CHAPTER IV:

RESIDUAL SHEAR STRENGTH AND ITS MEASUREMENT

4.1 INTRODUCTION

Soil engineers generally consider the strength of soils in terms of shear strength, which may be defined as the resistance offered by the soil to deformation along a discrete failure surface.[1] The emphasis placed on shear strength is a consequence of the fact that slope failure usually involves a shearing of the soil material under the influence of gravity. This is true of landslides (Ward, 1945, p. 172), and perhaps also of soil creep (Kojan, 1967). Therefore in studies of mass movements it is the shear strength characteristics of soils which must be studied, rather than other strength parameters.

The numerical value of shear strength, in terms of resistance offered per unit area of failure plane, is

1. Hvorslev (1960) has defined shear strength as the stress acting in the plane of failure at the time of failure.

determined in standard laboratory tests in which a sample of soil is forced to rupture along a discrete surface (Terzaghi & Peck, 1967). However, the value obtained is not a constant characteristic of the soil, but varies with the method of testing (Gillott, 1968). Indeed, soil shear strength is still not completely understood, principally because of its extreme complexity and dependence on a large number of other properties, including permeability and compressibility (Haefeli, 1950), fabric (Mitchell, 1954; Williamson, 1954; Rowe, 1972), type of pore fluid, pressure on the sample, rate of deformation, and others (Kenney, 1966).

The resistance offered by a cohesive soil (one in which some shear strength is present in the absence of a normal compressive load) to movement along a failure surface depends on the stress acting normal to the failure plane. A relationship similar to that of Figure 4.1 is shown by most soils.

This relationship may be expressed by the Coulomb equation,

$$S = C + \sigma \tan \phi \qquad \text{.....(1)}$$

which is merely the equation of the straight line shown in Figure 4.1. This equation assumes that the strength of the soil is governed by the total stress normal to the shear plane (σ). However, the application of stress to a

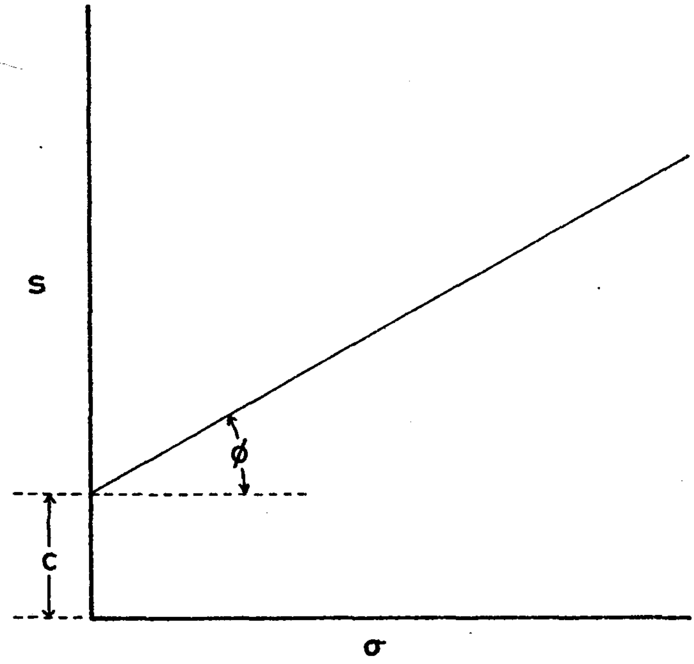


FIGURE 4.1
TYPICAL STRESS-STRENGTH PLOT
FOR A COHESIVE SOIL

soil may result in a temporary increase in the pore-water pressure, which counterbalances part of the increase in load, and therefore the effective stress (σ'), on which the strength actually depends, is given by

$$\sigma' = \sigma - u \qquad \text{.....(2)}$$

where u is the pore-water pressure (Capper & Cassie, 1963, p. 67). Hence to allow for the presence of positive and negative pore-water pressures developed in saturated and unsaturated soils, the Coulomb equation must be re-written in the form

$$S = C' + (\sigma - u) \tan \phi' \qquad \text{.....(3)}$$

or

$$S = C' + \sigma' \tan \phi' \qquad \text{.....(4)}$$

where the parameters C' and ϕ' are expressed in terms of the effective stress $(\sigma - u)$ acting normal to the slip surface (Lee, 1968). The concept of "effective stress" is semi-empirical, but its validity has been well established (Hvorslev, 1960).

Equation (4), known as the Coulomb-Terzaghi equation, assumes that the soil is homogeneous and isotropic (Bjerrum, 1951). It suggests that the shear strength is composed of two different components - the first, "cohesion" (c), is the same in all planes through any point in such a soil mass, but depends on the water content and hence varies from point to point. According to Tan (1947, p. 22), cohesion is essentially an intergranular stress phenomenon created by capillary forces arising from the moisture film at grain boundaries, and molecular forces between clay-sized particles.

The second component of shear strength is the frictional resistance, whose magnitude is proportional to the effective normal stress on the plane through the point under consideration (Bjerrum, 1951; Haefeli, 1950; Cooling & Smith, 1936; Karol, 1960).

It must be remembered that C and ϕ are not fundamental properties of a soil; they are merely constants derived from the geometry of the graph relating shear strength to normal stress (Figure 4.1). They depend on the conditions of testing, and are hence best termed the apparent cohesion (C) and angle of shearing resistance (ϕ) (Her Majesty's Stationery Office, 1952, p. 348).

In a stable slope, only a part of the available shear strength along a potential shear surface is required to counteract the shear force. It is usual to express the excess of strength in terms of the "factor of safety", which is the ratio of shear strength to shear stress. Hence we may write -

$$\Sigma\tau = \Sigma\frac{C'}{F} + \Sigma(\sigma - u)\frac{\tan\phi'}{F} \qquad .....(5)$$

where F is defined as the "factor of safety" (Skempton, 1964). For stable slopes, F is greater than unity; at failure, $\Sigma\tau = \Sigma s$, and $F = 1.0$.

4.2 DEVELOPMENT OF IDEAS ON "RESIDUAL" SHEAR STRENGTH

Until the 1950's, it was usual in the laboratory testing of soils to continue shearing a sample until a maximum strength value was reached, and to define this as the shear strength. Thus, Capper & Cassie (1963, p.66) define shear strength as the maximum resistance to shearing stresses.

However, it had been recognised much earlier that strength values below the maximum, reached by continuing to shear the sample after reaching the peak strength, might also be significant. For example, Tiedmann (1937, quoted by La Gatta, 1970) found that at very large displacements the shear strength of clays was only a fraction of the peak strength, and that it approached a constant value. Tiedmann called this "pure sliding resistance."

Again, one of the principal objectives of shear tests suggested by Hvorslev (1939) was to determine

> ".. the temporary or permanent decrease of the shearing resistance after failure. Most soils are subject to a decrease of the shearing resistance after failure and its ultimate minimum value may in some cases be as low as 20 per cent of the maximum value of the shearing resistance." (Hvorslev, 1939, p. 999).

Hvorslev termed this lower strength value "ultimate minimum shear strength." Haefeli (1950, p. 195) subsequently introduced the now accepted term "residual shearing stress", or more simply, "residual strength".

These ideas were largely neglected until the 1963 Rankine Lecture (Skempton, 1964). In this paper, Skempton re-analysed a number of previously studied landslides, for which standard soil shear strength tests (for peak values) had suggested that failure had occurred when the factor of safety was greater than unity (see p. 140). that is, when sufficient strength should have been present to prevent failure. Such discrepancies were first noted by Terzaghi (1936), and have been well-documented by other workers (Bjerrum, 1967; Gould, 1960; Kenney, 1968).

Table 4.1 lists some of the best-documented case records. In the right-hand columns, the effective-stress strength parameters (C' and ϕ') determined in the laboratory, are compared with the field value of ϕ' at the moment of failure, computed on the assumption that $C' = 0$. The discrepancies are readily apparent.

Skempton (1964) was able to show that the shear strength at failure must have been close to the residual strength, as measured by the technique to be described in Section 4.4. This discovery initiated a great deal of research into the nature and measurement of residual strength, and its importance for predictions of slope stability. Since the measurement of residual strength is fundamental in the present study, a brief review of some of the recent advances in this field is given below.

4.3 THE NATURE OF RESIDUAL STRENGTH

In a conventional shear strength test, it is found that as the clay is strained, so it builds up an increasing resistance; the maximum value which this resistance reaches is the "peak strength". In normal practice the

Table 4.1

Landslide Case Records

Name of slide	C' in tons per square metre	ϕ in degrees	ϕ'_r in degrees	ϕ' in degrees, computed from landslide
California Coast	-	25	-	12
Waco Dam	4	17	6	7 - 8
Saskatchewan	4	20	6	7 - 9
Dunvegan Hill	3.8	20	-	9
Little Smoky	2.0	22	-	12
Seattle Freeway	-	30	-	13
Balgheim	1.5	18	17	14 - 17
Sandnes	1.3	22	12 - 18	15 - 17
Jackfield	1.1	25	19	17
Walton's Wood	1.6	21	13	13
London Clay	1.6	20	16	16
Sudbury Hill	0.3	17	16	15

(Source: After Bjerrum, 1967).

test is stopped as soon as the peak strength has been clearly defined, and the value referred to simply as the "shear strength" of the specimen.

However, it is now well-known that if a shear test is continued beyond the point at which the peak strength is reached, the strength of the soil will decrease, and ultimately reach a certain value, the residual strength, which remains constant for any further straining. (Figure 4.2) (Haefeli, 1958; Hvorslev, 1960; Skempton, 1964; Bjerrum, 1967; Skempton & Hutchinson, 1969). Haefeli (1966, p. 143) defines residual strength as

> " .. all the shear strength remaining along the slip surface after the loss of soil cohesion...";

Skempton & Petley (1967, p.32) define it as

> "..the minimum possible resistance to shear".

It has been suggested that the residual strength represents a "critical state" at which a constant voids ratio[1] is maintained in the sample (Roscoe, Schofield, and Wroth, 1958; La Gatta, 1970; Parry, 1971), but this has not been investigated fully.

A series of tests under different effective pressures shows that the residual strengths are in accord with the Coulomb-Terzaghi law (Skempton, 1964), and they may thus be represented by the relation -

$$S_r = C'_r + \sigma' \tan \phi'_r \qquad .....(6)$$

It is generally considered that C'_r (the residual cohesion) is zero or negligibly small (Bjerrum, 1971; de Beer, 1967;

1. Voids ratio is the ratio of the volume of voids to the volume of solids in the soil sample (Karol, 1960).

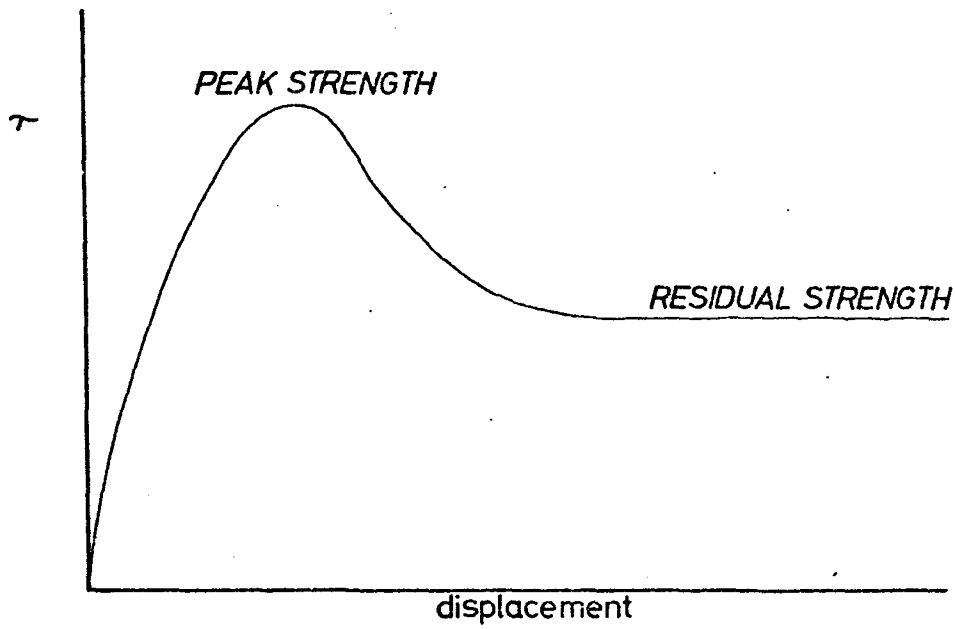


FIGURE 4.2
GENERALISED PEAK-RESIDUAL STRENGTH
PROPERTIES OF A SOIL

Skempton, 1964), and that residual strength is therefore of a frictional character only (Haefeli, 1966); however, Cullen & Donald (1971) have measured significantly non-zero values, as has the present writer (see Section 4.5).

In addition to the decrease in the cohesion component (C') on passing the peak strength, the angle of shearing resistance, ϕ', also decreases (James, 1971). In some soils the decrease is only one or two degrees, but in others it may be as much as 10 degrees (Skempton, 1964). The magnitude of this decrease is related to the strength of the soil (Bjerrum, 1967). Plastic soils show a very small decrease; brittle soils show a larger change.

To describe this phenomenon, Bishop (1967) has proposed to define the Brittleness Index, I_B, as the percentage reduction in strength on passing from the peak to the residual state.

Thus

$$I_B = \frac{S_f - S_r}{S_r} \quad (\%)$$

This is approximately equal to $(1 - \lambda_R) \times 100\%$, where λ_R us Haefeli's "residual coefficient" (Haefeli, 1965), which is defined as the ratio of residual shear strength to peak shear strength.

Hence,

$$\lambda_R = \frac{\tan \phi_r}{\tan \phi_f} = \frac{S_r}{S_f} = \frac{S_f - C}{S_f} = 1 - \frac{C}{S_f}$$

The greater the proportion of cohesion C in the peak value, the smaller the coefficient λ_R. However, this expression is in error in that it considers loss in cohesion to be the only change on passing from peak to residual strength conditions, whereas as noted previously, ϕ' also decreases. Hence the Brittleness Index is the preferable parameter for describing the loss in strength of a sample on passing to the residual state.

The Brittleness Index (I_B) varies considerably with the normal stress, being high at low stresses, and hence at the ends of slip surfaces, where the depth of soil above the failure plane is less. The relationship of I_B to normal stress is shown in Figure 4.3.

The decrease in the angle of shearing resistance referred to above is believed to be due largely to the formation of zones within the soil in which the platy clay particles are oriented in the direction of shear, as a result of "polishing" in the slip surface. This mechanism was first suggested by Hvorslev (1960), and Morgenstern & Tchalenko (1967) have since observed this preferred orientation of particles in their sections prepared from actual shear surfaces. They found that in the London Clay the zone showing

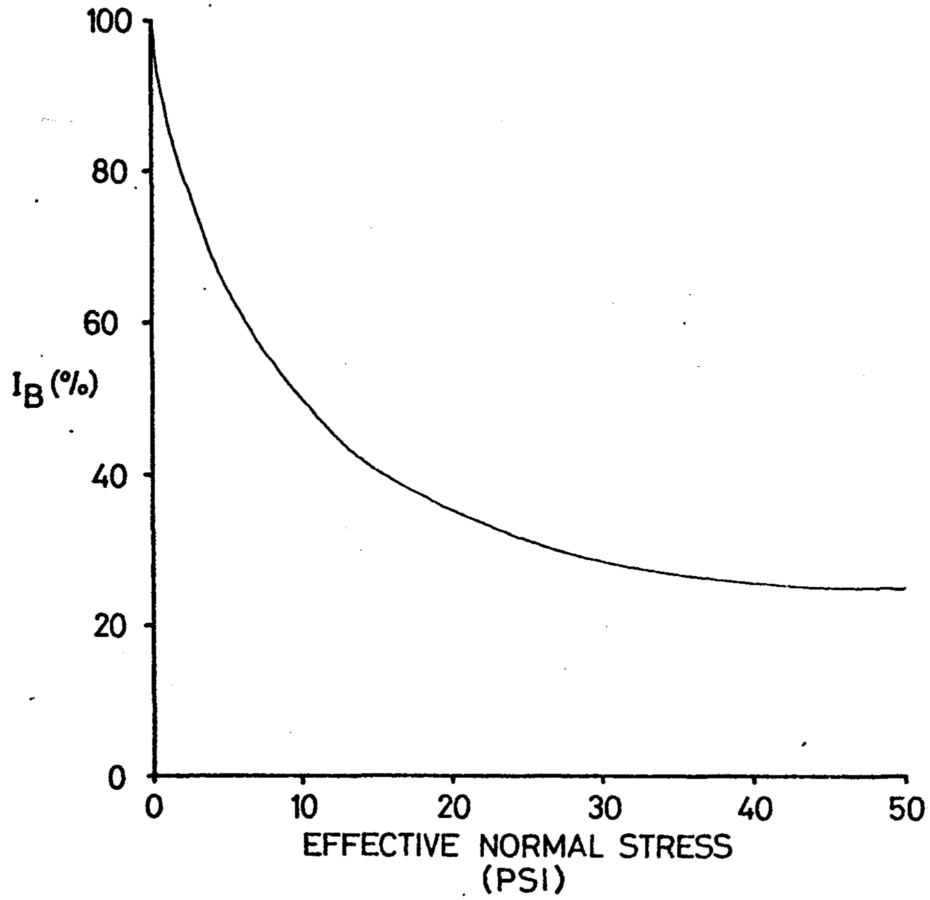


FIGURE 4.3
RELATIONSHIP OF I_B TO NORMAL STRESS
(After Bishop, 1967)

strong orientation was typically only 2 - 5 mm across. Skempton (1964, p. 83) found that natural slip surfaces in the Walton's Wood clay consisted of a continuous band in which the clay particles were very strongly orientated; this zone had a thickness of about 20μ. In addition to the main slip surface, Skempton found several secondary slip domains in which the clay showed moderate orientation (not necessarily parallel to the slip surface), and having a thickness of up to about one inch. On either side of this zone the clay was found to exhibit scarcely any orientation.

4.3.1 Factors Controlling the Residual Angle of Shearing Resistance (ϕ'_r)

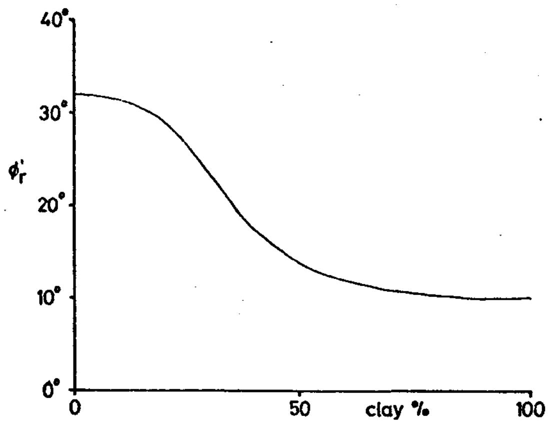
Few studies have been made of the soil characteristics which influence the residual strength, and a full understanding of this subject has yet to be reached.

Skempton (1964) suggested that ϕ'_r was inversely related to clay content, in the belief that with increasing clay content, a more perfect preferred orientation pattern could be formed, leading to a lower frictional resistance on the shear plane; in addition he suggested that an appreciable silt content might tend to increase the value of ϕ'_r by the influence of its higher angle of shearing resistance, but this suggestion has not been investigated. The relationship of ϕ'_r to clay content obtained by Skempton (1964) is shown in Figure 4.4a.

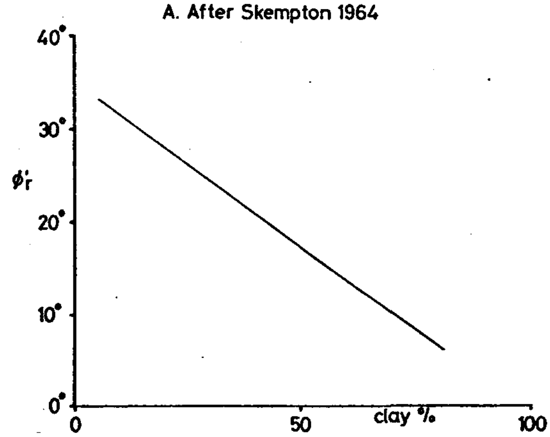
The results of Petley (1966, quoted by La Gatta, 1970) indicate a similar relationship of ϕ'_r to clay content (see Figure 4.4b); however, data taken from Kenney (1967) (see Figure 4.4c) fail to show any clear relationship between the two.

Kenney (1966, 1967) has made the only studies of the effect on ϕ'_r of the mineral composition of the soil, and of the size and shape of the particles composing it. Kenney (1967) tested samples of massive minerals, layer-lattice minerals, mineral mixtures, and artificial silica in the direct-shear technique to be subsequently described. He concluded that:

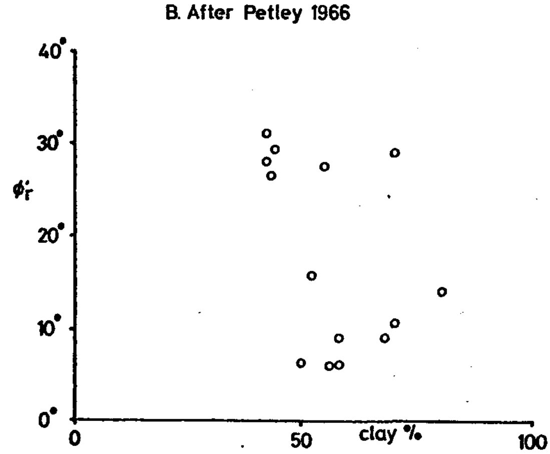
1) The residual strength of a soil is dependent on its mineral composition. The samples of layer-lattice minerals gave smaller values of ϕ'_r than the massive minerals. To a lesser extent ϕ'_r is also dependent on what Kenney (1967) has called the "system chemistry" - that is, the composition of the pore fluids and the nature of the ions adsorbed on the mineral particles.
2) The residual strength was not dependent in the range of testing on grain size or shape.



A. After Skempton 1964



B. After Petley 1966



C. Data from Kenney 1966

FIGURE 4.4
RELATIONSHIP OF ϕ_r' TO CLAY CONTENT

3) The residual strength is related to the effective normal stress.
4) The residual strength was not affected by the rate of strain.

Kenney (1966) tested various natural soils. He found that soils having the lowest values of ϕ'_r were those containing the largest amounts of the two low - ϕ'_r mineral groups, montororillonite and kaolin, and that as the content of massive minerals increased, so also did the value of ϕ'_r.

Apart from the findings of the studies reported above, little is known about the precise factors which control residual strength. In particular, no explanation has been offered as to why a residual cohesion component (C'_r) is sometimes present in measured values of residual strength, whilst in many samples it is zero or negligibly small.

4.4 THE MEASUREMENT OF RESIDUAL STRENGTH

The measurement of residual strength has been the subject of considerable debate amont soil engineers. The reader is referred to the recent papers of La Gatta (1970), Bishop et al. (1971), Cullen & Donald (1971), and references given in these papers, for amplification of the comments made here.

Residual strength is generally determined from one or more of three types of test -

1) Reversing direct shear box test
2) Triaxial compression test
3) Ring shear test

1) Reversing direct shear box test: The direct shear box is described by Her Majesty's stationery Office (1952, pp. 358-361), Akroyd (1957, pp. 109-111), and Capper & Cassie (1966, pp. 279-281).

The majority of residual strength determinations have been made using this device. Examples may be found in the work of Hamel & Flint (1972), Kenney (1966, 1967), Skempton (1964), Skempton & Petley (1967), West et al. (1971), Herrmann & Wolfskill (1966) and Bishop & Little (1967).

The shear box only allows a total displacement of about 0.3 inches along the failure plane; since several inches displacement are often required to define a stable value of residual strength, after each forward travel the box must be pushed back to its original position and the sample re-sheared. This process is repeated until a stable value of shear strength is reached. The technique is described more fully by Skempton (1964) and Cullen & Donald (1971). Skempton (1964) noted that the method is imperfect, since the motion is unlike that which occurs on a natural shear surface, where the displacement is in one direction only. However, Skempton (1964) and Skempton & Petley (1967) have obtained good agreement between the residual strength measured in this way and the strength measured along failure surfaces in natural landslides, and this is considered to give support to this technique as a valid test procedure. In addition, this form of testing has several advantages for residual strength determination, (Cullen & Donald, 1971) including -

a) Thin samples with reasonably rapid drainage can be used.
b) Specimens can be readily oriented in the correct direction of sliding, as on the natural slope.
c) Large displacements can be obtained by repeated reversals.
d) Large areas of failure surface can be tested.
e) Specimens can be readily wire-cut after consolidation to pre-form a failure surface, if desired.

f) The apparatus is simple to use.

However, the shear box also has several disadvantages, including -

a) A decrease in the cross-sectional area occurs during the test (Hvorslev, 1939).
b) The stress distribution is complex (see Roscoe, 1953).
c) With small normal loads, the upper frame will tend to "ride" over the sample and thus make the test results unreliable (Hvorslev, 1939).

2) The triaxial compression test: The triaxial test has been used by several workers, including Chandler (1966, 1969), Skempton & Petley (1967) and Skempton & Hutchinson (1969). The results of this type of test are found to be consistent with those of the direct shear test, which is to be preferred for the reasons given above.

3) The ring-shear (or torsion) test: Various types of ring-shear apparatus have also been used to determine residual strength (for example, see de Beer, 1967; Sembenelli & Ramirez, 1969; and Bjerrum, 1971), with both disc-shaped and annular specimens. Skempton, Hutchinson & Neville (1969)

have suggested that values of ϕ'_r measured in ring-shear (where the displacement is continuously in one direction) are lower than those obtained in other tests; however, de Beer (1967, using annular specimens) found the reverse to be true. Cullen & Donald (1971) note that the ring shear test has a number of disadvantages, chief among which is that there is no direct field evidence to substantiate this form of test.

However, Bishop _et al_. (1971) have recently argued in favour of a modified ring-shear device. They found that tests in ring-shear gave much lower values of residual strength than shear box or triaxial tests, and although their results disagree with a large number of previous analyses, they believe them to be more reliable because of their internal consistencey, and since the displacement is continuous in one direction, which they suggest is a closer approximation to field conditions.

4.5 THE TECHNIQUES USED IN THE PRESENT STUDY

In the present study, residual strength was measured in a direct shear box, since this was the only type of equipment available to the writer. The evidence presented above suggests that this is the most consistent and least complex method of measuring residual strength.

The equipment used was a standard Wykeham-Farrance 6 cm x 6 cm shear box, motor driven, and having a 400 - pound proving ring. This machine was in its original state unsuitable for the measurement of residual strength, and the writer found it necessary to spend a considerable amount of time designing and constructing a number of modifications before beginning testing.

Firstly, the standard rate of strain of 0.048 inches per minute, at which the shear box was designed to operate, was considerably too fast to allow a test to be run under drained conditions, which are required to define the effective stress parameters. Such a rapid rate of shear would develop positive pore-water pressures in the soil sample as explained by Osterman (1960, pp.347-348). Cullen & Donald (1971) found that a test rate of 0.010 inches per minute was slightly too rapid to reliably define residual strength in some samples.

Therefore it was decided to reduce the shear rate by about 10 times. It was considered that the cheapest and simplest way to achieve this was to modify the chain drive connecting the motor to the screw jack on the shear box chassis. The original sprocket ratio was 12 teeth (motor) to 20 teeth (screw jack), giving a rate of advance of 0.048 inches per minute. These sprocket wheels were removed and a countershaft was mounted between the motor and the screw jack. Four new sprocket wheels were then mounted, giving a ratio of 8 (motor) to 32 (countershaft) and 8 (countershaft) to 25 (screw jack). To achieve this, metal sleeves (to compensate for differences in the internal bore of the sprocket sheels) had to be mounted on all bearing shafts, and new chain, to suit the wider sprockets, fitted on both sides of the countershaft. This arrangement gave a rate of advance of 0.0052 inches per minute, or 5.2 t.p.m. (thousandths of an inch per minute). This was subsequently found to be slightly too rapid and the ratio altered to 8:32:8:32 giving a rate of advance of 0.0048 inches per minute (4.8 t.p.m.), which was subsequently used in all tests. The arrangement of the sprocket wheels is shown in Plate 4.1.

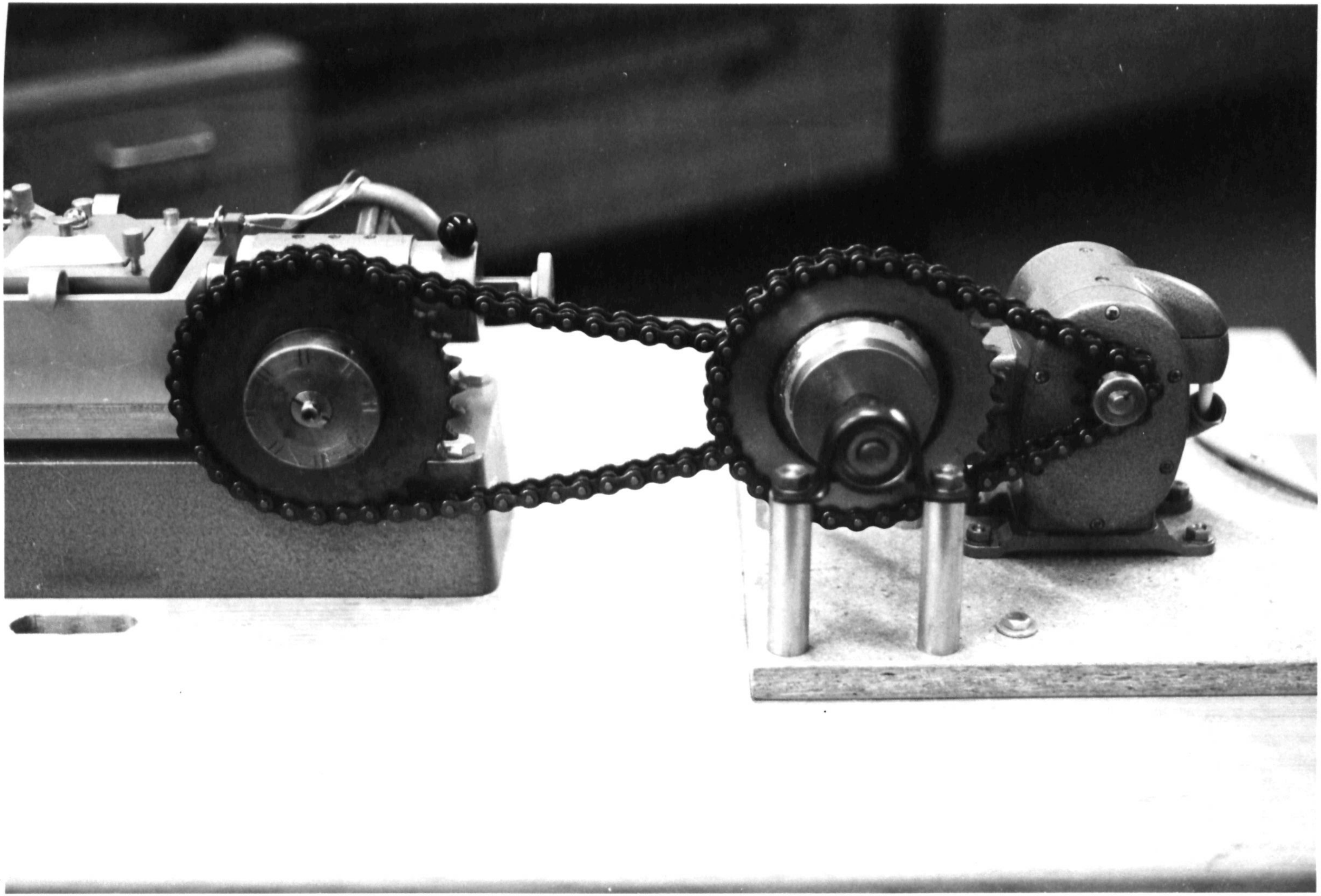


Plate 4.1 Modified shear box sprocket drive system.

By replacing the original sprocket wheels and chain, the sliding clutch had also to be removed, and hence adjustments to the shear box, necessary to mount the proving ring without tension, could not be made, since the highly down-geared motor could not be turned by hand. It was found, however, that by taking great care in setting-up before each test, no difficulty was encountered.

A second modification was required to safeguard the motor and to make the reversals of the machine automatic and motor-driven.

In the standard test procedure, the sample is sheared under motor-control in the forward direction, at the completion of which the motor must be quickly switched off (to prevent the current increasing to a damaging level, since no further displacement of the shear box is possible); the shear box carriage is then pushed by hand back to its original position, thus releasing the accumulated stress in the proving ring. This technique was used by Skempton (1964). However, it has several disadvantages:- first, the operator must be present at all times, and ready to switch off the motor at the instant the forward travel is completed.

If he happens not to notice at this particular instant, there is danger of the motor being damaged. Second, the need to perform reversals by hand demands more of the operators time, and introduces variables into the procedure, for example, the decision as to when a forward travel has been completed.

In addition, as trials in the early part of this year showed, pushing the lower half of the shear box by hand is liable to cause the two halves of the shear box to come into contact, especially when using low normal stresses, and to cause misalignment of the upper frame of the shear box, since the coupling with the shaft of the proving ring contains of necessity a certain amount of free-play.

To overcome these difficulties, it was decided to make the reversals automatic, hence safeguarding the equipment and making reversal procedure standardised and reproduceable. This was achieved by mounting two micro-switches, with both normally on and normally off positions, at opposite ends of the shear box carriage. (Plate 4.2) The position of mounting of the micro-switches was critical to the success of this idea, since the very small travel required to trigger the switches (about 0.01 inches) meant that precise location at the

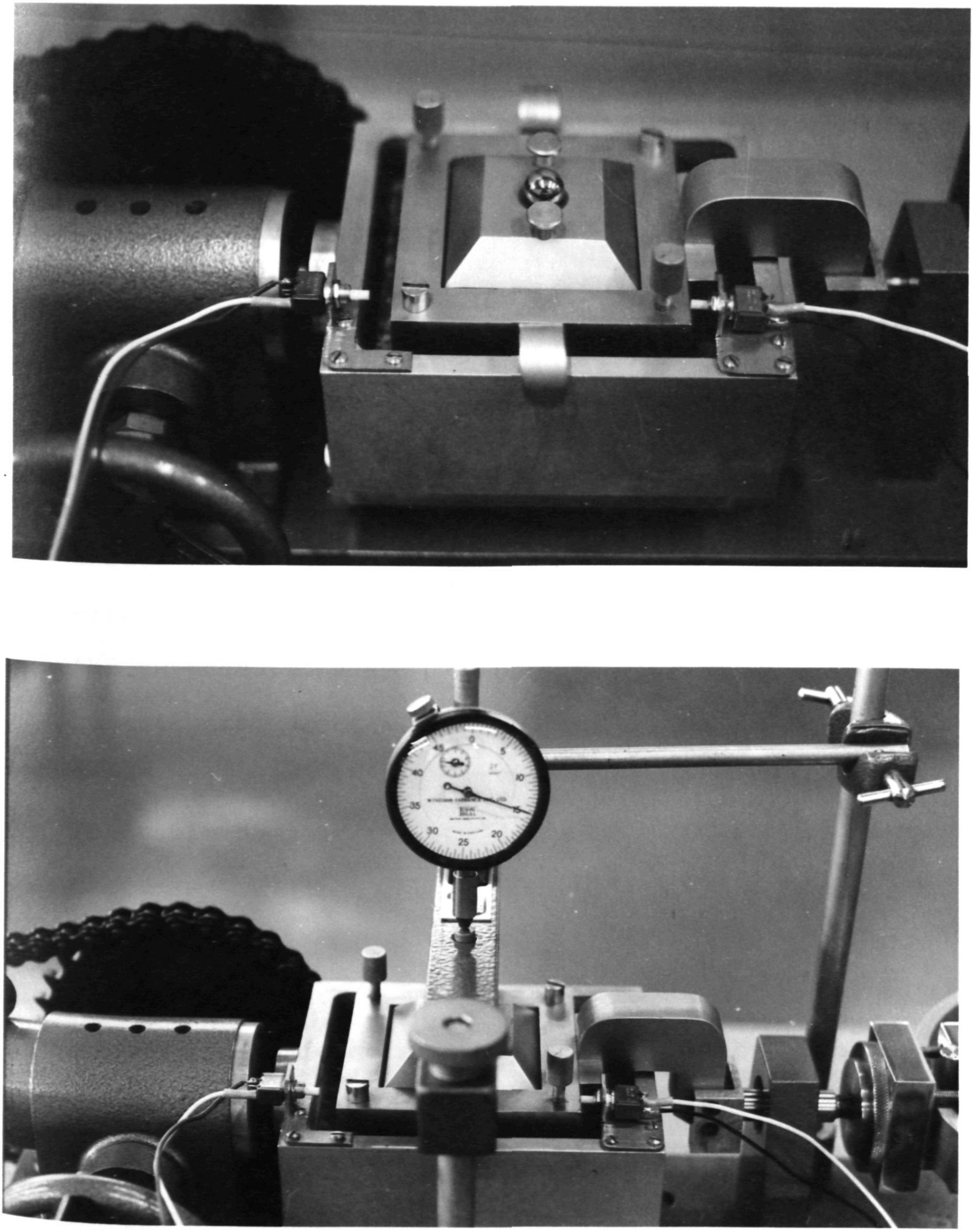


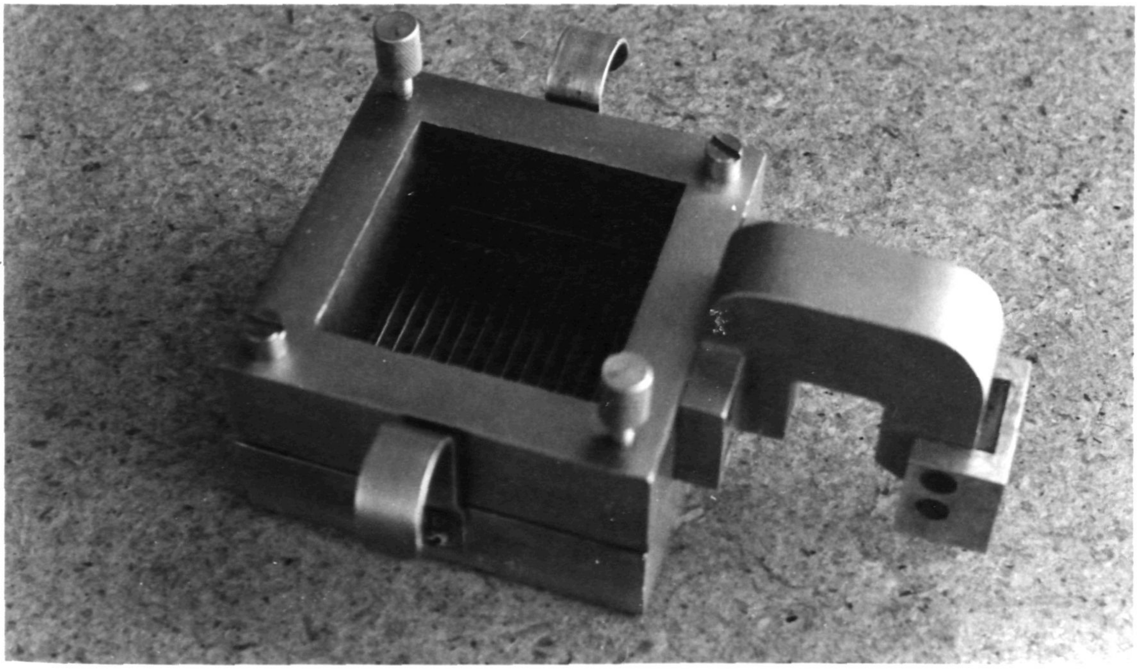
Plate 4.2 Positions of mounting of micro-switches.

extreme displacements of the shear box was necessary. The switches could not be mounted closer together, since this would have reduced the possible movement of the shear box. Therefore the writer constructed two brass brackets, each of separate design to conform to the available mounting positions on the shear box; these were attached with brass screws by drilling and tapping three holes at each end of the shear box carriage, and corresponding holes in the brackets, through which the screws could just pass. With these brackets rigidly held in place it was possible to adjust the micro-switch positions by moving the lock-nuts slightly on either side of the bracket. Then by repeated trials the switches were set in position so that they would be triggered precisely at the end of each travel of the shear box. The entire arrangement was made easily removable so that the box could be cleaned after each test.

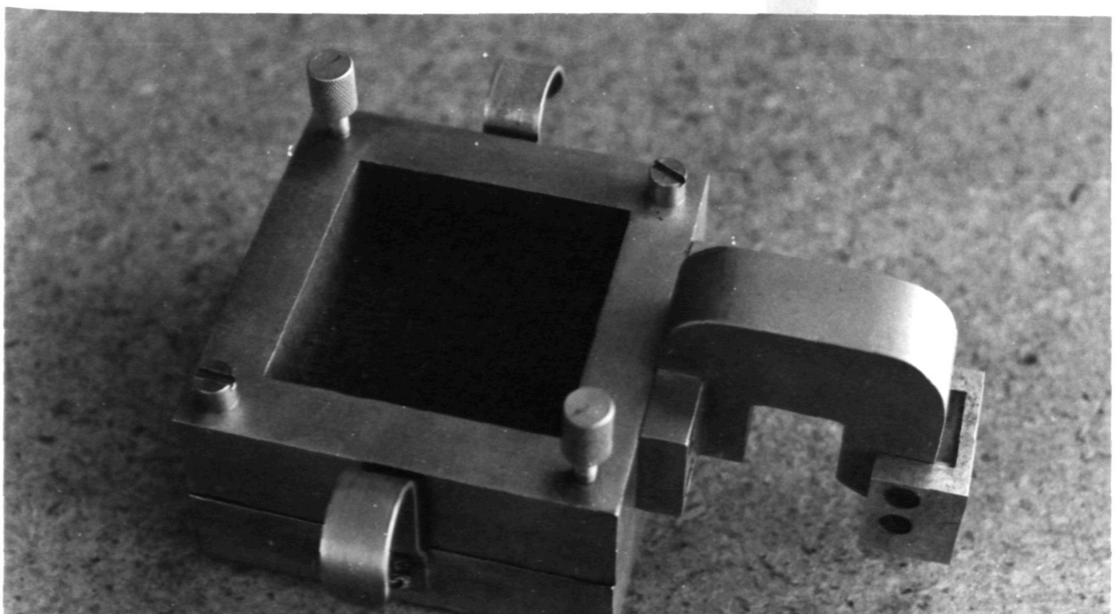
The switches were then wired to a relay switching system, especially designed and constructed for the purpose, in which the relay was off during the forward travel, but was energised when the forward motion of the shear box triggered the appropriate micro-switch. The windings of the motor were then tapped, and connected to the relay, such that tripping of the relay by the motion of the shear box would automatically reverse the direction of motion, by transferring the starter-capacitor to the opposite coil of the motor. The circuit for this system is given in Appendix D. Such a system ensures that the length of each travel is precisely the same, since it is governed by the micro-switches, and not by judgement. It was found to perform perfectly, the proving ring returning to a reading of precisely 0.00 at the end of each reversal.

Only remoulded samples were tested, these having been collected by hand augering. Sufficient soil was weighed out to give a sample of size 6 x 6 x 2 cm, with a density of about 1.5 gm cc^{-1}. This was then ground up and packed carefully into the shear box with continuous tamping-down until the sample was 2 cm thick. Perforated toothed-grids were inserted above and below the sample. (Plate 4.3) A small amount of the soil was retained and a gravimetric moisture content determined.

All samples were allowed to consolidate under the applied load until the micrometer guage indicated that the sample had reached a constant thickness. This period reached a maximum of 185 hours. The shear box carriage was filled with distilled water during this period and also during



a. Before loading sample, showing toothed-grid.



b. Completed sample.

Plate 4.3 Preparation of a soil sample.

the shear test, to ensure that the soil chemistry was not altered, and to prevent the sample from drying out.

Negative stage testing, as proposed by Cullen & Donald (1971) was employed with all samples. In this procedure, the sample, having been consolidated to the desired normal load, is sheared continuously until the residual strength is defined. The normal load is then reduced, and the same sample re-sheared until a new value of the residual strength is obtained. The sample is allowed to consolidate in response to the new load during the reverse travel. (In the present study, an additional period of up to 12 hours was allowed for consolidation after each decrease in the normal load.) This procedure is repeated until sufficient values have been obtained to accurately define ϕ'_r.

Because of the large displacements resulting from this procedure, some extrusion of the soil sample between the halves of the shear box was observed. This phenomenon was noted also by West et al. (1971), who found that it made their test results slightly erratic; no such problems were encountered by the writer, and all results were found to be precisely reproduceable (see Section 4.6).

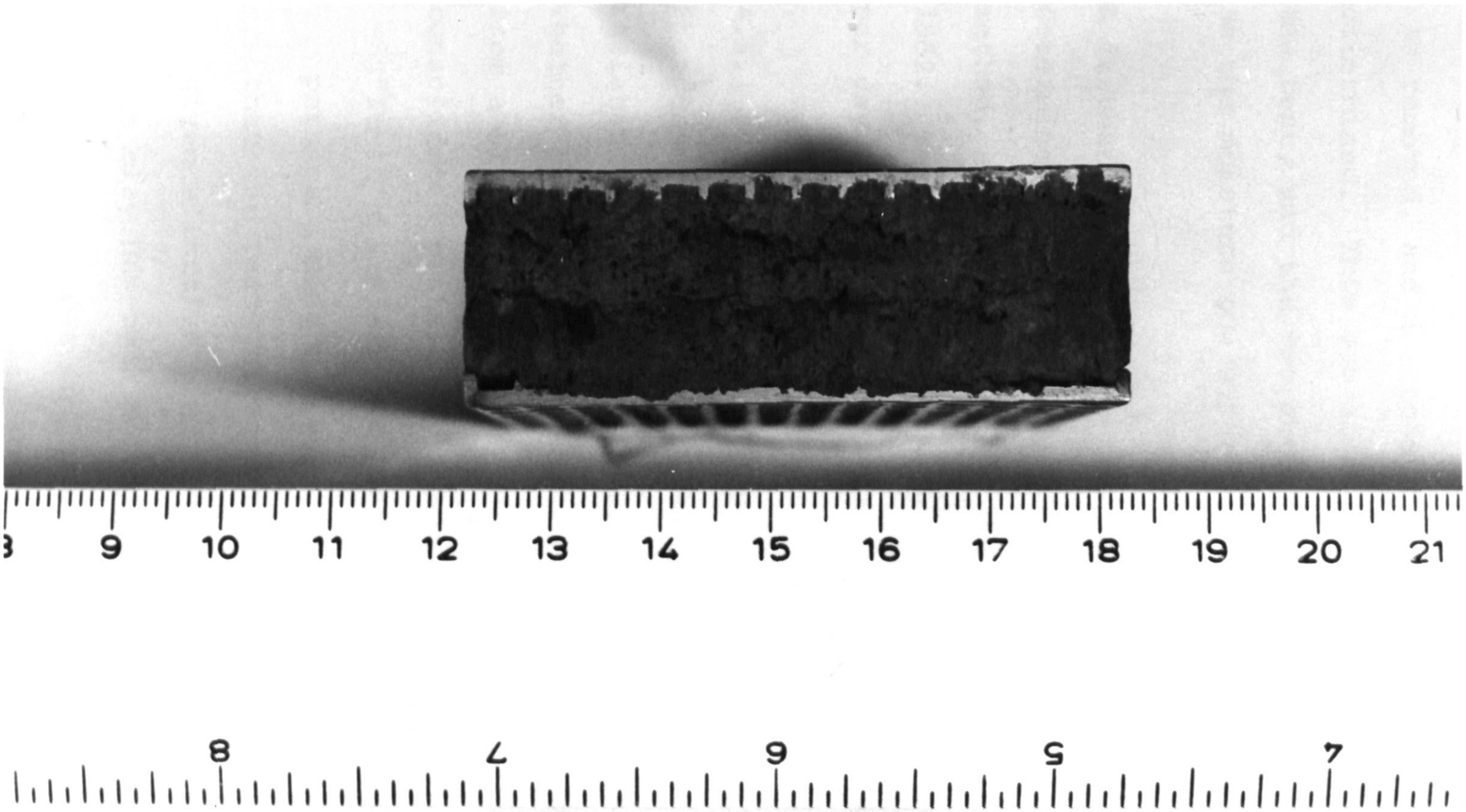


Plate 4.4 Sample after testing, showing location of the failure plane.

At the conclusion of each test, part of the sample was removed, and a second gravimetric moisture content determined. This was added to the value obtained before the test, and the two averaged. The average was taken as the moisture content of the sample during the test.

A further part of each soil sample tested was used to determine a particle-size distribution by the hydrometer method, as described by Her Majesty's Stationery Office, 1952. Testing was generally continued for about 13 hours per day for three or four days, for each sample.

4.5.1 Validity of the Use of Remoulded Samples

Skempton (1964) found that the residual angle of shearing resistance measured in the clays tested was the same whether the sample tested had been normally consolidated from a slurry, or was undisturbed (generally in an over-consolidated state). He therefore concluded -

> "It is therefore possible to suggest, with perhaps a slight degree of over-simplification, that the residual strength of a clay, under any given effective pressure, is the same whether the clay has been normally - or over-consolidated; .. if this idea is correct then the angle ϕ'_r should be a constant for any particular clay whatever its consolidation history; <u>depending</u>

only on the nature of the particles."
(Skempton, 1964, p.83).

This conclusion has since been confirmed by further experimental studies. For example, Bishop et al. (1971) have reported the results of a large number of tests using both undisturbed and remoulded samples. Some of these results are summarised in Table 4.2. As can be seen, there is no significant difference between the values obtained by the two methods.

Similar conclusions were reached by Kenney (1967), who found no residual strength difference when using undisturbed or re-moulded samples.

Indeed, these conclusions are predictable from a knowledge of what the residual strength represents. It is the strength measured along a failure surface which has completely disrupted the soil sample; any prior soil structure is destroyed by this action, and the strength measured is essentially that produced by the friction between the two halves of the specimen. Thus Kenney (1966) describes his test procedure as follows -

"Straining was continued, by driving the apparatus forwards and backwards many times,

Table 4.2

Residual angles of shearing resistance measured from undisturbed and remoulded samples

Sample	Test condition	ϕ'_r degrees
London Clay No.1	Remoulded	9.5
	Undisturbed	9.4
London Clay No.2	Remoulded	12.5
	Undisturbed	13.5
London Clay No.3	Remoulded	14.5
	Undisturbed	14.5
London Clay No.4	Remoulded	11.4
	Undisturbed	11.3

Source: Data taken from Bishop *et al*. (1971).

until the shear resistance of the sample had become constant and equal to its ultimate or residual value. In this way the effect on shear resistance of the fabric of the sample was brought to a common level in all tests." (Kenney, 1966, p. 53).

It may therefore be concluded that, as the data in Table 4.2 shows, the use of remoulded soil samples gives identical results to those obtained from undisturbed samples. Therefore the use of undisturbed samples is unnecessary in the case of residual strength, and the procedure adopted in the present study is considered to be valid.

In a similar way all studies (for example, Skempton, 1964; La Gatta, 1970; and Cullen & Donald, 1971) have found that ϕ'_r is essentially independent of soil moisture content. In the present study, samples were tested as near as possible to the field moisture content, which following Webb & Collins (1967) may be defined as the moisture content, expressed as a percentage by weight of the over-dry soil, of a soil in the field at any time. This was done purely for consistency, and any drying-out of the samples should have had little effect on the residual strength.

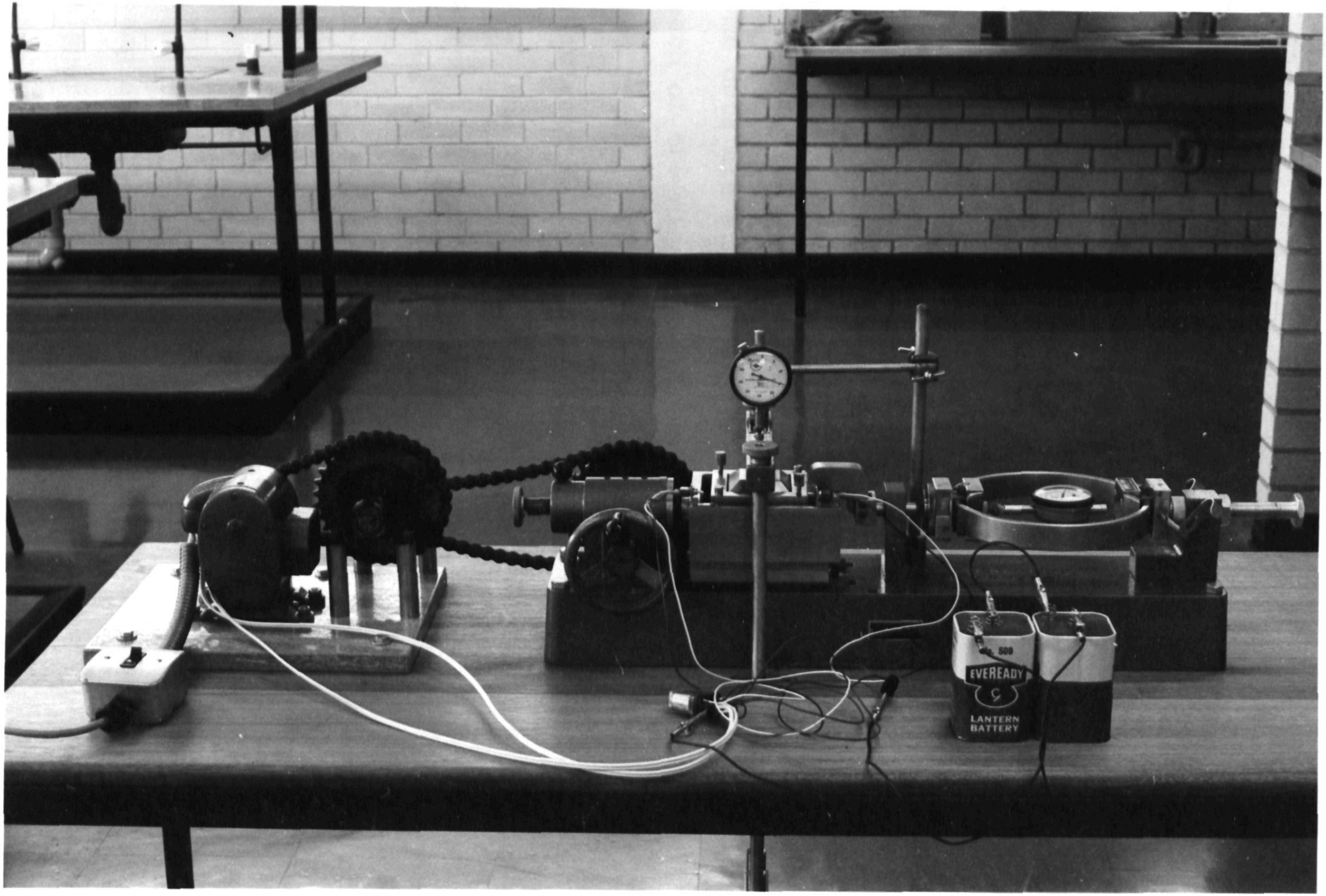


Figure 4.5 The complete shear box arrangement.

4.6 RESULTS OF SHEAR STRENGTH DETERMINATIONS

Nine soil samples, of which 8 were from soil B horizons and one from an A horizon, were tested during 49 days of laboratory work. These samples represented the soils from four slope profiles of varying form and inclination, as shown in Table 4.3. The positions on the slopes from which each sample was taken are shown in Figure 4.5.

One hundred and twenty five complete strength tests were performed on these samples, at varying normal stresses. The data are summarised in Table 4.4. The test data for each of the nine samples are summarised in Table 4.5 and Figure 4.6. The results of each test in terms of resultant values of ϕ'_r and C'_r are summarised in Table 4.6, and the relationship of residual strength to normal stress for each sample in Figure 4.7.

As Table 4.6 shows, disregarding the sample from the soil A-horizon (which was tested principally to check machine operation), six of the eight samples tested gave values of ϕ'_r in the range 20-25 degrees. The exceptions are samples P28 H3 90-120 and P22 H2 90-120, which gave much higher values (40 degrees and 38 degrees). These tests were respectively the first attempted (which was interr-upted by several errors in test procedure) and a test

Table 4.3

Details of slopes from which Samples were taken for Shear Strength Determination

Slope Profile Number	Mean Angle	Number of Samples
P 1	17.6	2
P11	13.5	3
P22	17.3	1
P28	13.9	3

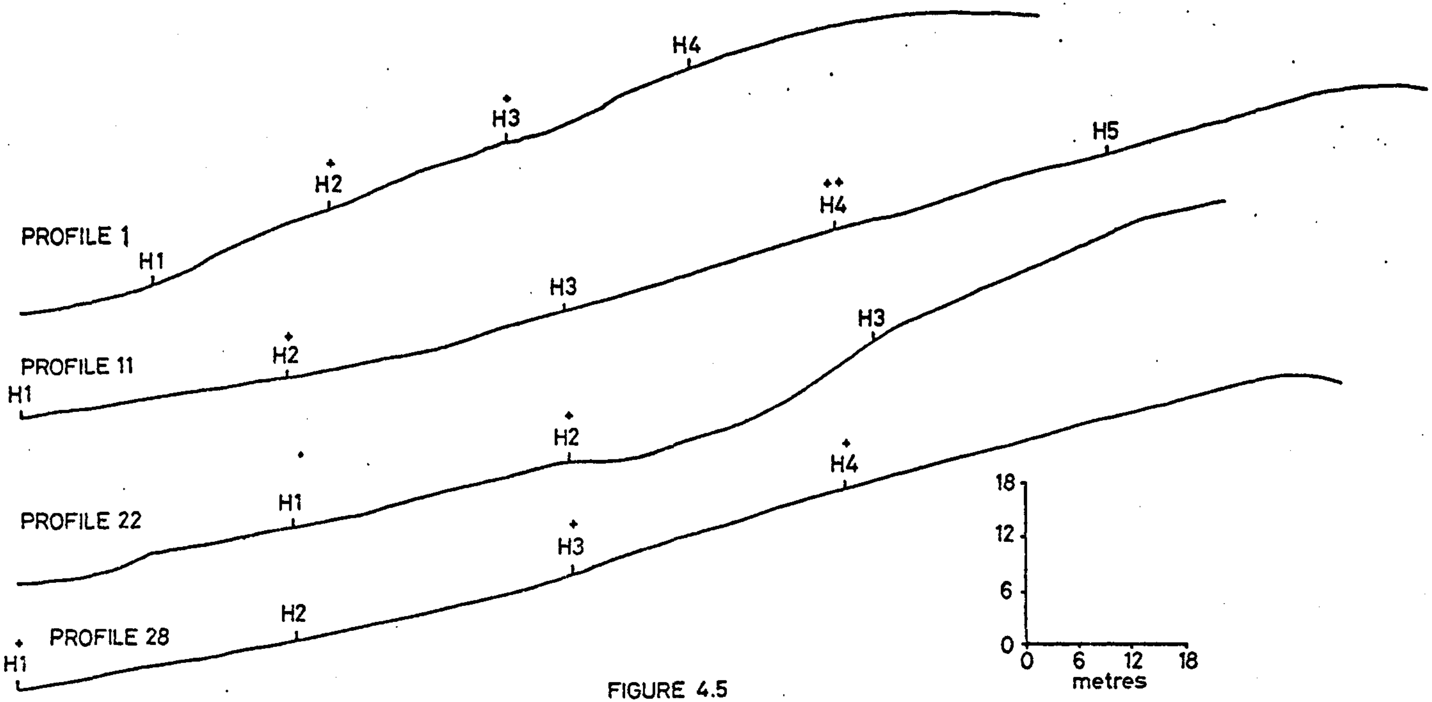


FIGURE 4.5

SITES FROM WHICH SOIL SAMPLES WERE TAKEN.

+ INDICATES TESTED SAMPLE

Table 4.4

Details of Tests Carried out on Each Soil Sample

Sample	Soil horizon represented	Number of tests performed	Number of different normal stress levels used
P1 H290-120	B	15	6
P1 H335-75	B	16	7
P11 H290-120	B	10	7
P11 H40-50	A	15	6
P11 H450-75	B	9	7
P22 H290-120	B	15	7
P28 H195-125	B	14	7
P28 H390-120	B	20	5
P28 H440-65	B	14	5

Table 4.5

Summarised Shear Strength Test Data

A) Sample Pl H290-120

Normal Stress, lb in^{-2}	Shear Strength, lb in^{-2} Minimum	Maximum
8.76	4.20	4.64
6.97	3.61	4.06
6.08	3.15	3.76
5.18	2.85	3.35
4.29	2.54	3.15
3.39	2.24	2.54

Results

	Minimum	Maximum
Average ϕ'_r	20 degrees	20 degrees
Average C'_r	0.57	0.96
Correlation coe-ficient	0.99	0.99
Regression equation	$S = 0.37\sigma_v + 0.95$	$S = 0.37\sigma_v + 1.40$

Test Information

Average moisture content: 21%

Pre-consolidation period: 43 hours

Total test duration: 5 days

Stain rate: 0.052 in min^{-1} Percentage clay: 50

Total displacement on shear plane: 7.5 inches

Table 4.5 (continued)

B) Sample Pl H3 35-75

Normal Stress, lb in^{-2}	Shear Strength, lb in^{-2}
9.66	4.82
8.76	4.52
7.87	3.98
6.97	3.91
5.18	3.07
3.84	2.91
2.50	2.36

Results

Average ϕ'_r: 22 degrees

Average C'_r: 0.61

Correlation coeficient: 0.99

Regression equation: $S = 0.34\sigma_v + 0.99$

Test Information

Average moisture content: 22%

Pre-consolidation period: 35 hours

Total test duration: 5 days

Strain rate: 0.052 in min^{-1} Percentage clay: 52

Total displacement on shear plane: 8 inches

Table 4.5 (continued)

C) Sample P11 H2 90-120

Normal Stress, lb in^{-2}	Shear Strength, lb in^{-2}
8.76	4.91
6.97	4.43
6.08	4.04
5.18	3.66
4.29	3.14
3.39	2.86
1.61	2.06

Results

Average ϕ'_r: 21 degrees

Average C'_r: 1.00

Correlation coefficient: 0.99

Regression equation: $S = 0.41\sigma_v + 1.45$

Test Information

Average moisture content: 22%

Pre-consolidation period: 40 hours

Total test duration: 5 days

Strain rate: 0.048 in min^{-1}

Total displacement on shear plane: 5 inches

Percentage clay: 55

Table 4.5 (continued)

D) Sample P11 H4 0-50

Normal Stress, lb in^{-2}	Shear Strength, lb in^{-2}
8.76	7.14
6.97	5.30
6.08	4.66
5.18	4.09
4.29	3.47
3.39	2.85
1.61	1.68

Results

Average ϕ'_r: 37 degrees

Average C'_r: 0.04

Correlation coefficient: 0.99

Regression equation: $S = 0.74\sigma_v + 0.32$

Test Information:

Average moisture content: 25%

Pre-consolidation period: 23 hours

Total test duration: 6 days

Strain rate: 0.048 in min^{-1}

Total displacement on shear plane: 7.5 inches

Percentage clay: 30.0

Table 4.5 (continued)

E) Sample P11 H4 50-75

Normal Stress, lb in^{-2}	Shear strength, lb in^{-2}
8.76	4.14
6.97	3.65
6.08	3.30
5.18	2.97
4.29	2.64
3.39	2.13
1.61	1.51

Results

Average ϕ'_r: 21 degrees

Average C'_r: 0.57

Correlation coefficient: 0.99

Regression equation: $S = 0.38\sigma_v + 0.94$

Test Information

Average moisture content: 25%

Pre-consolidation period: 24 hours

Total test duration: 4 days

Strain rate: 0.048 in min^{-1}

Total displacement on shear plane: 4.5 inches

Percentage clay: 45

Table 4.5 (continued)

F) Sample P22 H2 60-90

Normal Stress, lb in^{-2}	Shear Strength, lb in^{-2}
8.76	6.95
6.97	5.28
6.08	4.71
5.18	4.06
4.29	3.52
3.39	2.94
1.61	1.48

Results

Average ϕ'_r: 38 degrees

Average C'_r: 0.03

Correlation coefficient: 0.99

Regression equation: $S = 0.74\,\sigma_v + 0.30$

Test Information

Average moisture content: 16%

Pre-consolidation period: 163 hours

Total test duration: 7 days

Strain rate: 0.048 in min^{-1}

Total displacement on shear plane: 6 inches

Percentage clay: 48

Table 4.5 (continued)

G) Sample P28 H1 95-125

Normal Stress, lb in^{-2}	Shear Strength, lb in^{-2}	
	Minimum	Maximum
8.76	4.22	5.27
6.97	3.69	4.38
6.08	3.15	3.91
5.18	3.00	3.47
4.29	2.54	3.15
3.39	1.94	2.70
1.61	1.33	1.80

Results

	Minimum	Maximum
Average ϕ'_r:	23 degrees	26 degrees
Average C'_r:	0.34	0.65
Correlation coefficient:	0.99	0.99
Regression equation:	$S = 0.42\sigma_v + 0.67$	$S = 0.48\sigma_v + 1.04$

Test Information:

Average moisture content: 25%

Pre-consolidation period: 60 hours

Total test duration: 6 days

Strain rate: 0.048 in min^{-1} Percentage clay: 55

Total displacement on shear plane: 7 inches

Table 4.5 (continued)

H) Sample P28 H3 90-120

Normal Stress, lb in^{-2}	Shear Strength, lb in^{-2}
7.33	8.32
6.44	7.41
5.55	6.95
4.65	6.62
3.39	4.82

Results

Average ϕ'_r: 40 degrees

Average C'_r: 1.76

Correlation coefficient: 0.97

Regression equation: $S = 0.81\sigma_v + 2.34$

Test Information

Average moisture content: 29%

Pre-consolidation period: 16 hours

Total test duration: 7 days

Strain rate: 0.052 in min^{-1}

Total displacement on shear plane: 10 inches

Percentage clay: 52

Table 4.5 (continued)

I) Sample P28 H4 40-65

Normal Stress, lb in^{-2}	Shear Strength, lb in^{-2}
7.33	4.37
6.44	3.23
5.55	2.88
4.65	2.64
3.39	2.19

Results

Average ϕ'_r: 20 degrees

Average C'_r: 0.03

Correlation coefficient: 0.94

Regression equation: $S = 0.50\sigma_v + 0.30$

Test Information

Average moisture content: 21%

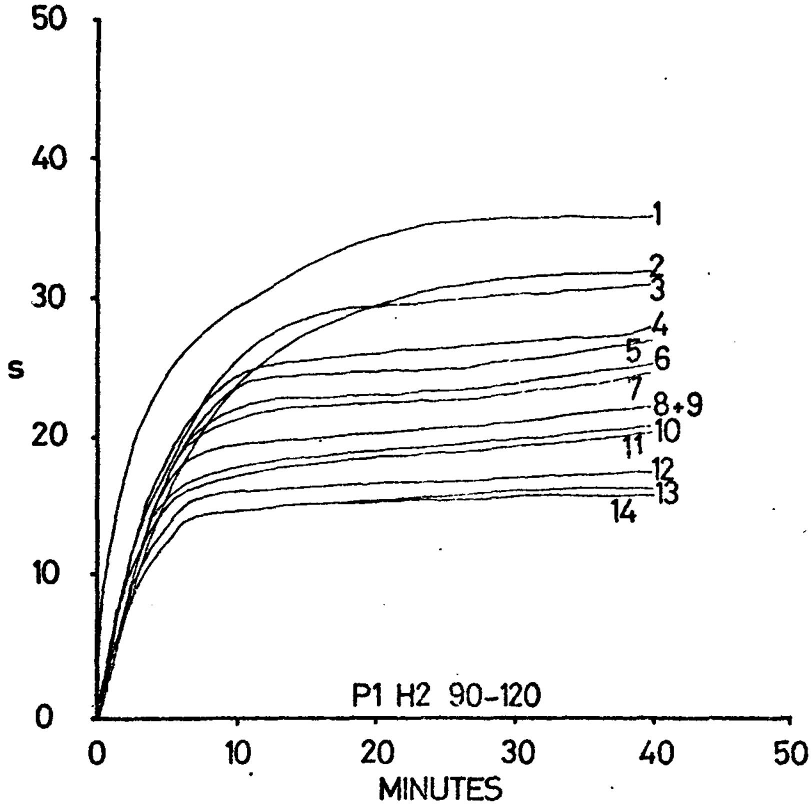
Pre-consolidation period: 45 hours

Total test duration: 4 days

Strain rate: 0.052 in min^{-1}

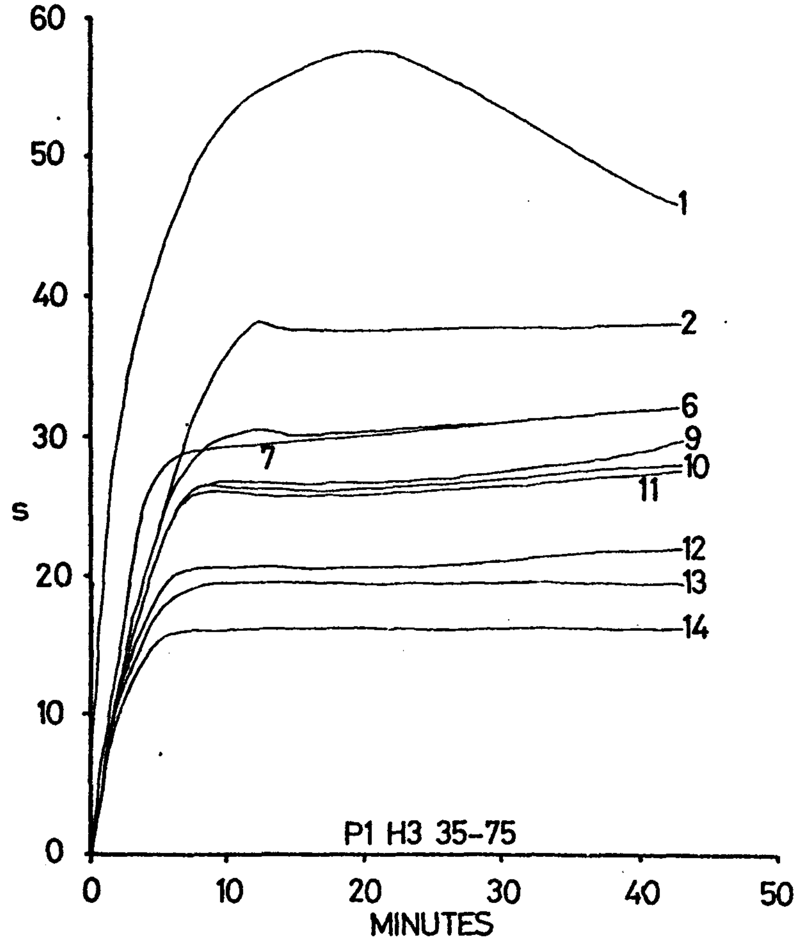
Total displacement on shear plane: 7 inches

Percentage clay: 48



50
40
30
20
10
0
s
1
2
3
4
5
6
7
8+9
10
11
12
13
14
P1 H2 90-120
0
10
20
30
40
50
MINUTES

FIG. 4.6



60
50
40
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10
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s
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2
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7
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P1 H3 35-75
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50
MINUTES

FIG. 4.6

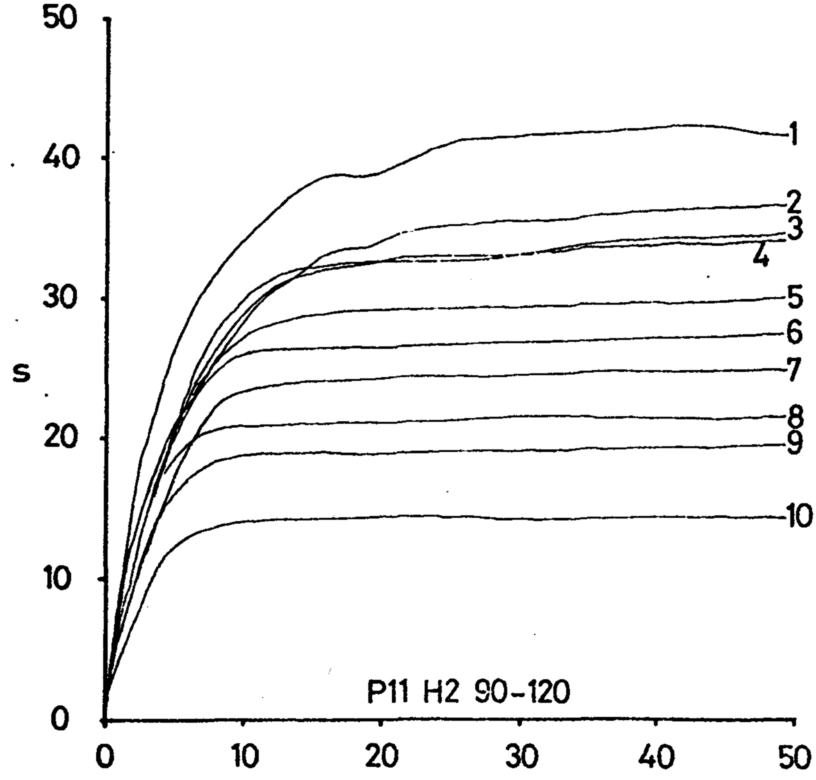
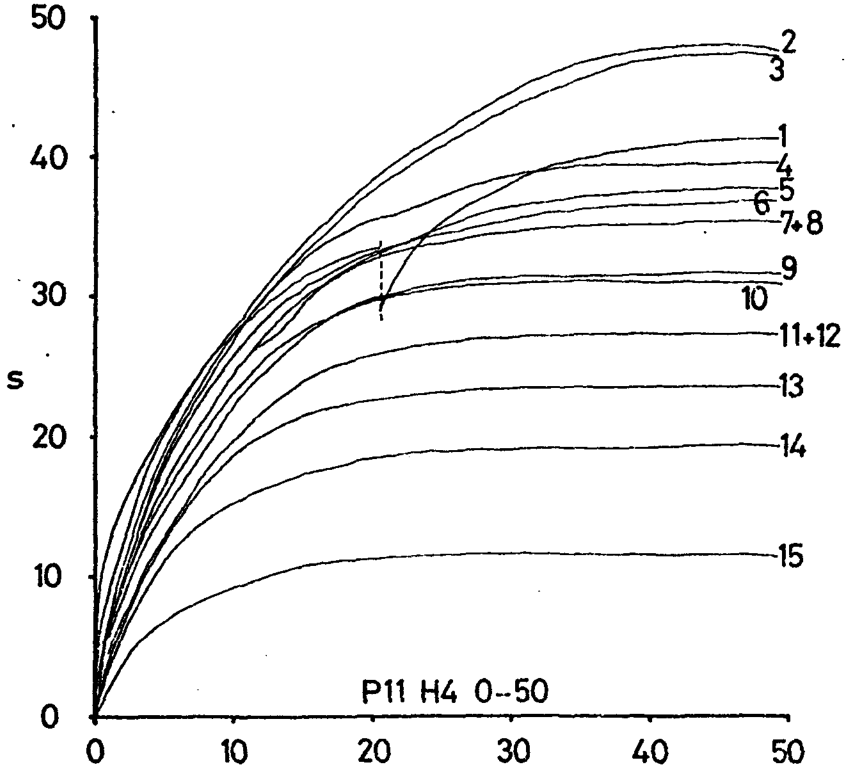


FIG. 4.6



50
40
30
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s
0
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40
50
P11 H4 0--50
2
3
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4
5
6
7+8
9
10
11+12
13
14
15

FIG. 4.6

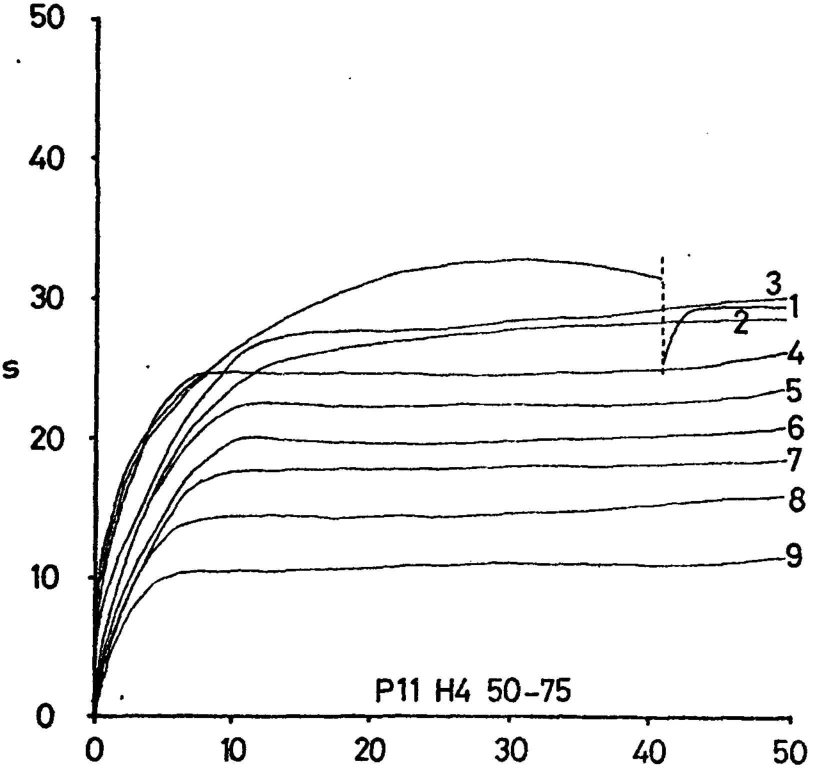
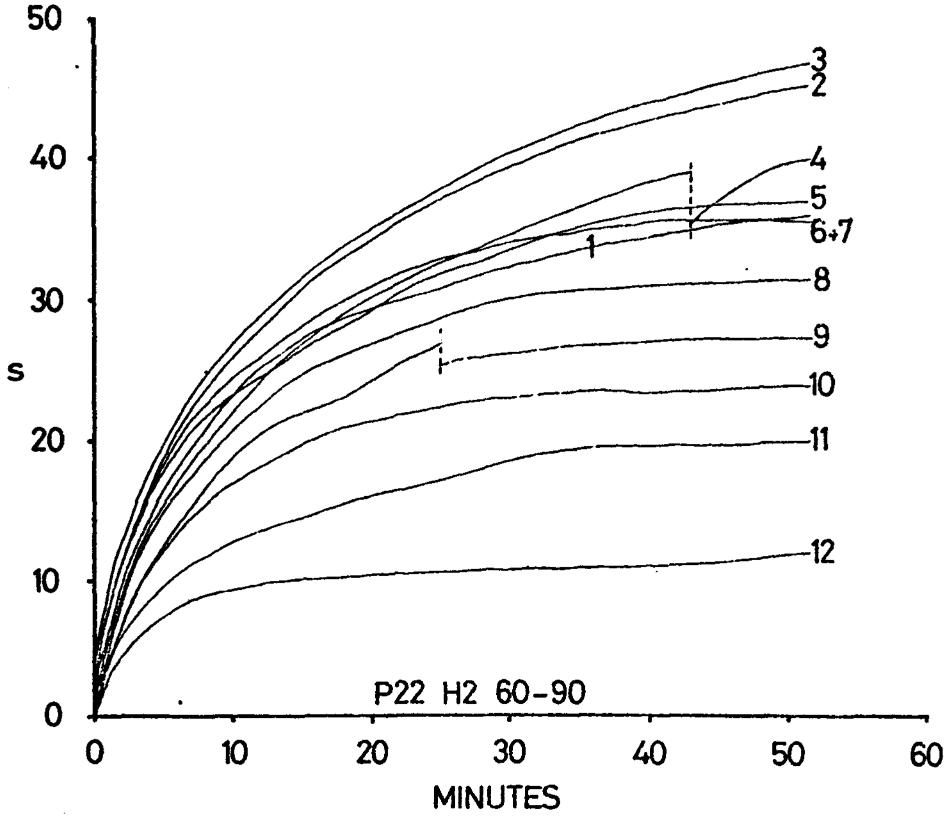


FIG. 4.6



50
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s
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6+7
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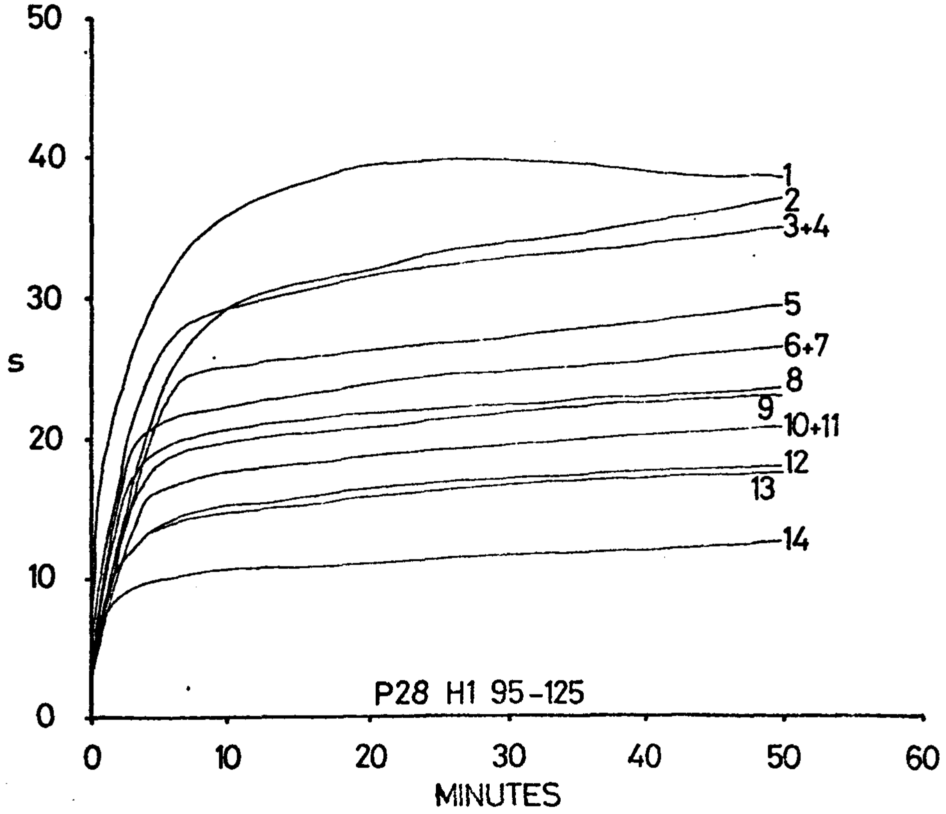
12
P22 H2 60-90
0
10
20
30
40
50
60
MINUTES

FIG. 4.6



50
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60
MINUTES
1
2
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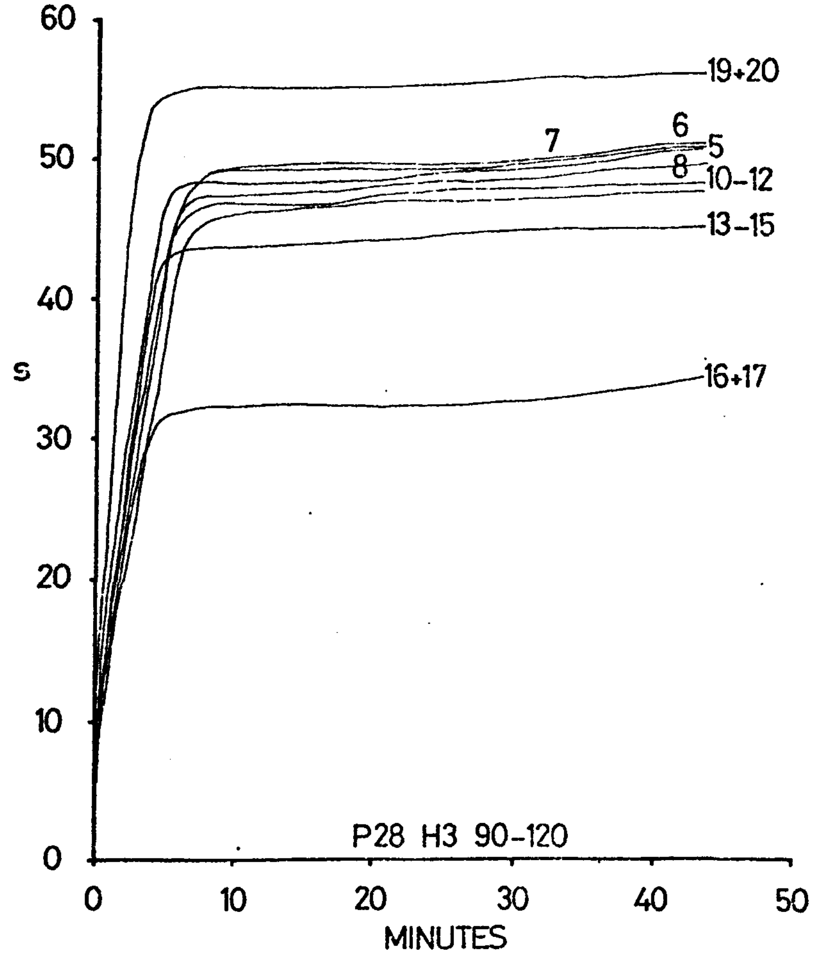
8
9
10+11
12
13
14
P28 H1 95-125

FIG. 4.6


60
50
40
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20
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s
19+20
7
6
5
8
10-12
13-15
16+17
P28 H3 90-120
0
10
20
30
40
50
MINUTES

FIG. 4.6

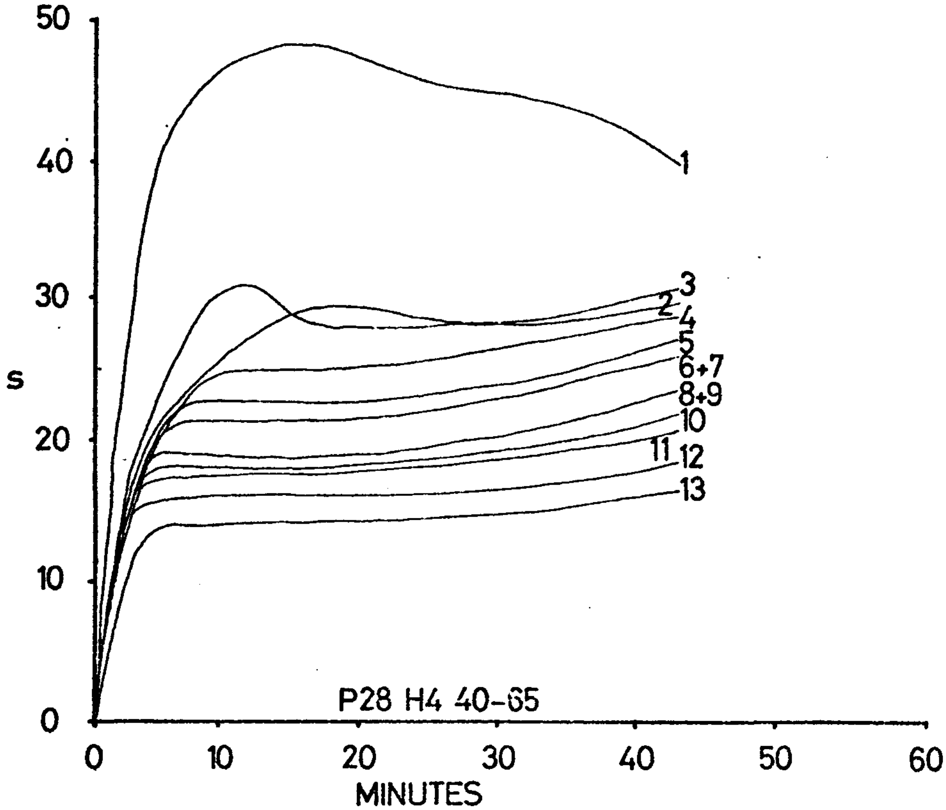

50
40
30
20
10
0
s
1
3
2
4
5
6+7
8+9
10
11
12
13
P28 H4 40-65
0
10
20
30
40
50
60
MINUTES

FIG. 4.6

Table 4.6

Summarised value of residual angle of shearing assistance and residual cohesion

Sample	ϕ'_r (degrees)	C'_r (lb in^{-2})
P1 H2 90-120	20	0.77
P1 H3 35-75	22	0.61
P11 H2 90-120	21	1.00
P11 H4 0-50	37	0.04
P11 H4 50-75	21	0.57
P22 H2 90-120	38	0.03
P28 H1 95-125	25	0.49
P28 H3 90-120	40	1.76
P28 H4 40-65	20	0.03

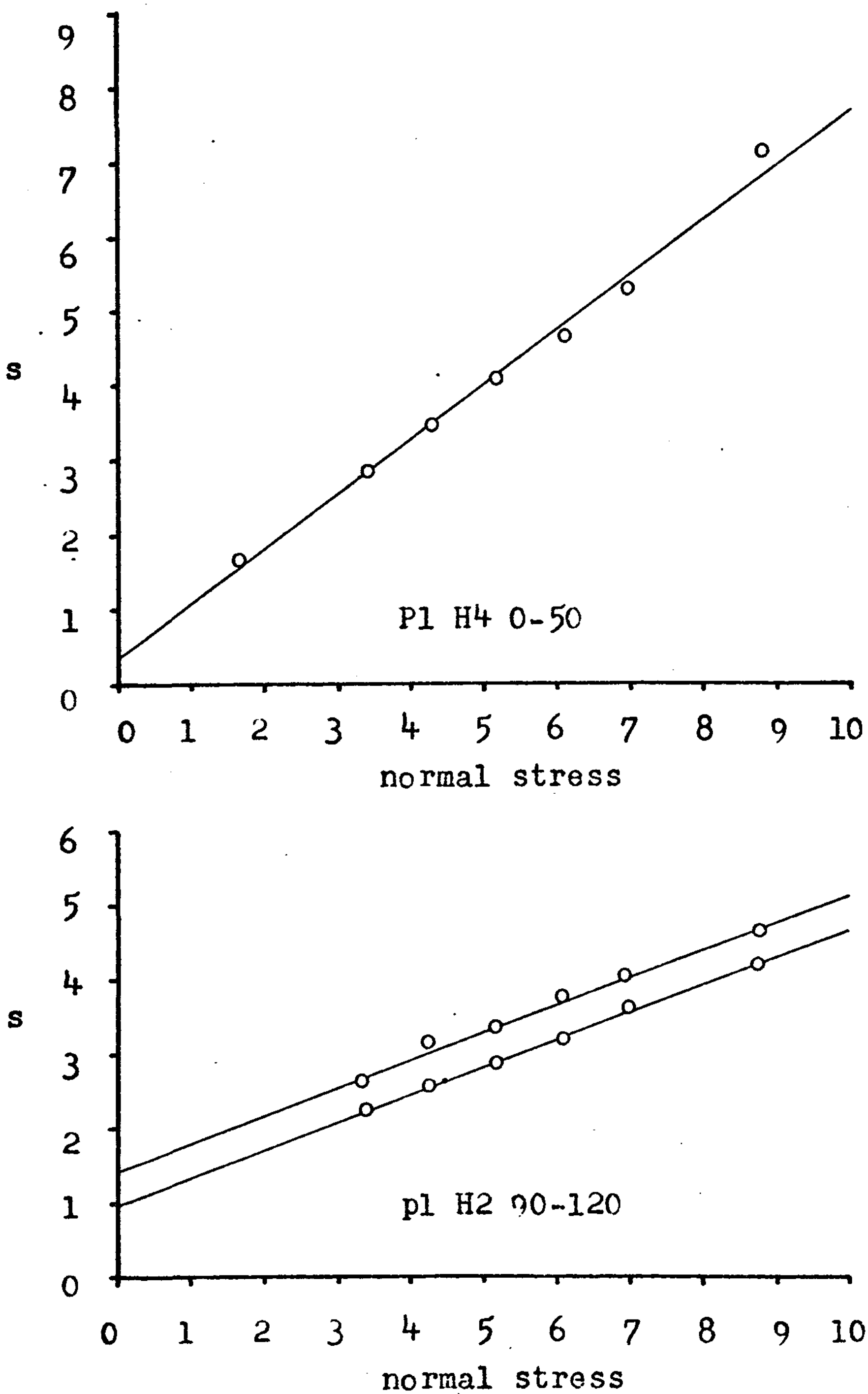


FIGURE 4.7

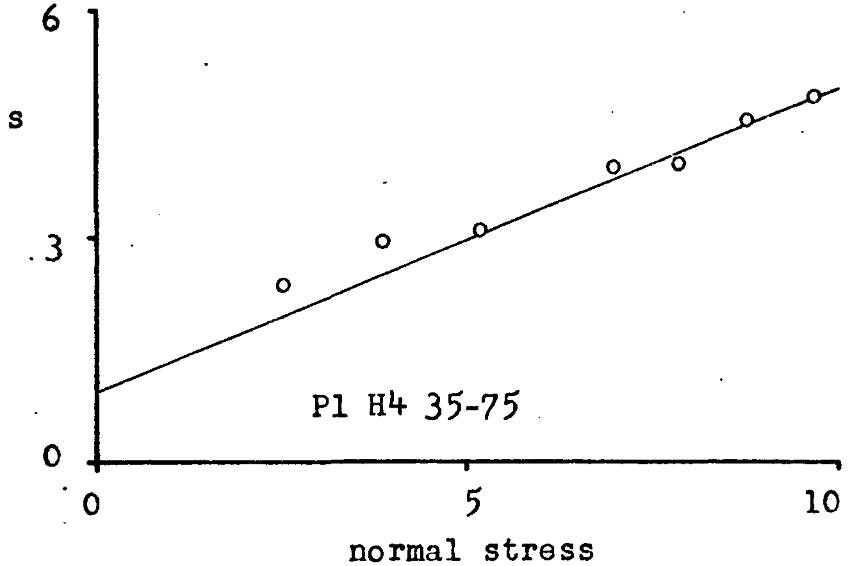
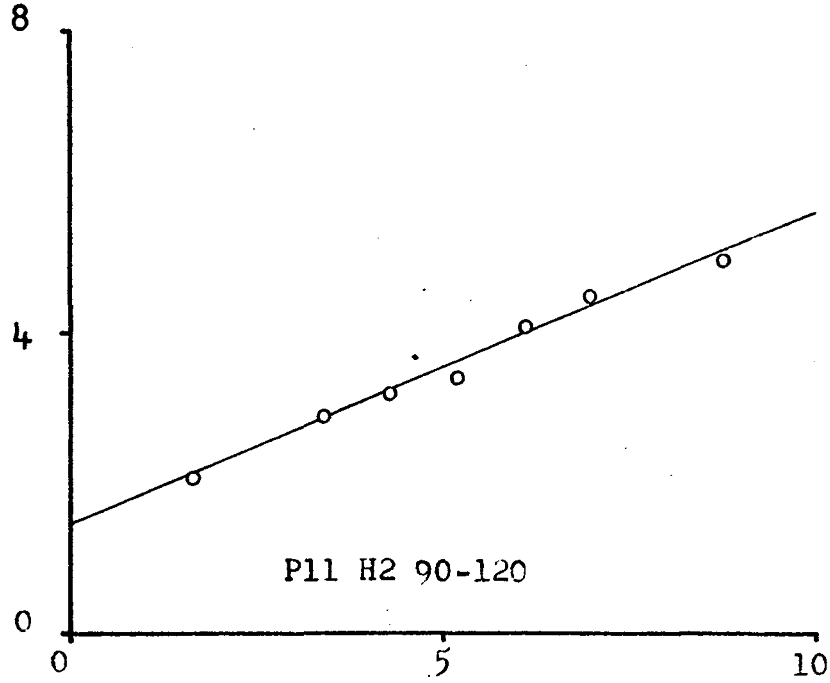


FIGURE 4.7 (cont.)



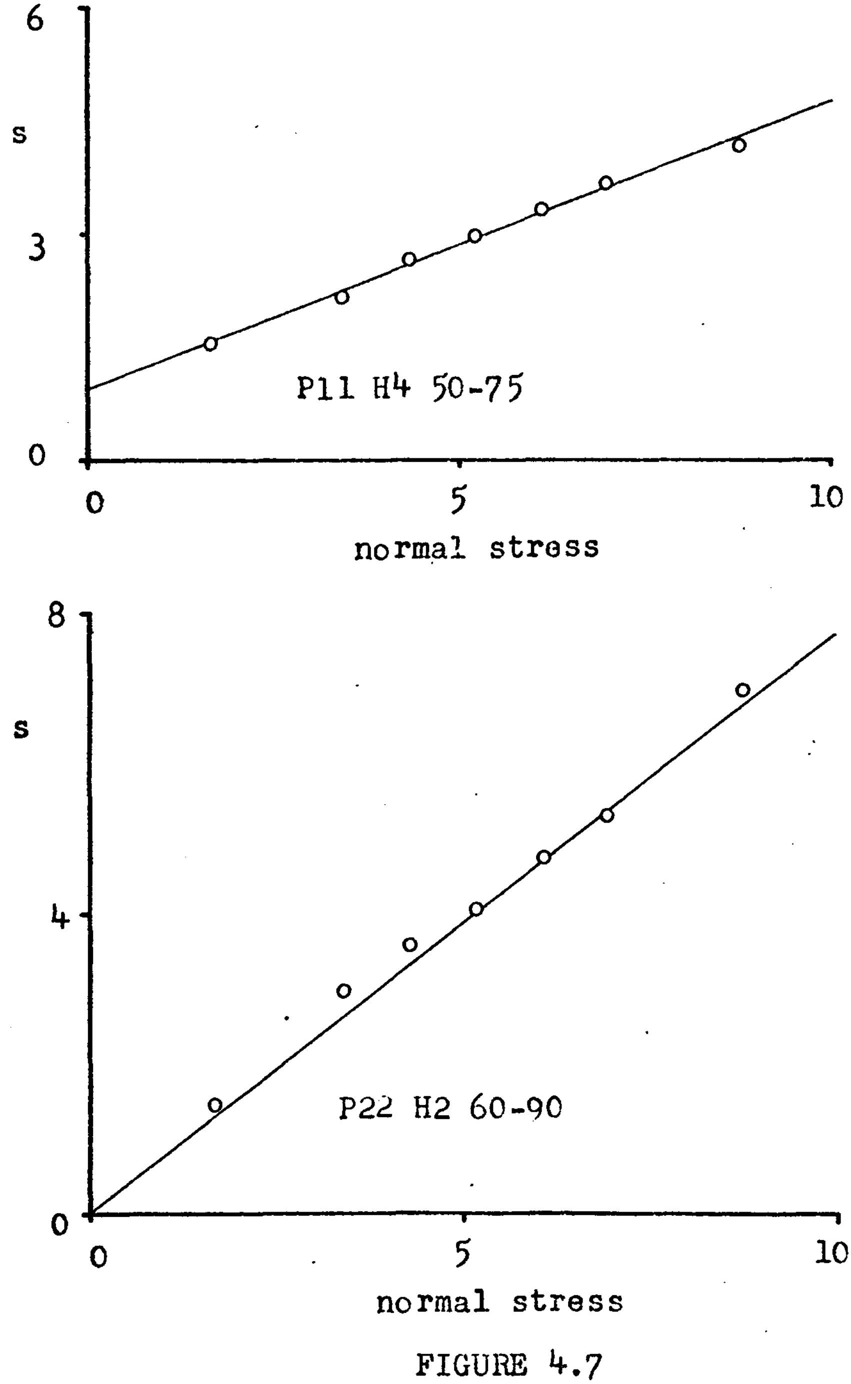


FIGURE 4.7
(cont.)

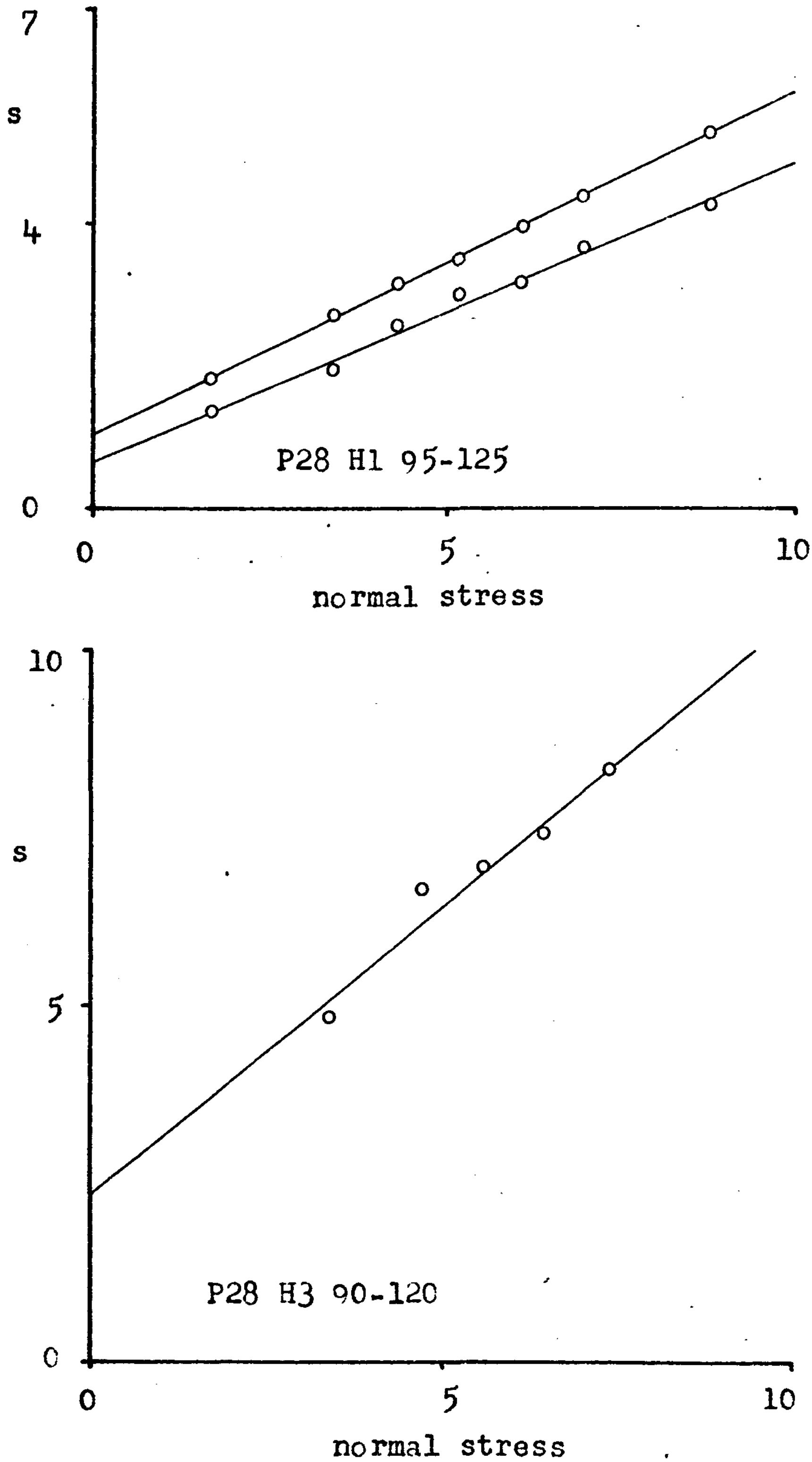

7
s
4
0
P28 H1 95-125
0
5
10
normal stress
10
s
5
0
P28 H3 90-120
0
5
10
normal stress

FIGURE 4.7

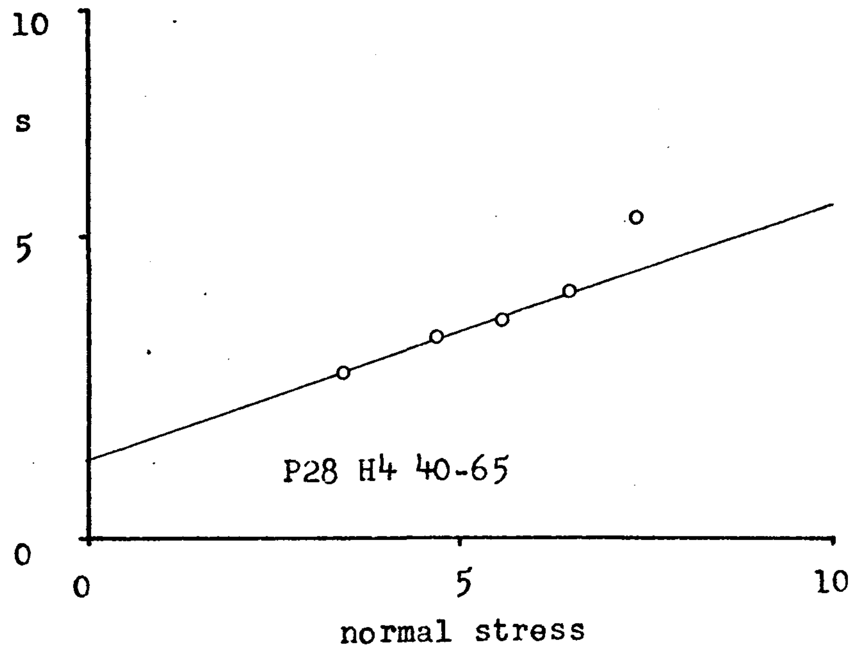


FIGURE 4.7
(cont.)

which was disrupted by a power failure during a week-end. Both results are therefore regarded as being unreliable, perhaps as a result of these interruptions to testing, which may have allowed the samples to consolidate for unusually long periods, hence altering the position of the shear plane with respect to the junction of the upper and lower halves of the shear box. The consistency of the test results suggests, however, that some more systematic error may have been involved.

Considering only the six acceptable tests on B-horizon material gives a mean value of ϕ'_r of 22 degrees, with a standard deviation of 1.9 degrees. Cohesion intercepts were found to be generally small but positive, (see Table 4.5).

Residual shear strength was found to generally increase at low stresses, as noted by Bishop et al. (1971). A typical relationship is shown in Figure 4.8. The significance of this fact will be considered in the next Chapter.

In some tests it was found that the upper and lower halves of the shear box came into contact during the test. When this occurred, the box was re-separated using

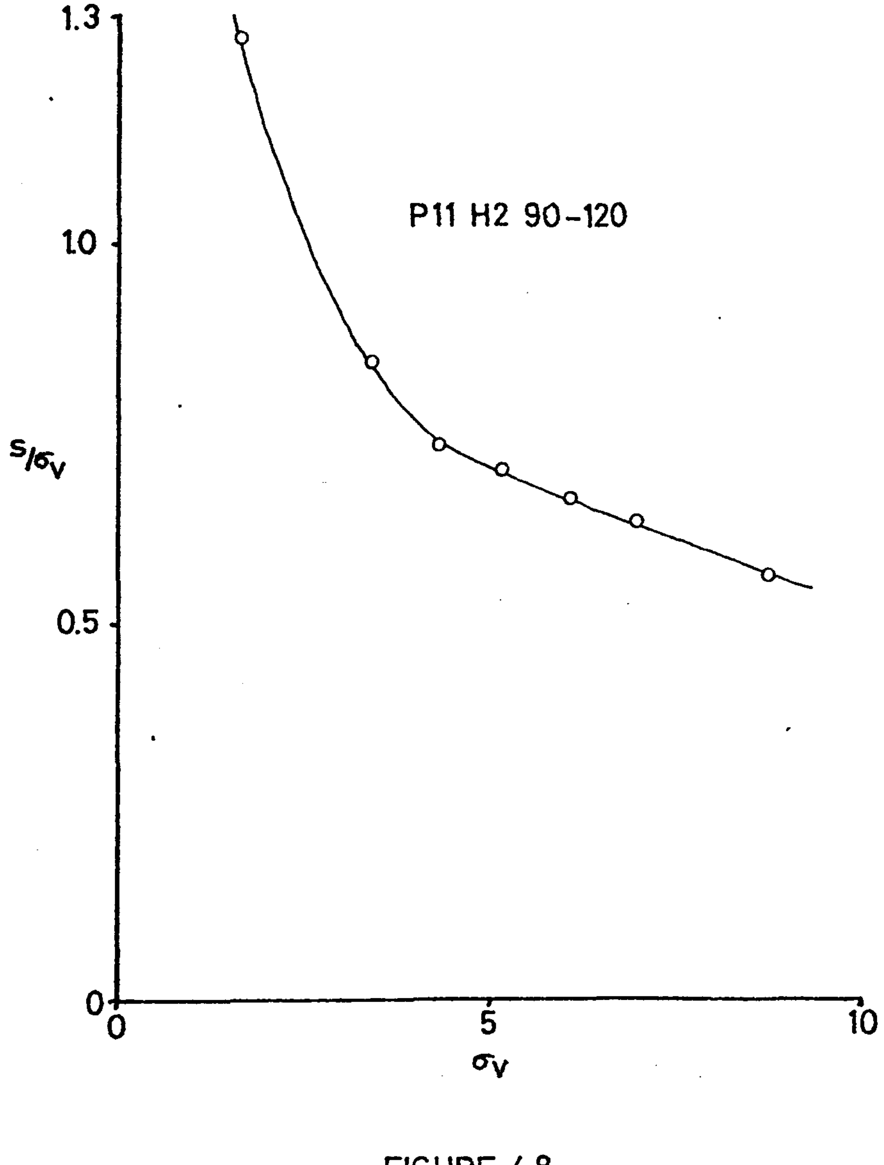


FIGURE 4.8
RELATION OF S_r TO NORMAL STRESS

the adjusting screws. The points at which this was caried out are marked in Figure 4.6.

Occasionally the shear strength of the soil was found to reach a constant value for about the first half of the test, and thereafter rise gradually. This behaviour was observed also by Hermann & Wolfskill (1966). The reason for its occurrence has not been investigated; La Gatta (1970) believes that this behaviour is peculiar to the reversing direct shear test, and may be due to some uninvestigated test error. In such tests, the constant strength value reached has been taken as the residual.

A few tests showed a gradual, but continuous, rise in strength throughout the tests, so that a constant residual value could not be defined. In such cases, the minimum and maximum strength values were recorded, and the average taken as the residual strength. Details of tests are included in Table 4.5.

No significant variation in soil strength properties along the slope profile was encountered, although the results available are too few to allow reliable conclusions to be reached.

CHAPTER V:

THE RELATIONSHIP OF HILLSLOPE FORM AND EVOLUTION TO THE PROPERTIES OF THE SLOPE MANTLE

5.1 ANALYSIS OF SLOPE STABILITY:

Before attempting to relate soil strength properties to slope form (Section 5.2) it is necessary to develop a stability analysis involving slope failure by landsliding. This analysis may then be used to allow for the effects of soil saturation and the movement of water within the soil, which may be expected to occur in actual storms within the study area from time to time (for example, as occurred in 1950. See Dept. Main Roads, 1952).

As was shown in Chapter 2, in the Razorback Range area the dominant slope process is landsliding, the landslides being essentially shallow planar movements which affect only the weathered mantle on the slopes, leaving the bedrock undisturbed. A suitable stability analysis for such a system may be developed from elementary statics.

Such an analysis (see Appendix A) leads to the conclusion that if the slope mantle is entirely saturated, the maximum stable slope angle, β , is given by the relation

$$\tan\beta_{(max)} = \frac{(\gamma - \gamma_w)}{\gamma} , \tan \phi \quad ....(1)$$

where γ is the unit weight of the soil, and γ_w the unit weight of water.

In most situations $\frac{(\gamma - \gamma_w)}{\gamma}$ is approximately 0.5, so that the ultimate angle of stability is about $\frac{1}{2}\phi'$ (Carson, 1971).

5.2 RELATIONSHIP OF MEASURED SOIL STRENGTH PARAMETERS TO SLOPE INCLINATION:

As was shown in Section 4.5, the average value of ϕ'_r measured was 22 degrees. If it is assumed that over geologic time the slope mantles in the study area become saturated on occasions (however rare), then using the relationship of equation (1) above, the ultimate angle of stability predicted on the basis of these laboratory measurements is about 11 degrees. The degree of correspondence of this value with observed field conditions will be discussed on the basis of four separate comparisons:

firstly, with the inclinations at which slopes are actually observed to become stable against landsliding in the Razorback area; secondly, with the inclinations of slopes which have been free of stream undercutting for a considerable period of time; thirdly, with the inclination of certain segments of the measured slope profiles, and fourthly, with the frequency histogram constructed using all slope angle readings made in the study area.

5.2.1 Inclination of slopes stable against landsliding:

Hazell & Kennedy (1957) and Dyer (1966) noted in independent studies of landsliding in the Razorback Range that landslides only occurred on slopes steeper than 10-12 degrees; identical conlcusions are reached in unpublished reports of the Department of Main Roads. The same values are given by Matson (1970) and Corbett (1972), whose sources of information (not acknowledged) are presumed to be the two papers cited above.

These values agree precisely with those predicted on the basis of the strength measurements made in the present study, which as shown above suggested that landsliding should only occur on slopes steep than 11 degrees. This agreement is evidence that the predicted ultimate angle of stability is in fact reflected in the landscape of the Razorback area. The correspondence between predicted and observed inclination limits for landsliding is precise, and even by itself may be regarded as unlikely to be a chance correlation.

None of the studies referred to above suggests a reason for the existence of a lower inclination limit for slopes affected by landsliding, nor has it ever been regarded as curious that such a limit exists. However, the findings of the present study show that this is explicable in terms of the strength properties of the clay soil resulting from the weathering of the Wianamatta shales, and of the worst groundwater conditions which can occur, namely, complete saturation of the soil profile.

Dyer (1966) suggested that landslides in the Razorback Range area only occur in the Camden Sub-group sediments (see Chapter 2) and that the much larger area of Liverpool Sub-group rocks is free of landslides. This conclusion has been quoted as fact by Quick (1968) and others. However, reconnaissance by the present writer has shown that this suggestion is incorrect, and that the earthflows typical of the Razorback Range occur throughout the entire area of outcrop of the Wianamatta Group rocks; they have been observed as far south as Moss Vale. Therefore the tentative suggestion is made that the value of 11 degrees suggested as the lower limit for landslide occurrence may have significance over this entire area, although particularly to the Razorback Range, and that it should be borne in mind in all large construction works that any slope steeper than this value is potentially unstable. Thus, for example, the suitability of land in the Razorback area for urban development (discussed by Matson, 1970), may need to be revised in view of the present finding; also, unless subsoil drainage can be installed, the Hume Highway across the Razorback Range remains susceptible to major collapse along much of its length, given another storm of the magnitude of that which occurred in 1950.

5.2.2 Inclination of slopes free from undercutting:

As can be seen from the geological map of the study area which has been prepared (Figure 2.4), a large deposit of allunium-colluvium mantles the valley floor between the northern and southern drainage divides near the eastern end of the study area, where the stream flows in a narrow and deeply incised gully through this material. The precise

Plate 5.1 Fill material in a tributary valley in the study area.

age of this deposit is not known; however, radiocarbon dating of contained charcoal fragments has suggested that it may have been present for at least several thousand years. (R.J. Blong, pers. comm.)

Therefore it may be hypothesised that during this period of isolation, the slopes which face onto this deposit would have been subject to repeated landsliding, so that their inclination would have been gradually reduced (in a manner to be discussed subsequently) towards the angle of stability against landsliding, at which time the relatively rapid change in slope angle would have ceased. There is no evidence to suggest that the period of time for which these slopes have been isolated is sufficient for this stage to have been reached; however, the inclinations of these slopes should show, if the stability analysis is correct, some tendency to approach an angle of about 11 degrees.

Four of the surveyed slope profiles were located on slopes which faced onto this deposit, and details of their inclination are summarised in Table 5.1.

Table 5.1

Inclination of isolated slopes in the study area

Profile Number	Mean slope angle
P7	10.7°
P9	13.0°
P11	13.5°
P47	11.4°
	mean: 12.1°

As can be seen from these data, the slopes do show a tendency to stand at about 11 degrees, the mean angle of the four being 12.1 degrees. Given the uncertainty as to the period for which these slopes have been isolated, this represents a good degree of correspondence with the predicted slope behaviour. A larger sample of slopes from other areas would be required to check this relationship.

5.2.3 Inclination of basal slope segments

As has been noted in Chapter 3, many slopes in the study area display a distinct basal rectilinear segment. On the ground the basal segments of some slopes may be seen to be formed from material deposited by old earthflows, as shown by the presence of subdued depositional lobes and undulations in the ground surface; in other cases, this evidence is only visible on air photographs, which reveal features too subdued to be noticed on the ground.

If indeed these basal slope segments represent material moved and deposited by landsliding during the evolution of the slopes, then again, according to the strength tests, they should stand at about 11 degrees, since having moved a considerable distance, the strength of the disturbed soil would have fallen to the residual value; in addition, the slipped material accumulating at the foot of a slope would be saturated relatively frequently. The relative moistness of these basal segments was noted in Chapter 2, where it was pointed out that they are characterised by the growth of the moisture-loving *melaleuca* spp. In addition, Willoughby (1967), using the microscopic techniques of Lafeber (1963, 1964), has shown that a preferred orientation of both soil pores and contained stone fragments (clay particles were not considered) is produced by landsliding, and this lends support to the idea that fabric changes, associated with the development of residual strength, do indeed occur in material moved by landsliding.

As can be seen from the data in Table 3.1, the basal slope segments do indeed stand at almost precisely 11 degrees, the mean of the 24 surveyed segments being 11.45 degrees. This is again a remarkable agreement with the predicted value.

It was also noted in Chapter 3 that whilst the inclination of the mid-slope and crestal rectilinear segments varied very widely, that of the basal segments varied only within a narrow range, the inclinations of the 24 segments having a standard deviation of only 2.3 degrees. An explanation of this markedly low variance is now apparent. It is a consequence of the fact that the inclination of these segments is controlled by the properties of the uniform clay soil which weathers from the Wianamatta shales, and not by variable factors such as gradient of the stream at the base of the slope, rate of stream downcutting, volume of runoff, vegetation cover, and so on, which are normally considered the determinants of slope angle (see for example, Strahler, 1950).

5.2.4 Slope angle frequency distribution:

As was shown in Chapter 3, a histogram constructed on the basis of 4,500 slope angle readings made within the study area showed a modal angle of about 12 degrees, the precise angle being unknown because of the 2-degree class width employed. The distribution was essentially symmetrical, with a slight secondary mode at about 22-24 degrees.

The mode at about 12 degrees is a very marked peak in the essentially symmetrical distribution; if any angle is "characteristic" of the study area (Young, 1961) then it is approximately 12 degrees. This matter will be discussed subsequently; it is merely presented here as evidence for the validity of the stability analysis employed, which suggested that all ground slopes should be adjusted by landsliding to an angle of approximately 11 degrees. The fact that a greater number of angle readings of about 12 degrees were made than of any other angle, and that the frequency distribution is symmetrically distributed about this value, may be taken as further evidence of the validity of this analysis. The "characteristic angle" revealed by the frequency distribution is thus seen to be a reflection of a known aspect of slope behaviour in the study area.

5.3 THE MODE OF SLOPE EVOLUTION IN THE STUDY AREA

On the basis of the four comparisons of predicted and observed slope behaviour discussed in the previous section, it may be concluded that the stability analysis applied to the slopes in the study area is valid, since it is confirmed by several independent comparisons with field observations.

The scheme of landslide behaviour developed from these ideas, and on which the present discussion of slope evolution depends, may be summarised as follows:

Landslides may occur on slopes of any inclination greater than about 11 degrees; they cannot occur on slopes of lesser inclination, since even soils whose strength is at its lowest, or residual, value, and which are entirely saturated, are stable at this angle, and therefore will not undergo shear failure. Any soil material which has moved downslope by landsliding will accumulate to form a deposit whose inclination is about 11 degrees, since it is at this angle that the moving mass becomes stable and ceases to flow. These ideas are confirmed by the field evidence, discussed above.

On the basis of this knowledge and the data on hillslope form which have been collected, the mode of slope evolution in the study area may be understood, at least in broad outline. The scheme whichhas been inferred is as follows -

Stream incision in the uplifted area in the vicinity of the Razorback Range has tended to produce moderate to steep slopes (typically of 20-35 degrees), which because of the fairly dry climate have always had a thin forest vegetation and a continuous grassy ground cover. The Wianamatta shales have weathered to produce a uniform clay soil on these slopes. The intense storms from which much of the rainfall of the area is decived, together with the layered geological sequence (consisting of sandstone beds interbedded with shales) which tends to carry infiltrating water out into the slope mantle, have meant that at times the clay soils mantling the slopes have become saturated, and in this condition they are unstable on the steep slopes produced by stream incision, and mass movement results. The mass movement takes the form of earthflows, since the shallow slope mantle becomes fully saturated and therefore moves downslope as a mass, rather than rotational slumps or similar forms, which require a deeper slope mantle and are produced by a localised failure only.

All those slopes whose inclination is greater than about 11 degrees may be affected by the landslides; however, in any given storm, south-facing slopes and those with concave plan curvature are more likely to be affected (see Chapter 2).

The material which is involved in the landslides moves slowly downslope and comes to rest only when it has attained a slope at which it is stable against further movement (again, about 11 degrees) or has become too dry to continue moving. Since the largest part of the area affected by a landslide in the study area is occupied by the depositional zone (Chapter 2), which must lie at a lower angle than the original slope, the landslides cause a slight decrease in overall slope angle.

The part of the slope from which the soil moved is then subject to more intense weathering, and the depth to which the bedrock was weathered is increased, producing a fresh slope mantle. This is then subject to further mass-movement as is any deposited material which remains at an angle of greater than 11 degrees, and the cycle repeated until the entire slope is reduced to the angle at which it finally becomes completely stable against landsliding.

This angle is indicated long before the final stage is reached by the angle of the material which accumulates at the base of the slope, having been moved there by land-sliding.

This "accumulation zone" gradually increases in length, and the steeper slope produced by stream incision gradually declines in angle by repeated landsliding, until the entire slope stands at the same angle. When the whole slope has been reduced to about 11 degrees in this way, essentially only the slow processes of creep are presumed to continue to operate; since this slope angle corresponds to that governed by the residual strength, even continuous (or mass) creep cannot operate (see Ter-Stepanian 1963, 1965 and Yen, 1969), and only seasonal creep may occur. The effects of creep have not been investigated.

Since the dominant slope process of mass-movement occurs principally during exceptionally intense storms, hillslope development in this area is essentially a discontinuous process. This was also the conclusion reached in the study of mass-movement by Rapp (1960).

The scheme of slope evolution described above is illustrated diagramatically in Figure 5.1.

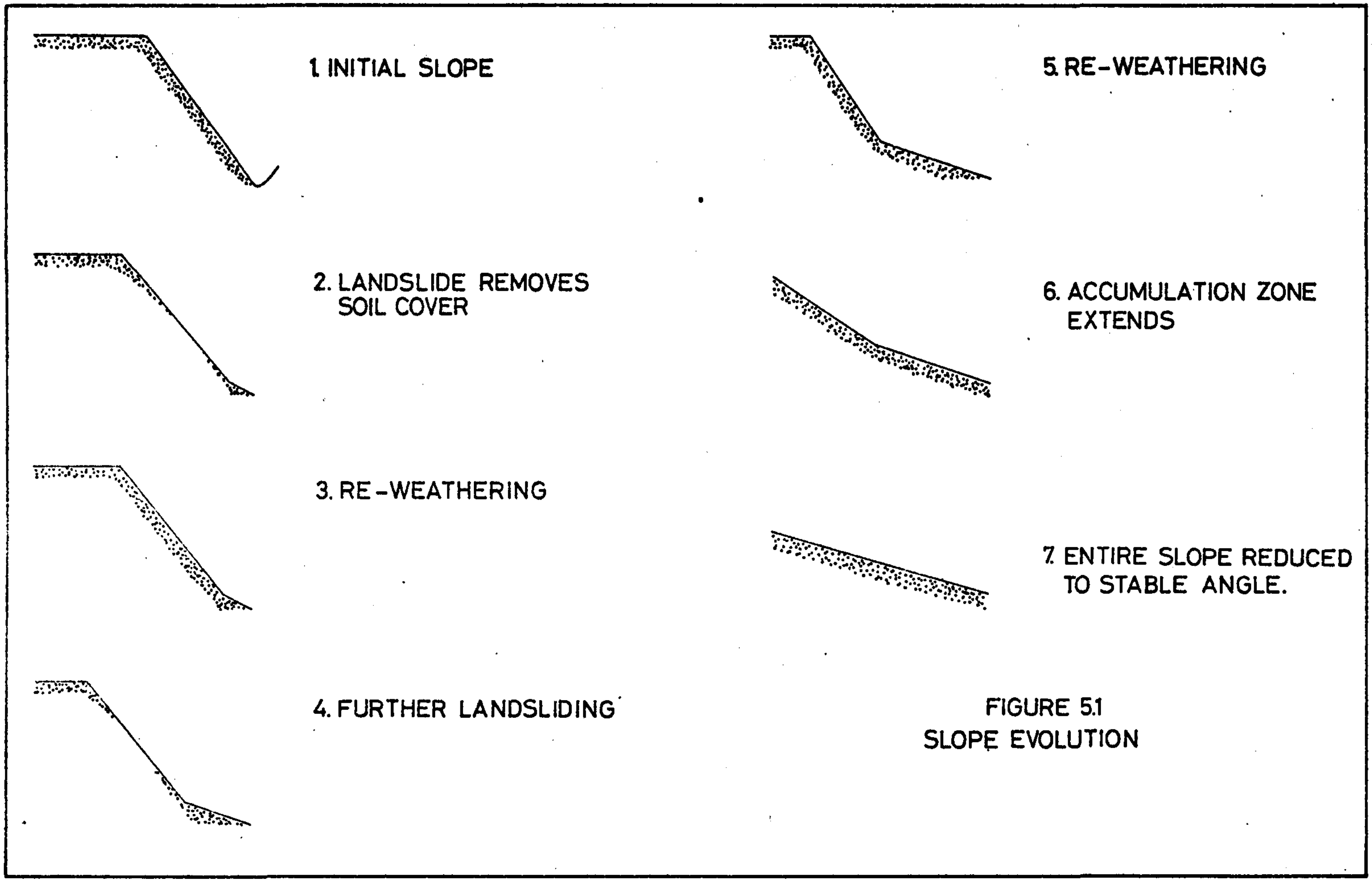


FIGURE 5.1
SLOPE EVOLUTION

A similar mechanism of slope evolution appears to be suggested by the data of Chandler (1970, 1971), who found that slopes on the escarpment in Northamptonshire could be divided into two sections. The upper section (14-20 degrees) was subject to rotational and trandational movements; the downhill extent of this section was represented by a break in slope below which the lower of the two sections formed an "apron" of more subdued relief, at an inclination of about 9 degrees. Chandler observed that the lower boundary of this lower section was formed by a series of interconnected lobes, very similar to those observed in the study area. As in the present study, the inclination of the lower slope sections was found by Chandler to correspond with that predicted using measured values of the residual angle of shearing assistance and the stability analysis given by Stempton & De Lory (1957).

Similar segmented slopes were also noted by Hutchinson (1967), in a study of the London clay. He found that abandoned coastal cliffs went through several stages of landsliding and concluded that:

> "The net result of the various types of mass movement described above is to effect a down-slope shift of weathered landslip debris which

under conditions of free degradation will gradually accumulate at the cliff foot. This accumulation protects the lower part of the cliff from further degradation and at the same time contributes to the overall reduction in inclination of the slope by advancing its foot. This principle was described a century ago, for Chalk cliffs, by Fisher (1866). At some time after the cessation of erosion at its foot, the slope profile will thus consist of two distinct components; an upper, degrading slope and a lower, less steeply inclined, accumulation slope." (Hutchinson, 1967, p.116).

Hutchinson also notes that:

"The accumulated material consists of heavily sheared and weathered slipped clay in which, by virtue of its situation at the slope foot, ground-water levels will tend to be close to the present ground surface. From an early stage in the free degradation process, therefore, the ultimate angle of stability against landsliding eventually to be reached by the whole slope, will be anticipated fairly closely in the average inclination of its accumulation zone." (Hutchinson, 1967, p.116).

It therefore appears that the scheme of slope evolution inferred from the field evidence in the present study agrees with the findings of other workers. The conclusion that slope inclination and evolution are controlled by the properties of the slope mantle is, however, a reversal of the normal relationship assumed to exist, whereby soil properties are usually seen as a function of slope inclination (for example, see Swan, 1970).[1] However, the possibility of such a control on slope development by the properties of the regolith was mentioned by Gilbert (1877, p.119) and later by Chorley (1959), whose preliminary comments are worth quoting here:

> "The importance of the physical properties of soils, in areas where soil-forming processes can keep pace with the erosive ones of sheetwash, creep, and mass movements, has been consistently ignored by geomorphologists, except on the most superficially qualitative level. Yet, in a soil-covered region it

1. Although Baulig (1940) regarded the regolith as the key element in the control of slope form and development, and Melton (1957) found a significant correlation between wet soil strength and valley side steepness.

is the stress necessary to shear off the surface soil particles which is more significant from the viewpoint of erosion than that necessary to fracture the associated bed-rock; and, except under the most intense or prolonged rain which results in the clogging of the whole depth of the soil column, it is the permeability of the soil layers ('infiltration capacity' of Horton, 1945) which controls surface run off - and therefore the intensity of superficial sheet erosion - rather than the permeability of the underlying rock. In these instances, bed-rock is not to be considered a parent of the related topography, but rather a grand-parent. It is quite possible for areas of differing bed-rock to present, under comparable climatic and vegetative conditions, landforms of similar geometry, simply because both rock types have weathered to produce soils having similar physical properties."
(Chorley , 1959, p.503).

As may be seen from Table 3.2 , the basal segment occupies more than 50% of the total length of some of the slope profiles surveyed in the study area. However, the slopes have clearly not reached the ultimate stage at which they will be essentially rectilinear and standing at about 11 degrees; only the older slopes at the eatern end of the study area approach this situation (Table 5.1). The steeper mid-slope segment is still a very prominent feature of most of the slope profiles, so that the landscape of the study area must be regarded as only a temporary feature which will be altered relatively rapidly by landsliding until all the slopes are degraded to a constant angle. In other words, the landscape does not yet fully reflect the operation of the processes which are forming it; it has been evolving for an insufficient period of time to become fully adjusted.

5.4 THE RELEVANCE OF RESIDUAL STRENGTH TO SLOPE STABILITY

A consideration overlooked in most studies which have attempted to relate hillslope evolution to the residual strength of the soil mantle (for example, Carson & Petley, 1970; Carson, 1971, and Hutchinson, 1967) is the explanation of how a slope which is not entirely composed of landslide debris (that is, which is not an "accumulation slope") can be lowered by landsliding to an angle determined by the residual strength, rather than the higher peak strength.

Hand augering in the study area has shown that those slopes which have almost reached the ultimate angle of stability (see Table 5.1) are not entirely composed of weathered debris; rather, they have a normal soil cover overlying bedrock at the usual depth.

The problem presented by these slopes in this : to have been reduced to the angle at which they are stable against landsliding, even employing the lowest or residual strength, landslides must have occurred on them when they were just slightly steeper than this angle. However, at such low angles, the peak strength of these soils (which must be greater than the residual strength) should have been sufficient to prevent failure. However, it clearly was not, since the slopes have been reduced to an angle controlled by the residual strength. Therefore the conclusion which must be made is that the peak strength of the soils was in some way overcome so that the residual value became that which governed slope stability.

Skempton (1964) originally explained this behaviour in terms of the mechanism of progressive failure, and this has become the accepted explanation. Discussions of this subject may be found in the papers by Bishop (1971), Taylor (1948, pp.342-345), Haefeli (1951, 1966), Turnbull & Hvorslev (1967), Peck (1967), Rowe (1969), James (1971), Henkel (1967), and Suklje (1967).

Progressive failure may be defined as "a simultaneous, or quasi-simultaneous, decay in both the c' and ϕ' parameters preceeding actual failure" (James, 1971, p.344). Peck (1967) has suggested that the failure of all clays which have not failed previously is probably progressive.

Progressive failure is associated with non-uniform stress distributions,which are universally present in slopes (see Bishop, 1967). The mechanism may be explained as follows - in such a slope, if the soil pore-water pressure rises, the shear stresses along a potential failure surface will increase in a non-uniform way; locally this stress may reach the peak value, and a localised failure will result. This leads to over-stressing elesewhere, and the zone of failure spreads; as this takes place, the shearing resistance begins to drop from the peak to the residual state as movement takes place. When the failure has extended to the whole of the slip surface the shearing resistance will vary from near the peak value at the point failing last to the residual value at others. Hence the angle at which the mass of soil is stable will be at some value between that determined by the peak and residual strengths; if movement continues or is later re-initiated, only the residual value is of concern.

Several other mechanisms for progressive failure have also been suggested - for example, Henkel (1967) has explained this phenomenon in terms of groundwater movements; Suklje (1967) and Bjerrum (1967) have stressed the role of soil creep as a possible means of initiating progressive failure.

Some actual field evidence has been collected which shows the importance of creep in initiating landslides. Suklje & Vidmar (1961) concluded that the peak strength of material involved in a large failure in Yugoslavia was overcome by creep perhaps centuries before the landslide occurred. Further details and additional field evidence are given by Saito (1961, 1965), and will not be repeated here.

The conclusion which is important for the present study is that several mechanisms exist by which peak strength may be overcome so that landslides may occur on slopes of inclination fully as low as that determined by the residual strength, but of course, no lower. Those mechanisms involving soil creep (Suklje, 1967) and the effects of the movement of groundwater through layered shale and sandstone geological systems (Henkel, 1967) may be particularly appropriate in the study area.

5.5 RELATIONSHIP OF FINDINGS OF THE PRESENT STUDY TO PREVIOUS STUDIES OF SLOPE EVOLUTION IN THE RAZORBACK RANGE:

In view of the fact that slopes in the study area do not yet fully reflect the operation of the processes which control their evolution (noted in Section 5.3) it is well to review the findings of the other two studies which have been made of slope development in the Razorback Range area.

Both Page (1966) and Quick (1968) have suggested that slopes in this area already stand at angles which are controlled by current processes on the slopes. For example, Quick (1968) measured only maximum valley-side slope sections (without specifying how these were identified), and grouped the readings made on each lithologic type into frequency distributions. She observed that:

> "The tendency of the slope angles to group closely about a mean is an expression of the adjustment of these slope segments to contemporary conditions of energy and resistance. Valley-side slope segments on various lithologies in the Razorback and Donald's Ranges may be regarded, therefore, as tending to a condition of dynamic equilibrium." (Quick, 1968, p.31).

and again -

> "Valley-side slope segments on various lithologies in the Razorback and Donald's Ranges were found to be tending to a steady state condition, as indicated by the discrete, close and recognisable statistical groupings of the segment angles." (Quick, 1968, p.87).

These findings must be regarded as an uncritical repetition of the ideas of Strahler (1950). As has been shown above, they are completely erroneous, since the maximum slope segments (or mid-slope segments of the present study) in the landscape of the Razorback Range do not reflect any equilibrium condition; they are merely temporary features, occurring at a wide range of inclinations, in the process of degradation of the landscape. The only slopes which reflect the outcome of the contemporary slope processes are the basal segments and those slopes which are nearing the ultimate angle of stability. It could validly be suggested that these slopes are adjusted to contemporary conditions of "energy" (gravity) and "resistance" (shear strength of the soil); however, to attempt to relate other slope segments to these conditions is clearly incorrect, for they have not yet reached this state.[1]

1. It is worth noting that Griffith Taylor (1958, p.172), writing about the Razorback Range observed that:

   "Landslides are very abundant also, which would seem to suggest rapid erosion in rather recently developed slopes",

   and suggested that the Range may be a small horst block.

5.6 SIGNIFICANCE OF THE INFERRED MANNER OF SLOPE EVOLUTION:

Carson (1969) is critical of much of the work in slope geomorphology for its failure to develop reliable models linking process and form. As he points out,

> "It cannot be overemphasised that the first task in understanding slope forms, as in understanding any system, is the derivation of a process-response model linking form (response) to a set of external variables which determine the processes at work. The nature of a physical system at any instant in time may be fully appreciated with reference to the model."
> (Carson, 1969, p.78).

Carson outlined the procedure by which a process-response model could be developed. The stages in this process are:-

(1) the identification of a process or set of processes and a determination of the rate at which the processes operate;

(2) the discovery of the manner in which the process

operates to change the form of the system in a small period of time; and

(3) the extrapolation of the geometric change in this small period of time, allowing for any feedback, until the system reaches equilibrium.

To the writer's knowledge, no geomorphic study has achieved the construction of a soundly-based process-response model which considered each of these stages. Mathematical models have been used to analyse the third stage (for example, see Culling 1963, 1965), and deductive theories developed in an attempt to proceed directly from the first to the third stages (see Chapter 1). However, none of these approaches has established the validity of the resultant model.

In the present study, the dominant slope process has been identified (see Chapter 2), its effects on slope form have been inductively determined and compared with predictions based on laboratory measurements, and the changes in form produced extrapolated to suggest the nature of the landscape which will be produced in the Razorback Range area when this process eventually ceases to operate, similar information was gathered by Hutchinson (1967) whose work has unfortunately been neglected by geomorphologists. The work of Carson & Petley (1970) and Carson (1971), which was described in Chapter 1, attempted to achieve such a model. Apart from containing errors of procedure, this work failed to give any indication as to the manner in which mass-movement affected slope form (that is, the second stage in the development of a process-response model).

Therefore the present study, having led to the construction of a process-response model by which the development of the landscape of the study area may be understood, allows various hypotheses regarding slope formation and evolution to be compared with the actual situation which exists in the field, at least in this area. No claim is made that the findings of the present study relate to any area other than that studied; however, it is felt that the application of the method employed to other areas might allow similar process-response models to be developed.

5.6.1 Dynamic equilibrium

The hypothesis that slope processes tend towards the establishment of a dynamic equilibrium which is reflected in the geometry of the landforms produced goes back to Gilbert's report on the Henry Mountains (Gilbert, 1877).

The existence of "dynamic equilibrium" in the landscape has been extensively debated by geomorphologists. Hack (1960) has argued in favour of the idea; he suggests that "... within a single erosional system all elements of the topography are mutually adjusted so that they are downwasting at the same rate. The forms and processes are in a steady state of balance and may be considered as time independent." (Hack, 1960, p.85). Ollier (1968), Bretz (1962), and others have argued against the existence of equilibrium forms; discussion on the subject is given by Leopold (1970), Howard (1965), Schumm &Lichty (1965), Conacher (1967), Ahnert (1964), and Melton (1957).

Whatever the theoretical merits of the idea, it clearly does not apply to the development of slopes in the study area. It has been shown that these slopes are undergoing an essentially continuous change in form, and are progressing towards an ultimate slope of gentle inclination. At no stage during this development does there appear to be scope for all elements of the topography to be "downwasting at the same rate"; the essence of the scheme is of degradation in some parts of the area and accumulation in others. Therefore it is believed that slopes in the study area must be considered in terms of their stage of evolution towards the ultimate slope, and not in terms of any steady-state equilibrium form.

5.6.2 Decline or retreat?

The issue of whether slopes develop through time by slope decline or slope retreat at a constant inclination has been a recurrent theme in geomorphology. No attempt is made to review this controversy here; the reader is referred to Twidale (1960), Young (1971) and Tanner (1956). However, it is worth noting that the findings of the present study show that the overall pattern of slope development in the study area is one of slope decline, which, however, occurs largely by replacement of the steep slopes produced by stream incision by slopes at lower inclinations. The steeper slope is gradually consumed by the latter, and only declines slowly in inclination itself (see Figure 5.1).

5.6.3 Characteristic and limiting slope angles:

The landscape of the study area may usefully be described in terms of "characteristic" and "limiting" slope angles. Young (1961) defines characteristic angles as "...those which most frequently occur, either on all slopes, under particular conditions of rock type or climate, or in a local region." (Young, 1961, p. 126), and limiting slope angles as "...those that define the range within which particular types of ground surface occur, or particular denudational processes operate." (Young, 1961, p.127)

As has been shown, (see Figure 3.22), an angle of about 12 degrees is characteristic of the study area; this is also the limiting angle below which landsliding is no longer operative, and this fact immediately reveals the genetic connection between the two. Angles of about 12 degrees are abundant in the study area because this angle results from the deposition of soil material by landsliding, and because this is the angle to which slopes decline under the influence of these processes. Having been produced at about this angle, there is only a very slow alteration in the inclination of such slopes, because the dominant process of mass-movement <u>can no longer occur</u> on them. Only the processes of creep and slopewash, which are extremely slow in comparison to the operation of mass movement, can then be operative. These slopes therefore remain at approximately the same inclination.

CHAPTER VI: CONCLUSION

It has only been possible to carry out a very incomplete study of hillslope form and evolution in the area studied during the seven months available for the completion of this thesis. However, it is felt that in a general way the dominant slope process and the manner of slope evolution under its influence have been determined.

The principal conclusions reached may be summarised as follows:

(1) The landforms of the Razorback Range area have been produced largely by processes of mass movement.

(2) The occurrence of mass movement on any slope is governed by the residual strength of the soil which weathers from the underlying shales.

(3) All slopes in the area which have inclinations greater than about eleven degrees will continue to be affected by landslides until their inclination is reduced to this angle, but no gentler slopes will be affected.

(4) Most slopes in the area have only reached this ultimate angle in their lower sections, and considerably steeper slopes are therefore common.

(5) Hillslope evolution occurs by replacement of steep slopes by slopes of about eleven degrees.

(6) Whilst the slopes in the area are at an early stage in this developmental sequence, slopes of about eleven degrees are already the most common in the landscape.

Rectilinear slope profiles have been recorded in many parts of the world. The control of slope inclination by the strength properties of the regolith may well provide an explanation for this phenomenon.