at the highest range of strain rates for Dromore. This is clearly evident from Figure 5.12.

A smaller range of strain rates was used on the clay from Warden Point, Isle of Sheppey. Despite this, a trend of increasing vertical movement with strain rate can be interpreted from the results. This is clearest from machine 1 where the trend is exhibited by all but at 44.52 mm/min where there is a decrease of 0.8 mm compared to the previous strain rate. Machine 2 decreases at low speeds then increases to mid speeds then decreases again to stabilize at higher rates. Vertical movements at their maximum for the Warden Point sample vary between 1.42 mm and 5.43 and exhibit more divergent results from the different apparatuses at higher speeds.

Compression which occurs during actual loading is also affected by increases in strain rate. Similar amounts of compression occur at low and mid range strain rates during loading as demonstrated by Figure 5.13. At rates from around 10 mm/min show greater dispersion in terms of the amount of vertical compression experienced during loading. Low speeds have a range of 0.228 mm to 1.19 mm, mid range speeds have a range of 0.39 mm to 1.34 mm compared to high speed's range of 0.338 mm to 3.02 mm. St. Catherine's Point samples display an increase in vertical compression with strain rate. As the rate increases the divergence between the machines for each of the samples is observed to increase. Dromore exhibits virtually no change fluctuating above and below almost a straight line for rates below 2 mm/min. Above 7 mm/min the general trend is one of increasing with strain rate but there are two anomalies at 7.42 mm/min and 22.26 mm/min which produce an increase of 1.5 mm and a decrease of 0.6 mm respectively. Warden Point, for which the smaller range of strain rates was investigated, exhibits an increase up to 2 mm/min and then a decreasing trend after 7 mm/min for machine 2. However machine 1 exhibits a trend of increasing with strain rate. Consequently divergence between the machines increases with strain rate. Taken as a whole, the range for vertical compression during loading is between 0.546 mm and 1.709 mm with a maximum variance between machines of 1 mm.

Figure 5.14 demonstrates that the linearity of displacement is fairly constant with the majority of the "R" squared results above 0.9. Furthermore, there is a slight trend of the linearity increasing with strain rate. At low speeds below 0.1 mm/min there are three which produced "R" squared values between 0.80 and 0.85 which indicates little or no correlation between the sets of values. These were for machine 1, Warden Point at 0.027 mm/min and St. Catherine's Point at 0.036 mm/min on both machines. However rates within the region of 1 mm/min exhibit less correlation with twelve "R" squared values below 0.9 including a St. Catherine's Point sample one on machine 1 at 1.34 mm/min with a value of 0.81. Those between 0.85 and 0.9 were from St. Catherine's Point sample one on both machines at 0.4452 mm/min, machine 1 at 0.5936 mm/min and 1.34 mm/min and machine 2 at 0.8904 mm/min; the others were from Dromore at 0.4452 and 0.8904 mm/min respectively. The higher rates produced less disparity with most "R" square's above 0.95 and almost all above 0.90, excluding three Warden Point tests at 22.26 mm/min and 44.52 mm/min on machine 2 which were at 0.71 and 0.72 respectively. Values of R squared were achieved between 0.85 and 0.9, for St. Catherine's Point, Sample 2 on machine 1 at 7.42 mm/min and Warden Point, machine 1 at 44.52 mm/min also did not prove to have a linear correlation.

Rate of volume change appears to decrease with increase in strain rate as shown in Figure 5.15, definition of rate of volume change being the vertical compression during loading divided by the horizontal movement during the same process. This shows an almost linear relationship with St. Catherine's Point, less evident with Warden Point and Dromore shows the largest divergence between machines. Furthermore at faster rates there is a larger horizontal displacement therefore the rate is lower because even though as Figure 5.6(b) demonstrates, vertical compression during loading increases, the horizontal displacement actually increases more rapidly therefore creating the impression of a reduction in rate.

Movement is affected by strain rate but clay plasticity and machine design also affect the actual quantity displaced.

5.8 The Effect of Apparatus Design on Soil Extrusion

Soil extrusion is directly affected by the rate of shear. This is implicitly linked to apparatus design although it is through both the displacement gauge readings and the obvious soil loss from the container post test makes the quantity appreciable. As soil loss increases this changes the void ratio and so has a direct affect on pore water pressures.

The different ring shears have differential amounts of gap. The Harvard apparatus has a 0.005 inch gap which causes excessive soil extrusion which according to La Gatta (1970) can lead to a maximum of 12% uneven normal stress distribution on disk samples. Taylor (1998) provides an excellent summary of Sassa's ring shear apparatus. Taylor (1998) noted that on the third generation of Sassa's ring shear it had a gap control mechanism. This gap control mechanism maintains the gap to 1/1000 mm variation. A servomotor maintains the gaps via a precise gap sensor via a feedback loop mechanism. The technical explanation for this is the gap is monitored by a precise gap sensor which sends a feedback signal to a servomotor which adjusts the gap accordingly. This reduces the soil extrusion and therefore reduces the change in the soil mixture.

Anayi (1990) whilst researching residual strength of clay at low normal stresses discovered some modification to the Bromhead ring shear apparatus was required. Specifically this modification by Anayi (1990) involved the introduction of vanes to improve the accuracy of the result. These cause the shear surface to form at mid height in the sample as opposed to near the surface. This modified ring shear apparatus requires less horizontal displacements for the clay to achieve residual strength and a Teflon coating inside the sample container was used to reduce extrusion.

Parathiras (1994) noticed that the NGI/IC ring shear test was not performing as it should during prolonged stages of fast shearing or at high rates of displacement and identified the cause as the gap control mechanism. The required solution was to develop a modification, which minimised soil loss, allowed for multistage, long duration and fast shearing tests on fine-grained soils. New confining rings were introduced which when compared to the old, resulted in a difference in residual strength values of between O and 2%. This modification resulted in tests losing up to seven times less soil than with the original rings. It also allows for prolonged stages of shearing. Parathiras added confining rings around the gap to prevent soil extrusion. The second generation of Parathiras' rings enable a steady residual strength to be achieved as the soil extrusion is reduced. Sassa (2004) details the next generation of intelligent system ring shears where the gap is controlled via a feed back mechanism thus keeping extrusion to an absolute minimum.

Previous research in this area indicates that soil extrusion not only affects pore water pressures but the actual formation of a steady residual strength. In this research using the Bromhead Ring Shear it is noted that initial moisture content, internal plates, strain rate, normal stress and preciseness of levelling the top of the sample all attribute to the amount of extrusion. Consequently all the above factors have an impact on residual shear strength.

5.9 Apparatus Design

Bromhead (2004b) concluded that strain rate affects were associated with apparatus design. For this research two Bromhead ring shears which are identical apart from the internal plate design were used. Machine 1 uses a "rough" or "coarse" texture and machine 2 uses a "grooved" pattern. This work was carried out concurrently with the strain rate effects work as all tests use the two machines.

Figure 5.16 and Figure 5.17 demonstrate the differences between the two types of plate. Overall the "rough" exhibits a positive trend with residual strength increasing with strain rate. As described in Section 5.3 the detailed analysis of the individual samples produces a less specific trend. St. Catherine's Point samples follow identical trends, the low speeds show disparity amongst the results, mid range exhibits trend of increase but a decrease in ϕ'_r of 5° changes the pattern; high speeds generate an increase with strain rate excluding 33.39 mm/min which results in a decrease in ϕ'_r of 1° compared to the previous strain rate of 22.26 mm/min. Warden Point exhibits no change for low speeds, then a decrease for 0.8904 mm/min and a tendency to increase with strain rate at high speeds. Dromore exhibits a trend of decrease then increase to levelling off at low speeds, decrease, increase, level off at mid range and the only difference at the highest speeds is an initial increase before the decrease. St. Catherine's and Dromore samples all tend to exhibit a reduction in Φ'_r with strain rate using the "grooved", even though the trend is slight with fluctuations within the results. Conversely Warden Point, Sheppey exhibits an initial decrease followed by an increase in Φ'_r with strain rate.

These trends are more evident in Figure 5.18. Excluding Sheppey the other clays appears to follow similar trends. It is at the highest strain rates where the discrepancies occur within the trends with Sheppey. Dromore follows a slightly shallower gradient of trend than the positive line. St. Catherine's Point contains anomalies at the 0.018 mm/min rate where there is disparity between the machines and the samples, the overall trend for St. Catherine's is still positive for "rough" and negative for "grooved".

The effects on post test moisture content are described within section 5.5, demonstrated in Figure 5.9 and on displacement within section 5.7 and exhibited by Figures 5.12 - 5.15. Discontinuous strain rate tests also investigate the effects of internal plates, the conclusions of which are described within Section 5.4 and are demonstrated in Figure 5.7.

The overall conclusion is that internal plates do affect the residual shear strength. Soil extrusion and therefore vertical deflection were demonstrated to be larger with the "rough" internal plate than the "grooved" in general and exhibit less linear rates of displacement. "Rough" internal plate in the post test moisture content investigations produced higher percentages overall than the "grooved" internal plate. Therefore positive and negative effects could be an artefact of apparatus design and not an actual strain rate effect.

5.10 Conclusion

In the latest report of research Fearon et al (2004) states that the only condition where negative rate effects occurred during the testing was at rates in excess of 1000 mm/min, where the sample had no pre cut surface and free water could reach the shear surface. Fearon et al describes the field conditions in which the likelihood of this to be rare due to the requirement of the shear surface to be "hydraulically linked" to the drainage layer. For example this would occur in a shear surface in clay which was above a granular layer or a clay layer within a fractured rock mass. This research supersedes previous research and thus further challenges the existence of negative rate effects being due to strain rate increases. It results in the further need for investigation into this area and increases the probability of the rate effects being due to other causes either individually or in conjunction with each other or with the increase of strain rate.

The investigations into the effect of strain rate on the residual shear strength parameter ϕ'_r; found on average there was a 1° increase per log cycle for the samples from St Catherine's Point , a just under 2° increase per log cycle for the sample from Warden Point and less than 1° decrease per log cycle for the sample from Dromore. None of these are a linear relationship there are fluctuations with all of these which would indicate influences in addition to strain rate.

Whilst investigating the effects of procedure, by changing the rate mid-test a measurable effect on the residual shear strength parameter ϕ'_r was observed. At the extremes up to a 7° increase when the rate is decreased using the sample from St Catherine's Point. When the rate is increased using the same sample up to a 6° decrease was observed. Analogously, for the same tests using the sample from St Warden Point up to a 4° increase when the rate is decreased. When the rate is increased using the same sample up to a 6° decrease was observed. Conversely, for exactly the same tests using the sample from Dromore, up to an 8° decrease when the rate is decreased. When the rate is increased using the same sample up to a 3° increase was observed. These observed effects may relate to the silt content.

Fearon et al research therefore implies the degree to which apparatus design affects the determination of residual shear strength is greater than has been previously assumed. In these investigations apparatus design even with only minor differences has been proved to have an effect on the established value. The Bromhead Ring Shear set up with different internal plates in the two machines used gave rise to different values of residual shear strength. Comparatively, of the two machines, the rough kriss cross internal plate was observed to expel soil at a faster rate than the machine grooved plate and produced higher post test moisture contents. Overall these investigations confirm that at moderate strain rate machine design is an important factor in the determination of residual shear strength.

Investigations into post test moisture contents showed that at the highest rates above 7mm/min the soil approaches its liquid limit and above 30mm/min it surpasses this limit and therefore behaves as a liquid. At the fastest rate, post test moisture content exceeds the liquid limit by as much as 15% for the samples from St Catherine's Point and Warden Point; and it was the same even for Dromore which is the siltiest of the clays at the same rate the liquid limit was 5% lower than the post test moisture content. This could indicate that at these fast rates the test changes from drained to undrained. Although it is classed as a total stress test in the British Standard (BS1377:Part7 Test 6 (1990) the ring shear test is actually a drained test. Boundary pore pressures which result from rotational effects and soil extrusion may build up which results in undrained pore pressures. Unless the test is run at a slower strain rate, for a longer duration or these pore pressures are measured. Therefore this appears as a rate effect and it cannot be argued that this is a drained test.

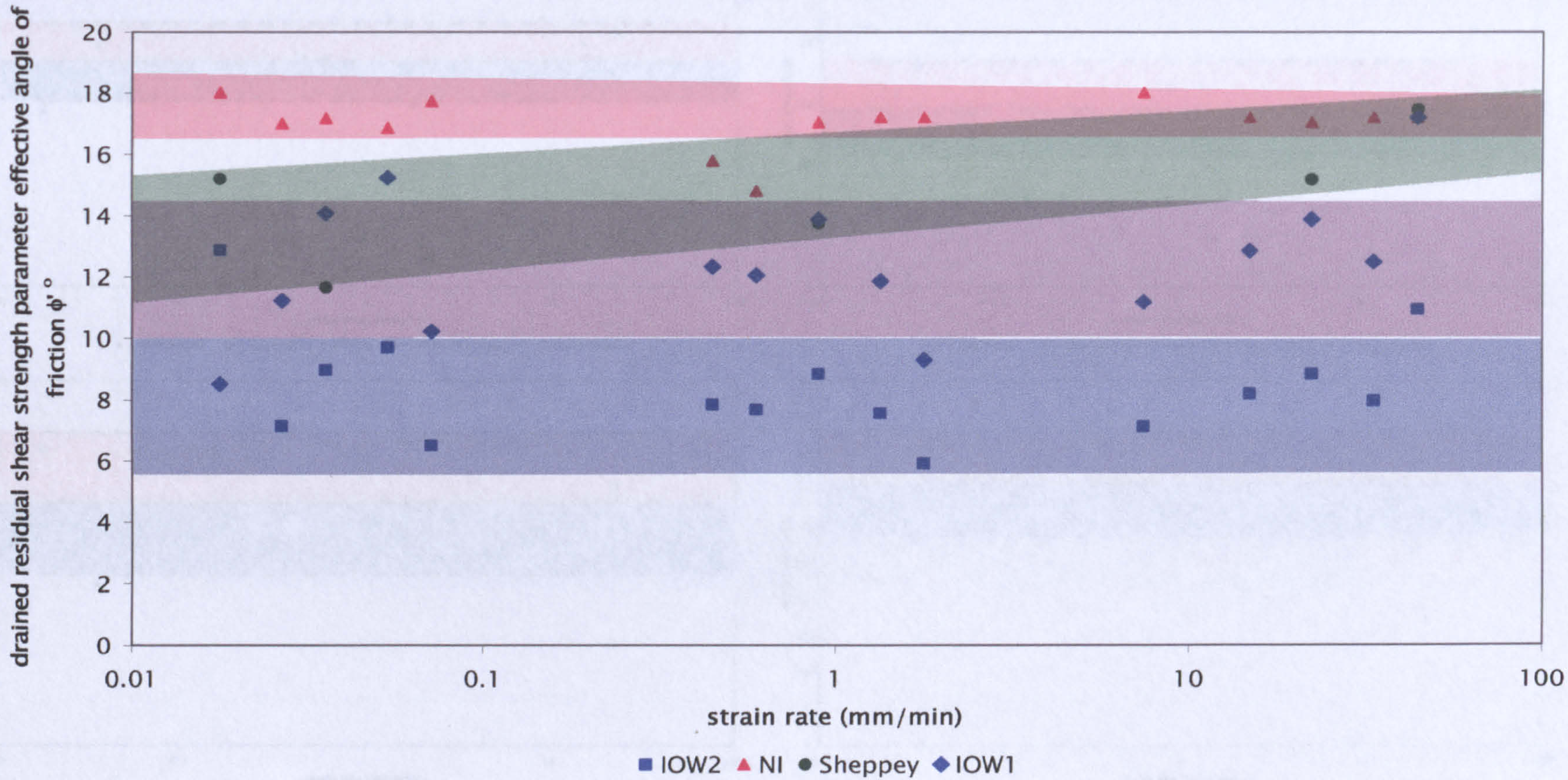


Figure 5.1: How varying the strain rate affects Φ'_r

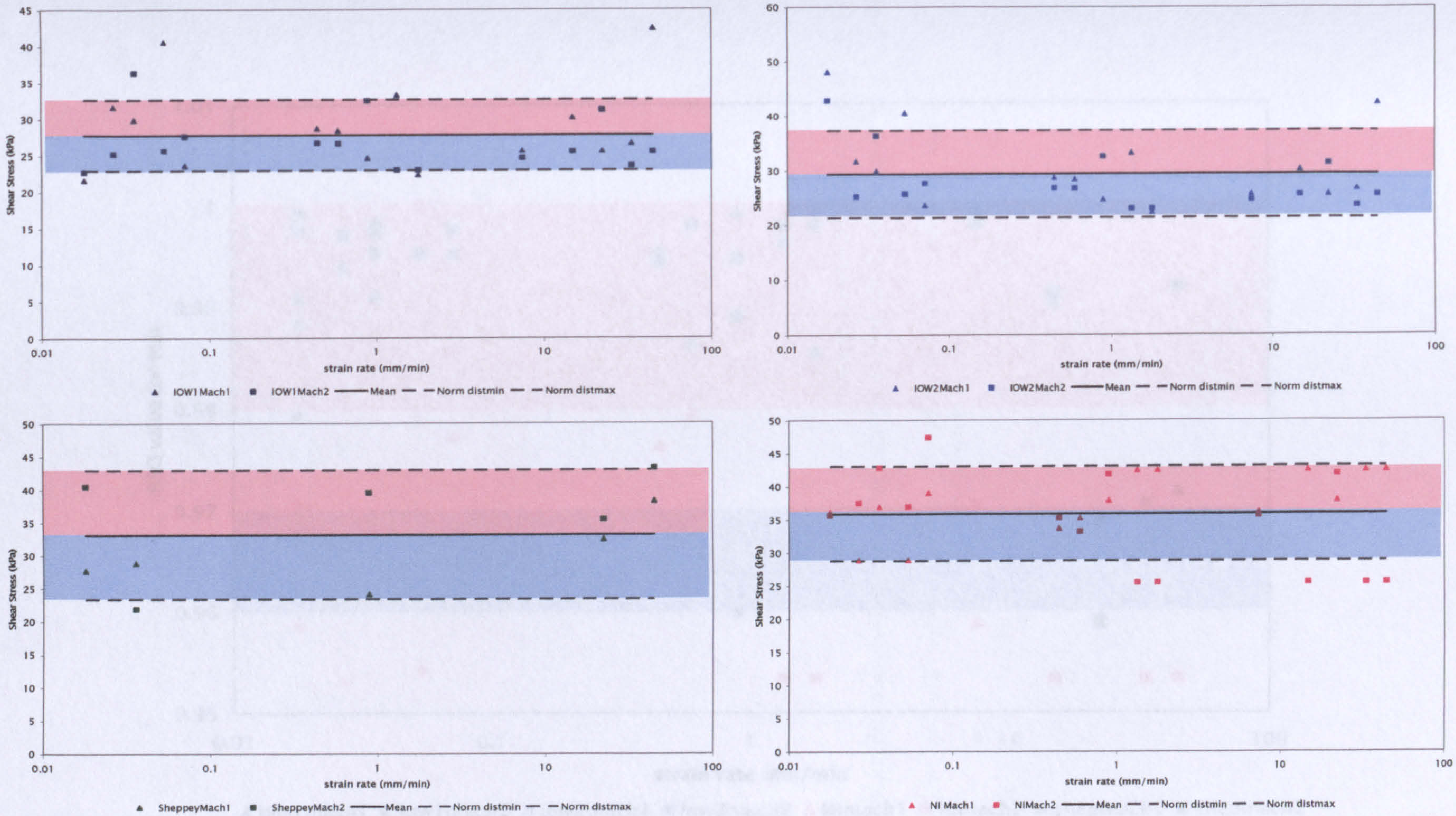


Figure 5.2: Normal Distribution analysis of rate of strain effects using 99% confidence at 198kPa

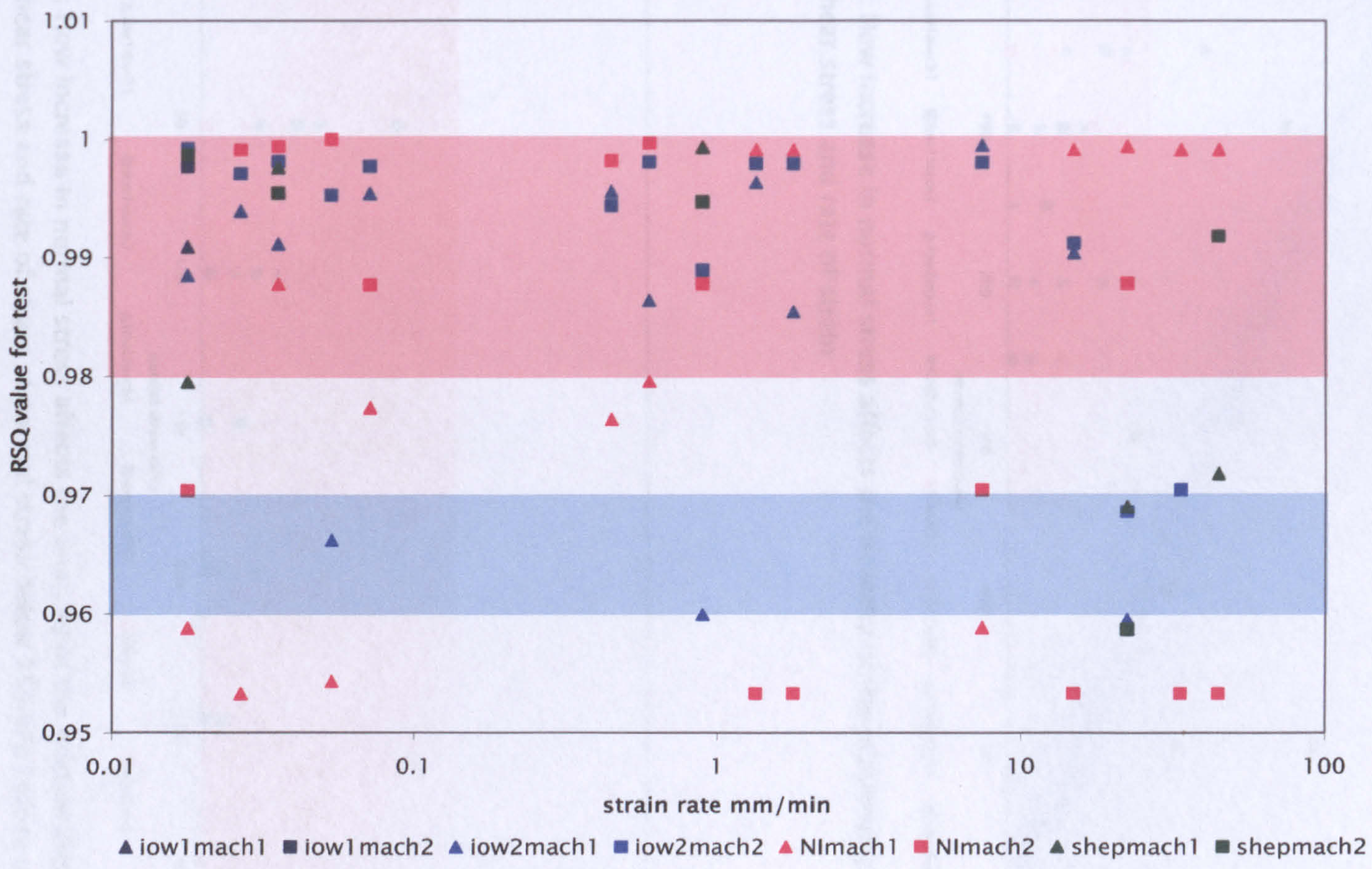


Figure 5.3: How strain rate affects linearity of the envelope.

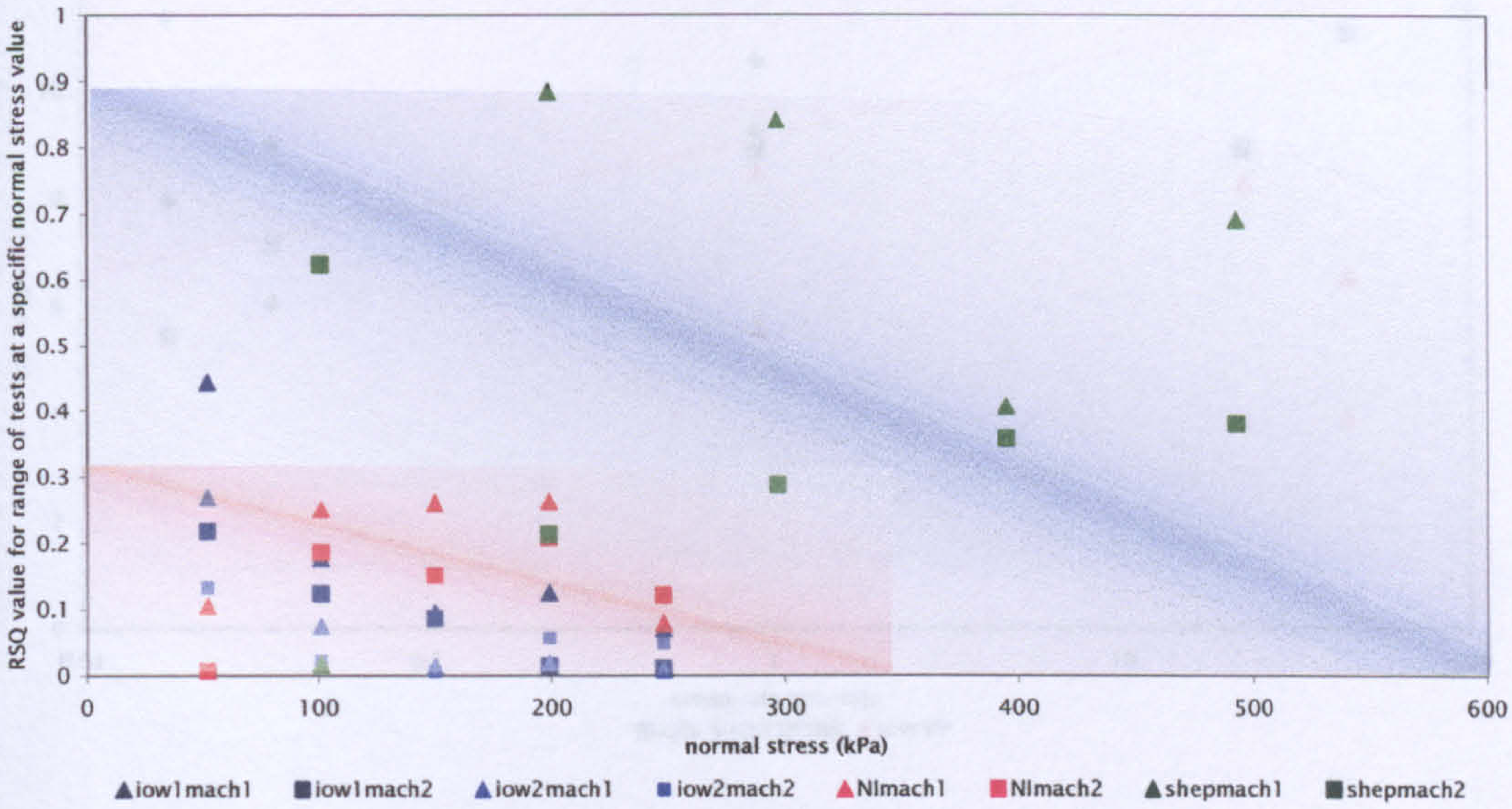


Figure 5.4: How increase in normal stress affects the linearity of the relationship between shear stress and rate of strain

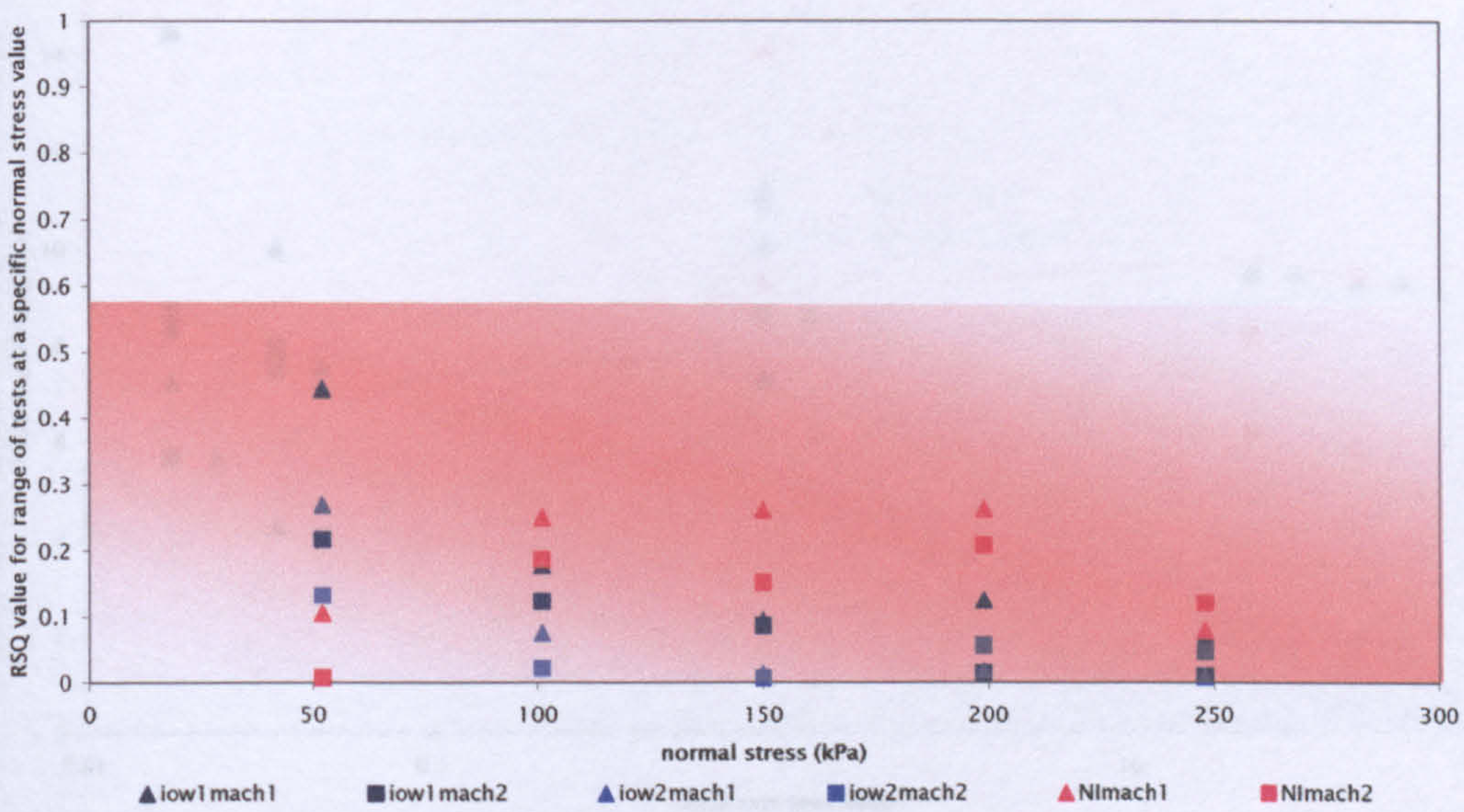


Figure 5.5: How increase in normal stress affects the linearity of the relationship between shear stress and rate of strain at normal stress below 250 kPa (Close up of lower range shown in Figure 5.4)

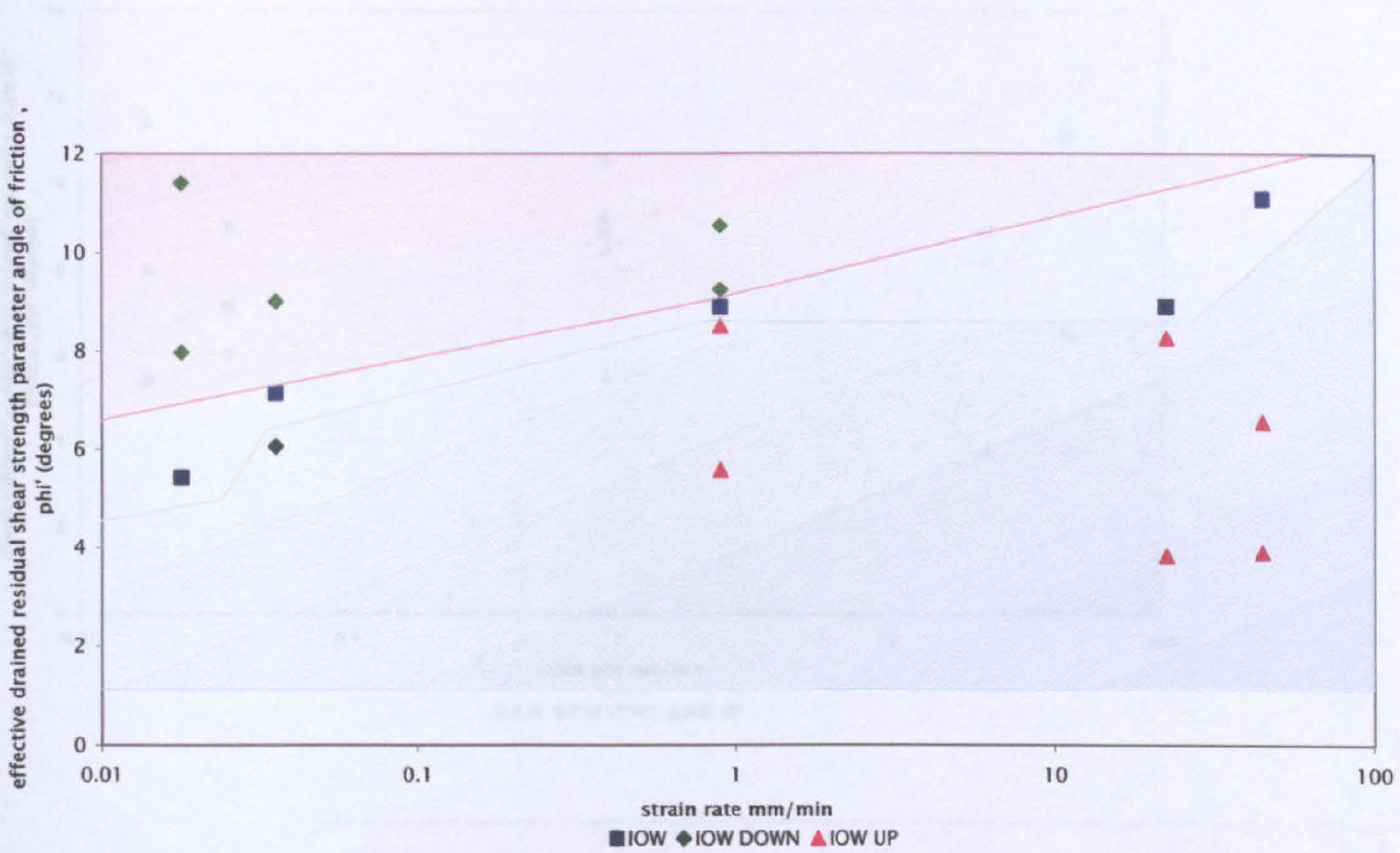


Figure 5.6: Discontinuous strain rate and its effects on the final value of Φ'_r using Gault clay from St Catherine's Point, Isle of Wight.

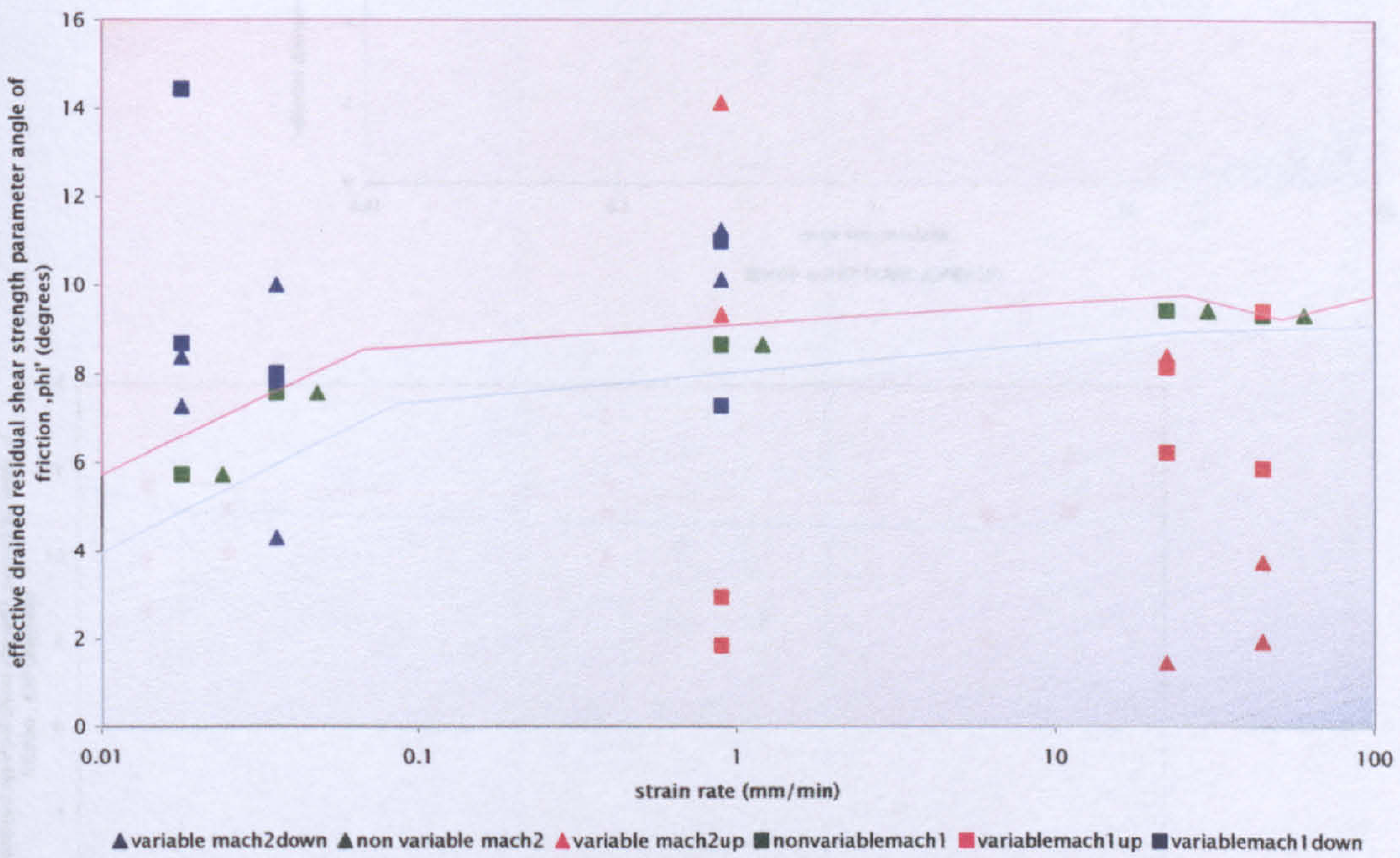
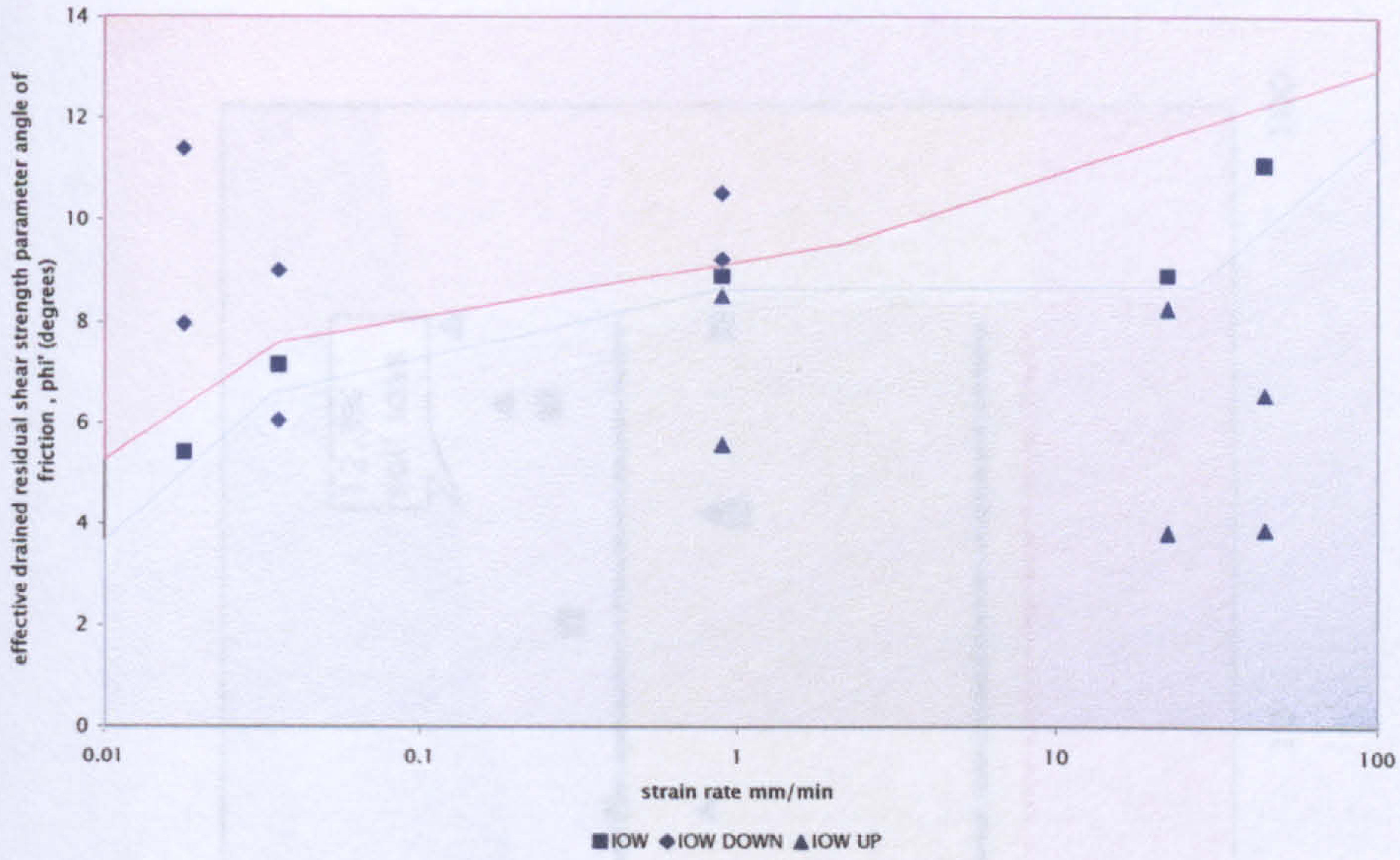
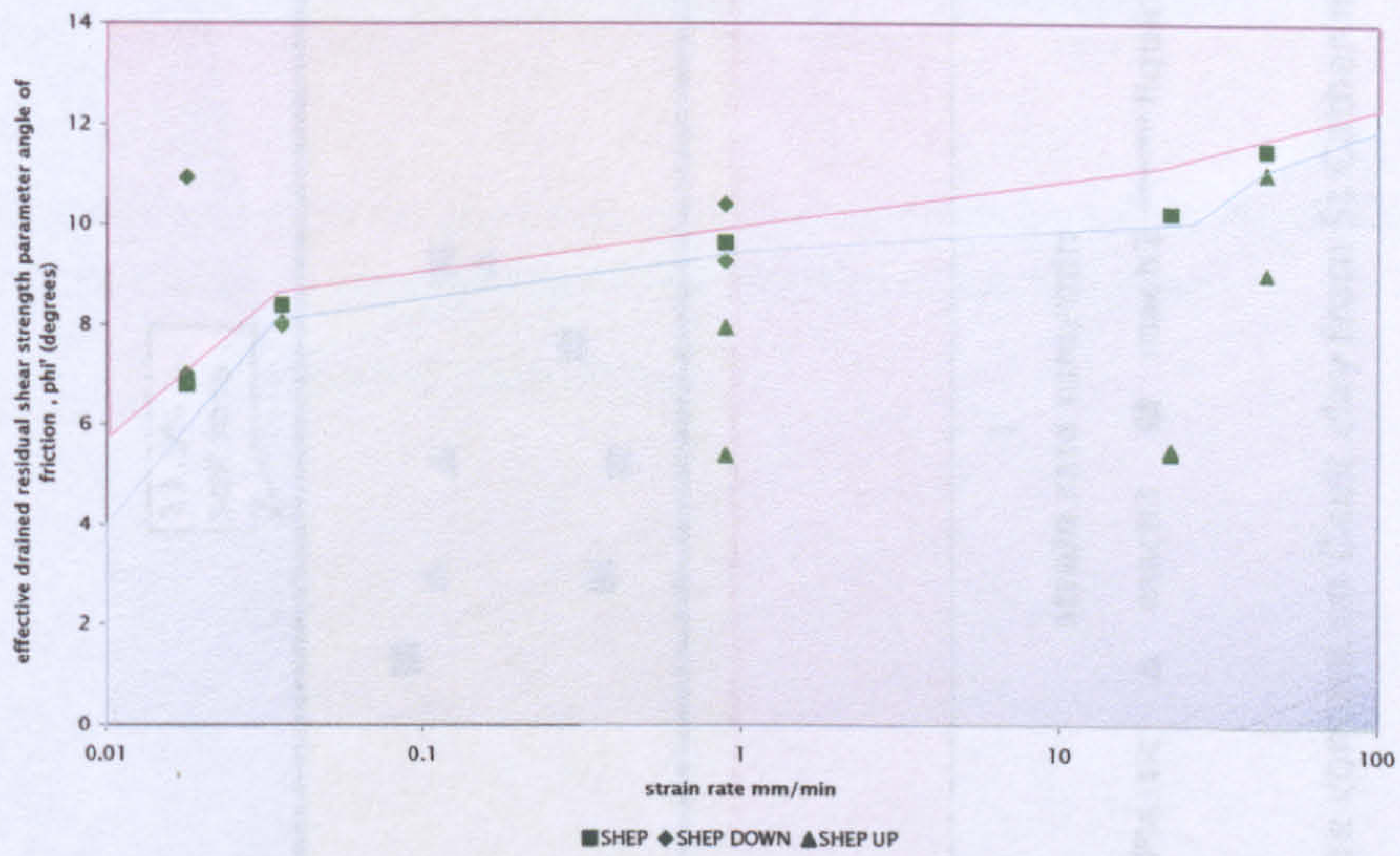


Figure 5.7: Discontinuous strain rate and the effect of the internal plate design on the final value of ϕ'_r using Gault clay from St Catherine's Point, Isle of Wight

N.B Machine 1 - Rough Internal plate design
Machine 2 - Grooved Internal plate design





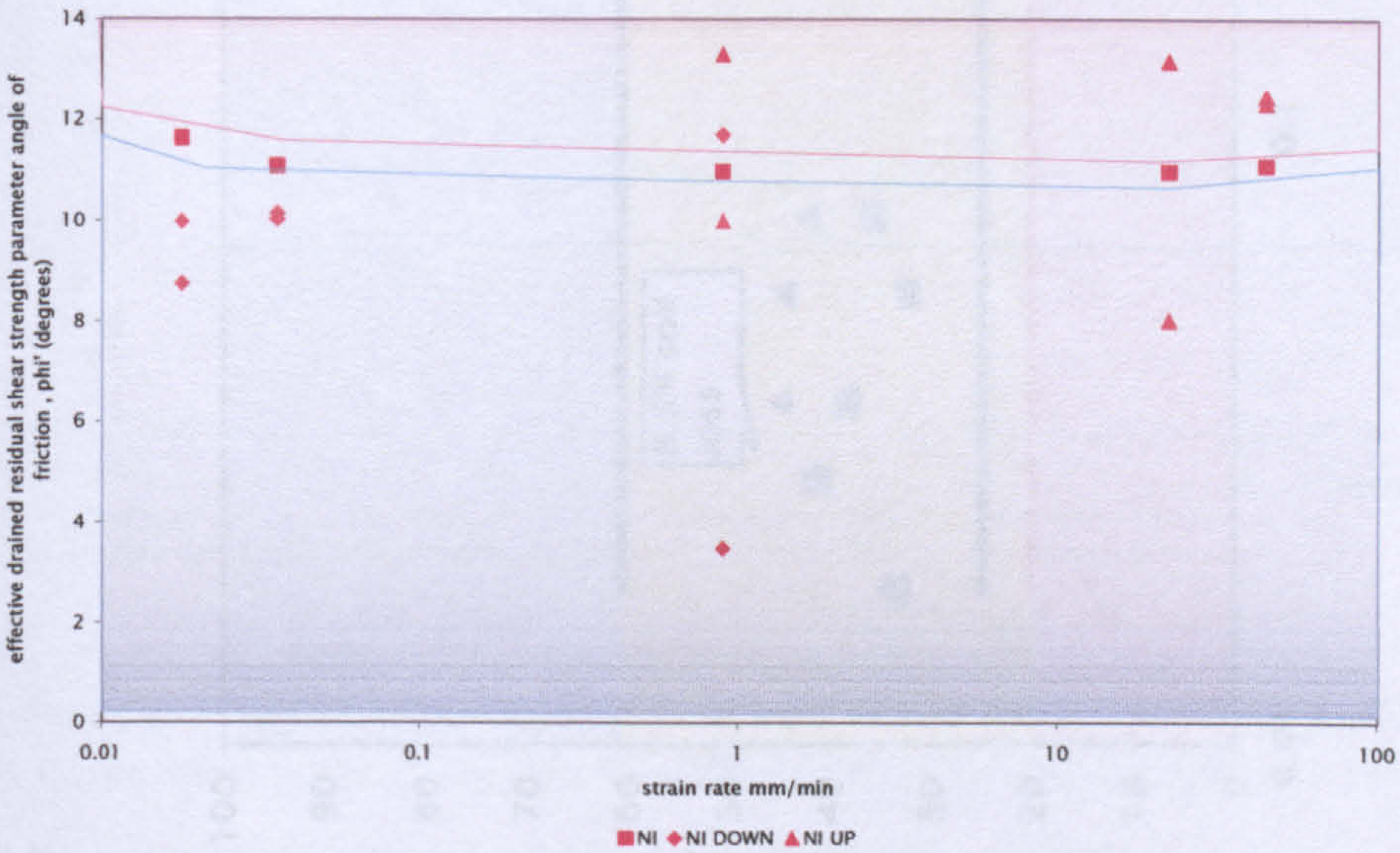


Figure 5.8: Role of clay plasticity on the effects of discontinuous strain rates

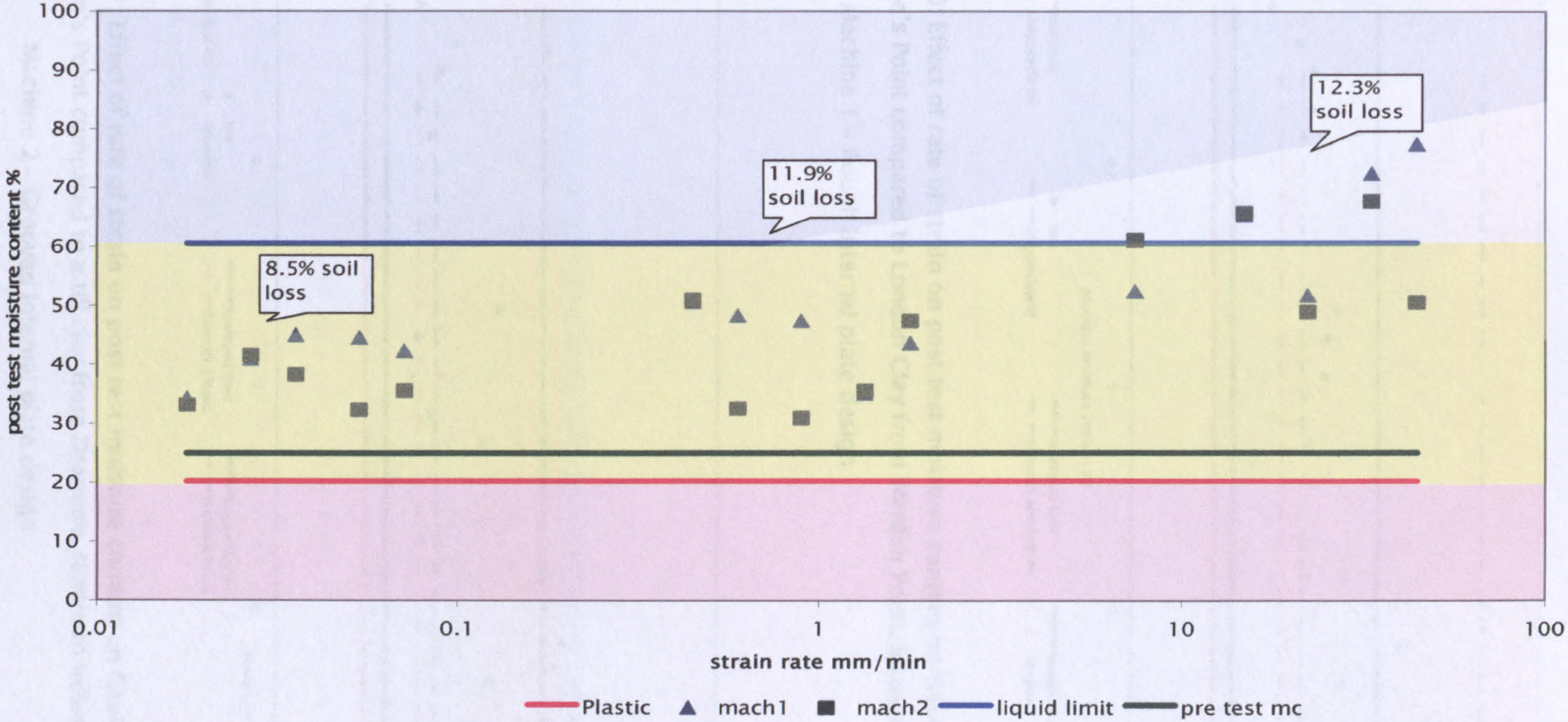


Figure 5.9: Effect of rate of strain on post test moisture content on Gault clay from St Catherine's Point

N.B Machine 1 - Rough Internal plate design Machine 2 - Grooved Internal plate design

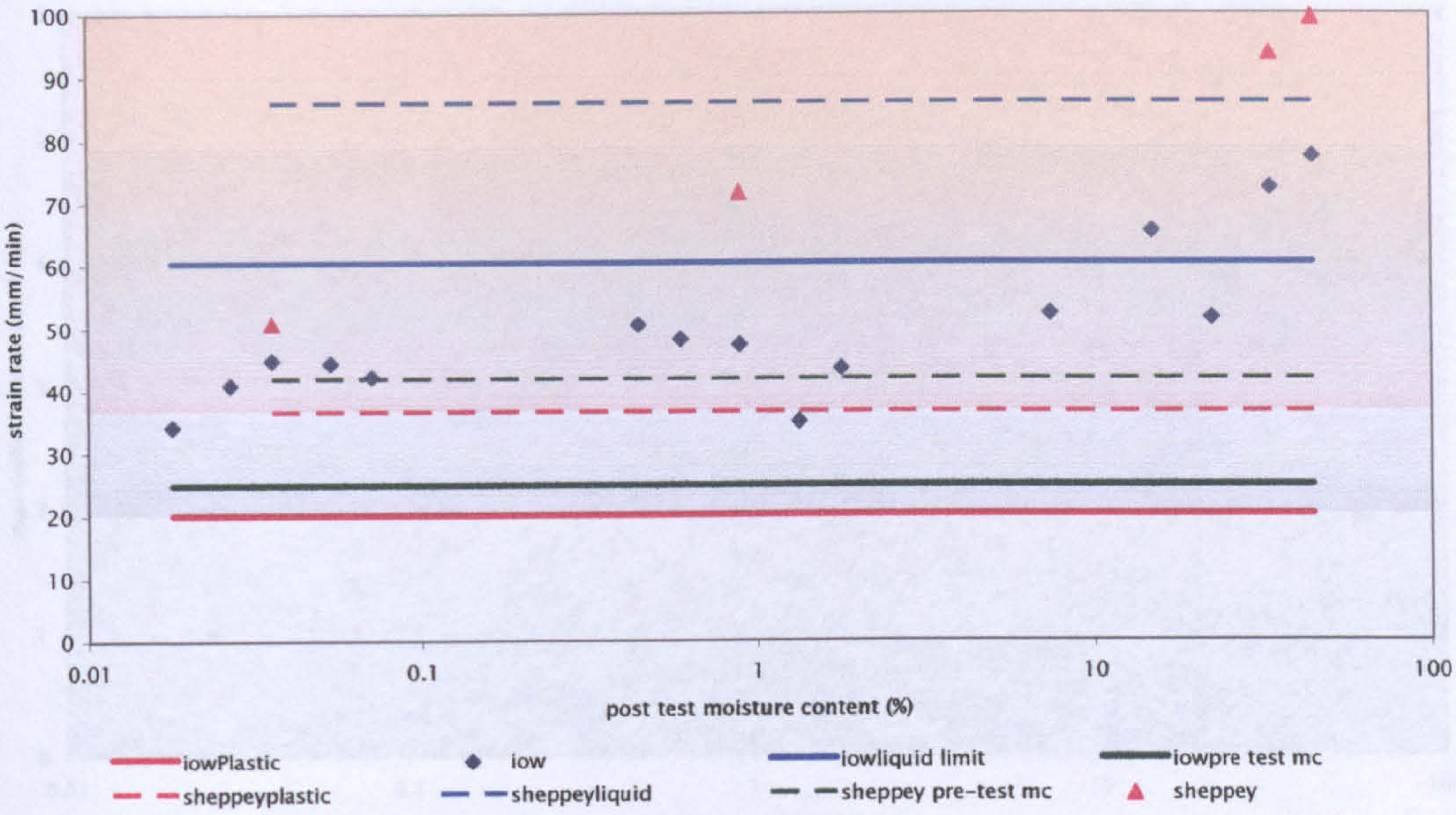


Figure 5.10: Effect of rate of strain on post test moisture content on Gault clay from St Catherine's Point compared to London Clay from Warden Point, Sheppey

N.B Machine 1 - Rough Internal plate design

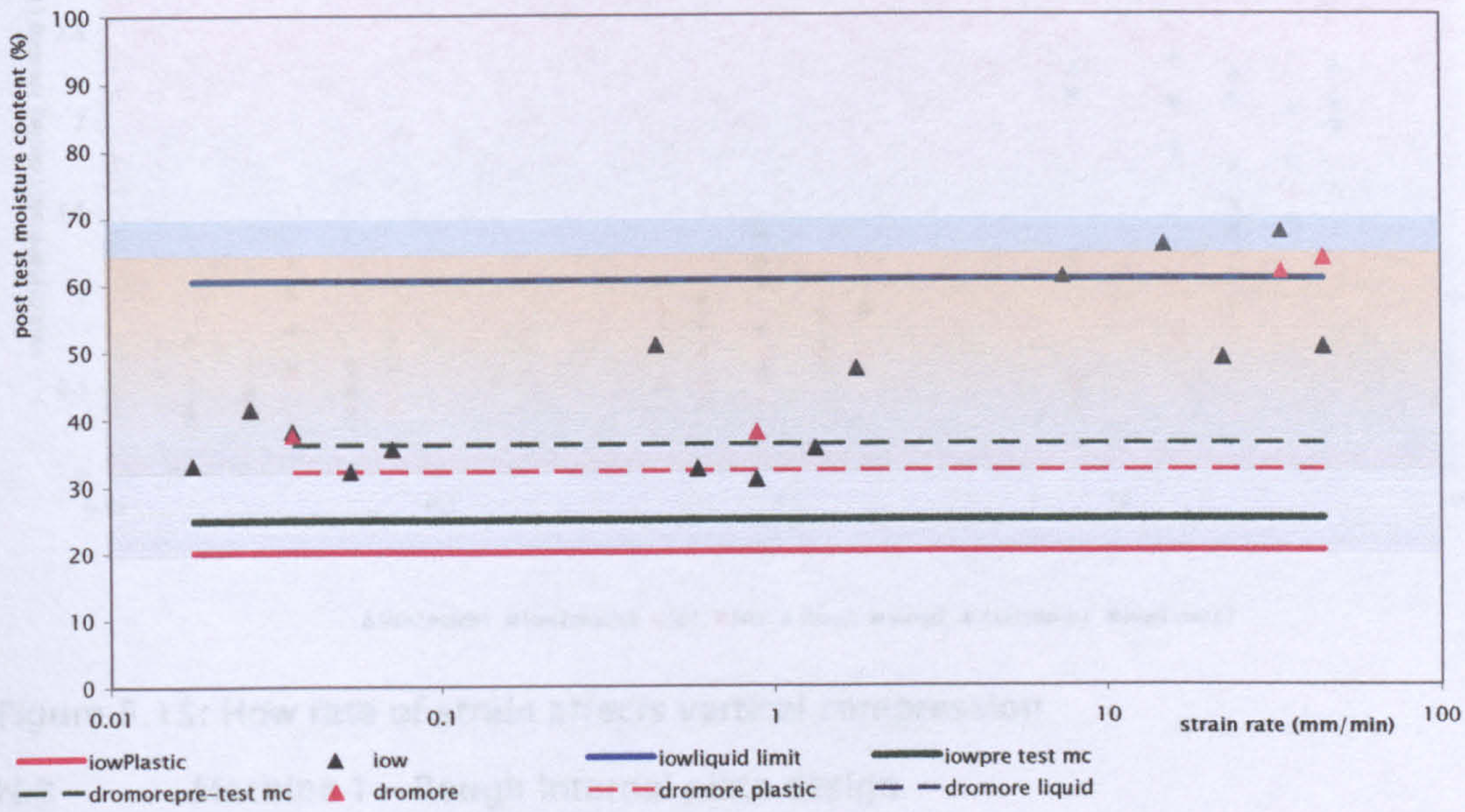


Figure 5.11: Effect of rate of strain on post test moisture content on Gault clay from St Catherine's Point compared to a till clay from Dromore, Northern Ireland

N.B Machine 2 - Grooved Internal plate design

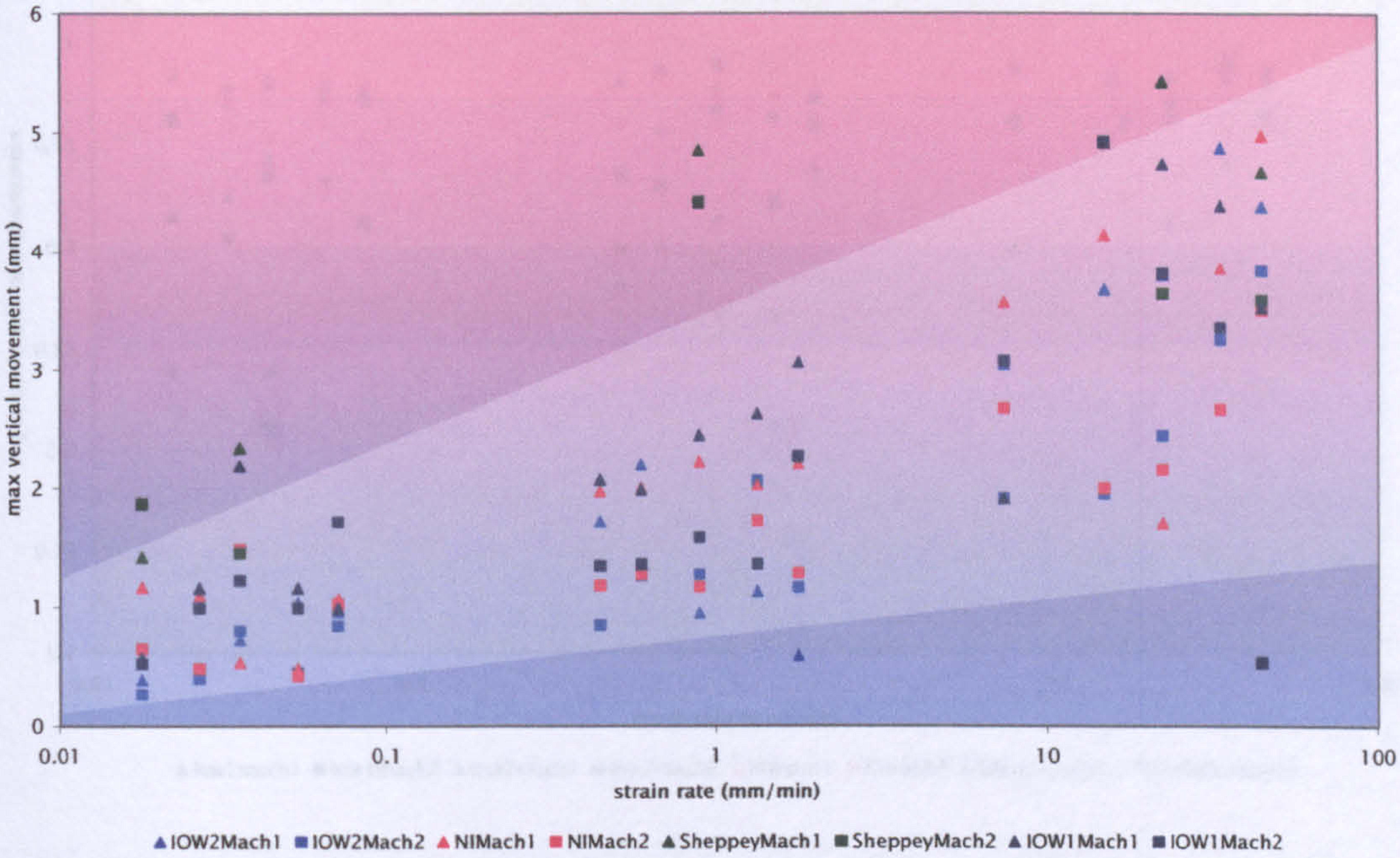


Figure 5.12: How rate of strain affects maximum vertical compression

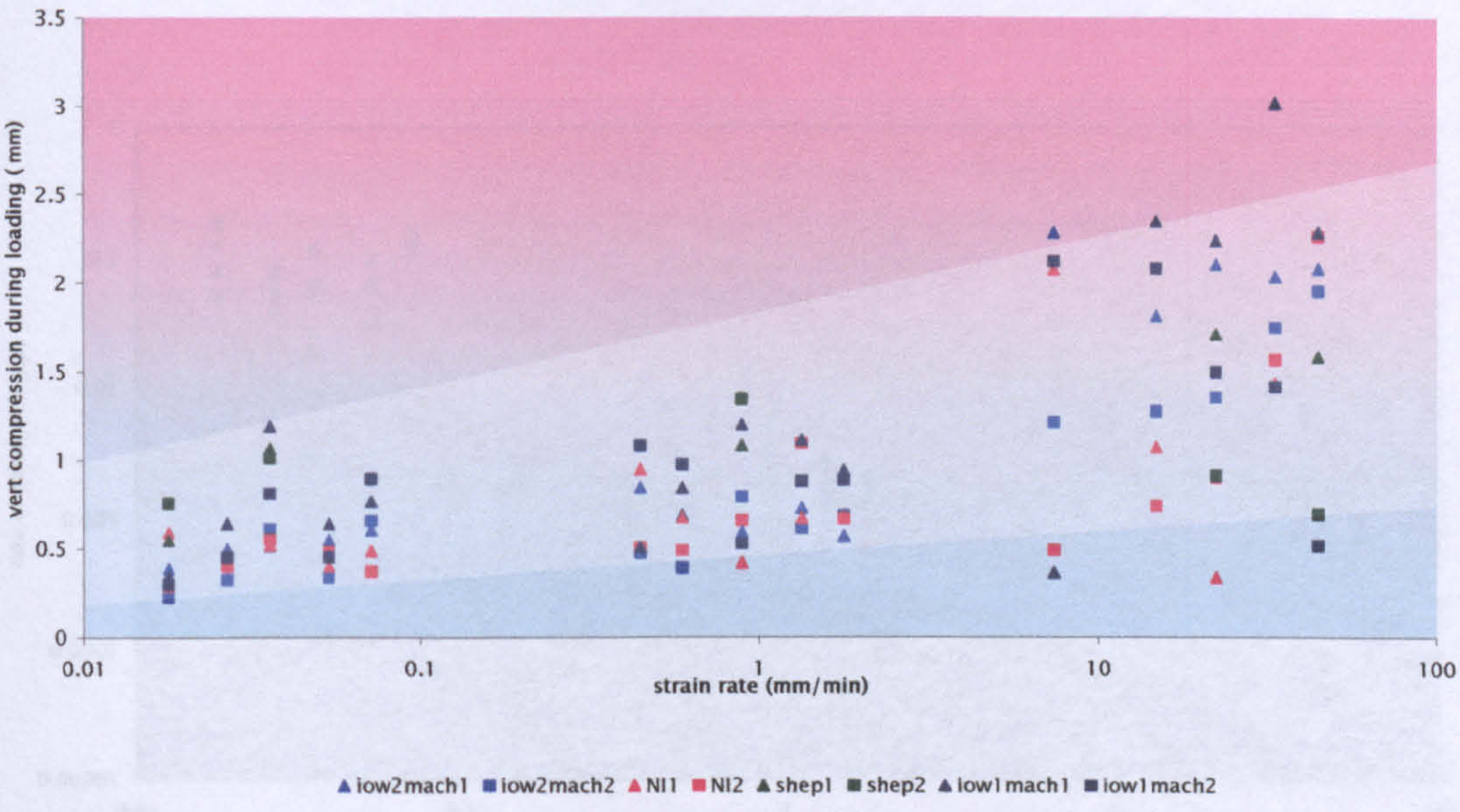


Figure 5.13: How rate of strain affects vertical compression

N.B Machine 1 - Rough Internal plate design

Machine 2 - Grooved Internal plate design

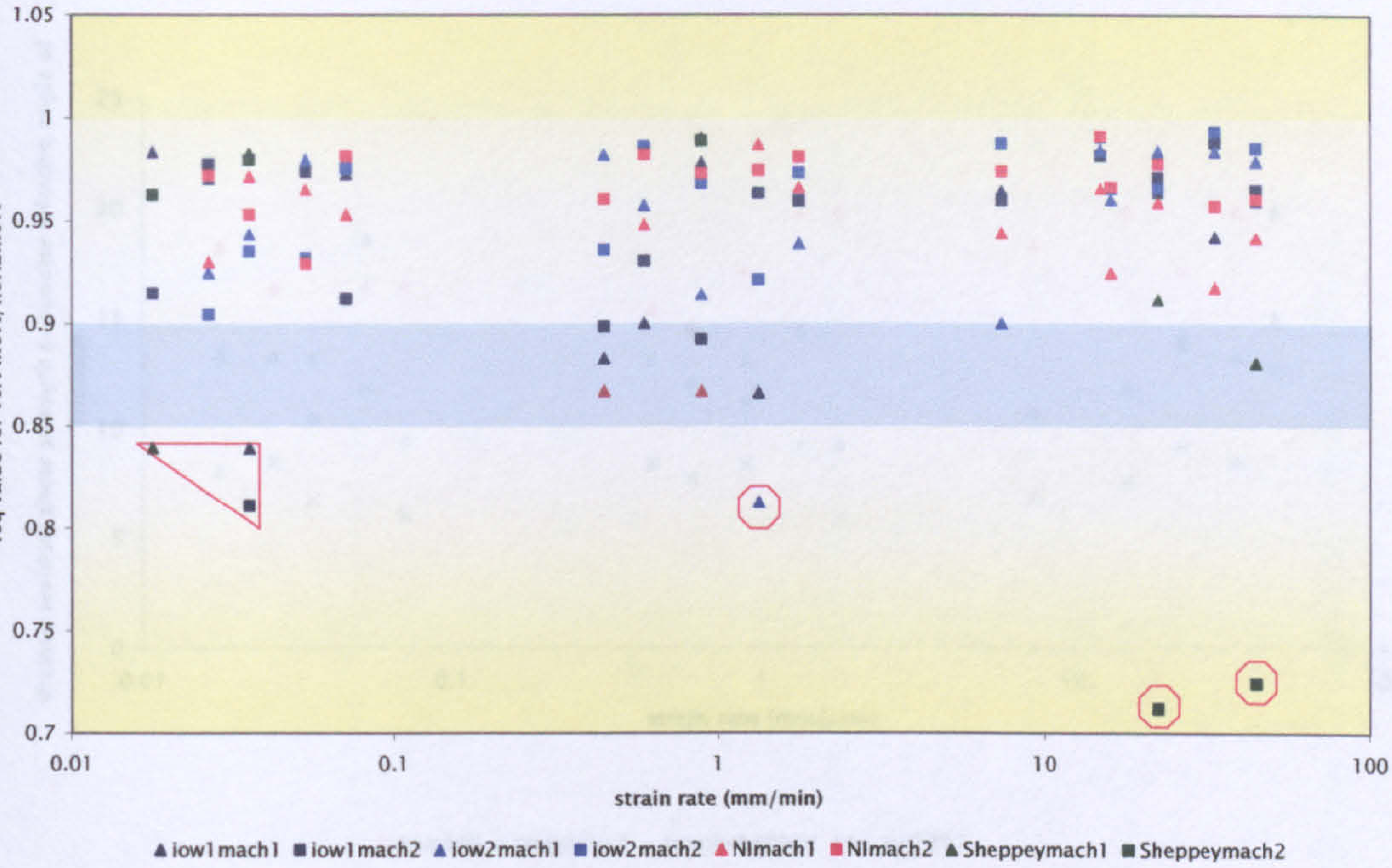


Figure 5.14: How rate of strain affects the linearity of the ratio of vertical compression to horizontal movement

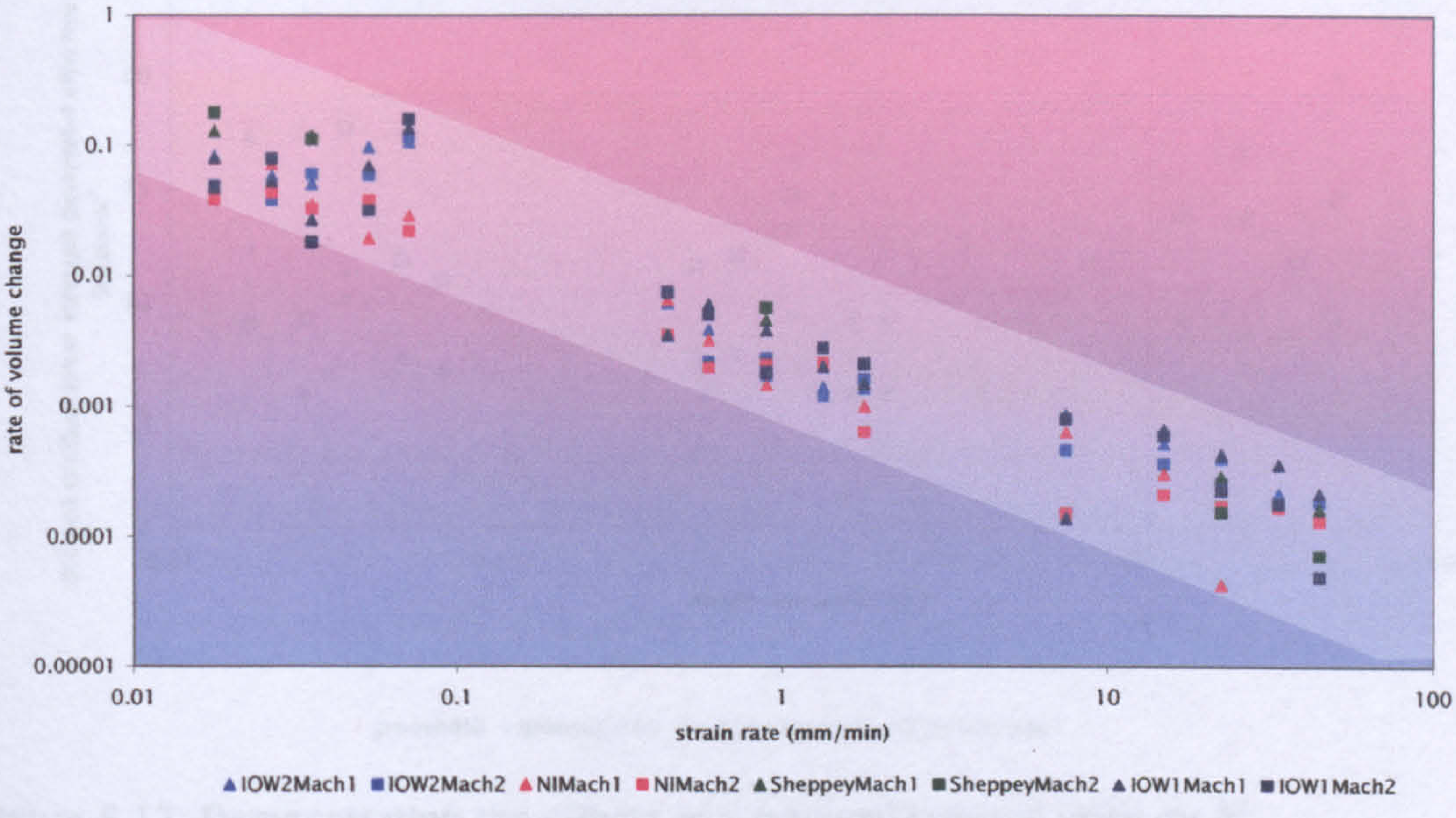


Figure 5.15: How rate of strain affects the ratio of vertical compression to horizontal movement

N.B Machine 1 – Rough Internal plate design
Machine 2 – Grooved Internal plate design

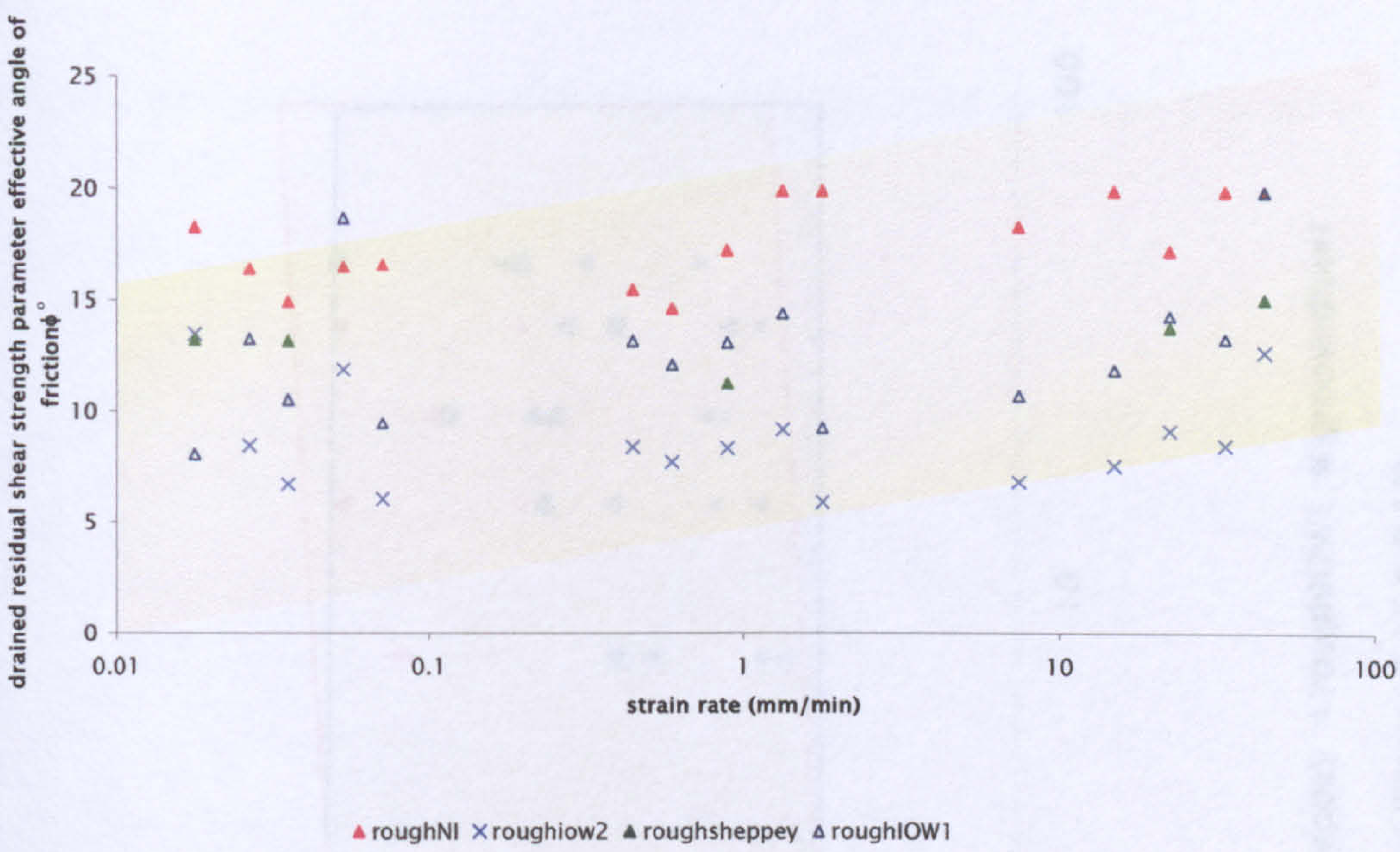


Figure 5.16: Demonstrating the effects of a rough internal plate on Φ'$_r$ incorporating clay plasticity

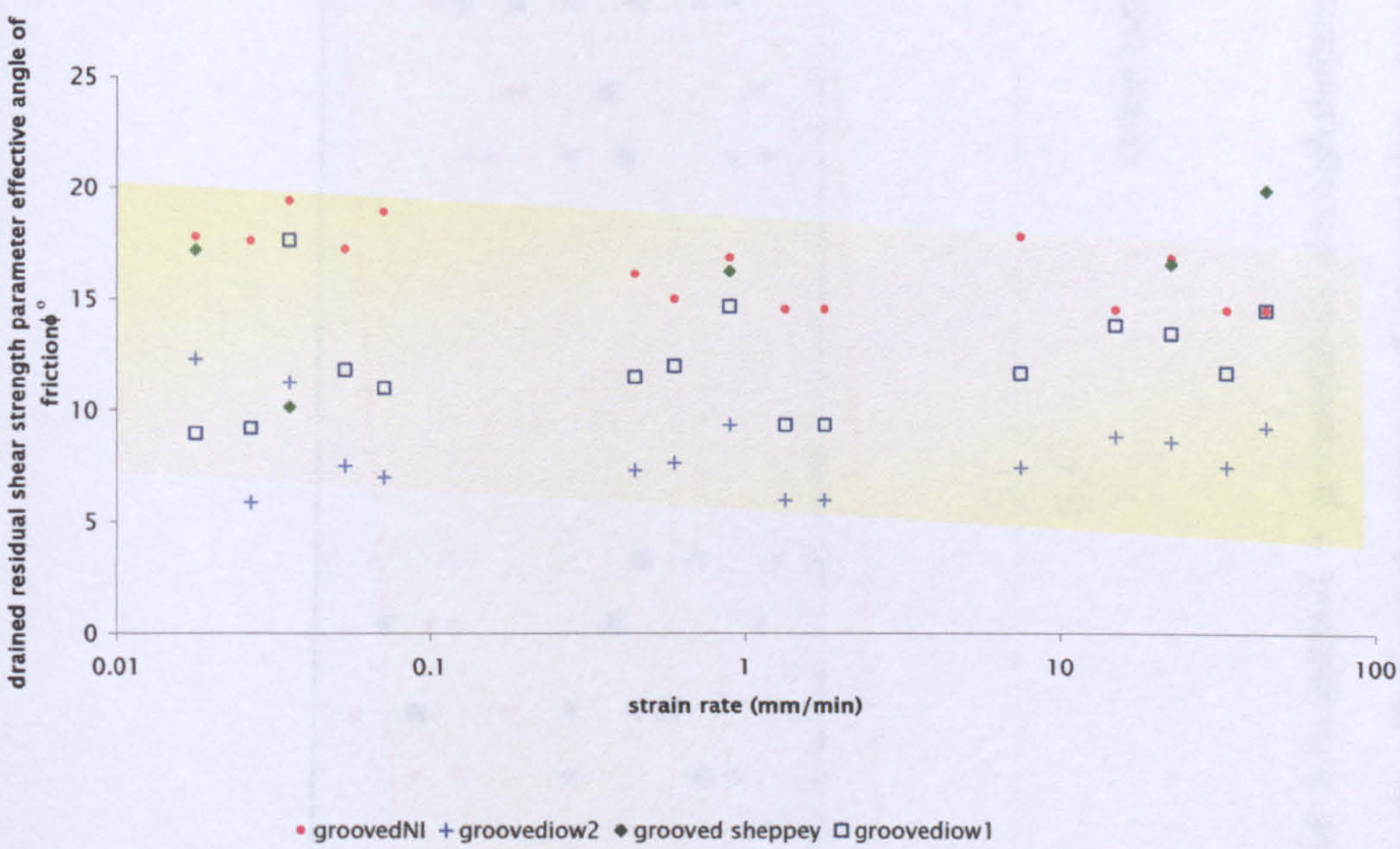


Figure 5.17: Demonstrating the effects of a grooved internal plate on Φ'$_r$ incorporating clay plasticity

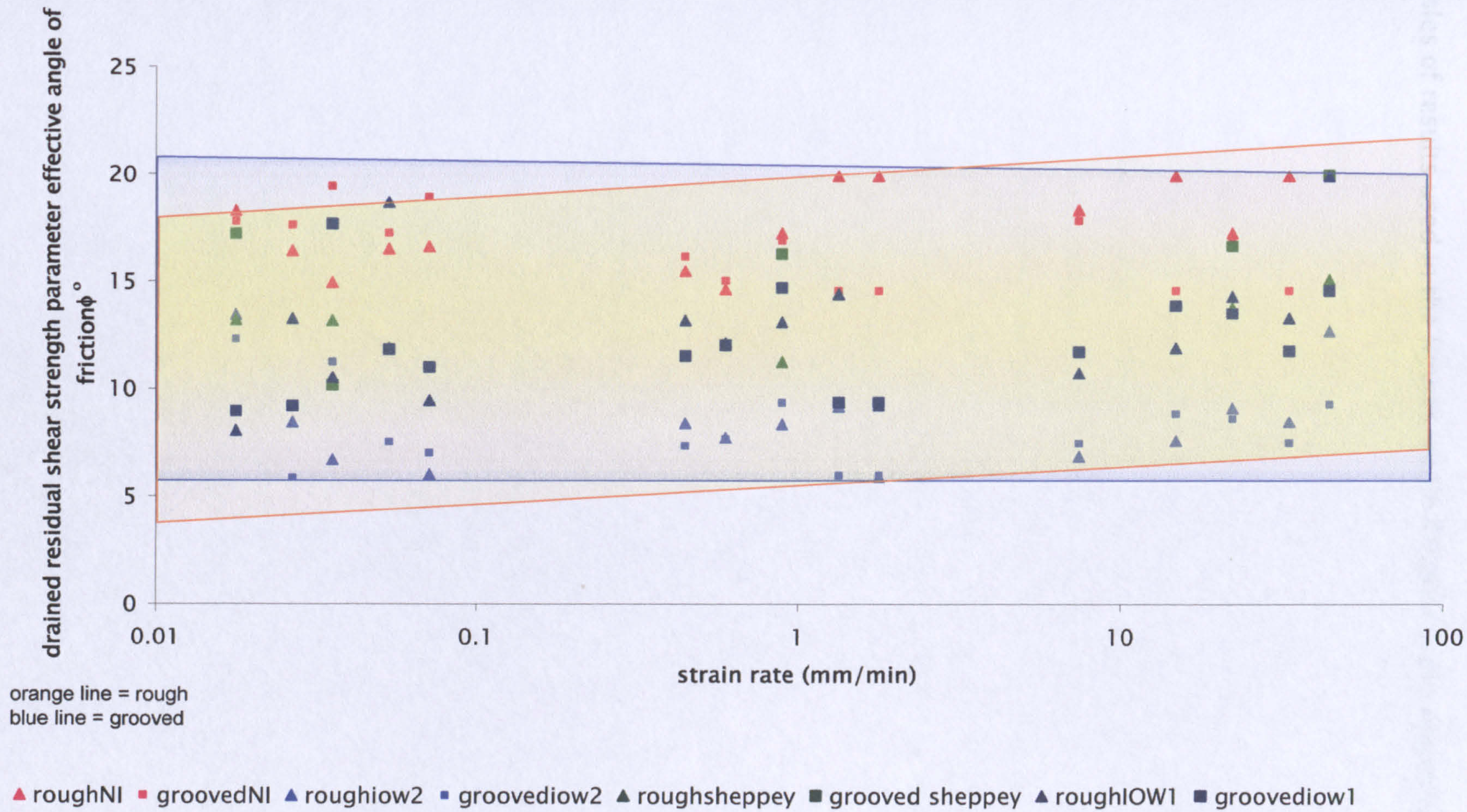


Figure 5.18: Demonstrating the effects of the pattern of internal plates (combined version of Figures 5.16 & 5.17)

For tables of results used in the figures within Chapter 5 see Appendix D

6 Investigations into the role of the application of normal stress

This chapter investigates the role of the application of normal stress in ring shear testing to determine residual shear strength. In general, normal stress can be applied either by weight loading or by air pressure. Air pressure can be applied with incremental increases, defined in this thesis; as stepped or discrete loading or at a continuous fixed rate of increase, defined in this thesis; as step-less or continuous loading. One of the Bromhead ring shears at Kingston has been modified by the manufacturers Wykham Farrence to facilitate pressure loading. An automatic pressure controller regulates the pressure to allow for step-less loading.

6.1 Background

Essentially, laboratory testing for residual shear strength can be performed using one of the two basic methods as described in Chapter 2. The first of these considers one specimen per normal stress on a natural slip surface but this is not investigated for this thesis. The other method involves one specimen per test carried out on an artificially formed slip surface with the normal stress applied at pre defined intervals over a set range.

The above are the two conventional approaches. The method which is investigated in this chapter was inspired by field observations and back analysis from the Warren Point Landslide, Sheppey on the Kent coast (Dixon & Bromhead, 2002; Bromhead, 2004b) and is an adaptation of the second method described above. Back analysis data plotted in Figure 6.1 suggests a straight line strength envelope for London Clay. Due to the process of back analysis a certain degree of homogeneity is assumed (see Chapter 7 for further explanation) nevertheless the data shown in Figure 6.1 does indicate that a landslide follows a path which it searches for. Observations and data from the landslide at Herne Bay (Bromhead, 1978) follow a similar pattern to the Warren Point landslide; thus reinforcing the indications from Warren Point that in the London clay stress envelope and the data from both landslides reveals a trend of seeking the stress path. Based on the principle of slope evolution a method of normal stress application was devised to replicate the stress path of the landslide.

In reality the Warren point landslide was observed to not only continue to move and rotate but to retain a constant head gradient, the top of the slope keeps a similar angle and thus preserves its shape. Movement was attributed to marine action eroding the toe of the landslide (Bromhead, 2004b) which when combined with the effects of the collapse of the rear scarp, increased the head loading. Essentially this process of erosion and collapse is the action which kept the system in equilibrium. In addition, surface mudslides eroded away some of the hill resulting in a "relatively constant overall slope" as Bromhead (2004) concludes. Figures 6.2 and 6.3 demonstrate the Warren Point and Herne Bay landslides ability to restore slope equilibrium.

When the field observations of slope equilibrium and other data from Warren Point and Herne Bay are linked it is apparent there is a set shape to which the landslide gravitates. If these movements are plotted they form a "stress path" which represents the average path of the entire landslide. It was from this that Bromhead (2004b) based his concept of step-less control for normal stress. Traditional systems have involved normal stress being applied at intervals which result in a stiff response for uploading and a gradual response for downloading as described in Section 3.2. Step-less loading by its basic definition does not involve sudden changes ensuring everything happens gradually. The aim of this system is to replicate the natural stress path for the landslide from which the sample comes, using a modified Bromhead Ring Shear with an air pressure system as shown in Figure 6.4.

There are many advantages to this less conventional approach to the application of normal stress. The more gradual loading system results in less disruption of the specimen and therefore reduced settlement and soil extrusion. By this method a greater number of points on the stress path are achieved and therefore a more detailed examination of the stress path is possible. This enables a detailed investigation of the stress path and the existence of residual cohesion. Stepped loading or conventional loading by the BS method involves rest period which according to Angeli et al. (2004) allow processes such as re-cementation to occur. Finally since both loading and unloading are gradual there is no difference in the behaviour of either as there are no dramatic changes in stress therefore it is possible to investigate the response solely due to strain without the added influence of stress changes.

This chapter describes principally the studies into replicating the stress path in both directions and the practicalities which surround it. Associated with these studies are a series of background issues which have been explored. Among these the difference between loading methods, variations in strain rate, the effect of plasticity index, the impact of the rate of step-less normal stress on post test moisture content and how soil displacement rates are affected by step-less loading. The samples used are from St. Catherine's, IOW; Warden Point, Sheppey and in some background investigations; from Dromore, Northern Ireland. Descriptions of the locations of these samples and of the samples themselves can be found in Section 1.3. The ring shears used are the modified Bromhead ring shear and one of the ordinary ring shears with the use of a VJtech automatic pressure controller and a Wykham Farrance data logger. The procedures for all investigations can be found in Chapter 4.

6.2 Normal Stress: method of application

At Kingston, the available ring shear apparatuses facilitate the use of pressure and weight loading during the testing procedure. This capability is available due to a specially adapted ring shear which was custom designed and built by Wykham Farrance during the early 1990's. For these investigations two Bromhead ring shears are used, which are identical except for the method of load application. Even the internal plates used within the sample holder are the same on these two particular ring shears. Figure 6.5 and 6.6 demonstrate the disparities in the ϕ'_r derived from the readings from these two apparatus. Using the sample from St Catherine's Point, Isle of Wight fifteen tests were performed on each machine over a range of strain rates between 0.017 mm/min and 44.52 mm/min. A smaller number of tests were performed over the same range of strain rates using samples from Dromore and Warden Point, Sheppey on the same machines as St Catherine's Point.

Generally the trends are the same for both machines although the apparatus adapted for pressure loading exhibited a tendency to produce marginally lower values of ϕ'_r. A more detailed examination of the specific results shows St. Catherine's Point sample to exhibit a sharp increase in ϕ'_r of 3°, followed by a more gradual increase in ϕ'_r of 1° over a range of five strain rates as depicted in Figure 6.5, (blue triangles) for conventional weight loading. Pressure loading for the same sample shows an increase in ϕ'_r of 1° , then a slight decrease in ϕ'_r of

0.5°, followed by an increase in ϕ'_r of 2°, then a final decrease in ϕ'_r of 1.5° as portrayed in Figure 6.5 (blue squares).

Comparing these two sets of results for St. Catherine's Point sample, for the different methods of load application shows that although the pressure loading exhibited less variation in ϕ'_r, normal weight loading produced a more consistent trend. The St. Catherine's Point sample was tested over a broader range of strain rates as demonstrated in Figure 6.6. By excluding the more extreme points it is possible to visualize trends of increasing in ϕ'_r with increasing strain rate. Although both pressure (squares) and weight (triangles) loading exhibit a large degree of variation within the results, pressure loading resulted in a smaller degree of scatter with a maximum of 9° minimum of 8°. Conventional weight loading in Figure 6.6 was observed to follow a more dispersed but still positive trend with increasing ϕ'_r with increasing strain rate. Pressure loading had a different response with a small degree of variation in ϕ'_r of less than 1° followed by a larger dispersion in ϕ'_r of greater than 3° and at the higher strain rates a variance in ϕ'_r of just 2°.

Dromore demonstrated similar trends for pressure (red squares) and normal weight (red triangles) load as exhibited in Figure 6.5. Analogous to St. Catherine's Point sample, slightly lower value of ϕ'_r were derived from the readings from pressure loading than for normal weight loading although the trends for both types of loading were negative, in other words Φ decreased with increasing strain rate. Closer examination of the normal weight loading (red triangles) for Dromore, shows that although there is an initial increase in ϕ'_r with strain rate of 1°, this is followed by a decrease of 3.5° in ϕ'_r over the other four strain rates which span from relatively slow speeds 0.036 mm/min to comparatively quick speeds of 45 mm/min. Dromore's pressure loading displayed greater variation as shown by the red solid line in Figure 6.5. The detailed trend is the same as normal weight load but the increase is 2° in ϕ'_r and then the decrease is in two parts, the first a gradual 2° in ϕ'_r followed by a more rapid 3° in ϕ'_r.

The last of the three samples to be tested produced the least consistent trends. Warden Point, Sheppey's sample, for normal weight loading (green triangles) displayed the same increasing ϕ'_r trend with strain rate as St. Catherine's Point, as shown in Figure 6.5, except there was a more rapid increase of 2° in ϕ'_r at the higher rates compared to the more gradual increase exhibited by the St. Catherine's sample. Pressure loading as shown by the green squares in Figure 6.5 produced no overall trend. Lower strain rates between 0.017 and 0.036 mm/min exhibited a decrease of 3° in ϕ'_r between them and then at the mid range rate of 0.9 mm/min, an increase of 4° in ϕ'_r occured. From mid range to the lowest of the high ranges at 22.5mm/min, there is a decrease in ϕ'_r of 4.5° but between 22.5 and 45 mm/min there is an increase of 1.5° in ϕ'_r. It may be possible to conclude that with the lower strain rates exhibiting higher degrees of ϕ'_r than the higher strain rates an interpretation of a trend of decreasing ϕ'_r with increasing strain rate; which would be the opposite trend to the normal weight loading is plausible for pressure loading.

Overall it is noted that higher values of the shear strength parameter ϕ'_r were achieved using pressure loading compared to conventional weight loading. This is perhaps a consequence of the greater uniformity of the normal stress when using pressure and a greater distribution between the clay particles.

6.3 Discontinuous strain rate with pressure loading

Varying the strain rate mid test was investigated within Chapter 5 Section 3 using the traditional weight loaded apparatuses. The effect of the application of the normal stress using pressure on this procedure is demonstrated in Figure 6.7. St. Catherine's, Dromore and Sheppey samples were all used and the investigations carried out in accordance with the procedure outlined in Chapter 4. A reduced range of strain rates was used, with only the following utilized, 0.018, 0.036, 0.8904, 22.26 and 44.52 mm/min. Validity was ensured by using three different samples. In total, twelve tests where the strain rate was changed mid-test post the consolidation stage per sample; plus the five control tests where the strain rate remained at a constant rate throughout the test.

From Figure 6.7 the sample from Dromore, (red coloured graph) generally when compared to a sample from the same location at a constant strain rate (black squares) in other words when the strain rate remains unchanged for the duration of the test, displayed an increase in ϕ'_r if the strain rate was increased (red triangles) and a decrease if the strain rate was reduced (blue diamonds). However, this general trend did not always occur as there are specific points where the opposite is true in three difference circumstances as demonstrated by Figure 6.7. The results using the sample from Warden Point, Sheppey, (green coloured graph) when compared to the black squares, representing the constant strain rate results, exhibited in general an increase in ϕ'_r whether increased (red triangles) or decreased (blue diamonds) to the extremes but a decrease in ϕ'_r when increased or decreased to mid range. On three occasions the opposite occurs as shown in Figure 6.7. A sample from St. Catherine's Point, Isle of Wight , (blue coloured graph) was also compared to its black squares, representing the constant strain rate results and the red triangles show where the strain rate is increased from a lower rate to higher rate and the blue diamonds indicate the opposite. A general trend of an increase in ϕ'_r when the strain rate is reduced from high to low but a decrease if strain rate goes from low to mid range, mid range to high or low to high with only two exceptions to this trend as shown in Figure 6.7.

Overall the trends observed are different to those observed in Chapter 5 Section 3 but still indicate that a different result is achieved by altering the strain rate mid test and consequently giving a different value of ϕ'_r than would have been achieved had the rate been constant.

6.4 Discrete and Continuous loading

Conventional procedure involves taking discrete points on the stress path and analysing the results to assume values of shear strength parameters. Maksimovic (1989, 1996) argues that the shear stress envelope is curved not linear. Skempton (1985) noted the curved relationship but said that the degree of curvature varied and therefore the most suitable method was linear analysis. These investigations were carried out using one of the Bromhead ring shears within Kingston Geotechnics research with procedures which are described in Chapter 4. An automatic pressure controller, manufactured by V.J. Tech Ltd., a datalogger by Wykham Farrance and a normal manual pressure regulator were utilized during these investigations. Figure 6.8 and 6.9 demonstrate the differences between discrete and continuous loading. The apparatus used has been specifically adapted by Wykham Farrance to enable pressurized loading as part of an EPSRC research grant during the late 1990's. For these investigations samples from Warden Point, Sheppey, where the idea for step-less loading came from according to Bromhead 2004(b) and St. Catherine's Point, Isle of Wight were used. Overall three tests at different rates of loading and unloading were carried out using clay from Warden Point to ascertain which rate tracks the stress path best. The same system was used on a sample from St Catherine's Point for two tests. The results discussed reflect the rate of loading which best follows the stress path for each sample within this section and compared to a loading process which is at discrete intervals. All tests are discussed in Section 6.5. In addition, a control test using discrete intervals was performed for both samples. All tests were run at a strain rate of 0.036 mm/min.

The sample from Warden Point, Sheppey exhibited a steeper gradient with continuous loading (black triangles) than with discrete loading (purple squares). The envelope was shown to be disjointed on the upward ramp in Figure 6.8 but more linear on the downward ramp. Both discrete and continuous loading gave slightly higher values for the initial shear stress on the return trip i.e. when normal stress was being unloaded. Discrete loading produced a linear relationship although this takes account of only a fraction of the actual stress path. There is a large jump of 15 KN/m^2 in shear stress when approximately 300 KN/m^2 normal stress is being applied, which occurs at thirty three and a third hours into the ramp up of the pressure over a short duration of just sixteen and a half minutes. Actual applied pressure, which is monitored throughout the test, was regular and had no notable variations for the entire duration of the test. Therefore the cause may be where the shear-zone has been worn away and a new one is formed. Continuous loading as exhibited in Figure 6.8 produced higher values of shear stress for the same normal stress than discrete loading. Warden Point, Sheppey demonstrated a slight degree of curvature but was more linear on the downwards ramp of pressure than the upwards.

St. Catherine's Point, Isle of Wight sample exhibited greater agreement between the discrete (black plus signs) and continuous loading (blue diamonds) patterns as demonstrated in Figure 6.9. The upward path follows a more disjointed trend than the downward which is virtually linear with only a slight bend in the middle region at approxiamately 300 KN/m^2 normal stress. The upward trend curves more and initially displays more shear stress for the same normal stress by 15 KN/m^2 compared to the downward trend. The path between 250 KN/m^2 is the same or almost the same for upward and downward using St. Catherine's and is less than 10 KN/m^2 higher than the discrete loading line as exhibited in Figure 6.9.

Figures 6.8 and 6.9 indicate that the higher the plasticity index the more gradual the rate at which the upward and downward loading needs to be. The lower the plasticity index according to Figures 6.8 and 6.9 the closer the continuous path relates to the discrete line.

6.5 Rate of loading

The basis of this technique is the cycle of landsliding and erosion which is described in Dixon & Bromhead (2002) using Warden Point as example. A natural cycle occurs where there is a landslide which slowly creeps downwards, then mudslides and shallow creep deposits seep forward until a central mudslide is formed which is then eroded away by marine erosion thus restoring equilibrium. By using an automatic pressure controller it is possible to analyze how the clay reacts to small infinite changes in normal stress and thus gain a clear understanding of the natural processes. For these investigations the specially adapted ring shear was used, along with the datalogger and the automatic pressure controller. Tests were carried out at two rates of decline on both samples and a week long test using Sheppey. St. Catherine's Point performed better on the first two with closer agreement with the upward and downward paths.

St. Catherine's Point, as demonstrated by the red triangles on Figure 6.10, showed a very curved envelope for 400 KN/m^2 in twenty-four hours up (17 kPa/hr) and six hours down (67 kPa/hr), the downward path going above and below the upward path and sort of interlacing with it but not quite following the same path. Blue diamonds on Figure 6.10 demonstrate a twenty-four hour ramp up and twelve hour ramp down (34 kPa/hr) over 400 KN/m^2. This relationship is more linear and closely correlates with the discrete loading for St. Catherine's Point, Isle of Wight.

Warden Point, Sheppey; three rates were utilized in addition to the two rates used with St. Catherine's, a rate of 6 kPa/hr up and 5 kPa/hr down which are the grey triangles in Figure 6.11. Blue diamonds represent the twelve hour decline (17 kPa/hr up and 34 kPa/hr down) and red circles the fastest of the three which involve the mere six hour ramp down (17 kPa/hr up and 67 kPa/hr down). The twelve hour ramp down and the slowest rate test displayed the closest agreement indicating similar paths used. At six up five down for this slower test indicated a steeper path and gave rise to slightly higher values of shear stress when compared to the 17 up 34 down test. In comparison, the fastest test took a more gradual path than the other two as demonstrated by Figure 6.11. The relationship between the discrete tests and the continuous tests indicates that the shear stresses are higher with the latter than the former possibly due to the ability for more gradual movement of pore pressures under these conditions. Warden Point, Sheppey resulted in a less smooth path when compared to St. Catherine's and exhibited less correlation with its discrete line indicating a completely different path is followed for continuous loading.

Detailed examination of these paths demonstrated in Figure 6.11 show the 17/34 and 6/5 to reach the same values at either end of the path. 17/67 and 17/34 were noted, using Sheppey, to have various points which miss the actual line, indicating the limits of the proving rings to respond quickly enough to the changes. Larger intervals between readings for the 6/5 path enabled more time for the machine to adjust and overall, even though the machine was running at the strain rate, more time for the sample to respond to the load. Warden Point, Sheppey responded the best to this more gradual approach. St. Catherine's responded best to the 17/34 where, although the decline is quicker than the upward rate, there is enough time for the sample to respond. As described in Chapter 3, declines usually take longer than going up the stress path but with continuous unloading, the path is followed even when it is quicker for St. Catherine's Point, indicating that perhaps there was enough time for the pore pressures to dissipate.

Figure 6.12 and 6.13 compare and contrast the different responses the samples gave to the loading and unloading rates. The sample from Warden Point, Sheppey (shown as diamonds for continuous and squares for discrete in both Figure 6.12 and 6.13) did not relate with either rate to the discrete path to which it is compared. St. Catherine's Point sample (shown as triangles for continuous and plus signs for discrete in both Figure 6.12 and 6.13) does not respond well to 17/67 but closely correlates to its discrete path with 17/34; all indicate a slightly curved stress path but to different degrees. Continuous loading mostly produces higher shear stress values due to its facility to allow for the pore pressures to dissipate thus giving a more "effective" stress test. This system relates better to the natural process of regaining equilibrium and thus is more representative of the real world. Natural processes seldom involve sudden changes in normal stress therefore continuous is better than discrete. In conclusion the role of plasticity seems to govern the rate at which the perfect stress path is achieved as demonstrated by figures 6.10, 6.11, 6.12 and 6.13 and also the rate at which the normal stress can be removed and still follow the same path as the uploading.

6.6 The effect of rate continuous loading and unloading of normal stress on settlement and post-test moisture content.

Continuous loading at various rates and their effects on settlement and post test moisture content were monitored using the methods described within Chapter 4. These investigations were carried out using a sample from Warden Point, Isle of Sheppey although as a comparison, some tests were carried out on a sample from St. Catherine's Point.

Initially investigations into the length of the unloading process were undertaken. Chapter 3 notes different methods of unloading procedure and it was observed that the shear stress took longer to return to its initial value if the tension was not removed from the proving rings. The effect on settlement shown in Figure 6.14 and 6.15 indicates how the faster rate displays not only a steeper gradient during the unloading process but also it is more stepped than at the slower rates. Loading of normal stress was achieved for these tests at the same rate and they display analogous gradients but the faster rate test displays more settlement during the formation of the shear zone than the lower rate of loading. This is due to the non homogeneous nature of the clay. Any scatter is a result of the rate of unloading combined with the data logger logging at five minute intervals which causes some anomalous readings as a result of the linear transducer not having time to stabilize. As a comparison the tests were rerun using a sample from St. Catherine's Point, Isle of Wight. Figure 6.16 demonstrates an identical settlement path for both Sheppey and Isle of Wight. Even the scatter appears in the same locations indicating that the cause must be consistent for both tests. The largest test was run over a period of 8 days and involved a very slow ramp up of the normal stress and an even slower ramp down. At a third of the rate of the previous tests loading of normal stress this gave a more gradual settlement curve. Unloading at a rate of one kPa per hour less than loading resulted in a fairly flat settlement curve with little or no dilation. Previous tests at 67 kPa/hr and 34kPa/hr unloading rate produced up to 0.25mm of dilation; this by comparison produced less than 0.125mm.

Post-test moisture content was investigated for all the five tests undertaken as part of this research. Sheppey and Isle of Wight displayed different trends. Table 6.1 demonstrates, with reference to the initial moisture content, how the different clays with their respective plasticity's produce dissimilar responses. The London clay sample from Warden Point, Sheppey with the three tests performed indicated a trend of increasing post test moisture content to a point, in this case a twelve hour ramp down at 34kPa/hr. This involved an increase from fourteen percent and forty-three percent for a six hour ramp down at 67kPa/hr; then an increase from fourteen percent to fifty-nine percent for a twelve hour ramp down at 34kPa/hr. This trend displayed a decline on the 6kPa up 5kPa down test when for an increase from fourteen percent to only forty-six percent was observed, all of which though increased the moisture content to a minimum of six percent above the plastic limit. Conversely, the gault clay sample from St. Catherine's Point displayed a different trend. At thirty -six percent the sample was sixteen percent above the plastic limit. The higher rate of decrease, 67kPa/hr resulted in an increase from thirty-six percent to forty-six percent, the lower rate produced a decrease of four percent thus indicating that for this type of clay the trend is for an increase in moisture content post-test. Subject to further investigation this would suggest that dependent on the plasticity index, the pore pressures within the sample more or less dependent on the rate of normal stress removal and possibly of application.

In conclusion, the limited investigations have revealed that the settlement and the post-test moisture content are affected by the rate of application and removal of normal stress.

6.7 Conclusion

Methodology of application of normal stress if via continuous ramped loading removes the problems caused by rest periods as per BS test therefore producing a more accurate result. This particular type of normal stress loading procedure facilitates the possibility of replication of the stress path thus enabling a closer examination of the existence of residual cohesion which this research suggests exists. The extremes of settlement and post test moisture content are reduced by continuous loading but are affected by rate of unloading. Care must be taken to do this slow enough to replicate the stress path. Variable loading rates using pressure produce analogous trends to conventional loading as expected.

Clay Sample	Pressure up	Pressure down	Plastic Limit	Liquid Limit	Initial Moisture Content	Post Test Moisture Content
Gault	17	67	20.1	60.5	36	46
		34				32
London	17	67	36.7	86	14	43
		34				59
	6	5				46

Table 6.1 - Post Test Moisture Contents after a step-less normal stress ring shear test

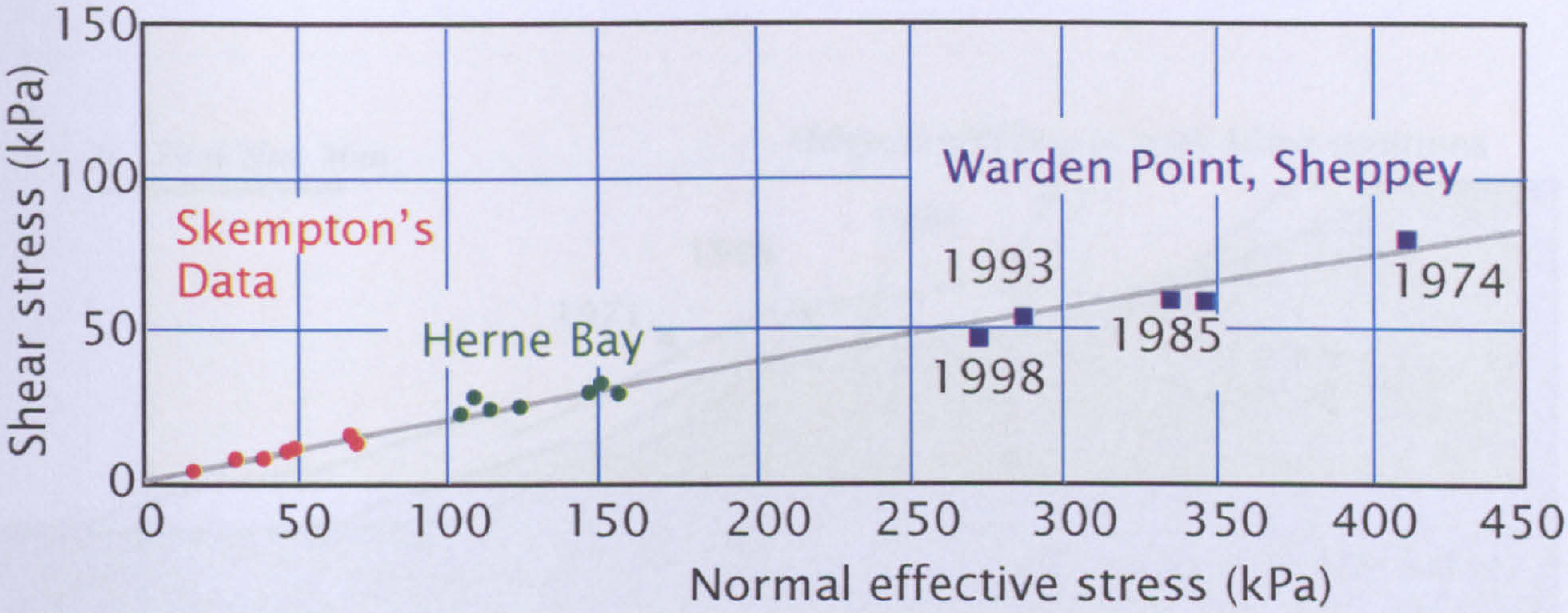


Figure 6.1 Shear strength envelope for London Clay based on back analyses.

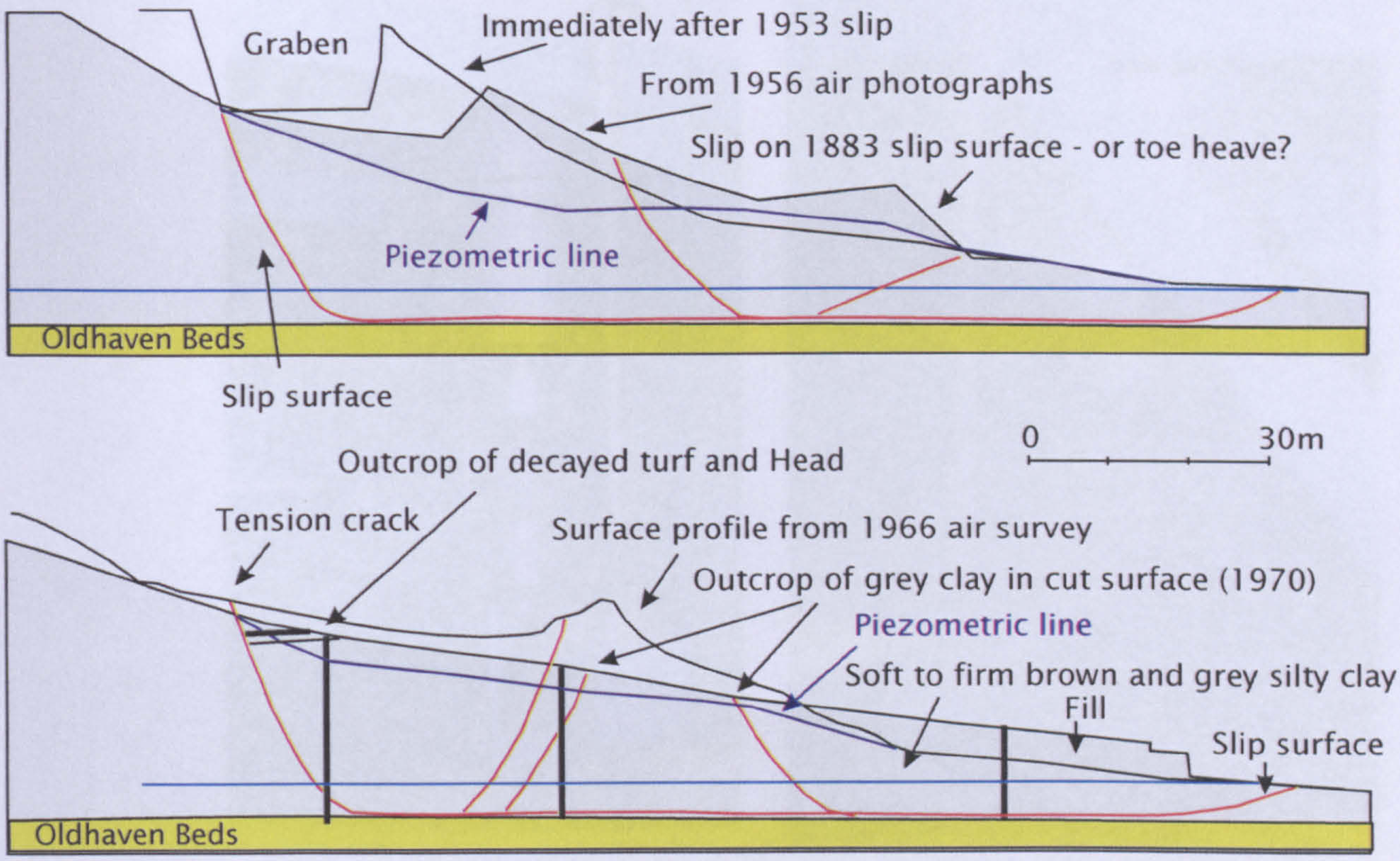


Figure 6.2 Evolution of cross section of the Miramar landslide, Herne Bay, 1953-1970. (After Bromhead, 1978)

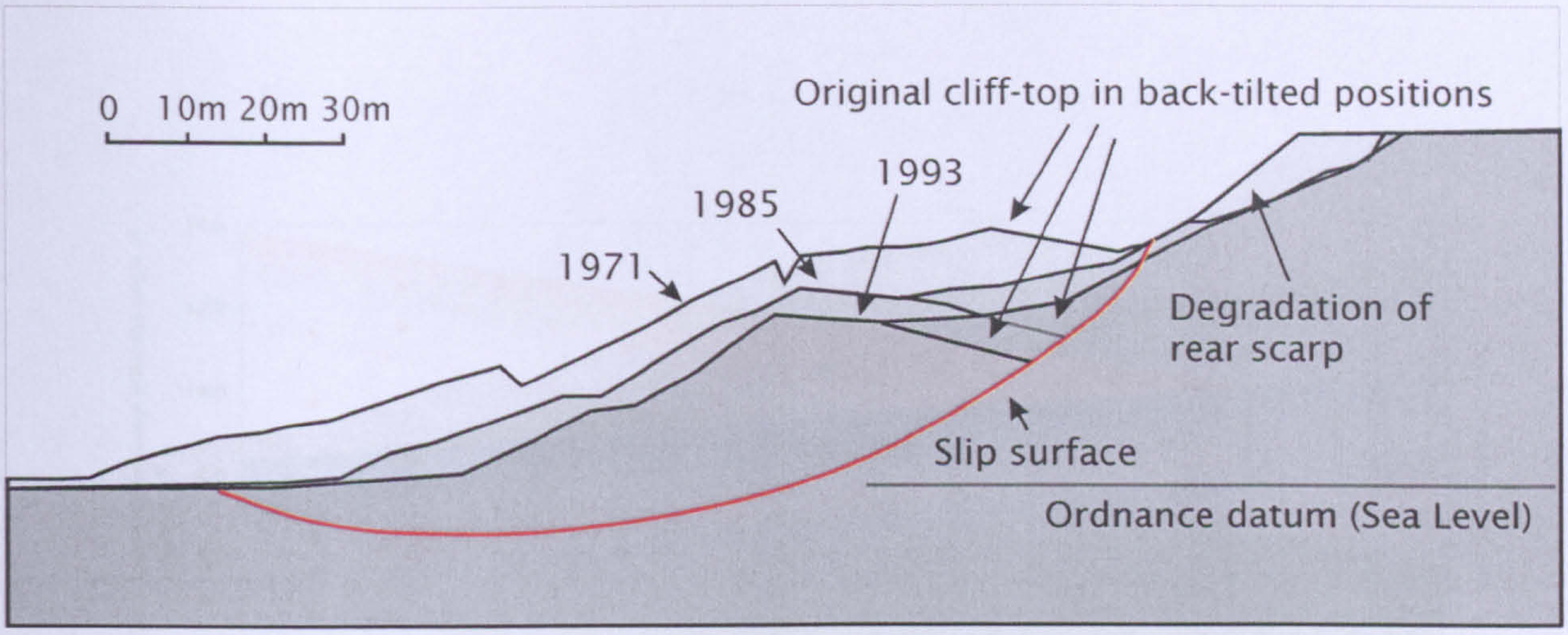


Figure 6.3 Evolution of cross section of Warden Point landslide 1971-1993, (After Dixon & Bromhead, 2002).

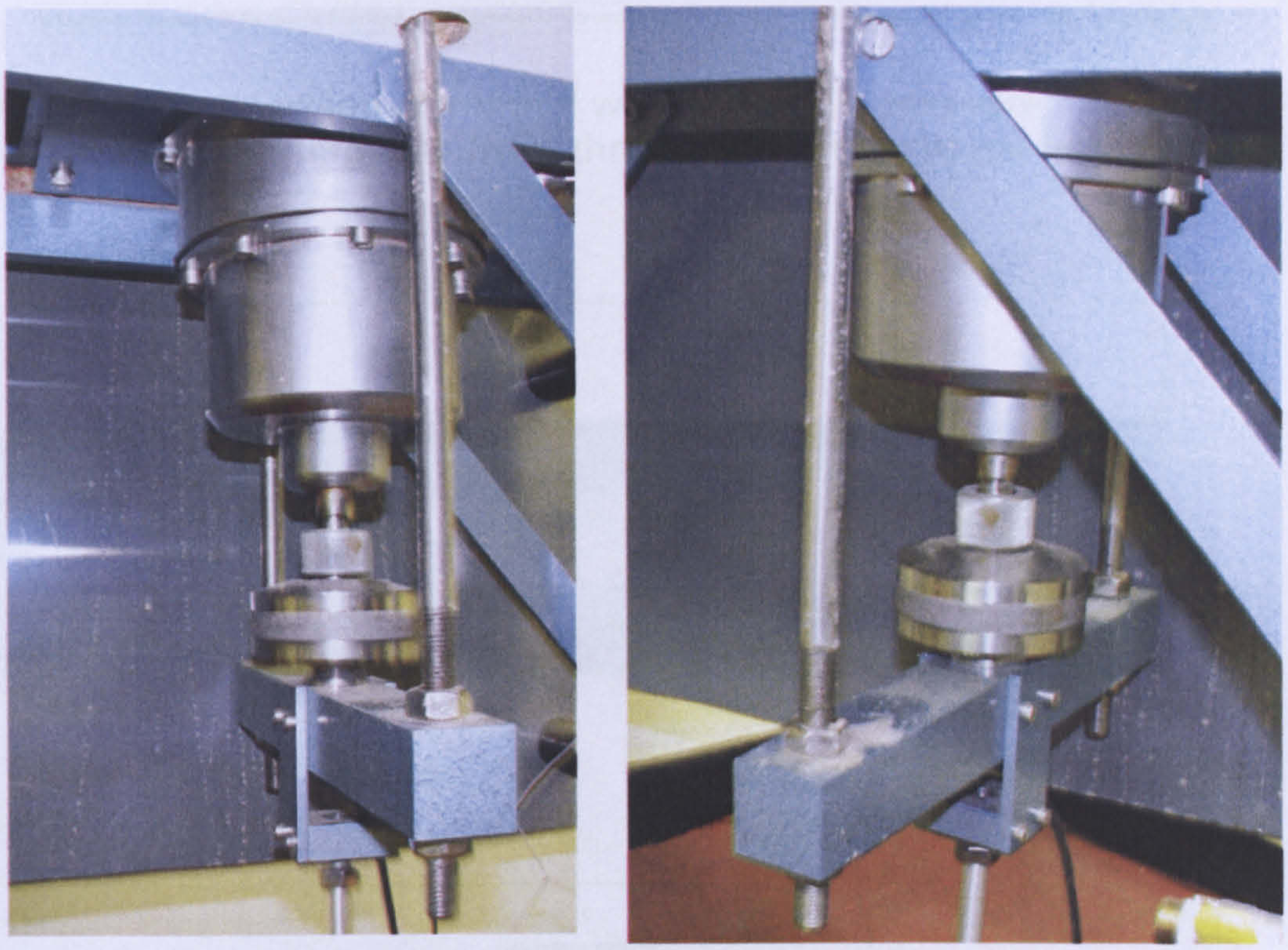


Figure 6.4 Modifications to normal load control, Bromhead ring shear.

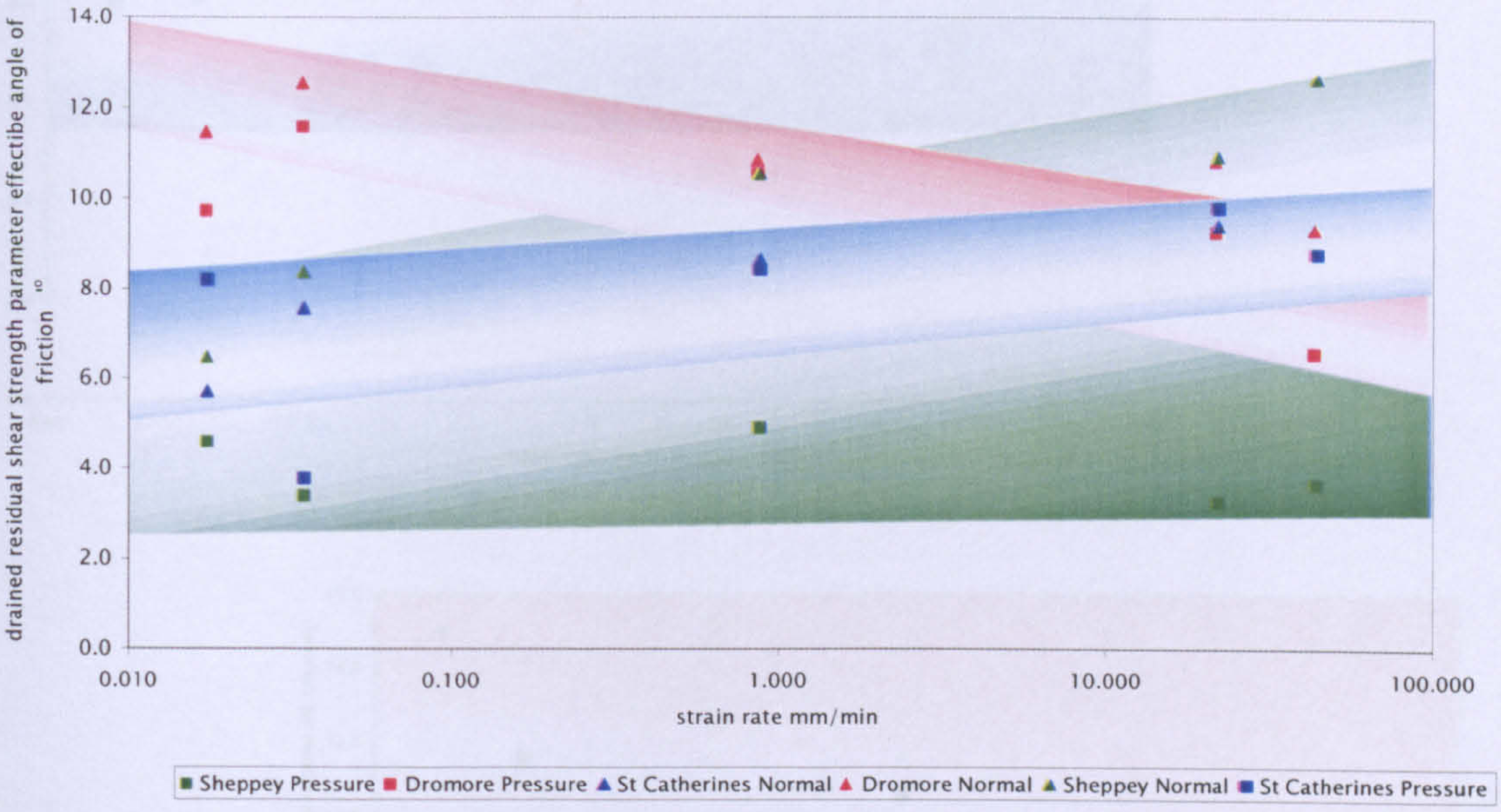


Figure 6.5: A comparison of normal weight and air pressure loading using clay from three different locations in the UK

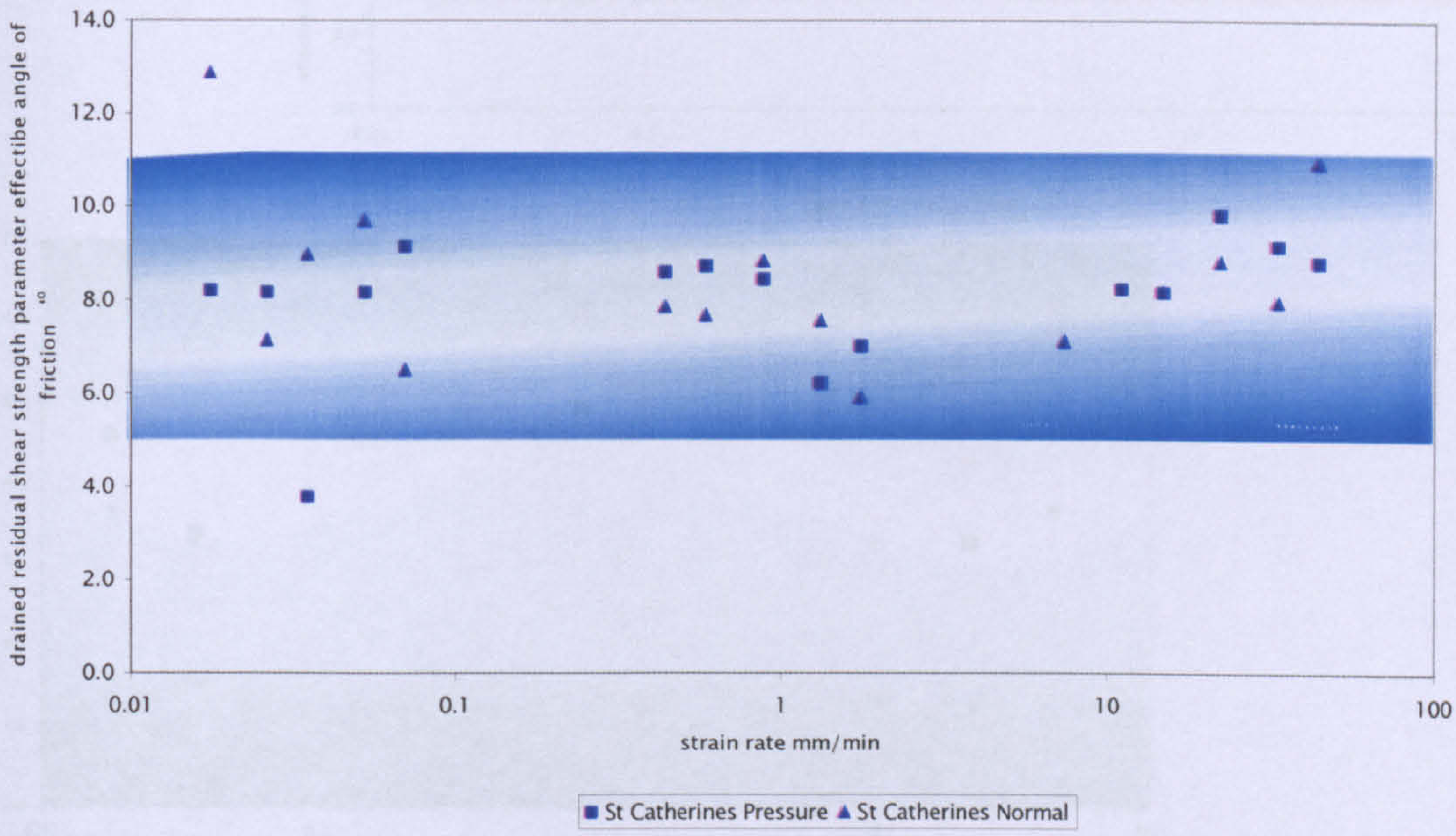


Figure 6.6: A Detailed Direct comparison of normal weight and air pressure loading using clay from St Catherine's Point, I.O.W.

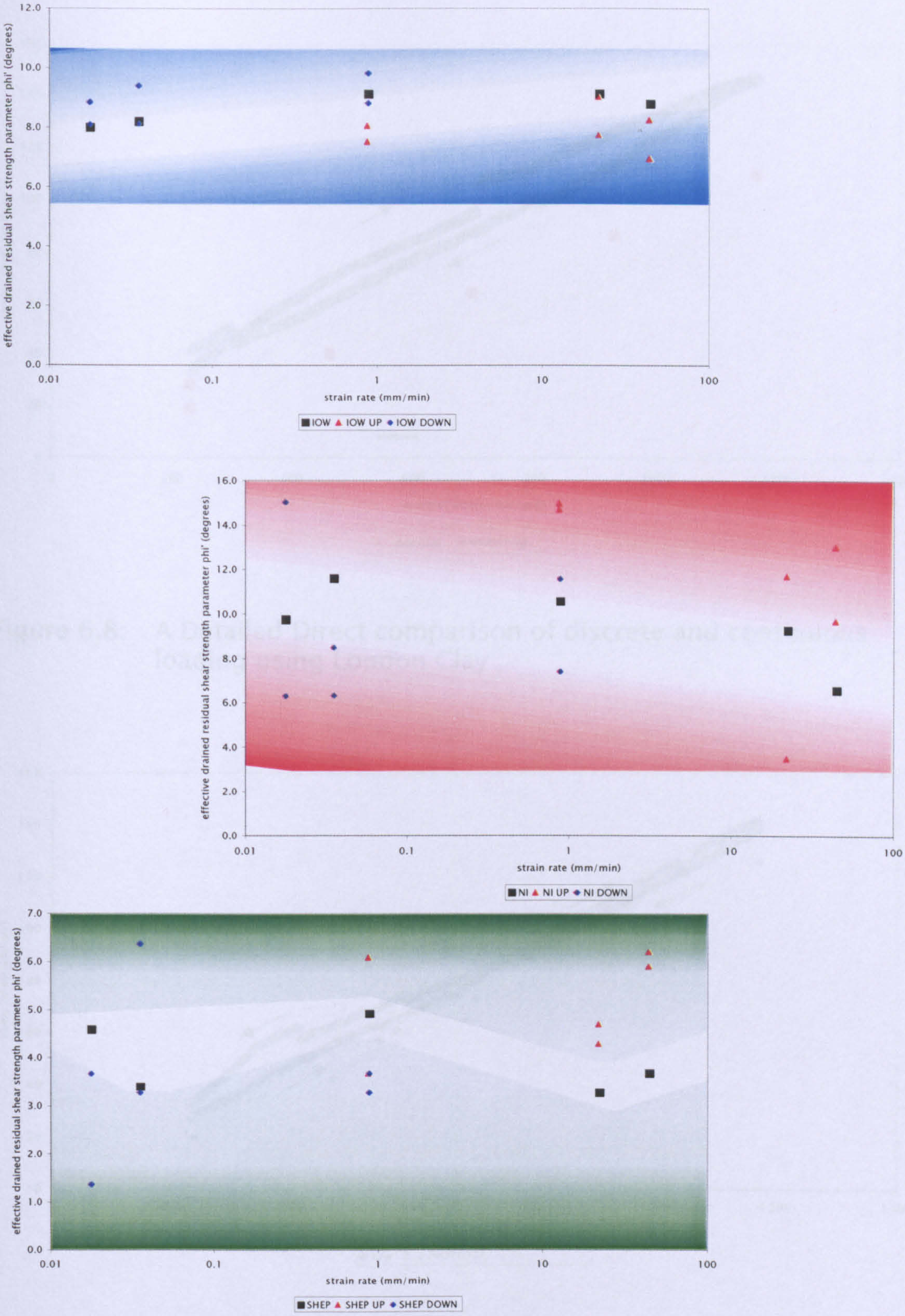


Figure 6.7: A comparison between continuous and discontinuous strain rate

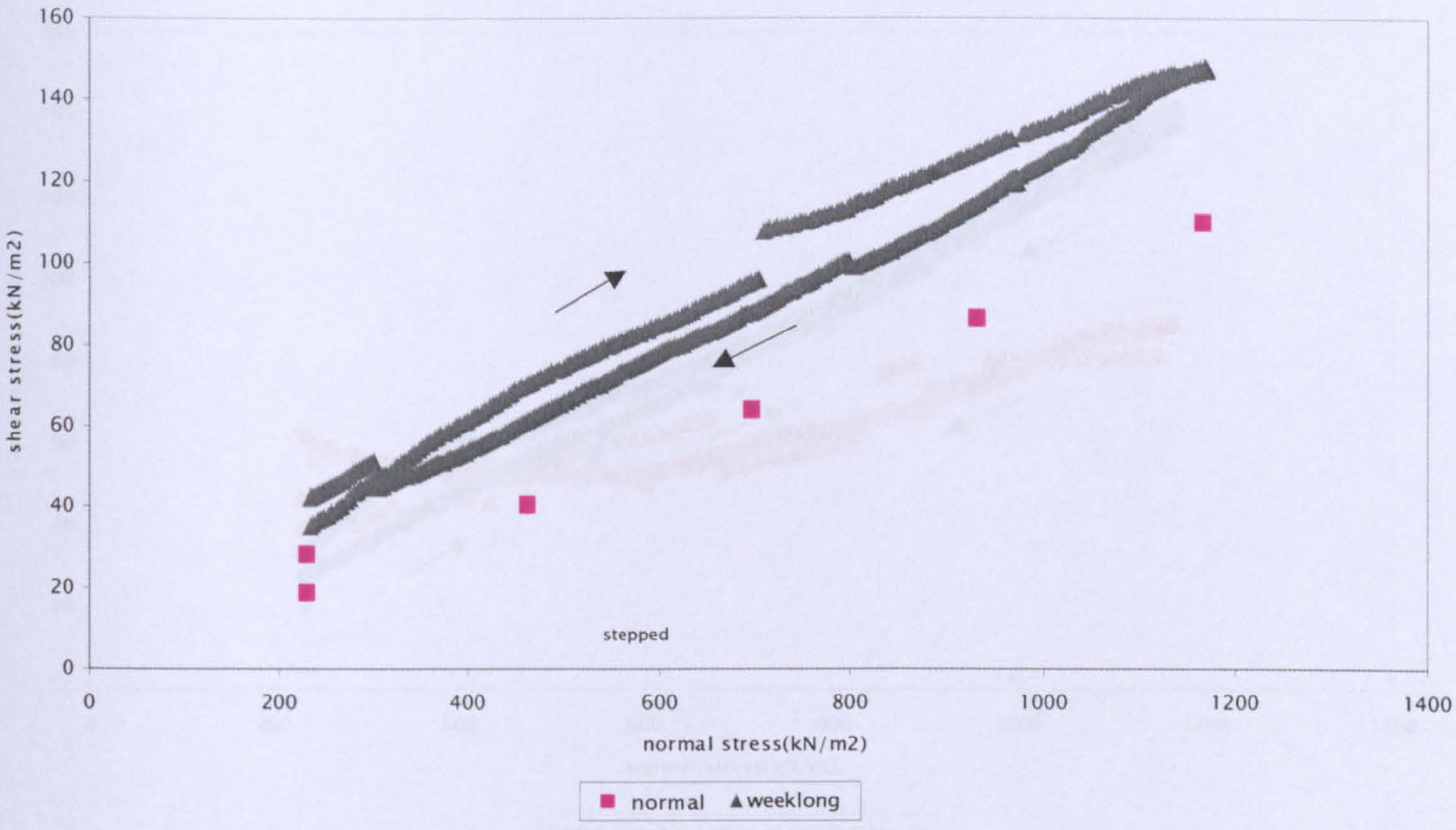


Figure 6.8: A Detailed Direct comparison of discrete and continuous loading using London Clay

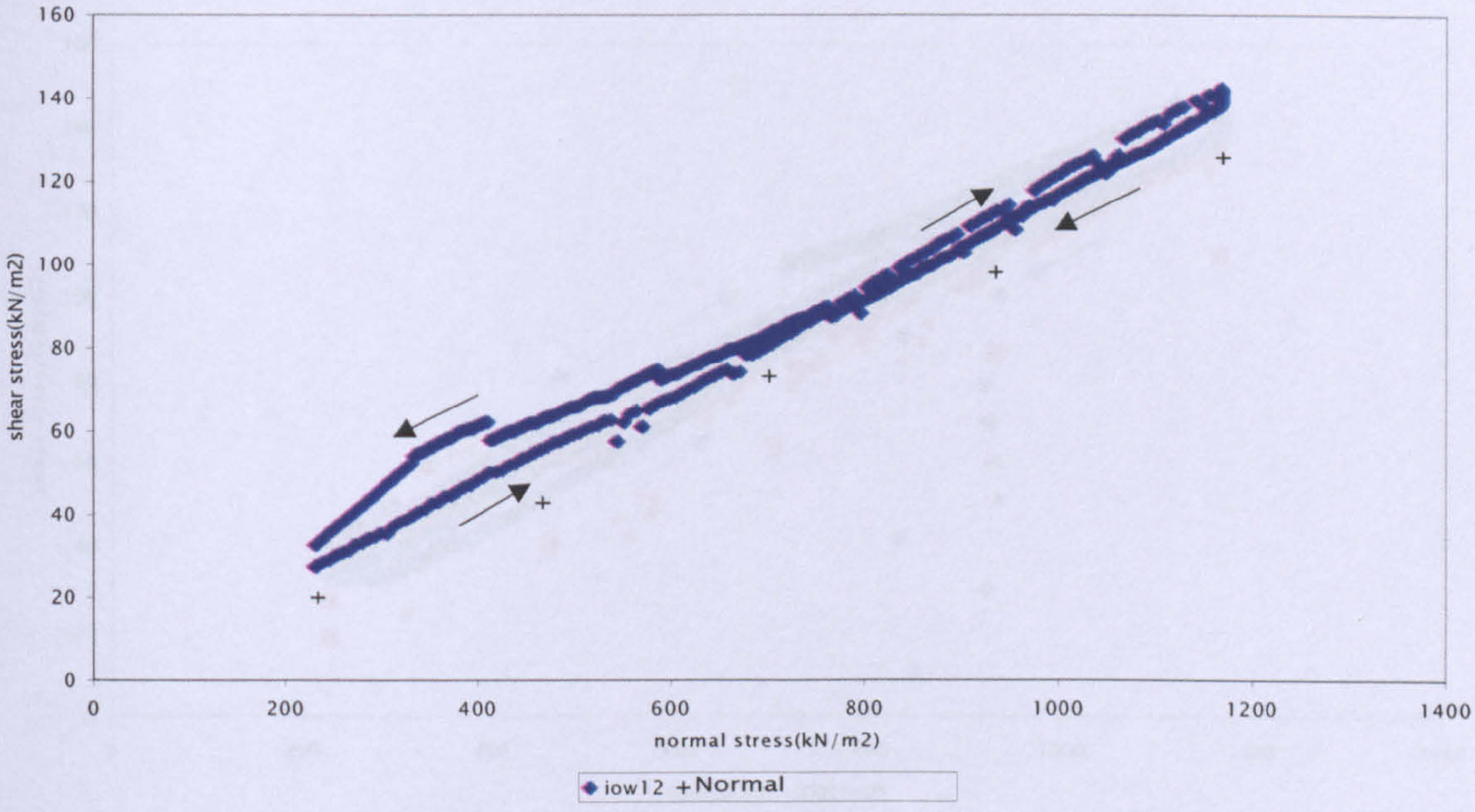


Figure 6.9: A Detailed Direct comparison of discrete and continuous loading using Gault Clay

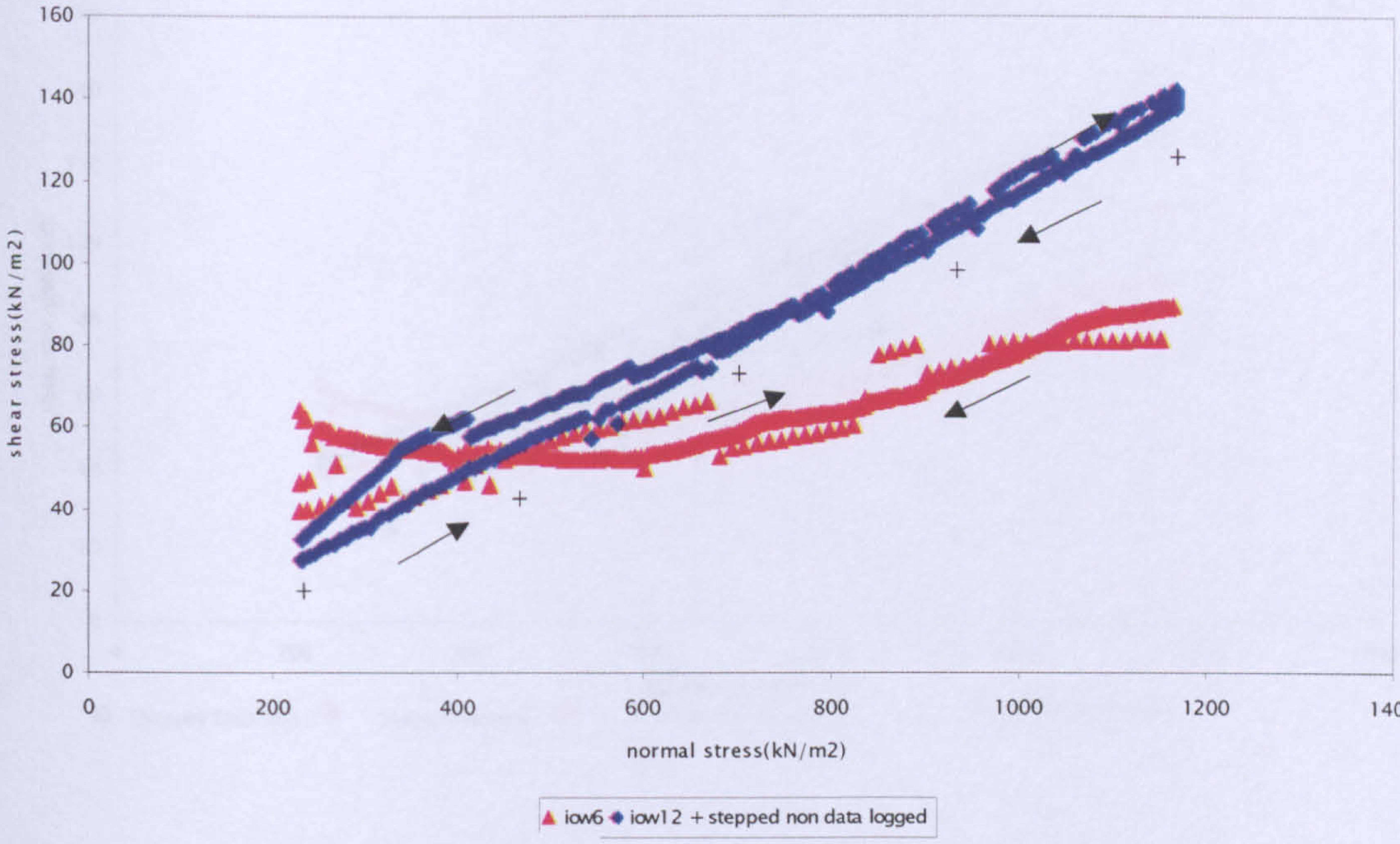


Figure 6.10: Direct comparison of discrete and continuous loading using Gault Clay exhibiting all rates of loading

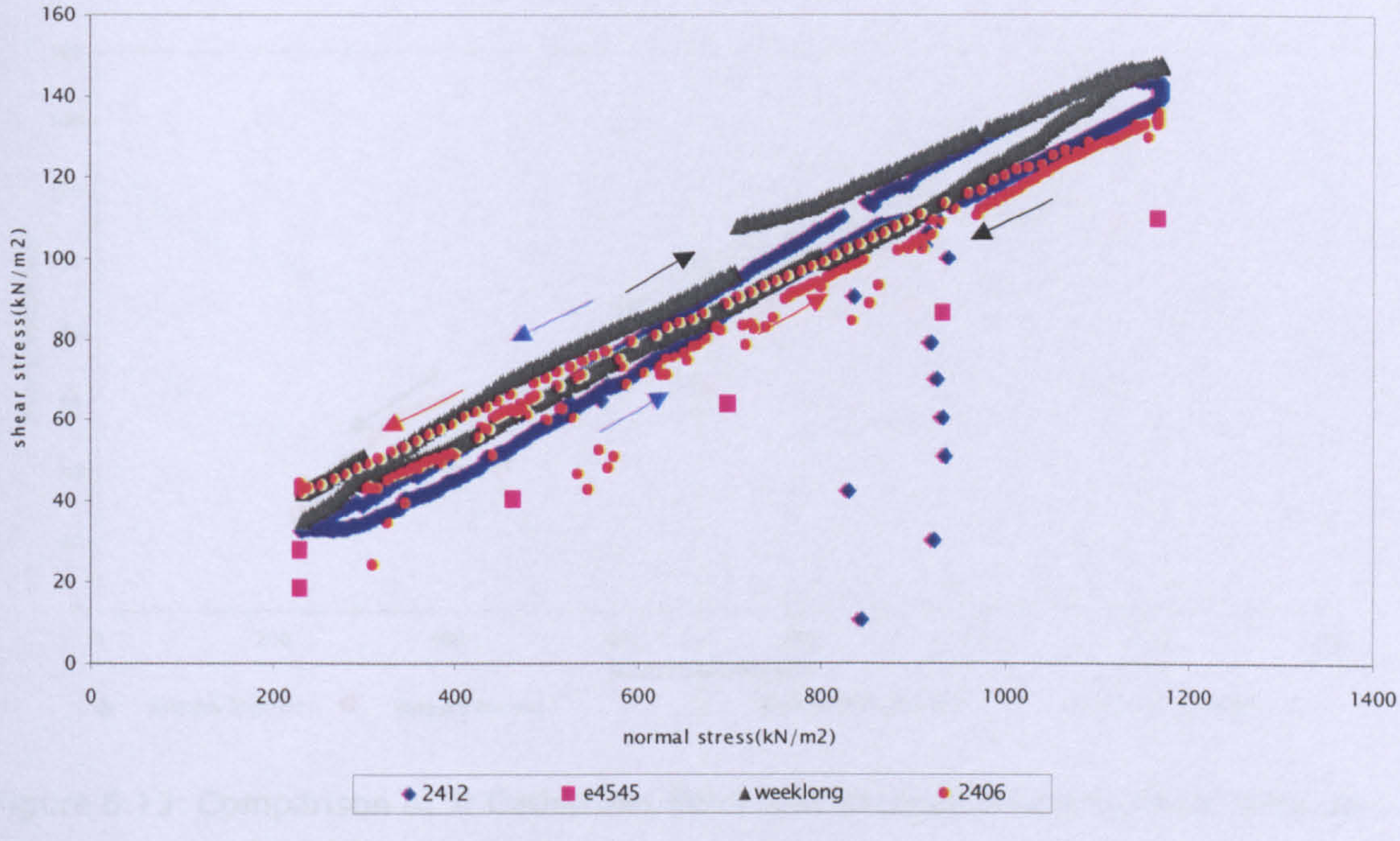


Figure 6.11: Direct comparison of discrete and continuous loading using London Clay exhibiting all rates of loading

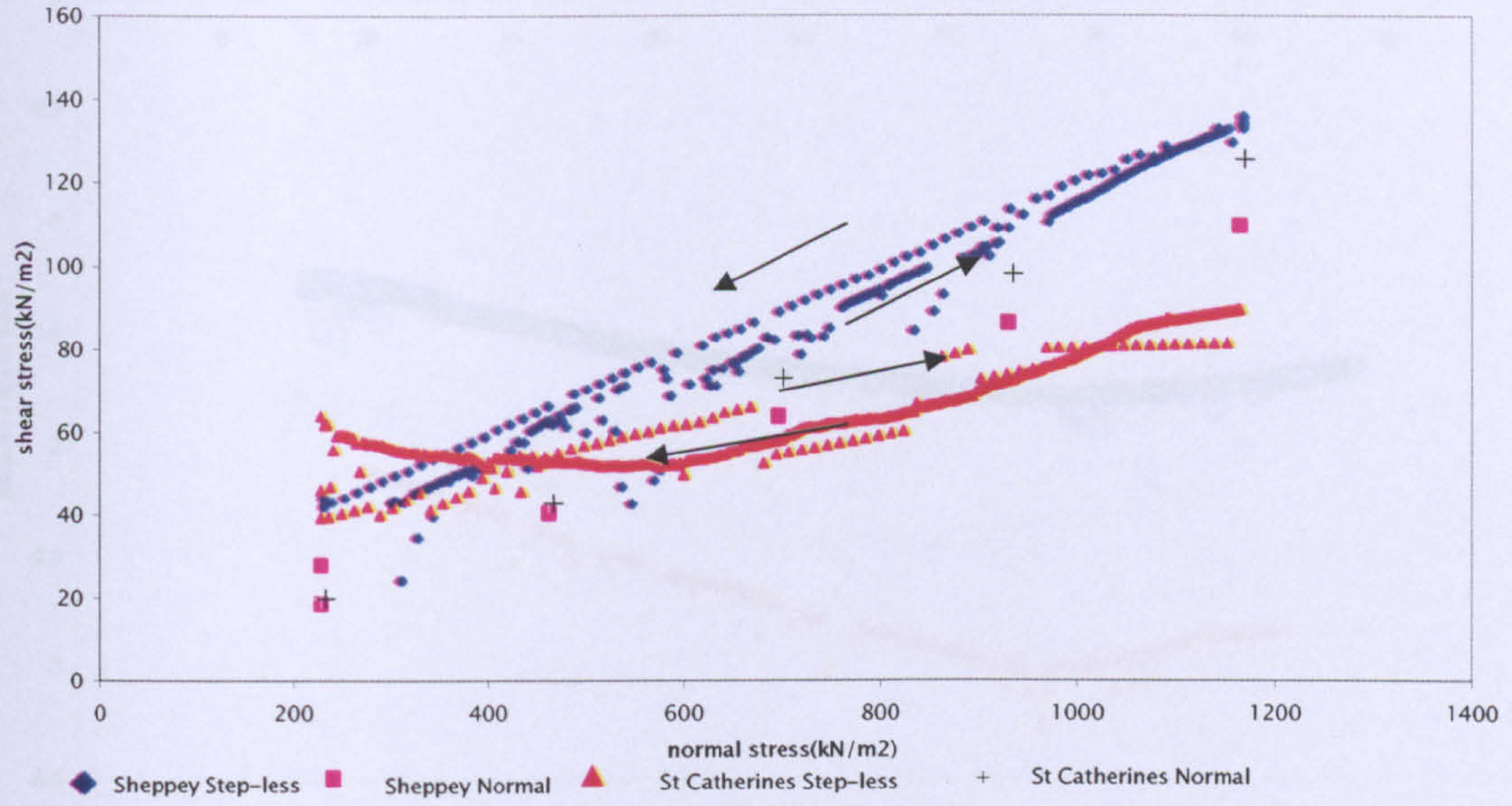


Figure 6.12: Comparison of St Catherines Point and Sheppey with a 24 hour ramp up and a 6 hour ramp down

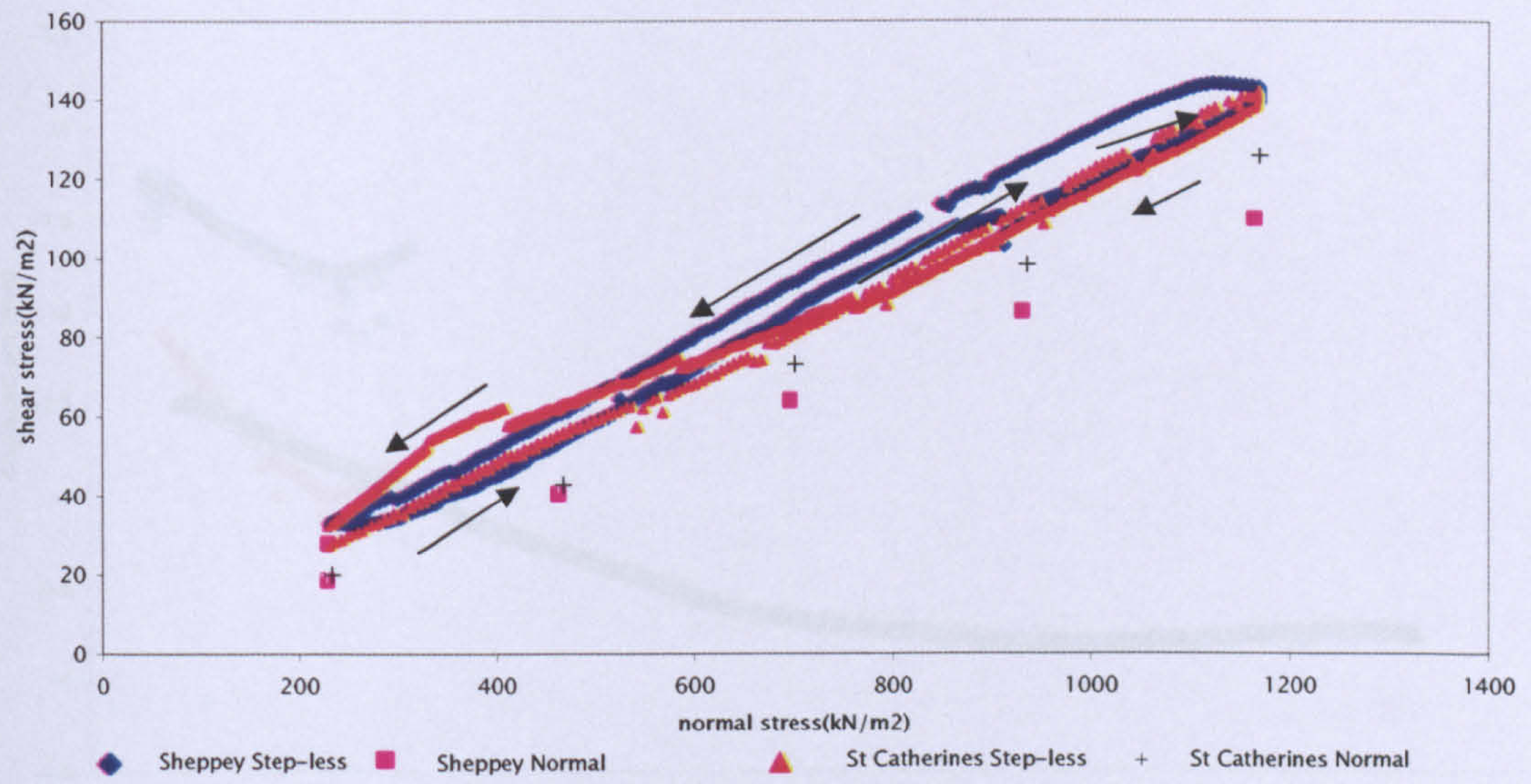


Figure 6.13: Comparison of St Catherines Point and Sheppey with a 24 hour ramp up and a 12 hour ramp down

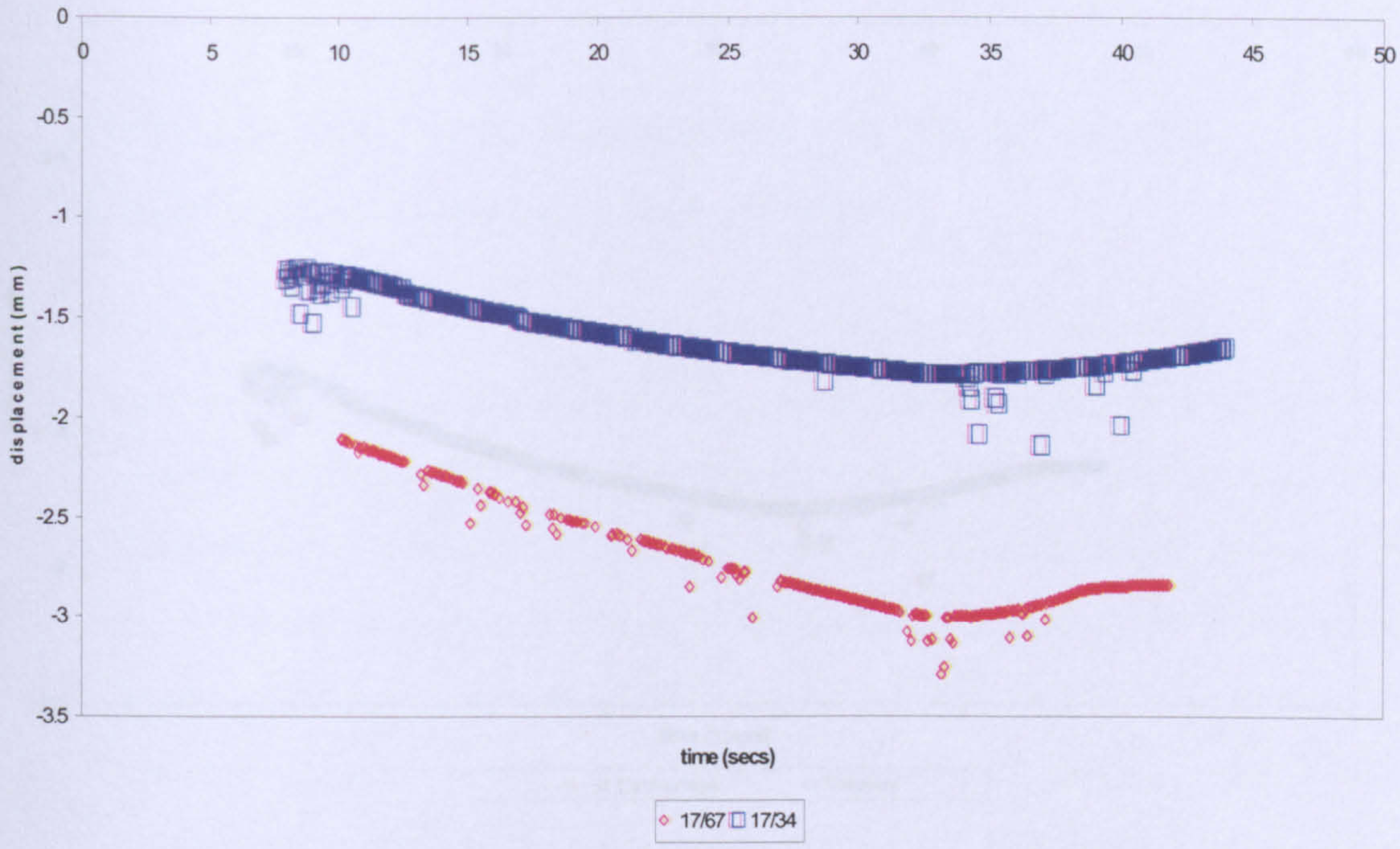


Figure 6.14: Settlement at the faster rates of loading for London Clay

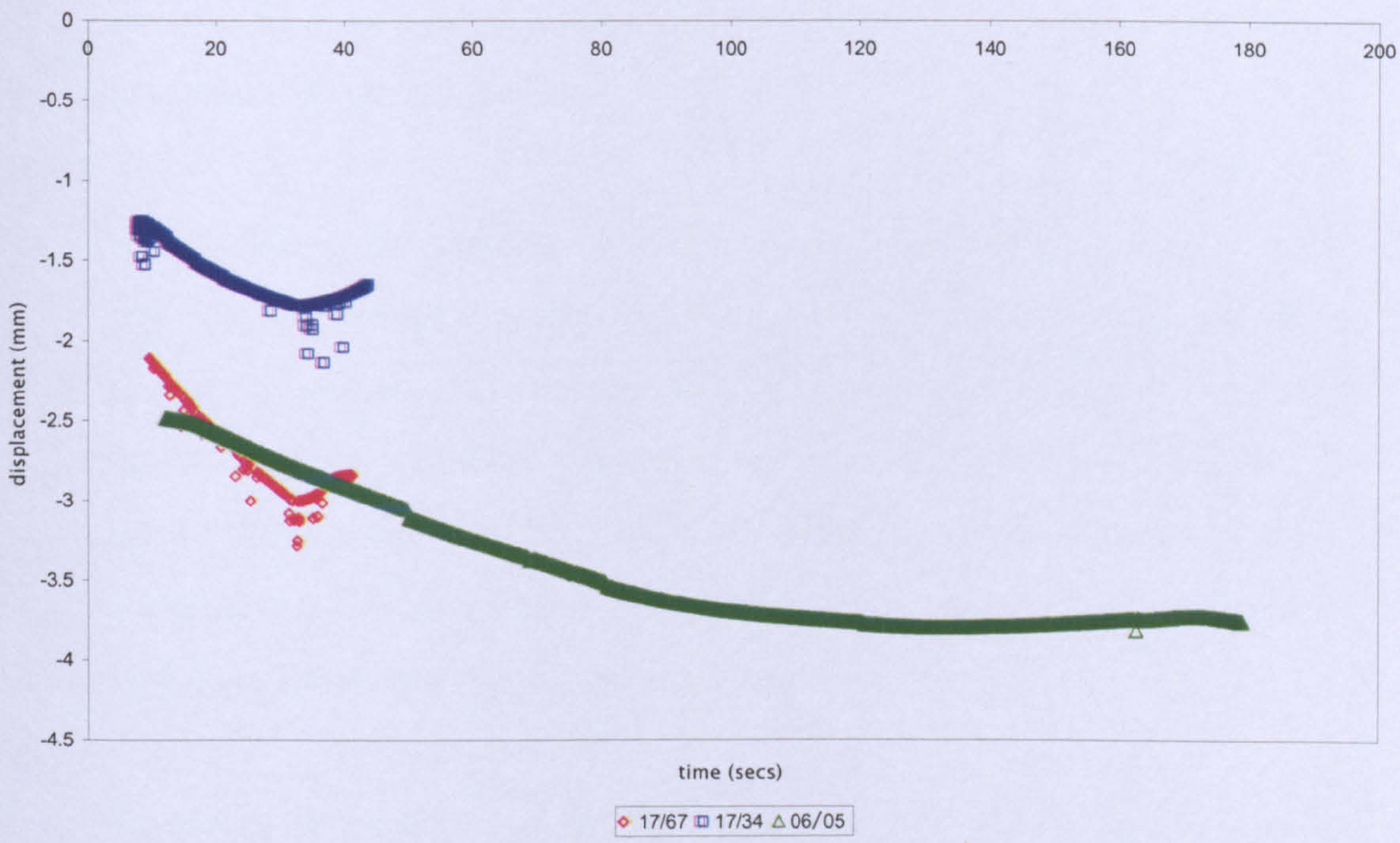


Figure 6.15: Settlement at all rates of loading for London Clay

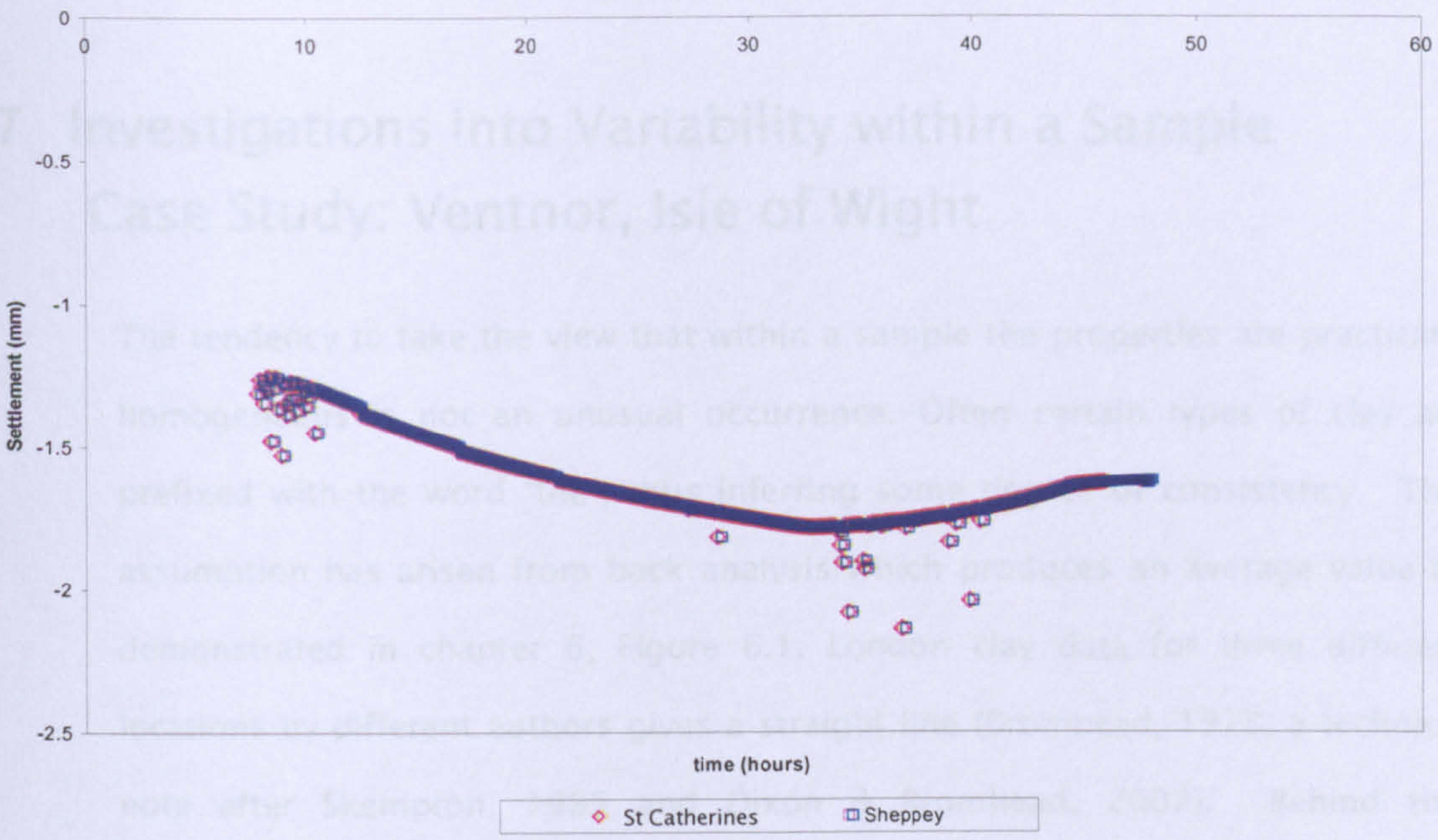


Figure 6.16: A comparison of rates of settlement for clay from St Catherines and Sheppey

7 Investigations into Variability within a Sample Case Study: Ventnor, Isle of Wight

The tendency to take the view that within a sample the properties are practically homogeneous is not an unusual occurrence. Often certain types of clay are prefixed with the word "the", thus inferring some degree of consistency. This assumption has arisen from back analysis which produces an average value as demonstrated in chapter 6, Figure 6.1. London clay data for three different locations by different authors gives a straight line (Bromhead, 1978; a technical note after Skempton, 1985 and Dixon & Bromhead, 2002). Behind this phenomenon is the fact that back analysis produces an average value thus the straight line. Bromhead et al. (2000) investigated variability within a sample and examined the results statistically. From this they concluded that within one sample there is a degree of variability. This means that even the same sample on using an identical test set up would not necessarily give the matching result on two indistinguishable specimens.

London Clay is not exposed in many locations therefore it is not very easy to examine the variability within the whole deposit, due mainly to the densely populated area known as the London basin. Examining the variability within IOW Gault was possible through a landslide hazard study in Ventnor, South IOW. For the IOW there is one main problem. Gault and clays in sandrock are actually very similar and have a tendency to be amalgamated together. Therefore samples of both were investigated for the project.

To put it in context the Isle of Wight, one of Britain's favourite holiday destinations, is slowly slipping into the sea due its numerous landslide complexes which are predominately on the south coast. Located in a secluded south eastern corner Ventnor was very popular in Victorian times due to its cleansing air but in recent years the age of the septic tank on the outskirts coupled with being within a major landslide zone has led to some of the town vanishing.

This chapter uses samples taken at various levels from Ventnor, Isle of Wight to investigate the assumption that "the gault", this blue grey clay has non variable properties within the same sample. In addition included in this chapter is a review of the geological and topographical history of the Isle of Wight, the Undercliff and Ventnor.

7.1 Varibility within a Sample

Bromhead et al.'s (2000) investigations into the innate variability within a sample of London Clay from Warden Point demonstrate that clay is not homogeneous. Their investigations used samples from along the basal shear of a coastal landslide which were repeatedly tested and then statistically analysed. These results indicate the shear strength parameter ϕ' had a standard deviation of 1°. Back analysis over the years has given the impression that clays are homogeneous, they are a naturally derived deposit and therefore by their very nature variable. Back analysis uses an average value and therefore distorts reality slightly.

The Gault is easier to sample than London clay as it is more exposed and therefore more accessible. For this research the samples came from a landslide hazard study in Ventnor. The following sections describe the geological and historical context of the landslides location.

7.2 Geological and Historical Influences on the Isle of Wight.

The Isle of Wight is situated on the South Coast of Great Britain, separated from the county of Hampshire by the Solent. The island is fairly small - only fifty-five square miles in area - with a perimeter of sixty miles. The widest part of this jagged "rhombus" shaped island north-south is between Cowes and St. Catherine's Point, which is approximately thirteen miles. Between Bembridge Foreland and the Needles is the widest part east-west and that is only approximately twenty-three miles. The main river is the Medina which flows from St. Catherine's Downs to Cowes. Lighthouses to protect shipping from the dangerous rocks are located at The Needles to the west and St. Catherine's Point, the most southerly point of the island. There is also to the east at Bembridge a Warner lightship.

The island has two ranges of hills - the larger of the two is the Chalk Downs which run from Culver, which is at the north-eastern end of Sandown right across the Island to its westerly tip at The Needles. The smaller range cuts off the district of the Undercliff from the rest of the Island. These run just from near Shanklin to St. Catherine's Point, are known as the Madeira of England and is where many of my samples are from. The geology of the Island is all sedimentary, formed within the most recent one percent in terms of geological time. Many years of mass movement, faulting and folding followed by erosion have exposed what is beneath the surface. It is thought that the oldest rocks which are visible were part of the huge river valley of which is now the South of England and the English Channel, over 110 million years ago. The clays and sands were eventually laid down after the land level was reduced below the sea level. As the sea level increased the chalk was laid down, the remainder of this is visible at Tennyson Down.

About 40 million years ago this sea had been reduced to a sub tropical island complete with lagoons, marshes and crocodiles! Then 30 million years ago, the internal stresses within the Earth's Crust resulted in buckled and folded rock layers. Those rock layers were then subjected to uplift and erosion. Over time the Isle of Wight evolved to its present shape but remained linked to the mainland until some time after the last Ice Age had melted which had resulted in increased sea levels. This obviously increased the size of the Solent Estuary which all occurred approximately only 77,000 years ago.

The rocks of this island fall into three categories of geological time, Cretaceous, Palogeogene and Neogene and are up to 110 million years in age. The Palogeogene are mainly in the north and are younger, maximum of 54 million years old. The Cretaceous under lie the southern half of the island and are older, up to 110 million years old. The rest are either Neogene rocks the youngest rocks or classed as Quaternary geology. Hutchinson & Bromhead (2002) describe the Quaternary geology of the Island, specifically regarding the effects of periglaciation since the Island has never been glaciated. Hutchinson & Bromhead in the same paper also discuss the effects of the channel river on the shape of the Island during the period of fluctuating sea level. It is thought during this period of time that The Needles - Ballard Point chalk ridge - was breached. Hutchinson & Bromhead also give an excellent summary of the tectonic movements and it is therefore beyond the scope of this thesis to detail them.

The Island is renowned for its geology. A past Director of the then Geological Survey, John Flett, according to Hutchinson & Bromhead (2002) remarked in 1921 that "no district of England of equal size is more interesting to a geologist than the Island of Wight". This is clearly visible from Figure 7.1. Its geology, beauty parts of the Island are known as the Madeira of England, are not the only attributes for which the Island is renowned. As Hutchinson & Bromhead (2002) noted the coastal landslides were particularly "varied and fascinating". These landslides are particularly evident along an area of coastline known as the Undercliff.

7.3 The Undercliff

Situated on the South Coast of the Island, it is just 6 miles long with a width that varies from ¼ to ½ a mile. On the landward side the Undercliff is bounded by a 200 ft cliff. This is followed by a secondary cliff which in some areas has been eroded away to form a patch of rough broken land which slopes rapidly down towards the shoreline. The Undercliff despite the many slips in the last three or four centuries, with the exception of a place known as Windy Corner, has exhibited the same outline shape for many years. Windy Corner was the site of a major landslide in 1928 where a road and a large area of cliff were destroyed during a major slip. Blackgang Chine and St. Catherine's Point itself have sites of major slips in recent years. The theme park built on the sterile area known as Blackgang Chine regularly has to move or repair its attractions due to the slips which occur mainly during the winter months. The movements at St. Catherine's Point, where many of the author's samples were collected, are active landslides. The Lighthouse at St. Catherine Point is monitored by GPS satellites for any movement. There is movement and the lighthouse complex according to Hutchinson et al (2002) is moving faster than the main slide. The Lighthouse is not tilting above the maximum allowable amount and therefore is still serviceable. The details of the slides have been summarized by both Hutchinson et al (2002) mainly from the conclusions of Hutchinson et al (1991). Clarke

(2004) in an unpublished paper noted that the mudslides at St Catherine's Point are less complex than those at other locations in the Undercliff. The reasons behind this are:-

- The fact that St Catherine's Point is largely uninhabited.
- The toe of the landslide is not in line for direct marine action as it discharges on to the large apron of debris at the base of the slides
- The source area of the landslide is actually isolated from the chalk aquifer of the chalk and upper greensand of St Catherine's Hill according to Hutchinson et al. (1991) by the occurrence of a massive block slide.

Therefore activity is entirely down to rainfall and this area represents a control which other more inhabited areas can be compared to.

The underlying geology is a major cause of the landslides. The geology of the Undercliff consists mainly of cretaceous rocks and landslide debris. A large percentage of the south coast of the Isle of Wight consists of lower greensand. Behind the Undercliff the rock strata goes further up the sequence where in the few places the chalk has been eroded away the upper greensand is still intact as shown in Figure 7.2. Beneath the upper greensand in the Undercliff and in the other areas of the Island, are extensive bands of gault known locally as the "blue slipper" due to its blue/grey colouring and its reputation as being the principal cause for the major landslips which make up the Undercliff.

In their keynote paper, Hutchinson & Bromhead noted the importance of even subtle changes in low dips. They concluded the change of even a degree or two in the component of seaward dip in the Undercliff had resulted in an "abrupt change in style of landsliding between the St. Catherine's Point slides" and the slides just NW up the coast on Gore Cliff. The actual seaward components of dip controlling the basal slip surfaces according to Hutchinson& Bromhead (2002) are only +1.5° to - 0.1°. That a difference of less than two degrees results in an abrupt change in such a short distance is proof of the implications of the difference.

The Undercliff is subject to coastal attack from two directions which makes it one of the most exposed sections of coastline on the island. There are stretches of coast between SW and just west of W-S-W which have fetches from right across the Atlantic. These large fetches result in waves of anything between 3 and 4 m high according to Hutchinson & Bromhead (2002), which they noted were between a bearing of 120° and 270° in a specific sector coastline. Hutchinson & Bromhead (2002) were quoting Figures for Bonchurch at the eastern end of the Undercliff but the heights and fetches apply to the stretches of coastline which face towards the Atlantic. The coastline which faces between SE and SW is subject to fetches across the English Channel. These shorter fetches result in shorter wave heights. There are no coastal defences as such at St. Catherine's Point, as the large blocks of upper greensand act the same as a rock breakwater and cushion the effect of the waves on the shoreline.

The town of Ventnor is just up the coast from St. Catherine's and is one of the largest towns in the Undercliff.

7.4 Ventnor

Situated on the south east coast of the Island, it is actually a sub-box and as a consequence neither the north nor eastern winds frequent it. The town was popular in the Victorian times for its therapeutic water; people would come from miles around just to bathe in the sea at Ventnor, hence the Victorian style architecture which is different to other towns on the Island. The town is built on a hill, in a theatre formation, most houses face the sea. Ventnor is, in fact, almost as renowned for its zigzag and steps as Venice is for its canals. The actual hill on which the town is built is 800 ft high. It has a steep slope with a gradient of approximately one in four stopping approximately 20 ft before the shore line.

Hutchinson & Bromhead (2002) provide an excellent summary of the research done by Hutchinson et al (1991a), Hutchinson et al (1991b) and Rendel Geotechnics (1993) on the Ventnor Park Model. This is a confirmed refined combined model between Gore Cliff and St. Catherine's Point Landslide models. Hutchinson & Bromhead (2002) describe it as a "general combination of compound slides to seaward, seated in the sand rock, and backed by multiple compound-rotational slides seated in the mid-gault". There is unfortunately a limit to this definition by Hutchinson & Bromhead (2002) as the situation cannot be precisely defined "without further geological mapping and sub surface investigation".

As part of the "living" with landslides scheme, the town's main sewerage system has been substantially reconstructed Hutchinson & Bromhead (2002), the water mains, sewers and cess pits have all undergone a process of more rigorous maintenance and reduction in leaks. The main sewerage area covers the heavily built up sector about Nat. Grid. Ref SZ 541 767, which approximately coincides with the Ventnor-St. Lawrence boundary and Bonchurch which includes Lowtherville to the rear of the Undercliff. Hutchinson & Bromhead (2002) note that the sewerage is collected behind East Ventnor Bay and then pumped inland up the coast to Sandown. The more remote, less built-up areas of the Undercliff, NE of Bonchurch and W of the Ventnor-St. Lawrence boundary are reported by R. McInnes (pers comm) to Hutchinson et al (2002)) to be still using septic tanks and cess pits. Non mains sewerage results in large fluctuations of the ground water levels which has a destabilizing effect on slope stability.

Hutchinson & Bromhead (2002) concluded from research done in the early nineties that the main seat of the landslide is in sand rock horizon 2d and noted the observations of Dike (1972) that the layers were discontinuous for sentimentological reasons.

The samples used in this chapter were boreholes drilled as part of research by IOWCC/Halcolw and taken from the western outskirts of Ventnor Town in three different locations as shown in Figure 7.3. BHI was 180 m from the shoreline and 40 m above sea level, BH2 was further back at 440 m from the shoreline and 104m above sea level and BH3 was 70 m above sea level and 335 m horizontally from the shoreline. BHI, situated in a less steep area of Ventnor and nearest of the three bore holes to the coast, comprises mainly of chalk debris, then sand rock of varying horizons. Tests for this chapter were done on samples from sand rock 2d and sand rock 2b. Moving up the slope away from the coast to a steeper gradient where BH3 is located. BH3 comprises of upper greensand, passage beds, gault (where most of the samples for this chapter are from), carstone and sand rock of 3 horizons. The samples from this borehole were from gault and one from sand rock 2e. BH2 is situated the furthest from the coast in area with a lower gradient than BH3 and is by far the longest borehole. BH2 comprises of chalk, chertbeds, malm rock, passage beds, gault, carstone and 5 horizones of sandrock. The samples for this chapter were taken from the gault and sandrock horizon 2c from a range of depths throughout the specified layers of borehole. Ring shear tests, plastic limit, liquid limit and moisture content tests were then performed to examine the properties of the layers and investigate the interrelationships.

A record of the mechanisms involved and exact locations of the boreholes is contained within Appendix A.

7.5 The Sandrock Layers

At the base of all four boreholes, including the previous programme's borehole, there are up to five layers of sandrock as shown on Figure 7.3. The layers dip towards the coast and start at between 8 m above to -18 below and are approximately 42 m deep at maximum to 36 m at minimum. Categorized into 2(a), 2(b), 2(c), 2(d) and 2(e) this research programme used samples from two layers 2(b) and 2(d). It is within 2(d) which the shear zone is inferred on Figure 7.3. Although the samples are from different boreholes and different layers, definite trends can be observed in Figures 7.4, 7.5 and 7.6. These layers are sandy clay and therefore use the procedures outlined in chapter 4.

Figure 7.4 clearly indicates a trend of decreasing effective drained angle of friction residual shear strength parameter, Φ'_r with depth. A more detailed analysis of the samples within sandrock layer 2(d) indicates an increase in Φ'_r of 3.5° over three metres in depth but 170 m horizontal distance as the specimens are from borehole 1 and 3. The sample from borehole 2 which is within layer

2(b), indicates that over twenty three metres in depth, Φ'_r decreases by 7.3° compared to the sample from 2(d) from a horizontal distance of two hundred and fifty metres from borehole 1. The last sample was from four metres below and two hundred and fifty metres of horizontal distance from the previous but from within the same layer 2(b). Indicating a trend of decreasing Φ'_r with depth, this sample displayed a 1.5° reduction in Φ'_r compared to the previous sample. Although a general trend can be observed variations within the strata and the actual layers are also evident from Figure 7.4.

The soil properties from within the sandrock layers show an overall trend of increasing percentage moisture content with depth as shown in Figure 7.5. Initial sample moisture content has a general trend of increasing with depth. More detailed analysis shows a 2.3% decrease over three metres in layer 2(d) and that there is no significant change within 2(b) over nearly four metres. Between the lowest sample in 2(b) and the highest sample in 2(d), there is an increase in moisture content of 4.3%. This would indicate that the moisture content rises with depth.

Plastic limit results display a similar trend of increasing with depth. Detailed layer analysis shows a decrease of two percent within layer 2(d) over three metres and an increase of 2.5% for layer 2(b) over nearly four metres. An increase of five percent was indicated from the results between the lowest sample from 2(d) and the highest sample from 2(b) which as shown on Figure 7.5 indicates a trend of increase in plastic limit with depth.

Liquid limit results from these samples show a definite trend of increasing moisture content with depth as shown in Figure 7.5. The same trend is observed within the actual layers. Layer 2(d) sample results indicate a 4.3% increase over three metres. For layer 2(b) the increase is larger, 8% over nearly four metres. Between the maximum depth for layer 2(d) and minimum depth for layer 2(b), a distance of twenty three metres, there is an increase of 21.7%. This confirms the trend of increasing liquid limit with depth.

Similar trends were observed for plasticity index, I_p, as shown in Figure 7.5 which had an overall increase with depth. Analysis of the layers indicated an increase of 6.4% over three metres within layer 2(d) but within 2(b) there was a 5.5% increase over nearly four metres. The interlayer trend indicated an increase in Ip with depth and the results showed a 27.3% increase over twenty three metres, confirming the trend of increasing I_p with depth.

The relationship between plasticity index I_p and effective drained residual angle of friction shear strength parameter, Φ'_r as shown in Figure 7.6, display a trend of decreasing I_p with increasing Φ'_r. Layer 2(d) shows a decrease of 6% I_p with a decrease of 3.5° over approximately four metres but this is within the inferred shear zone. Further down layer 2(b) displays an identical trend to the overall, the Ip decreases by 5.5% and the Φ' increases by 1.5° in three metres. The interlayer relationship indicates an increase of four degrees but an 18.8% decrease in Ip which confirms the overall trend.

The layers do interrelate to each other and they have similar trends of properties even if they are from separate boreholes hundreds of metres apart. Detailed analysis of these results indicates that values within a layer are within the same range of values - there are no extreme values. Between 2(d) and 2(b) the values display a trend of increasing with depth. These results identified a trend of higher values for layer 2(b) than 2(d) except for the effective drained residual friction angle shear strength parameter where the opposite was observed. Layer

2(d) is inferred as a shear zone in Figure 7.3 which would explain the higher values of Φ'_r.

7.6 The Gault Layer

This layer comprises of firm to very stiff dark blue grey clay and is approximately 45m narrowing to 30 m from in depth borehole two to borehole three. This layer is not continuous to the coast and at some point between 180 m and 355 m from the coast is replaced by chalk debris which is a direct result of a previous landslip. Eventually at less than 200 m from the coast the gault reappears but as a layer less than 7 m deep as a previous research programme's borehole PI indicates. The following analysis is the result of British Standard laboratory tests as detailed in Section 4.6 and Kingston University Ring Shear Procedure as detailed in Section 4.2

Figure 7.7 indicates a general trend of increasing angle of friction with depth. This would indicate that the gault was stronger with depth which would be as a result of the greater normal stress being applied by the self weight of the soil resulting in the clay being compressed which reduces the distance between particles and therefore a stronger matrix forms, hence clay increasing its strength over time with loading due to settlement. From Figure 7.3 there is a possible shear zone which is approximately 45 m above sea level. On a more detailed examination of the results, borehole 3 initially shows an increase in Φ'_r of 4.5° with depth between 26 m and 29 m from the surface at approximately 43 to 46 m above sea level. Following that, the results indicate a decrease of almost eight degrees in the next four metres followed by a trend of slow increase of only two degrees for the next 14 m. Results of borehole two indicate one situation to be similar with an initial decrease of eight degrees over the first metre. Over the next nine metres there is a trend of slow increase in Φ'_r of one and a third degrees, then after that over the next ten metres there is a decrease in Φ'_r of four degrees followed by the next twelve metres which see an increase in Φ'_r of six degrees. High values at around 28-29 m relative to the surface of borehole 3 correspond to a shear zone on the cross section of Figure 7.3. Another high value within the results was the final increase although lower than at 28-29 m, it could be as a result of the weight of soil or indicate a developing shear zone.

The soil properties are shown on Figure 7.8. Sample moisture content was shown to be fairly stable at between 15% and 30% although a slight trend of decreasing moisture content with depth which is evident from Figure 7.8. There are slight fluctuations within the results for borehole 3 over the first three metres. There was an initial increase of one percent, followed by a decrease of thirteen percent over the next four metres, then an increase of five percent over the next ten but a decrease of two percent over the next five metres. For borehole 2 the first metre shows little change but is approximately three percent lower than borehole 3 at the same level. It then decreases by four percent over the next metre to increase by half a percent over the next seven, then decreased by two percent over the next ten to increase by the same amount over the next twelve. In relation to the groundwater level which is at 60 m they are all within it and therefore were saturated samples. The increase at the end borehole 2 could coincide with the groundwater table in 2a resulting in the increase.

Plastic limit tests on the samples displayed an overall trend of decreasing plastic limit with depth. A more detailed analysis showed fluctuations in the relationship. Borehole 3 displayed an initial increase of three percent over the first three metres, then a decrease of seven percent over the next four metres.

The next ten metres resulted in two percent increase, followed by a six percent decrease. For borehole 2 the trend was similar to borehole three. Initially over the first two metres there was a four percent increase, the next seven metres saw a five percent increase followed by a trend of decreased moisture content at plastic limit with increased depth below ground level of fourteen percent over twenty two metres. Similar detailed trends were observed in boreholes.

The overall trend for liquid limit on these tests is shown in Figure 7.8 to be increasing with depth. Detailed analyses of the graphs indicate fluctuations within the trend. Variations between boreholes resulted in slightly different trends. Borehole 3 shows an initial decrease of less than half a percent over the first three metres, followed by an increase of thirteen percent over the next twenty metres, then a higher percentage increase of twenty-five percent over five metres follows. The overall trend for borehole 3 is easier to define as liquid limit increasing with depth as it almost exactly follows that trend. Borehole 2 is not as simple. Initially over the first three metres liquid limit decreases by almost three percent, which is followed by an eighteen percent increase over the next eighteen metres. In contrast this is followed by a decrease of seven percent over the next twelve metres. Between the two boreholes the most major difference is the liquid limit at 47.37 m below surface on borehole 3, which results in a 95% liquid limit, and at the same relative height to 72 m above sea level (top of borehole 3) at 48.85 m in borehole 2 there is a difference of almost thirty percent as liquid limit is only 67%. Borehole 2 in general had lower values of liquid limit than borehole 3. The only values which were almost identical were within the shear zone at around 26-29 m below surface level.

The relationships between depth and plasticity index are similar to those between liquid limit and depth. From Figure 7.9 an overall trend of increasing plasticity index with depth is shown. When the borehole results are analysed in more detail there are fluctuations within the trend. For borehole 3 the results indicate an initial four percent decrease over the first three metres, then a twenty-five percent increase over the next thirteen metres. Then due to the large liquid limit at this point, there is a thirty percent increase in plasticity index over the next five metres. Initially the results indicate a similar trend for borehole 2, a decrease of seven percent, at approximately the same depth as borehole 3 within the shear zone but with higher values than those achieved with borehole 3. Over the next eighteen metres the liquid limit increases by twenty percent, this rate of increase then reduces to about 0.003% over the next twelve metres.

The relationship between plasticity index and the effective drained residual shear strength parameter, Φ'_r is more complicated as shown in Figure 7.9. Overall there is a slight trend of decreasing plasticity index, I_p with increases in Φ'_r. More detailed analysis shows borehole 3 to indicate an initial increase of thirty-one percent with an increase of three degrees with a thirty percent increase between 5.3 and 6.9 degrees. Between six and seven degrees the I_p decreases by just over fifty percent then between seven and eleven degrees there is a four percent decrease in I_p. For borehole 2 the trend shows greater fluctuation. Initially there is a three degree increase in Φ'_r for a seventeen percent decrease in I_p, then a degree increase in Φ'_r results in a three percent decrease in I_p over an increase of half a degree for Φ'_r. Between 11.5° and 13.2° Φ'_r there is an increase of eighteen percent in I_p which is in direct contrast to the next five degree increase for a thirteen percent decrease in I_p.

As Figures 7.7, 7.8 and 7.9 indicate there are general trends between the soil properties and depth but the interrelationships in this non homogenous material result in no absolute correlations within a varied sample such as this.

7.7 Conclusion

These investigations confirm Bromhead et al. (2000) conclusions that clay is innately variable and non homogeneous as opposed to the results produced by back analysis which are an artefact of the process. This average variation in Φ'_r of a 2° per metre change could result in misleading test results.

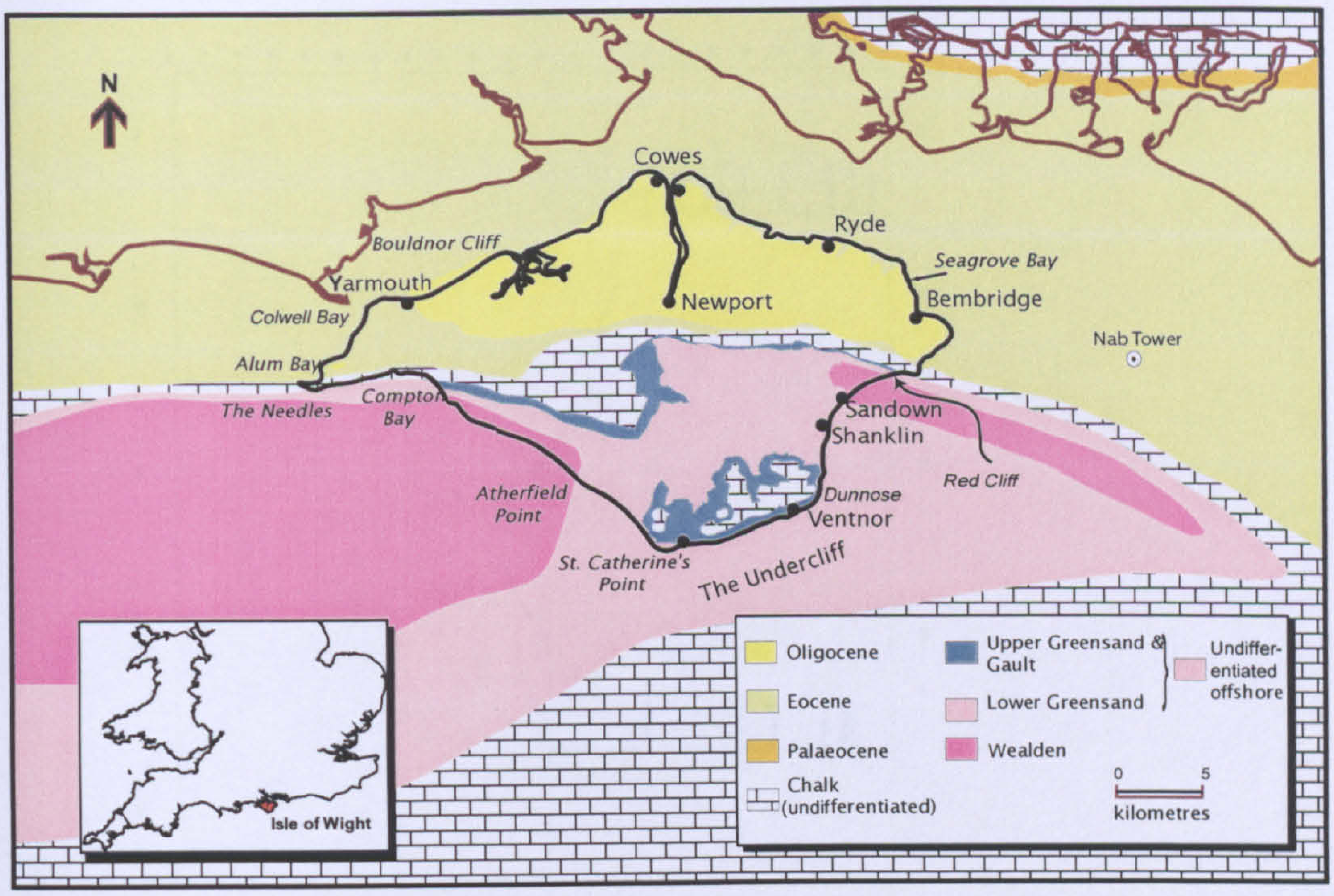


Figure 7.1 Geological Map of the Isle of Wight.
(Based on BGS map, and after Hutchinson & Bromhead, 2002).

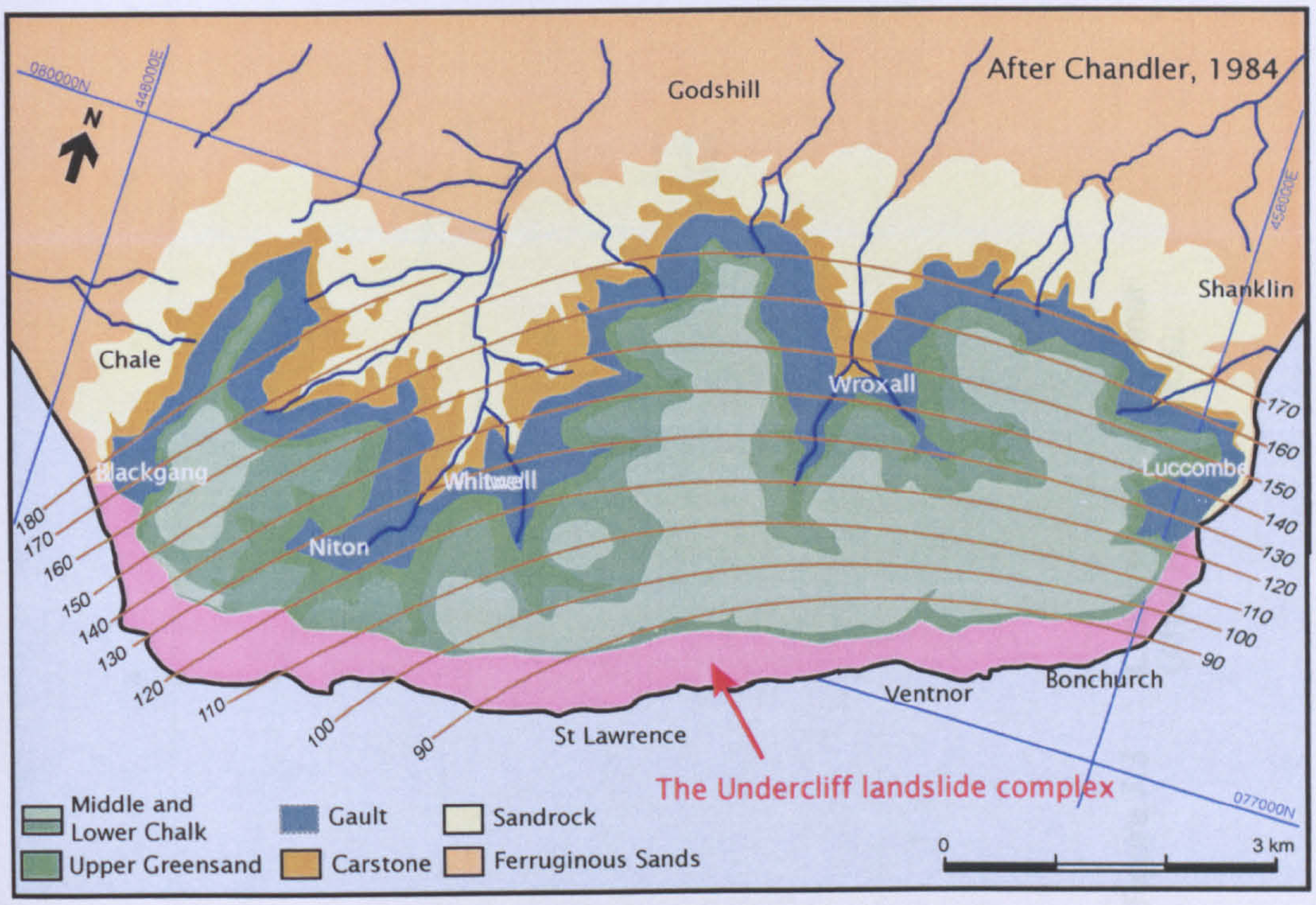


Figure 7.2 Detailed Geological Map of the Ventnor Undercliff area, Isle of Wight, showing contours for the Upper Greensand freestone.
(After Chandler, 1984; Hutchinson & Bromhead, 2002).

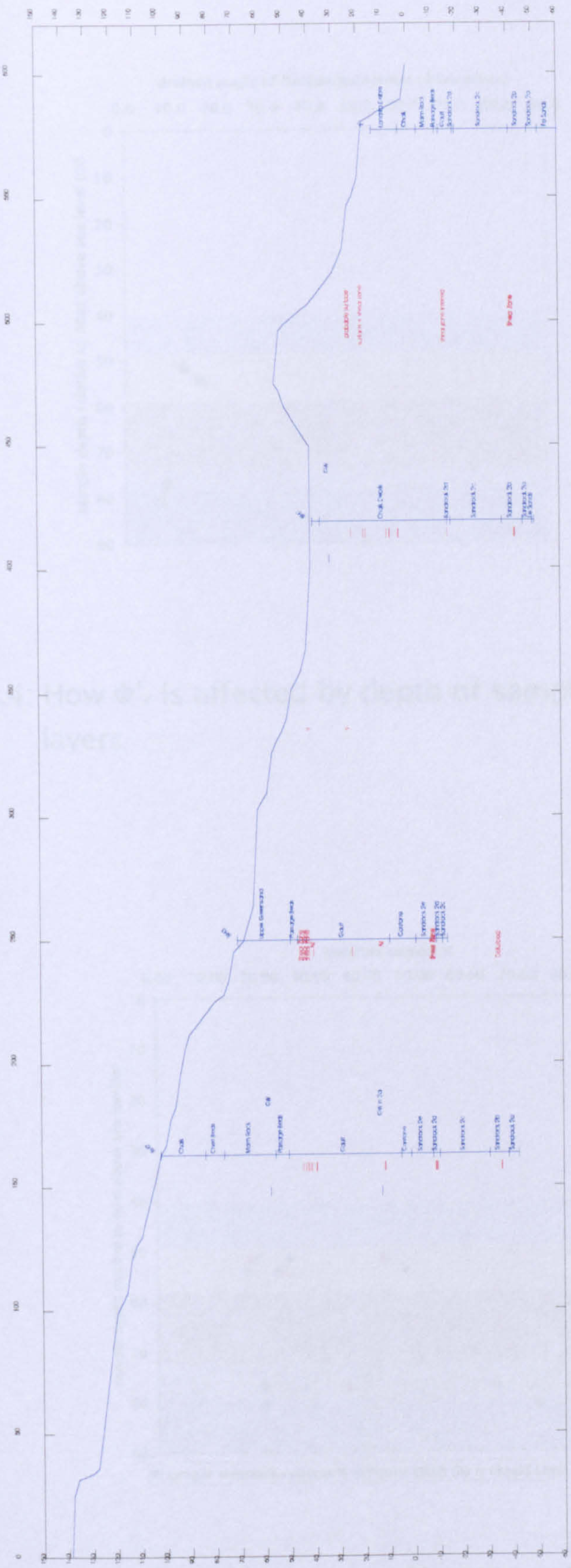


Figure 7.3 Cross section of Ventnor. (After Halcrow, 2003).

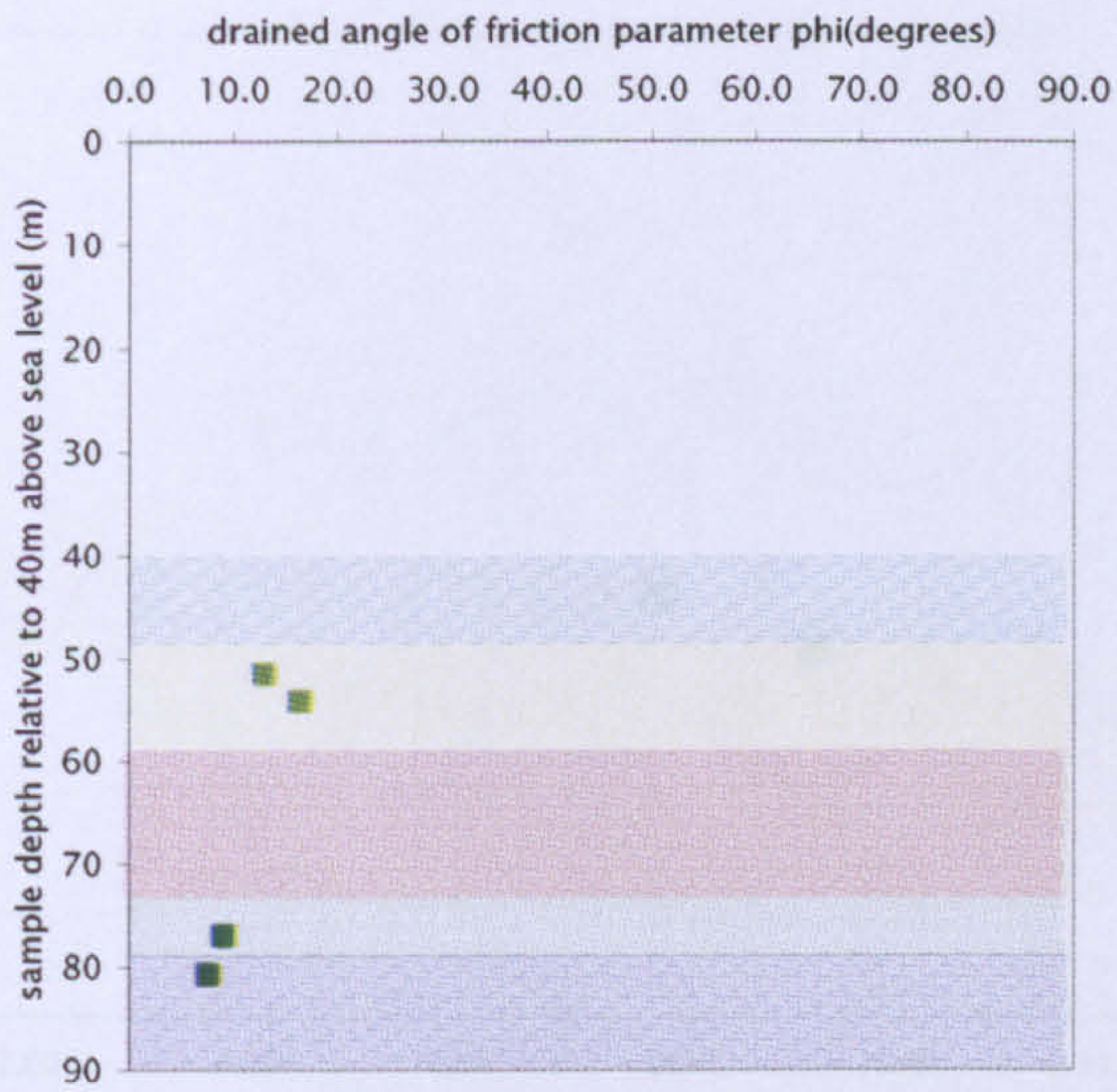


Figure 7.4: How Φ'_r is affected by depth of sample in the Sandrock layers

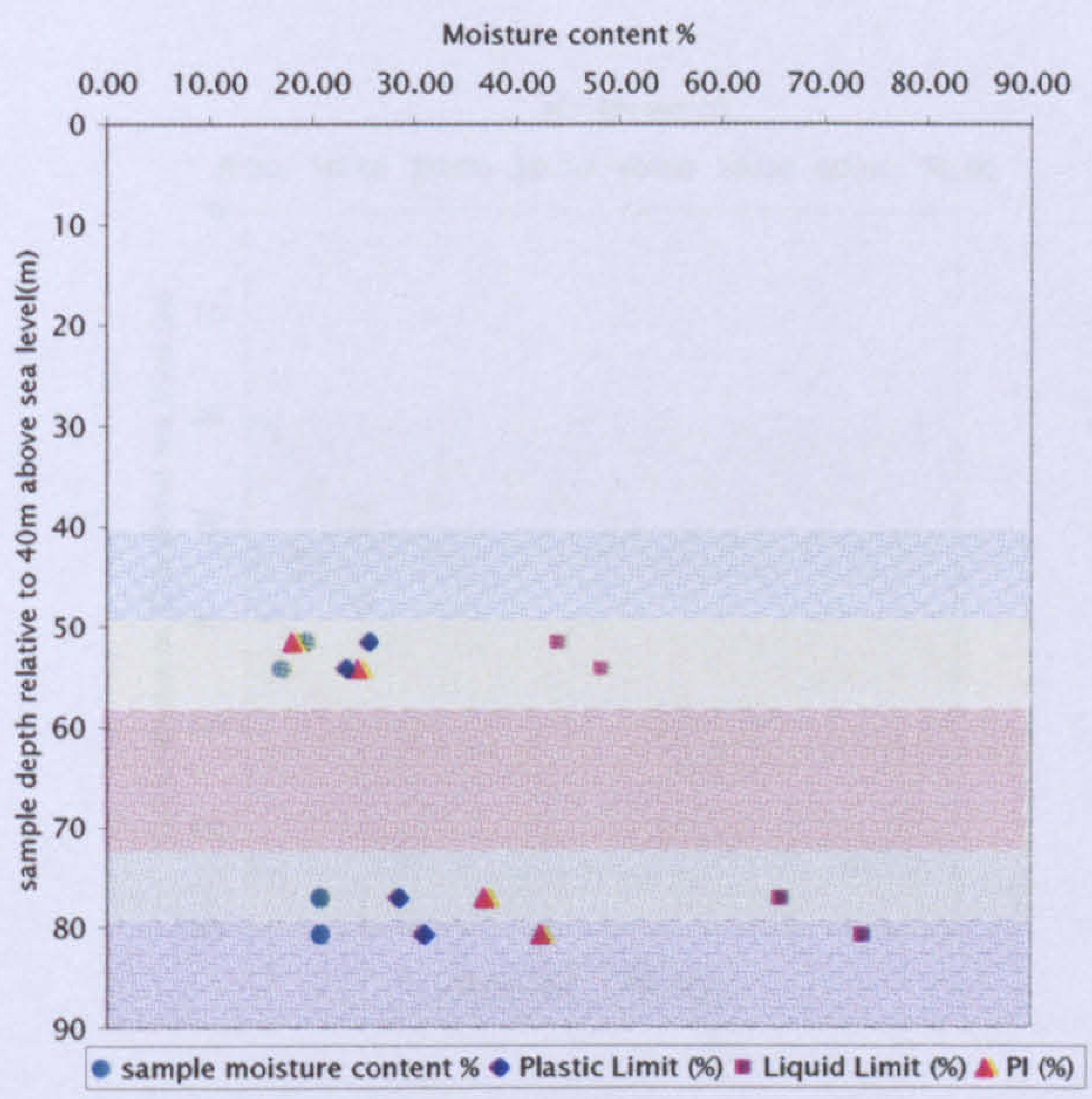


Figure 7.5: How soil properties are affected by depth of sample in the Sandrock layers

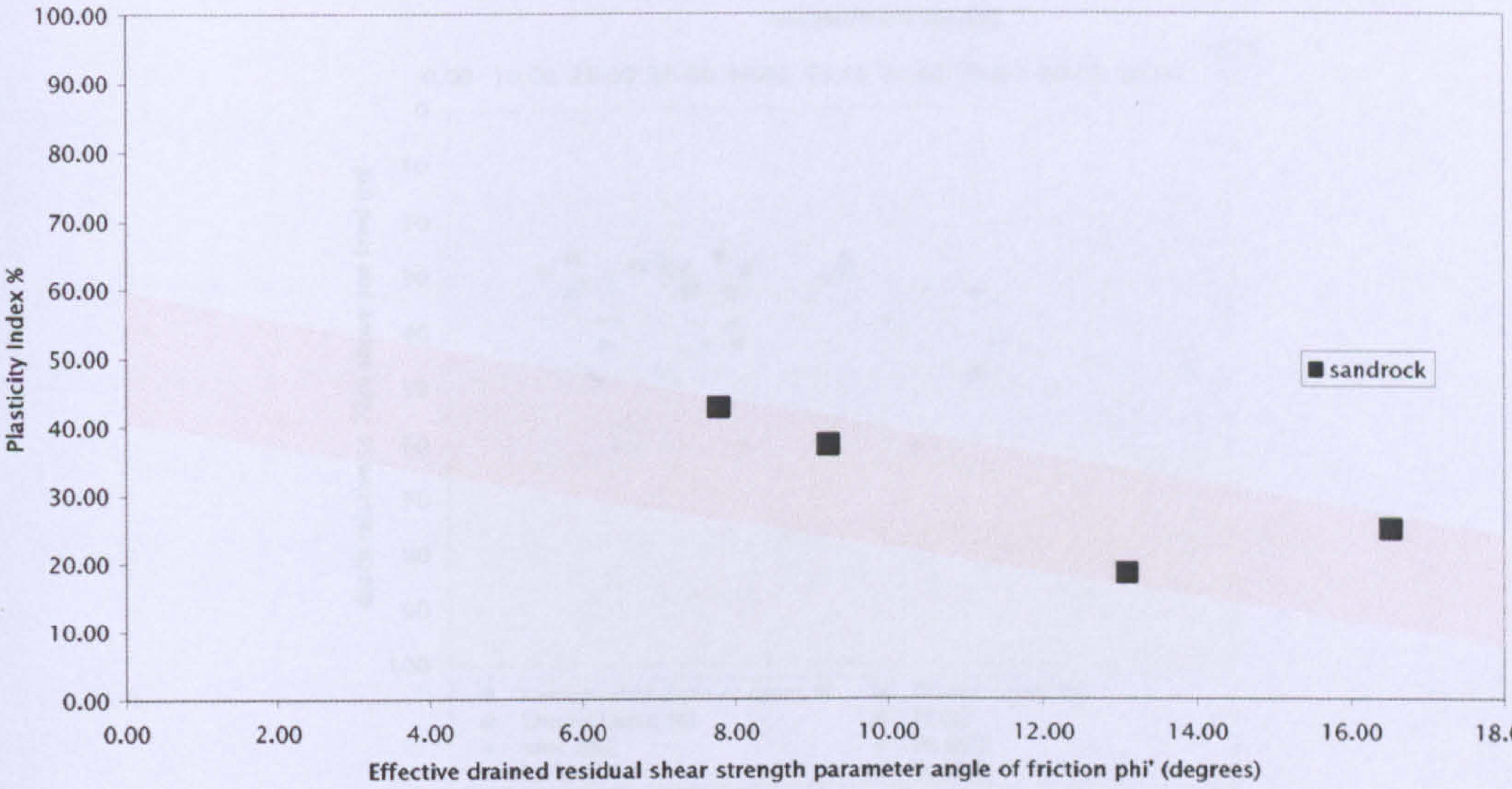


Figure 7.6: The relationship between Plasticity Index I_p and Φ'_r in the Sandrock layers

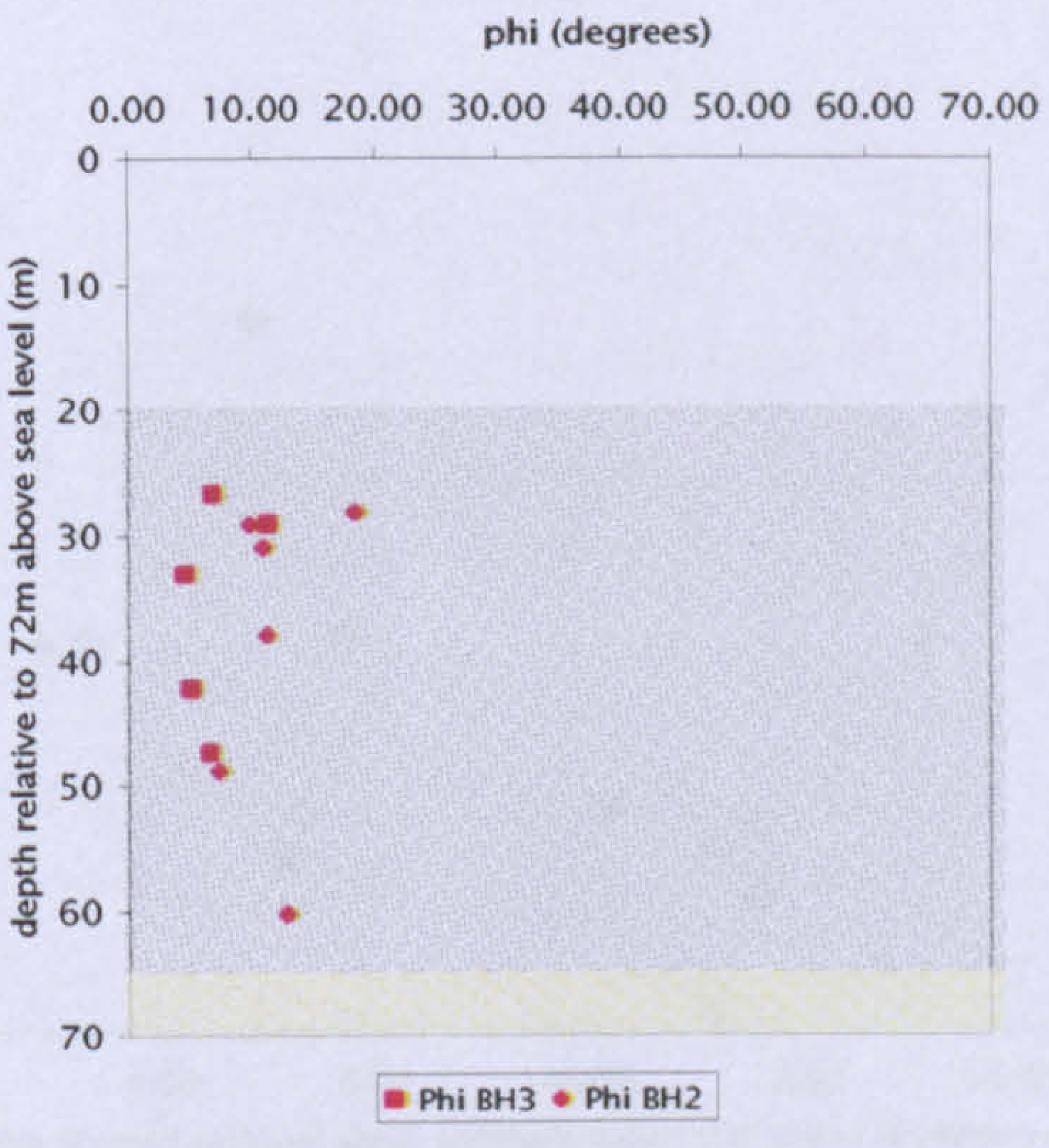


Figure 7.7: How Φ'_r is affected by depth of sample in the Gault layer

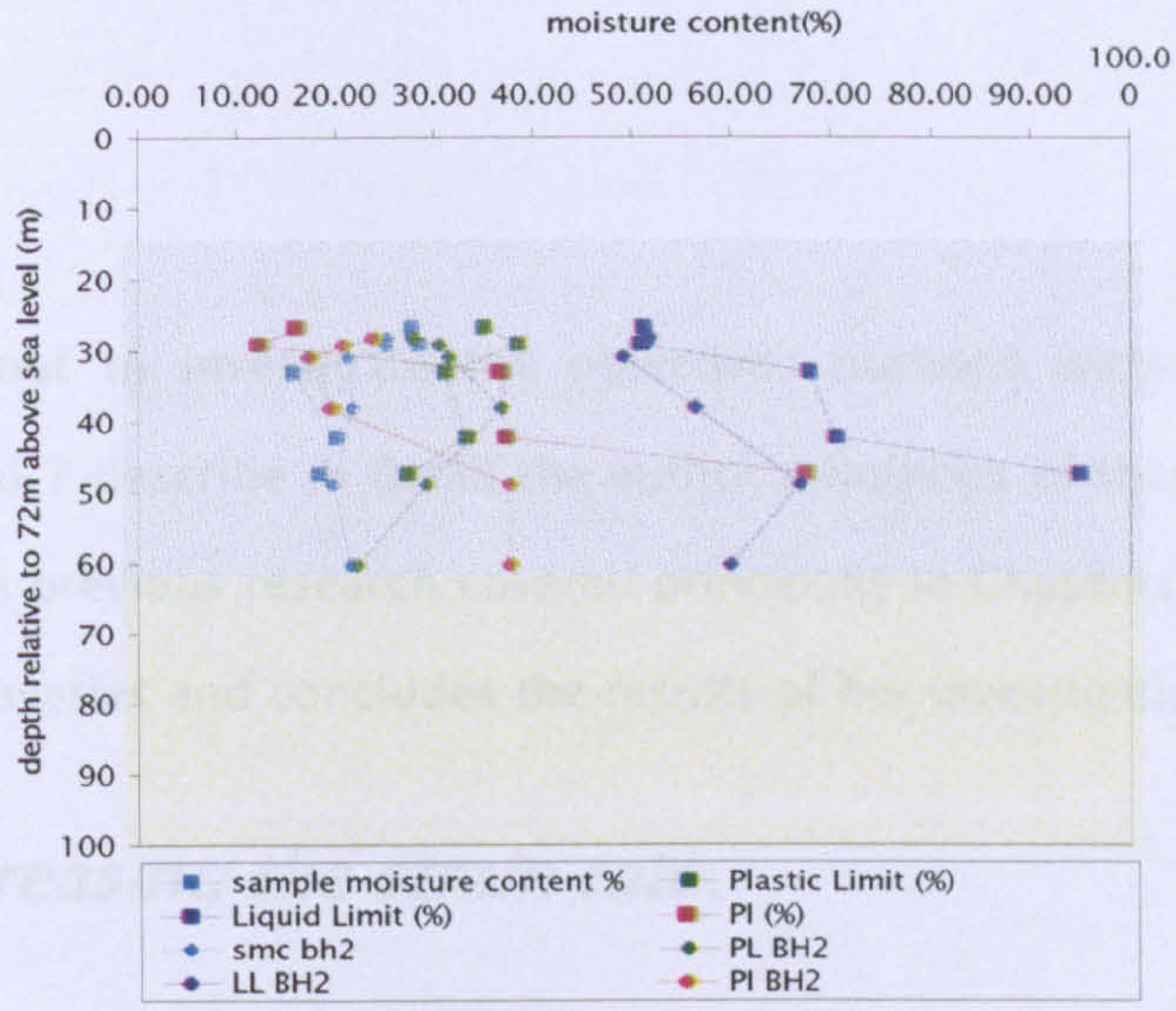


Figure 7.8: How soil properties are affected by depth of sample in the gault layer

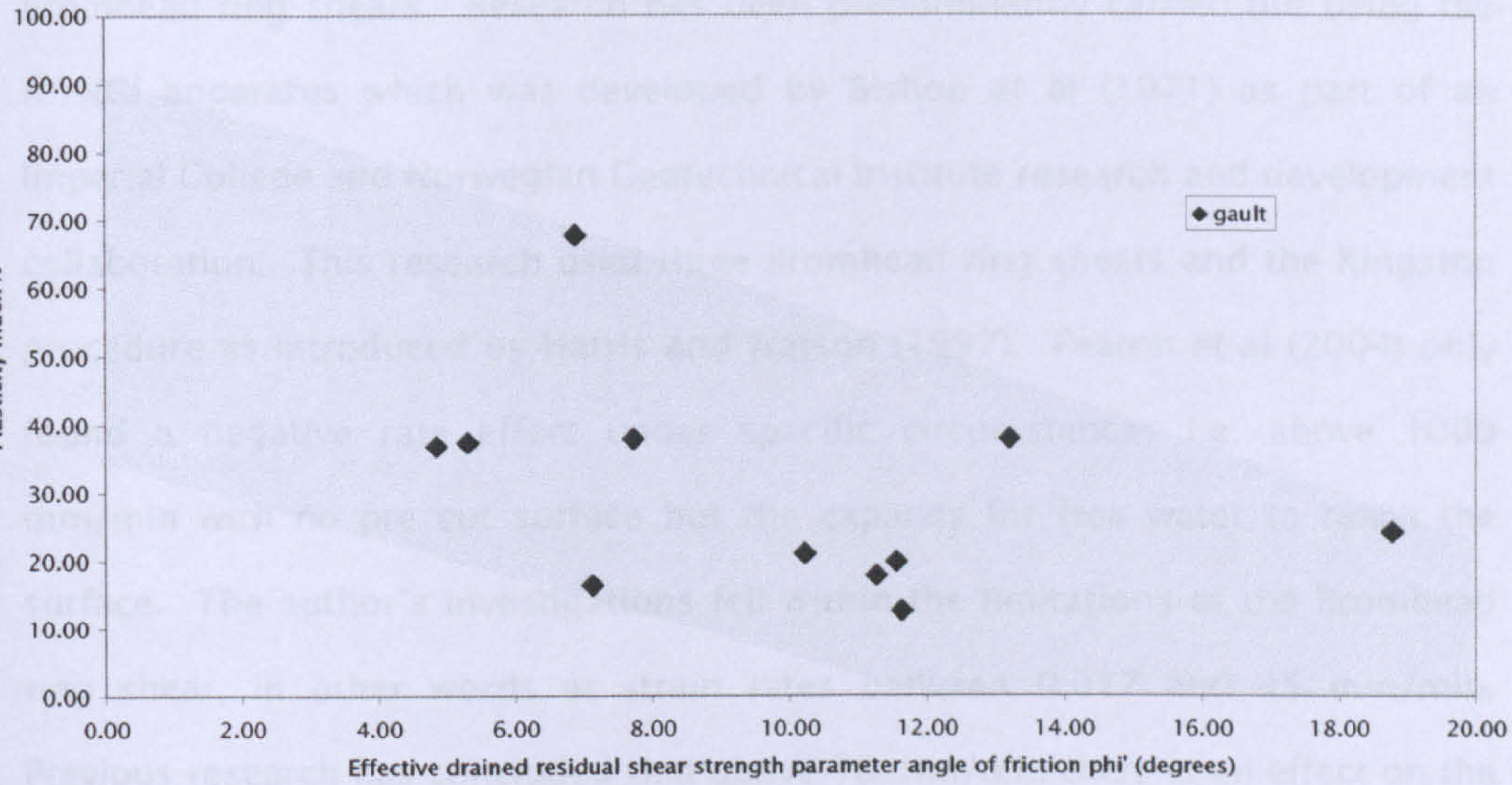


Figure 7.9: The relationship between Plasticity Index I_p and Φ'_r in the gault layer

8 Conclusion

This thesis set out to investigate the objectives outlined within Chapter 1. Chapters 5, 6 and 7 describe in detail the author's findings of these objectives. This chapter links previous research covered principally in Chapters 2 and 3 with the author's discoveries and concludes the results of her investigations.

8.1 Effect of increasing the strain rate.

Explaining why landslides move at the rate they do has been an active area of research since the 1960's. Numerous authors have investigated this area with notable contributions from Lupini (1981), Lemos (1986), Tika (1989), Parathiras (1994) and Taylor (1998). Investigations have been carried out using a range of apparatuses from early beginning using the Hvorslev apparatus and the shear box through the Harvard and Cambridge apparatuses to the IC/NGI and Bromhead ring shears. Research has been predominantly carried out using the IC/NGI apparatus which was developed by Bishop et al (1971) as part of an Imperial College and Norwegian Geotechnical Institute research and development collaboration. This research used three Bromhead ring shears and the Kingston procedure as introduced by Harris and Watson (1997). Fearon et al (2004) only found a negative rate effect under specific circumstances i.e. above 1000 mm/min with no pre cut surface but the capacity for free water to reach the surface. The author's investigations fell within the limitations of the Bromhead ring shear, in other words at strain rates between 0.017 and 45 mm/min. Previous research has concluded that above 10 mm/min there is an effect on the exhibited value of residual strength. The small ring shear is adequate for the purpose for which it was designed i.e. to investigate slow rates of strain primarily on clays which correspond to movement rates of British Landslides. Strain rate effects are associated with tests performed at moderate or high rates of strain which relate in practical terms to fast moving landslides. Noticeable effects are the quantity of soil extrusion and the changes in un-sheared material due to the process. The investigations which are detailed in Chapter 5 indicate the rate effects are an artefact of design and extrusion of soil as shown by the use of marginally different machines. In conclusion the small ring shear is not appropriate for fast strain rate tests due to the amount of extrusion which occurs as a result. This research calls into question whether any other machine which extrudes soil is producing a rate effect or an artefact of design which causes the extrusion.

The author's results in general display a non linear relationship between strain rate and shear stress (taken at a particular normal stress), using the two normal loaded ring shears. An analogous relationship was also observed between strain rate and Φ'_r although specific trends were observed with each of the samples. St. Catherine's Point, Isle of Wight exhibited a trend of increasing Φ'_r with increasing strain rate, therefore a positive rate effect in general whilst at higher strain rates less divergence was observed. Warden Point, Sheppey displayed a positive rate effect. Dromore was more linear and exhibited less rate dependence than the other samples. In conclusion only positive or neutral rate effects were observed but there was decreasing divergence within the results as the strain rate was increased. It was also observed that although the shear stress varied in its relationship with normal stress, it fell within the 95% confidence levels when the strain rate was increased over the stated range. As the normal stress was increased there was a decreasingly linear relationship with shear stress across the range of strain rates and the "R" squared value reduced accordingly.

Conversely, with the Bromhead ring shear, which has been adapted to facilitate pressure, did not exhibit the same trends. In general there was no consistent trend over any of the three samples or within the range of strain rates. St. Catherine's Point gave an analogous trend to the other machines but with a slightly more gradual but still a positive effect. Dromore gave the opposite response therefore a negative effect. Warden Point had no recognisable trend but displayed a mixture of negative and positive effects.

In conclusion, positive, neutral and negative rate effects exist, although negative effects may be restricted to certain apparatus set ups and specific samples, as Fearon et al (2004) implied. Therefore the "rate effect" may not be due to the increased strain rate but due to some other contributing factor.

8.2 Factors affecting residual strength

Investigations were carried out into the effects of apparatus design, clay plasticity and discontinuous strain rate during testing for residual strength. These ring shear tests were undertaken using one very highly plastic clay, one highly plastic clay and one clayey till of high plasticity - Warden Point, Sheppey, Kent; St. Catherine's Point, Isle of Wight and Dromore, Northern Ireland respectively. Previous research which has been carried out into these areas is summarized in Chapter 3.

Apparatus design and its effects were investigated using the three Bromhead ring shears all of which are slightly different. Machine 1 is the oldest of the three with a rough internal plate but normally weight loaded; Machine 2 has a grooved internal plate but otherwise is identical to Machine 1; and Machine 3 is identical to Machine 2 but it has been specially adapted by Wykham Farrance to facilitate pressure loading as well as conventional weight loading as a source of normal stress. The author concludes there are two facets to apparatus design which affect the residual shear strength: the method of normal stress application and the design of the internal plate. Chapter 5 Section 7 describes how, when a different design of internal plate is used, a different trend is achieved. During the tests the rough internal plate gave a slightly positive effect, the grooved a slightly negative effect. This would indicate that the type of internal surface within the ring shear itself could affect the outcome of residual strength. These general trends were observed over the range of samples used but more specifically the different samples were affected to different degrees by the plates and therefore the effect is likely to be plasticity dependent. Examining the three sample's results showed the moderately plastic clay of St. Catherine's to exhibit the least response to the internal plates. Internal plates in the Bromhead ring shear are in direct contact with the shear surface as it forms just below the top platen consequently the change of texture would produce a response, leading to a different result to than expected. By using the two machines and taking an average the effect is positive since the rough positive effect cancels the grooved negative effect.

To apply normal stress by pressure, the apparatus is designed differently to the conventional set up. Pressure is applied directly on to a load arm which bisects the sample whereas conventional techniques have a 10:1 lever arm configuration. From the results of the investigations discussed within chapter six, it can be concluded that there was less disparity the achieved Φ'_r for normal stress applied by pressure than by traditional weight loading techniques as the strain rate was increased. In general Φ'_r for both procedures fell within an acceptable margin of each other.

The samples were categorized by plasticity index and the different responses were noted. Dromore is a clayey till and often displayed the most observable response. For the internal plates and normal stress application methods, it is Dromore which exhibited the greatest degree of dependence despite exhibiting little response to change in strain rate. St. Catherine's Point is a highly plastic clay; of the three this showed the least dependence on internal plates and normal stress application. Overall trends for both were the same and with the normal stress application there was little difference in ϕ'_r. Warden Point, Sheppey exhibited the most confused responses, giving the least positive result for the rough and an inconsistent for the grooved internal plate; the same applies to normal stress application where it gave no fixed trend for pressure and a positive effect for conventional. All this would indicate that the higher the value of Plasticity Index the less consistent the trend of response. Interestingly it was the he clayey till which exhibited the greatest dependence on the testing conditions.

A common practice when using the ring shear test is to start the apparatus at one speed, usually a lower strain rate to form the shear zone, and then increase the rate once the sample has failed. Investigations into whether this procedure actually resulted in the same strength as a continuous strain rate produced a conclusion that whether pressure or conventional the same residual strength is not achieved. The trend was clearer for conventional loading techniques using St. Catherine's Point, with an increase in strain rate resulting in a negative effect and the opposite for a decrease in strain rate post shear-zone formation. For pressure loading almost all the St. Catherine's Point sample gave a positive effect with any change in strain rate. This trend was generally exhibited by both machines with one or two exceptions. Warden Point, Sheppey demonstrated a similar trend for discontinuous strain rate with conventional loading to St.

Catherine's but with pressure, exhibited mainly positive effects with the value of Φ'_r increasing whatever the change in strain rate in most cases. Dromore conversely exhibited, strain rate increases on both loading techniques produced an increase in Φ'_r when compared to continuous and decreases resulted in the opposite. Therefore, the two high plasticity clays produced the opposite response to each other and the very high plasticity clay was inconsistent. In conclusion, one fact is very apparent, changing the strain rate during the test results in a change of value. The only variable is the increase or decrease which appears to have some link with possibly the silt content.

To summarize, apparatus design does affect the residual shear strength, plasticity index affects how design of the apparatus and procedure affect the final value. Changing the strain rate affects the value of the residual shear strength parameter Φ'_r achieved and therefore residual strength. The reason for these affects is linked to pore pressures and the ability for free water to be able to access the shear-zone. Changing the rate post shear-zone formation results in an alteration to the rate at which the shear-zone is removed which affects the final value of residual strength. A more concentrated normal stress results in the pore pressures being reduced as the sample is compressed more and therefore the shear stress is higher due to the removal of the pore pressures therefore resulting in a larger angle of Φ'_r. The degree to which the sample is affected by this depends on the liquid limit. In other words the lower the liquid limit the greater affect on the clay when the pore pressures are expelled.

8.3 Strain rate effects on other properties

During a ring shear test the vertical displacement is monitored throughout the test. The effect of increasing the strain rate results in an increased amount of soil extrusion especially in the NGI/IC and various authors have attempted to minimise this extrusion. Parathiras' (1994) main thrust in his thesis was to develop confining rings which reduced soil extrusion. Eventually he succeeded in limiting the amount of soil loss but could not prevent it entirely. Other aspects which have been investigated are the role of pore pressures (Fearon et al, 2004). Post test moisture content compared to pre-test, plastic limit and liquid limit revealed a trend of increasing moisture content after the test with increasing strain rate overall.

Displacement in this programme was analysed in terms of maximum displacement, vertical compression during loading and the displacement rate as indicated in Chapter 5. The linearity of displacement rate was also investigated and it was concluded that increasing the strain rate exhibited no substantial effect on this relationship. Displacement rate, that is vertical compression divided by horizontal movement, was found to exhibit an inverse trend. This is actually a result of the horizontal displacement increasing at different rates and therefore the amount of vertical compression although larger takes place over a longer distance and therefore appears to be smaller. Maximum vertical movement increases with strain rate due to the effect of the greater number of rotations resulting in more soil extrusion, therefore it is to be expected that the larger vertical displacement occur at the higher rates. Vertical compression during loading is defined as the amount of load applied between the second load being uploaded and the last load being downloaded to the original normal stress. In other words this is a constant time frame for all tests. The horizontal displacement is not taken into account just the amount the sample compresses during these particular stages of the test. At the lower speeds the change in strain rate had little effect. Conversely at the higher rates there is a definite increase in the amount of compression during this time. Increases in normal stress combined with strain rates mostly above 10 mm/min result in large quantities of soil extrusion as the sample is almost ground down by this combined effect.

Post test moisture content exhibits a trend of increasing with strain rate. The lower rates display a maximum of 25% increase in moisture content post test. There is less vertical displacement at these rates indicating a greater percentage of the actual soil sample remains. Analogous to St. Catherine's Point sample, Dromore and Warden Point, Sheppey exhibited trends of increasing post test moisture content with strain rate. Conversely at the higher rates there is a greater percentage increase of moisture content and the samples go from just above their respective plastic limit to above their respective liquid limit. This is because at higher rates there is less actual soil left so the water content increases proportionally which would result in a reduced shear stress. Dromore and Warden Point, Sheppey demonstrated similar results thus indicating that in high rates the test is not a drained effective stress test but more towards an undrained total stress test. This is due to the proportions of soil and water changing which affects the movement of the pore pressures in the sample. The increase in water content could be due to the gap via which the soil is extruded, because water from the water bath must be able to get to the sample; therefore the change in pore pressures occurs. Other increases could occur when the normal stress is reduced, which dilates the sample and therefore water further enters the equation.

Overall the investigations into rate effects indicated that the increases in strain rate produce a change in pore pressure. These changes result at higher rates in the test being undrained instead of drained and therefore becoming a total stress test instead of an effective stress test.

8.4 Normal Stress

The concept of normal stress applied using pressure, as opposed to regular weights, was initially introduced so testing could be undertaken on ships. As an extension to this idea, continuous gradual loading of normal stress arose from Dixon and Bromhead (1991, 2002) investigations on Warden Point, Isle of Sheppey, based upon the natural process of restoring equilibrium. Landslides evolve in response to toe erosion, mass movement and surface erosional processes but they actually follow a specific stress path which is akin to the laboratory test following a failure envelope. Step-less control of normal stress not only allows a detailed examination of the stress path but semi automation of the test as well. This research overall indicates that there is negligible curvature and therefore implies that residual cohesion exists in some tests. Semi-automation would be a significant advantage in a commercial laboratory because it reduces labour intensity of the test. By reducing the size of the steps in the loading, it becomes a virtually continuous process and therefore is an extension to the Harris and Watson (1997) procedures which with removal of the BS specified rest periods avoid the problems of processes such as re-cementation (Angeli et al., 2004).Continuous loading takes account of the slow nature of this process whereas discontinuous loading only indicates specific points. Bromhead (2004b) cited this as one of the positive features of this process.

The investigations into the effect of pressure noted that within a small margin the values of shear stress achieved using pressure applied normal stress were the same as for conventional weight loading. This small variation is a consequence of the non homogenous nature of the clay. Discontinuous strain rate under pressure loading as opposed to conventional loading, achieved different trends to those previously observed for the traditional methods of normal stress application. However, there was still a disparity between the Φ'_r achieved and the one which is attained if the rate remains constant, thus confirming that if the rate is altered after the shear zone is formed, the resultant Φ'_r is not consistent with an unchanged rate post failure.

Comparisons between discrete and continuous loading using both the St. Catherine's Point and Warden Point samples indicated the role of plasticity index in the dependence upon the rate of load application and to how closely continuous correlated to discrete loading. St. Catherine's Point sample exhibited a stress path which returned by almost the same route with 17kPa/hr application and 34kPa/hr removal of normal stress. Similarly the sample from Warden Point displayed a trend which used to not quite the same degree, the same stress path for both application of load at 6kPa/hr and removal of load at 5kPa/hr of normal stress. This would indicate that plasticity index has a role in the rate required for normal stress application and removal, Warden Point having the higher value of plasticity and requiring the slower rate.

Rate of normal stress application and removal during continuous loading investigated the shape of the stress path and whether the same path was followed in both directions. St. Catherine's Point displayed a gradual, almost horizontal relationship, in other words on a minimal change in shear stress compared to any change in normal stress for 17kPa/hr up 67kPa/hr down. As load was applied a slightly curved response was achieved, on removal this "zigzagged" above and below the line. Conversely, at the slower rate of 17kPa/hr up 34kPa/hr down a steeper gradient of 1 in 9.15 compared to 1 in 23.7 at the faster rate. A value of 0.98 "R" squared was also achieved at this slower rate compared to 0.81 thus indicating a linear relationship as opposed to a more curved relationship. Warden Point, Sheppey produced a consistently linear relationship between shear stress and normal stress with a linear trend value exceeding 0.9 in terms of "R" squared details of which are shown in Table 8.1. Even at the slowest rate, Warden Point sample did not replicate the same path when the load was removed compared to when it was applied. Cohesion values were quite high for four out of the five which could indicate Maksimovic's (1989, 1996) curved envelope but this would require investigation of normal stresses of below 100kPa. The sample from Warden Point produced more consistent values of Φ'_r; with a range of 0.25° and with the lower rates within 0.17°. St. Catherine's Point had a range of 3.81° over the two rates.

Overall continuous loading is a more realistic model of natural processes as Bromhead (2004b) notes but the rate at which the loading is applied or removed appears to be dependent upon the plasticity of the clay, as there is an indication of a degree of rate sensitivity. Conventional loading at discrete intervals gave values of 7.55° and 8.36° for St. Catherine's Point and Warden Point respectively at the strain rate at which these tests were undertaken (0.036 mm/min). This is a further indication that strain rate is not the sole factor which affects or determines residual strength. Pressure loading at the same strain rate gave value of 8.2° for St. Catherine's Point, Isle of Wight and 3.39° for Warden Point, Sheppey. Nevertheless, clay is not an homogenous material and there is variability in the value of Φ'_r within even the same sample.

Furthermore, the investigations on settlement and post-test moisture content indicate further sensitivity to the rate of application and removal of normal stress. There are also indications that plasticity index has an affect on the rate required to replicate the same stress path in both directions.

Continuous loading is subject to following:-

- Rate of load sensitivity.
- Plasticity index.
- To the quantity of sample available.

Care needs to be taken not to exhaust the amount of sample available before the end of test. In conclusion this does as Bromhead (2004b) replicate natural processes but requires further work to achieve a method of rate of load selection to ensure the correct path is followed.

Sample	Rate of Application (kPa/hr)	Rate of Removal (kPa/hr)	RSQ = "R" squared	Cohesion (kN/m^2)	Gradient	Φ' (rads)	Φ' (deg)
St. Catherine's Point	17	34	0.98	8.1	1 in 9.15	0.109	6.23
	17	67	0.81	34.3	1 in 23.7	0.042	2.42
Warden Point	6	5	0.98	10.8	1 in 8.49	0.117	6.71
	17	34	0.93	0.93	1 in 8.29	0.120	6.88
	17	67	0.97	12.1	1 in 9.62	0.104	6.53

Table 8-1: Results summary for Step-less loading

8.5 Variability within a sample

Back analysis gives the impression that samples are homogeneous in nature. The results of James (1970), Bromhead (1978) and Dixon & Bromhead (1986) all form a straight line relationship therefore indicating a degree of homogeneity in London clay (Figure 6.1). Hutchinson et al (1980) indicates the same for the gault and Chandler (1974) for the lias.

In actual fact there is evidence of the variability as Bromhead et al. (2000) discovered within a fairly small section of London clay and within this research at Ventnor in the gault. Statistical parameters were suggested by Bromhead et al. in the form of a standard deviation of 1° for ϕ'_r; this research suggests a standard deviation of between 2.5° and 4° for ϕ'_r dependent on the location of the borehole so 4° overall within the gault at Ventnor for ϕ'_r. Back analysis is an average value and hence the reason for the convergence as shown in Figure 6.1 on the single envelope thus removing the innate variability of the clay. Therefore the clays are taken to be homogeneous but since the clay is a natural material and not man made, homogeneity is an unlikely concept.

London clay, is inaccessible to a large extent, huge areas of it are concreted over. This makes investigations of London clay difficult and only possible either at the coast or at a select number of locations within the basin. Conversely, the gault on the Isle of Wight due to the lack of development and small resident population is accessible. Access for this research was part of a Landslide Hazard Study by South Wight Council and other groups. Ventnor is on an active landslide and therefore constantly monitored along with the majority of the Undercliff, which makes up most of the South Coast of the Isle of Wight, for any movement. Therefore there is no shortage of samples to demonstrate the innate variability of the gault which is the complete opposite to London clay. Overall it is wrong to assume homogeneity in a natural material and clay is no different.

Bromhead et al (1999) examined variability within a sample of London clay. They investigated the effect of increasing normal stress. Likewise, Chapter 7 investigates the variability within a sample. From three boreholes at various depths, a range of samples were taken. Various properties were analysed to investigate how even in the same layer these varied. For the gault layer, Figure

8.1 demonstrates how the most variable property is liquid limit and this tends to increase as it gets closer to the top of the stratum. Plastic limit, to a lesser degree, does the same. Shear strength parameter Φ'_r increases the closer to the top of the stratum it is. Similar results were exhibited for the shear strength parameter, Φ'_r in Chapter 7 using a smaller population of samples which Kingston tested as part of the overall programme. Plastic and liquid limits tended to increase with depth, which indicates how even using the same sample the variability that is achieved in the results. Conversely, in the sandrock layer, Φ'_r decreased slightly with depth whereas the other properties all increased with depth. Sandrock displayed a kind of uniformity that the gault layer does not.

Figure 8.1 from Halcrows; which is the complete results from the investigation (Chapter 7 contains the results from samples which Kingston analysed), demonstrates how varied the samples properties are even at the same depth. The Halcrow's graph confirms the non homogenous nature of the material which is known as clay.

8.6 Summary

Residual shear strength is affected by the apparatus design, soil extrusion and by the way the load is applied. In addition the effect of non homogeneous nature of the material itself has its own implications. Clay plasticity has the effect of changing the sensitivity to the above factors. The lower the plasticity index the less dependence on strain rate but the higher dependence on load application and apparatus design.

Post test moisture content increases with strain rate which indicates that the movement of water changes with strain rate. At the fastest rates this is above the liquid limit which indicates a change from drained to undrained which obviously will have an effect on the shear strength parameter ϕ'_r as the test becomes a total stress test and not an effective stress test. Applying the normal stress at a continuous rate of loading reduces the quantity of soil extrusion and obviously the disruption to the sample. In addition it is more realistic as replicates the landslide stress path.

In conclusion the small ring shear apparatus is excellent for what it was designed for; the analysis of British Landslides. In other words tests run at slow strain rates, where the soil extrusion is virtually non existent and therefore the problems do not arise. At moderate and fast strain rates the soil extrusion is increased which results in changes in pore pressures and this has implications on the nature of the test. Continuous loading of normal stress reduces soil extrusion and therefore makes the investigation of moderate and fast strain rates more practical and removes the influence of stress changes. This would therefore make the small ring shear more practical for these investigations.

The question still remains though as soil extrusion is unavoidable is any apparatus which extrudes soil suitable for investigating moderate and fast strain rates. Answer is of course; no but with the use of continuous loading, thus with the removal of the implications of stress changes would therefore increase the suitability of any such apparatus for these investigations.

Rate effects are caused by soil extrusion and the associated pore pressure changes which are artefacts of the design of the apparatus. These effects are then perpetuated by stress changes and the non homogeneous nature of the clay.

"Value" is:
degrees for ring shear ϕ',
and % moisture content
for Atterberg limits

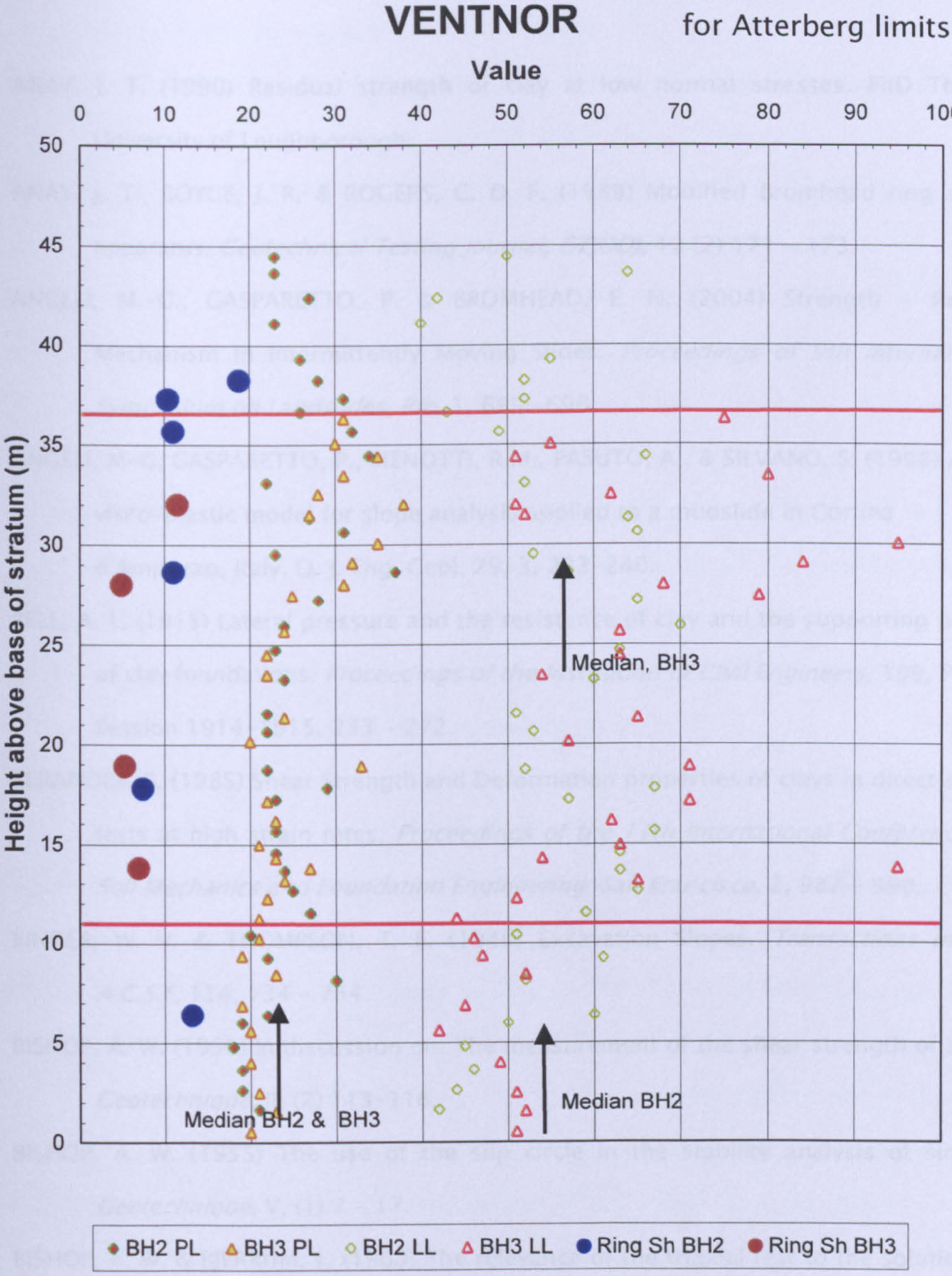


Figure 8.1 **Plasticity characteristics versus height above base of stratum for Gault**

(After Halcrow, 2003).

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Appendix A

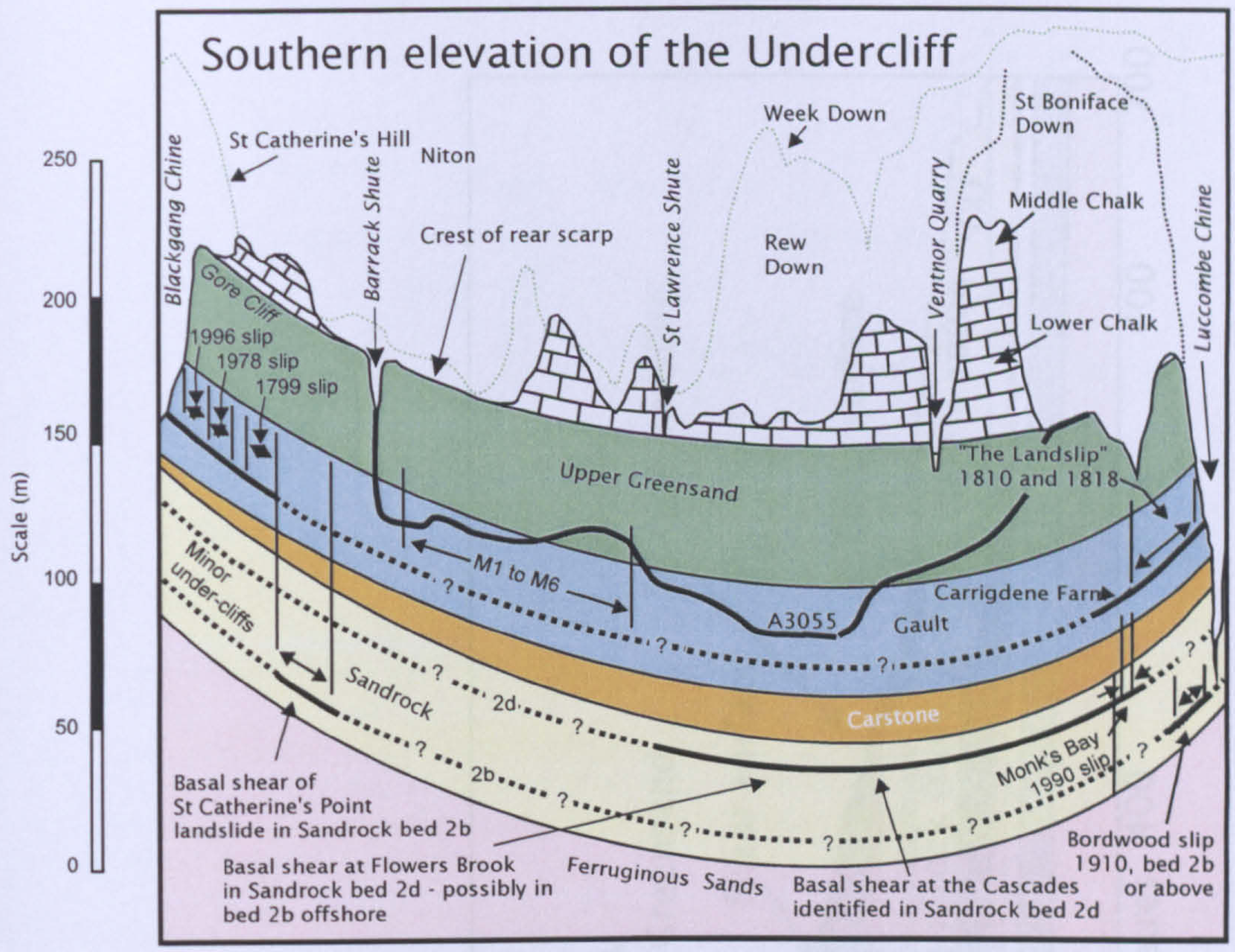

Figure A.1 Southern Elevation of the stratigraphy of the Ventnor Undercliff, of the Isle of Wight. (Based on Hutchinson & Bromhead, 2002). This shows the locations of the principal clay beds in the Gault and Sandrock (Lower Greensand), tested in this study.

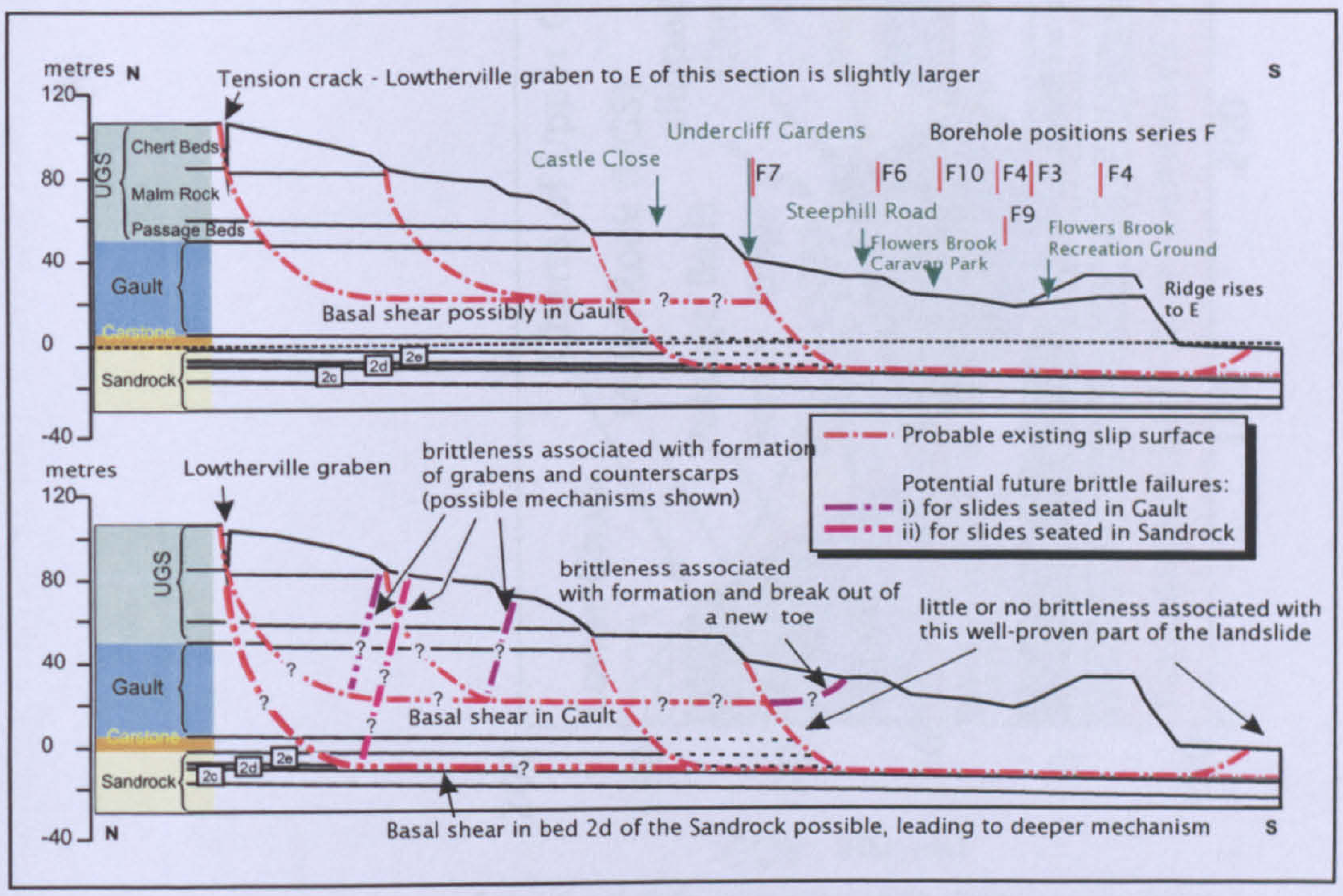

Figure A.2 The context for drilling boreholes which provided the samples was to determine which of the above mechanisms was most likely to be the dominant one, and hence how the Undercliff in west Ventnor was most likely to develop.

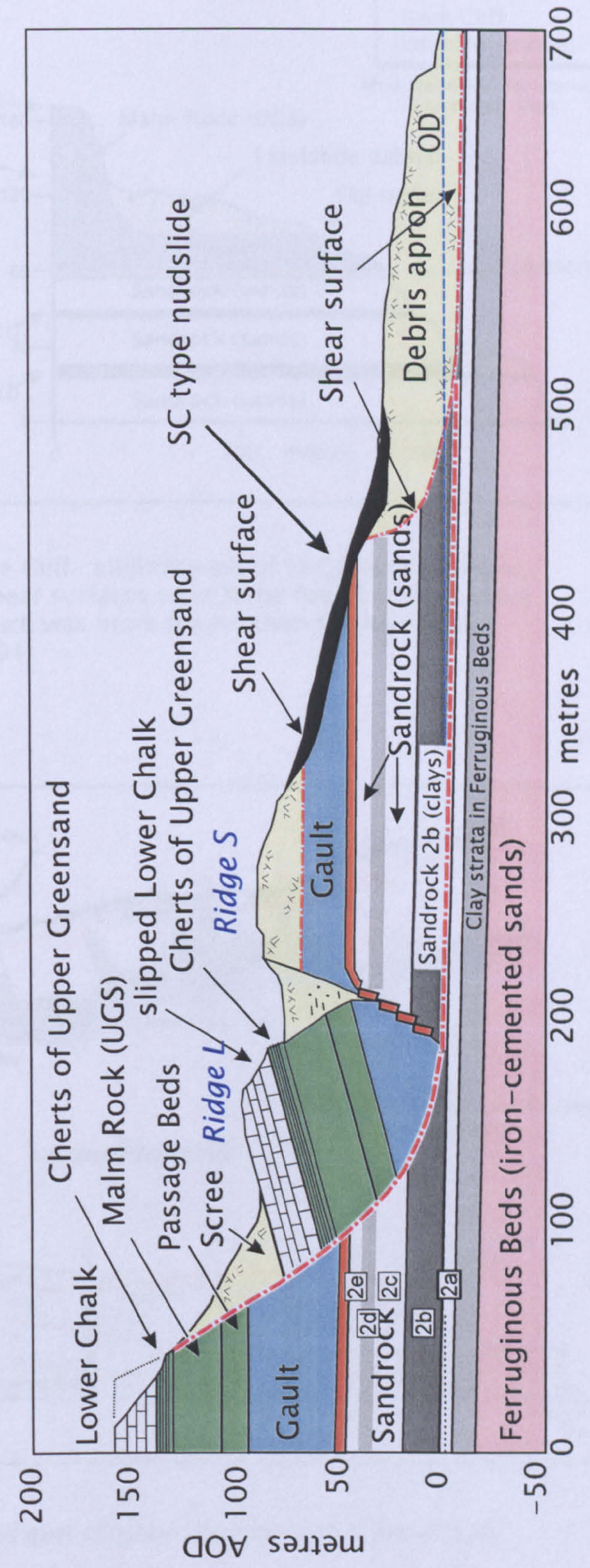

Figure A.3 Main cross section of the landslide complex at St Catherine's Point (Hutchinson et al., 1991; Hutchinson & Bromhead, 2002). The original investigations showed that the basal shears were located in the clay beds of the Sandrock.

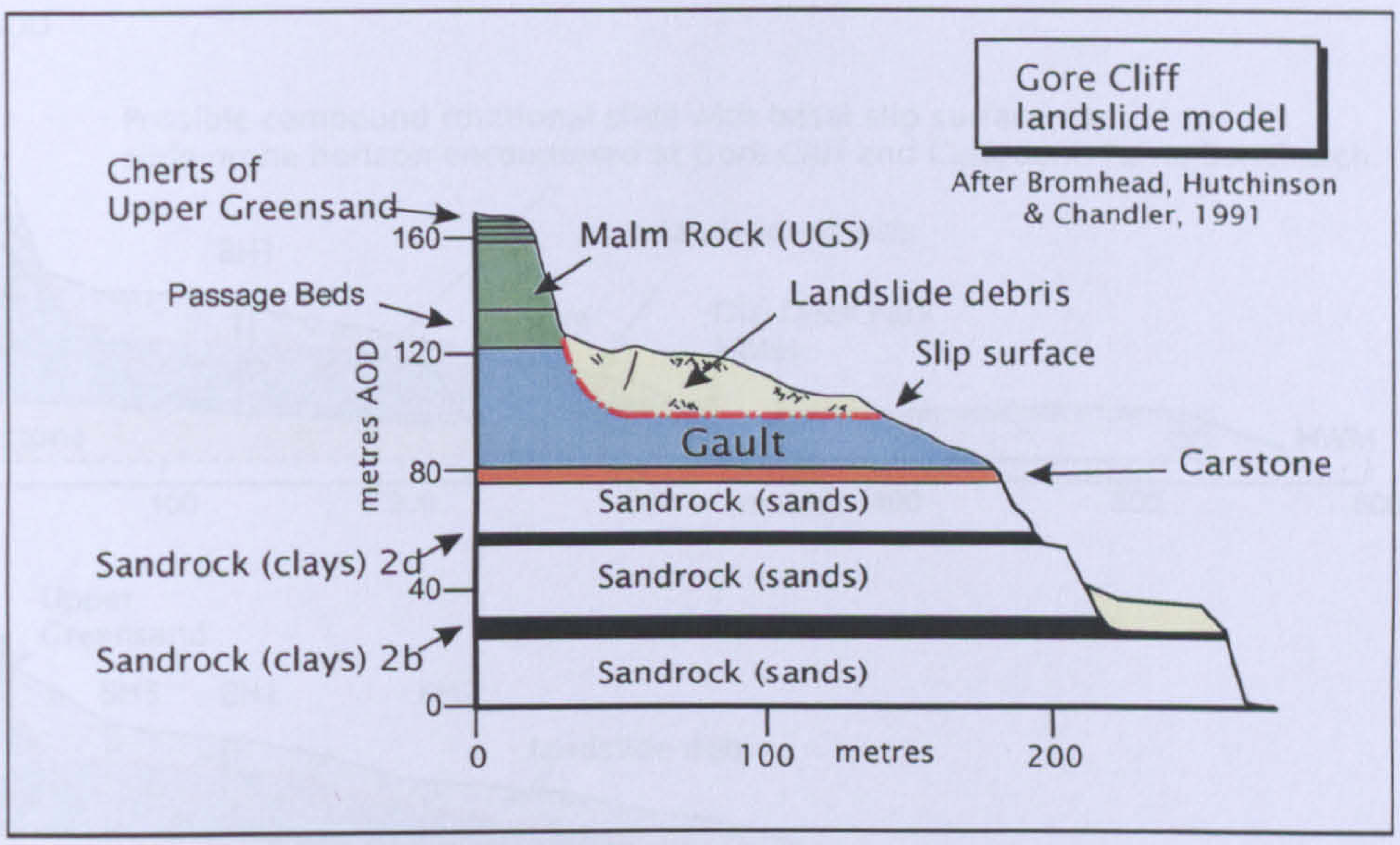

Figure A.4 Investigations at Gore Cliff, slightly west of St Catherine's Point, showed that basal shear surfaces were to be found in the central part of the Gault, which was more plastic than the lowest part (Bromhead et al., 1991)

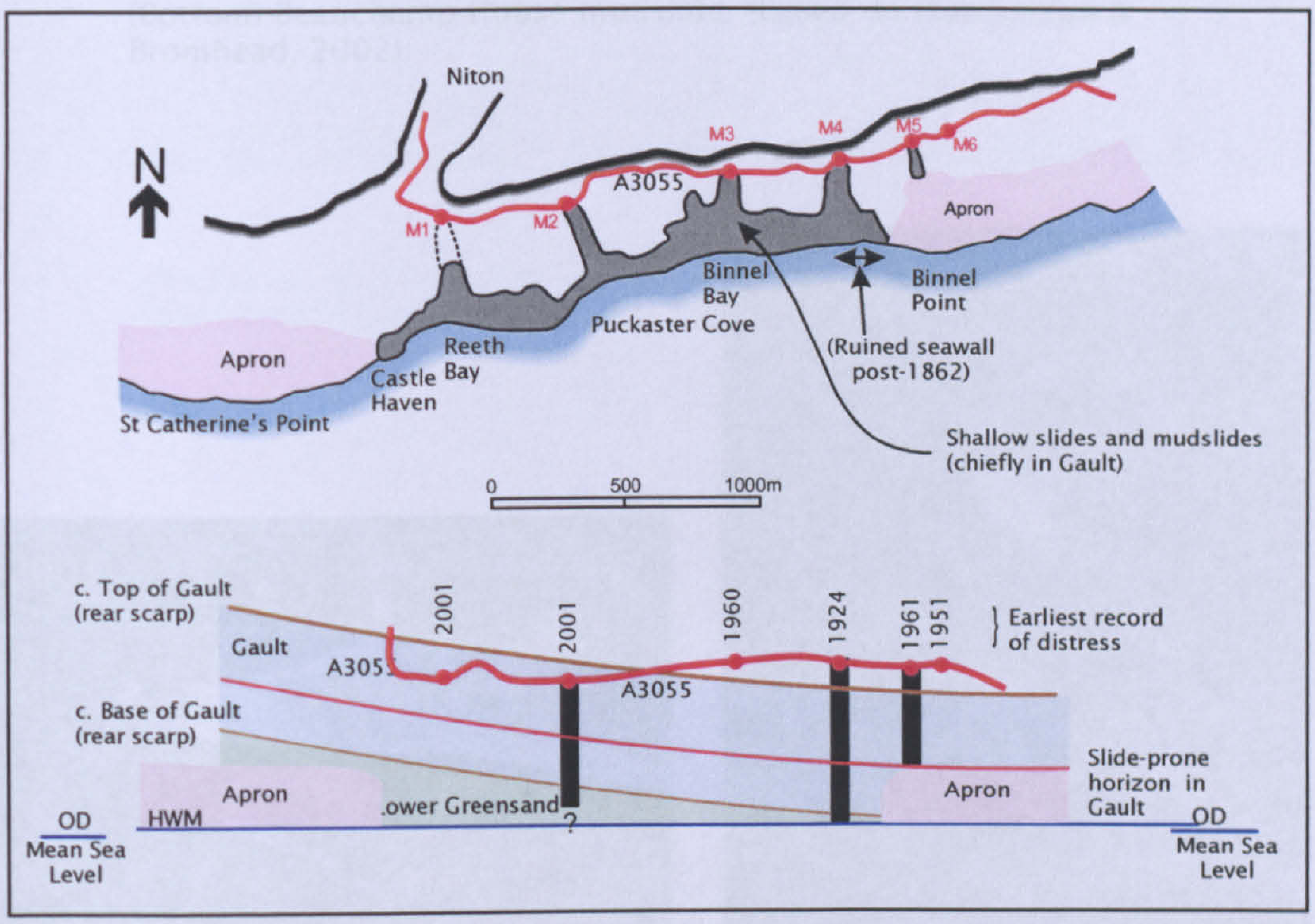

Figure A.5 Location of mudslides east of Niton. (Hutchinson & Bromhead, 2002)

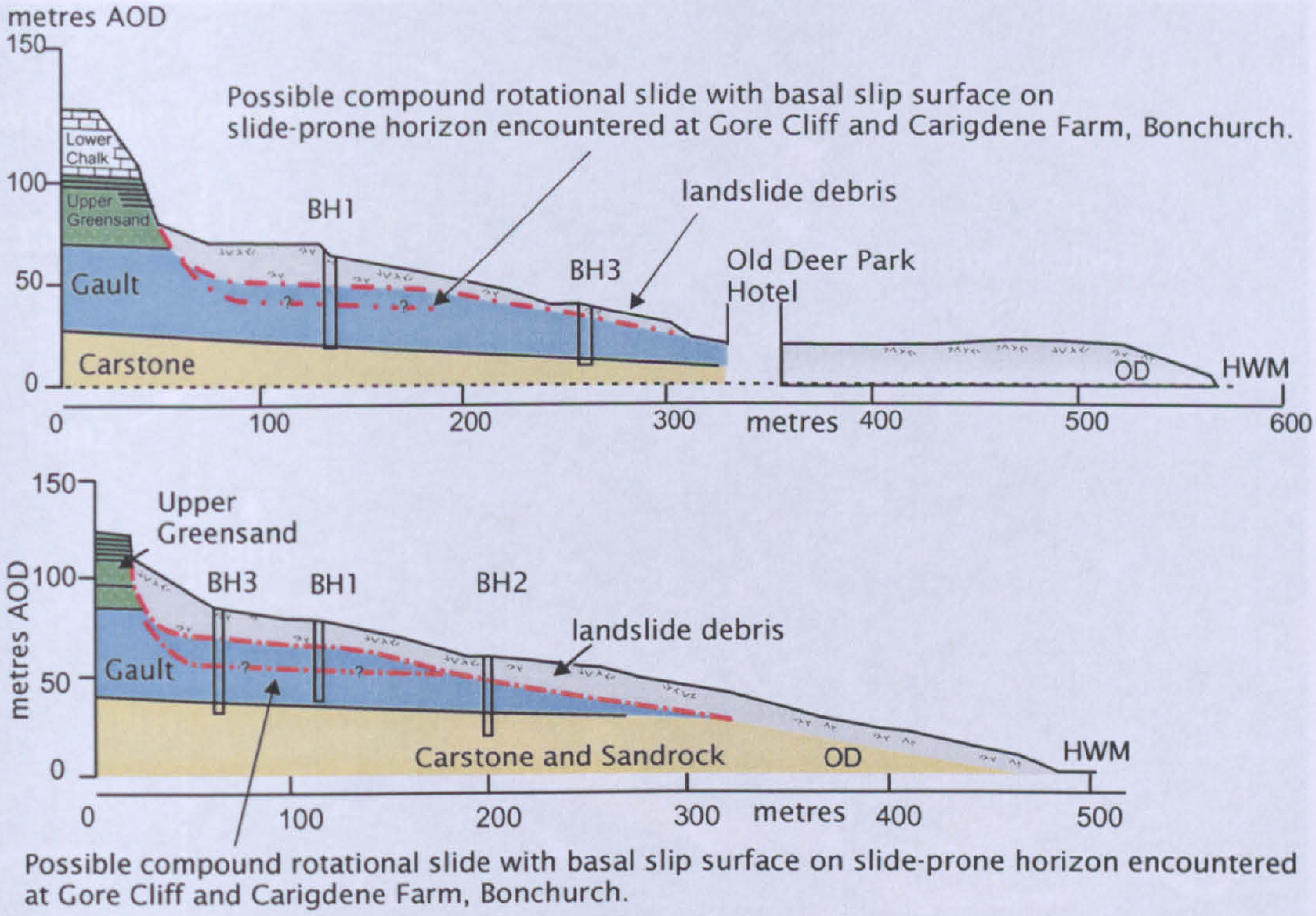

Figure A.6 Mudslides active in the winter of 2000-1, originating in the more plastic upper part of the Gault. (Top) Woodlands mudslide. (Bottom) Beauchamp House mudslide. (Based on Hutchinson & Bromhead, 2002).

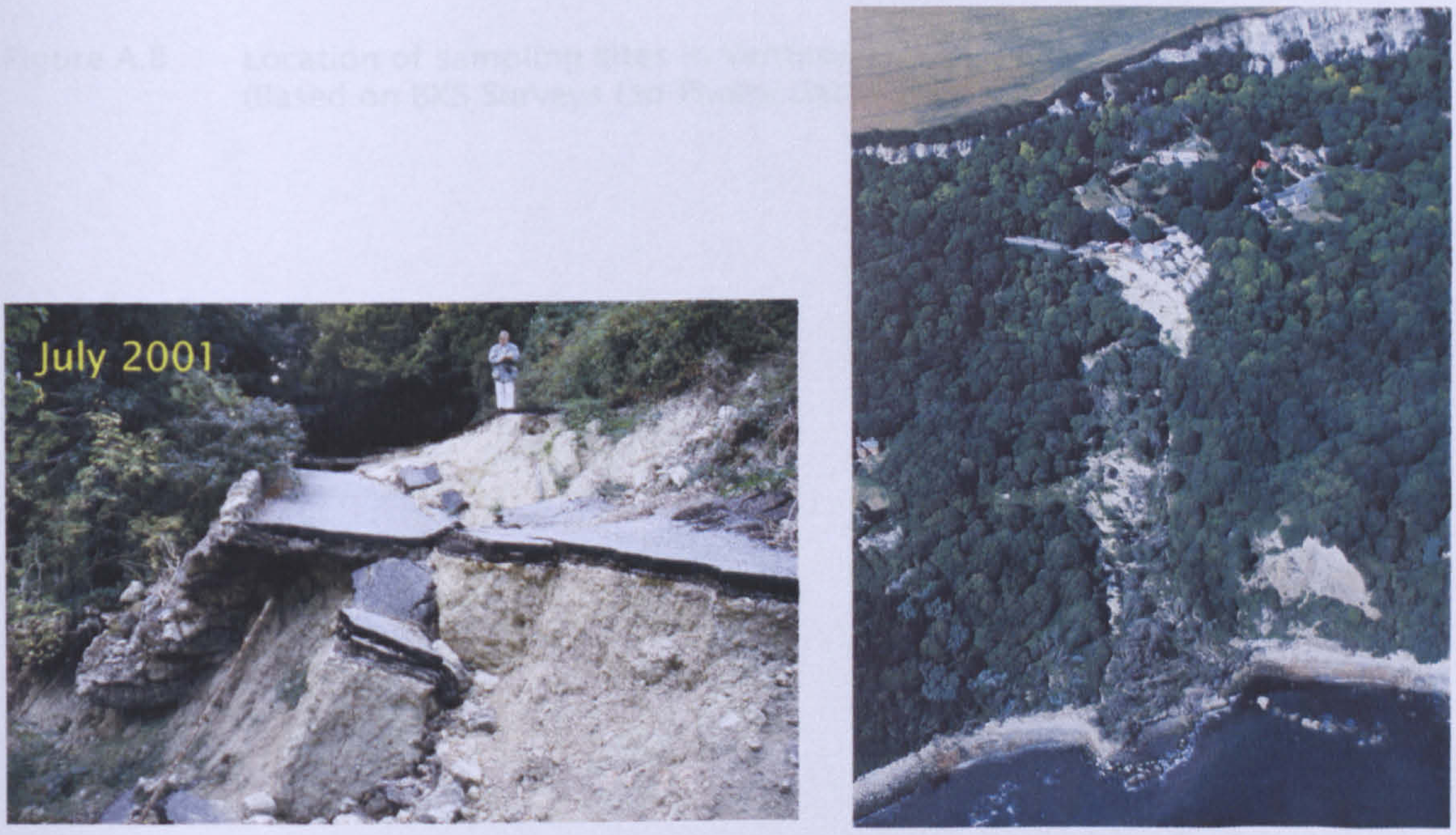

Figure A.7 (Left) Main scarp of Beauchamp House slide. (Photo E. N. Bromhead. (Right) Aerial view of Woodlands mudslide (Source unknown).

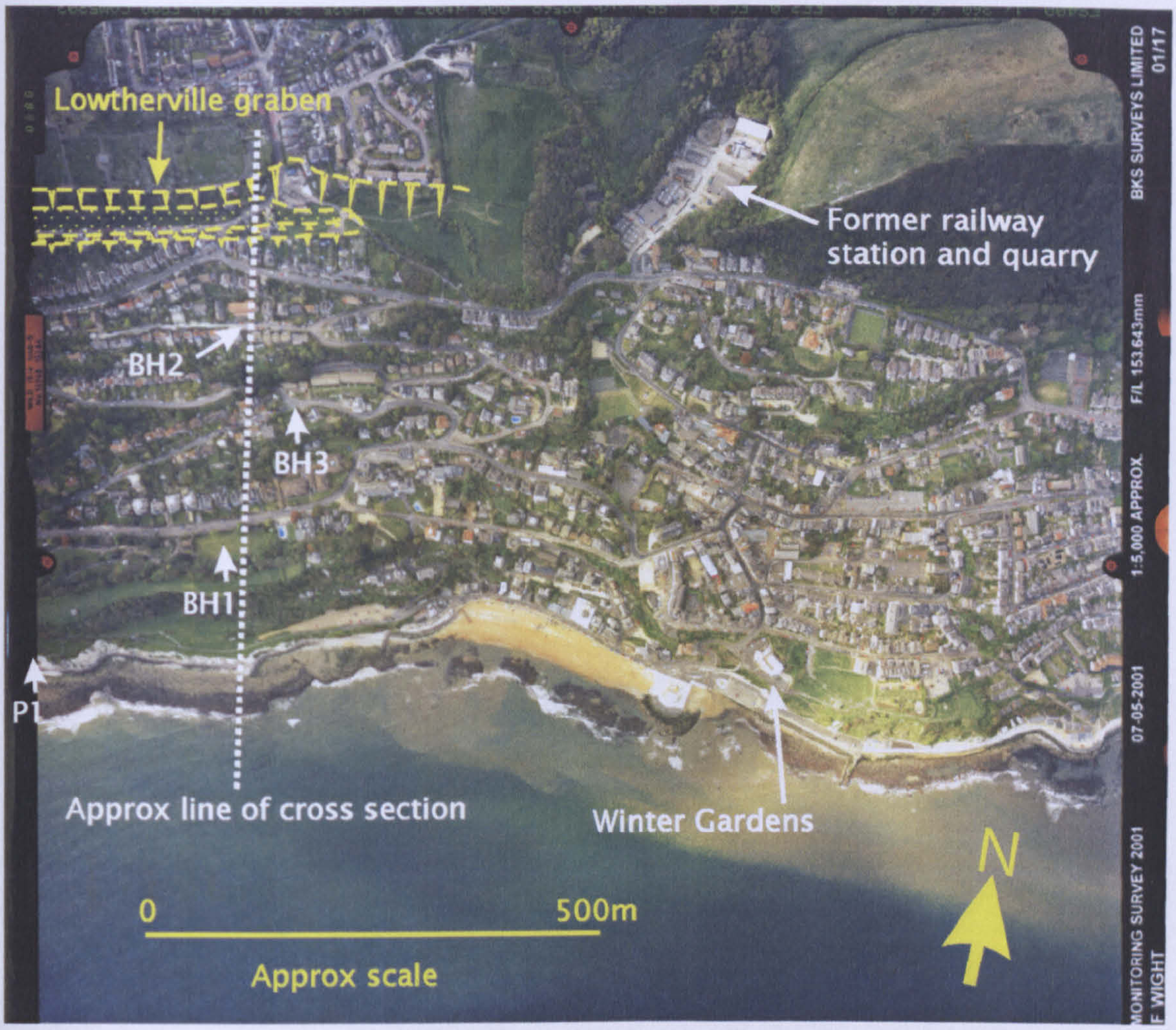

Figure A.8 Location of sampling sites in Ventnor, Isle of Wight. (Based on BKS Surveys Ltd Photo, dated 2001).

Appendix B

BASAL INCORPORATION IN MUDSLIDES AT THE SLIDE-FLOW INTERFACE – SOME REMAINING QUESTIONS

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ABSTRACT: A process termed *basal incorporation* (Hutchinson, 1970) has been proposed to explain aspects of the field behaviour of mudslides. It is one of a number of aspects of mudslide behaviour identified as a result of research into coastal mudslides in the UK. This paper reconsiders the mechanisms for the process of basal incorporation, and the practical significance of the process is discussed.

Keywords: Mudslide, Mudflow, Basal Incorporation, Basal shear.

1 INTRODUCTION

Hutchinson (1970) describes a set of investigations carried out to determine the mechanics and physical properties of a coastal mudflow on the North Kent Coast. One of the principal findings of this research was that movements took place as sliding along a basal shear surface. As a result of this finding, he has successfully made the case for changing the taxonomy of such movements from mud*flow* to mud*slide* – although since the latter is used as a "catch all" term for mass movement in the North American tabloid press, this classification does not please all.

The second of the main findings of Hutchinson's (1970) study was that significant artesian excess pore water pressures existed within the mudslide body. These pore pressures are generated within the head of the mudslide by *undrained loading*, where small slips and falls of debris onto the head of the mudslide induce stress – related pore pressure changes. The theme of undrained loading was further explored in detail by Hutchinson,Bhandari (1971), and by Hutchinson *et al.* (1974). Today it is taken as axiomatic that undrained loading is a fundamental mechanism of mudslide activity and mechanics. Figure 1 shows a sketch, based on a Figure in Hutchinson, Bhandari (1971), which describes the mechanism, and Figure 2 shows a simplified version of Hutchinson's Beltinge mudslide (current terminology).

The third of the main findings of Hutchinson's study, which originates in his earlier surveys of coastal landslides in SE England, is that mudsliding tends to be the dominant form of mass movement in cliffs subject to moderate rates of marine erosion. According to this scheme of things, rotational landsliding was the response to rapid marine erosion, mudslide activity the response to moderate rates of marine erosion, and where toe erosion was very low or non-existent, for example in artificially defended or naturally abandoned cliffs, mudslide activity would reduce the slopes to a condition of equilibrium. Fourthly, the undrained loading mechanism is used to explain some aspects of the displacement and velocity behaviour of mudslides, for example, surges.

Finally, through consideration of volumes contributed by various processes to the mass transport within a mudslide, Hutchinson concluded that approximately 25% of the volume of a mudslide system was contributed via a process of bed scour, which he termed "basal incorporation". Basal incorporation is a mechanism that is clearly important in promoting parallel retreat of a cliff line.

This paper considers aspects of the field behaviour of mudslides and the details of soil mechanics that lead to the process of basal incorporation.

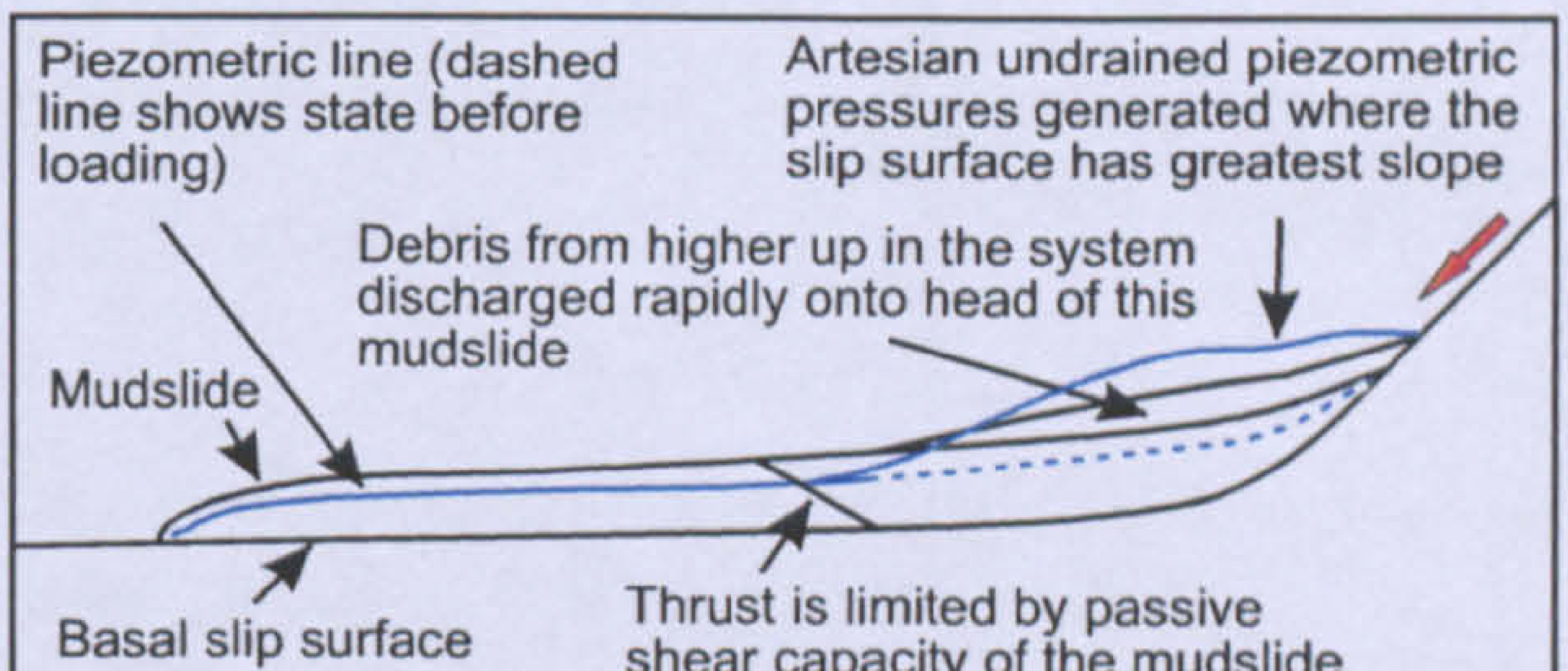

Figure 1. Undrained loading mechanism (Hutchinson, 1970).

2 EXTENT OF THE PROBLEM OF MUDSLIDES IN THE UK

Mudslides are a widespread problem on coastal slopes of clays in many areas of the UK, particularly on the south and east coasts. Detailed examination of numerous elongate mudslides show that they owe their origins to careless disposal of waste water onto sloping ground, in particular waste water from domestic sanitation where this is disposed of *via* septic tank and soakaway systems (e.g. Watson, Bromhead, 2000). However, surface water drainage from roofs and particularly from large paved areas such as car parks can also be a problem. Water from highway drainage may well give rise to related problems if it is not carefully disposed of.

Mudslide activity is also a part of larger scale landslide systems. For example, rotational landslides create landforms which trap and accumulate runoff, concentrating it at specific discharge points which often develop local, parasitic mudslide activity in response to the increased water contents in the subsoil at these locations (see, for example, Bromhead, 1979 or Bromhead *et al.*, 1998). Where the water contents rise beyond a critical level, the soil is unable to develop slickensides, and it is thought that flow takes place on a surface above the basal shear. The mass movement then becomes a mudflow proper. When the water input ceases, the slurry gains strength, and reattaches to the basal shear.

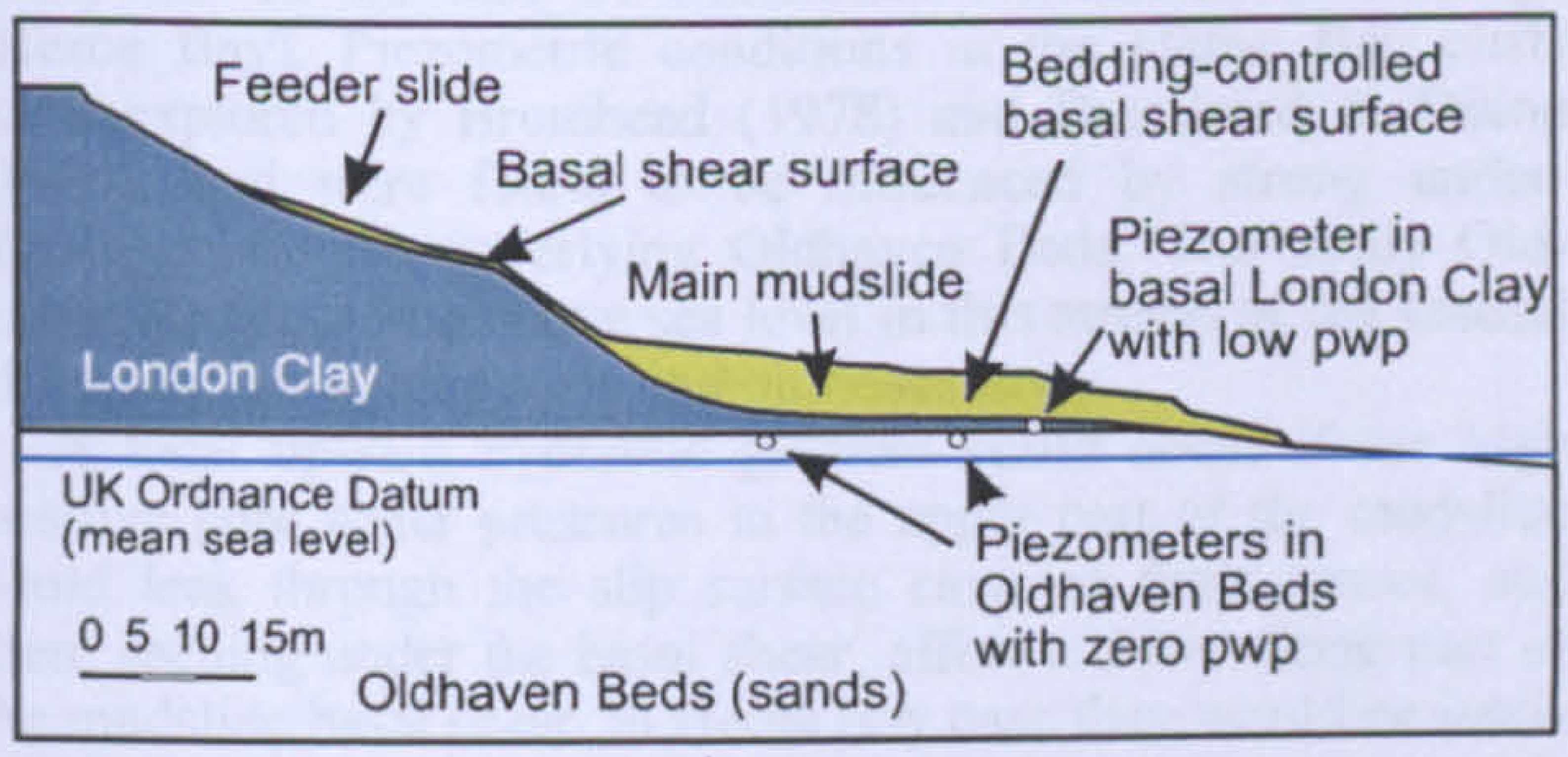

Figure 2. Hutchinson's Beltinge Mudflow (mudslide) Flow II shown in a simplified cross section (Hutchinson, 1970)

Most of the landslide prone coastal areas in south and southeast England are developed in interbedded sequences of clays and limestones or sandstones. These sequences have gentle dips as a rule, often less than 2 degrees, and may dip landwards or seawards, or along the coast, or in some combination of these directions. Complex landslides with basal shear surfaces following a particularly weak bed in the sequence are common.

Where the geological structure permits, e.g. the Undercliff of the Isle of Wight (Hutchinson, Bromhead, 2002), related complex landslides occur for many kilometres along the coast, forming benches and terraces. Equally common are terraces developed from mudslides, again with a common basal shear. It is not clear whether these develop along a preferred basal shear in the same manner as do the compound slides, and from then on, there is no basal incorporation, or whether they develop initially at a higher elevation and end up at the common basal shear by processes of basal incorporation which can go no further. Clearly, if the latter process is the dominant one, basal incorporation is an important mechanism. Figure 3 shows a double cascade of mudslides modelled on the examples to be found in the cliffs east of Bournemouth and Christchurch.

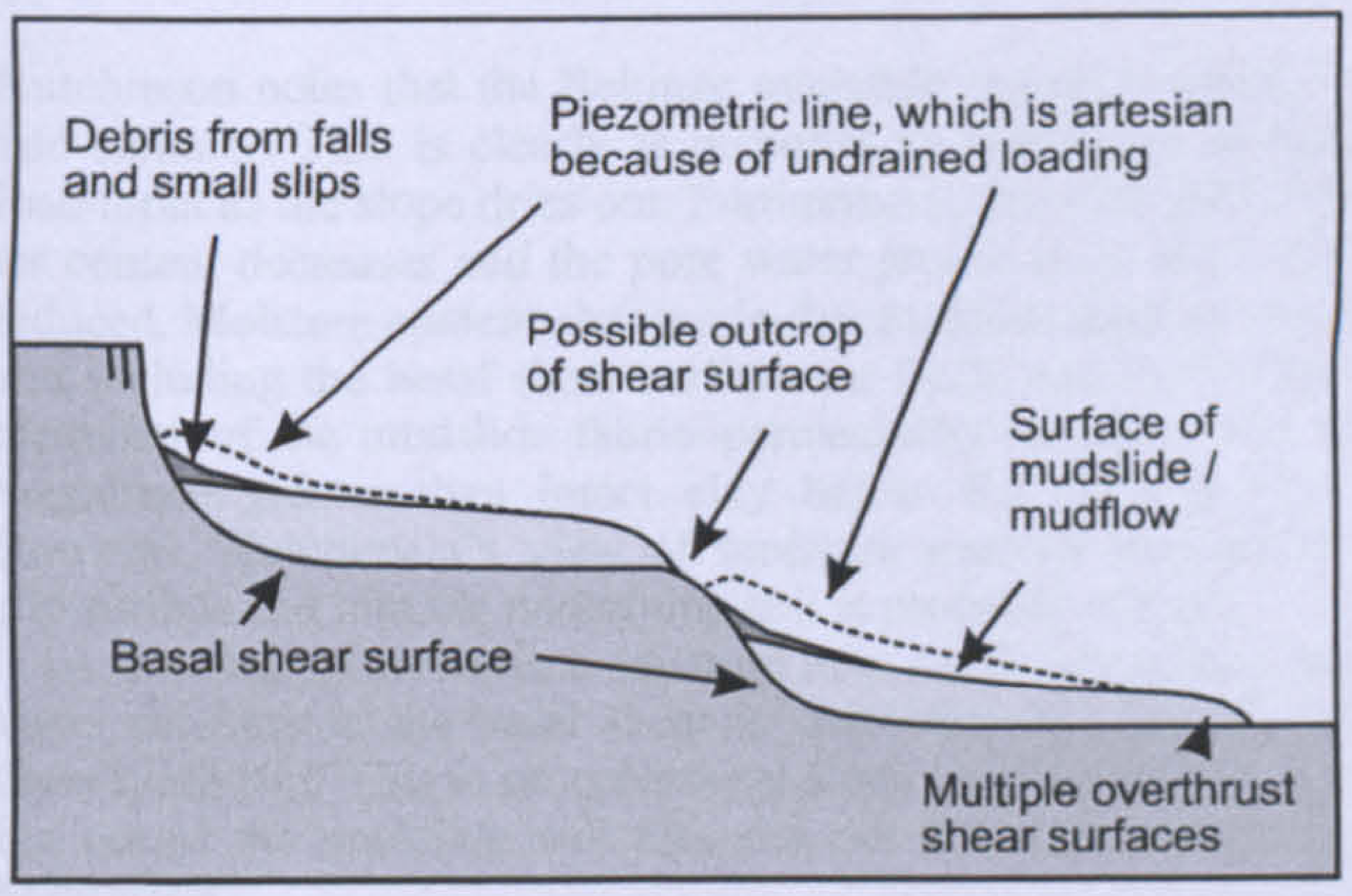

Figure 3. A mudslide cascade, with sequences of induced undrained loading.

3 THE PROBLEMS

There are a number of problems associated with the mechanisms proposed by Hutchinson to account for basal incorporation. Furthermore, the Beltinge mudslide (particularly the lowermost mudslide element in the cascade) is not an ideal case in which to explore the mechanism, for reasons of geological structure. Hutchinson (1970) postulates that some of the effect results from scraping on the basal shear by hard particles in the mudslide. This is difficult to imagine in London clay, where the only hard particles are infrequent calcareous septarian nodule fragments and pyritised fossil remnants (in the Lias, in contrast, there would be abundant limestone fragments). Moreover, they are carried in a "mudflow" with a shear strength of 5 – 15 kN/m^2, and would not be carried down the slip surface if they met with any significant resistance while scraping.

It is also improbable that the second and alternative mechanism postulated by Hutchinson is entirely correct. This mechanism involves the swelling and softening of insitu clay under the mudslide as it is unloaded in a saturated environment, often with high r_u values, so that the water content of the insitu clay rises to a value approaching that of a mud flow matrix, "thus facilitating its incorporation". The explanation ignores the earlier observation of the existence of basal shear surface. No shear stress *greater* than the residual strength can be transmitted through such a shear surface, and as a result, soil at a similar pore water pressure, but *not* sheared, and thus at a higher shear strength, cannot be sheared out of the mudslide bed. This poses what seems to be an insuperable problem – how can soil be sheared from any kind of peak strength with shear stresses only at residual strength levels?

However, it is observed in the field (at least in the summer, when these mudslides are generally less active, somewhat dried out and opened up by marine erosion) that layers of clay, often of very different colours, are repeatedly overthrust. This is sketched in Figure 4.

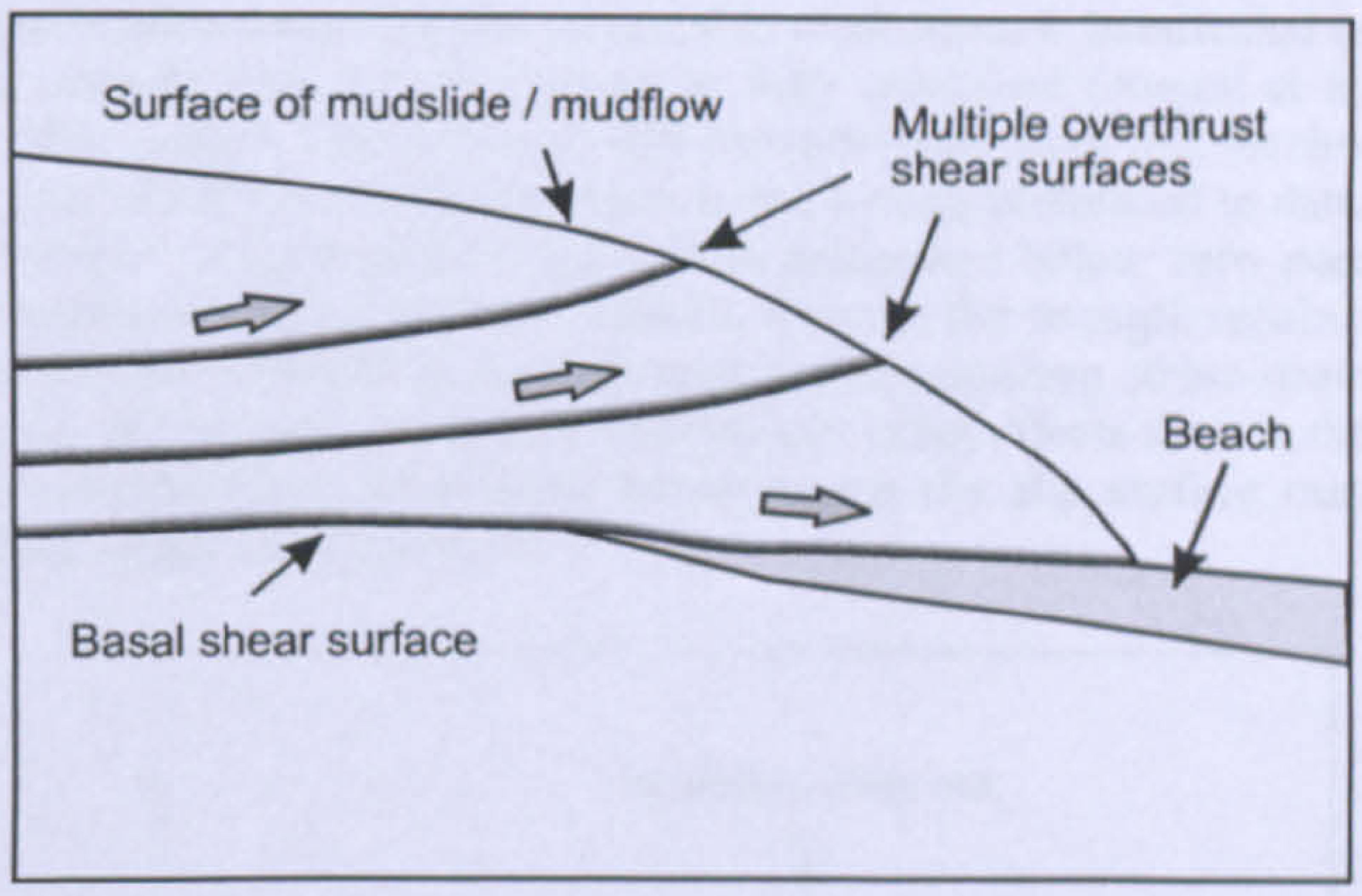

Figure 4. Overthrust repetitive shears in a mudslide toe.

It is first of all important to explain why Hutchinson's Flow II is not in an ideal location to explore mechanisms of basal incorporation. Much of the basal shear of this mudslide is developed about 1m above the contact between the base of the London Clay and the underlying sandy Oldhaven Beds. At Herne Bay, 3 comparatively large compound landslides have their basal shear at this approximate location (Bromhead, 1978). It is thus a preferred slide prone horizon, and if the mudslide has developed along this horizon *ab initio*, then it can have had no basal incorporation. Furthermore, other smaller slides and mudslides have basal shears that follow this particular horizon. Outside the Herne Bay area, slides in earthworks have developed parallel to, and slightly above, the base of the London Clay, e.g. the slides in the cutting at New Cross (Gregory, 1844) and at Harefield (Author's files). If such preferred horizons are slide-prone, then they will be readily picked out by mudslides, but basal incorporation must then cease (or never happen) if they are to be followed thereafter. Alternatively, does basal incorporation occur until a mudslide reaches a stratum which inhibits further channel deepening?

4 ALTERNATIVE MECHANISMS FOR BASAL INCORPORATION

4.1 *Upward water flow*

Clearly, a softened peak strength underneath the basal shear could be weaker than the slip surface if an upwards hydraulic gradient existed. Could such a process exist in the field? Certainly not in the case of Hutchinson's mudslide at Beltinge (Herne Bay). Piezometric conditions in the Herne Bay cliffs were explored by Bromhead (1978) and Bromhead & Dixon (1984), and were found to be influenced by strong underdrainage into the underlying Oldhaven Beds. The sandy Oldhaven Beds outcrop above sea level in this stretch of the coastal cliffs, and are to all intents and purposes dry.

A local upward hydraulic gradient could occur if the high positive pore water pressures in the upper part of the mudslide could leak through the slip surface close to their source, and then, seeping under the basal shear, affect a down slope part of the mudslide basal shear. In Herne Bay case they would be intercepted by the strong under drainage, and thus the mechanism is also improbable.

4.2 Strength regain on the slip surface as a rate effect

A further possible mechanism arises out of the rate effects in residual shear strength reported by several authors. If the residual strength were to rise as the rate of shear increased, (a positive rate effect) it might rise sufficiently to allow the transfer of shear stresses large enough to migrate the shear surface downwards.

Several things conspire to prevent this. Firstly rates of shear are too low to force the reported effects to occur. Secondly, the amount of change (from $\Phi'_r = 10°$ to $20°$) is too large to occur at field rates of shear. Thirdly, in the Authors' experience, such rate effects are artefacts of the testing apparatus and method anyway. Fourthly, it must not be ignored that the rate of sliding increases as the mudslide is becoming more fluid, and of a lower shear strength, and so is losing its ability to transfer shear stresses.

It is therefore particularly difficult to identify mechanisms operating during the steady-state movement of a mudslide that could give rise to basal incorporation. It is much more likely that the mechanism is intermittent. A possibility is as follows.

4.3 Strength regain on the slip surface when there is a cessation of movement due to drying out

Hutchinson notes that the Beltinge mudslide ceased to move by late summer. This is clearly is response to reductions in head load input as the slope dries out. Furthermore, the slide mass water content decreases and the pore water pressures in it are also reduced. Moisture content changes in the mudslide mass down to and including the basal shear surface are facilitated by the permeability of the mudslide fabric–permeability that is orders of magnitude greater than intact clay below the shear surface. However, Hutchinson's view of moisture transfer through the slip surface and into the underlying soil is probably correct.

Accordingly, therefore, a situation may result where the pore water pressure in the basal shear is somewhat less than in the underlying soil. This is an ephemeral state, because further drying out of the mudslide will also dry out the clay beneath the basal shear.

The residual shear strength of a slickensided zone is a normally-consolidated property by virtue of remoulding. Consider the voids ratio and effective pressure on an actively sliding residual slip surface. On a void-ratio / effective stress plot, the soil is at the end of a virgin compression curve (Figure 5). If shear is held constant, and the effective stresses are increased, the soil moves away from the residual shear strength failure envelope (Figure 6). It also traces out a new section of virgin compression curve (Figure 5). This is the progression through late summer in the mudslide. Movements are small or non existent at this time.

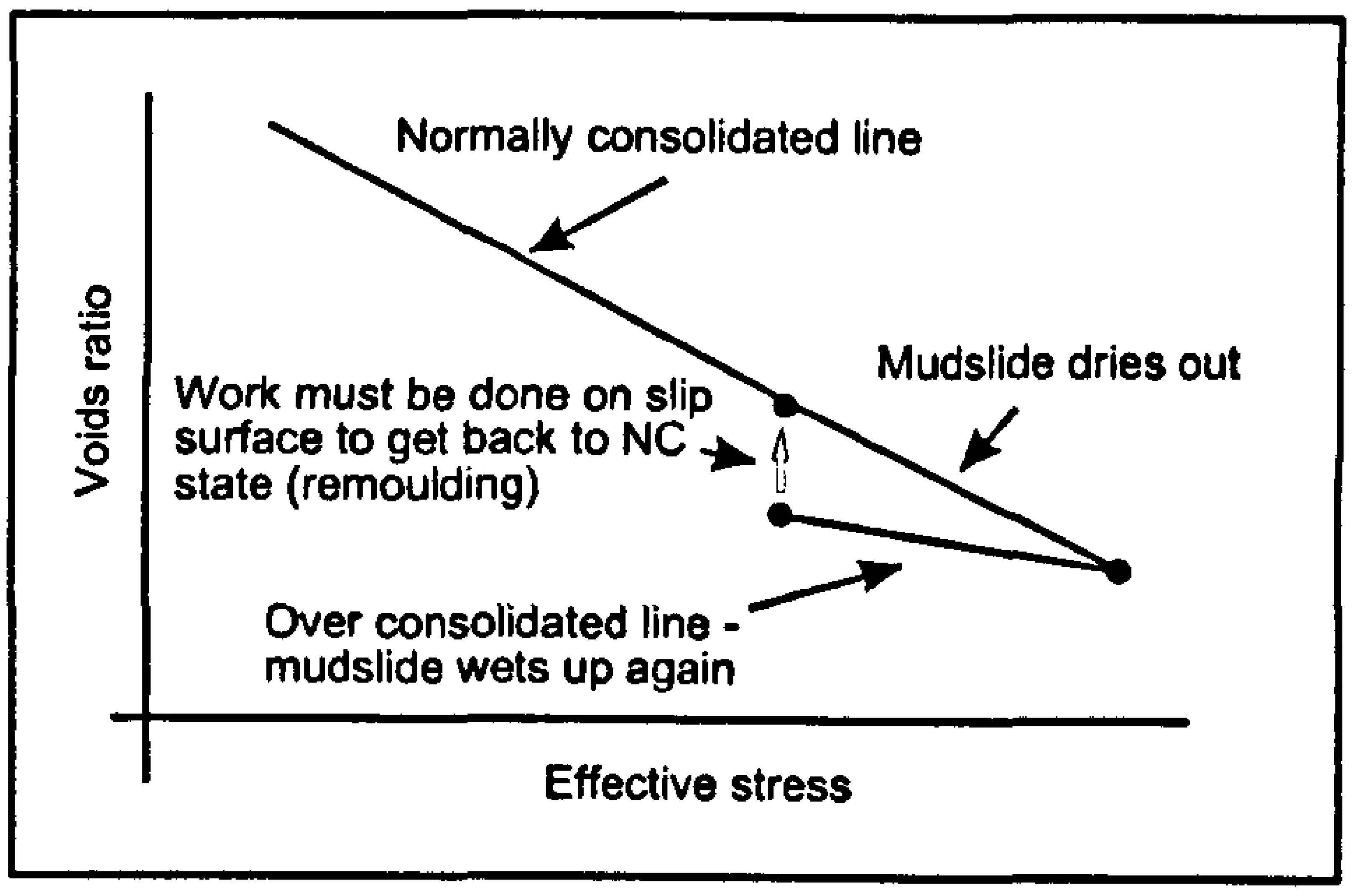

Figure 5. The voids ratio changes in a mudslide basal shear through a drying cycle.

It is difficult to see the mudslide wetting back up to the previous wettest state, including the generation of undrained loading induced pore water pressures, the following winter. However, this is an extreme limit to how far the water pressures may rise. If this was the case, and shear stresses had been "frozen in" to the dried-out mudslide through the summer, the stress state would move back onto the residual shear strength envelope (Figure 6). However, the voids ratio must follow an overconsolidated voids ratio / effective stress line (Figure 5). The slip surface is then overconsolidated, and exhibits a strength gain. It requires appreciable additional work to reinstate the normally-consolidated clay condition, and this work input is manifested as a strength gain. In tests yet to be fully published (Angeli et al. 1996; Angeli, Gasparetto, 2003) strength gains from this mechanism of 20% are not uncommon in the testing performed to date. Further strength gains if the soil is desiccated below zero pore water pressure are possible. Indeed, some of the strength regains, which are manifest as a small peak in the reloading stress-strain plot, are so large that it is postulated that other effects such as the re-establishment of mineral bonds across the slip surface may play a part in this process.

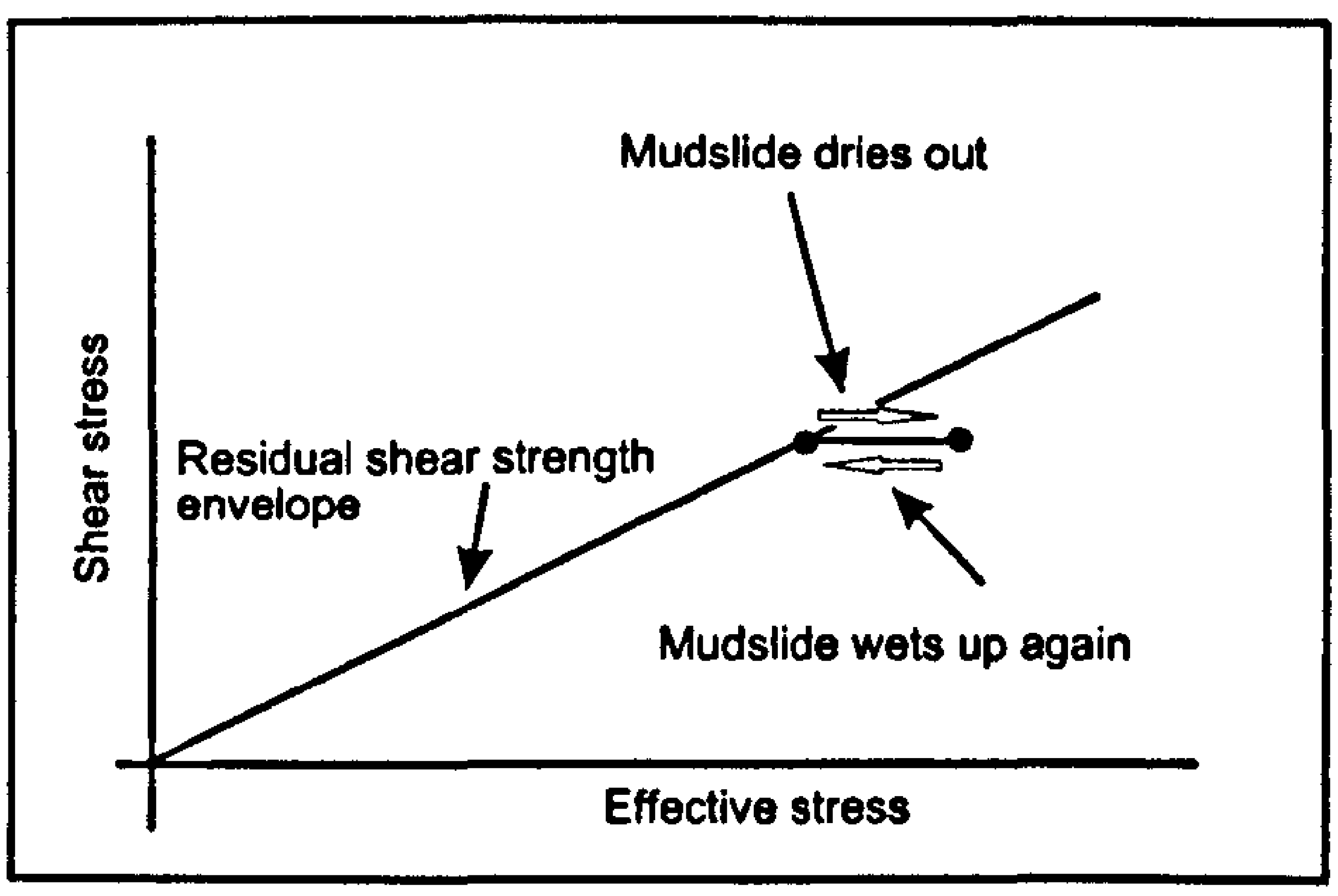

Figure 6. The stresses on the mudslide basal shear through a drying cycle.

If we have a strength gain on the basal shear, and a weakening of the clay beneath the basal shear, the system may transfer sufficient shear stress for a shear-based basal incorporation mechanism to be possible, although it requires a further build up of shear stress to overcome the increased resistance. This build-up occurs during the following winter and spring (rainy season) due to a fresh supply of debris at the head of the mudslide system. The moisture profile in the mudslide resulting from surface drying is likely to inhibit the formation of a new shear surface at a higher elevation than the original shear – formation of a new shear at a lower elevation is of course, a "basal incorporation" shear.

Several aspects of behaviour arise from this postulated mechanism. Firstly, the brittleness associated with formation of a new basal shear will lead to an element of surge in the mudslide deformation, a factor in all first-time failures in brittle soils and rather different to surges resulting from mudslides finding a steeper track (*e.g.* Hutchinson et al., 1974). Secondly, both the old basal shear and the new basal shear will eventually accumulated in the mudslide snout (*cf* Hutchinson's 1970 observations and Figure 4)

The mechanism is critically dependent on the mudslide drying out, at least in part, to lock the existing basal shear, and for UK climatic conditions this requires a comparatively shallow mudslide. Territories with a much higher summer (or dry season) soil moisture deficit will be able to "heal" slip surfaces at greater depths.

5 SOME PRACTICAL CONSIDERATIONS ARISING FROM BASAL INCORPORATION

The senior Author has been involved with the routing strategy for the oil and gas pipelines through inland upland mudslide affected terrain. The primary routing strategy has been to identify primary, i.e. most active, landslide hazards and to avoid them. However, several large mudslides cannot be avoided and must be crossed. The intention is to bring the pipelines beneath the basal shear surface of the mudslide.

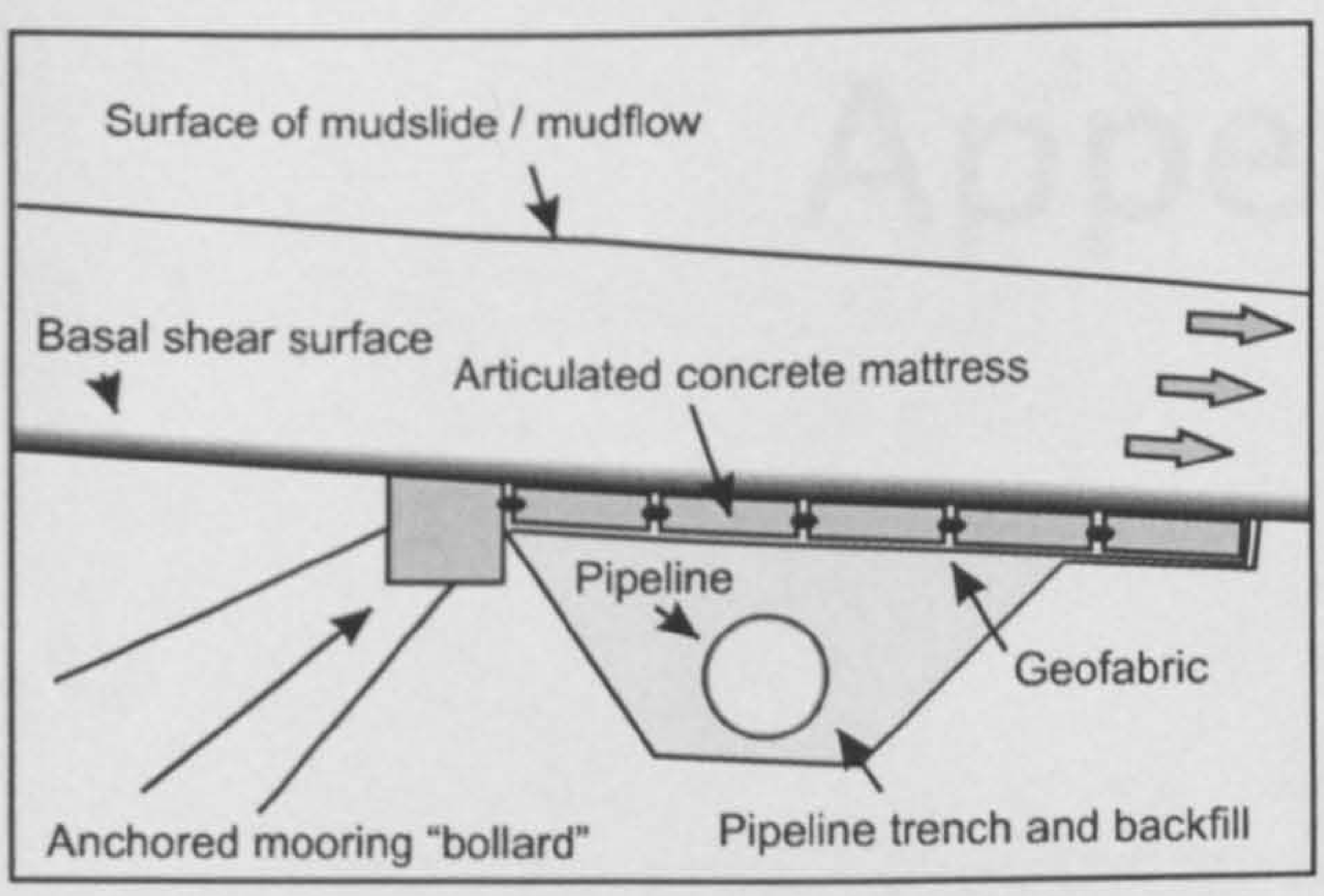

Figure 7. An anti basal incorporation system (Bracegirdle, *pers. comm.*)

Basal incorporation and slide-related deepening of its channel is significant to the safe performance of the pipelines throughout their design life.

Two types of remedial measure have been proposed to address these problems. In the first method, excavation of the mudslide channel bed is to be inhibited by the construction of buried check dams. Check dams at the surface are widely-used to prevent down cutting of gully channel with consequent oversteepening of gulley side slopes. The buried dam structure is constructed at the same time as the pipeline trench. Clearly, it is important to keep mudslide activity confined to its original channel – the buried check dam must not therefore force any change in mudslide channel alignment, or this could affect unprotected parts of the pipeline system.

The alternative design relies on capping the backfilled pipeline trench with a heavy, articulated, concrete mat (Bracegirdle, *pers. comm.*). A geofabric filter under this mat (figure 7) inhibits the migration of fines. The mat needs to be securely anchored at the upstream end.

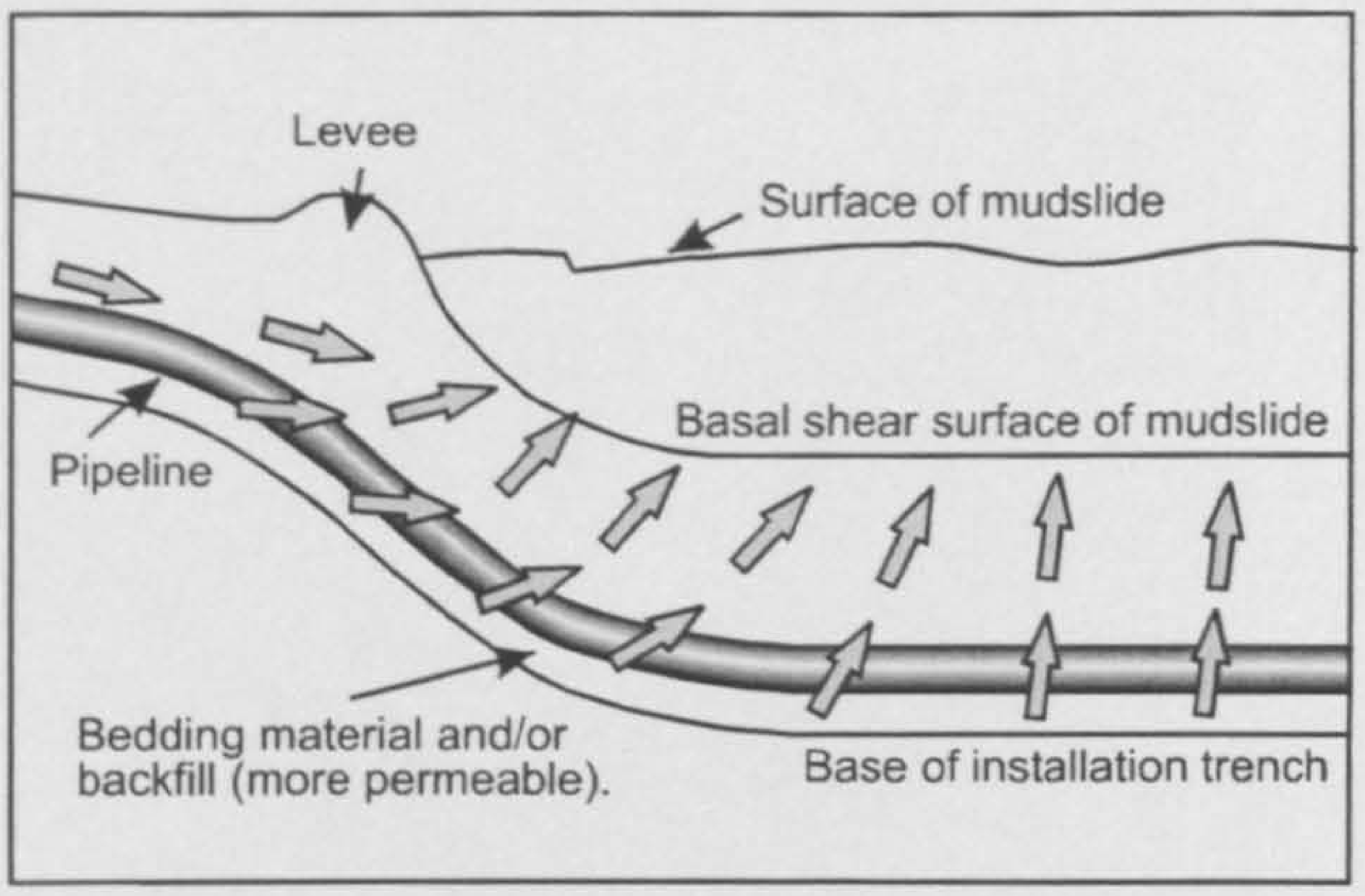

Figure 8. Hydraulic gradients in a pipeline trench under a mudslide.

In both cases, the design permits the relief of pore pressures in the trench fill or bedding (figure 8), which are likely to be sub artesian regardless of the efficiency of the trench blocks installed to inhibit the movement of water along the pipeline.

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Appendix C

Activity of Gault mudslides near St Catherine's Point, Isle of Wight

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Activity of Gault mudslides near St Catherine's Point, Isle of Wight

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Introduction

Coastal sites where clay beds outcrop are classic locations for landslides. Slide types and sizes are related to structural geology, but slide activity is the result of a complex interplay of factors, including marine erosion, rainfall and site hydrology, and lastly human interference with the *status quo.*

This paper describes briefly a series of shallow slides, which may be classified as debris slides or mudslides, within a much larger coastal landslide complex. Preliminary stability analysis based on residual shear strengths determined in a series of ring shear tests at Kingston University are made and the factors which control mudslide activity are explored.

Geological setting

The 11km-long Isle of Wight Undercliff coastal landslide complex is formed from an extensive series of slides that often have bedding-controlled basal shear surfaces. These basal shears occur within a limited range of the clay beds in the Gault, and in the underlying Lower Greensand (LGS) where clay strata are interbedded with weak sandstones (Hutchinson & Bromhead, 2002). Not only are there massive compound block slides with bedding-controlled basal shears, but there are numerous debris slides and mudslides that are superimposed on the basic pattern of block slides. The debris is mainly sourced from the Upper Greensand (UGS) and Lower Chalk, which overly the Gault. The overall pattern of slide types within the landslide complex relates to the presence of an asymmetric SSE-trending plunging syncline, the axis of which crosses the Undercliff in the vicinity of St Lawrence. This general geological structure is shown on Figure 1, with contours of a marker horizon within the Upper Greensand (Chandler, 1984; Hutchinson & Bromhead, *op. cit.*) revealing the overall syncline.

Weather

It has been shown (Ibsen & Brunsden, 1997; Ibsen, 2002) that the incidence of landslides affecting the Undercliff is increasing, and that this increase correlates with the increase in average rainfall and with particular periods of heavy rain. Further increases in landslide activity are anticipated as a consequence of climate change. Typically, average annual rainfall is about 800mm, of which half is lost to evapo-transpiration, and half is available to infiltrate.

Mudslide activity

The nature and thickness of the debris slides and mudslides (collectively mudslides in the following) makes them particularly sensitive to single-year or single-winter rainfall, and it was therefore unsurprising that the particularly wet winter 2000-2001 stimulated a series of serious ground movements which affected the A3055 Undercliff Drive, in once place entirely severing it. Figure 2 shows an intermediate scarp of this particular mudslide, together with a cross section. It is readily seen that a large part of the source area is occupied by ancient debris slides shearing along bedding-controlled surfaces in the Gault, and then spilling over the outcrop of more competent beds, usually as a series of lobate slides, forming accumulations of slide debris on the seaward margins of the Undercliff.

Mudslide movements in the Undercliff (Figure 3) tend to be sedate - *Slow* on the Cruden & Varnes (1996) scale - but they are nevertheless able to obliterate everything in their path, destroying elegant Victorian "Marine Villas", parks, pasture and gardens, severing roads and the infrastructure and generally laying waste to the countryside.

This paper concentrates on the western end of the Undercliff, but the problems are widespread, and only space restraints prevent a fuller review.

Mudslide areas with damage to property

Areas containing the mudslides in the western part of the Undercliff include (from west to east) the slopes under Gore Cliff, Blackgang (Bromhead *et al.*, 1991); the mudslides that are part of the larger St Catherine's Point landslide system (Hutchinson *et al.*, 1991; Hutchinson & Bromhead, 2002), and the mudslides in the Niton – St Lawrence area (Hutchinson & Bromhead, 2002).

The Gore Cliff slopes have lost virtually all of the lobate mudslides to marine action, leaving the bedding-controlled source slides perched high in the cliff. Since the major slippages of 1978 and 1991 destroyed the dwellings in this area, and cut off their septic-tank wastewater inputs, there has been little activity in this region. Neither 1978 nor 1991 had particularly high rainfalls, and this is believed to point to the role of wastewater disposal in slope instability.

In contrast, the slopes east of Niton are well populated. This semi-rural area does not have mains sewerage, and all wastewater is disposed of *via* septic tanks and soakaways, which adversely affect the water balance. The A3055 Undercliff Drive follows a winding path that takes it over the heads of several mudslides, and thus renders it almost certain that damage will be experienced at times of elevated mudslide activity. An extensive system of deep land drains are currently under construction to improve stability in an area where the mudslides have damaged and destroyed a number of dwellings, and have been the subject of attention by national press and television reports.

St Catherine's Point

Hutchinson *et al.* (1991) describe the system of compound block slides at St Catherine's Point and its activity is described by Bromhead *et al.* (1989). The Ordnance Survey chose to locate its GPS base stations at Trinity House lighthouses, of which St Catherine's Point lighthouse was one. It became evident that the lighthouse was moving (although this was already known from a local geodetic survey), and the movements in the wet winter 2000-2001 are summarised by Hutchinson *et al.* (2002).

Mudslide SC8 (see site plan, Figure 3) was active in the winter of 1960, and severed the access road to Knowles Farm and the lighthouse complex. The access road is very distorted, and this shows some recurrence of movement since the road was reconstructed the following year. In recent times, however, the mudslides at the western end of the ridge have been more active. This includes mudslide SC2, which reactivated in 1988-1989, and has moved intermittently since, eventually breaking through the debris apron and finding an indirect path into Watershoot Bay. Mudslides SC3 and SC4 became active in the winter of 2000-2001, which has already been noted to have been wet and well able to provoke landslide movements elsewhere in the Undercliff.

The mudslides at St Catherine's Point are in many ways less complicated than those elsewhere on the Undercliff. Firstly, there are no inputs of water from dwellings or highway drainage in the source area. Secondly, the source area has been isolated from the aquifer of the Chalk and Upper Greensand of St Catherine's Hill by the occurrence of a massive block slide (Hutchinson *et al.* 1991). Finally, the toes of the mudslides discharge onto a massive low-lying apron of debris on which the St Catherine's lighthouse and Knowles Farm have been built, and are not acted on directly by marine erosion. They therefore represent a control, which demonstrates the effect of rainfall on mudslide activity: greater activity elsewhere must be the result of additional factors, of which human activity is the main one.

Sampling and testing the sheared Gault Clay

The basal shears of the mudslides in question are formed from insitu and transported Gault clay, and the body of the mudslide is a mixture of Gault Clay and debris from Upper Greensand and Lower Chalk sources. It was regarded as important to measure the residual shear strength parameters for this clay, and large disturbed samples were taken on two occasions from the exposures in the vicinity of Watershoot Bay. The geotechnical index properties of this very dark grey plastic clay are listed in Table 1. It is believed to have a mixture of clay minerals, with illites dominant.

The clays were tested in the geotechnical laboratory at Kingston University, where there are three "Bromhead" pattern ring shear machines, one modified by the manufacturer, Wykeham Farrance international, for stepless normal load control. As well as testing the clay for a range of normal effective stresses, the effects of strain rate and testing procedure were investigated.

Results of the testing programme

The main findings of the testing programme are that the residual shear strength of the Gault clay in so far as it affects these mudslides is best characterised by the average parameters:

$c' = 0$ and $\phi' = 16.6°$

There is a small rate effect, seen as a rise in residual shear strength of 1.3% for a 2500 fold increase in shearing rate from 0.017 mm/min to 45 mm/min. Increase in shearing rate of this magnitude causes significant changes in the extrusion of clay from the sides of the apparatus, and it is concluded that the work done in extrusion is a significant factor in the apparent strain-rate dependent increase in shear strength. At field rates of shearing (appreciably slower than laboratory rates) the strain rate effect is negligible.

Analysis of mudslide behaviour

At Gore Cliff, samples of the bedding-controlled basal shear surface had previously been sampled and tested, showing ϕ'_r to be about 10.5°, with very little scatter. Back-analysis of the main debris slides indicated that they would be stable under this residual strength until piezometric levels rose in the debris to a pore pressure ratio of r_u=0.4 or more. Under mean annual rainfall conditions, the pore pressure ratio (where measured) is more like r_u=0.3 or less. To raise the piezometric level takes an excess of rainfall above the mean. Typically, a wet winter (or year) is one with more than 60% above the mean rainfall. Assuming a similar loss of water to evapotranspiration, and that the infiltrating rain fills up pores in the debris of which the slide masses are composed, then a wet winter will give about 0.6 – 1.0m of piezometric rise. For the main slides, which are often 10m or more in thickness, this piezometric rise is often insufficient to destabilise them to any degree.

The lobate mudslides (like the SC series mudslides in Figure 3) are steep, and their basal shears are characterised by a higher and more variable residual shear strength. Moreover, they are thin (Figure). Under wet winter conditions they can be readily destabilised with a 0.6 – 1.0 m rise in piezometric level, or the likely result of a wet winter (analysis by infinite slope method).

Effects of future climate changes

Climate change predictions for the south of the Isle of Wight up to 2080 (McInnes *et al.*, 2000) anticipate a 23% increase in winter rainfall, but a 20% decrease in summer rainfall during the next half-century. Since the winter rainfall even today is more significant than summer rainfall, this represents a significant rise overall. The annual probability of wet winters (as defined above) is anticipated to rise from 2% to 11%. This increases the probability of consecutive wet years, and indeed, the probability of runs of wet years. Temperatures and sea levels will also rise. These climatic factors can be expected to correspondingly increase the frequency of bouts of severe mudslide activity in the Undercliff.

Discussion

The measured residual strengths display a variability that is entirely concordant with the irregular mixture of source materials that form the mudslides. However, with the mean properties measured the stability analysis shows that the mudslides at St Catherine's Point are stable under summer conditions and ordinary winter conditions, but are readily destabilised by extremes, such as wet winters. This matches the field observations of periods where the mudslides are active.

Applying the same methodology to mudslides elsewhere (e.g. in the Niton – St Lawrence area) it is readily seen that mudslides are active more frequently than would be the case than if they were destabilised by rainfall alone. Given that this area is populated, and that the population is not served by mains sewerage, it is an irresistable conclusion that the inputs of wastewater into the groundwater body are at least in part a reason for the increased mudslide activity here.

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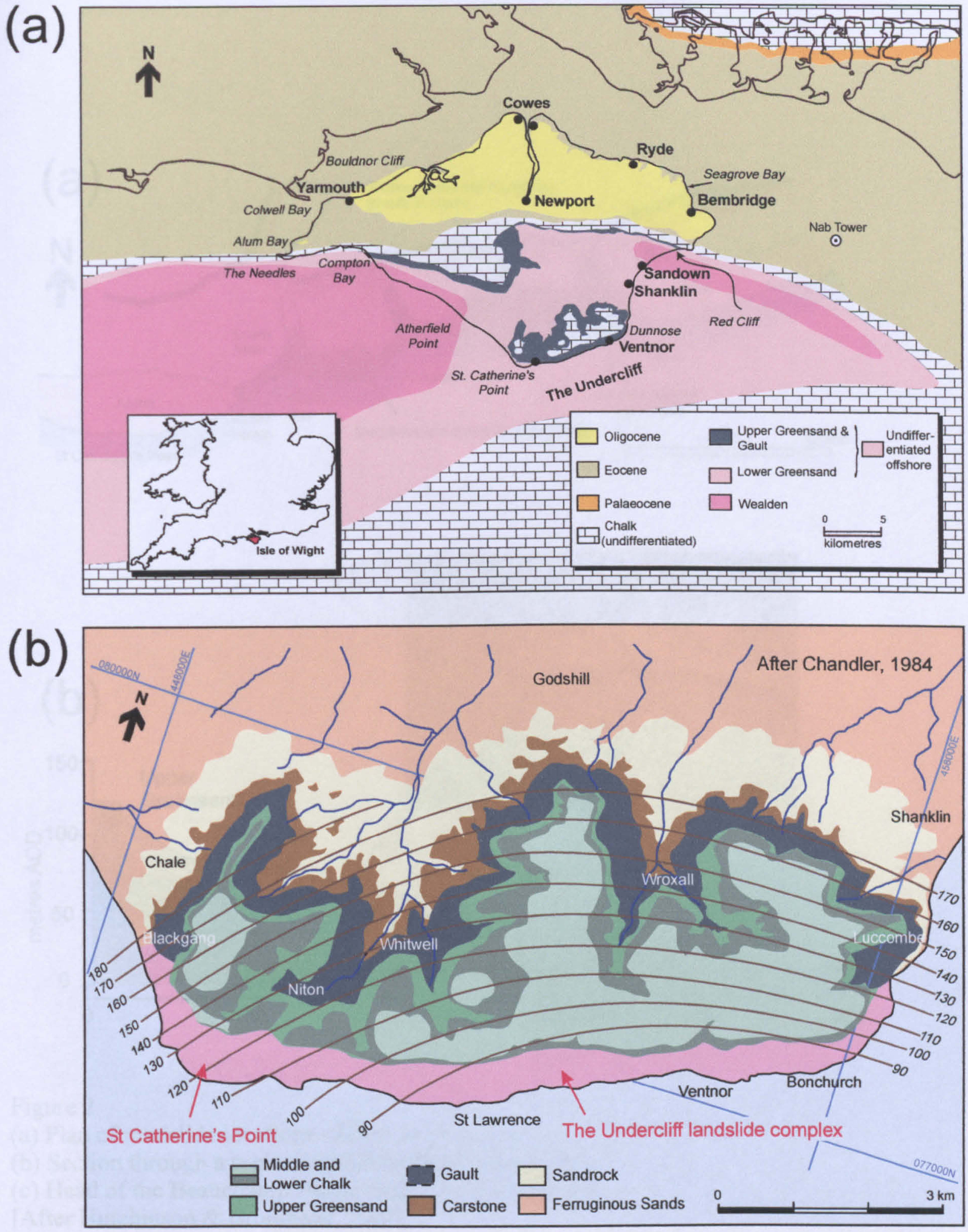


Figure 1
(a) General geological setting and location of Isle of Wight
(b) Detailed geology and structure of the Undercliff, especially the plunging syncline evidenced by contours of a marker horizon.
[After Chandler, 1984 and Hutchinson & Bromhead, 2002]

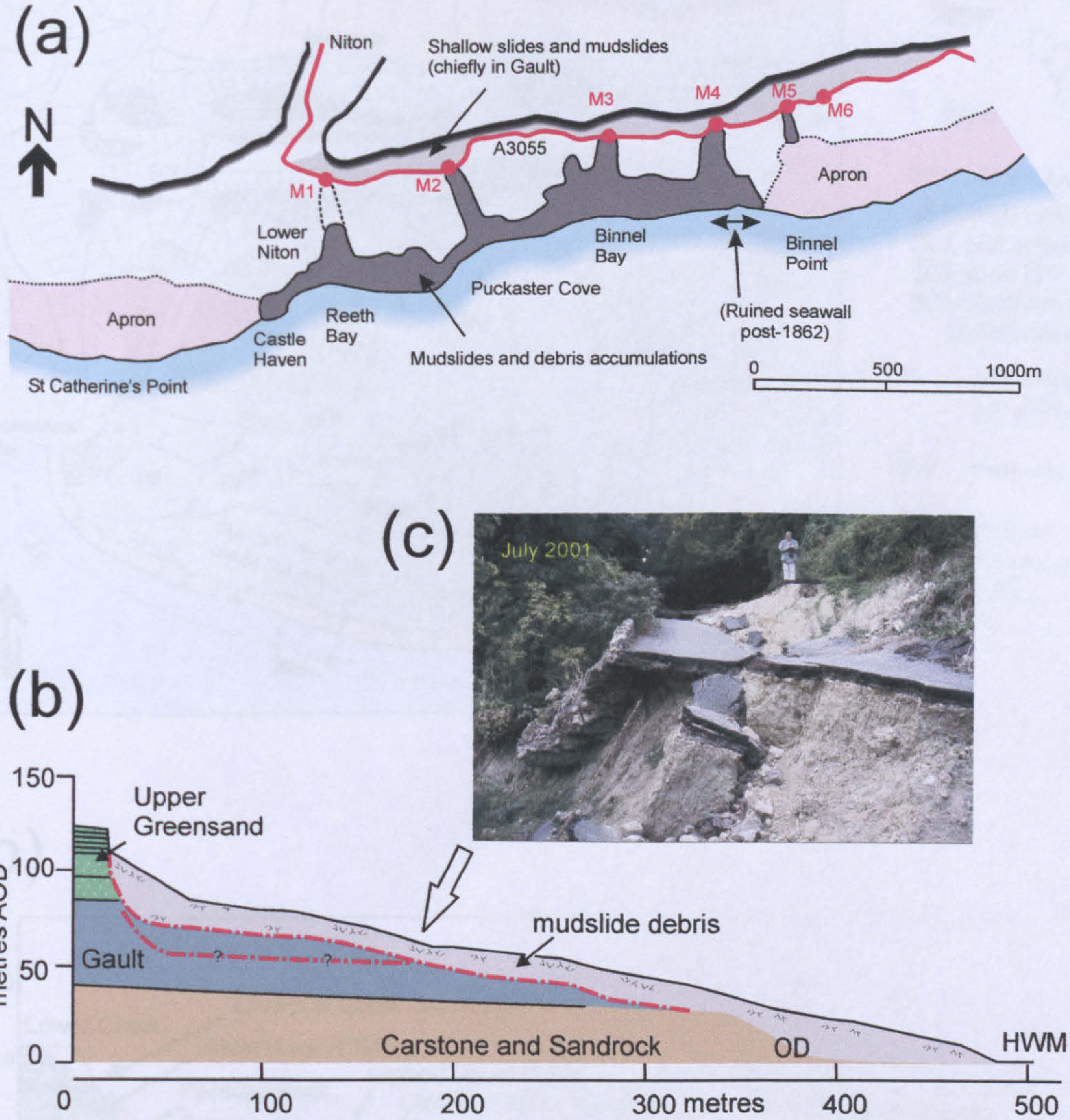
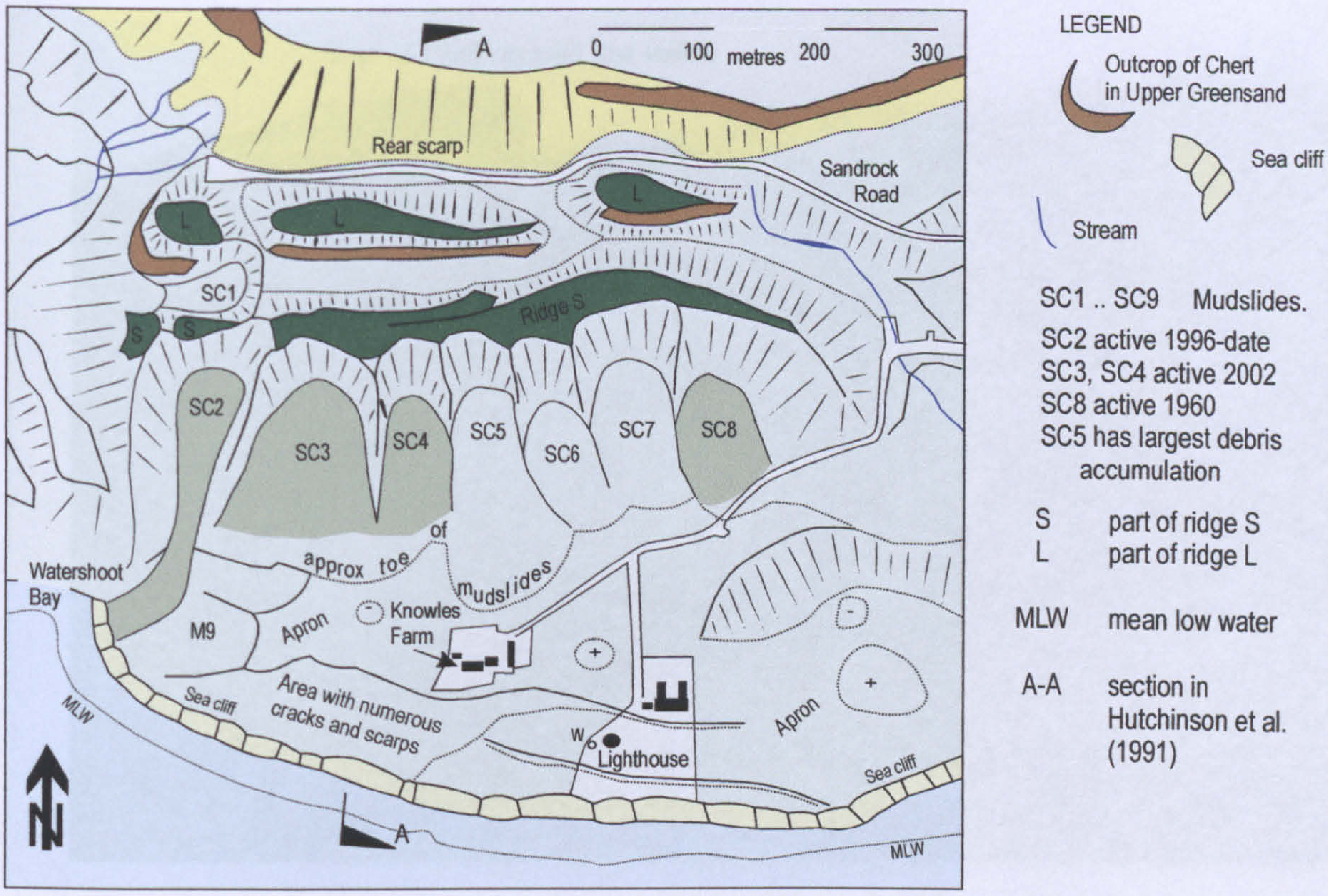


Figure 2
(a) Plan of mudslide locations - Niton to St Lawrence
(b) Section through a typical mudslide (Beauchamp House)
(c) Head of the Beauchamp House mudslide, M2 on plan
[After Hutchinson & Bromhead, 2002]

(a)



(b)

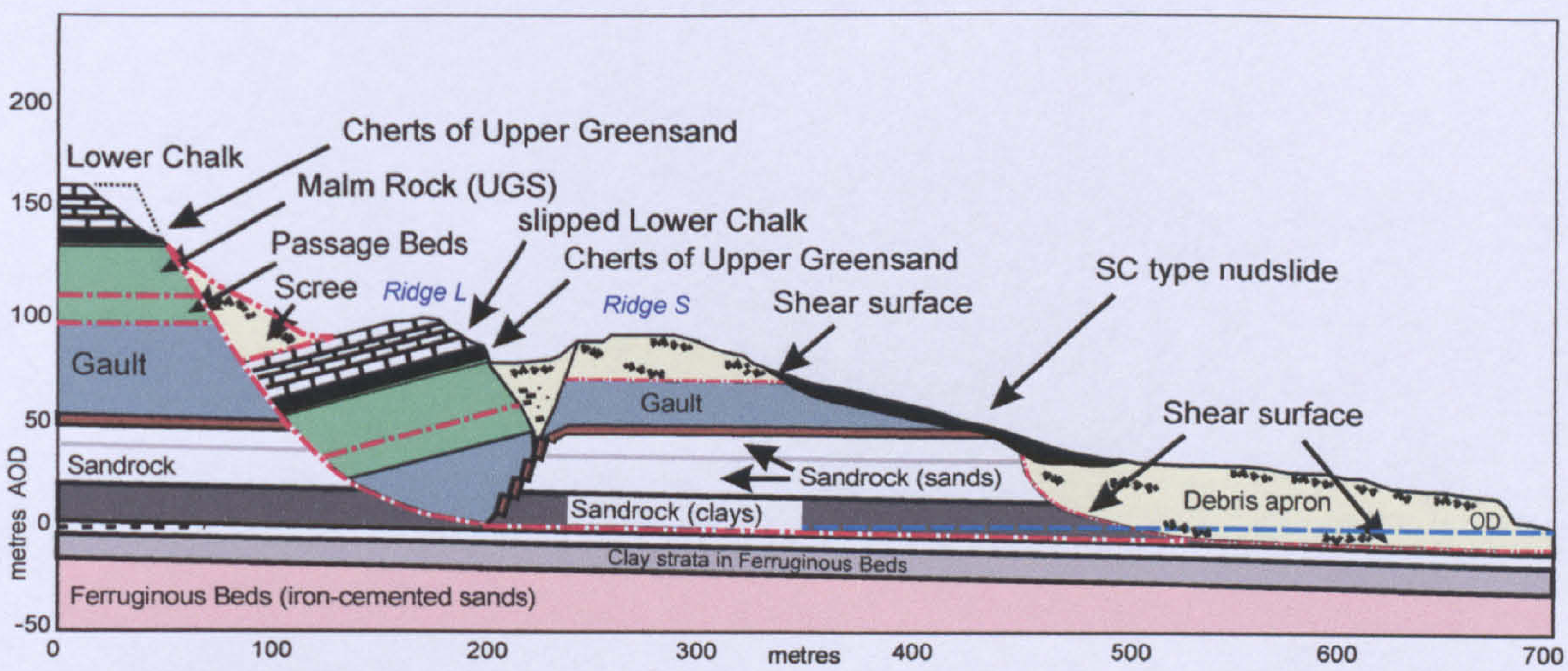


Figure 3
(a) Plan of St Catherine's Point landslide system
(b) Section through St Catherine's Point landslide system, showing relationship of main block slide and mudslides
[After Hutchinson, Bromhead & Chandler, 1991]

Figure 4
Mudslide SC2

Appendix D

strain rate deg/min	mm/min	IOW1	IOW2	NI	Sheppey
0.024	0.018	8.5	20.6	18.3	15.2
0.036	0.027	11.3	11.3	17.2	
0.048	0.036	14.2	14.2	17.4	11.6
0.072	0.053	15.4	15.4	17.1	
0.096	0.071	10.3	10.3	18.0	
0.6	0.445	12.4	12.4	15.9	
0.8	0.594	12.1	12.1	14.9	
1.2	0.890	14.0	14.0	17.2	13.7
1.8	1.336	11.9	11.9	17.4	
2.4	1.781	9.3	9.3	17.4	
10	7.420	11.2	11.2	18.3	
20	*14.840*	*12.9*	12.9	17.4	
30	22.260	14.0	14.0	17.2	*15.2*
45	33.390	12.6	12.6	17.4	
60	44.520	17.4	17.4	17.4	17.5

Average Results for Strain rate effect on ϕ'_r

Clay Location	Plastic Limit	Liquid Limit
St Catherine's Point, IOW	20.1	60.5
Dromore, NI	32	60.5
Warden Point, Isle of Sheppey	37	86

Index tests on clay samples

strain rate deg/min	mm/min	IOW1	IOW2	NI	Sheppey
0.024	0.017808	8.1	21.5	18.5	13.2
0.036	0.026712	13.3	13.3	16.6	
0.048	0.035616	10.6	10.6	15.1	13.1
0.072	0.053424	18.9	18.9	16.7	
0.096	0.071232	9.5	9.5	16.8	
0.6	0.4452	13.2	13.2	15.6	
0.8	0.5936	12.2	12.2	14.7	
1.2	0.8904	13.2	13.2	17.4	11.2
1.8	1.3356	14.5	14.5	20.2	
2.4	1.7808	9.3	9.3	20.2	
10	7.42	10.7	10.7	18.5	
20	14.84	11.9	11.9	20.2	
30	22.26	14.4	14.4	17.4	13.7
45	33.39	13.3	13.3	20.2	
60	44.52	20.2	20.2	20.2	15.0

Rough Internal Plate - Mach ine 1 on ϕ'_r

strain rate deg/min	mm/min	IOW1	IOW2	NI	Sheppey
0.024	0.017808	9.0	19.6	18.0	17.2
0.036	0.026712	9.2	9.2	17.8	
0.048	0.035616	17.9	17.9	19.7	10.1
0.072	0.053424	11.9	11.9	17.4	
0.096	0.071232	11.0	11.0	19.2	
0.6	0.4452	11.5	11.5	16.3	
0.8	0.5936	12.1	12.1	15.1	
1.2	0.8904	14.8	14.8	17.1	16.2
1.8	1.3356	9.4	9.4	14.7	
2.4	1.7808	9.4	9.4	14.7	
10	7.42	11.7	11.7	18.0	
20	14.84	13.9	13.9	14.7	
30	22.26	13.6	13.6	17.1	16.6
45	33.39	11.8	11.8	14.7	
60	44.52	14.7	14.7	14.7	19.9

Grooved internal plate- machine 2 on ϕ'_r

Final Strain rate mm/min	IOW UP	NI UP	SHEP UP	Bromhead Ring shear gear settings
0.8904	5.6	13.3	8.0	e3060c4545
44.52	3.9	12.5	11.0	e3060a6030
0.8904	8.5	10.0	5.4	ec4545
22.26	3.8	8.0	5.5	ea4545
22.26	8.3	13.2	5.5	ca4545
44.52	6.6	12.3	9.0	c4545a6030
	IOW DOWN	NI DOWN	SHEP DOWN	
0.035616	6.1	10.1	8.0	ce4545
0.017808	8.0	8.7	7.0	c4545e3060
0.017808	11.4	10.0	10.9	a6030e3060
0.8904	10.5	11.7	9.3	a6030c4545
0.035616	9.0	10.0	8.0	ae4545
0.8904	9.2	3.5	10.4	ac4545
	IOW	NI	SHEP	
0.017808	5.4	11.6	6.8	e3060
0.035616	7.1	11.1	8.4	e4545
0.8904	8.9	11.0	9.6	c4545
22.26	8.9	11.0	10.3	a4545
44.52	11.1	11.1	11.5	a6030

Average results for Discontinuous strain rate on ϕ'_r

strain rate deg/min	mm/min	IOW2	NI	Sheppey
0.024	0.017808	5.14	11.8	7.1
0.048	0.035616	6.7	9.6	8.4
1.2	0.8904	8.4	11.1	8.7
30	22.26	9.2	11.1	9.6
60	44.52	12.9	12.9	10.3

Summary results of continuous strain rate - Machine 1 on ϕ'_r

Bromhead Ring shear gear settings	inc in rate	Final Strain rate mm/min	IOW2	NI	Sheppey
e3060c4545	0.872592	0.8904	1.8	10.5	5.9
e3060a6030	44.50219	44.52	5.9	11.7	9.2
ec4545	0.854784	0.8904	2.9	10.7	7.9
ea4545	22.22438	22.26	6.2	16.0	6.0
ca4545	21.3696	22.26	8.1	13.9	4.2
c4545a6030	43.6296	44.52	9.4	12.2	9.5
ce4545	-0.85478	0.035616	7.8	10.6	6.6
c4545e3060	-0.87259	0.017808	8.7	9.7	2.8
a6030e3060	-44.5022	0.017808	14.4	13.1	9.3
a6030c4545	-43.6296	0.8904	11.0	11.1	9.0
ae4545	-22.2244	0.035616	8.0	11.0	6.5
ac4545	-21.3696	0.8904	7.3	2.8	8.4

Machine 1 - rough internal plate on ϕ'_r

strain rate deg/min	mm/min	IOW2	NI	Sheppey
0.024	0.017808	5.7	11.5	6.5
0.048	0.035616	7.6	12.6	8.4
1.2	0.8904	8.6	10.9	10.6
30	22.26	9.4	10.9	10.9
60	44.52	9.3	9.3	12.7

Summary results of continuous strain rate - Machine 2 on ϕ'_r

Bromhead Ring shear gear settings	inc in rate	Final Strain rate mm/min	IOW2	NI	Sheppey
e3060c4545	0.872592	0.8904	1.8	10.5	5.9
e3060a6030	44.50219	44.52	5.9	11.7	9.2
ec4545	0.854784	0.8904	2.9	10.7	7.9
ea4545	22.22438	22.26	6.2	16.0	6.0
ca4545	21.3696	22.26	8.1	13.9	4.2
c4545a6030	43.6296	44.52	9.4	12.2	9.5
ce4545	-0.85478	0.035616	7.8	10.6	6.6
c4545e3060	-0.87259	0.017808	8.7	9.7	2.8
a6030e3060	-44.5022	0.017808	14.4	13.1	9.3
a6030c4545	-43.6296	0.8904	11.0	11.1	9.0
ae4545	-22.2244	0.035616	8.0	11.0	6.5
ac4545	-21.3696	0.8904	7.3	2.8	8.4

Machine 2- grooved internal plate on ϕ'_r

Rate deg/min	Rate mm/min	Plastic Limit	Moisture Content post test	Liquid Limit	Moisture Content Natural Sample
0.024	0.017808	20.1	34.3	60.5	24.9
0.036	0.026712	20.1	40.9	60.5	24.9
0.048	0.035616	20.1	44.9	60.5	24.9
0.072	0.053424	20.1	44.5	60.5	24.9
0.096	0.071232	20.1	42.2	60.5	24.9
0.6	0.4452	20.1	50.5	60.5	24.9
0.8	0.5936	20.1	48.2	60.5	24.9
1.2	0.8904	20.1	47.3	60.5	24.9
1.8	1.3356	20.1	35.1	60.5	24.9
2.4	1.7808	20.1	43.5	60.5	24.9
15	7.42	20.1	52.3	60.5	24.9
20	14.84	20.1	65.3	60.5	24.9
30	22.26	20.1	51.6	60.5	24.9
45	33.39	20.1	72.3	60.5	24.9
60	44.52	20.1	77.3	60.5	24.9

Machine 1 - IOW Post test Moisture Content

Rate deg/min	Rate mm/min	Plastic Limit	Moisture Content post test	Liquid Limit	Moisture Content Natural Sample
0.048	0.035616	37	50.7	86	41.9
1.2	0.8904	37	71.57	86	41.9
45	33.39	37	93.6	86	41.9
60	44.52	37	99.3	86	41.9

Machine 1 - Sheppey Post test Moisture Content

Rate °/min	Rate mm/min	Plastic Limit	Moisture Content post test	Liquid Limit	Moisture Content Natural Sample
0.024	0.017808	20.1	33.1	60.5	24.9
0.036	0.026712	20.1	41.5	60.5	24.9
0.048	0.035616	20.1	38.2	60.5	24.9
0.072	0.053424	20.1	32.2	60.5	24.9
0.096	0.071232	20.1	35.5	60.5	24.9
0.6	0.4452	20.1	50.9	60.5	24.9
0.8	0.5936	20.1	32.5	60.5	24.9
1.2	0.8904	20.1	30.8	60.5	24.9
1.8	1.3356	20.1	35.5	60.5	24.9
2.4	1.7808	20.1	47.3	60.5	24.9
15	7.42	20.1	61.0	60.5	24.9
20	14.84	20.1	65.6	60.5	24.9
30	22.26	20.1	48.8	60.5	24.9
45	33.39	20.1	67.6	60.5	24.9
60	44.52	20.1	50.4	60.5	24.9

Machine 2 - IOW Post test Moisture Content

Rate deg/min	Rate mm/min	Plastic Limit	Moisture Content post test	Liquid Limit	Moisture Content Natural Sample
0.048	0.035616	32	37.6	60.5	36
1.2	0.8904	32	37.9	60.5	36
45	33.39	32	61.6	60.5	36
60	44.52	32	63.6	60.5	36

Machine 2 - Dromore Post test Moisture Content

strain rate deg/min	mm/min	IOW2	NI	Sheppey
0.024	0.018	8.2	9.7	4.6
0.036	0.027	8.2		
0.048	0.036	3.8	11.6	3.4
0.072	0.053	8.2		
0.096	0.071	9.1		
0.6	0.445	8.6		
0.8	0.594	8.7		
1.2	0.890	8.4	10.6	4.9
1.8	1.336	6.2		
2.4	1.781	7.0		
10	7.420	8.2		
20	14.840	8.2		
30	*22.260*	9.8	9.3	3.3
45	33.390	9.2		
60	44.520	8.8	6.6	3.7

Continuous Strain Rate using Pressure loading – on ϕ'_r

Final Strain rate mm/min	IOW UP	NI UP	SHEP UP	Bromhead Ring shear gear settings
0.8904	8.1	14.7	6.1	e3060c4545
44.52	7.0	9.7	5.9	e3060a6030
0.8904	7.5	15.0	3.7	ec4545
22.26	7.7	3.5	4.7	ea4545
22.26	9.0	11.7	4.3	ca4545
44.52	8.3	13.0	6.2	c4545a6030
	IOW DOWN	NI DOWN	SHEP DOWN	
0.035616	8.1	8.5	6.4	ce4545
0.017808	8.9	15.0	1.4	c4545e3060
0.017808	8.1	6.3	3.7	a6030e3060
0.8904	9.8	7.4	3.7	a6030c4545
0.035616	9.4	6.3	3.3	ae4545
0.8904	8.8	11.6	3.3	ac4545
	IOW	NI	SHEP	
0.017808	8.0	9.7	4.6	e3060
0.035616	8.2	11.6	3.4	e4545
0.8904	9.1	10.6	4.9	c4545
22.26	9.1	9.3	3.3	a4545
44.52	8.8	6.6	3.7	a6030

Discontinuous Strain Rate using Pressure loading - on ϕ'_r

	Phi' (deg)	sample moisture content %	sample depth (m)	Plastic Limit (%)	Liquid Limit (%)	PI (%)
BH1	7.8	21.0	80.74	31.2	74.0	42.8
BH1	16.5	17.3	54.15	23.6	48.6	25.0
BH2	9.2	21.0	137.00	28.8	66.1	37.3
BH2	13.2	21.7	90.25	22.3	60.2	37.8
BH2	7.7	19.8	78.85	29.3	67.1	37.8
BH2	11.6	21.9	68.00	36.9	56.6	19.8
BH2	11.3	21.4	60.97	31.6	49.3	17.7
BH2	10.2	25.3	59.22	30.7	51.6	21.0
BH2	18.8	25.4	58.25	27.9	52.0	24.1
BH3	13.1	19.6	81.55	25.8	44.4	18.5
BH3	6.9	18.5	47.37	27.3	95.0	67.7
BH3	5.3	20.2	42.25	33.3	70.7	37.4
BH3	4.9	15.9	33.15	31.2	68.0	36.8
BH3	11.6	28.6	29.15	38.5	50.9	12.5
BH3	7.1	27.8	26.74	35.0	51.3	16.3

Ventnor Variability Results