THE STABILITY OF SLOPES IN CHALK

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A THESIS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY IN THE
FACULTY OF ENGINEERING OF THE UNIVERSITY OF SURREY.

JUNE, 1974
SUMMARY

A field and laboratory investigation of the slope stability of the Chalk is described. A review of previous work on rock slope stability and a working hypothesis based on the "state of the art" are presented. The engineering geology of the Chalk is reviewed.

Natural and artificial slopes in chalk have been investigated and typical slope angles and types of failure recognised. Movements of the walls of experimental trial pits have been recorded and the causes of these movements are discussed.

Two main types of slope instability have been recognised: slope degradation and major slope failures. Fractures present within the Chalk are largely responsible for controlling the major failures.

A regional analysis of the fracture pattern within the Chalk and its effect on slope stability are described. Measurements have been made of the spacing of fractures and geophysical methods used for in situ determinations. Different types of fracture surfaces have been recognised in the field and investigated in the laboratory. The effect of fracture surfaces on shear strength is discussed, and a simple sliding apparatus for testing fractures has been devised.

A model is suggested for the development of slopes in chalk and the effects of the interacting factors which determine stability are discussed. Criteria are given which allow a practical approach to the design of new slopes and the assessment of existing slopes. Some suggestions are made for further work.
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LIST OF SYMBOLS

A  Area
E  Young's Modulus of Elasticity
E_d  Dynamic Modulus of Elasticity
E_{d1}  Dynamic Modulus of Elasticity - first loading
E_{d2}  Dynamic Modulus of Elasticity - second loading
J_1  Shear fracture set
J_2  Shear fracture set
J_3  Tension fracture set
J_4  Tension fracture set
J_x  Shear fracture set
J_y  Shear fracture set
N  Value determined by standard penetration test
S  Shear strength
U  Pressure due to water along fracture
V  Pressure due to water-filled tension crack
V_1  Seismic velocity in layer 1
V_2  Seismic velocity in layer 2
V_3  Seismic velocity in layer 3
C  Cohesion of intact rock
C_i  Effective cohesion
C_j  Cohesion of joint or fracture
C_r  Cohesion at residual shear strength
C_{u'}  Cohesion in undrained conditions
C'  Cohesion with respect to effective stress
C_{r'}  Cohesion at residual shear strength with respect to effective stress
\( e \) Voids ratio
\( i_s \) Saturation moisture content
\( k \) Permeability
\( m \) Moisture content
\( n \) Porosity
\( q_{ult.} \) Ultimate bearing capacity
\( u \) Hydrostatic water pressure
\( \alpha \) Angle of inclination of fracture plane as measured from horizontal
\( \beta \) Angle of inclination of plane of failure as measured from horizontal
\( \chi_b \) Bulk density
\( \chi_o \) Dry density
\( \nu \) Poisson's ratio
\( \sigma_n \) Normal stress
\( \phi \) Angle of frictional shearing resistance
\( \phi_f \) Angle of frictional shearing resistance of fracture surface
\( \phi_i \) Angle of frictional shearing resistance due to irregularities
\( \phi_j \) Angle of frictional shearing resistance of joint surfaces
\( \phi_{peak} \) Angle of frictional shearing resistance at peak shear strength
\( \phi_r \) Angle of frictional shearing resistance at residual shear strength
\( \phi^r \) Angle of frictional shearing resistance at residual shear strength with respect to effective stress.
\( \phi_s \) Angle of statical frictional shearing resistance

\( \phi_u \) Angle of statical frictional shearing resistance in undrained conditions

\( \phi' \) Angle of frictional shearing resistance with respect to effective stress

\( \tau \) Shear stress
1. INTRODUCTION

Until recently the stability of rock slopes has received little attention from the engineer and the geologist, particularly when compared with that given to soil slopes. Although rock slopes are numerous around the coasts of Britain, comparatively few rock slopes have been excavated inland for engineering purposes. The increasing scale of excavation both in civil engineering and open-cast mining has led in the last decade or so to a number of deep excavations being made, for which stability has been essential. The increasing demands for safety generally have also meant that a more rigorous appraisal of such steep slopes is necessary.

Although some theoretical work on the stability of rock slopes was found to be in existence, at an early stage in the research programme the inadequacy of available data obtained from field and laboratory investigations was recognised as a frequent cause of the difficulties encountered in the analysis of stability. The research was therefore directed mainly to an investigation of rock slopes in the field and the laboratory rather than a theoretical study.

The purpose of the research was then initially to recognise the various factors which affect the stability of slopes in rock. If these various interacting factors contributing to slope development could be recognised and then their relative importance assessed on a regional scale, the prediction of stability for any given slope might become possible.

To recognise and assess these discrete factors affecting stability, the decision was made to limit the investigation to one
rock type. The Chalk was selected as it is relatively homogeneous, is of extensive occurrence, and is convenient for study in South-east England. As the work was concerned with the stability of rock slopes, failures involving sliding of the Chalk on underlying incompetent strata were excluded from consideration.

As very little previous published work was found to exist on the detailed analysis of slope stability in the field, the following were considered essential.

(a) The recognition of 'safe' angles of slope determined from natural chalk slopes.

(b) The recognition of the types of slope failure that are possible in chalk slopes.

(c) The determination of any movements of steep excavated faces, the cause of any such movements and their contribution to rock instability.

The importance of pre-existing planes of weakness was recognised early in the research programme. As a consequence the following also became necessary.

(d) An investigation of the extent to which a regional fracture pattern is recognisable in the Chalk.

(e) An investigation of the variation of fracture spacing in the Chalk and its determination.

(f) The recognition of different types of fracture surfaces and their assessment.

The objective of the research was therefore to investigate these
complex interacting factors and to provide a realistic assessment of them in the field, supported by laboratory work as necessary. If an understanding of the interaction of these factors and their control on the behaviour of chalk slopes could be achieved, then a more practical approach to the assessment of the stability of existing and proposed slopes would be possible.
2. REVIEW OF PREVIOUS RESEARCH WORK

2.1 Introduction

Few detailed published works were found to exist on the stability of slopes in chalk. Consequently, a survey was made of all literature on rock slope stability. This was found to be necessary to gain an understanding of the factors affecting stability and the mechanics actually involved in slope failures. Each failure is to some extent unique because of the varied nature of the Earth's crust, and reference has therefore been made to case histories of particular failures. Previous work tends to be of three main types: the geographical approach; the description and analysis of specific slope failures; and rock mechanics techniques. In addition, there is some specific literature that is concerned with chalk slopes and this is described separately. The bibliography on slope stability produced by Imperial College (Hoek, 1970a) has been found to provide a useful source of relevant published literature. In each of the following sections the published works are considered in chronological order.

2.2 The geographical approach

Much geographical literature exists that is concerned with the types of slopes that are developed under different environmental conditions. A recent survey of work on slopes has been made by Young (1972). In addition to describing the types of slopes in existence on various lithologies and under various climatic conditions, the geographical approach is often concerned with the development of slopes as a dynamic process. Young states that in humid climates the upper limiting angle for the presence of a
regolith, or the lower limit of a free face is 40-45°. For
continuous vegetation and soil cover with no rock outcrops the
upper limit is 30-36°. The lower limit for the occurrence of
well-developed terracettes is 32-35°.

Examination of geographical literature has not been found
to be particularly worthwhile. Most of this literature deals
with the morphology of slopes and their denudation chronology,
rather than assessments of their present stability. In
addition, there appears to be little attempt to examine the
processes operative at present both on and within particular
slopes.

Ward (1945) attempted to bring together the geographical-
geological approach and the engineering approach. He describes
possible causes of mass movements and recognises a number of
types of failure. These are: soil erosion, creep and solifluxion;
fragmental slides (dry, partially saturated, saturated or seepage
instability); rock falls and slides; detritus slides; and
rotational shear slips.

2.2 Actual slope failures

A number of specific case studies of rock slope failures
have been published, for example, Müller (1964), Price and Knill
(1967) and Arrowsmith (1971). A detailed examination of these
studies is not included as they are considerations of specific
slope failures in a variety of rock types. Each failure has,
to some extent, its own characteristics. Great variations, often exist between failures in different types of rock. Failures in the same rock type may also differ as a result of differences in the prevailing geological situations.

The published accounts of slope failures indicate the importance of thorough engineering geological investigations to provide reliable data with which to assess the stability of any particular slope. In many failures there may have been old landslips already present at the locations described, for example Broili (date unknown). In other cases groundwater has been of great importance, for example Sherrell (1971). Most failures have resulted from the interaction of several factors.

The specific case studies examined indicate that the following are the main factors which may cause instability in rock slopes:

(a) Strata dipping towards slope;
(b) Joints, faults, or other planes unfavourably inclined in relation to face of slope;
(c) Presence of planes or layers with low shear strength, e.g. clay layers;
(d) Existence of permeable competent strata overlying impermeable incompetent strata;
(e) Water seepage towards slope;
(f) High water table in slope giving high hydrostatic pressures;
(g) Weathering of slope face (especially freeze-thaw action in temperate climates);
(h) Undercutting of toe of slope;
(i) Presence of old landslips.
2.4 The rock mechanics approach

Although considerable published work existed on the theoretical behaviour of rock masses under load, until relatively recently few attempts were made to determine the in situ factors that affect stability. The present research is primarily a field and laboratory study of rock slopes. Thus, previous work dealing with theoretical aspects of rock masses or mechanical characteristics of intact rock and rock masses is not described, except where it specifically relates to rock slopes. The present understanding of rock slopes has resulted from:

(a) the recognition of the in situ factors controlling stability and the types of instability that may occur;
(b) the development of theoretical and graphical methods for the solution of rock slope problems; and
(c) the determination of field parameters that allow the use of simple mechanics principles for assessing stability.

As described in Section 2.2 Ward (1945) attempted to recognise the causes and types of instability in natural slopes. In the case of slopes in hard rock, Ward distinguished between frequent small-scale rock falls resulting from the effects of near surface weathering, and less frequent larger-scale rock falls resulting from undercutting of sea or river cliffs. Ward also recognised rock slides occurring along planes of weakness such as bedding planes, fault zones, clay layers, often further weakened by frost and/or water. These types of instability have been included in a comprehensive classification of mass movements on slopes proposed by Hutchinson (1968).
A theoretical study of rock slopes was undertaken by Trollope (1961 and 1968). He identified three main types of rock masses:

(a) uniform, unjointed rock;
(b) rock with randomly orientated fissures; and
(c) block jointed rock.

In the case of the latter two types he comments that determination of appropriate $c'$ and $\phi'$ values on a large scale in the field is of major importance, although laboratory determined values of $c$ and $\phi$ may be used in the case of uniform, unjointed rock. The present author has found the factors involved in rock slope stability to be too complex for such generalised theoretical solutions to be widely applied.

Terzaghi (1962) stressed the importance of joint planes in determining the stability of slopes on unweathered rock. He also described the modifications that may occur near the surface of steep slopes, such as the development of sheeting joints parallel to erosion surfaces and changes produced by weathering. Like Ward, he recognised rock falls and rock slides as being the two main types of failure. Terzaghi considered the normally applied equation for shearing resistance to be valid for potential failure planes in rock slopes:

$$ S = c_i + (p - u) \tan \phi $$  \hspace{1cm} (2.1)

where, $S$ = shearing resistance at a given point, $P$, of a potential surface of sliding in a porous and saturated material;

$c_i$ = its effective cohesion;

$p$ = unit pressure at $P$;

$u$ = hydrostatic pressure in water located next to point, $P$;

$\phi$ = the angle of shearing resistance of rock.
The effective cohesion, \( c_i \), depends on the continuity of the joints in a section across the rock mass. The portions of a section in intact rock are considered to be 'gaps'. This gives:

\[
\frac{c_i}{c} = \frac{A - A_g}{A}
\]

(2.2)

where, \( c \) = cohesion of intact rock,

\( A \) = total area of section through rock,

\( A_g \) = total area of gaps within the section.

As a consequence of this reduction in the value of cohesion, the further reduction caused by the development of superficial jointing near the slope surface, and the brittleness of jointed rock, Terzaghi considered the influence of cohesion on the stability of slopes in jointed rock to be relatively unimportant. Other workers, notably Patton (1966), have examined the apparent cohesion of failure surfaces, and this is discussed more fully in Chapter 9.

Terzaghi also concluded that as the angle of jointed rock might be more than 65°, the critical slope angle for hard, massive rocks with a random joint pattern and without seepage pressures is about 70°. Where the slope is on stratified sedimentary rock, the critical slope angle depends on the relative orientation of the bedding planes and the cross joints (Fig. 1). The cross joints are assumed to be perpendicular to the bedding planes. Where the bedding planes are horizontal the critical slope angle is vertical since no slide is possible. If the bedding planes dip into the slope at an angle \( \alpha \), then provided \( 90^\circ - \alpha \) is equal to or smaller than the angle of friction of the cross joints, no failure is possible.
Fig. 1.(a) Critical slope angle for strata with regular cross joints.

Fig. 1.(b) Critical slope angle for strata with staggered cross joints. (after Terzaghi, 1962).
If the cross joints are staggered then the critical slope is steeper. If the bedding planes dip towards the slope at an angle less than an angle of friction $\phi_f$ of $30^\circ$, the critical slope is $90^\circ$. Where $\alpha$ is more than $30^\circ$ the critical slope is equal to $\phi_f$.

In his paper Terzaghi also described the effect of variations in the water table on the stability of steep rock slopes. Joints tend to be more open near the slope and the quantity of water entering the joint system will be greater than elsewhere. The water table may therefore be locally higher. Water in joints exerts a pressure which Terzaghi termed cleft water pressure, that may lead to slope failure. During the winter, freezing of water in joints near the surface may prevent the natural flow of water from the slope, and this may cause a further rise in the water table giving increased cleft water pressures resulting in slope failure.

Terzaghi quotes statistics collected by the Norwegian Geotechnical Institute which indicated that rock falls and slides are most frequent in April when the snows melt, and in October when rainfall is heaviest. Most of the major slides occur in April when joints are still blocked with ice near the surface, and snowmelt is producing large quantities of water to enter the joint system causing a rise in the water table. After a period of severe weather during which the least stable slopes undergo failures, there may then be a number of years before the 'fresh' slopes and other slopes deteriorate sufficiently for more failures to occur. In his paper Terzaghi mentions that in some areas there is evidence for deep seated slides, particularly in the geological past. The paper by
Terzaghi has been found particularly useful in developing the author's working hypothesis described in Chapter 3.

A series of papers published by the American Highway Research Board in 1963 were concerned with rock slope stability. Those by Philbrick, Ritchie, and Rausch et al. are particularly relevant. The papers are primarily concerned with deep artificial cuts in rock. Philbrick described the factors requiring consideration in the design of cut slopes in rock. The geological factors requiring investigation include:

(a) the extent of various rock types present;
(b) the depth of weathering;
(c) the orientation of planes traversing the rock; and
(d) the water table and subsurface drainage.

Climatic conditions and the orientation of the cut are considered to be factors influencing the rate and type of weathering processes that affect the slope. Philbrick stresses the importance of fitting the slope to the materials present rather than vice versa.

The determination of the shape of rock segments composing the slope must be made. In some cases what would have been stable cut slopes are made unstable by excessive blasting which reduces the size of the rock segments. Philbrick also briefly describes the protection of cuts e.g., by rock bolting or surface coverings, the use of benches as rock catchers, and the drainage of surface and subsurface water. He then gives examples of the design of cuts in durable rock, non-durable rock, and combinations of the two.
The paper by Ritchie describes the problem of rock falls and the means of containing them. Various outcrops subject to rock falls were studied and a slow motion camera used to film the trajectories of the falling rocks. He differentiates between the ways in which rocks move down a slope. On very steep slopes of 0.25 : 1 (76°) rocks tend to fall. On less steep slopes of 0.5 : 1 (64°) they bounce, and on gentler slopes of 1 : 1 (45°) they roll. They may be contained by fences at the base of the slope. Ritchie also describes the problem of rocks rolling down steep talus slopes and he found that these could be effectively contained by a 2m high shock absorbing chain link fence.

Rausch et al. describe preliminary work in a research programme aimed at detecting instability before failure of a rock slope occurs, and the predetermination of precise, safe maximum slope angles with known factors of safety. They describe the problem of designing slopes for highways, and also open pit mines, where previous practice had been to use 'cut and try' methods. They concluded that open pit failures were due to the influence of rock structure and water seepage, and set out to examine the stress in and strength of the slope of a deep pit.

Some of the results of the detailed investigations at this deep pit were later discussed by Long et al. (1966). They define the main factors affecting stability as the stress geometry, rock structure, environmental conditions, and excavational procedures. The stresses acting in a slope depend on the regional stress distribution and the modification of that distribution produced
by the slope. Photoelastic model studies showed that gravity induced stress will normally be concentrated in the toe of the slope and that tensional stress exists in the floor near the toe (Fig. 2). In the models using a homogeneous isotropic material a theoretical shear plane exists at an angle of 55° from the horizontal, and this was found to be independent of the magnitude of the stresses. The latter were increased as the height of the pit wall increased. Berms did not increase stability. Concave slopes were more stable than convex slopes, since, in the former case, the slope is affected by horizontal tensional stress.

Study of open pit mines showed that failures were nearly always along planar or structural discontinuities in the rock. In natural rock slopes structural features were also found to be the main factor affecting stability. A system of recording and plotting strikes and dips of planes is described by Long et al. Like Terzaghi they emphasise the effects of weathering and seasonal changes in the level of the water table. They also mention as a possible cause of failure weakening of the toes of slopes by overbreaking during excavation.

Wittke (1967) described the shear strength of joints in a slope in granite and its effect on the design of anchors to stabilise the slope. The granite was traversed by two systems of plane parallel joints, and sliding was possible along the line of their intersection. For practical purposes Wittke adopts the standard Coulomb-Mohr equation for failure:

\[ \tau = C + \sigma_n \tan \phi \]  

(2.3)
Fig. 2. Levels of strain and shear plane developed in photelastic models of various slopes (after Long et al. 1966).
He then describes how the factor of safety against sliding parallel to the intersection of the joints may be calculated. Basic mechanics principles are used to determine the resisting and disturbing forces, although this case is complicated by the irregular shape of the sliding mass. Having calculated these forces the anchor force to resist motion can be estimated.

Morgenstern (1968) distinguished between deformation problems requiring calculation of displacements in rock, and failure problems concerned mainly with shearing resistance of rock. The latter category includes the stability of rock slopes. He described the solution of rock slope problems using limit equilibrium methods employing the Coulomb-Mohr failure criterion with respect to effective stress. The methods applied were essentially those adopted in soil mechanics separating the sliding mass into a set of slices or wedges. Morgenstern described the importance of structural discontinuities in the rock mass and the effect of variations in these on the shearing resistance of the mass. He concludes that the inclination of planes of weakness will primarily control the shape of the critical slip surface, but that detailed field observation in a variety of rock types will be required before the movement of rock masses will be fully understood.

Morgenstern distinguishes between peak and residual shear strength and suggests that the available resistance of a rock mass will lie between these limits. Progressive failure leading to a decrease in the shear strength may result from physical and chemical weathering and/or local stress concentrations. The effect
of water pressure in reducing the shearing resistance is also considered to be important, but the water pressure distribution may also be reliably found using in situ tests.

John (1968) describes the use of equal area nets and the friction cone concept for solving rock slope problems. Two types of sliding movement are considered as examples: wedge type movement on two planes, and block type movement on one plane only. Stability limits and factors of safety may be calculated by this graphical method. Although angles of friction determined from shear tests may be assigned to the failure planes, cohesion cannot be allowed for in this method. In his conclusions John stresses the importance of collaboration between geologist and engineer in most rock mechanics studies.

In discussion of John's paper, Panet (1969), Vigier (1969) and Goodman (1969) have described the application of the stereographical technique to different and more complex rock slope problems, and Vormeringer (1969) describes a two dimensional planar construction.

Londe et al (1969) considered that the stability of most rock slopes could be considered in terms of the stability of a tetrahedral volume of rock. The tetrahedron is defined by the intersection of three planes within the slope. The method of construction described involves all possible movements of the tetrahedron drawn on a sphere. Such a method is clearly more difficult and more involved in practice than employing the equal area net for the three dimensional representation.
Hoek (1970b) draws attention to the lack of readily available information for the solution of slope stability problems in open-cast mines. He comments that although there is much published information on slope stability, most is devoted to theoretical analysis which may usually be applied only by specialists. Hoek reviews the mechanical characteristics of rock and soil and the mechanics of slope failure. He particularly notes the influence of joint roughness and water pressures on shear strength. Hoek derives slope design charts for plane and circular failures and describes how these may be applied to specific slope problems. He also describes some of the more complex failures that may occur such as sliding of a wedge of rock. Hoek comments that although theoretical solutions based on vector analysis exist, such as that by Londe already described, the major problem is the determination of reliable field data rather than theoretical solution. Although with soils, the standard laboratory tests for the determination of strength characteristics are usually reliable, in the case of rock masses the shear strength depends on such factors as the strength of the intact material, and the frequency, orientation, inclination, roughness and strength of joints and bedding planes, and further work will be required before these aspects are fully understood. Hoek also suggests the use of analysis of actual slope failures in the field to determine the shear strength of the rock mass involved. The determination of groundwater conditions he also considers to be important.

In 1971 Hoek and Boyd emphasised the importance of structural
discontinuities in controlling rock slope stability. They describe three main types of failure:

(a) planar slides;
(b) block toppling; and
(c) wedge slides.

The occurrence of toppling or sliding is believed to depend on the geometry of the sliding blocks composing the slope and the inclination and friction of the plane along which failure occurs. A simple model to demonstrate toppling failure is described.

The factors affecting coastal stability are described in a paper by Muir Wood (1971). Four main groups of factors are recognised: (a) topography; (b) geology and hydrology; (c) historical; and (d) marine. He describes briefly the special problems that exist in the stability of coastal rock slopes. These include the possibility of high pressures in fissures caused by breaking waves. To stabilise coastal rock slopes knowledge of the fissuring is required. Protection of the toe is of prime importance. Muir Wood considers that the ground characteristics are usually insufficiently well-known to justify the use of refined analytical techniques, except as an academic exercise after a failure has occurred. Ground water may reduce stability by causing internal erosion and by reducing the effective normal stress across potential failure surfaces. Also described are some methods of stabilising unstable coastal slopes. In conclusion, Muir Wood comments "it is usually far more important to be able to recognise the significant factors affecting stability than it is to attempt to apply any very precise method of analysis".

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Hoek, Bray and Boyd (1975) deal specifically with methods for the solution of wedge failures. The methods employed are engineering graphics, spherical projections and analytical solutions. The combination of graphical and analytical techniques is also described. The authors stress the sensitivity of slopes to water pressures.

Londe (1973 a) dealt at greater length with water seepage in rock slopes. The two main effects are erosion of material due to the velocity of flow and a reduction in the stability caused by the pressure of water. He considers that three types of conditions may exist in which different seepage flows may occur. These three types of flow occur in porous rocks, rock with plane fissures, and rock with small channels. Methods of improving drainage in slopes by drainage and grouting are discussed with reference to actual examples. Londe (1973 b) also shows how the spherical three dimensional analysis described in his earlier paper (Londe, 1969) may be undertaken using plane projection methods and how these techniques may be applied to specific problems.

At the time of writing a review of the present 'state of the art' concerning the design of rock slopes and foundations has been presented by Hoek and Londe (1974). The report gives a useful summary of the techniques available and attempts to identify those areas where further research is required.

2.5 Previous work on slopes in chalk.

There is considerable geographical literature concerned with chalk areas. Most of this literature is purely descriptive or is concerned with the geological history leading to the development of
the present landscapes. Except in a few cases this literature has not been found particularly useful in defining in detail the factors that lead to stable or unstable conditions in slopes.

Clark (1965) has drawn attention to the importance of literature concerned with chalk landforms and the general lack of objective scientific data. Only those published geographical works that are considered especially relevant to the present research are therefore mentioned. There is only limited geological and engineering literature available. Although a number of papers deal with the large scale landslips of chalk involving the underlying Gault Clay and/or Upper Greensand, few papers specifically deal with the stability of chalk in steep slopes and the types of failure that may occur therein.

Fisher (1866) examined the disintegration of a chalk cliff and concluded that the recession of such a cliff due to weathering is in the form of a parabolic curve. McDakin (1895) described how in 1891 a house at the foot of the chalk escarpment near Danton Farm, outside Folkestone, was destroyed by an 'avalanche'. The cause of the avalanche was believed to be the sudden thawing of snow on the frozen chalk at the top of the Downs. The water released carried down loamy material on top of the chalk.

The possible relationship between surface topography and the hardness of the underlying chalk was examined in the South Downs by Bull (1936). He concludes that the Melbourn Rock, a harder band, does tend to form erosion levels although this is not so in every case. Sparks (1949) has attempted to show a much more detailed relationship between topography and the effects of geology and
lithology. Small differences in the permeability of the various horizons of chalk forming the South Downs might account for some variations in topography. The work of Bull and Sparks is also discussed in Chapter 5.

Much geographical literature has also been concerned with the origins of the dry valleys. These may be due to:

(a) lowering of the water table level associated with scarp retreat;

(b) a high water table at the end of the Pleistocene allowing spring sapping and stream flow; or

(c) freezing of saturated chalk to give an impermeable material able to support stream flow and normal erosion.

The origin of dry valleys is discussed by, for example, Sparks (1960).

Work by Ollier and Thomasson (1957) in the Chilterns indicates that many slopes of 3° or less consist of chalk covered by flinty clay or loam, and only in steeper slopes, for example 18°, does chalk occur at the surface. They also conclude that there is a dominant west facing asymmetry of the valleys which dates from the period of active valley formation. However, valley asymmetry is a subject of some debate (see for example Young, 1972).

Clark (1965) presents surveyed profiles of chalk slopes. The types of slopes were divided into three groups: scarp coombes; truncated coastal valleys; and inland dip slope valleys. Scarp coombes were typified by slope angles in the 21-25° range. Maximum slope angles were in the range 29-33°. Slopes steeper than 32-33°
were found to show increasing signs of instability. In the Devil's Kneadingtrough the upper slopes reach 35.5° but are unstable. Coastal valleys show concentrations at 10-11° and 6-8° and inland valleys show concentrations at 3-8°. Clark considers that the characteristic slope angles of 8-9°, 26-28° and 32° may represent particular stages in slope development. He concludes that there appears to be no relationship between slope and lithology and/or permeability. This might suggest origins for many of the landforms in glacial or periglacial times when lithological factors would be of minimal influence. There is some relationship between asymmetrical valleys and the structural geology.

Smart et al (1966) comment that near Sugarloaf Hill, Folkestone, the topography suggests the occurrence of landslips affecting the Chalk. Scares and bulging of the toes on the Gault Clay are cited as evidence. However, these landslips probably involve the Gault Clay and are related to the nearby rotational landslips of Folkestone Warren that is outside the scope of the present investigation. A description of the geology of the Folkestone Warren landslip is given by Smart et al (1966). Detailed investigations of the landslip are described and discussed by, for example, Toms (1946) and Hutchinson (1969).

Toms (1966) describes a number of slopes in chalk. At Saltdean a chalk face has been cut back at 75° to the horizontal and although without vegetation, it is apparently stable. Yet, in 1940, after exceptional rainfall and cold, a cliff inclined at 65-70° between Abbotscliff and Dover was subject to major cliff falls. On the slopes
of deep railway cuttings in the Middle and Upper Chalk in Surrey and Sussex vegetation will not remain permanently on slopes steeper than about 45°. On slopes steeper than this a period of continuous rainfall followed by freezing and then thawing results in 'bursting' of the surface and removal of the vegetation. This lends support to the hypothesis of Terzaghi described in Section 2.4.

Hummocky topography developed on the Chalk east of Newmarket has been described by Worssam and Taylor (1963). The topography consists of ridges and hollows that are oval in plan and 45-135 m. long. The crests of the ridges are as much as 2 - 2.5 m. above adjacent hollows. Trenches indicate that the ridges are of chalk and the hollows contain sand. The origin of the sand is uncertain. The hummocky ground is suggested to be due to solution of the chalk or periglacial action in the Pleistocene. The most steeply ridged areas occur where the Totternhoe Stone and the Melbourn Rock are at or near the surface. The topography is therefore related to the outcrop of horizons that allow free passage of ground water. Solution of the chalk would be concentrated in these horizons leading to subsidence of the ground surface. Alternatively, the susceptibility of chalk to frost action may be the main cause. In support of this are the following: (a) the occurrence of this topography in Cambridgeshire but not to the south, relating to the similar occurrence of fossil patterned ground; (b) its development at the level of the First Terrace suggesting a Weichselian age; and (c) its coincidence with the outcrops of water-yielding chalk that might be more susceptible to frost action.

Ward (1971) gives a brief description of work undertaken in
connection with the M40 motorway cutting through the Chilterns. A survey of excavations in the Chilterns revealed that none were deeper than about 18m., but that some with near vertical faces, up to 18m. high, had remained stable for many years. A major factor in the stability of these slopes is the limited amount of superficial frost action possible in confined quarry faces. On site boreholes revealed most discontinuities to be inclined at angles of more than 50° to the horizontal. Ward states that "A major problem was to obtain a realistic value of the effective stress parameters c' and $\phi'$ for these discontinuities". A value of $\phi' = 35^\circ$ was obtained from drained shear box tests on joint and fault surfaces. Analyses of slides were made using a value of $\phi' = 35^\circ$, to give values of $c' = 5-700$ KN/m$^2$ according to the weight of the slide.

Hutchinson (1972) shows a correlation between the incidence of chalk falls on the Kent coast and climatic factors viz. average monthly effective rainfall and average number of days of air frost per month. He notes, however, that the exact mechanisms by which these climatic factors affect stability still have to be established. They may include pore and fissure water pressures, ice action opening joints or preventing water flow, and the increased weight of a potentially unstable mass by an increase in its degree of saturation. Wave action is perhaps important in the case of sea cliffs. Hutchinson describes the investigation of a fall in cliffs at Hackemdown Point, Joss Bay, Thanet. Drained shear box tests on samples of the chalk gave a peak strength envelope defined by
$c' = 131 \text{ KN/m}^2$ and $\phi' = 42^\circ$. Tests on smooth planar surfaces to determine the drained residual shear strength gave $c'_r = 0$ and $\phi'_r = 30^\circ$. A wedge type stability analysis in terms of effective stress was undertaken by Hutchinson. The analyses were made assuming zero water pressure on the failure surface, since piezometers indicated only tidal fluctuations, although fissure water pressures may have existed prior to failure. An analysis was made, however, assuming a tension crack half-filled with water. The maximum average shear strength possible in the chalk examined was found to be 216 KN/m$^2$. This indicates that the maximum height of a vertical cliff in that chalk is 30.7 m.

Arber (1973) describes the landslips in West Dorset and South-east Devon involving strata from the Keuper Marl to the Chalk. A large landslide involving the Chalk affects the coastal area between Lyme Regis and Axmouth. The landslide has been attributed to the lower part of the Foxmould (Upper Greensand) being reduced to a quicksand after exceptionally heavy rainfall, followed by sliding on the underlying impermeable Lias strata. More recent evidence indicates that the slipped masses dip landwards and are therefore blocks formed as a result of rotational sliding along slip planes (Ward, 1945 and Arber, 1973). These landslips are therefore of a similar character to that at Folkestone Warren and are not specifically dealt with in this investigation.
3. THEORY OF ROCK SLOPE STABILITY

3.1 Introduction

The review of previous research work revealed that a number of possible factors which may affect the stability of rock slopes have been recognised. The relative importance given to these factors varies from author to author, and is also dependent on the nature of the actual slope under discussion. The present author therefore considered the adoption of a working hypothesis for rock slope stability to be necessary. This allowed closer definition of those factors requiring greater examination. The papers by Terzaghi (1962) and Hoek (1970b) have been found particularly useful in developing a working hypothesis suitable for this study which is primarily a field and laboratory investigation rather than a theoretical analysis.

3.2 Working hypothesis

If rock is without fractures and unweathered then it is able to form a vertical slope of considerable height. Terzaghi (1962) has shown that for even a comparatively weak rock (compressive strength = 34,500 kN/m², and unit weight = 2723 kg/m³) the critical height for a vertical slope would be 1267 m. Once exposed at the surface, however, the face of such a slope would begin to weather, pieces of rock will spall off giving rock falls, and the angle of the slope will begin to reduce (Fig. 3). If the face of the slope is undercut at the base by, for example, fluvial or marine action, then the unsupported rock may eventually fail by fracturing (Fig. 4).
Fig. 3 Effect of weathering on a vertical slope in rock without fractures.

Fig. 4 Effect of erosion on a vertical slope in rock without fractures.

Fig. 5 Effect of dip of fractures on stability.
In practice, however, all rocks near or at the surface of the Earth are fractured mainly as a result of: (a) stresses acting within the Earth; (b) stress relief as the rock mass becomes exposed at the surface following erosion; and (c) weathering. As rock has therefore already failed by development of these fracture planes the strength of the fractured rock en masse is considerably less than if it were unfractured. Slope failure may therefore take place by movement of masses of rock along these fracture planes. Hence, the orientation of these planes with respect to the slope is of major importance. If all the fracture planes are dipping into the slope then assuming no undercutting of the slope occurs, no major instability should develop, although the slope angle will reduce by weathering and minor rock fall (Fig. 5). Even slopes with failure planes dipping quite steeply towards the face of the slope may be stable in the short-term. Such slopes, however, must be susceptible to a progressive reduction in stability. Possible causes of this progressive reduction are changes in the level of the water table within the slope and weathering effects on the face of the slope.

The rock mass forming the slope will possess a primary fracture system that has resulted from stresses operative within the Earth. Stress relief and weathering will tend to develop much closer fracturing near the surface, and a secondary fracture system will therefore become superimposed on the primary system. The mass of rock immediately behind the face of the slope will
Fig. 6 (a). Development of secondary fracture system

Fig. 6 (b). Variations in water table levels
therefore have a much higher permeability than the remainder. After exceptionally heavy rainfall the water table level may locally rise in the vicinity of the slope where the higher permeability allows a greater concentration of ground water flow (Fig. 6). If, during the winter, water held within this zone of increased permeability freezes, the natural drainage of the ground water in the slope may be prevented. The result may be a build-up of water within the slope which may exert additional stresses on the fracture planes sufficient to overcome their shear strength and cause failure.

The characteristics of the fracture surfaces may also be changed by ground water flowing along the fractures or by weathering processes. Ground water may remove fracture infillings or lubricate fractures infilled with, for example, clay. The result may be a reduction in the shear strength of the fracture with consequent failure of the slope.

The control on stability exerted by fractures therefore depends on their orientation with respect to the slope and the nature of the surfaces. Failure may occur by movement along a single fracture (planar failure) or by movement along two intersecting fractures (wedge failure). Toppling failures may occur where steeply dipping fractures are present.

3.3 Basic mechanics of failure

The Coulomb-Mohr failure envelope is frequently used for the definition of the shear strength characteristics of rock. The
failure envelope is defined by the equation:

\[ \tau = c + \sigma_n \tan \phi \]  

(3.1)

where \( \tau \) = shearing resistance along failure surface,

\( c \) = cohesion,

\( \phi \) = angle of friction, and

\( \sigma_n \) = total normal stress acting on failure surface.

Rock subjected to a shear stress will deform in an approximately elastic manner until a peak shear stress is attained. Sliding then occurs and is associated with a reduction in the shear stress. The peak strength represents the point of failure of the intact rock, and once failed the shear strength reduces until a relatively constant value is reached. This is the residual strength and represents the shear strength of the plane of failure (Fig. 7). For hard rocks such as basalt the difference between peak and residual strengths may be very great, whereas for soft rocks such as shale the difference may be slight.

If the fractures within the slope are discontinuous or are interlocking, fracture of the rock will be necessary before movement is possible and the shear strength of such fractures will be considerably greater than in the case of smooth continuous fractures. The roughness of the fractures is also of importance. The angle of friction of the fracture, often denoted by \( \phi_f \), may need to be increased by an amount \( \phi_i \) corresponding to the roughness of the fracture (Fig. 8).
Fig. 7. Peak and residual strength

Fig. 8. Strength of fractures in rock

Fig. 9. Basic mechanics of simple planar sliding failure.

Fig. 10. Effect of water pressures on mechanics of planar sliding.
\[ \tau = c_j + \sigma_n \tan (\phi_j + \phi_1) \]  \hspace{1cm} (3.2)

where, \( \tau \) = shearing resistance along failure surface,
\( c_j \) = cohesion of joint or fracture forming failure surface,
\( \phi_j \) = angle of friction of joint or fracture forming failure surface,
\( \phi_1 \) = additional component to angle of friction produced by irregularities on the failure surface,
\( \sigma_n \) = total normal stress acting on failure surface.

Water along a fracture will bend to exert a pressure, denoted by \( u \), that opposes the normal stress, \( \sigma_n \), acting across the failure plane. The shear strength hence becomes:

\[ \tau = c_j + (\sigma_n - u) \tan (\phi_j + \phi_1) \] \hspace{1cm} (3.3)

Rock slope problems may often be solved by applying basic principles of mechanics. Like some other authors, for example Muir Wood (1971), the present author considers that frequently solutions involving basic mechanics are preferable to more sophisticated methods of solution since the data available is usually insufficiently accurate to warrant those more precise methods.

The mechanics involved in a simple planar sliding failure are shown in Fig. 9. Limiting equilibrium occurs when:

\[ W \sin \beta = c_j A + W \cos \beta \tan (\phi_j + \phi_1) \] \hspace{1cm} (3.4)

where, \( W \sin \beta \) = disturbing force acting down the plane,
\( c_j \) = cohesion of fracture plane,
\( A \) = base area of sliding mass,
\( W \cos \beta \) = normal force acting across the plane, and
\( (\phi_j + \phi_1) \) = angle of friction of fracture plane.
The factor of safety for such a condition is:

\[ F = \frac{c_J A + W \cos \beta \tan (\phi_J + \phi_s)}{W \sin \beta} \quad (3.5) \]

Additional forces may result from water pressures (Fig. 10). As a result of these forces the equation for limiting equilibrium becomes:

\[ W \sin \beta + V = c_J A + (W \cos \beta - U) \tan (\phi_J + \phi_s) \quad (3.6) \]

The accurate application of the above equations depends on the determination of realistic values for the shear strength parameters \( c \) and \( \phi \). As in most cases the shear strength involved is that of the fractures rather than the intact rock, the residual shear strength parameters are those usually required. In these cases \( c_r \) is normally zero and only \( \phi_r \) requires determination. The density of the rock involved may be obtained by laboratory testing. The volume of the rock mass involved may be determined if the orientations of the potential failure planes have been measured in the field.
4. ENGINEERING GEOLOGY OF THE CHALK

4.1 Introduction

Despite the extensive occurrence of chalk in England there is little published information giving complete accounts and data of the engineering geology of the Chalk. For this reason a reasonably detailed account is given here. Published data for the Chalk has been collected and is summarised in Table I. The present author has also undertaken a number of tests on samples of chalk and the results of these are given in later chapters. The most detailed accounts and discussions appear in the Proceedings of the Symposium 'Chalk in Earthworks and Foundations' (1966). In these Proceedings, Higginbottom described the engineering geology of chalk, Wakeling described the characteristics of chalk as they affect foundations, Lewis and Croney as they affect road construction, and Toms as they affect earthworks.

4.2 Distribution

Chalk outcrops extensively over the South and East of England and there are also smaller outcrops in Scotland and Northern Ireland. In England the outcrop (including that beneath superficial deposits) covers about 15% of the surface area (Higginbottom, 1966). In East Anglia the Chalk is extensively covered (about 75% of the outcrop) by glacial deposits, particularly boulder clay. In the South of England the Chalk may gradually dip beneath more extensive coverages of Reading Beds or Thanet Sands, and, in places, it is overlain by thin deposits of Clay with Flints.

In southern England the Chalk has been affected by earth
Table I (a). Published data on the Lower Chalk

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$^{a\#}$ S. varianz zone (Chalk Marl). $^{a\#\#}$ H. subglobus zone (upper white part). $^{a\#\#\#}$ H. subglobus zone (Gray Chalk). $^{a\#\#\#\#}$ S. varianz zone (Chalk Marl). $^{a\#\#\#\#\#}$ S. varianz zone (Glaucotic Marl). $^{a\#\#\#\#\#\#}$ S. varianz zone. $^{x^7}$ Determined by plate loading tests. $^{x^8}$ Extreme values. $n$. Normal values.
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<th>is%</th>
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x^1 T. lata zone.  x^2 Twin Marl, T. lata zone.  x^3 probably T. lata zone.  x^4 Melbourne Rock.
Table I(c). Published data on the Upper Chalk

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x M. Coreguminum zone.
Table I(c) (continued) Published data on the Upper Chalk

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<td>TOMS 1966</td>
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*1 B. mucronata zone.
Table I(c). Published data on the Upper Chalk

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<th>𝜉_D</th>
<th>m%</th>
<th>Cu</th>
<th>𝐶'</th>
<th>𝜙'</th>
<th>E_d</th>
<th>Q_u</th>
<th>N-Value</th>
<th>Depth metres</th>
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</table>

Note: Ed determined by plate bearing-tests. (H) refers to horizontal tests; all others are vertical.
Table I(d). Published data on the Chalk (undifferentiated)

<table>
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<th>REF. NO.</th>
<th>LOCALITY</th>
<th>$X_D$ Kg/m$^3$</th>
<th>$n$ %</th>
<th>$e$</th>
<th>$i_o$ %</th>
<th>$k$ m/s</th>
<th>$C_u$ Kg/m$^2$</th>
<th>$C'_u$ Kg/m$^2$</th>
<th>$g'$</th>
<th>$m_{v}$ m$^2$/min</th>
<th>$E_{v}$ Mg/m$^2$</th>
<th>N-value</th>
<th>Reference</th>
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<td>0-6-11</td>
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<td></td>
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<td>HIGGINBOTTOM (1966)</td>
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<td></td>
<td>30-160</td>
<td>SIMM (1974)</td>
<td></td>
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<td>MZ</td>
<td>51-6</td>
<td>10</td>
<td></td>
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<td></td>
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<td>20-9</td>
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<td>30-160</td>
<td>SIMM (1974)</td>
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<td>30-160</td>
<td>SIMM (1974)</td>
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</table>

* Values for Middle and Upper Chalk. E. Extreme values. N. Normal values.
movements. The effects of these earth movements that occurred in
the Alpine Orogeny produced a number of major but simple folds such
as the Wealden Dome and the Hampshire Basin. Only in South Dorset,
the Isle of Wight and the Hog's Back in Surrey, is the Chalk more
severely folded and fractured.

4.3 Lithology

Geologically, the Chalk is divided into three main divisions:
Lower, Middle and Upper. These divisions are subdivided by fossils
into zones. Within the Chalk a number of distinct lithologies occur
and these are believed to reflect changes in the depth of the sea
at the time of deposition (see, for example, Wells and Kirkaldy,
1966).

At the base of the Lower Chalk is the Chloritic Marl, a sandy
glaucolithic marl. It contains much detrital material as, at that
time, a land mass was probably still supplying large quantities of
sand and mud. The proportion of clay in the Lower Chalk may reach
50% and the properties are more like those of heavily consolidated
marl. The upper part of the Lower Chalk is more massive although
still grey and marly. It is sometimes referred to as the Grey
Chalk. In the Chilterns and East Anglia the base of the massive
grey chalk is hard and is called the Totternhoe Stone. At the top
of the Lower Chalk are the plenus Marls consisting of yellowish
or greenish grey marls about 0.5 - 1.0 m. thick.

The base of the Middle Chalk is marked by a nodular bed termed
the Melbourn Rock. It consists of about 3m. of hard grey chalk with
nODULES. These nodular beds of chalk were probably formed during periods of temporary shallowing of the sea. Most of the Chalk was probably deposited in shallow sea about 180 m. deep. The Melbourn Rock and the overlying chalk of the *Inoceramus Labiatus* zone has a gritty texture caused by fragments of shell. The remainder of the Middle Chalk is white and massively bedded; although at the top flints and seams of marl occur.

Like the Middle Chalk, the base of the Upper Chalk is nodular and some marl bands occur. The basal hard chalk is called the Chalk Rock. Particularly in East Anglia another bed of hard chalk, the Top Rock, occurs some 10 m. above the Chalk Rock. The Upper Chalk mainly consists of uniform, soft, white chalk with flint as nodular bands or tabular layers.

Considerable variations in the thickness of the Chalk occur. These are shown in Figure 11.

A.4 Composition

The soft white chalks consist of material of size 0.0005 - 0.1 mm. For most samples a separation is possible into a coarse fraction and a fine fraction. The coarse fraction (20-30% of the total) is usually of size 0.01 - 0.1 mm. (coarse silt). This fraction consists of pieces of shell from creatures such as molluscs and echinoids, and complete and fragmentary skeletons of foraminifera. The fine fraction (70-80% of the total) is of size 0.0005 - 0.004 mm. (clay and fine silt). This fraction consists of intact and fragmentary skeletons of coccoliths (Black, 1953).
Fig. 11. Vertical sections showing variations in thickness and lithology of the Chalk (after Wells and Kirkaldy, 1966).
Each coccolith consists of a single calcite crystal. The original organic structures are normally preserved and no cementing agents are present. The strength is probably therefore due to mechanical interlock of particles and pressure solution at inter-granular boundaries (Higginbottom, 1966).

As has been stated in Section 4.3, the proportion of clay in the Chalk Marl may reach 50%. The overlying Grey Chalk has a clay content of 10-20%. An investigation of marl bands from the Middle Chalk (Ward et al., 1969) has shown them to have a calcite content of 50-80% of dry weight with an average value of 70%. X-ray and d.t.a. analyses showed the presence of quartz, montmorillonite and illite. The quartz and illite contents did not exceed 6% and the montmorillonite content was found to be 10-30% by weight with an average of 17%. The montmorillonite occurred in thin layers in which calcite is dominant. These layers were in an overall succession with calcite dominant (about 97%).

For Upper Chalk samples from the South Downs, Sparks (1949) gives the following determinations of insoluble residue.

<table>
<thead>
<tr>
<th>Zone</th>
<th>Insoluble residue (as % of dry weight of chalk)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actinocamax quadratus</td>
<td>1.40</td>
</tr>
<tr>
<td>Hagenokia beds</td>
<td>1.90</td>
</tr>
<tr>
<td>Offaster dilula</td>
<td>4.10</td>
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<tr>
<td>Marsupitas testudinarieus</td>
<td>3.33</td>
</tr>
<tr>
<td>Uintacrinus</td>
<td>1.53</td>
</tr>
</tbody>
</table>
These results indicate that there is some variation of clay content even in the typical white chalk. Sparks has used these differences to explain the topographical changes that occur in the South Downs. Hutchinson (1969) gives an insoluble residue of 2.5% for a sample of air dry Middle Chalk from near Folkestone Warren.

4.5 Index properties of chalk

4.5.1 Density

The range of values published for the dry density of the Lower Chalk is 1602 - 2243 kg/m³. Values of dry density of the Middle and Upper Chalk are significantly lower. The range of values for the Middle Chalk is 1375-1762 kg/m³, and that for the Upper Chalk 1264 - 1803 kg/m³. An exception to these ranges is a value of 1922.4 kg/m³ for the dry density of the Melbourn Rock quoted by Meigh and Early (1957). However, a higher value would be expected for this bed as it is one of the hard bands of chalk. Values of bulk density are typically 400 - 500 kg/m³ more than those of dry density. The greater density of the Lower Chalk is probably the result of a greater pre-consolidation load and the higher clay content giving increased compression of the original sediment.

4.5.2 Moisture content

Meigh and Early (1957) quote a value of 11.3% for Lower Chalk from Merstham, Surrey, and Hutchinson (1969) gives a value of 12.0% for a sample obtained from the Chalk Marl at Folkestone. Published moisture content values for the Middle Chalk show a range of 6.6 - 34.1% with a concentration between 23% and 28%. For the Upper Chalk
the range of values is 3.2 - 40% with a concentration between 25% and 35%. These figures indicate that generally the moisture content of the Chalk increases vertically. However, there are great variations that presumably reflect such factors as the state of fracturing, groundwater levels, and the amount of infiltration.

4.5.3 Porosity and voids ratio

Higginbottom (1966) gives a normal range of 21 - 30% and an extreme range of 17 - 41% for the porosity of the Lower Chalk. Duncan (1969) gives values of 18.7% for a sample of Lower Chalk, 19.5% for Melbourn Rock and 47.3% for Upper Chalk. For Middle Chalk and Upper Chalk taken together Higginbottom quotes a normal range of 41-50% and an extreme range of 34-53%. These determinations indicate that, as with natural moisture content, values increase upwards through the succession. The voids ratio also exhibits this increase (see Table 1). These increases in natural moisture content, porosity and voids ratio coincide with decreases in density upwards through the succession. The changes presumably reflect the decreased clay content and more open texture of the chalks higher in the succession.

4.5.4 Saturation moisture content

Higginbottom (1966) gives the extreme range for saturation moisture content of Lower Chalk as 8-23%, although Duncan (1969) quotes a value of 3.28% for one sample. For Middle and Upper Chalk Higginbottom quotes an extreme range of 19-44%. However, most determinations, Lewis and Cronay (1966), Parsons (1967) and Duncan (1969), appear to be within the range 21-28%
Lewis and Croney have classified chalk on the basis of saturation moisture content. They define soft chalk with a saturation moisture content of between 20 and 30%, medium chalks with between 15 and 20% and hard chalks with below 15%. They found that an approximately linear relationship exists between the amount of frost heave and the saturation moisture content.

4.5.5 Permeability

Higginbottom (1966) notes that the permeability of chalk as determined in the laboratory is about $10^{-8}$ m/sec. Despite the high porosity of chalk, the permeability is relatively low probably because of the small size of the pore spaces. However, the in situ permeability or transmissibility is suggested by Higginbottom to be about $10^2 - 10^4$ greater. This is the result of gradual but continual seepage from the chalk into the fissures.

Grange and Muir Wood (1970) give values of in situ permeability for the Lower Chalk as follows: Glauconitic Marl and Chalk Marl $10^{-6} - 10^{-8}$ m/sec; Grey Chalk $10^{-5} - 10^{-7}$ m/sec; and white chalk $10^{-5} - 10^{-6}$ m/sec. The permeability of the Chalk therefore increases up the succession.

4.5.6 Plasticity

For a sample probably from the Terebratulina late zone of the Middle Chalk, Hutchinson (1969) gives values as follows: liquid limit 29%; plastic limit 23%; plasticity index 6%. Ward et al gave the following values for a sample from the Twin Marl (Terebratulina late zone) at Mundford, Norfolk: liquid limit 58.1%; plastic limit 20.4%; and plasticity index 37.7%. A sample from the Micraster
coranginum zone of the Upper Chalk at Joss Bay, Kent gave values of: liquid limit 31%; plastic limit 23%; and plasticity index 8% (Hutchinson, 1972).

4.6 Mechanical characteristics of chalk

4.6.1 Unconfined compressive strength

No published work has been found that documents the results of unconfined compressive strength tests on samples of chalk.

4.6.2 Shear strength parameters of intact chalk

Drained triaxial compression tests on samples of Middle Chalk from Mundford, Norfolk gave $c'$ as 6.96 kN/m$^2$ and $\phi'$ as 25.8°. (Ward et al., 1969). Lake and Simons (1970) obtained the following values from drained triaxial tests on samples of Upper Chalk: $c'$ range 0-255 kN/m$^2$, average 35-50 kN/m$^2$; and $\phi'$ range 25-46°, average 33.5-38.5°. The values obtained were found to depend partly on the type of samples used in the tests. In conclusion Lake and Simons state that "No correlation has been found between the results of either drained or undrained triaxial compression tests and 'N' value, and these laboratory tests do not appear to give a reliable indication of the strength of the chalk in situ".

Wakeling (1966) obtained values of $c' = 0$ and $\phi' = 39^\circ$ using chalk samples from Newbury. Meigh and Early (1957) report values of $c' = 697$ kN/m$^2$ and $\phi' = 21^\circ$ on chalk from Coulsdon, Surrey. Tests on chalk from various localities carried out by Kee and Clapham (1971) gave $c'$ values of 0-207 kN/m$^2$ and $\phi'$ values of 26-43° with an average $\phi'$ of 39°. Values of $c'$ would seem to normally be in the range of 0-200 kN/m$^2$ and $\phi'$ about 39°.
Results of undrained triaxial compression tests show a large variation. Some examples of \( c_u \) values obtained are: 65 and 70 kN/m\(^2\) (Simm, 1966); 55-223 kN/m\(^2\) (Broadhead, 1966); 124-331 kN/m\(^2\) (Lake and Simons, 1970); 576 kN/m\(^2\) (Lewis and Cronen, 1966); 1518 and 2070 kN/m\(^2\) (Meigh and Early, 1957). Values of \( \phi' \) also vary greatly, for example: 10° (Wakeling, 1966); 12° and 16° (Meigh and Early, 1957); 0-22° (Broadhead, 1966); 37° (Lewis and Cronen, 1966); 34° and 0-40° (Simm, 1966). The higher \( \phi' \) values are not necessarily associated with high \( c_u \) values. The values suggest that either the tests themselves are somewhat unreliable or that the variation of values for chalk is extremely large. The values obtained from triaxial tests do not appear to be a suitable means of classifying chalk or of comparing chalk from different localities.

4.6.3 Shear strength parameters of fractured chalk

Hutchinson (1972) has obtained peak values of \( c' = 131 \) kN/m\(^2\) and \( \phi' = 42° \) on surfaces of chalk from Joss Bay tested in a shear box. He also obtained residual values, \( c'_r = 0 \) and \( \phi'_r = 30° \). Hutchinson (1969) gives values of \( \phi'_r \) of 20° and 23° on cut-plane tests on Lower Chalk from Folkestone Warren. The values of \( \phi' \) on these cut-plane tests are significantly lower than those obtained from the triaxial tests.

4.7 In situ condition of chalk

4.7.1 Introduction

Especially near the surface the condition of chalk varies considerably. It is usually much more fractured than at depth and may be traversed by swallow holes and 'pipes'. More severely
fractured chalk that may have been affected by frost heave or solifluction also occurs.

4.7.2 Near surface fractured chalk  

The layers of chalk near the surface are usually very fractured; the fractures often being open. These open fractures are thought to be due to frost shattering, especially during glacial times, solution, disturbance by roots and gravity. Sometimes the fractures are very open. Higginbottom (1966) describes fractures parallel to the contours that were encountered during construction of the Medway Bridge. These were probably opened by gravitational movements and were up to 50 mm. wide and at intervals of about 0.5 m.

4.7.3 Solifluction chalk and head  

Seasonal thawing of chalk during the Pleistocene Ice Age resulting in slow downhill movement of water-logged chalk debris is believed to be the cause of the solifluction chalcks, often called coombe deposits. These deposits occur in valley bottoms, at the foot of steep slopes and as lobate deposits in dry valleys or below scarp slopes. The deposits usually consist of poorly sorted accumulations of angular chalk and flint debris. Some deposits are bedded indicating redistribution of original debris and deposition by flowing water.

In some places a deposit may occur, which, although similar to solifluction chalk, has been formed in situ. This chalk is intensely frost shattered and disturbed. The chalk fragments are often angular, vary in size, and are deposited in a pasty matrix.
Flint bands and bedding planes continue through this chalk although they may be disturbed. This type of chalk is a form of head since it has not formed as a result of transportation.

4.7.4 Recognition of grades of chalk

Various attempts have been made to recognize a series of grades of chalk according to the degree of fracturing and/or hardness of the chalk. Dixon (see Palmer, 1966) has suggested the following classification related to 'N' values.

<table>
<thead>
<tr>
<th>Grade</th>
<th>N value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak chalk (including</td>
<td>0 - 20</td>
<td>Pieces of hard chalk 25 - 50 mm. in diameter in a matrix of putty-like chalk with a soft to firm clay consistency.</td>
</tr>
<tr>
<td>frost shattered and</td>
<td></td>
<td></td>
</tr>
<tr>
<td>solifluctioned chalk)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium hard chalk</td>
<td>20 - 40</td>
<td>Similar to weak chalk but with 150 - 300 mm. pieces of hard chalk. Matrix firm to stiff.</td>
</tr>
<tr>
<td>Hard chalk</td>
<td>over 40</td>
<td>Chalk that broke only with difficulty between finger and thumb or required a hammer to break it.</td>
</tr>
</tbody>
</table>

A more detailed grading of chalk at Mundford, Norfolk was made by Ward et al (1969). The classification is a function of three factors that were recognised as influencing stiffness of the chalk viz. hardness, spacing and orientation of joints and tightness of joints. The classification adopted was that given overleaf.
<table>
<thead>
<tr>
<th>Grade</th>
<th>Name</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>V</td>
<td>Structureless melange</td>
<td>Unweathered and partially weathered angular chalk blocks and fragments set in a matrix of deeply weathered remoulded chalk. Bedding and joints absent.</td>
</tr>
<tr>
<td>IV</td>
<td>Friable to rubbly chalk</td>
<td>Unweathered or partially weathered chalk with bedding and jointing. Joints and small fractures closely spaced ranging from 10 mm. apart to about 60 mm. apart. Joints commonly open up to 200 mm. and infilled with weathered debris and small unweathered chalk fragments.</td>
</tr>
<tr>
<td>III</td>
<td>Rubbly to blocky chalk</td>
<td>Unweathered medium to hard chalk with joints 60 mm. to 200 mm. apart. Joints open up to 3 mm., sometimes with secondary staining and fragmentary infillings.</td>
</tr>
<tr>
<td>II</td>
<td>Medium hard chalk</td>
<td>Joints more than 200 mm. apart. When dug out for examination purposes this material does not pull away along joint surfaces but fractures irregularly. Most of the T. jata and H. planus zones where unweathered and fractured were placed within this grade at Mundford.</td>
</tr>
<tr>
<td>I</td>
<td>Hard, brittle chalk</td>
<td>Details as for grade II, but the chalk is harder. Rock beds such as the Chalk Rock, Top Rock, and other partially recrystallised beds, and tougher shell-rich beds where secondary iron cementation has hardened the material, were placed within this grade.</td>
</tr>
</tbody>
</table>

Grades IV and V were found to result largely from weathering and to be independent of lithology. However, grades I and II are unweathered and their separation reflects lithological differences.

Wakeling (1970) suggests that some correlation exists between the
'N' value obtained in the standard penetration test and the grade of chalk. He also gives in his classification an additional grade to cover solifluction chalk. The remaining grades are the same as those adopted by Ward.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Brief description</th>
<th>'N' Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>VI</td>
<td>Extremely soft structureless chalk containing small lumps of intact chalk</td>
<td>below 8</td>
</tr>
<tr>
<td>V</td>
<td>Structureless remoulded chalk containing lumps of intact chalk</td>
<td>8 - 15</td>
</tr>
<tr>
<td>IV</td>
<td>Rubbly, partly weathered chalk with bedding and jointing. Joints 10 - 60 mm. apart, open to 20 mm. and often infilled with soft remoulded chalk and fragments</td>
<td>15 - 20</td>
</tr>
<tr>
<td>III</td>
<td>Rubbly to blocky unweathered chalk. Joints 60 - 200 mm. apart, open to 3 mm. and sometimes infilled with fragments</td>
<td>20 - 25</td>
</tr>
<tr>
<td>II</td>
<td>Blocky medium-hard chalk. Joints more than 200 mm. apart and closed.</td>
<td>25 - 35</td>
</tr>
<tr>
<td>I</td>
<td>As for grade II but hard and brittle</td>
<td>over 35</td>
</tr>
</tbody>
</table>

Fookes and Horwill (1970) have drawn attention to the dangers of using a grading classification for comparing strata at different localities. For example, the discontinuities present at a particular locality will depend not only on weathering but also on the local tectonic history.
4.7.5 In situ tests on chalk

The standard penetration test (S.P.T.) and plate-bearing test have been used frequently to determine the condition of chalk in situ. These tests are not considered by the present author to be particularly relevant to the investigation of rock slope stability, and they have been mainly used to determine the strength en masse of the chalk for foundations.

A number of problems associated with the standard penetration test are known to exist. Higginbottom (1966) has suggested that the boring tools used in the test may soften the chalk sufficiently during driving to produce results that are not typical of the actual in situ chalk. Wakeling (1970) also suggests that unrepresentative values may be obtained due to borehole disturbance, and that in situ visual examination should be used wherever possible.

Lake and Simons (1970) have shown an approximate correlation between S.P.T. value and deformation modulus, although better results were obtained using small scale plate loading tests. No correlation was found to exist between the S.P.T. values and the results of drained or undrained triaxial compression tests. As mentioned in Section 4.6.2, Lake and Simons suggest that the laboratory tests do not give a reliable indication of the strength of in situ chalk.

4.7.6 Swallow holes and pipes

Swallow holes occur where solution has caused fractures to open sufficiently to allow the inflow of large amounts of surface run off. Most swallow holes have a cone-shaped profile tapering downwards. They may be 30 m. in depth.
The swallow holes frequently become infilled with sand, clay and flint gravel from deposits overlying the chalk which have collapsed into them. The infilled swallow holes are termed pipes.

The distribution of swallow holes and pipes is closely related to the junction between the Chalk and the overlying Tertiary strata (Higginbottom, 1966). The Tertiary strata are usually less permeable than the Chalk and allow greater surface run off which then tends to percolate underground on reaching the Chalk outcrop. Away from the Chalk-Tertiary junction swallow holes and pipes are normally absent because erosion has presumably removed them. The junction between the Chalk and the overlying Tertiary beds is frequently very uneven due to solution.

4.7.7 Groundwater

Ineson (1962) has suggested that water percolates through the more permeable surface layer of chalk and is then absorbed into the fracture system and also slowly into the pores of the chalk. At depth the water becomes concentrated in specific open fractures especially in the zone of seasonal water table.

Areas of high transmissibility correspond with river valleys and dry valleys and anticlinal folds. Areas of low transmissibility correspond with synclinal folds. Water flow is often concentrated in the harder bands of chalk such as the Melbourn Rock. Where these harder bands outcrop they frequently give rise to springs (see for example Fordham, 1965). Faults tend to prevent groundwater movement because at depth they may contain crushed chalk of low permeability.
The Lower Chalk has few water-bearing fractures and the higher clay content of this chalk may prevent the fractures remaining open. In addition the Lower Chalk has tended to be always below the regional water table and groundwater circulation has been limited.

The maximum elevation of the water table is usually in February or March and the minimum elevation in September or October, although variations do occur (Smith, 1972). Larger variations of the seasonal water table occur on high ground and slopes than in the valley bottoms.

4.7.6 Effects of ice

As mentioned in Section 4.7.3 the solifluccion and head types of chalk are believed to have been formed by freeze-thaw action, resulting mainly from the Pleistocene Ice Age. Chalk has also been found to be susceptible to the effects of freezing under present climatic conditions in Britain.

Higginbottom (1966) describes the likely mechanism. Water held in the pores remains fluid because capillary forces tend to depress its freezing point. Suction occurs and the super cooled pore water moves by capillarity towards the cracks and joints where it freezes. Ice lenses are then formed, usually along bedding planes, with resulting heave of the surface.

As mentioned in Section 4.5.4 Lewis and Cronen (1966) have shown a linear relationship between frost heave and saturation moisture content. The increase in height of their specimens after subjecting to the frost heave test varied from about 20% for hard chalk to about 130% for soft chalk.
Freeze-thaw action is generally considered to be responsible for much of the spalling of exposed faces of chalk. There is some evidence from experimental work to support this (see, for example, Carson and Kirkby, 1972).
5. CHARACTERISTICS OF CHALK SLOPES

5.1 Introduction

Slopes developed in chalk were examined in some detail in an attempt to define those factors that affect their degradation and eventual stability. Three main categories of slopes were recognised: (a) natural inland slopes; (b) natural coastal slopes, usually subject to active erosion; and (c) artificially excavated slopes.

Natural inland slopes were investigated primarily with the aim of establishing the natural stable slope angles for chalk. The natural coastal slopes undergoing active erosion were examined to determine any failure mechanisms operative within steep slopes. Artificially excavated slopes e.g. quarry faces and cuttings, were also investigated to determine the existence of any failure mechanisms acting within these slopes, and additionally, to recognise separate factors that may affect such artificial slopes. Having identified the main factors that affect stability of these chalk slopes, types of mass movements involved could be recognised. The localities studied during field work by the author are shown in Fig. 12.

5.2 Natural inland slopes

Natural inland slopes were examined to determine the types of slopes and slope angles that are achieved after long periods of degradation. The methods adopted for this study were: measurement of slope angles from Ordnance Survey maps; field examination of slopes; and the examination of oblique and vertical aerial photographs.
5.2.1 Measurement of slope angles from Ordnance Survey maps

Slope measurements have been made from Ordnance Survey 1:63,360 Maps of the North Downs and South Downs. The method used was measurement of the horizontal distance between two selected contours for the steeper sections of scarp slopes, and of the contour difference in that distance. These measurements permitted calculation of the average angle of slope. These calculations are open to possible error because of limitations imposed by the scale of the maps used. For an 11° slope the largest possible error is to record a minimum slope angle of 9° or a maximum slope angle of 14°. With steeper slopes the possible error is greater. For a 21° slope the largest possible error is to record a minimum slope angle of 16° or a maximum slope angle of 29°. However, such large errors are only likely in a limited number of cases. The method was used to give an indication of the slope angles achieved by steep natural slopes in chalk.

Slopes were measured at 0.8 km. intervals along the North Downs scarp on O.S. Sheet 170 and at similar intervals along the South Downs scarp on O.S. Sheet 183. The results for the North Downs (Fig. 13) show concentrations of steep slopes at 8°, 13° and 16°. Two slopes of 25° were recorded. Results for the South Downs (Fig. 14) indicate concentrations of slope angles at 14° and 25°. Three slopes of 29° were recorded. Slope angles measured for the South Downs are significantly greater than those for the North Downs. The results indicate that natural inland chalk slopes frequently reach 13-29°.

To investigate the distribution of chalk slopes over a large area a slope angle map was constructed of North Hampshire (Figs. 15 and 16).
Fig. 3.14. Histograms of slope angles measured from (a) O.S. Sheet 170 (part of North Downs) and (b) O.S. Sheet 183 (part of South Downs).
Fig. 16. Detail of northwest part of Slope Angle Map.

Scale: 1:63,360.
The method of construction adopted was that described by Ollier and Thomasson (1957). The slope angle map shows that slopes steeper than 9° do not often occur. The steeper slopes (i.e., in excess of 14°) are concentrated in the north, east and south of the map area. These steep slopes generally coincide with the main chalk scarps. Inspection of the Geological Survey 1:63,360 Maps indicates that these scarps are related to eroded anticlinal folds. The erosion of these anticlinal folds presumably leads to scarp retreat along the limbs of the folds (Fig. 17). The steep slopes in the east of the map area are along the western side of the Wealden Dome. In the south-east the steep slopes are related to the Warnford Anticline and the Meon Valley Anticline. Another area with steep slopes in the south coincides with the Winchester Anticline. An east-west line of steep slopes occurs near the northern margin of the map area. These slopes are related to the Kingsclere Anticline and the Ham Anticline.

In all these instances erosion has been sufficient to expose the Middle and Lower Chalk in addition to the Upper Chalk. The steep parts of these slopes are formed mainly of Middle Chalk. Steep slopes are sometimes developed in Upper Chalk, but only rarely in Lower Chalk. Steep slopes are achieved in Lower Chalk only where it forms the lower part of a slope mainly formed by Middle Chalk e.g. near Hawkley (SU 7329). The usual relationship between steep angled slopes and inward facing scarps formed along anticlinal axes may be modified. Wooldridge and Linton (1955) comment that the Lower Chalk outcrop in the Ham-Woodhay inlier forms a bench rather than a scarp-walled inlier. They explain this modification by marine planation in the Pliocene.
(a) Erosion at crest of anticline.

(b) Erosion of scarps on each limb of fold.

Fig. 17: Scarp Retreat
The relationship between steep slope angles and the anticlinal axes suggests that the steep slopes result not only from the relative resistance to erosion of Lower, Middle and Upper Chalk and underlying strata, but also from the denudation chronology of the area.

Steep slope angles of 14-22° occur in the north-west of the map area, some 3 km. to the south of the main scarp face. These steep slopes in the upper part of the Bourne Valley are aligned and coincide with the trend of the axis of the Heydon Hill Anticline.

Slopes with angles of 9-14° are largely confined to: the gentler parts of the scarp slopes described above; the upper portions of dry valleys cut in the dip-slopes; and, in a few instances, the side slopes of river valleys. Where these steeper slopes exist along the river valleys they presumably indicate that active lateral erosion by these rivers has ceased only in relatively recent geological history.

Low angled slopes (less than 9°) occur over much of the area of the map. Slope angles of less than 3° broadly coincide with the anticlinal axes developed in the centre of the map area e.g. the Farleigh Wallop Anticline. The relationship between structure and topography is that often described as 'uninverted relief'. Wooldridge and Linton (1955) have noted that in this region the anticlinal axes are marked by low swelling ridges and the synclinal tracts by minor longitudinal valleys e.g. the valleys of the Micheldever Stream and the upper Itchen.

5.2.2 Field examination of slopes

A large number of natural inland slopes in chalk have been observed
in the field. These observations indicated that only exceptionally does the slope angle exceed 30°. More detailed investigations were made in an area where the slope angles exceed this value.

Field measurement of slope angles was undertaken at Dunstable Downs where the scarp of the Chiltern Hills is characterised by steep slopes. The field measurements were made using an Abney Level. Slopes on the steeper parts of the scarp at this locality varied from 26° to 30°. A coombe at Five Knolls (TL 0020) showed the following variation: north side (south facing) 26°; centre 27°; and south side (west facing) 30°.

A prominent cutting in the Chalk occurs below Whipsnade Zoo on the B4540 road (SP 9916). The steep slope is probably a combination of the natural scarp and steepening produced by construction of the road. The cutting is some 17 m. high and has a slope angle of 32°. Lower down the scarp the slope steepens to 38° in a 10 m. high cutting. The slope exhibits terracettes and small hummocky slips occur. These show poor exposures of pebbly chalk in a sandy chalk matrix. This steep slope of 38° would appear to be only just stable. However, the terracettes may indicate that the slope is stabilising naturally and not that it is potentially unstable. The lower part of the scarp slope is at an angle of 17°.

At Pegsdon the dry valley in which one of the trial pits was located (see Section 6.2.2) has side slopes of 25-35°. Some parts of the steeper east facing slope exhibit terracettes with loose pebbly chalk exposed beneath tussocks of grass. Nevertheless the slope is
well vegetated with grass, shrubs and small trees and no signs of major instability are present. The trial pit excavated on one side of the valley proved that apparently in situ chalk was present at a depth of only 0.6 m. The valley sides are of chalk with only a thin covering of loam. The slopes are of the form shown in Fig. 17, and have presumably been 'carved' by erosion. This is in contrast to the methods of degradation observed by the author for many natural and artificial slopes which have a free face of in situ chalk exposed. Where active erosion has ceased these slopes accumulate extensive scree at their foot. The scree eventually extends upward to hide the free face. (Fig. 16).

5.2.3 Examination of oblique and vertical aerial photographs

Oblique and vertical aerial photographs have been examined for surface indication of instability. The areas investigated were mainly in the South Downs and in Kent. Terracettes were found to be a common feature of the steeper scarp slopes. No other signs of instability affecting inland slopes were observed. Tension cracks were noted behind high coastal chalk cliffs at a number of localities.

5.3 Natural coastal slopes

Natural coastal slopes examined included those undergoing active erosion by the sea, and also those that are no longer being actively eroded or are protected by sea-defence works. These cliffs and steep slopes were investigated with the aim of establishing how these slopes are affected by erosion, and how they continue to degrade when active erosion ceases.
Slope formed by, for example, fluvial action

Waste mantle

Vertical slope formed by, for example, marine erosion

Scree and waste mantle

Fig. 18. Variations in slope degradation.
5.3.1 Coastal slopes undergoing active erosion

Many kilometres of coastal chalk cliffs have been examined by the author. Their examination was considered to be essential since these cliffs allow the failure mechanisms actively operating within steep chalk slopes to be clearly studied. The coastal sections studied by the author have included the Thanet coast, St. Margaret’s Bay, Shakespeare Cliff, Eastbourne-Brighton, Isle of Wight, and the Beer area (Fig. 12).

Where active erosion is in progress cliffs cut in the chalk are often inclined at 90° to the horizontal and are sometimes slightly overhanging. These vertical cliffs occur frequently in the areas studied e.g. White Ness near Margate, East Cliff near Ramsgate, Beachy Head, Friar’s Bay near Newhaven, Telscombe Cliffs and Freshwater Bay (Isle of Wight). The angle at which the cliffs are inclined, is, however, frequently reduced from the vertical to some lesser angle. A number of methods by which the reduction in angle is achieved have been observed. These are:

1. Planar failures
   A. Translational sliding
   B. Block sliding
   C. Inverted block removal
   D. Tension-shear
   E. Fracture controlled rock fall
   F. Irregular rock fall

2. Wedge failure
   G. Wedge sliding
   H. Inverted wedge removal

3. Complex failures
   I. Complex sliding

4. Superficial failures
   J. Circular slip
   K. Mudflow

5. Miscellaneous failures
   L. Swallow hole controlled.
These different types of failure are shown diagramatically in Fig. 19.

1. Planar failures

The most usual way for the cliff angle to be reduced is by failure of part of the cliff by movement along a single failure plane. The failure plane is usually a prominent joint plane inclined at a steep angle, typically 65-85°, towards the coast. The strike of the joint plane is usually parallel or nearly parallel with the trend of the coast. Examples of these failure surfaces and potential failure surfaces are given in Table II.

Many of these planar failures have been observed to have occurred along planes that are coincident in orientation with a dominant fracture set. In such cases failure has therefore presumably taken place along a pre-existing plane of weakness within the rock mass. That movement is often along pre-existing planes is also suggested by the occurrence of fracture planes with overlying masses of rock that appear unstable. In such cases these fracture planes are the potential failure planes. At Whitecliff Bay and Alum Bay in the Isle of Wight failure planes are developed along bedding planes. At these localities the dip of the bedding is sufficiently steep (60-75°) for these planes to be potential failure surfaces. At the other localities listed in Table II the dip of the bedding is 0-10°, insufficiently steep for them to act as potential failure surfaces.

The distinction between failures occurring along joints, faults and bedding planes and planes of unconformity is considered to be
1. PLANAR FAILURES

A. Translational Sliding

B. Block Sliding

(i) Backward limiting fractures inclined inwards from face

(ii) Backward limiting fractures inclined vertically

(iii) Backward limiting fractures inclined towards face

Fig. 19. Types of failure
C. Inverted block removal

D. Tension-shear

E. Fracture controlled rock fall

F. Irregular rock fall

Fig. 19 (continued) Types of failure
2. WEDGE FAILURES

G. Wedge sliding

H. Inverted wedge removal

3. COMPLEX FAILURES

I. Complex sliding

Fig. 19 (continued). Types of failure.
4. SUPERFICIAL FAILURES

J. Circular slip

K. Mudflow

Fig. 19 (continued). Types of failure.
<table>
<thead>
<tr>
<th>Locality</th>
<th>Geological horizon with zones (Rams, 1900)</th>
<th>Dip of failure surface</th>
<th>Strike of failure surface</th>
<th>Trend of joint</th>
<th>Remarks</th>
<th>Type of failure where known</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. East side of Foreness Point, Thanet (TR 3871)</td>
<td>U. Chalk: <em>Marsupites testidinarius</em> <em>Eintacrinus socialis</em></td>
<td>(a) 90°N.</td>
<td>N 60°W.</td>
<td>N 45°W.</td>
<td>Limonite stained failure surface</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) 65°N.</td>
<td>N 65°W.</td>
<td>N 40°W.</td>
<td>Failure surface along joint plane</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) 83°N.</td>
<td>N 60°W.</td>
<td>N 40°W.</td>
<td>Potential failure surface along joint plane</td>
<td></td>
</tr>
<tr>
<td>2. Botany Bay, Thanet (TR 3971)</td>
<td>U. Chalk: <em>Marsupites testidinarius</em> <em>Eintacrinus socialis</em></td>
<td>(a) 90°</td>
<td>N 60°W.</td>
<td>N 45°W.</td>
<td>Limonite stained failure surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) 89°W.</td>
<td>N 56°W.</td>
<td>N 45°W.</td>
<td>Failure surface along prominent joint</td>
<td></td>
</tr>
</tbody>
</table>
### TABLE II. (continued) Examples of the orientation of planar failure surfaces and potential planar failures

<table>
<thead>
<tr>
<th>Locality</th>
<th>Geological horizon with zones (Rowe, 1902)</th>
<th>Dip of failure surface</th>
<th>Strike of failure surface</th>
<th>Trend of coast</th>
<th>Remarks</th>
<th>Type of failure where known</th>
</tr>
</thead>
<tbody>
<tr>
<td>3. South side of Hackernow Point, Thanet (TR 3970)</td>
<td>U. Chalk: <em>Marsupites testudinarius</em>, <em>Uintacrinus socialis</em></td>
<td>(a) 67°E</td>
<td>NO°</td>
<td>NO°</td>
<td>Failure surface</td>
<td>D</td>
</tr>
<tr>
<td>5. East Cliff, Ramsgate, U. Chalk: Thanet (TR 3965)</td>
<td><em>Micraster coranguinum</em></td>
<td>(a) 66°E</td>
<td>N8°E</td>
<td>N26°E</td>
<td>Failure surface</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) 70°E</td>
<td>N11°E</td>
<td>N28°E</td>
<td>Failure surface</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) 65°E</td>
<td>N44°E</td>
<td>N28°E</td>
<td>Stepped failure surface</td>
<td>D</td>
</tr>
<tr>
<td>Locality</td>
<td>Geological horizon with zones (Rowe, 1900)</td>
<td>Dip of failure surface</td>
<td>Strike of failure surface</td>
<td>Trend of coast</td>
<td>Remarks</td>
<td>Type of failure where known</td>
</tr>
<tr>
<td>----------</td>
<td>------------------------------------------</td>
<td>-----------------------</td>
<td>--------------------------</td>
<td>---------------</td>
<td>---------</td>
<td>---------------------------</td>
</tr>
<tr>
<td>6. Pegwell Bay, Thanet (TR 3664)</td>
<td>U. Chalk: Uintacrinus socialis, Micraster corangulunum</td>
<td>(a) 90</td>
<td>N60°W.</td>
<td>W78°W.</td>
<td>Failure surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) 80°W.</td>
<td>N24°W.</td>
<td>W78°W.</td>
<td>Failure surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) 84°W.</td>
<td>N46°W.</td>
<td>W78°W.</td>
<td>Failure surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(d) 88°S.</td>
<td>N90°E.</td>
<td>W78°W.</td>
<td>Failure surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e) 70°S.</td>
<td>N70°E.</td>
<td>W78°W.</td>
<td>Potential failure surface</td>
<td>D</td>
</tr>
<tr>
<td>7. South side of St. Margaret's Bay, Kent (TR 3644)</td>
<td>U. Chalk: Micraster corangulunum, Micraster cortestudinarium</td>
<td>(a) 90</td>
<td>N37°E.</td>
<td>N40°E.</td>
<td>Potential failure surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) 65°S.</td>
<td>N63°E.</td>
<td>N40°E.</td>
<td>Failure surface</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) 80°S.</td>
<td>N84°E.</td>
<td>N40°E.</td>
<td>Failure surface</td>
<td>B</td>
</tr>
<tr>
<td>Locality</td>
<td>Geological horizon with zones (Rowe, 1966)</td>
<td>Dip of failure surface</td>
<td>Strike of failure surface</td>
<td>Trend of coast</td>
<td>Remarks</td>
<td>Type of failure where known</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>---------------------------------------------</td>
<td>------------------------</td>
<td>---------------------------</td>
<td>----------------</td>
<td>--------------------------------------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td>8. Eastbourne, Sussex (TV 6096)</td>
<td>M. Chalk: Terebratulina lata Inocerasus labiatus</td>
<td>73°S.</td>
<td>N70°E.</td>
<td>N35°E.</td>
<td>Failure surface</td>
<td></td>
</tr>
<tr>
<td>9. West side of Eirling Gap, Sussex (TV 5496)</td>
<td>U. Chalk: Micraster coranginum</td>
<td>(a) 64°S.</td>
<td>N35°W.</td>
<td>N55°W.</td>
<td>Failure surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) 81°S.</td>
<td>N41°W.</td>
<td>N55°W.</td>
<td>Potential failure surface along joint plane</td>
<td></td>
</tr>
<tr>
<td>10. West of Newhaven Harbour (TV 4399)</td>
<td>U. Chalk: Goniatethus quadrata</td>
<td>66°S.</td>
<td>N85°W.</td>
<td>N85°W.</td>
<td>Failure surface along joint plane</td>
<td>E</td>
</tr>
</tbody>
</table>


<table>
<thead>
<tr>
<th>Localities</th>
<th>Geological horizon with zones (Rowe, 1900)</th>
<th>Dip of failure surface</th>
<th>Strike of failure surface</th>
<th>Trend of coast</th>
<th>Remarks</th>
<th>Type of failure where known</th>
</tr>
</thead>
<tbody>
<tr>
<td>11. East side of beach access, Peacehaven, Sussex (TQ 4000)</td>
<td>J. Chalk: <em>Gonioteuthis quadrata</em></td>
<td>(a) 54°S.</td>
<td>N69°E.</td>
<td>N67°W.</td>
<td>Potential failure surface along joint plane</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) 67°S.</td>
<td>N53°W.</td>
<td>N67°W.</td>
<td>Potential failure surface along joint plane</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) 67°S.</td>
<td>N47°W.</td>
<td>N67°W.</td>
<td>Failure surface</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(d) 80°S.</td>
<td>N83°E.</td>
<td>N67°W.</td>
<td>Extensive failure surface</td>
<td></td>
</tr>
</tbody>
</table>


TABLE II. (continued) Examples of the orientation of planar failure surfaces and potential planar failures

<table>
<thead>
<tr>
<th>Locality</th>
<th>Geological horizon with zones (Rowe, 1900)</th>
<th>Dip of failure surface</th>
<th>Strike of failure surface</th>
<th>Trend of coast</th>
<th>Remarks</th>
<th>Type of failure where known</th>
</tr>
</thead>
<tbody>
<tr>
<td>12. West side of beach access, Peacehaven, Sussex (TQ 4000)</td>
<td>U. Chalk: Goniotethis quadrata</td>
<td>(a) 75°E.</td>
<td>N21°W.</td>
<td>N67°W.</td>
<td>Failure surface</td>
<td></td>
</tr>
<tr>
<td>13. Whitecliff Bay, Isle of Wight (SZ 6385)</td>
<td>U. Chalk</td>
<td>(a) 75°N.</td>
<td>N80°E.</td>
<td>N55°W.</td>
<td>Limonite stained failure surface</td>
<td></td>
</tr>
<tr>
<td>14. Culver Cliff, Isle of Wight (SZ 8534)</td>
<td>L. and M. Chalk</td>
<td>30°S.</td>
<td>N54°W.</td>
<td>N67°E.</td>
<td>Failure surface along joint plane</td>
<td></td>
</tr>
<tr>
<td>Locality</td>
<td>Geological horizon with zones (Rowe, 1900)</td>
<td>Dip of failure surface</td>
<td>Strike of failure surface</td>
<td>Trend of coast</td>
<td>Remarks</td>
<td>Type of failure where known</td>
</tr>
<tr>
<td>------------------------------------------------------------------------</td>
<td>--------------------------------------------</td>
<td>------------------------</td>
<td>---------------------------</td>
<td>----------------</td>
<td>-------------------------------------------------------------------------</td>
<td>----------------------------</td>
</tr>
<tr>
<td>15. East side of Freshwater Bay, Isle of Wight (SZ 8534)</td>
<td>U. Chalk</td>
<td>(a) 46°S.</td>
<td>N59°W.</td>
<td>N50°W.</td>
<td>Potential failure along slickensided joint plane</td>
<td>A/B</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) 54°S.</td>
<td>N73°W.</td>
<td>N50°W.</td>
<td>Potential failure surface along joint plane</td>
<td>A/B</td>
</tr>
<tr>
<td>16. West side of Freshwater Bay, Isle of Wight (SZ 8534)</td>
<td>U. Chalk</td>
<td></td>
<td></td>
<td></td>
<td>Failure surface</td>
<td>A</td>
</tr>
<tr>
<td>17. Alum Bay, Isle of Wight (SZ 3085)</td>
<td>U. Chalk</td>
<td>(a) 62°N.</td>
<td>N67°E.</td>
<td>N70°E.</td>
<td>Failure surface along bedding plane</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) 73°N.</td>
<td>N62°E.</td>
<td>N70°E.</td>
<td>Failure surface along bedding plane</td>
<td>A</td>
</tr>
</tbody>
</table>
important. For example, bedding planes have been observed to be usually better developed as separation planes, more constant in orientation, and more extensive than sets of joint planes. Fault planes although not frequently developed within the Chalk have also been observed to have peculiar characteristics e.g. they often contain infillings of brown clay. The importance of recognising different fracture surfaces is discussed in Chapter 9.

Where the planes are inclined at angles of up to 80-85° to the coast, failure appears to take place mainly by sliding of an unstable mass of rock along a stable mass beneath. Where the planes are inclined at or near the vertical (a dip of 80-90° to the coast) failure appears to occur not by sliding but by toppling of a vertical columnar mass of rock resulting in a large-scale rock fall. Two main types of planar failure may therefore be recognised depending on the predominance of sliding or of tension.

4. Translational sliding

Although a distinction may be made between sliding and tensional modes of planar failure, actual failures observed in the field appear somewhat more complex. Planar failures involving sliding have been found to show some marked differentiating features. A few failures have been observed that appear to have taken place by movement of an entire mass of unstable rock upon stable rock below; the movement having occurred along a joint plane. After movement the cliff profile corresponds to that of the failure plane. Evidence for such complete translational failures with a single probably rapid movement in the
large masses of fresh chalk occurring at the base of the cliff involved. Examples of this type of sliding failure have been observed in Thanet and the Isle of Wight.

In the cliffs to the south of Dumpton Gap, Thanet, a failure had occurred along a joint plane with a dip of $65^\circ$. The strike of the failure plane was $N 40^\circ E$; that of the coast being $N 25^\circ E$. At Alum Bay, Isle of Wight, a similar but larger failure had occurred along a plane with a dip of $65^\circ N$, and a strike of $N 62^\circ E$. The strike of the coast at that locality is $N 70^\circ W$. The orientation of the failure plane was the same as that of the bedding, and presumably coincided with a bedding plane. At Whitecliff Bay, Isle of Wight, a surface along which a similar failure may have occurred in the past few years was noted. The possible failure plane had a dip of $65^\circ N$, and a strike of $N 76^\circ W$, and coincided with a bedding plane. At this locality the cliff section is aligned $N 55^\circ W$.

B. Block sliding

Of more frequent occurrence are planes along which sliding is occurring irregularly. The rock overlying the failure plane does not slide rapidly 'en masse' but moves as a series of discrete blocks over a period of time. Initially blocks of rock at the base of the mass overlying the potential failure plane are removed. In the coastal sections this removal is effected mainly by processes such as wave action, solution, weathering and gravity. Blocks are then progressively removed leaving the unstable rock mass less well supported and a larger sliding failure may then occur or the block
removal process continues until all the unstable rock is removed. The size of the blocks involved in each stage of failure has been found to depend largely on the frequency of fracturing within the rock and also the extent of the fracture surfaces. Each block is usually bounded by prominent fractures.

At St. Margaret's Bay, Kent, progressive block failure has been observed (Plate I). In one example, the failure plane was developed along a joint inclined at 65°S. with a strike of N63°E. Blocks sliding down this failure plane were terminated backwards by joints dipping 50°W. with a strike of N28°E. The coast has an orientation of N40°E. The failure plane dips seawards but the joints terminating the blocks dip landwards. In another example at the same locality the failure plane along which sliding occurred was inclined at 80°S. with a strike of N84°E. The blocks were terminated backwards by joints dipping 55°W. and striking N38°E. The fracture spacing at this locality was a minimum of 1 - 1.5 m. and so individual blocks were large. The minimum volume of blocks is estimated to be 2 cu.m.

Another example of progressive block sliding failure has been examined at Culver Cliff, Isle of Wight (Plate IA). The orientation of the coast is N87°S. The surface along which movement occurred had a dip of 30°S. and a strike of N54°W. The blocks are bounded in the rear by bedding planes dipping at 68°N. and striking N62°W. Laterally the blocks are bounded by joints dipping at 60°N. and with a strike of N55°E. A typical block was measured and found to be 2m. wide, 4m. high and 3m. deep. Individual blocks were traversed by some minor fractures.
Plate I. Progressive block sliding failure (Type 1B) at St. Margaret's Bay, Kent.
Plate Ia. Progressive block sliding failure (Type 1B) at Culver Cliff, Isle of Wight.
In some places the blocks were more irregular in shape mainly because of the presence of another set of fractures with a dip of 38°S. and strike of N71°W. intersecting the other joints and bedding planes.

C. Inverted block removal

At the foot of Shakespeare Cliff, Dover, an extensive plane (dip 74°W., strike N34°E.) inclined landwards is exposed. The rock below the plane has been removed by the sea. The plane extends to the base of the cliff and a possible plane inclined seawards on which block sliding might occur is not exposed. Few joints were present in the section and this presumably assists the cliff in remaining apparently stable even if little support exists at the base. The strike of the exposed plane coincides with the orientation of the coast.

D. Tension-shear

Another type of failure has been observed at some localities that appears to involve both sliding and tensional effects. In some respect, this is intermediate between the translational and block sliding, and rock falls. Failures of this type often involve an entire section of cliff. The profile of the cliff after failure is composed of two parts. The upper part is normally vertical and the lower part is inclined towards the coast. The lower segment of the profile has been found to be typically inclined at 65 - 75°. The surface of the lower part of the profile is usually relatively smooth, although definite ridges or striae aligned in the direction of maximum slope frequently occur (Plate II). Where failure had been recent loose, rubbly and powdery chalk was observed on these surfaces. Some of the surfaces of failures observed in Thanet are stepped in appearance, each 'step' corresponding
Plate II. Tension-shear failure (Type 1D) at East Cliff, Ramsgate.
to the position of a bedding plane. The upper part of the profile is not smooth or ridged in a downward direction. Instead the surface is composed of blocky, irregular chalk. This vertical segment frequently constitutes one third to one half of the total length of the vertical profile.

Incipient failures of this type have marked tension cracks at the top of the profile. Such cracks are sometimes observable in the ground above the incipient failure. Large cracks of this type have been observed, for example, near Beachy Head. Such cracks are sometimes apparent on aerial photographs.

An example of an incipient failure at Pegwell Bay, Thanet, is shown in Plate III. The cliff at this locality is about 20m. high. A vertical plane extends down to about 8m. below the top of the cliff. The upper part of the vertical plane is a tension crack that is open to a depth of about 4m. Just below a prominent flint band the vertical plane is replaced by an inclined plane. The inclined plane slopes seawards at about 70°. The strike of this potential failure plane does not correspond exactly to that of any fracture set recognised at this locality. (see Section 7.8).

The strike of the potential failure plane was N70°E, whereas the mean strike of the nearest parallel fracture set has been measured as N67°W. The mean dip of that fracture set is 89°, quite different from the 70° dip of the inclined potential failure plane. Although the tensional crack is probably mainly developed along the strike of an existing fracture plane, the field evidence suggests that the inclined part of the failure plane may develop independently of any
Plate II: Inevitable tension-shear failure (Type ID) at Pégase, Bouy, Thonêt.
pre-existing fracture planes. Although usually utilising existing fracture planes these failures may be partially or completely independent of pre-existing fracture planes. The inclined failure planes probably represent shear failure of the chalk. The smoothness of the failure surfaces together with the ridges and strike in the directions of movement and the chalk debris on the surfaces are all in support of this. These failures may be described as tension-shear failures.

Another example of a tension-shear failure plane has been examined at East Cliff, Ramsgate (Plate II). This failure took place some time ago and the cliff section is being rapidly modified by active sea erosion. The orientation of the coast at this locality is N28°E. The lower inclined part of the failure plane has for the most part a strike of N11°E and a dip of 70°W. The failure plane tends to curve into prominent pre-existing fracture planes with a strike of N42°W and a dip of 60°N. The actual failure plane appears in the cliff as a resolution of the direction of the coastline and of the dominant set of pre-existing fractures. Field work has shown that there is also a poorly developed set of fractures with a similar strike to that of the failure plane.

The lower inclined part of the failure surface is smooth and has ridges extending downwards in the direction of probable movement. In places the surface is slightly stepped. Each step corresponds to a bedding plane or more especially a flint band or layer of small flint nodules that occur as horizontal bands at frequent intervals in the chalk of this area. Apart from this effect the bedding planes do not
appear to determine the modes of failure or the orientation of the failure surface. Other examples of tension-shear failures have been observed in Thanet, notably near Foreness Point, Botany Bay and Hackendown Point. They have also been seen in the cliff sections at Peacehaven and Beachy Head.

E. Fracture controlled rock fall

A type of planar failure has been observed that appears to involve tension only. In this type the unstable mass fails by gravity and the failure may be termed a rock fall. Rock falls have been found to be a common mode of failure of vertical or near vertical chalk cliffs undergoing active sea erosion. The scale of these rock falls is extremely variable. Field work has shown that many of the larger rock falls are controlled by pre-existing fractures. The size of any one rock fall appears to be mainly limited by the occurrence of a pre-existing fracture of large enough area to provide a definite gap between the unstable mass involved in the rock fall and the relatively stable rock behind.

A fracture plane controlling a large rock fall of this type near Newhaven, Sussex, is shown in Plate IV. The vertical cliffs at this locality are some 30 m. high. The backward extent of the rock fall has been limited by an extensive fracture plane dipping 86°3 with a strike of N85°W. The fracture plane is inclined slightly seawards. The strike of the fracture plane coincides with the trend of the coastline. The upper part of the cliff is overhanging and is formed of blocky chalk. When visited this overhanging part of the cliff appeared unstable. The fracture plane belongs to a set of fractures
Plate IV. Fracture controlled rock fall (Type 1E) at Newhaven, Sussex
that are well-developed in this area with a mean strike of N67°W. and a mean dip of 82°S. (see Section 7.4.4). The fracture plane is smooth and in places covered with limonite. Some striations vertically orientated occur on the surface. However, these appear to be 'true' slickensides rather than markings caused by recent sliding of a failing mass of rock. No irregularities of the plane occur due to the bedding of flint bands indicating that the fracture is a structural fracture or joint rather than a plane formed recently by shearing of the chalk during failure of an unstable mass of rock. Laterally the rock falls are terminated by fractures trending at or near right angles to the orientation of the coastline.

A second example of extensive fracture planes controlling rock falls has been examined near Peacehaven, Sussex. At this locality the cliffs are about 30 m. high. Prominent fracture planes are orientated typically with a dip of 74°W. and a strike of N30°W. The coastline trends N67°W. Where the fractures intersect the cliffs, extensive planes are exposed as shown in Plate V. The planes extend uniformly from the base of the cliff almost to the top. Near the top the chalk is loose and very fractured and the planes are less prominent. The fracture surfaces are extensively stained with yellow limonite. Each plane forms the backward termination of a rock fall. The rock fall progresses between two consecutive parallel planes as erosion proceeds. No indications of sliding along these fractures were found.

E. Irregular rock fall

At some localities rock falls have been observed that are apparently unrelated to pre-existing fracture planes. Such rock falls are normally
Plate V. Fracture controlled rock fall (Type IE) at Peacehaven, Sussex
of much smaller size than the fracture controlled rock falls, and they take place mainly in blocky or rubbly chalk that is loose and weathered. The more intensive fracturing of the chalk, presumably produced by weathering processes, means that when failure occurs, for example by the sea undercutting the cliff, it is not controlled greatly by the major fracture planes. Small rock falls of this type have been noted at several localities e.g. Botany Bay, Thanet, Freshwater Bay (Isle of Wight) and at Weybourne, Norfolk.

2. Wedge failures

Failures and potential failures of masses of chalk have been observed where the controlling factor is not just one single plane but two or more inclined planes. These wedge type failures are much less common than the planar failures. They usually occur where two existing inclined fracture planes intersect to give a mass of rock that with erosion of the cliff face becomes unstable and moves forwards.

G. Wedge sliding

A potential wedge sliding failure has been examined at East Cliff, Ramsgate. The cliff is cut mainly in Upper Chalk of the Micraster coranguinum zone. At this locality the cliff is about 30 m. high and is aligned N28°E. Two prominent fracture planes intersect in the cliff as shown in Plate VI. One fracture was found to have a dip of 61°N. and a strike of N62°W., and the other fracture a dip of 61°N. and a strike of N40°W. The fractures extended upwards nearly to the cliff top. The fractures had smooth surfaces, but that with
Plate VI. Wedge sliding failure (Type 2G) at East Cliff, Ramsgate.
a strike of \( N82^\circ W \) had a stepped appearance with each step occurring at a bedding plane. The two fracture planes had relative displacements of up to 0.1 m., and may perhaps therefore strictly be termed faults rather than joints. Some ridges orientated vertically downwards are visible on the fracture surface striking \( N82^\circ W \), suggesting that movement occurred mainly along that plane. A plan of the potential failure is shown in Fig. 20.

2. Inverted wedge removal

As already mentioned at some localities the sea has eroded the foot of cliffs along a single fracture plane inclined landwards. Cases have also been noted where the sea has eroded the foot of a cliff along two intersecting planes. The mass of rock initially removed is defined by the two planes. The result may be described as inverted wedge removal.

An example of this mode of failure has been examined at Peacehaven. Prominent fracture planes occur in the lower part of the cliffs. The upper 7 - 10 m. of the 30 m. high cliffs is composed of softer loose chalk and the main fracture planes become less prominent. The two intersecting fracture planes have strikes of \( N21^\circ W \) and \( N51^\circ E \), and dips of 75\(^\circ\) and 65\(^\circ\), respectively. The surface of the fracture with a strike of \( N21^\circ W \) is smooth and covered by limonite. The fracture with a strike of \( N51^\circ E \) has less limonite on its surface and some broken flint occurs along the plane indicating that at some time movement may have occurred. A plan of the intersecting fractures is given in Fig. 21. Sea erosion appears to be gradually removing the wedge shaped mass of rock formed by the intersecting fractures.
Fig. 20. Plan and section of potential wedge failure at East Cliff, Ramsgate.
Fig. 21. Plan and section of inverted wedge failure at Peacehaven.
3. Complex failures

A more complex type of fracture controlled failure has been observed at some localities. Instead of involving one fracture plane as in planar sliding or two fracture planes as in wedge failures, these more complex failures involve control by more than two existing fracture planes.

I. Complex sliding

One type of complex failure takes place by sliding. The rock masses involved in these failures are of irregular shape, as opposed to the usual cuboidal blocks formed by block sliding failure and the wedge-shaped masses formed by wedge sliding failure. The sliding process may occur along several fracture planes at any one time.

A fairly simple example of such a multi-fracture controlled failure has been examined at Whitecliff Bay (Isle of Wight). At this locality the bedding planes dip 65°N. and strike N76°W. The high angle of dip allows the bedding planes to be significant fractures affecting the stability of the cliff. The strike of the bedding is close to that of the coastline which is N55°W. Three main existing fractures sets control the shape of the rock masses involved in the failures. A sketch of a typical failure examined at this locality is shown in Fig. 22. Two fractures limit the extent of the rock mass laterally. These fractures have strikes of N16°E. and N60°E., with dips of 90° and 65°, respectively. These fractures also limit the backward extent of the mass. The vertical extent of the mass is controlled by the distance between the bedding planes dipping at
Fig. 22. Detail of potential complex sliding failure in cliffs at Whitecliff Bay, Isle of Wight.
At this locality these bedding planes are 0.2 - 0.3 m. apart. The mass so determined fails by sliding on the lower limiting bedding plane surface. The sliding in this particular failure although taking place mainly down the bedding plane, might also partially occur on the lateral limiting fracture plane striking N16°E., which would tend to divert the block slightly eastwards. The degree of control exerted by individual fracture planes clearly depends on their relative orientations.

The bedding plane involved in this potential failure is nearly smooth and is partly coated with limonite. Lower down the exposed bedding plane a black coating occurs. This is thought possibly to be salt. The fracture striking N16°E. is also nearly smooth and coated with limonite. Some vegetation, mainly grass and mosses, occurs along the surface trace of the fractures striking N60°E. Complex fracture controlled failures of a similar nature have been observed in the structurally complex chalk of Man o’ War Cove and Durdle Cove in Dorset.

A. Superficial failures

J. Circular slip

At a few localities circular slips have been observed but they have been confined to weathered and loose chalk and solifluction chalk. These slips encountered have been of only small magnitude and have affected only the loose chalk at the top of the cliffs. The maximum height of cliff involved in those slips observed has been about 1.5 m. The failure planes have been circular but steeply inclined backwards so that only a relatively small mass of chalk is
involved in the failure. Such circular failures have been observed in rubbly chalk which has been produced by the movement of normally bedded chalk in large rotational slips at White Nothe in Dorset. Other failures of this type have been observed in poorly graded solifluction chalk at Freshwater Bay, Isle of Wight and at Peacehaven, Sussex.

K. Mudflow

Mudflows of chalk have been noted at a few localities. Like the circular slips they only involve weathered loose chalk or solifluction chalk or chalk that has already been reduced to small particles by mass movement. Those mudflows observed have covered only relatively small areas. Minor mudflows of this type have been observed at Dumpton Gap, Thanet.

The latter two types of failure (types J and K) have not been examined in detail by the author. The condition of the chalk involved in these failures is such that they may be better considered as soil slopes rather than rock slopes. Nevertheless the two types are mentioned for completeness.

5. Miscellaneous failures

L. Swallow hole controlled

The shape of cliffs undergoing coastal retreat is thought to have been determined at a few localities by the occurrence of old swallow holes. Inlets, roughly circular in plan and about 7 - 10 m. in diameter, have been eroded out by the sea. Their occurrence is not readily explained by the orientation of fracture planes. However, the chalk surrounding these inlets has been found to have fractures that are
very open and the chalk is closely fractured and blocky. The fracturing has been found to decrease rapidly away from the inlet. The author considers that these may be the sites of old swallow holes or the open-fractured chalk immediately below the 'hole' itself through which water percolated downwards, thus increasing the openness of the fracturing. These features have been observed particularly well at North Foreland, Thanet.

Another rather larger circular inlet has been noted on the east side of Freshwater Bay (Isle of Wight). Again the chalk surrounding the inlet has close fracturing, and limonite staining occurs down to the base of the cliff which, at this locality, is some 25 m. high. Not enough examples of these features have been studied to confirm that they give rise to 'swallow hole controlled' failures of steep chalk slopes. However, the possibility is thought to exist.

5.3.2 Coastal slopes not undergoing active erosion

Many cliff sections of chalk in south-east England are not now subject to marine erosion as a consequence of various sea defence works. Some of these cliffs have been artificially graded to a supposedly safe angle of slope. These protected coastal slopes have been examined to investigate the manner in which degradation is proceeding, and to determine if any of these slopes appear potentially unstable.

Most of the observed faces exhibited chalk that appeared much more fractured than that in the cliffs being actively eroded. The
effect of the increased fracturing is to produce loose chalk fragments at the surface. As a consequence of the increased fracturing the main regional fracture sets are much less readily apparent in these sections than in those undergoing active erosion. Although fracturing tends to increase with age, some faces that have not undergone erosion by the sea for many years have been found to show only a limited development of this secondary fracturing. Fracture spacing in the Chalk is dealt with more fully in Chapter 6.

The majority of protected cliff faces examined appeared stable, although in some cases they remained vertical or near vertical, and nearby unprotected cliffs in similar types of chalk were undergoing failures of one or more of the types described in Section 5.3.1. Most planar failures observed in actively eroded chalk cliffs have been along existing fractures planes dipping at angles of 80-90°. Indeed, the majority of all measured fractures have been found to have dip angles in excess of 80°, and most fracture sets have mean dip angles of 90° (see Chapter 7).

In protected faces that are graded back from the vertical, for example to 70° at Saltdean, Sussex, failures along any steeply inclined fractures is unlikely. In protected faces that are vertical the increased fracturing presumably due to weathering, tends to cause loose fragments to fall off the exposed surfaces. As this process continues, the face tends to be slowly graded back from the vertical. More material tends to fall from the upper part of the cliff than the lower part so making the slope inclined backwards.
Although artificial grading and natural grading tend to make large failures unlikely in these protected faces, some cases of potential instability have been observed. At Dover, behind the Cross-Channel Car Ferry Terminal, is a chalk slope that has been protected for many years. Muir Wood (1971) notes that the cliffs at Dover have been protected from the sea for about 400 years, and in places have weathered back less than a metre to an angle of about 80°. The slope at Dover investigated by the author is some 70 m. high and is in hard chalk. The orientation of the slope is N40°E, and it is inclined at an angle of 65°. The slope is partly vegetated with grass, shrubs and small trees, but appears rather unstable since some apparently loose blocks and slipped masses of chalk occur. At one place a fault is present with a strike of N36°E, similar to the trend of the slope, and a dip of 55°S. Slickensides occur on the surface of the fault plane. This plane is considered to be one along which a translational sliding failure could possibly take place. A joint with a strike of N20°E, and a dip of 55°S., and hence a similar orientation to that of the fault, was also observed at this locality.

In a protected cliff at Westbrook Bay, Margate, steeply inclined fracture planes striking nearly parallel with the coast have been observed. The orientation of the coast is N60°W., and a typical potential failure plane has a strike of N37°W, and a dip of 63°N. The cliffs are near vertical and about 10 m. high. If active sea erosion was taking place failure would probably be by fracture controlled rock fall. However, in this protected slope triggering of failure by erosion of the foot of the cliff cannot occur. This
slope is therefore presumably stable provided no other factors are
activated that might reduce stability and 'trigger' a failure.
Similar steeply dipping fracture planes aligned parallel with the
coast, and which may be potential failure planes, have been observed
in protected faces at Western Undercliff, Ramsgate, and Eastern
Esplanade, Ramsgate.

On the west side of St. Mildred's Bay, Margate, a protected
cliff face has been examined that shows two intersecting planes. The
planes appear to be those along which a wedge failure has occurred,
or along which a wedge-shaped mass of rock has been removed during
construction of the sea-wall. At this location the nearly vertical
cliff is orientated N50°W., and its height is about 10m. The
fracture planes involved strike N27°W. and N65°W., and have dips of
69°N. and 67°N. respectively. A plan of this probable wedge failure
is shown in Fig. 23.

Few of the recently protected slopes support vegetation. Slopes
steeper than about 65° do not normally appear to become vegetated
apart from occasional mosses and lichens. Grasses and small shrubs
have been observed growing in fractures filled with rock fragments
and clay on slopes of about 65-70°.

In protected cliffs west of Newhaven Harbour chalk is overlain
by some 7 m. of Thanet Sands. The chalk forms a cliff inclined at
about 40-50°. The sands are sliding and being washed from the top
part of the cliff to form debris piled up against the chalk at the
base of the cliff. Hence, the old cliff of chalk is becoming buried
beneath the softer debris.
Fig. 23. Plan and section of wedge sliding failure at St. Mildred's Bay, Margate.
5.4 Artificially excavated slopes

Many steep slopes have been examined in quarries and road cuttings. The form of these artificially excavated slopes has often been found to differ from that of natural coastal slopes. Usually some degree of stability is required for artificial slopes. Only short-term stability is necessary for most working faces of quarries, whereas long-term stability is required for road cuttings. Differences also derive from the methods of excavation employed.

Many faces examined have appeared stable. A major factor affecting the stability has been found to be the strike and dip of existing fracture planes with respect to the orientation of the face. Some examples will be described which illustrate the variation in the stability of the artificially excavated slopes investigated.

At Redbourn, Hertfordshire, a quarry that has been abandoned for many years shows a 10 m. high vertical face of chalk, with an additional lower 10 m. hidden by a scree of loose chalk. The main face of chalk appears stable. The stability of the face is attributed to the absence of any fracture planes orientated adversely with respect to the face. One set has a mean strike of N35°W., similar to that of the face, and with a mean dip of 90°. Two other fracture sets are orientated almost at right angles to one another. They have mean strikes of N32°E. and N67°W., and mean dips of 81° and 83°, respectively. No fracture planes are therefore inclined towards the face in such a way as to give major joint controlled failures. The dominant rectangular fracture pattern together with the near horizontal bedding planes, tend to
give cuboidal blocks of chalk. The sides of the cubes are normally 0.5 - 1.0 m. in length.

In another part of the same quarry a failure occurred in a 5 m. high face of chalk. Numerous tree-roots were present extending down the failure plane and they appear to have contributed to failure of the slope. The failure plane conforms to an existing joint plane with a strike similar to that of the chalk face and with a dip of 80° towards the face.

Examination of the joint plane adjacent to the failure showed that it was very open with an average width of 50-100 mm. More tree roots could be seen extending down the fracture. The presence of roots would allow water to readily percolate down the fracture. The failure is believed to have occurred in January - February 1974, a period of high rainfall in this area.

The failure is of the fracture controlled rock fall type. The failed chalk forms a pile of blocks at the base of the slope. The largest blocks found were about 200 mm. x 100 mm. x 150 mm. The chalk involved in the failure, when in place, did not extend more than about 1.0 m. back from the face, and was therefore already blocky and partly weathered.

At a quarry at Pitstone, Bucks., the upper part of the Lower Chalk and the lower part of the Middle Chalk are exposed. At the top of the Lower Chalk the plenus Marls are well-developed. In some parts of the quarry the plenus Marls occur as two distinct marl bands each about 0.15 m. thick, separated by a band of harder and less fragmented chalk about 0.6 m. thick. A number of sets
of fracture planes have been recognised at this locality. Two
dominant fracture sets are orientated with respect to one
another and to one of the faces of the quarry, in such a way as
to give frequent wedge type failures. From measurement of two
such fracture planes typical strikes of N62°E. and N20°W. were
obtained, with dips of 72°S. and 66°E. respectively. The trend of
the face is N35°E. Figure 24 shows a plan of the resulting wedge
failure.

The wedge failures affect chalk both above and below the
level of the plenus Marls. Most of the surfaces of the fracture
planes involved exhibit slickensides. The slickensides tend to
be aligned in the direction of maximum dip suggesting that
movement may have occurred along these planes at some time in
geological history. The face is some 7-10 m. high and the wedge
failures generally involved chalk to a depth of 3-5 m. below the
surface, but failures involving the whole height of the face have
been observed occasionally.

Observations at a quarry at Ashwell, Hertfordshire, have
revealed the occurrence of wedge type failures. These differ
from the wedge failures recorded at Pitstone in that they occur
along existing fracture planes that intersect at right angles,
and the planes usually have much steeper angles of dip. Typical
fracture planes involved have strikes of N45°E. and N45°W. with
dips of 74°W. and 65°W. respectively. The wedge failure
involving these fractures is shown in Plate VII.
Plate VII. Wedge sliding failure (Type 2G) at Ashwell, Hertfordshire.
Fig. 24. Plan and section of potential wedge sliding failure at Pitstone, Bucks.
At this locality fractures have also been recorded that do not contribute to an unstable situation. This is because of their orientation with respect to that of the face. The fractures form two intersecting sets. Typical strikes are N18°E. and N45°E. with dips of 74°E. and 84°W. respectively. The situation at the quarry at Ashwell is as shown in Fig. 25a. If, however, the face of the quarry had been orientated at N45°W., these fracture-planes would intersect in such a way as to make a wedge type failure likely (Fig. 25b).

In November, 1971 a cutting in chalk on the M4 Motorway was inspected prior to the opening of the Motorway to traffic. The cutting to the south-east of Swindon exposes chalk of three main lithologies. Chalk with well-spaced fractures is overlain by a prominent marl band about 0.2-0.3 m. thick, and this is overlain by white chalk with numerous fractures imparting a blocky nature to it. The cutting has been constructed with a vertical face of chalk 4 m. high. Above the face, the cutting is grassed and inclined at about 60° for about another 3 m.

Measurements were made of fracture planes mainly in the northern face of the cutting which appeared to be the least stable. Some of the fractures were found to have much lower dip angles than those recorded at many localities. Dip angles as low as 50° were measured. When considered in relation to the trend of the face of the cutting, two types of failure were considered to be possible. Planar translational sliding failures were thought to be possible along fracture planes that strike in directions similar to that of the cutting. The cutting is orientated approximately
Fig. 25. Effect of dips of intersecting fractures and orientation of slope on stability.
H75°W. and the fracture planes along which failure could occur have strikes of, for example, N66°W. and N85°W., with dips of 65°S. and 50°S. respectively. Wedge sliding failures were thought to be possible along intersecting fracture planes inclined towards the excavation. Typical strikes for these planes are N26°W. and N40°E. with dips of 55°W. and 80°E.

When the face of the cutting was seen in March, 1973, wedge failures had occurred. The failures appeared to have been of the expected type. Most were relatively minor involving only the upper part of the vertical face of exposed chalk. Potential wedge failures have also been observed during excavation of a slope at Lewes, Sussex.

One kilometre east of Royston, Hertfordshire, the A505 road, passes through a cutting in chalk. Although chalk is exposed in places, the cutting is well vegetated with side slopes mainly of 30-40°. At some time between 1914 and 1920 a landslide occurred which blocked the original cutting that existed along this road (Crow, 1972). The cutting at that time was much narrower and the sides steeper. The landslide originated from the northern side of the cutting. Measurements of fracture planes were made in the chalk exposed at this locality. However, the fracture planes do not appear to be orientated in such a way as to give rise to instability. The difficulty of making reliable measurements in the poor exposures at this locality may account for no relationship apparently existing between fracture planes and the known failure. Little documentary evidence for similar failures involving inland
chalk slopes has been found by the author.

At localities where 'pipes' with unconsolidated materials extending down into the chalk are exposed in steep slopes, the unconsolidated materials usually slip down as they tend to degrade more readily than the chalk. Although not strictly failures of chalk these 'pipes' are features so closely related to the chalk that they are considered worthy of examination also. The effect of the slips is to produce much more irregular faces than might otherwise exist. The pipes may also provide channels along which water may permeate more readily especially if the infilling is sand and/or gravel. Examples of these 'pipe' failures have been observed at South Mimms, Hertfordshire, and Lenham, Kent.

5.5 Discussion

The investigations of natural inland chalk slopes which appear to have been stable for very many years indicated that a slope angle of 38° is the upper limit for stability under present climatic conditions in Britain. Even slopes at this angle may show signs of instability such as terracettes and small hummocky slips. The exact upper limit for the angle of stability probably varies somewhat depending on the thickness of soil on the slope, extent and type of vegetation cover, geographical position and microclimate. All chalk slopes steeper than 38° must therefore be regarded as potentially unstable.

The examination of the coastal chalk cliffs showed that many of them are actively failing, or are unstable or potentially unstable.
The dominant factor controlling failure of these cliffs was found to be the fractures present within the chalk. As a consequence a field investigation of the fractures present in the chalk was undertaken. The results are described in detail in Chapter 7.

The most common method of failure of the vertical or near-vertical cliffs was found to be planar failure. These failures involved movement of a mass of rock controlled by a single failure plane. Laterally the failure would be limited by other pre-existing planes, or, in some cases, fracture of the chalk mass. The field evidence indicated that most of these planar failures occurred mainly along pre-existing fractures. A decisive factor in determining the type of planar failure to occur was found to be the angle of inclination of the pre-existing fracture plane that is to act as the failure surface. Where these planes are inclined at angles of up to 80-85° to the coast, sliding is dominant, but when the planes are inclined at angles of 80-90° to the coast, rock fall is dominant. Most fracture sets in the chalk have been found to have mean dip angles of 80-90° (Section 7.5) and this would seem to account for the frequent occurrence of these steep planar failures in chalk cliffs.

In Kent and Sussex the bedding planes dip at angles usually of less than 10°. As a consequence, they do not exert significant control on the failure of chalk cliffs. However, in the Isle of Wight the bedding planes are inclined at angles of 65-70°, and they are therefore able to exert significant control on the types of failure that occur. The joint planes are usually not vertical.
as they frequently are in Kent and Sussex, but instead are commonly inclined at angles of 25-70°. As a result they also tend to exert a different pattern of control on failures.

Since many of the dominant fractures in the Isle of Wight are inclined at angles of 25-70°, translational and block sliding failures occur quite frequently. At Whitecliff Bay and Alum Bay where the trend of the coast is almost parallel with that of the strike of the bedding, these sliding failures occur along the bedding planes which dip towards the coast at angles of 65-70°. On the south side of Culver Down sliding failures take place not along the bedding planes but along joint planes typically dipping seawards at 30-50°. Failures occurring along such inclined planes have also been observed in Man o' War Cove and Durdle Cove in Dorset.

The distinction between areas of relatively simple geological structure and areas of more complex structure is clearly very important when considering the types of instability likely to occur.

Of the types of failure recognised by the author, two are limited to slopes which are being actively eroded at the base. These two types of failure are inverted block removal and inverted wedge removal. Removal of rock in this way often produces instability in the remainder of the cliff which may then undergo one of the other modes of failure.

The tension-shear failure which has been recognised by the author is believed to differ fundamentally from the other main types of failure described. Whereas those failures are largely controlled by pre-existing fractures, the tension-shear failure types of failure described. Whereas those failures are largely involves fracture of the intact chalk, although little was suggested
that initially the position of these failures may be influenced by pre-existing fractures, there is adequate evidence to indicate that the shear surface develops across intact chalk. More detailed analysis of these failures involves consideration of the shear strength of the intact chalk, whereas analysis of the other types of failure recognised usually requires investigation of the shear strength of the pre-existing fractures along which movement takes place. A study of fracture surfaces in the Chalk is described in Chapter 9.

In those areas in which the structural geology is relatively simple (and the dip of the bedding is less than $25^\circ$), the most common modes of failure have been found to be planar and wedge failures involving control by existing fractures inclined at angles normally exceeding $65^\circ$ and usually greater than $80^\circ$. In the tension-shear failures examined, the lower part of the failure plane, which is considered to have been the product of shear failure of the intact chalk, has been found to be inclined at an angle of $65^\circ-75^\circ$. If $65^\circ$ is taken as the minimum value for shear failure of the intact chalk, and few failures have been encountered involving existing fractures with dips of less than $65^\circ$, that angle would seem to be the normal lower limit for major instability in chalk slopes.

Vegetation has been observed on slopes as steep as $65^\circ$. At Dover a slope inclined at that angle, although appearing rather unstable, is partly vegetated with grass, shrubs and small trees.
Slopes steeper than about 65° do not appear to support vegetation except for occasional mosses and lichens.

Slopes which have been reduced by major movement to angles of less than 65° would therefore seem subsequently to suffer only minor instability, perhaps resulting from frost action, with a gradual reduction in the angle of slope. Although the study of natural chalk slopes indicated that the upper limit for stability is about 36°, this may be somewhat misleading since these slopes were formed in part under climatic conditions different from the present conditions. Many of these slopes were initiated or transformed under glacial and/or wetter climates than those of the present. Such relatively severe climatic conditions probably caused more instability in slopes than occurs today. The numerous solifluction deposits in chalk areas is an example of this. Chalk slopes steeper than 36° may therefore perhaps be stable under present climatic conditions, and possibly even slopes as steep as 65°. Certainly, many quarry faces and cliff faces protected from sea erosion appear quite stable, even when vertical. These vertical faces are invariably in chalk that is cut by vertical joints only and in which the bedding is horizontal or nearly so.

In areas in which the structural geology of the Chalk is more complex and the bedding dips at angles of more than 25° such as the Isle of Wight and Dorset, the modes of failure have been found to be more varied. This variation reflects the control exerted by the fracture sets which are generally inclined at angles of 25-70°. The occurrence of prominent continuous fractures dipping
at about 30-50°. leads to frequent failures along these planes in south facing coastal slopes. Thus the normal lower limit for major instability of 65° recognised in areas of simple structural geology is not valid for these more complex areas.

The author suggests that a critical factor may be the angle of dip of the bedding planes (Fig. 26). Where the angle of dip of the bedding does not exceed 25° failure by sliding or rock fall is extremely unlikely to take place along these planes. The lowest angle noted for a failure plane by the author is 30°. If the angle of dip of the bedding is 25° then most fracture planes will be orientated at 65°, assuming that most joint fractures are developed perpendicular to the bedding as has been shown by field work (Chapter 7). As has already been suggested 65° is probably the lower limit for major instability in areas of simple geology. For areas with a dip of bedding of 25° or less slope angles of up to 65° are therefore likely to be relatively safe.

Where the angle of dip of the bedding exceeds 25° then failure is possible along the bedding planes themselves. In addition, the joint fractures will be orientated at angles of less than 65°, again assuming that most joint fractures are developed perpendicular to the bedding. Failure by sliding will then be possible along these joint planes as well as by sliding along the bedding planes, although the exact nature of the movement will be controlled by such factors as the direction of the planes relative to the slope, and the nature and continuity of the fractures. For areas with a dip of bedding exceeding 25° only slope angles of 25° or less are likely to be stable in the long-term.
Vertical slope stable where bedding dips towards face at <25° and joints inclined inwards.

Slope stable at angle of up to 65° where bedding dips inwards away from face at >25° and joints inclined towards face.

Where bedding dips towards face at >25° stable slope angle coincides with that of bedding.

Where bedding dips inwards away from face at >25° and joints towards face, stable slope angle coincides with that of joints.

Fig. 26. Effect of bedding and jointing on stability.
5.6 Conclusions

(a) The upper limit for the long-term stability of natural chalk slopes is normally about 36°.

(b) The following types of failure may be recognised:

1. Planar failures
   A. Translational sliding
   B. Block sliding
   C. Inverted block removal
   D. Tension-shear
   E. Fracture controlled rock fall
   F. Irregular rock fall

2. Wedge failures
   G. Wedge sliding
   H. Inverted wedge removal

3. Complex failures
   I. Complex sliding

4. Superficial failures
   J. Circular slip
   K. Mudflow

5. Miscellaneous failures
   L. Swallow hole controlled.

(c) The dominant factor controlling failure of steep chalk slopes is the orientation of pre-existing fractures.

(d) Where failure planes are inclined at angles of up to 80-85° sliding is dominant.

(e) Where failure planes are inclined at angles of 80-90° rock fall is dominant.

(f) Tension-shear failures involve shear failure of the intact chalk.

(g) Where the dip of the bedding is less than 25° in areas of simple geological structure, slopes inclined up to 65° may
be relatively stable.

(h) Where the dip of the bedding exceeds 25° in areas of complex geological structure, all slopes steeper than 25° may be potentially unstable.
6. MOVEMENTS IN EXCAVATED SLOPES

6.1 Introduction

The examination of literature on rock slope stability revealed that no attempts appear to have been made to investigate in detail what movements occur at the faces of excavations in rock. To investigate the extent of stress relief in new excavations and the possible movements that might occur in steep slopes, in situ measurements were attempted.

The decision was taken to excavate two trial pits; one being in a shallow slope where movements might be expected to be small; and the other in a steep slope where movements might be greater. Much difficulty was encountered in selecting suitable sites and in obtaining permission from landowners to excavate the trial pits. Eventually two sites were settled: one at South Mimms, Hertfordshire; the other at Pegsdon, Bedfordshire.

Satisfactory data was obtained from the trial pit at South Mimms mainly because it was sited on private land. However, only limited data was obtained from the trial pit at Pegsdon. The pit was located near a public right of way and on several occasions it was 'attacked' by vandals.

6.2 Description of the sites

6.2.1 South Mimms

The site selected at South Mimms was near to the Castle Lime Works Quarry (TL 228027). Topographically the site is located at a height of about 100 metres on the west side of the shallow valley of the Mimms Hall Brook. The geology and hydrogeology of the Mimms Hall Valley have been described by Wooldridge and Kirkaldy (1937).
The quarry exposes varying thicknesses of brown sand and reddish brown sandy clays of the Reading Beds overlying Upper Chalk. The thickness of the Reading Beds is generally 1 - 3 metres. The base of the Reading Beds is marked by the Bull Head Bed consisting of glauconite stained flints. The upper surface of the chalk is very irregular and in places is broken by pipes. The pipes contain various mixtures of clay, sand and gravel. They often extend to a depth of 10 m, or more below the upper surface of chalk. Isolated lenses and fracture infillings of sand and/or clay also occur at intervals throughout the exposed face of chalk. The chalk at this locality has been referred to the lower part of the zone of Miroaster cornuaquium (Kirkaldy, 1950). The chalk is exposed to about 15 m, in a vertical working face.

The pit to be monitored was excavated approximately 50 metres north of the working face of the quarry on land sloping at 2-3° eastwards. The possibility of the movements in the trial pit, and particularly the south face of the pit, being influenced by the much larger excavation of the quarry was recognised. However, as mentioned in Section 6.1, the choice of sites was limited. The nearby quarry did have the advantage of allowing detailed examination of the chalk present in the area.

6.2.2 Pegsdon

The site chosen at Pegsdon was in the Pegsdon Valley at a point about 1 km south of the village (TL294119). The Pegsdon Valley is a dry valley cut in the chalk scarp. The valley has
steep side slopes and also a steep southern termination. The floor of the valley is flat. Sparks and Lewis (1957) have shown that this is caused by infilling. They found that the infill consists of a layer of chalk rubble, up to 2 m. thick, overlain by a light brown rubbly loam, also up to 2 m. thick. The chalk rubble consists of rounded pellets and angular fragments of chalk in a matrix of finely divided chalk. It overlies weathered chalk grading down into solid chalk. The light brown rubbly loam consists of topsoil, chalk fragments and glacial erratics. Near the base of the loam is a darker layer with molluscs, and above that a pebble band. Sparks and Lewis suggest that the flatness of the valley floor may be due in part to cultivation, as the land was ploughed during the 1914-1918 war. The ploughing would also tend to cut into the bottom of the waste mantle on the side slopes causing instability and downhill creep. Small scars do occur on the valley sides at present.

The trial pit was excavated on the eastern side of the valley near to where the valley changes direction from being aligned N.N.W. - S.S.E. to being aligned N.N.E. - S.S.W. The pit was sited approximately 8 m. above the floor of the valley, and at an altitude of about 140 m. A slope of 26° was measured along the valley side.

6.3 Excavation of the trial pits

The trial pits were hand dug. Although intended to be circular in plan, this was found to be difficult to achieve in
practice, and consequently square pits were excavated. In plan both pits were 2 metres square. The walls of the pits were vertical, as measured with a plumb line. The depth of the pit at South Mimms was 2 m. The pit at Pegsdon was excavated to a depth of 1.5 m., as measured on the upper side of the pit. Excavation to depths greater than these would only have been possible with great difficulty and with the expenditure of a vast amount of time and effort.

At both sites wooden lids were provided. At South Mimms rabbits were retrieved from the pit on two occasions and so the lid was given a covering of polythene.

On completion of measurements the trial pit at Pegsdon was filled with the chalk that had been extracted. The top soil and tussocks of grass were also carefully replaced. On excavation the chalk and top soil had been banked up on the hillside with some difficulty, ready for later replacement. The trial pit at South Mimms is to be filled in the near future.

6.4 Geological description of the trial pits
6.4.1 South Mimms

The faces of the pit showed 0.3 m. of rubbly chalk at the top. This consisted of pieces of chalk, averaging 25 mm. in diameter, set in a finer matrix of chalk, sand, clay and flint pebbles. Two cavities were also present in this rubbly chalk. They were about 150 mm. across, 100 mm. high and 150 mm. in depth. One was in the west wall and the other in the south wall. They
are presumably rabbit burrows, or have resulted from solution of the chalk.

Below the rubbly layer the chalk was white and blocky. Most blocks had sides of about 50 - 75 mm. Most of the fractures separating the blocks were closed, although a few were just open, up to a maximum of 1 mm. This close fracturing decreased downwards in the pit faces.

Although the chalk was fractured some more prominent and extensive joint planes were observed. These became more prominent towards the bottom of the pit. Below a depth of about 1.5 m. the fractures in the pit walls were mainly of this type and were some 0.15 - 0.3 m. apart. The orientation of these primary fractures at this locality have been measured and the results are given in Section 7.4.1.

6.4.2 Pegadon

The trial pit at Pegadon exposed 0.1 m. of top soil overlying 0.15 m. of chalky soil. Beneath blocky white chalk was present. The blocks were about 0.1 m. in length in the direction parallel to the bedding planes, and about 30 mm. in height. The blocks tended to interlock with one another. Some fractures were open up to 2 mm. The intensity of fracturing decreased downwards in the faces of the pit. Few prominent joints like those observed in the trial pit at South Mimms, were present.

6.5 Instrumentation

A number of methods were considered for measuring the possible movements of the walls of the trial pits. These included the use...
of electrical resistance strain gauges, stretched wires and transducers. An attempt was made to attach electrical resistance strain gauges to freshly excavated faces of chalk. The wet nature of the chalk made fixing of the strain gauges difficult. Using a blow lamp small areas of chalk were dried sufficiently to allow fixing with an epoxy resin, but, because of the difficulty of using a blow lamp in the field for drying out the chalk and for soldering, the use of strain gauges was subsequently abandoned.

The use of stretched wires and transducers was discounted because of cost, and also because these instruments would have to remain in the pits and could be easily tampered with by vandals. The latter contention was proved correct at least in the case of the trial pit at Pegsdon.

The method eventually adopted was that of measurement between markers in the excavated faces of the pits using a removable rod. The rod consisted of an alloy tube with a dial gauge fixed at one end, and a short rod shaped to a point at the other end (Plate VIII and Figure 27). Nails, 150 mm. long, previously having had their heads punched, were driven into the sides of the pits at regular intervals. The nails were placed in vertical rows with one row of nails in each face. The nails were 150 mm. apart (Figure 28).

Measurements were made between pairs of nails on opposite sides of each pit (Plate IX). These measurements were carried out by placing the point of the dial gauge on one nail and the point at the other end of the rod on an opposing nail. Then the dial gauge reading was recorded. This was repeated for each opposing set of nails.
Plate VIII. Instrument used for measurements at trial pits.
Part A  Aluminium alloy tube (38 mm diameter, 1 mm thick tubing).
Part B  Mild steel bar (13 mm diameter with welded stud for dial gauge).
Part C  Mild steel bar (7 mm square section with end shaped to a point).
Part D  Dial gauge (50 mm travel, reading to 0.025 mm).

Fig. 27. Detail of Measuring Instrument.
Fig. 23. Layout of reference nails at trial pit at South Mimms.
Plate IX. Instrument in use at trial pit at South Mimms, Hertfordshire.
At each time of recording the air temperature was measured to allow for relative expansion or contraction of the alloy tube. The coefficient of linear thermal expansion of the alloy tube was found by comparing measurements of the tube at different temperatures. The coefficient of linear thermal expansion was found to be $22 \times 10^{-6} / ^\circ C$. This value compares well with that given for aluminium in standard reference works. For each series of readings with the instrument a correction for temperature was therefore applied. The correction applied for a difference in temperature of $1^\circ C$ was $31.75 \times 10^{-3}$ mm./$^\circ C$.

A temperature correction was not applied for the mild steel bar that composed the pointer and the attachment for the dial gauge at the ends of the alloy tube. However, the error introduced is believed to be small. The total length of mild steel bar used in the measuring instrument is 256 mm. If the coefficient of linear thermal expansion of the mild steel is taken as $11 \times 10^{-6} / ^\circ C$ (Thurston, 1951) the correction for the largest temperature difference experienced, that of $11.8^\circ C$, would be only $3 \times 10^{-2}$ mm. A temperature correction was not applied for the nails.

6.6 Results of the measurements at the trial pits at South Mimas

6.6.1 Introduction

The trial pit was excavated on the 8th and 9th October, 1971. Measurements were commenced immediately after placement of the nails following completion of the pit on 9th October. Readings were taken at weekly intervals until 29th November, 1971, and then at less frequent intervals until 29th January, 1974.
The movements recorded over this period for each wall of the pit are plotted in Figures 29 and 30. The movement measured between each pair of nails has been attributed to movement of both the opposing walls, although clearly one wall may have moved relatively more than the other. The method of measurement employed did not allow this to be determined. The total movement recorded between any pair of nails was therefore divided equally into two and each part of the movement was then attributed to a particular wall. Inward movements of a wall are considered positive and outward movements negative.

6.6.2 Observed movements of the north and south walls of the trial pit

The largest movements occurred in the upper part of the pit and were positive. The largest total movement observed during the period was one of \( +434 \times 10^{-2}\text{mm} \), recorded between the uppermost nails, set 1. Much of this movement, \( +198 \times 10^{-2}\text{mm} \), occurred over the first week of observation. This initial positive movement was observed also in sets 2 - 7, but the size of the movement decreased rapidly downwards. Sets 9 - 12 showed a negative movement over the first week. The size of this negative movement increased downwards. For example, for set 12 the movement was \( -45 \times 10^{-2}\text{mm} \).

These initial movements were followed by a negative movement observed for all sets and reaching a minimum on 22nd November, 1971. This movement was generally one of about \( -50 \times 10^{-2}\text{mm} \).

From the latter part of November 1971 until March 1973 a general positive movement was recorded. No record is available for set 1 for
Fig. 29. Recorded movements of East and West Walls of Trial Pit at South Mimms.
Fig 30. Recorded movements of North and South walls of Trial Pit at South Mimms.
Fig. 30(a). Comparison of typical observed movements of walls of trial pit at South Mimms with rainfall and frost records.
the period 19th February, 1972 to 24th July, 1973. During this period a girder was installed above the pit in an attempt to measure vertical displacements of the floor of the pit. The presence of the girder prevented measurement between the nails of set 1. For the remaining sets a high positive value had been achieved by March-May 1973.

This positive movement was then followed by a negative movement in May-July 1973, and then a general positive movement reaching another peak in October 1973. This was followed by a decline in November-December 1973, and a further rise in January-February 1974.

The more frequently the measurements were made then the more erratic the results appear to be. Although a series of these positive and negative movements were recorded, all sets exhibit a general positive trend over the period.

6.6.3 Observed movements of the east and west walls of the trial pit

The general trend of results for the east and west walls is similar to that observed for the north and south walls. The largest movements occurred with the uppermost set of nails, and were positive movements. The largest total positive movement was +255 x 10^{-2} mm, recorded for set 1. As in the case of the north and south walls, a large movement, 92 x 10^{-2} mm., occurred in the first week of observation. During the first week following excavation this initial positive movement was also recorded for sets 2-5, with the size of the movement decreasing rapidly downwards. Set 6 showed a negative movement over the first week, while sets 7 - 12 tended to remain constant.
Unlike the north and south wall movements, the largest initial positive movements were recorded for sets 1 - 5 two or three weeks after excavation of the pit. The initial movements were followed, like the north and south wall movements, by a negative movement observed for all sets and reaching a minimum on 22nd or 29th November, 1971. This movement was generally about \(-50 \times 10^{-2} \text{ mm}\).

As with the north and south walls, a general positive movement was recorded between the latter part of November 1971 and March 1973. No record is available for set 1 for the period 19th February, 1972 to 24th July, 1973, for the same reason as given for the north and south wall set 1 in Section 6.2.2. For the remaining sets this positive movement reached a peak in March-May 1973. This was followed by a negative movement in May-July 1973, and then a general positive movement reaching another peak in October 1973. For most sets this was followed by a negative movement in November 1973 and another positive movement in December 1973-February 1974.

As in the case of the north and south wall movements, the more frequently the measurements were made then the more erratic the movements appear to be. Nevertheless all sets exhibit a general positive trend over the period.

6.7 Results of the measurements at the trial pit at Pegadon

6.7.1 Introduction

The trial pit was excavated on the 25th May, 1972. Due to unforeseen circumstances measurements could not be commenced until 1st June, 1972. This is considered to be a major drawback since any initial movements of the walls of the pit were not measured.
Readings were taken at intervals until 1st March 1973.

The movements recorded over this period for each wall of the pit are plotted in Figure 31. As in the case of the trial pit at South Mimms, the total movement recorded between any pair of nails has been divided equally into two parts and each part of the movement attributed to a particular wall. Inward movements of a wall are considered positive and outward movements negative.

Numerous gaps occur in the records as a result of the attacks on the pit by vandals. With nail set 2 of the north-south walls a new set of nails had to be installed and readings recommenced on 30th June, 1972. With nail sets 5 and 7 of the east-west walls new sets of nails had to be installed and readings recommenced on 10th July, 1972. A major disruption of the pit by vandals prevented many readings on 1st March, 1973. The decision was then taken to stop any further recording at this site.

6.7.2 Observed movements

The observed movements are irregular. However, this must be partly due to the discontinuities in the series of readings. The possibility also exists that any large positive movements in the week immediately after excavation, like those observed at South Mimms, may have been missed completely. Nevertheless, there is a general tendency for a positive movement to have occurred in June 1972. This is then followed by a negative movement with or without a subsequent positive movement.

The largest positive movements occurred at set 2 of the north and south walls and set 2' of the east and west walls. Movements
Fig. 31(e). Observed movement of east and west walls of trial pit at Pegsdon.
Fig. 3(b). Observed movement of north and south walls of trial pit at Pegsdon.
recorded for the sets placed in the lower part of the pit tend to be negative. Final observations at all five sets in the north and south walls are negative. Final observations at three of the seven sets in the east and west walls are positive.

6.7.3 Experimental failure of wall of trial pit

After the decision in March, 1973 to cease further measurements at the trial pit at Pegsdon, an attempt was made to induce failure of one of the walls of the pit. In May, 1973, the east wall of the pit was undercut at the base for a distance of 0.5 m. (Fig. 32a). When inspected in July of that year the wall still appeared stable. The width of the undercut block was then reduced in width to 0.75 m. and was separated laterally by excavation for a distance of 0.5 m. back from the face on each side (Fig. 32b). Vegetation was also removed from its upper surface. Thus, the block was adjoined to neighbouring chalk only at the rear. The block still appeared stable.

When visited in October, 1973 failure had occurred. The block had collapsed into the pit. The relative freshness of the failed chalk and that occurring on the failure plane suggested that the failure had taken place very recently. The failure surface showed indications of shearing of the intact chalk. Wedge-shaped masses of chalk with upper surfaces inclined downwards were present in the wall of the pit. Loose pieces of chalk and powdered chalk were present on the surfaces. The chalk behind the failure surface was extremely wet.
Fig. 2. Dimensions of block involved in experimental failure at trial pit at Pegsdon: (a) May to July 1973; (b) after July 1973.

Fig. 32(c). Field sketch of wall of trial pit after failure.
The dip of the upper surfaces of the wedge-shaped masses was mainly 45° N.W. The dip of these planes did not correspond to that of any of the fractures measured in the trial pit prior to the experimental failure. These planes are therefore considered to have been formed as a result of the failure. However, the chalk in the walls of the pit before failure was blocky (see Section 6.4.2) and the size of the wedge-shaped blocks after failure was found to be similar to that of the blocks prior to failure. The blocks adjacent to the failure plane had presumably been pulled downwards during movement of the falling mass of chalk. Some fracturing of the intact chalk forming the blocks may have occurred to produce the pieces of chalk and powder left on the failure surface.

The interlocking of this blocky chalk appears to make it extremely stable at least in slopes only a metre or so in height. No frost occurred during the period of the experiment but high rainfall (55 mm.) was experienced in the last two weeks of September, and the failure may have taken place at that time.

6.8 Investigation of possible causes of movements observed in the trial pits

6.8.1 Introduction

As a consequence of the difficulties experienced at the trial pit at Pegsdon no attempt has been made to explain these results in detail. The results from the trial pit at South Mimms are considered to be reliable although there are sources of error resulting from the method of measurement adopted and from the
general difficulties of making in situ measurements.

Two main aspects of these results require explanation:

(a) the general trend for movement of the faces of the pit towards the excavation; and

(b) the irregular nature of the movements over the period of measurement.

A number of possible causes for the observed movement may be considered. These are: (1) stress relief; (2) movements along fracture planes; (3) thermal expansion/contraction; (4) moisture; (5) ground water levels; (6) frost; (7) biological factors.

6.8.2 Stress relief

Removal of the rock from the pit results in removal of part of the load on the surrounding rock. As a consequence relaxation of the surrounding rock occurs with an associated decrease in the state of stress in the rock mass. The relaxation is expressed as an inward movement of the pit walls (Fig. 33)

Emery (1966) has suggested that there are two constituent parts to this relaxation: an immediate elastic rebound, largely volumetric and a time-dependent effect. The time-dependent relaxation commences in the new surface and then affects rock gradually further away from the face, but with gradually decreasing effect.

Large inward movements of the walls of the trial pit were recorded immediately following excavation. In the upper part of the pit walls these inward movements were about 0.5 mm - 2.0 mm. They occurred during the first week of excavation. Since the time interval is so short, processes dependent on climatic factors would have had
Fig. 33. Stress relief as a possible cause of movement of walls of trial pit.

Fig. 34. Movement along a fracture as a possible cause of movement of walls of trial pit.
insufficient time to operate effectively. During that week no frost was recorded although some rainfall occurred. The author therefore considers that this movement is probably due to relaxation of the chalk forming the walls of the pit.

6.8.3 Movements along fracture planes

Movement may occur along fracture planes exposed in the pit walls. Exposure of planes dipping into the excavation and removal of supporting rock may initiate movement of the rock overlying these planes (Fig. 34).

To investigate the possibility of movement along continuous fractures exposed in the walls of the trial pit, dip and strike measurements were made of all such fractures that were present. Separate measurements were made for each wall of the pit. The fractures observed are shown plotted stereographically in Fig. 35. The pattern formed by these fractures is discussed in Chapter 7.

The majority of fractures strike either north-west or south-east, but only in the east wall of the pit are both types of fractures observed (Fig. 36). In the east wall these two fracture directions both have dips inclined towards the pit. In the west wall both types of fractures dip away from the face. The north and south walls each have one type of fracture dipping towards the pit and one away from it. The east wall would therefore appear to be the most susceptible to movement along fracture planes.
Fig. 35(a). Stereogram of fractures observed in east wall of trial pit at South Mimms.

Fig. 35(b). Stereogram of fractures observed in west wall of trial pit at South Mimms.
Fig 35(c). Stereogram of fractures observed in north wall of trial pit at South Mimms.

Fig 35(d). Stereogram of fractures observed in south wall of trial pit at South Mimms.
Fig. 36. Main fracture orientations in relation to walls of trial pit at South Mimms.
6.3.4 Thermal expansion/contraction

Two effects are possible as a consequence of temperature changes:
(a) volumetric changes caused by thermal expansion
   and contraction of the chalk; and
(b) fracturing of the chalk as a consequence of the
   relative expansions and contractions.

There is little published evidence available as to the exact
effect of thermal changes on rock. Thermal expansion caused by
temperature changes of at least $300^\circ$C may exert sufficient forces
to shatter rock, and temperature changes may cause enough
differential movement amongst stones to cause them to move downslope
(see, for example, Carson and Kirkby, 1972). Carson and Kirkby (1972)
have also referred to the disputed importance of thermal changes as
a cause of rock weathering. Nevertheless, the available evidence
indicates that the temperature range at the trial pit at South Mimms
is insufficient for thermal changes alone to cause fracturing of the
chalk.

There must be, however, a relative expansion and contraction of
the chalk surrounding the excavation due to temperature changes. To
investigate this effect a laboratory study was undertaken. Dry cores
of chalk (moisture content 0%) were extracted from a block sample of
chalk obtained from the quarry at South Mimms adjacent to the trial
pit. Apart from the moisture content, the chalk composing the cores
was similar to that exposed in the central part of the walls of the
trial pit. The cores of chalk were then subjected to two cycles of
freezing/thawing. The results are given in Table III.
Table III. Results of freeze-thaw tests on dry cores of chalk

<table>
<thead>
<tr>
<th>Temperature °C</th>
<th>Contraction in length (mm.) as compared with original length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Core 1</td>
</tr>
<tr>
<td>Start</td>
<td>16</td>
</tr>
<tr>
<td>Cycle 1 Day 1</td>
<td>-9</td>
</tr>
<tr>
<td>Day 2</td>
<td>16</td>
</tr>
<tr>
<td>Cycle 2 Day 1</td>
<td>-9</td>
</tr>
<tr>
<td>Day 2</td>
<td>16</td>
</tr>
</tbody>
</table>

Length of each core: 76.0 mm.
Diameter of each core: 76.0 mm.

Each cycle consisted of one day at a temperature of 16°C, and one day at a temperature of -9°C. Using the results from cycle 1 of these tests a thermal coefficient of linear expansion of $36.39 \times 10^{-6}/°C$ is obtained, and using the results from cycle 2 a value of $25.98 \times 10^{-6}/°C$. No published values for the coefficient for chalk have been found.

Duncan (1969) gives a general value of $1.3 \times 10^{-6}/°C$ for the coefficient of thermal expansion of calcareous rocks but comments that actual values for particular samples may vary considerably. He also notes that calcareous rocks have the lowest values of the coefficient.

For a temperature range of 0-80°C, Benoit (in Fowle, 1921) gives the following coefficients of thermal expansion for Iceland Spar: parallel to crystal axis $26.31 \times 10^{-6}/°C$, and perpendicular to crystal axis $5.44 \times 10^{-6}/°C$. The value of $26.31 \times 10^{-6}/°C$ is
similar to that obtained by the author using the results from cycle 2. The relatively high value obtained by the author may be due to the absorption of some water by the samples during testing.

6.8.5 Moisture

Two effects resulting from moisture which may be responsible for movements of the walls of the trial pit are:

(a) wetting and drying of the chalk; and

(b) removal of loose chalk at the surface or within the mass by water.

(a) Wetting and drying of the chalk

The chalk in the faces of the pit was observed to be moist throughout the year. Tests on samples obtained from the pit showed that the moisture content of the chalk is usually 26-29%. The saturation moisture content of the chalk was found to be 29-32%. The moisture content of the chalk in the walls of the pit is therefore at or close to the saturation value.

When observed after periods of dry weather the chalk at the surfaces of the pit walls had dried out considerably, but the chalk at a distance of only 0.1 - 0.2 m. behind the faces, was still very wet. Significant changes of moisture content do therefore occur at the faces as a result of wet or dry periods of weather. These changes are not, however, thought to occur in the chalk generally, which would seem to be usually at or near its saturation moisture content value. The effects of
wetting and drying are therefore considered to be mainly limited to the surfaces of the walls of the pit, and for a distance of about 0.1 - 0.2 m. back from the faces.

To investigate the possibility of attributing some or all of the movements of the pit walls to changes in moisture content of the chalk, swelling tests were undertaken. Two cores of chalk were extracted from block samples obtained from the quarry adjacent to the trial pit at South Mimms. Each core, 75 mm. in diameter and 75 m. high, was placed in a glass container. A glass plate was placed on the upper surface of the core, and a dial gauge, reading to 0.01 mm., was arranged to measure any upward deflection of the specimen. The container was then filled with water to a height mid-way up the core. Both cores remained under test for several weeks. No swelling was measured during this period. The cores were then allowed to dry out. No shrinkage was measured during drying. The tests were carried out at a temperature of 14-17°C.

The results obtained by the author from these tests are similar to those obtained by Meigh and Early (1957). Swelling and shrinkage of the intact chalk resulting from changes in moisture content are not responsible for the movements that were measured in the trial pit.

(b) Removal of loose chalk by water

The possibility exists that water moving through the chalk may remove and transport small pieces of loose rock. Solution of the chalk may also occur. This internal erosion might then give
rise to some minor instability with possible movement of blocks and chalk. Also possible is the removal of loose chalk from fractures reducing their shear strength and causing movement of chalk overlying the fracture. The possible changes in the character of fracture planes in this way is discussed more fully in Chapter 9.

Evidence for the internal erosion of chalk by water has been obtained from observations at a locality near Rammore, Surrey (Farrar, 1973). Water emerging from chalk near the foot of a road cutting deposited particles of chalk presumably eroded from within the mass of chalk behind. Water emerging from widened joints has also been noted by the author at Alum Bay, Isle of Wight, and at Ashwell, Hertfordshire (Section 9.2.10). The joints have presumably been widened by solution and/or erosion of the surrounding chalk.

Although the surfaces of the pit at South Mimms have always been moist, no water has been observed flowing from the walls or lying on the floor of the pit. The chalk at that locality would therefore seem to be free-draining. Chalk removed by water either as fragments or in solution would therefore probably be carried mainly downwards, rather than laterally. The pit is known to lie well above the general water table level (see Section 6.8.6). Thus, although solution and erosion are often possible, they are not considered likely causes of the movements observed in the trial pit.
6.8.6 Ground water levels

Seasonal changes in the ground water level in chalk areas may be 30 m. or more. Large fluctuations occur especially beneath valley sides and interfluves and small fluctuations below valley floors (Smith, 1972).

The present author believes there to be two main effects of such ground water level fluctuations. These are:

(a) changes in the character of the chalk in the zone between minimum and maximum water table levels; and

(b) changes in the forces acting within the chalk.

(a) Changes in the character of the chalk

Cycles of saturation and drying of the chalk in the zone of seasonal water table fluctuation will tend to create open fractures. Any loose material filling the fractures will be removed by water percolating downwards, and the chalk will also tend to be gradually dissolved thereby increasing the width of the fractures.

(b) Changes in the forces acting within the chalk

Cycles of saturation and drying do not cause swelling or shrinkage of the chalk. However, a rise in the level of the water table will increase the apparent density of the chalk that becomes saturated, since the density of water is much greater than that of the air that is displaced. The effect is therefore to increase the overall density of the mass of chalk. Although pore water pressures are generally considered to be insignificant in the case of rock, there is the possibility
of fracture water pressures resulting from the filling of open fractures with water. The effect of the increased density of the mass may be to exert forces sufficiently great to cause movement of the walls of the pit. The increased density may give rise to forces sufficient to overcome the shear strength of some fracture planes resulting in movements along them.

Available data regarding ground water levels in the South Mimms area has been investigated by the author. The nearest well to the trial pit for which records are available is at Clare Hall, South Mimms. This is located 2.5 km. S.S.W. of the trial pit. However, The Thames Conservancy did not begin records until November, 1972 and few conclusions may therefore be reached about the changes in ground water levels. Seasonal fluctuations for this well are small, being only about 0.5 - 0.6 m. The well is located at an altitude of about 115m., on a low ridge, about 10 - 15 m. above the valley floor to the south which has a branch of the Mimmshall Brook flowing in it.

According to Wooldridge and Kirkaldy (1937), wells in the vicinity of Water End, where the Mimmshall Brook normally disappears underground down swallow holes, show that the water table usually lies about 15m. beneath the bed of the stream. Even after torrential rain on one day in 1936 a rise in the level of the water table of only 2m. was recorded. Wooldridge and Kirkaldy also note that the water table in the chalk in the area of the Mimmshall Brook is abnormally high (see Fig. 37). They attribute this to the numerous swallow holes in the area.
Fig. 37(a). Ground water level contours for the drainage system of the Mimms Hall Brook. (after Wooldridge and Kirkaldy, 1937).

Fig. 37(b). Contours for the top of the Chalk in the area of the drainage system of the Mimms Hall Brook. (after Wooldridge and Kirkaldy, 1937).
which allow a large proportion of the rainfall to flow into the chalk.

Fig. 37 shows the average ground water levels for the Mimmshall Valley. The average ground water level beneath the trial pit is at about 73 m. above Ordnance Datum. The ground level at that location is 100 m. O.D. At no time was water present on the floor of the quarry near the trial pit, which until a few years ago was deeper, the floor being at a level of about 76 m. O.D. As the average ground water level is therefore about 25m. below the floor of the trial pit, the chalk is free draining all year round. Ground water levels are not responsible directly for any of the movements observed.

6.8.7 Frost

The susceptibility of chalk to frost action has been described by, for example, Higginbottom (1966) and Lewis and Cronney (1966). Lewis and Cronney have shown that the softer varieties of chalk are particularly susceptible.

The frost heave test described by Lewis and Cronney (1966) has been used by the present author to assess the susceptibility of chalk similar to that in the walls of the trial pit at South Mimms. Two cores of chalk were tested. The frost heave recorded over the period is shown graphically in Fig. 38.

Sample A had a 2 mm. wide horizontal fracture at about mid-height and during test a thick ice lens developed along this fracture. Sample B was an intact sample with no fractures. However, a larger heave was recorded than with sample A. The results of the
Fig. 38. Results of frost heave tests on two samples of chalk.
tests are given below.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Natural moisture content (%)</th>
<th>Moisture content after test (%)</th>
<th>Frost heave (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>23</td>
<td>34</td>
<td>50</td>
</tr>
<tr>
<td>B</td>
<td>26</td>
<td>32</td>
<td>76</td>
</tr>
</tbody>
</table>

To investigate the effects of frost action on chalk in more detail some further tests were undertaken. Two cores extracted from larger block samples obtained from the quarry at South Mimms adjacent to the trial pit were left in water for 24 hours. The cores of chalk were then subjected to freezing. After removal from the freezer, measurement, and leaving for 12 hours at a temperature of 16°C, the cores were soaked in water. Both cores disintegrated along cracks. The cracks extended right across the specimens, mainly longitudinally. No further cycles of testing were therefore undertaken with these specimens. No cracks were observed in the specimens prior to test. Had cracks been present the chalk would probably have disintegrated during coring. The possibility exists that these cracks were caused by the coring, although the initial saturation of the cores should have enabled their discovery.

The results of these freezing tests are given in Table IV together with the results of the tests already described in Section 6.8.4.
Table IV. Results of freeze-thaw tests on cores of chalk

<table>
<thead>
<tr>
<th>Temperature °C</th>
<th>Test Series A (dry cores)</th>
<th>Test Series B (soaked cores)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Core 1</td>
<td>Core 2</td>
</tr>
<tr>
<td>Start</td>
<td>16</td>
<td>0.00</td>
</tr>
<tr>
<td>Cycle 1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Day 1</td>
<td>-9</td>
<td>-0.07</td>
</tr>
<tr>
<td>Day 2</td>
<td>16</td>
<td>-0.02</td>
</tr>
<tr>
<td>Cycle 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Day 1</td>
<td>-9</td>
<td>-0.08</td>
</tr>
<tr>
<td>Day 2</td>
<td>16</td>
<td>-0.03</td>
</tr>
</tbody>
</table>

Length of each core: 76.0 mm.
Diameter of each core: 76.0 mm.

These results indicate that dry chalk is largely unaffected by freezing and thawing, whereas wet chalk is severely affected. The expansion produced by freezing of water is about 9 per cent of its volume. In the case of chalk the force exerted by this expansion would seem to be sufficient to cause fractures to develop. The water is held within pores in the chalk, and freezing of this water exerts forces that eventually overcome the bonds between the constituent particles. The author suggests that the term pore ice pressure might usefully be employed to describe this phenomenon.

The evidence described indicates that freezing and thawing of chalk in the presence of moisture is a process that does significantly
affect the characteristics of chalk, and that this might be responsible for the movements observed in the trial pit.

6.6.8 Biological factors

Penetration by roots may exert forces on the rock and these have been observed to cause failure of quite large masses of chalk as described in Chapter 5. Roots and burrowing activity by animals will tend also to increase the permeability of the near surface rock. Cavities that were possible rabbit burrows were found in the loose chalk exposed in the upper part of the pit faces (Section 6.4.1). No entrances to inhabited burrows were found at the surface near the pit. Only grass was present in the immediate vicinity of the pit and the roots did not extend more than about 30 mm. into the loose chalk. The evidence suggests that biological factors are not a major cause of the movements observed in the trial pit.

6.9 Discussion

The results of the measurements made at the trial pits, especially those at South Mimms, showed that over a period of time movements do occur in steep chalk slopes. The method adopted for measurement was of necessity relatively simple. It was open to some errors such as the thermal expansion and contraction of the nails that must have occurred as mentioned in Section 6.5. Nevertheless the size of the movements measured and the consistency of the results suggest that the method adopted was reasonably precise.
The largest inward movement recorded was 4.3 mm. for the uppermost part of the north and south faces. The largest inward movements were recorded in the upper parts of the faces of the pit. These movements occurred in rubbly chalk and such chalk would be expected to be less rigid than the normal blocky chalk beneath. Yet, even in the blocky chalk inward movements of up to 1.5 mm. were measured.

The pattern of movements recorded at the trial pit at South Mimms consists of three main phases as illustrated in Fig. 39. Phase 1 took place in the week immediately after excavation and consisted of a large positive (inward) movement in the upper part of the pit. The size of the positive movement decreased downwards and in the lower part of the pit a small negative movement occurred. Some part of this phase 1 movement may well have occurred during excavation and therefore escaped measure. Phase 2 consisted of an overall negative movement which reached its acme six weeks after excavation of the pit. Phase 3 resulted in a general positive movement of the walls of the pit. Although this movement was not continuous but irregular, the general positive trend continued until measurements ended in March, 1974.

Investigation of the causes of the observed movements has suggested that the phase 1 movement is due to stress relief resulting from excavation of rock originally supporting the chalk in the faces of the pit. The phase 2 movement which consisted of a relative movement of the chalk back into the face was thought
Movement from initial measured position on 9.10.71. (mm).

Ground level

Floor of pit

192

19272

1.3.74

Vertical Scale: 1:15

Fig. 39. Observed movement of East and West Walls of Trial Pit at South Mimms on 19.2.72 and 1.3.74.
Movement from initial measured position on 9-10-71. (mm.)

Ground level

19-2-72

1-3-74

Floor of pit

Vertical Scale 1:15

Fig. 39(b) Observed movement of North and South Walls of Trial Pit at South Mimms on 19-2-72 and 1-3-74.
to be possibly due to shrinkage of the chalk as it dried out on exposure to the atmosphere. The tests described in Section 6.8.5 showed, however, that no volume change occurs with changes in moisture content of the chalk. The movement may be attributable to lower temperatures recorded at that time. The movement to be explained is one of about 0.3 mm. The decrease in average temperature between the phase 1 and phase 2 readings is about $10^6\text{C}$. (Appendix 1). Assuming the thermal coefficient of linear expansion of the chalk to be $30 \times 10^{-6}/\text{C}$. (Section 6.8.4), then to explain the movement of 0.3 mm., chalk for a distance of 1 m. back from the face would be involved. Temperature changes may therefore explain the phase 2 movement and some of the other irregular movements which occurred.

The phase 3 movement resulted in a general inward movement of the pit faces. In the early stages this movement was greater in the upper part of the pit and decreased regularly downwards. This regularity became much more obscure as the period of measurement progressed. Particular disintegration of the faces of the pit was noted between July and October 1973. This occurred two years after its excavation. The chalk which was extremely wet appeared very shattered and pieces had spalled off the faces. The shattering was especially intense on the east face. By the end of October, 1973 the north and south faces also appeared shattered although the chalk had dried out considerably. Some disintegration was also occurring along joint surfaces intersecting the pit (Fig. 40). Small masses of rock were becoming detached and falling down to the
Fig. 40(a). Fracture controlled disintegration of an exposed face.

Fig. 40(b). Field sketch of upward disintegration along a fracture plane in the trial pit at South Mimms.
floor of the pit. The disintegration was limited backwards by a prominent joint.

When visited in December, 1973 the central parts of the walls of the pit had undergone significant decay and the chalk was very shattered and loose. This is reflected in the positive movements shown in Figs. 29 and 30. In March, 1974 further disintegration upwards along a joint with a dip of 66°S. was occurring in the north wall of the pit. If left that part of the pit would not be stable until disintegration back along that joint to an angle of slope of 66° had been achieved.

Significant deterioration in the condition of the pit therefore occurred some two years after its excavation. The rate of inward movement would seem to accelerate with time. Excepting the large movements immediately after excavation, the first indication of another large inward movement was in March, 1973, by which time the pit had been subjected to two winters. Observation of the chalk extracted from the trial pit showed that it was relatively unaffected by exposure during the first winter and retained its blocky character. Yet, after exposure to the second winter, it had been completely reduced, presumably by frost action, to small fragments and powder.

6.10 Conclusions

(a) Significant movements do occur in the faces of steep exposed chalk slopes.

(b) Stress relief is initially a significant cause of movement.

(c) In the long-term frost action is mainly responsible for disintegration and movement of faces of chalk.

(d) These movements may occur at an accelerating rate and reflect increasing disintegration of the exposed chalk.
7. Fracture Pattern of the Chalk

7.1 Introduction

Most rocks are traversed by fractures. These fractures are usually joints, but in some cases may be faults or other planes such as cleavage and schistosity. The term 'fracture' is used here to include all types of fracture planes that may occur e.g. faulting, jointing, cleavage, schistosity, although in the case of chalk only bedding planes jointing and faulting are present. Study of literature describing case histories of rock slope failures indicates that these pre-existing fracture planes are those along which movement often occurs. Field work by the author described in Chapter 5 has confirmed this.

Apart from a few brief or vague references to the joint system within the Chalk (for example, Peake, 1967), only one detailed investigation appears to have been undertaken. In an investigation of the geological factors affecting jointing, Denness (1969) examined jointing at four sites in the Chalk. The sites were at Dover, Dartford, Beachy Head and Alum Bay. Bedding was found to be a major factor affecting the jointing.

In competent sedimentary strata the fractures usually form a definite pattern related to the stress pattern of patterns that have acted on the strata during their geological history. Consequently, these fracture patterns are frequently related to the major fold axes. Price (1966) describes the fracture patterns and indicates that in simply folded rocks two sets of tension joints occur, one set parallel to the fold axis and one set at right angles.
to the fold axis, together with two sets of shear joints, these being cut by the set of tension joints at right angles to the fold axis (Fig. 41). In addition, other joints may be developed, for example shear joints may occur at the crest of anticlinal folds with the acute angle made by the joints being intersected by the fold axis. Faults may also be related to the joint pattern, and Price describes the fault patterns that may be present in a horizontally bedded stratum.

A detailed field investigation was made by the author to ascertain if a definite fracture pattern exists within the Chalk, and if so, the extent to which the pattern remains constant over a given area. If definite fracture directions are recognisable over sufficiently large areas then the prediction of likely directions of failure becomes possible. During the field work the types of fractures present, the spacing of fractures and the nature of the fracture surfaces were also noted. The results of these aspects of the work are discussed more fully in Chapters 8 and 9.

7.2 Structural geology of the chalk

Price (1966) suggests that the fracture patterns developed in competent strata are frequently related to the fold axes. Recognition of the major fold axes for Chalk areas was therefore made together with major faults. The main structures present within the Chalk outcrop are shown in Fig. 42.

The Chalk has been affected by only one major period of earth movements, the Alpine Orogeny commencing some 70 million years ago.
Fig. 41. Typical system of jointing developed on the limb of a fold. (after Price, 1966).
Fig. 42. Map of main structures affecting the Chalk of South-east England.

Amphitheatres
Overturned Formations
Synclines

\[ < 0.5 \text{ Kellen} \]
As this Orogeny was centred in Continental Europe the intensity of effects in Britain decrease northwards. Consequently, the only areas where the Chalk is affected by complex structural movements are South Dorset, the Isle of Wight and the Hog's Back, near Guildford. In South Dorset the Chalk is involved in faulting and folding along the northern limb of the Weymouth Anticline and its eastern extension into Ballard Down and the monocline of the Isle of Wight. At the Hog's Back is another faulted monoclinal fold. Elsewhere the Chalk is affected only by relatively simple 'open' folds such as the London Basin and the Wealden Dome. Folds of smaller amplitude and still of simple form often occur on the limbs of the main folds (e.g. in the Lewes area of Sussex).

Major folds are largely absent from the Chalk of East Anglia, Lincolnshire and the Yorkshire Wolds, and the Chalk dips gently eastwards at less than 10°. However, in these areas some less well known structural modifications have occurred as a result of glacial action during the Pleistocene. These structures include thrust planes that are thought to be due to the 'pushing' effect of ice advances. One such thrust plane has been observed at Barkway in North-east Hertfordshire.

7.3 Field procedure

At each locality visited the dip and strike of each observed fracture was recorded. Measurements were made using a standard compass-clinometer manufactured by M.D.S. Ltd. Care was taken to ensure that the fracture to be measured was not just a
weathered surface. Dirty faces were therefore cleaned of loose material beforehand. Many fractures exposed in weathered faces were found to change their orientation a short distance in from the exposed face. The distinction between fractures produced by weathering and the more extensive probable tectonic fractures was therefore important, but this was found to be relatively straightforward if the faces were cleaned before measurements were made.

In addition to recording the orientation of the fractures at each locality, other factors have also been noted where possible. These have included the distinction between joints and faults, the orientation of the bedding, the nature of the fracture surfaces, and the spacing of fractures. Fracture spacing is dealt with in Chapter 8 and the nature of fracture surfaces in Chapter 9.

Localities visited at which fractures have been measured were mainly in the Chilterns, the South Downs, and the Isle of Thanet. These areas were selected initially because the geological structures are relatively simple. However, a number of measurements have also been made in other areas such as the Isle of Wight.

7.4 Results of directional measurement of fractures

7.4.1 South Mimms

Initially, the measurement of fractures was undertaken by the author at South Mimms, Hertfordshire. These measurements were undertaken in connection with the trial excavation in which movements were monitored (see Chapter 6). The movements that were
observed in the walls of the excavation were considered to be possibly related to the fracture pattern. Measurements of the dip and strike of the fractures were made in the walls of the excavation and also in the faces of the nearby quarry. All the fractures observed were joints. Of these joints, 89% had a dip of more than 75°. The lowest dip angle recorded was 51°.

The measurements made at South Mimms are shown plotted in stereographic projection in Fig. 35. The standard Wulff Net has been used for all stereographic projections that have been constructed during the author's work. The method adopted is described by, for example, Phillips (1971).

The fractures plotted in Fig. 35 indicate two main concentrations of fracture strike directions. One concentration occurs between N20°E. and N60°E. A second concentration occurs between NO°W. and N55°W. These well marked groupings of fracture strikes suggested that the fracturing within the Chalk at this locality was not randomly orientated, but might conform to some definite pattern. In Fig. 43 the strikes of the fractures are plotted as a frequency polygon. This figure indicates the concentration of fracturing into six main directions. The mean strike values for each of these concentrations are: N27°E., N50°E., N67°E., NO°W., N27°W., N49°W.

Structurally, South Mimms is located on the northern side of the London Basin. The strike of the northern limb of this major synclinal fold is N.E.-S.W. The pattern of fracturing postulated by Price for this type of fold is shown in Fig. 44a.
Fig. 43. Frequency polygon of percentages of fractures observed at South Mimms plotted against their strikes.
Rq. 44-(a). Expected fracture pattern for fold with N.E.-S.W. axis.

Fig. 44(a). Expected fracture pattern for fold with N.E.-S.W. axis.

Fig. 44(b). Recorded fracture pattern at South Mimms.
The mean strikes of fractures recorded at South Mimms are shown for comparison in Fig. 44b. The mean strikes of N49°W. and N0°W. would therefore seem to correspond with the expected shear fractures denoted by \( J_1 \) and \( J_2 \). The mean strikes of N50°E. and N27°W. probably correspond with the expected fractures denoted by \( J_3 \) and \( J_4 \).

In addition to being located on the northern limb of a major syncline the South Mimms area lies on an anticlinal flexure with a N.E.-S.W. axis. There is also the possibility of 'anticlinal warping' along a N.W.-S.E. axis (Wooldridge and Kirkaldy, 1937). Such anticlinal folding along a N.W.-S.E. axis might be expected to result in the formation of an additional set of shear fractures as indicated in Fig. 45. These fractures are denoted by \( J_x \) and \( J_y \). For fracture directions other than these the notation given by Price has been adopted. The mean strikes of N27°E. and N67°E. recorded at South Mimms correspond with this additional set of shear fractures.

As indicated in the stereogram of fractures measured at South Mimms (Fig. 35), the intensity of fracturing varies from one set of fractures to another. The \( J_3 \) and \( J_y \) fractures are developed particularly well. The pronounced \( J_y \) fracture direction might be expected since it corresponds to the tension fracture direction parallel to the N.E.-S.W. axes of both the major synclinal fold and the better developed of the anticlinal folds.

**7.4.2 The Chilterns**

The results of the fracture survey at South Mimms indicated that a definite fracture pattern is present and that this pattern
Fig. 45(a). Shear joints, $J_1$ and $J_2$, formed in response to folding along N.E.-S.W. axis.

Fig. 45(b). Shear joints, $J_x$ and $J_y$, formed in response to folding along N.W.-S.E. axis.
may be related to the structural geology of the area. Having recognised a pattern at one particular locality, an attempt was made to distinguish a pattern on a regional scale.

Measurements of the dip and strike of fractures were undertaken at nineteen localities in the northern Chilterns. The localities are shown in Fig. 46. A distinction was made in the field between joints and faults.

The results have been analysed in several ways. The localities were divided into a southern group and a northern group. The northern group included localities in south Cambridgeshire, west Essex and north-east Hertfordshire, east of a line from Hertford to Hitchin. The southern group included localities in east Buckinghamshire, south Bedfordshire and Hertfordshire, west of the Hertford-Hitchin line. The results of the survey undertaken at South Mimms are included in the results for the southern group of localities. For each locality the mean strike of each group of joints was calculated. The mean strike values calculated are shown plotted as frequency polygons in Fig. 47. For both the northern and southern groups six main joint sets may be recognised. A summary of these systematic joint sets is presented in Table V.

The strike directions of these joint sets are similar to those recognised at South Mimms. There is an indication that the $J_1$, $J_2$ and $J_4$ joints in the northern group of localities are rotated anticlockwise by about 10-15°, compared with the equivalent joint sets in the southern group of localities.
Fig. 46. Localities in the northern Chilterns at which fractures have been measured. Generalised base and top of Chalk shown. National Grid lines indicated at 20km. intervals.
Fig. 4.7. Frequency polygons of number of stations at which dominant joint sets are developed plotted against their strike for (a) Northern Chilterns and (b) Southern Chilterns.
The $J_2$, $J_x$ and $J_y$ joints in the northern group are rotated anticlockwise by about 5-10°. This is to be expected as the strike of the Chalk changes from approximately $N45^\circ E$ at southern localities to $N35^\circ E - N40^\circ E$ at northern localities.

The mean strike directions for all the localities sampled in the Chilterns are listed in Table V. All joints measured in the Chilterns are plotted as a frequency polygon in Fig. 48. The six main joint directions recognised earlier are still apparent. The formation of the joints is suggested to be similar to that already described for the locality at South Mimms. The $J_1$ and $J_2$ joint sets are considered to be shear fractures related to the main synclinal fold of the London Basin. The $J_3$ and $J_4$ joint sets are probably tension joints. The $J_4$ direction bisects the acute angle between the $J_1$ and $J_2$ joint sets, and the $J_3$ direction coincides with the strike of the Chalk, and the axis of the secondary flexures developed on the northern limb of the main synclinal fold. The acute angle between the $J_x$ and $J_y$ joint sets is usually bisected by the $J_3$ joint set direction. The $J_x$ and $J_y$ joints may be shear joints related to the secondary flexures.

Wooldridge and Linton (1955) suggest that a series of parallel anticlinal and synclinal folds with axes trending N.W. - S.E. exist in the Chilterns. They cite as evidence the lines of Bocene outliers aligned parallel with the strike of the Chalk.

The structural geology of the Cambridge area has been described in some detail by Worssam and Taylor (1969). One locality sampled by the author, that of Reach, occurs in this area. The immediate
TABLE V. Summary of systematic joint sets recognised in the Chilterns

<table>
<thead>
<tr>
<th>Joint set</th>
<th>Modal strike direction</th>
<th>Mean dip angle</th>
<th>% of localities at which set developed</th>
<th>% of localities at which pair of sets developed</th>
</tr>
</thead>
<tbody>
<tr>
<td>J₁</td>
<td>N80°W, N60°W, N70°W,</td>
<td>90° 87°S, 90°</td>
<td>78 90 84</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(100°) (120°) (110°)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J₂</td>
<td>N10°W, N5°E, NO</td>
<td>84°E, 87°E, 85°E</td>
<td>100 70 64</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(170°) (005°) (000°)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J₃</td>
<td>N45°E, N50°E, N50°E,</td>
<td>65°N, 88°E, 88°N</td>
<td>67 60 63</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(045°) (050°) (050°)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J₄</td>
<td>N45°W, N50°W, N45°W,</td>
<td>88°N, 89°N, 88°N</td>
<td>89 80 84</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(135°) (150°) (135°)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jₓ</td>
<td>N20°E, N30°E, N25°E,</td>
<td>68°W, 81°W, 66°W</td>
<td>56 50 53</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(020°) (030°) (025°)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jᵧ</td>
<td>N65°E, N65°E, N65°E,</td>
<td>69°N, 90° 90°</td>
<td>56 60 58</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(065°) (065°) (065°)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

NOTE. Data is shown separately for: the northern group of localities sampled; the southern group of localities sampled; and all localities sampled.
Fig. 48. Frequency polygon of percentage of observed joints in the Chilterns plotted against their strikes.
vicinity of Reach is dominated by a synclinal axis trending N50°E. The joints recognised at that locality are in accord with this structure. They are a tension joint set \( J_2 \) parallel to the axis of that fold, and the regional strike, and two sets of shear joints \( J_1 \) and \( J_2 \). Little other detailed structural data appears to be available for the area studied by the author.

The mean strike directions of joint sets measured at each of the localities sampled are shown in Fig. 49. In Fig. 50 faults are shown plotted on a stereogram. Few faults were observed and they do not appear to be confined to definite directions.

### 7.4.3 Beer, Devon

The results of the fracture survey conducted in the Chilterns indicated that a definite fracture pattern is recognisable over a large area, and that the pattern is related to the overall structure. In the Chilterns the relation of the fracture pattern to the structure had been achieved by correlating fractures measured at isolated exposures with structures suggested by other workers. At Beer in East Devon the opportunity was taken to attempt a more controlled correlation between fracture pattern and structure.

The vicinity of Beer is dominated by a synclinal fold called the Beer Syncline (Smith, 1965). In the cliffs the
Fig. 49. Map of strikes of fracture sets at localities in the northern Chilterns.
Fig. 50. Stereogram of faults observed in the Chilterns.
following strata are exposed:

<table>
<thead>
<tr>
<th>Zone</th>
<th>Approximate thickness (metres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Chalk</td>
<td></td>
</tr>
<tr>
<td>Micraster cortestudinarium</td>
<td>10</td>
</tr>
<tr>
<td>Holaster (Stenotaxis) planus</td>
<td>20</td>
</tr>
<tr>
<td>Middle Chalk</td>
<td></td>
</tr>
<tr>
<td>Terebratulina lata</td>
<td>50</td>
</tr>
<tr>
<td>Inoceramus lebiatus</td>
<td>5</td>
</tr>
<tr>
<td>Cenomanian Limestone</td>
<td>2</td>
</tr>
<tr>
<td>Upper Greensand (Top Sandstones)</td>
<td>seen to 2</td>
</tr>
</tbody>
</table>

The accessible part of the section in which dip and strike measurements were made consisted of Upper Greensand, Cenomanian Limestone and the lower part of the Middle Chalk. Measurements were made at three separate places in the cliffs at West Beach.

At two of these places the dip of the bedding was about 5°E; at the third place the dip increased to 15-20°E. The dip and strike measurements for these three places are shown plotted on stereograms in Figs. 51 a, b, c. The stereograms suggest the presence of two sets of shear joints, $J_1$ and $J_2$, and two sets of tension joints, $J_3$ and $J_4$. Tension joint $J_4$ intersects the acute angle between the shear joints, and tension joint $J_3$ is aligned parallel to the axis of the fold. The stereogram (Fig. 51 c) has tension joints $J_3$ dipping mainly at angles of 60-70°W, but this is consistent with the increased dip of the bedding observed at that place.

The pattern of jointing observed at Beer confirms that the jointing in the Chalk is directly related to the directions of
Fig. 51(a). Fractures observed at Locality 1, Beer, Devon.

Fig. 51(b). Fractures observed at Locality 2, Beer, Devon.
Fig. 51(c). Fractures observed at Locality 3, Beer, Devon.
the structural axes. The results also indicate that this relationship between jointing and folding is maintained for both large and small scale folds.

7.4.4 The South Downs

The results of the directional measurement of fractures in the Chilterns and at Beer indicated a good correlation between fracturing and the orientation of the fold axes. This work was therefore extended to two other Chalk areas in South-east England, the South Downs and Thanet (see Section 7.4.5).

The mean strike directions of fractures at localities examined in the South Downs between Brighton and Eastbourne are plotted in Fig. 52. All the fractures observed were joints. A frequency polygon of the strike directions of all the measured joints is given in Fig. 53. The mean strike directions of the fractures recognised at these localities are also listed in Table VI. The mean strike direction for each fracture set varies very little from station to station, although only at one station are all six fracture directions developed. The fracture pattern is very similar to that recognised in the Chilterns.
Fig. 52. Map of strikes of fracture sets at localities in the South Downs.
Fig. Frequency polygon of percentage of all joints observed at localities in the South Downs plotted against their strike.
### TABLE VI Summary of systematic joint sets recognised in the South Downs

<table>
<thead>
<tr>
<th>Joint set</th>
<th>Modal strike direction</th>
<th>Mean dip angle</th>
<th>% of localities sampled at which set developed</th>
<th>% of localities sampled at which pair of sets developed</th>
</tr>
</thead>
<tbody>
<tr>
<td>J₁</td>
<td>N20°W. (160°)</td>
<td>88°E.</td>
<td>67</td>
<td></td>
</tr>
<tr>
<td>J₂</td>
<td>N40°E. (040°)</td>
<td>89°W.</td>
<td>58</td>
<td>50</td>
</tr>
<tr>
<td>J₃</td>
<td>N60°W. (100°)</td>
<td>89°W.</td>
<td>63</td>
<td></td>
</tr>
<tr>
<td>J₄</td>
<td>N10°E. (010°)</td>
<td>88°E.</td>
<td>58</td>
<td>50</td>
</tr>
<tr>
<td>Jₓ</td>
<td>N60°E. (060°)</td>
<td>87°W.</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td>Jᵧ</td>
<td>N45°W. (135°)</td>
<td>90°</td>
<td>67</td>
<td>42</td>
</tr>
</tbody>
</table>

#### 7.4.5 The Isle of Thanet

Measurement of fractures has also been undertaken in the cliff sections of Thanet. During the field work a distinction was made between joints and faults. The mean strike directions of the joints at localities examined are plotted in Fig. 54.

The mean strike directions of the joints recognised at these localities are listed in Table VII. The fracture pattern is similar to that recognised in the Chilterns and South Downs. The intensity of the jointing varies between joint sets. The Jᵧ joints are particularly well-developed, as are the J₁ and J₄ joints. There is no readily apparent reason for the especially
Fig. 54. Map of strikes of fracture sets at localities in the Isle of Thanet. National Grid lines indicated at 5 Km. intervals.
prominent jointing in the directions.

TABLE VII Summary of systematic joint sets recognised in the Isle of Thanet

<table>
<thead>
<tr>
<th>Joint set</th>
<th>Modal strike direction</th>
<th>Mean dip angle</th>
<th>% of localities sampled at which set developed</th>
<th>% of localities sampled at which pair of sets developed</th>
</tr>
</thead>
<tbody>
<tr>
<td>( J_1 )</td>
<td>N20°W. (160°)</td>
<td>69°E.</td>
<td>72</td>
<td>50</td>
</tr>
<tr>
<td>( J_2 )</td>
<td>N40°E. (040°)</td>
<td>67°E.</td>
<td>67</td>
<td></td>
</tr>
<tr>
<td>( J_3 )</td>
<td>N50°W. (100°)</td>
<td>68°E.</td>
<td>67</td>
<td>50</td>
</tr>
<tr>
<td>( J_4 )</td>
<td>N5°E. (005°)</td>
<td>61°E.</td>
<td>89</td>
<td></td>
</tr>
<tr>
<td>( J_x )</td>
<td>N70°E. (070°)</td>
<td>65°E.</td>
<td>56</td>
<td></td>
</tr>
<tr>
<td>( J_y )</td>
<td>N60°W. (120°)</td>
<td>90°</td>
<td>94</td>
<td>50</td>
</tr>
</tbody>
</table>

7.4.6 The North Downs

Although only a limited number of localities have been sampled, sufficient evidence has been obtained to indicate that a similar fracture pattern exists in the North Downs in Kent to that existing in the South Downs and Thanet. The mean strike directions of fractures at localities visited in the North Downs are shown in Table VIII.

TABLE VIII Mean strike directions of systematic joint sets recognised in the North Downs

<table>
<thead>
<tr>
<th>Joint set</th>
<th>Mean strike direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>( J_1 )</td>
<td>N20°W. (160°)</td>
</tr>
<tr>
<td>( J_2 )</td>
<td>N33°E. (033°)</td>
</tr>
<tr>
<td>( J_3 )</td>
<td>N59°W. (091°)</td>
</tr>
<tr>
<td>( J_4 )</td>
<td>NO°. (000°)</td>
</tr>
<tr>
<td>( J_x )</td>
<td>N53°E. (053°)</td>
</tr>
<tr>
<td>( J_y )</td>
<td>N54°W. (126°)</td>
</tr>
</tbody>
</table>
The measurement of fractures that has been described in Sections 7.4.1 - 7.4.6 was undertaken in areas in which the structural geology is relatively simple. Those areas are mainly affected by open folds of varying magnitude. To examine the extent to which fracturing is determinable and is consistent in areas of more complicated structural geology, measurement of fractures was made at localities in the Isle of Wight. The results of these measurements indicate that even in areas of more complicated structural geology fracture orientations are still reasonably consistent, and the same fracture orientations may be measured at localities several kilometres apart.

As the fracture pattern in the Chalk of the Isle of Wight is more complex than in the other areas examined, a comprehensive investigation of the strike orientations would require more data than that obtained by the present author. Nevertheless, a frequency polygon plot of the strike directions of all the fractures measured indicates the presence of at least three main fracture sets (Fig. 55). When the group of fractures with a mean dip of 100° was analysed in more detail, it was found to be composed of at least two separate fracture sets. One fracture set corresponds to the bedding planes with a mean strike of 100° and a typical dip angle of 65-68° N. The second fracture set has a similar mean strike of 100° but a typical dip angle of 50-55° S. The bedding planes have been included in this analysis.
Fig. 55. Frequency polygon of percentage of observed fractures in the Isle of Wight plotted against their strikes.
as the field work described in Chapter 5 has shown that they are particularly important in controlling slope failures in the Isle of Wight.

7.5 Analysis of dip measurements of fractures

7.5.1 Introduction

Recognition of the main fracture directions is considered by the author to be important in the determination of the effect of these fractures on the stability of rock slopes. In addition, the dip of these fractures is of great importance. For example, a fracture sloping downwards towards the face of a slope, and inclined at an angle of 70° from the horizontal, will normally be more likely to cause instability than a fracture with similar strike also sloping downwards towards the face of the slope, but inclined at an angle of 20° from the horizontal.

A study has therefore been made by the author of the dip angles of fractures measured in the field. This study has been undertaken for data collected in the Chilterns, the Isle of Thanet, the South Downs and the Isle of Wight.

7.5.2 The Chilterns

The investigation of the strike directions of fractures enabled six main fracture sets to be recognised in the Chilterns (see Section 7.4.2). The dip angles of each of these six joint sets have been analysed separately for localities in the northern Chilterns and for localities in the southern Chilterns. The results are shown diagrammatically for the northern Chilterns
in Fig. 56, for the southern Chilterns in Fig. 57, and for all localities sampled in the Chilterns in Fig. 58. Except for joint set, J3, the mean dip angles of each of these sets are similar for the northern and southern Chilterns (Table IX).

**TABLE IX Results of Analysis of Dip Angles measured in the Chilterns**

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Mean Dip Angle</th>
<th>% of Joints</th>
<th>Mean Dip Angle</th>
<th>% of Joints</th>
<th>Mean Dip Angle</th>
<th>% of Joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>J2</td>
<td>84°27'</td>
<td>17</td>
<td>67°23'</td>
<td>21</td>
<td>85°27'</td>
<td>18</td>
</tr>
<tr>
<td>Jx</td>
<td>88°40'</td>
<td>17</td>
<td>61°40'</td>
<td>14</td>
<td>86°40'</td>
<td>16</td>
</tr>
<tr>
<td>J3</td>
<td>83°20'</td>
<td>7</td>
<td>88°20'</td>
<td>60</td>
<td>88°20'</td>
<td>21</td>
</tr>
<tr>
<td>Jy</td>
<td>89°20'</td>
<td>6</td>
<td>90°20'</td>
<td>36</td>
<td>90°20'</td>
<td>19</td>
</tr>
<tr>
<td>J1</td>
<td>90°00'</td>
<td>6</td>
<td>87°00'</td>
<td>14</td>
<td>90°00'</td>
<td>8</td>
</tr>
<tr>
<td>J4</td>
<td>88°20'</td>
<td>35</td>
<td>89°20'</td>
<td>27</td>
<td>88°20'</td>
<td>32</td>
</tr>
</tbody>
</table>

The results indicate that the dip angles of joints are extremely variable. Although only 1% of all the joints measured in the Chilterns had a dip of 90°, 56% of the joints are inclined at dip angles greater than 80°. The lowest dip angle recorded was 40° but only 2% of joints had a dip angle of less than 50°.

**7.5.3 The Isle of Thanet**

Six fracture sets have been recognised in the Isle of Thanet (Section 7.4.5). The results of an analysis of the dip angles of joints belonging to each of these fracture sets is shown diagrammatically in Fig. 59. The mean dip angles for each of
Fig. 56. Frequency polygons of percentages of joints plotted against their angles of dip for each of the six joint sets recognised at localities in the northern group of those investigated in the Chilterns.
Fig. 57. Frequency polygons of percentages of joints plotted against their angles of dip for each of the six joint sets recognised in the southern group of localities investigated in the Chilterns: (a) Sets $J_1, J_x, J_3$. 
Fig. 57 contd. Frequency polygons of percentages of joints plotted against their angles of dip for each of the six joint sets recognised in the southern group of localities investigated in the Chilterns: (b) Sets $J_y$, $J_1$, $J_4$.
Fig. 58. Frequency polygons of percentages of joints plotted against their angles of dip for each of the six joint sets recognised at localities in the Chilterns.
Fig. 51. Frequency polygons of percentages of joints plotted against their angles of dip for each of the six joint sets recognised at localities in the Isle of Thanet.
these joint sets and the percentage of joints with a dip of 90°
are given in Table X.

TABLE X. Results of Analysis of Dip Angles measured in the
Isle of Thanet

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Mean Dip Angle</th>
<th>% of Joints with dip angle of 90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>J_4</td>
<td>81°E.</td>
<td>20</td>
</tr>
<tr>
<td>J_2</td>
<td>67°E.</td>
<td>9</td>
</tr>
<tr>
<td>J_x</td>
<td>85°S.</td>
<td>11</td>
</tr>
<tr>
<td>J_3</td>
<td>88°S.</td>
<td>11</td>
</tr>
<tr>
<td>J_y</td>
<td>90°</td>
<td>22</td>
</tr>
<tr>
<td>J_1</td>
<td>88°E.</td>
<td>17</td>
</tr>
</tbody>
</table>

The results are similar to those obtained in the Chilteurns:
18% of joints have a dip of 90°, and 60% of the joints are inclined
at dip angles greater than 80°. The lowest dip angle measured for
a joint was 11°, but only 4% of joints had a dip angle of less than 50°.

7.5.4 The South Downs

Six fracture sets were recognised in the South Downs (Section 7.4.4).
The results of an analysis of the dip angles of each of these fractures
sets is shown diagrammatically in Fig. 60. The mean dip angles for
each of these sets and the percentage of joints with a dip of 90° are
given in Table XI.
Fig. 60. Frequency polygons of percentages of joints plotted against their angles of dip for each of the six joint sets recognised at localities in the South Downs.
### TABLE XI. Results of Analysis of Dip Angles measured in the South Downs

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Mean Dip Angle</th>
<th>% of Joints with dip angle of 90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>J_4</td>
<td>88°E</td>
<td>0</td>
</tr>
<tr>
<td>J_2</td>
<td>89°W</td>
<td>11</td>
</tr>
<tr>
<td>J_x</td>
<td>87°N</td>
<td>8</td>
</tr>
<tr>
<td>J_3</td>
<td>89°N</td>
<td>9</td>
</tr>
<tr>
<td>J_y</td>
<td>90°</td>
<td>16</td>
</tr>
<tr>
<td>J_1</td>
<td>88°W</td>
<td>13</td>
</tr>
</tbody>
</table>

The results differ from those obtained in the Chilterns. Only 8% of all the joints measured have a dip of 90°, and only 40% of all the joints are inclined at dip angles greater than 80°. The lowest dip angle measured for a joint was 15°, but only 6% of joints had a dip angle of less than 50°.

### 7.5.5 The Isle of Wight

The results of an analysis of the dip angles of fractures measured in the Isle of Wight is shown diagrammatically in Fig. 61. This shows two main concentrations of dip angles at 70°N and 50°S. The concentration at 70°N corresponds to the bedding. Only 5% of the fractures measured have a dip angle of 90°, and only 11% have a dip angle exceeding 80° (Fig. 62). The lowest dip angle measured for a fracture was 5°, and 33% of the fractures observed had a dip angle of less than 50°. Thus there is a much greater variation in dip angles in the Isle of Wight than in the other areas investigated.
Fig. 61. Frequency polygon of percentage of observed fractures in the Isle of Wight plotted against their dip angles.
Fig. 62. Percentage of fractures observed in the Isle of Wight plotted against their dip angles: (a) all fractures; (b) excluding bedding planes.
7.6 Discussion

The measurement of fracture orientation in the areas investigated indicates that definite fracture patterns may be recognised. These fracture patterns have been shown to be related to the regional and local structural geology. This result is believed to be of some importance. It shows that fractures tend to be confined to particular directions that may be determined in the field. If the structural geology including the directions of the fold axes, is known already then the likely fracture pattern may be predicted prior to field measurements. However, although the main fracture directions may be predicted if sufficient details of the structural geology are known, the relative intensity of the fracturing in these directions is not readily predictable. The survey undertaken in Thanet where one fracture direction was found to be especially dominant, illustrates the variation that may occur.

The directions and angles of dip of the fractures have been found to vary. Although fractures belonging to a particular set do tend to have similar dips, some large variations have been observed. This variation may indicate that a larger number of fracture sets may exist, and that some of the fracture sets recognised in this work may be constituted by what are actually several separate sets of fractures. More detailed investigations would be necessary to establish if this is so.
Although at each locality and in each area the recognition of a number of main fracture directions has been possible, sometimes fractures have been observed that do not readily correspond to those main directions. Therefore, at a particular locality there may occur any number of fractures that are not in predictable directions. The method adopted for the recognition of the fracture sets is based on the determination of a mean strike value, and the particular concentrations may involve fractures with strikes usually varying over about 20° of the compass.

Most fractures observed in the Chalk have been joints. For example, in Thanet faults which were encountered more frequently than elsewhere, represent 5.2% of all the observed fractures. In the Chilterns faults represent 2.6% of the observed fractures. No directional concentrations of these faults has been recognisable.

The recognition of fracture directions is considered to be of importance when assessing the stability of rock slopes. Failure of steep rock slopes normally occurs along pre-existing fracture planes as discussed in Chapters 3 and 5. To assess the stability of a specific rock slope the orientations of the fractures present within that slope are required. If the orientations of these fractures are known a three dimensional assessment of the stability is possible. The orientations of any fracture or fracture set is usually considered in terms of its dip and strike.

The results of the author's work indicate that the main strike directions of the fractures are reasonably constant for a particular
area, defined in terms of geological structure. The author's work has defined these strike directions for certain Chalk areas. The likely strike directions to be encountered within a specific existing slope may be predicted then with some certainty. The strike direction of these fractures may then be used in a stability analysis of the slope under consideration.

The dip angles and directions within the Chalk are somewhat less uniform than the strike directions. However, some important points have emerged from the work. Most fractures are inclined at angles at or near the vertical. For example, in the Chilterns 92% of all observed joints are inclined at angles of 70° or more (as measured from the horizontal).

The average angle for each fracture set is usually 90°, or near to 90°. When the results from the Chilterns, the Isle of Thanet and the South Downs are compared (Table XII), the majority of fractures are found to have dip angles greater than 80°. This probably explains why the majority of failures observed in chalk slopes (Chapter 5) take place by planar rockfalls rather than by sliding. In the South Downs area sliding failures would be expected to be more common than in the Chilterns or Isle of Thanet, since only 40% of joints have dip angles exceeding 80° and 8% of joints have dip angles of less than 50°. There is therefore some variation from area to area.
TABLE XII Comparison of results of analyses of dip angles measured in the Chilterns, Isle of Thanet, South Downs and Isle of Wight.

<table>
<thead>
<tr>
<th></th>
<th>% of fractures with dip angle of 90°</th>
<th>% of fractures with dip angle exceeding 80°</th>
<th>% of fractures with dip angle of less than 50°</th>
<th>Minimum dip angle measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chilterns N.</td>
<td>16</td>
<td>54</td>
<td>3</td>
<td>40°</td>
</tr>
<tr>
<td>S.</td>
<td>26</td>
<td>66</td>
<td>2</td>
<td>45°</td>
</tr>
<tr>
<td>All</td>
<td>19</td>
<td>56</td>
<td>3</td>
<td>40°</td>
</tr>
<tr>
<td>Isle of Thanet</td>
<td>18</td>
<td>60</td>
<td>4</td>
<td>11°</td>
</tr>
<tr>
<td>South Downs</td>
<td>8</td>
<td>40</td>
<td>6</td>
<td>15°</td>
</tr>
<tr>
<td>Isle of Wight</td>
<td>5</td>
<td>11</td>
<td>33</td>
<td>5°</td>
</tr>
</tbody>
</table>

Note. Dip angles for Chilterns, Isle of Thanet and South Downs are those for joints only. Dip angles for the Isle of Wight are for bedding planes and joints.

The results for the Isle of Wight show a significant variation from those obtained for the Chilterns, Isle of Thanet and South Downs. Dip angles of bedding planes have been included in the analysis of fractures observed in the Isle of Wight since, as already mentioned, they control many of the slope failures which have been seen there.

In the Chilterns, Isle of Thanet and South Downs the dip of the bedding rarely exceeds 10° and therefore it does not exert such a significant control on slope failures.

In the Isle of Wight 79% of observed fractures have a dip angle of less than 80°, and sliding failures would therefore be expected to be more common than in the other areas. The field evidence described in Chapter 5 does suggest that sliding failures...
are more frequent in the coastal slopes of the Isle of Wight than in the other areas examined.

7.7 Conclusions

(a) Definite fracture patterns may be recognised within the Chalk.

(b) The fracture patterns recognised are related to regional and local structural geology.

(c) The fracture pattern is consistent over a large area if the geological structure remains constant.

(d) There is a large variation in the dip angles for any one fracture set, but each fracture set does tend to have a mean dip angle of 80-90°.

(e) In the areas of relatively simple structural geology (Chilterns, Isle of Thanet, South Downs) 94-97% of joints have dip angles exceeding 50°.

(f) The dominant dip angles of joints show significant variations in the three areas of simple structural geology examined: vertical joints are most frequent in the Chilterns, whereas joints with dip angles of less than 60° are more frequent in the South Downs.

(g) In the areas of relatively simple structural geology faults are not encountered frequently.

(h) No directional pattern has been recognised for those faults measured.

(i) In the area of more complex structural geology examined (the Isle of Wight) dip angles are more variable than in the areas of simple structural geology, with 33% of all fractures being inclined at dip angles of 5-50°.
6. FRAC TURE SPACING

6.1 Introduction

The variation in the spacing of fractures within the Chalk was considered to be a factor requiring investigation. Fractured rock is less likely to form steep slopes. Several authors have proposed schemes for the description of fracture spacing, for example Deere (1968), Fookes and Denness (1969). That given by Deere has been found useful during field work:

<table>
<thead>
<tr>
<th>Description</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very close</td>
<td>50 mm</td>
</tr>
<tr>
<td>Close</td>
<td>50 - 300 mm</td>
</tr>
<tr>
<td>Moderately close</td>
<td>300 mm - 1 m</td>
</tr>
<tr>
<td>Wide</td>
<td>1 - 3 m</td>
</tr>
<tr>
<td>Very wide</td>
<td>3 m</td>
</tr>
</tbody>
</table>

Attempts have been made to classify chalk into a series of grades as described in Section 4.6. Although recognition of these grades is partly dependent on the amount of fracturing, little work appears to have been undertaken to determine the extent and cause of the variations in fracturing. Early in the research programme field work indicated that the amount of fracturing present is not related only to the lithology.

Two main approaches have been adopted. Firstly, field studies have been undertaken to assess the extent and causes of variations in fracture spacing. Secondly, geophysical techniques were used to investigate the possibility of providing a relatively simple and accurate method for the determination of the fracture spacing.
0.2 Observational field studies

The spacing of fractures has been investigated at a large number of localities. Distinction was made between primary and secondary fracture systems. The primary system consists of the main joints that occur as sets and are therefore fairly constant in direction, and form planes of large areal extent. The primary system also includes some faults. The secondary fracture system consists of minor fractures which do not occur as sets, and each fracture is of limited areal extent. The secondary system is usually found superimposed on the primary system. There are some exceptions, for example, the close fracturing that occurs in marl bands which is a function of the lithology, and the crushed chalk in a finer matrix that may occur along major faults.

8.2.1 Primary fracture system

As these fractures are tectonic in origin their spacing is not greatly affected by depth. Although spacings for each set of fractures are fairly constant at each locality, there is great variation between sets. The average primary fracture spacings recorded at various localities are given in Table XIII. Detailed measurement of spacing between fractures belonging to different sets at each locality would have been interesting because of the variation noted between sets. However, this was not undertaken as part of the present research.

The primary fractures in the Chalk were found to have an average spacing of 0.3 - 1.3 m. The fractures are commonly 0 - 10 mm wide. At localities where the chalk is significantly
<table>
<thead>
<tr>
<th>Locality</th>
<th>Type of slope</th>
<th>Geological horizon with zones (Rowe, 1900)</th>
<th>Lithological description</th>
<th>Primary fracture spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Redbourn, Herts. (TL 124104)</td>
<td>Disused quarry</td>
<td>U. Chalk</td>
<td>Medium-hard white chalk with tabular flint layers</td>
<td>50-700 mm.</td>
</tr>
<tr>
<td>Nuthampstead, Herts. (TL 395328)</td>
<td>Working quarry</td>
<td>U. Chalk</td>
<td>Soft white chalk</td>
<td>1 m.</td>
</tr>
<tr>
<td>Ashwell, Herts. (TL 296401)</td>
<td>Working quarry</td>
<td>Melbourn Rock and Middle Chalk</td>
<td>Hard and medium white chalk</td>
<td>500 mm.</td>
</tr>
<tr>
<td>Litlington, Herts. (TL 316418)</td>
<td>Disused quarry</td>
<td>Lower Chalk, <em>plenus Marls</em>, Melbourn Rock</td>
<td>Marly chalk and hard white chalk</td>
<td>75-200 mm.</td>
</tr>
<tr>
<td>Fort Hill, Margate, Thanet (TR 355714)</td>
<td>Protected sea cliff</td>
<td>U. Chalk: <em>Marsupites testudinarius</em></td>
<td>Soft white chalk</td>
<td>300-600 mm.</td>
</tr>
<tr>
<td>West side of St. Mildred's Bay, Thanet (TR 322705)</td>
<td>Protected sea cliff</td>
<td>U. Chalk: <em>Marsupites testudinarius</em></td>
<td>White chalk</td>
<td>600 mm.</td>
</tr>
<tr>
<td>West side of Westgate Bay, Thanet (TR 3170)</td>
<td>Protected sea cliff</td>
<td>U. Chalk: <em>Marsupites testudinarius</em></td>
<td>White chalk</td>
<td>600 mm.</td>
</tr>
<tr>
<td>Locality</td>
<td>Type of slope</td>
<td>Geological horizon with zones (Rose, 1900)</td>
<td>Lithological description</td>
<td>Primary fracture spacing</td>
</tr>
<tr>
<td>----------------------------------</td>
<td>-------------------</td>
<td>--------------------------------------------</td>
<td>--------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>West side of Palm Bay, Thanet (TR 5771)</td>
<td>Protected sea cliff</td>
<td>U. Chalk: Marsupites testudinarius</td>
<td>White chalk</td>
<td>150-300 mm.</td>
</tr>
<tr>
<td>East side of Palm Bay, Thanet (TR 5871)</td>
<td>Eroding sea cliff</td>
<td>U. Chalk: Marsupites testudinarius</td>
<td>White chalk</td>
<td>225-300 mm.</td>
</tr>
<tr>
<td>Foreness Point, Thanet (TR 5871)</td>
<td>Eroding sea cliff</td>
<td>U. Chalk: Marsupites testudinarius</td>
<td>White chalk</td>
<td>300 mm.</td>
</tr>
<tr>
<td>Joss Bay, Thanet (TR 4070)</td>
<td>Eroding sea cliff</td>
<td>U. Chalk: Marsupites testudinarius</td>
<td>White chalk</td>
<td>0.3-1.0 m.</td>
</tr>
<tr>
<td>North Cliff, Broadstairs, Thanet (TR 3958)</td>
<td>Eroding sea cliff</td>
<td>U. Chalk: Marsupites testudinarius</td>
<td>White chalk</td>
<td>600 mm.</td>
</tr>
<tr>
<td>Dumpton Gap, Thanet (TR 3966)</td>
<td>Eroding sea cliff</td>
<td>U. Chalk: Marsupites testudinarius</td>
<td>White chalk</td>
<td>300-600 mm.</td>
</tr>
</tbody>
</table>
### TABLE XIII. (continued) Examples of average primary fracture spacings in the Chalk

<table>
<thead>
<tr>
<th>Locality</th>
<th>Type of slope</th>
<th>Geological horizon with zones (Rowe, 1900)</th>
<th>Lithological description</th>
<th>Primary fracture spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>East Cliff, Ramsgate, Thanet (TR 5965)</td>
<td>Protected sea cliff</td>
<td>U. Chalk: Marssonites testudinarius, Micraster ocarinaulum</td>
<td>Hard chalk with nodular flints</td>
<td>300-600 mm.</td>
</tr>
<tr>
<td>West Cliff, Ramsgate, Thanet (TR 7564)</td>
<td>Protected sea cliff</td>
<td>U. Chalk: Micraster ocarinaulum</td>
<td>White chalk</td>
<td>150-300 mm.</td>
</tr>
<tr>
<td>St. Margaret's Bay, Kent (TR 7543)</td>
<td>Eroding sea cliff</td>
<td>U. Chalk: Micraster ocarinaulum</td>
<td>White chalk with flint nodules</td>
<td>1.0-1.3 m.</td>
</tr>
<tr>
<td>Dover, Kent.</td>
<td>Protected sea cliff</td>
<td>U. Chalk: Micraster ocarinaulum, Micraster corynestudinarius</td>
<td>Hard white chalk</td>
<td>1.0-1.5 m.</td>
</tr>
<tr>
<td>Lenham, Kent. (TQ 913525)</td>
<td>Disused quarry</td>
<td>M. or U. Chalk</td>
<td>White chalk</td>
<td>1.0-2.0 m.</td>
</tr>
<tr>
<td>Lewes, Sussex (TQ 423404)</td>
<td>Highway cut</td>
<td>M. Chalk</td>
<td>Hard white and grey chalk without flints</td>
<td>0.3-1.0 m.</td>
</tr>
<tr>
<td>Lewes, Sussex. (TQ 425099)</td>
<td>Disused quarry</td>
<td>M. Chalk</td>
<td>Hard white chalk</td>
<td>150-300 mm.</td>
</tr>
</tbody>
</table>
softer or marly in character the average spacing is much reduced. In such cases the spacing is usually 75 - 300 mm. However, the primary fractures may usually be traced from normal chalk through the marl band and into normal chalk again. The primary fractures are less distinct in the marl bands because of the other numerous fractures present.

In the vicinity of the more important faults the primary fracture spacing has been observed to decrease markedly. In addition to an increase in the fracture intensity the jointing tends to change so that it is inclined at the same angle as the fault-plane. This is particularly well seen in the cliff section at Pegwell Bay, Thanet.
8.2.2 Secondary fracture system

The secondary fracture system is variable in its development. Where the chalk is extremely hard then, even in slopes subject to weathering, there may be no development of secondary fractures. In a pit at Barkway in north-east Hertfordshire the Top Rock is exposed but exhibits few primary fractures and no secondary fractures at all. In fresh exposures secondary fractures are normally absent. Presumably they are developed as a result of exposure. The two processes that the author believes to result in the present day development of these secondary fractures are stress relief and weathering. Secondary fractures will be present even in freshly excavated slopes if the chalk forming those slopes has undergone weathering or movement in geological time. Marly chalks also exhibit very close fracturing as do some of the brecciated chalks occurring along fault-planes. The extent of development of the secondary fracture system would therefore seem to depend primarily on the length of exposure to weathering, the intensity of that weathering, and the lithology of the rock itself.

The average spacing between secondary fractures has been observed to be 20 - 200 mm. The average secondary fracture spacings recorded at various localities are given in Table XIV. In many exposed faces where the chalk appears very shattered, the joints have been found to close up rapidly away from the face. At an old quarry at Merstham, Surrey, narrowly spaced, open fractures were found to close up completely at a depth of only 150 mm from the face. In most cases the secondary fracturing is absent or only poorly developed below a depth of 1 - 2 m.
## TABLE XIV. Examples of average secondary fracture spacings in the Chalk

<table>
<thead>
<tr>
<th>Locality</th>
<th>Type of slope</th>
<th>Geological horizon with zones where known</th>
<th>Lithological description</th>
<th>Secondary fracture spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Holaster planus</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>M. Chalk: <em>Terebratulina lata</em></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### TABLE XIV. (continued) Examples of average secondary fracture spacings in the Chalk

<table>
<thead>
<tr>
<th>Locality</th>
<th>Type of slope</th>
<th>Geological horizon with zones where known</th>
<th>Lithological description</th>
<th>Secondary fracture spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pegsdon Valley, Herts.</td>
<td>Trial pit</td>
<td>M. Chalk</td>
<td>Blocky white chalk</td>
<td>30-100 mm.</td>
</tr>
<tr>
<td>(TL 119294)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lewes, Sussex.</td>
<td>Disused quarry</td>
<td>M. Chalk</td>
<td>Hard white chalk</td>
<td>150-300 mm.</td>
</tr>
<tr>
<td>(TQ 425099)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>South Mims, Herts.</td>
<td>Working quarry</td>
<td>J. Chalk: Micraster coranguinum</td>
<td>White chalk</td>
<td>20-100 mm.</td>
</tr>
<tr>
<td>(TL 228025)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
8.3 Geophysical field investigations

Geophysical techniques have been used by the author to attempt to provide a method for the in situ determination of fracture spacing. The use of seismic refraction surveys to measure compressional seismic velocities, and the correlation of these velocities with in situ rock quality, has been described by several authors, see for example Deere (1968) and Coon and Merritt (1970). The seismic velocity has been found to depend primarily on the lithology, the moisture content, and the degree of fracturing present. Seismic velocity measurements have therefore been made in chalk of different types, and the velocities determined have been compared with the observed fracturing.

8.3.1 Method

The compressional seismic velocities were determined using an Huntec FS3 seismograph. Determinations were made from the surface using a hammer and steel plate as the source of energy. At nearly all localities clear seismic records were obtained (Fig. 63). The localities selected included the sites of the two in situ excavations described in Chapter 6. Other localities were chosen to provide results on a range of chalk lithologies.

8.3.2 Results at trial excavations

At the site of the monitored pit at South Mimms, seismic traverses were undertaken prior to its excavation. Two traverses were made: one in a north-south direction and the other in an east-west direction. The north-south traverse gave two separate velocities: $V_1 = 910 \text{ m/sec}$ and $V_2 = 1390 \text{ m/sec}$. These velocities indicated the presence of an interface at a depth of 4.8 metres.
Fig. 63. Typical seismic record.
Plan of traverses

Table XV. Results of seismic traverses at Pegsdon.

<table>
<thead>
<tr>
<th>Before excavation</th>
<th>V&lt;sub&gt;1&lt;/sub&gt;: 375 m/sec</th>
<th>V&lt;sub&gt;2&lt;/sub&gt;: 375 m/sec</th>
<th>V&lt;sub&gt;3&lt;/sub&gt;: 300 m/sec</th>
<th>V&lt;sub&gt;4&lt;/sub&gt;: 150 m/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>8-12-71</td>
<td>315 m</td>
<td>315 m</td>
<td>300 m</td>
<td>300 m</td>
</tr>
<tr>
<td></td>
<td>1.6 m</td>
<td>1.6 m</td>
<td>1.8 m</td>
<td>1.8 m</td>
</tr>
<tr>
<td></td>
<td>V&lt;sub&gt;5&lt;/sub&gt;: 510 m/sec</td>
<td>V&lt;sub&gt;6&lt;/sub&gt;: 300 m/sec</td>
<td>V&lt;sub&gt;7&lt;/sub&gt;: 750 m/sec</td>
<td>V&lt;sub&gt;8&lt;/sub&gt;: 600 m/sec</td>
</tr>
<tr>
<td></td>
<td>7.5 m</td>
<td>7.5 m</td>
<td>7.5 m</td>
<td>7.5 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>After excavation</th>
<th>V&lt;sub&gt;1&lt;/sub&gt;: 630 m/sec</th>
<th>V&lt;sub&gt;2&lt;/sub&gt;: 510 m/sec</th>
<th>V&lt;sub&gt;3&lt;/sub&gt;: 375 m/sec</th>
<th>V&lt;sub&gt;4&lt;/sub&gt;: 510 m/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>30-6-72</td>
<td>630 m</td>
<td>510 m</td>
<td>375 m</td>
<td>510 m</td>
</tr>
<tr>
<td></td>
<td>1.7 m</td>
<td>1.6 m</td>
<td>1.6 m</td>
<td>1.6 m</td>
</tr>
<tr>
<td></td>
<td>V&lt;sub&gt;5&lt;/sub&gt;: 750 m/sec</td>
<td>V&lt;sub&gt;6&lt;/sub&gt;: 750 m/sec</td>
<td>V&lt;sub&gt;7&lt;/sub&gt;: 750 m/sec</td>
<td>V&lt;sub&gt;8&lt;/sub&gt;: 750 m/sec</td>
</tr>
<tr>
<td></td>
<td>7.5 m</td>
<td>7.5 m</td>
<td>7.5 m</td>
<td>7.5 m</td>
</tr>
</tbody>
</table>
The east-west traverse gave a single velocity $V_1 = 955 \text{ m/sec}$.

After excavation of the trial pit, the seismic traverses were repeated. The north-south traverse gave a seismic velocity, $V_1 = 955 \text{ m/sec}$. The east-west traverse gave a seismic velocity, $V_1 = 855 \text{ m/sec}$. Initially, the change in velocity was considered to be possibly due to the inward movement of the walls of the pit, or to a change in the moisture content of the chalk. A large inward movement of the sides of the pit might have caused opening of fractures in the surrounding chalk and this would have led to a decrease in the seismic velocity. Alternatively, an increase in the moisture content would have resulted in an increased seismic velocity. As the results were not conclusive, since in one direction an increase in velocity occurred over the period and in the other case a decrease occurred, further measurements were made in connection with the excavation of the second trial pit at Pegsdon.

Seismic traverses were made in four directions using the site of the proposed excavations as centre. These traverses were then repeated after excavation of the trial pit. The results of these seismic traverses are given in Table XV. The results indicate a marked increase in seismic velocity after excavation. Before excavation two main groups of seismic velocity were recorded: a $V_1$ of about 300 - 375 m/sec and a $V_2$ of 600 - 750 m/sec. The velocities indicated the presence of two or possibly three layers. A significant interface was recorded at a depth of about 1.6 m.
6.3.3 Results at other localities

The seismic refraction results obtained at South Mimms and Peganon suggested that the seismic velocities were related to the amount of fracturing present in the chalk. Measurements of seismic velocity were therefore undertaken at a number of localities on different types of chalk. The results are given in Table XVI.

6.3.4 Directional variation of fracture spacing

The work described in Chapter 7 revealed that fracturing in the chalk usually conforms to a well-marked pattern. This suggested that the fracture spacing might vary according to direction. The possibility was therefore thought to exist that the seismic velocities measured might depend on the direction in which the seismic traverse was carried out.

To investigate the variation of seismic velocity in the chalk according to direction a number of seismic refraction traverses were made at a chalk quarry at Nuthampstead, Hertfordshire. The traverses were made on a flat surface from which all overburden and loose chalk had been removed. The quarry exposed soft Upper Chalk. The work was undertaken in November when the water table in the chalk in this area is normally at its lowest, and hence variations in seismic velocity due to moisture near the surface are probably at a minimum. A borehole in the same valley 4 Km. south of the quarry, and at a similar altitude of 105 m., had a rest water level of 70.9 m. in September 1964 (Cole, 1970).
### TABLE XVI. Results of seismic velocity determinations at different localities

<table>
<thead>
<tr>
<th>Locality</th>
<th>Terrain</th>
<th>Geological horizon with zones where known</th>
<th>Measured velocities</th>
<th>Observed lithologies</th>
<th>Observed average fracture spacing</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Redbourn, Herts.</td>
<td>Level surface above disused quarry</td>
<td>U. Chalk</td>
<td>$v_1 = 500$ m/sec.</td>
<td>Medium and hard white chalk with tabular flint layers</td>
<td>50-700 mm.</td>
<td>June 1972</td>
</tr>
<tr>
<td>Hatfield, Herts.</td>
<td>Gently sloping ground</td>
<td>U. Chalk</td>
<td>$v_1 = 455$ m/sec.</td>
<td>Soft white chalk</td>
<td>50-500 mm.</td>
<td>January 1972</td>
</tr>
<tr>
<td>Pitstone, Beds.</td>
<td>Level surface above working quarry (1 m. below level of <em>plenum</em> Marls)</td>
<td>L. Chalk: <em>Holaster supraboobus</em></td>
<td>$v_1 = 1250$ m/sec.</td>
<td>Medium and hard white and grey chalk</td>
<td>0.5 - 1.0 m.</td>
<td>July 1972</td>
</tr>
<tr>
<td>Locality</td>
<td>Terrain</td>
<td>Geological horizon with zones where known</td>
<td>Measured velocities</td>
<td>Observed lithologies</td>
<td>Observed average fracture spacing</td>
<td>Date</td>
</tr>
<tr>
<td>------------------</td>
<td>--------------------------</td>
<td>--------------------------------------------</td>
<td>---------------------</td>
<td>----------------------</td>
<td>----------------------------------</td>
<td>------------</td>
</tr>
<tr>
<td>Pitstone, Beds.</td>
<td>Above feature made by Melbourn</td>
<td>Inoceramus labiatus and Melbourn Rock</td>
<td>( V_1 = 800 \text{ m/sec.} )</td>
<td>Medium and hard white chalk</td>
<td>0.5-1.0 m.</td>
<td>July 1972</td>
</tr>
<tr>
<td>Hardy's Monument, Dorset.</td>
<td>Level surface on chalk ridge</td>
<td>Gravel (Bagshot Beds?) ( V_1 = 800 \text{ m/sec.} ) (gravel?)</td>
<td>( V_2 = 2000-2500 \text{ m/sec.} )</td>
<td>No exposure</td>
<td>November 1972</td>
<td></td>
</tr>
<tr>
<td>Beer, Devon.</td>
<td>Level surface above sea cliffs</td>
<td>Holaster planus ( 6 \text{ m.} ) ( V_1 = 750 \text{ m/sec.} )</td>
<td>( V_2 = 1800 \text{ m/sec.} )</td>
<td>White chalk</td>
<td>0.5-1.0 m.</td>
<td>November 1973</td>
</tr>
</tbody>
</table>
The traverses were made in four directions: NO°, N45°E., N90°E., and N45°W. The results are given in Table XVII.

**TABLE XVII. Variation of seismic velocity with direction at Nuthampstead.**

<table>
<thead>
<tr>
<th>Direction of traverse</th>
<th>Seismic velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>NO°</td>
<td>785 m/sec.</td>
</tr>
<tr>
<td>N45°E.</td>
<td>475 m/sec.</td>
</tr>
<tr>
<td>N90°E.</td>
<td>600 m/sec.</td>
</tr>
<tr>
<td>N45°W.</td>
<td>570 m/sec.</td>
</tr>
</tbody>
</table>

In Figure 64 the velocities are plotted as a seismic ellipse. The ellipse illustrates the higher velocity in the north-south direction, the intermediate velocity in the east-west direction, and the lower velocities in the N.E.-S.W. and N.W.-S.E. directions.

Measurements of the dip and strike of fractures were made in exposed faces of chalk surrounding the area used for the seismic traverses. A stereogram of these fractures is shown in Figure 66.

The average fracture spacing for each main set of fractures was also measured. These spacings were found to be: for N.-S. fractures, 0.5 m.; for N.W.-S.E. fractures, 1.25 m.; for E.-W. fractures, 1.25 m. Almost all E.-W. fractures and some N.W.-S.E. fractures contained flints, mostly broken, and clay. The N.-S. fractures did not contain flints. Many N.W.-S.E. joints were limonite-stained.

If these fractures are plotted schematically with a fracture spacing corresponding to that measured in the field, the grid shown in Figure 65 is obtained. Lines A, B, C, and D correspond to the directions of the seismic traverses. This diagram explains partly why higher velocities are obtained in certain directions. In traverse A for a set distance only three fractures are crossed and a high velocity would be expected. In traverse B five
Fig. 64. Seismic ellipse for Upper Chalk at Nuthampstead, Herts.

Fig. 65. Relationship between seismic traverses and fracture spacings at Nuthampstead, Herts.
I. N-S. Fractures
2. N.W-S.E. Fractures
3. E-W. Fractures.

Fig. 66. Stereogram of typical fractures observed at Nuthampstead, Herts.
fractures are crossed and a lower velocity would be obtained. 
In traverse C six fractures are crossed and an even lower velocity 
would be obtained. Traverse D is an exception since only four 
fractures are crossed, and yet, in practice, a low seismic velocity 
was obtained. However, this low velocity might be due to the 
profile line corresponding to a fracture line during the seismic 
traverse. The N.W.-S.E. fractures were noted to be particularly 
well-marked; in most cases being open fractures with limonitic 
surfaces.

6.3.5 Discussion of geophysical field results

The seismic velocity determinations made at the in situ 
excavations at South Mimms and Pegsdon indicate that chalk does 
not give a single seismic velocity. Instead variations occur 
that appear to be related to the state of the chalk. A gradual 
increase in velocity with depth is not observed. Sudden changes 
in velocity with depth are found to exist, presumably at interfaces 
between different types of chalk. At Pegsdon significantly higher 
velocities were recorded in the near surface chalk along the traverse 
lines in June, as compared with those observed in the previous 
December. In early December the water-table in the chalk is 
comparatively low whereas in June it is high. The increased seismic 
velocity measured in June is therefore thought to reflect the 
markedly higher moisture content of the chalk at that time. The 
chalk with a high velocity, 750 m/sec, appeared to be unaffected 
by the possible increase in moisture content. The movements 
oberved in the in situ excavations are considered to be too small
to have any effect on the seismic velocity of the surrounding chalk.

Since the author's work was undertaken, Grainger, McCann and Gallois (1973) have published a paper that describes similar changes in seismic velocity thought to be due to the effects of saturation. They found that only the more rubbly chalk (of their grades IV and V) was affected. The velocity of structureless and unjointed chalk (grade V) was raised from 700 m/sec. to 1950 m/sec.

The results obtained from the localities investigated by the author suggest that four main seismic velocities are typical of different types of chalk: 500 m/sec; 700-800 m/sec; 1000-1250 m/sec; and 1800-2500 m/sec. Velocities of about 500 m/sec have been obtained only on Upper Chalk and appear typical of very soft chalk or loose near surface chalk. The range 700-800 m/sec. has been obtained particularly on chalk that fractures into roughly rectangular blocks with side dimensions of 10-100 mm. Velocities of 1000-1250 m/sec were recorded on chalk in which the joints were more widely spaced, generally about 1 m. apart. Of the highest velocities measured, 1800-2500 m/sec., two measurements were obtained on the Upper Chalk of West Dorset which is often harder and denser than that of South East England. A high velocity of 2222 m/sec. was obtained on the Melbourn Rock at Pitstone, Bedfordshire.

The results of the traverses at Nuthampstead, Hertfordshire, indicate that where a definite fracture pattern is present in the chalk, the seismic velocities obtained depend upon the direction in

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which the traverse is undertaken. The seismic velocity is significantly reduced if the traverse is made in such a direction that a large number of fractures are crossed.

8.4 Geophysical laboratory investigations

The in situ measurements indicated that the seismic velocity within the chalk depends on two main variables. These are the amount of fracturing and the degree of saturation. To investigate in greater detail the effects of these two factors on the chalk, tests were undertaken in the laboratory where measurements could be more closely controlled than in the field.

8.4.1 Method

The velocities were measured in the laboratory using Pundit ultrasonic equipment manufactured by C.H.S. Instruments Ltd. The chalk samples used were 76 mm diameter cores extracted from block samples of chalk obtained from the Upper Chalk at Redbourn, Hertfordshire. Four series of tests have been undertaken. Two series of tests (Series A and B) were undertaken on cores of chalk with a variable number of horizontal fractures across the diameter of the cores. The horizontal fractures were produced by sawing of originally intact cores. One core of chalk was tested intact; a second had one horizontal fracture at mid-height; a third core was separated into three equal portions by two horizontal fractures; and a fourth core was separated into four equal portions by three horizontal fractures. The first set of tests (Series A) was undertaken on dry cores (moisture content 0%). The cores were
then soaked for 24 hours and tested in a saturated state (Series B).

Velocity determinations were also made on four cores that had been previously tested to failure in unconfined compression. Consequently, these cores possessed vertical fractures, some of which were closed, others open to a maximum of 1 mm. The cores were tested in a dry condition (moisture content 0%). The velocity was determined along the length of each core (Series C) and across the diameter of each core (Series D).

8.4.2 Results of ultrasonic tests

The results of the tests are given in Table XVIII. The tests on dry cores of chalk (Series A) indicate that there is a marked decrease in velocity as the number of fractures across the core is increased. This decrease in velocity was particularly great when three fractures were present across the core. Apart from the core of intact chalk the effect of saturation was to increase the velocity as compared with the dry state. Results of tests on the cores that had previously been tested to failure in unconfined compression, and consequently contained fractures, indicated that the largest decrease in velocity occurred when the fractures were open.

8.5 Discussion

The distinction between primary and secondary fracture systems is believed to be important. The results of the field investigation at the trial pit at South Mimas indicated that a secondary fracture system may develop near the face of an.
TABLE XVIII. Results of Ultrasonic Tests on Chalk Cores

<table>
<thead>
<tr>
<th>Ref. No.</th>
<th>State of Core</th>
<th>Velocity (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Series A Dry</td>
</tr>
<tr>
<td>1</td>
<td>Intact</td>
<td>2055</td>
</tr>
<tr>
<td>2</td>
<td>1 horizontal fracture at mid-height.</td>
<td>1904</td>
</tr>
<tr>
<td>3</td>
<td>2 horizontal fractures separating 3 equal portions of core.</td>
<td>1495</td>
</tr>
<tr>
<td>4</td>
<td>3 horizontal fractures separating 4 equal portions of core.</td>
<td>760-866</td>
</tr>
</tbody>
</table>

Series C and D

<table>
<thead>
<tr>
<th>Ref. No.</th>
<th>State of Core</th>
<th>Velocity (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Series C Across length of core</td>
</tr>
<tr>
<td>5</td>
<td>Open vertical fractures</td>
<td>1720-1661</td>
</tr>
<tr>
<td>6</td>
<td>Intact - 2 vertical cracks open to 0.25mm.</td>
<td>2052</td>
</tr>
<tr>
<td>7</td>
<td>Intact - 4 vertical cracks open to 0.25-0.5mm.</td>
<td>1683</td>
</tr>
<tr>
<td>8</td>
<td>Several vertical cracks, some open, some closed.</td>
<td>2222</td>
</tr>
</tbody>
</table>
excavation within two years of its exposure (Section 6.9). In contrast, at some localities where hard chalk has been exposed for many years no secondary fractures have developed. Even where a secondary fracture system is present it seldom extends more than 1-2 m. away from the face. This is approximately the maximum depth to which frost action operates in this country.

The seismic velocity measurements at various localities showed that the seismic method is able to detect different types of chalk. The range of seismic velocities obtained on chalk by the author is 300-2500 m./sec. This range in velocity has enabled the four main varieties of chalk described in Section 8.3.5 to be distinguished. The field results also indicated that the seismic velocity in loose and rubbly chalk is increased significantly by a rise in moisture content. The laboratory ultrasonic tests are in support of this. In the case of fractured samples of chalk the velocity was increased by up to 950 m./sec. as a result of saturation. Intact and slightly fractured samples exhibited only a small change in velocity when saturated.

Some correlation is possible between the velocities determined in the field and those obtained in the laboratory (Table XIX). Velocities for chalk with wide fracture spacing are similar in the laboratory and in the field, being about 2000 m./sec. Velocities for closely fractured chalk were also similar when measured in the field and in a dry condition in the laboratory, being generally about 800 m./sec. The author suggests that further work on the
TABLE XIX. Comparison of Laboratory Ultrasound and Field Seismic Velocity Measurements on Chalk

<table>
<thead>
<tr>
<th>Type of chalk</th>
<th>State of fracturing</th>
<th>Velocity in dry condition m/sec.</th>
<th>Velocity in wet condition m/sec.</th>
<th>Velocity in field m/sec.</th>
<th>Type of chalk</th>
<th>State of fracturing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal white</td>
<td>Intact</td>
<td>2055</td>
<td>1960</td>
<td>1800-2250</td>
<td>Hard chalk</td>
<td>Wide fracture spacing, 1-3 m.</td>
</tr>
<tr>
<td>Upper Chalk</td>
<td>1 fracture Fracture spacing 35 mm.</td>
<td>1904</td>
<td>2050</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal white</td>
<td>2 fractures Fracture spacing 23 mm.</td>
<td>1495</td>
<td>1870</td>
<td>1000-1250</td>
<td>Normal white chalk</td>
<td>Moderate fracture spacing, 1 m.</td>
</tr>
<tr>
<td>Type of chalk</td>
<td>State of fracturing</td>
<td>Velocity in dry condition m/sec.</td>
<td>Velocity in wet condition m/sec.</td>
<td>Velocity in field m/sec.</td>
<td>Type of chalk</td>
<td>State of fracturing</td>
</tr>
<tr>
<td>--------------</td>
<td>---------------------</td>
<td>---------------------------------</td>
<td>---------------------------------</td>
<td>------------------------</td>
<td>--------------</td>
<td>---------------------</td>
</tr>
<tr>
<td>Normal white Chalk</td>
<td>3 fractures Fracture spacing 18 mm.</td>
<td>760-866</td>
<td>1050-1710</td>
<td>700-800</td>
<td>Normal white chalk</td>
<td>Close fracture spacing, 10-100 mm.</td>
</tr>
<tr>
<td>Upper Chalk</td>
<td></td>
<td></td>
<td></td>
<td>500</td>
<td>Loose near surface chalk</td>
<td>Very close fracture spacing, less than 10mm.</td>
</tr>
<tr>
<td>Chalk</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Loose particles and fragments.</td>
<td></td>
</tr>
</tbody>
</table>

Note. The characteristics of the chalk used in the ultrasonic tests were as follows: \( \rho = 1550 \text{ kg/m}^3 \); \( m = 26-29 \% \); \( E = 4300-4500 \text{ MPa/m}^2 \); \( \nu = 0.25 \); \( q_u = 1.5-2.2 \text{ MPa/m}^2 \).
correlation of field and laboratory velocity determinations would be profitable. If a better correlation was to be developed, the accurate recognition of different fracture spacings in rock masses in the field would become possible.

The author's work at Nuthampstead, Hertfordshire, on the directional variation of velocity suggests that the velocities determined by the seismic method in the field are very dependent on the fracture spacing. The range in velocities measured on chalk is considered to be sufficiently large for relatively small variations in the fracture spacing to be recognised. These velocities are believed by the author to be sensitive to changes in the fracture spacing because of the relative homogeneity of the chalk in any one area. Where the hardness of the chalk varies greatly then seismic velocities are likely to change independently of the fracture spacing.

8.6 Conclusions

(a) A distinction may be made between primary and secondary fracture systems.

(b) Secondary fracture systems may develop near the face of an excavation within two years of exposure.

(c) Secondary fracture systems rarely extend more than 1-2 m. back from the face.

(d) Four types of chalk are recognisable on the basis of field seismic velocity determinations:

   (i) 500 m./sec. - soft or loose near surface chalk.
(ii) 700-800 m./sec. - blocky chalk with close fracture spacing of 10 - 100 mm.;

(iii) 1000-1250 m./sec. - normal chalk with moderate fracture spacing of about 1m.;

(iv) 1800-2500 m./sec. - hard chalk with wide fracture spacing of 1-3m.

(e) The seismic velocity in loose chalk is increased significantly by an increased moisture content.

(f) The seismic velocity in chalk with moderate or wide fracture spacing is not significantly affected by changes in moisture content.

(g) The seismic velocity in chalk varies with:
   (i) the hardness;
   (ii) the fracture spacing.

(h) Directional variations in seismic velocity corresponding to directional variations in fracture spacing may be recognised if the hardness of the chalk is relatively uniform in the area under consideration.

(i) Field seismic velocity determinations correlated with laboratory ultrasonic measurements may provide an accurate and inexpensive method for the recognition of fracture spacing in rock.
9. FRACTURE SURFACES IN THE CHALK

9.1 Introduction

The shear strength of intact rock is known to be greatly reduced by the presence of fracture planes. This is particularly so when the direction of shear is parallel or nearly parallel to these planes. However, the likelihood of movement occurring along these planes is affected by the exact nature of the fracture surfaces present.

Hoek (1970) has drawn attention to the importance of assessing fracture roughness. He also describes the work of Patton (1966) who has shown that the angle of the potential failure planes in slopes is dependent on the roughness of the failure surface.

The nature of fracture surfaces has been investigated by the author in two ways. Firstly, during the field investigations the types of failure surfaces encountered were recorded. Secondly, laboratory tests have been conducted on fracture surfaces.

9.2 Field studies

Three main types of joint surfaces have been described by Duncan (1969). These are qualitatively described as smooth, rough or keyed. Fookes and Denness (1969) have proposed a more detailed classification. Field work during the present investigation has revealed a number of different types of fractures in the Chalk. These fractures have been classified mainly according to the type of surface present. Some of these types correspond to the types defined by Fookes and Denness. The main types of fracture
surfaces observed are described below in 9.2.1 - 9.2.6. In addition, three main types of filled fractures have been observed and these are described in 9.2.7 - 9.2.9.

9.2.1 Smooth surfaces

Most main fractures are straight and bounded by smooth, white surfaces of chalk. All such fractures observed have been joints. Joints with surfaces of this type have been observed at all localities visited.

9.2.2 Limonite stained surfaces

Most major fractures have been observed to be bounded by surfaces that are stained by limonite, and in a few cases possibly hematite. The extent of the limonite staining is variable. In some exposures it is very extensive and gives rise to very prominent yellow-brown surfaces. Field work indicates that the limonite tends to occur where the fractures are particularly open. The limonite, hydrated iron oxide, could have been formed by the alteration of iron-rich minerals in the Chalk or, alternatively, it may have been introduced from overlying deposits. The author considers that deposition of the limonite took place from water percolating along the fractures. At localities where fracture surfaces of this type occur, all major fractures have usually been affected. This suggests that these surfaces have been formed by the movement of large quantities of water through the Chalk. In addition to being open such fractures are usually very extensive (Plate V).
9.2.3 Polished surfaces

Polished surfaces have been observed only in the harder varieties of chalk, and in chalk that has been subject to severe tectonic movements. They are of infrequent occurrence. Some surfaces of this type have exhibited traces of slickensiding. The polished nature of these surfaces may therefore be the result of the movements of one surface of the fracture over the other surface.

9.2.4 Slickensided surfaces

Slickensided surfaces have been observed at many localities. Such surfaces are not confined to the areas which have suffered especially from tectonic movements such as South Dorset and the Isle of Wight. They have been observed at localities throughout the Chalk outcrops studied. Although best developed in the harder types of chalk they also occur in soft chalk.

The exact nature of the slickensided surfaces varies. Some slickensides are indistinct and grade into the polished surfaces described in Section 9.2.3. The slickensides sometimes take the form of grooves and ridges. The maximum amplitude of the ridges above the grooves observed has been 3mm. Crests of ridges are usually 3 - 10 mm apart. The ridges and grooves forming the slickensides are often of chalk only, but some are partly or completely composed of the following: limonite; calcite; a black unidentified mineral; a greenish unidentified mineral (perhaps green calcite).

Some slickensides occur along fractures which have displaced
the bedding of the chalk. These faults usually have only small
downthrows, typically 0.5 - 2.0 metres. The ridges and grooves are
always orientated parallel to the direction of probable movement.
Slickensides have also been observed along fractures which do not
appear to be associated with any displacement of the chalk on either
side of the fracture plane.

9.2.5 Powdery surfaces

Powdery surfaces appear to occur particularly in soft chalk or
chalk that is wet. The powder is usually of very fine particles of
chalk, although some small angular fragments of chalk do occur
occasionally. The powder tends to adhere strongly to the adjacent chalk.
Powdery surfaces have been observed along joints and faults.

9.2.6 Interlocking surfaces

Surfaces of this type have been observed much less frequently
in the Chalk than in other competent rocks. Some minor fractures
exhibit conchoidal surfaces which are somewhat interlocking, and a
few irregular surfaces have been observed. However, no major joints
with interlocking surfaces have been seen, and only a few faults
show surfaces of this type. The general softness of the Chalk
presumably causes smooth fractures to develop. Internal erosion
by percolating water may also assist in the creation of smooth
fracture surfaces.

9.2.7 Chalk filled fractures

Some fractures observed have been filled with chalk fragments
and powder. In some instances fragments and powder have become cemented together to form a hard rock, which has been termed calcrete. Such fractures seem to be of two main types. Firstly, they may be open joints that have become filled with loose chalk. These are usually near surface features and do not extend far below the surface, perhaps 30-40 metres in extreme cases. Secondly, these fractures may be fault planes along which shearing has occurred and a breccia has been formed. Unlike many fault breccias the larger chalk fragments involved are rounded and not angular. The chalk has presumably been rounded during the brecciation process and this appears again to reflect the relative softness of the chalk.

9.2.8 Clay and sand filled fractures

Like the chalk filled fractures, these have been observed only occasionally. Some fractures are filled with clay only, some with sand only, others with mixtures of clay, sand and flint pebbles. Also like the chalk filled fractures, they are of two types. Firstly, they may be near surface features allied to swallow holes and pipes. These features are not discussed in detail here since their characteristics have been described elsewhere (for example, West and Dumbleton, 1972) and their possible effects on slope stability are discussed in Sections 5.3.1 and 5.4 of the present work. Secondly, these filled fractures may occur apparently unrelated to near surface swallow holes.

The latter type of fractures are usually filled with clay only or with clay and some silt. The fractures may be either joints or faults. Presumably the clay has been introduced from
above, and deposited in the once open fractures by water. At some localities sub-parallel near horizontal clay filled fractures have been observed. In North Hertfordshire and South Cambridgeshire these have been described by Bromley (1967) as horizontal thrust planes formed by ice action in the Pleistocene. The present author, however, believes that many such clay horizons may represent the changing levels of the water table within the chalk, probably at a time when the water table was much higher than at present. At Barkway, Hertfordshire, a 0.8 m. band of chalky boulder clay occurs along a plane inclined at 50°W. which is also believed to be a thrust plane formed in Pleistocene times (Bromley, 1967).

9.2.9 Flint filled fractures
At several localities the chalk is traversed by fractures filled with broken flints. Such fractures have been found always to be faults. The flints are crushed as a result of the fracturing that occurs during their formation. The crushed flints may occur alone or more often they are surrounded by chalk fragments and powder. If near the surface (less than a depth of 10 m.) the crushed flints sometimes occur in brown clay. At some localities crushed flints occur apparently unrelated to any fracture planes.

9.2.10 Water flow along fractures
The surfaces of most fractures observed in the field have been moist. Even when the chalk bounding the fractures has been dry, excavation to about 0.2 – 0.5 m. has usually revealed the chalk behind the face to be moist. The author has often found the
natural moisture content of chalk to be close to its saturation value (Section 6.6.5).

Although the fracture surfaces are normally moist, free water moving along fractures has been encountered at a few localities only. Near Ranmore in Surrey water has been observed emerging from chalk near the base of a road cutting. On emergence the water has deposited chalk presumably eroded from within the slope (Farrar, 1973).

At Alum Bay, Isle of Wight, water has been seen to emerge from a widened joint. The opening in the face of the slope measured about 0.7 m. wide and 1.0 m. high. The joint had a strike of N30°W and a dip of 90°. The joint has presumably been enlarged as a result of solution and erosion by the water flowing along it. Although on emergence the water is at beach level, the joint was sufficiently open for the water to be seen flowing downwards along the fracture plane. The fracture is in Upper Chalk close to its junction with the overlying Reading Beds. The ground water was flowing northwards. The unconformable junction between the Chalk and the Reading Beds also dips northwards at an angle of about 60°.

At Ashwell, Hertfordshire, the author has also noted water flowing from joints in the Chalk. The springs emerge from the Totternhoe Stone. Fordham (1965) quotes records for the period September 1963 to December 1964 which give a minimum flow of 2400 m³/day in December 1964 and a maximum flow of 8300 m³/day.
in April 1964. He also comments that a low flow was observed for a few weeks in the winter of 1963 when frozen ground prevented percolation.

The author has found no field evidence of water becoming 'ponded up' in fractures with resulting excess water pressures. Nevertheless the possibility does exist, particularly if the fractures near the slope face become plugged with ice during winter preventing natural drainage of the slope.

Chalk slopes are therefore considered to be normally free-draining. Water flow in the Chalk appears to take place mainly by seepage through the mass of rock rather than by flow along the fractures. In the Lambourn Valley, Berkshire, the rate of vertical seepage through the Upper Chalk above the water table has been measured as 0.88 m. per year (Headworth, 1972).

9.2.11 Conclusions from field studies

Field studies have revealed the existence of a number of different types of fracture surfaces and fractures. Most fractures have smooth surfaces or surfaces that are covered by limonite or chalk powder. The Chalk bounding the fractures is normally at a moisture content close to the saturation value. Since most fractures are planar and relatively continuous, the nature of the surfaces is believed to be the major factor determining the friction that may be developed along these planes.
9.3 Laboratory studies

The usual laboratory methods for testing fractured blocks of rock are the triaxial compression test and the direct shear box test. Goodman (1970) has drawn attention to the disadvantages of using the triaxial test on rock with fractures. One major disadvantage is the uneven stress distribution that has been found to exist. Goodman also considered the disadvantages arising from use of the direct shear box for testing rock fractures. These disadvantages are the uneven stress distribution that occurs in the box, and the confinement of the sample that prevents any rotations in the fracture plane. He suggests that one possible solution might be to omit the sides of the box.

Another difficulty associated with the use of the standard shear box for testing rock fractures, that the author believes to be of importance, is the relatively small size of the sample used. As rock fractures are features of considerable extent and their surfaces may show significant variation, the testing of the largest possible surface area of the fracture is desirable. Although large size shear boxes have been made to overcome this particular problem, they are not generally available and are complex and expensive to construct.

In an attempt to overcome the problems associated with the use of the standard shear box, a simple piece of apparatus was constructed. This apparatus was used to determine the angle of statical friction of fracture surfaces of rock. Tests were undertaken using the apparatus on a variety of chalk fracture surfaces.
9.3.1 Friction measuring laboratory apparatus

The apparatus consists of two timber platforms hinged together along one side. The hinges allow one platform to be raised or lowered as desired. A scale allows the angle at which the platform is inclined to be measured. The apparatus is shown in Fig. 67.

The samples to be tested were placed in wooden moulds with fracture surfaces uppermost. Quick setting plaster was then poured into the moulds and allowed to set. After removal from the moulds, the blocks were placed on the moveable platform of the apparatus so that the fracture surfaces were in contact. The platform was then raised until sliding of the upper block on the lower block took place. Sliding of the lower block was prevented by a ridge at the lower end of the moveable platform. The angle at which sliding occurred was recorded. The angle at which sliding occurred was recorded. The angle of sliding was found to be measurable to the nearest $0.5^\circ$. The shear strength is not measured directly in the test but may be obtained from the relationship,

$$\tau = \sigma_n \tan \phi_s \quad (9.1)$$

where $\tau$ = shear strength of fracture,
$\sigma_n$ = normal stress acting on fracture,
$\phi_s$ = angle of statical friction of fracture.

Surcharges placed on the blocks were used where the behaviour under different normal loads was to be examined. However, very high normal loads were found to be difficult to achieve by this method. 10 kg. was found to be the maximum surcharge easily used.
The author believes that this test may simulate in situ conditions of sliding more closely than the conventional shear box, since movement is induced by the vertical load rather than an externally applied shearing stress. In addition, the apparatus overcomes the problems of lateral restraint discussed earlier.

9.3.2 Tests on dry fractures

Tests were undertaken on prepared blocks of chalk. The chalk blocks were cut from larger masses obtained from the Upper Chalk near Redbourn, Hertfordshire. Each block had one carefully cut surface representing a smooth fracture surface. The contact area of the blocks was 184 mm.x 111 mm. The moisture content of the blocks was 0%. The blocks were prepared in moulds and tested as described in Section 9.3.1.

Tests were undertaken firstly using only the prepared blocks, and then with increasing surcharges; these being 2, 4, 6, 8 and 10 kg. For each series of tests sliding of one block on the other was repeated and on each occasion the angle at which sliding commenced was recorded. This angle was found to decrease by as much as 3 or 4° after only six slides. After lightly brushing the surfaces and testing again, the angle of sliding was found to increase significantly (Fig. 68).

To investigate the effects of continued movement the sliding of one block upon the other was repeated many times. The results of these tests are shown in Fig. 69. The data has been
Fig. 67. Apparatus for determination of statical angle of friction of fracture surfaces.

Fig. 68. Effect of brushing (B) of fracture surfaces on the statical angle of friction ($\phi_s$).
statistically treated using a standard time series technique.

The results indicate the following:

(a) The angle of sliding initially decreases and then tends to increase with continued movement.

(b) The greatest initial decrease and greatest subsequent increase occur in the case of zero surcharge.

(c) The higher surcharges tend to show only small variations in the angle of sliding.

(d) The angle of sliding varies from a minimum of $34.4^\circ$ to a maximum of $40.2^\circ$ in the case of zero surcharge.

(e) The average value for the angle of sliding obtained from all these tests was $37^\circ$.

Examination of the sliding surfaces showed that fine powder had been formed on the surfaces after only a small amount of sliding, perhaps two or three slides. After more continued sliding there was found to be a build-up of powder, often 2-3 mm. thick, on the surfaces, together with grooving (up to a depth of 2 mm.) of any part of the surfaces not covered by powder. With the higher surcharges small fragments of chalk, 1-2 mm. in diameter, and mostly angular, were observed on the surfaces. These effects are shown in Plates X and XI.

9.3.3 Tests on wet fractures

Field observations have suggested that even although most slopes in chalk lie above the permanent water table, fracture surfaces are frequently wet, and the chalk appears to have a high moisture content. Tests were therefore carried out on blocks of
Plate X. Upper surface of dry fracture after continued sliding.
Plate XI. Lower surface of dry fracture after continued sliding.
chalk in a wet condition. The blocks were prepared from larger samples obtained from the Upper Chalk at Redbourn, Hertfordshire. The prepared blocks had sawn surfaces to represent smooth fracture planes as with the blocks used in the dry tests. The in situ moisture content of the chalk was found to be 23-26%. Before testing on the sliding apparatus the prepared block samples were soaked in water for 24 hours. The moisture content of the saturated samples was determined as 25.2%. The natural moisture content of the chalk at the Redbourn locality is therefore at or extremely close to the saturation moisture content.

Measurement of sliding angles was carried out as in the previous series of tests. The results of the tests are shown in Fig. 70. The data has been statistically treated in the same way as the previous set of results. Tests were only undertaken for the blocks alone with no surcharge. The main results of these tests are as follows.

(a) The angles of sliding increase with continued movement from a minimum average value of 21.2° to a maximum average value of 41.2°.

(b) After the large initial increase in the angle of sliding, the angle of sliding remains fairly constant at an average value of about 40°.

(c) Some very erratic individual values were recorded in the tests. These were as low as 13° and as high as 62°.

Examination of the surfaces showed that after some continued sliding (about 15 slides) a wet chalk sludge had developed on the surfaces. With further sliding the sludge became thicker and reached 1mm. in thickness. A few small chalk fragments were also
Fig. 70. Effect of continued movement on statical angle of friction ($\phi_s$) for wet test blocks. (Data has been statistically treated).

(a) Irregular fracture surface

(b) High points removed and powder fills troughs

(c) Powder and fragments on fracture surface

Fig. 71. Effect of sliding on nature of fracture surface.
present on the surfaces. These effects are shown in Plates XII and XIII.

The particularly low angles of sliding were recorded when free water was present on the chalk surfaces. In these cases sliding occurred suddenly and rapidly. Pushing of the blocks together by hand pressure from above had the effect of increasing the angle of sliding, usually to about 50-60°, but not higher than any angles of sliding recorded by sliding of the blocks under their own weight.

9.4 Discussion

As was expected with the results of tests on dry fractures, the angle at which sliding occurred decreased, at least initially, with continued movement. This would tend to occur as any irregularities of the surfaces are removed by the sliding of one block over the other. However, the later increase in the angle of sliding after further movement was not expected. The cause of these changes in the angle of sliding achieved is believed to be the powder and fragments which develop on the fracture surfaces.

Initially, the formation of fine powder may fill in some of the voids on the fracture surface and assist in the lowering of the angle of tilt required for movement to occur. The possible situation is shown in Fig. 71. After this initial stage, however, the formation of additional powder (often 2-3 mm. thick) on the surfaces, and the grooving (up to a depth of 2mm.) on surfaces not covered by powder, increase friction and make sliding more difficult. The small, mostly angular, fragments of chalk, 1-2 mm. in diameter,
present when the higher surcharges were used, must have a similar
effect. After the build-up of material along the sliding plane,
the blocks often tended to suddenly stick or slide irregularly.

At higher surcharges the fragments, powder and grooves were
not found to cause such a great increase in the angle of sliding
as that obtained, for example, with zero surcharge. However, the
powder and grooving still had the effect of causing sticking and
irregular movement as one block moved upon the other. With the
maximum surcharge used, that of 10 kg., the average angle of
sliding showed an increase from a minimum value of 36.4° to
a maximum of 38.0°. An increase or decrease of this amount
occurring on an in situ fracture might still be significant.

The field studies have revealed that many fractures in the
Chalk do contain infillings or have material coating the surfaces.
The laboratory tests indicate that if these infillings or coatings
are removed, by, for example, percolating water, there will be a
decrease in the friction developed along the fracture. If the
fracture is inclined at an angle close to that for sliding along
the fracture to easily occur, then a reduction in friction could
initiate movement along the fracture surface. The laboratory tests
indicate that removal of the powder and fragments could decrease
the angle of friction of the fracture surface by as much as 6°.
Much larger changes have been shown to be possible if the effects
of moisture are considered. The presence of water, and the possible
formation of wet sludge along the fracture, could reduce the
angle of friction of the surface by as much as 20°.

Work undertaken by many researchers indicates that the shear strength of rock reaches a peak value at failure and then decreases as movement occurs along the failure plane until a residual value is obtained. Fracture surfaces in rock are therefore characterised by an angle of residual sliding resistance $\phi_r$. Hendron (1968) has indicated that the shearing resistance along a discontinuity is dependent on the amount of previous relative displacement that has occurred between the rock surfaces. The results of the author's tests support this, and, in addition, indicate that the $\phi_r$ value may vary significantly for the same rock type depending on the type of fractures and fracture surfaces present. The tests also indicate that in some cases the $\phi_{peak}$ value may decrease to a $\phi_r$ value and then increase to a new $\phi_r$ value. This is shown in Fig. 72.

Patton (1966) has shown that the shear strength of discontinuities is partly dependent on the magnitude of the normal load. At low normal loads irregularities of the fracture plane are not readily removed during movement. Consequently, there is an additional component of friction present that does not exist if the fracture plane is smooth. This additional component is the angle between the irregularities and the sliding surface (denoted by $\phi_z$). At low normal loads, Patton found that displacements occur that are perpendicular to the direction of the shearing force. At high normal loads failure of any irregularities on the fracture surface occurs and there is no dilatant vertical movements. The failure
Fig. 72(a). Variation of the angle of frictional sliding resistance with displacement of fracture surfaces.

Fig. 72(b). Observed variation of the angle of statistical friction with displacement for fracture surfaces of dry chalk.
envelopes obtained by Patton are shown in Fig. 73. The vertical distance between the two failure envelopes (x) represents the size of the shearing resistance lost with displacement. So, although there is no cohesion intercept, there is some internal 'cohesion' created by the irregularities. This reaches a maximum value when the irregularities are sheared off, and is constant for higher normal loads. For curve OA the cohesion is directly proportional to the normal load; for curve AB it is independent of the normal load.

In the author's sliding tests on chalk the average values for the sliding angles obtained at each increment of normal load are as follows:

<table>
<thead>
<tr>
<th>$\sigma_n$</th>
<th>$\phi_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.69 kN/m²</td>
<td>37.5°</td>
</tr>
<tr>
<td>1.65 kN/m²</td>
<td>37.0°</td>
</tr>
<tr>
<td>2.64 kN/m²</td>
<td>36.5°</td>
</tr>
<tr>
<td>3.62 kN/m²</td>
<td>38.0°</td>
</tr>
<tr>
<td>4.60 kN/m²</td>
<td>37.0°</td>
</tr>
<tr>
<td>5.58 kN/m²</td>
<td>37.0°</td>
</tr>
</tbody>
</table>
Fig. 73  Failure envelopes for smooth and irregular fracture surfaces (after Patton, 1966).
These values for \( \sigma_n \) and \( \phi \) may be substituted in the standard equation:

\[ S = \sigma_n \tan \phi \]  \hspace{1cm} (9.2)

In Fig. 74 these calculated shear strength values have been plotted against normal load. The points lie on a straight line passing through the origin. Since the normal loads used are small, this curve may equate with the OA segment of Patton's curves (Fig. 73). The curve could also equate with the OC segment of Patton's curve since the tests on chalk were undertaken on relatively smooth surfaces. However, even with these tests at low normal loads there is some evidence that as normal load increases, so the angle of sliding decreases. The typical end of test value for each increment of normal load obtained in the sliding tests on chalk are as follows.

<table>
<thead>
<tr>
<th>( \sigma_n )</th>
<th>( \phi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.69 ( \text{kN/m}^2 )</td>
<td>40.0°</td>
</tr>
<tr>
<td>1.65 ( \text{kN/m}^2 )</td>
<td>38.0°</td>
</tr>
<tr>
<td>2.64 ( \text{kN/m}^2 )</td>
<td>38.5°</td>
</tr>
<tr>
<td>3.62 ( \text{kN/m}^2 )</td>
<td>38.5°</td>
</tr>
<tr>
<td>4.60 ( \text{kN/m}^2 )</td>
<td>37.0°</td>
</tr>
<tr>
<td>5.58 ( \text{kN/m}^2 )</td>
<td>37.0°</td>
</tr>
</tbody>
</table>

If these values are substituted in the standard equation for shear strength the curve shown in Fig. 75 is obtained.
Fig. 74: Shear strength ($s$) calculated using average values obtained for the static angle of friction ($\phi_s$) plotted against normal load ($\sigma_n$).
Fig. 75  Shear strength (S) calculated using end of test values obtained for the static angle of friction ($\phi_s$) plotted against normal load ($\sigma_n$).
9.5 Conclusions

(a) A number of different types of fracture surfaces and infillings occur in the Chalk.

(b) Recognition of the type of fracture surface or infilling present at a particular location is important because it affects the shearing resistance of the fracture plane.

(c) With movement fracture surfaces in chalk may show a reduction in frictional resistance, but this may increase again as powder, grooves, and fragments occur on the plane.

(d) The average laboratory determined angle of frictional resistance for dry fracture planes in chalk is \(37^\circ\).

(e) Removal of infillings may lower the frictional resistance.

(f) Moisture significantly lowers the frictional resistance.

(g) The frictional resistance of fracture surfaces is load dependent.

(h) The behaviour of fracture surfaces is complex and requires further study.
10. GENERAL DISCUSSION

10.1 Introduction

The author considers that the conclusions arrived at from the present research confirm in the field some of the general theories of rock slope stability developed by earlier workers. In addition the conclusions show more specifically how some factors, which have not been dealt with fully elsewhere, affect the stability of slopes in chalk. Nevertheless many of these conclusions are believed to be applicable to other types of rock. In this discussion the model of slope development which has become apparent as a result of the work is described firstly, and then some of the other more detailed results will be discussed and their wider implications considered.

10.2 Model of slope development

The field work has indicated that two main types of instability occur: major slope failure and slope degradation. These two types of instability are closely related to the stages in the reduction of the angle of slope. In a fresh vertical slope, whether excavated naturally or by man, stability of that slope is initially controlled by the orientation of the pre-existing fracture planes. Different types of failure are possible depending on the orientation and intersection of these pre-existing fracture planes in relation to the orientation of the face of the slope. Where these fracture planes are inclined at angles of up to $80-85^\circ$ sliding of the failing mass of rock occurs. Where the fracture planes are inclined at angles in excess of $80-85^\circ$ the unstable mass fails by rock fall.
Sliding failures are possible along planes inclined at angles as low as $30^\circ$ from the horizontal.

The likelihood of failure along the pre-existing fracture planes depends on a number of factors, the most important being:

(a) the continuity of the fracture;

(b) the nature of the fracture surface; and

(c) the occurrence of other fractures to define the lateral extent of the failure.

In some cases failure may occur predominantly within intact chalk (Type D, tension-shear failure), either related or unrelated to the system of pre-existing fractures. These major failures may therefore result in a rapid and large reduction in the angle of slope.

Where the face is being actively eroded at the base, it will be subject to continued retreat by major slope failures. Two types of failure have been recognised which are limited to such situations (Type C, inverted block removal, and Type H, inverted wedge removal).

Where the face is no longer subject to major failures or active erosion, it undergoes the second type of instability, slope degradation. A secondary fracture system becomes superimposed on the primary fracture system as a result initially of stress relief, and then of weathering of the face. Frost action is the main weathering process that affects chalk, and significant fracturing and loosening of the chalk forming the face may occur after exposure to two winters with severe frost. This process of degradation
results in the loosened pieces of rock becoming detached from the face and falling under gravity to the base of the slope. As the pieces of fallen rock accumulate at the base of the slope so they protect it from further weathering. Weathering is therefore increasingly concentrated on the upper part of the slope which continues to degrade until an angle of slope is achieved at which vegetation becomes established and protects the slope from further degradation (Fig. 76).

10.3 Groundwater

Many slope failures in various types of rock have been attributed to the effects of ground water. However, the present author has found only limited evidence to suggest that ground water is a major cause of instability in chalk slopes.

Since chalk en masse is relatively permeable and homogeneous, slopes formed by it are normally free-draining. The water table is therefore usually at a low level within the slope. Hutchinson (1972) has demonstrated that in Thanet the phreatic surface is low. Even at distances of 1 km. from the coast the phreatic surface is at an average elevation of only +2m. O.D. Near the coast the phreatic surface is affected by the tides.

Even slopes excavated in chalk below the level of the water table are likely to become free-draining quite rapidly since the high permeability of the chalk allows the water table to adjust quickly to a new level (Fig. 77). In slopes excavated below the water table level, the greatest pressures due to ground water
Fig. 76. Stages in the degradation of a typical chalk slope.
1. Water table level before excavation of slope.
2. Water table level after excavation of slope.

Fig. 77. Effect on water table of slope excavated below that level.
will therefore prevail immediately after excavation. As drainage of the slope occurs the level of the water table will fall and ground water pressures will be reduced. Such excavations below the water table are therefore likely to be most unstable during and immediately following excavation.

High water pressures within the slope due to, for example, poor natural drainage, the blocking of water-bearing fractures by clay, or high water levels in fractures after periods of high rainfall, are likely to be infrequent in chalk slopes. Nevertheless, significant variations in the water table levels within the Chalk do occur. For example, in the northern Chilterns rest water table levels in a well on Dunstable Downs, Bedfordshire, showed a rise of 28.5 m. between September, 1968 and March, 1969, and rest water levels in a well at Offley, Hertfordshire, showed a rise of 34.6 m. between September, 1968 and February 1969. Both rises occurred after high winter rainfall. As mentioned in Section 9.2.10, Fordham (1965) has commented that an exceptionally low flow from the Totternhoe Stone at Ashwell Springs, Hertfordshire, in the winter of 1963 was due to frozen ground preventing percolation. Such large rises in the level of the water table caused by high rainfall and/or interruption in the natural drainage due to freezing, may result in slope instability.

Water flow from slopes in chalk has been noted at a few localities. In these cases the water has been observed to flow from fracture planes. The author’s field investigations, however,
indicate that seepage from steep slopes in this way is unusual. There is evidence to suggest that most flow in the Chalk is by seepage flow rather than by fracture flow (Headworth, 1972).

Electron microscope photographs of samples of Middle and Upper Chalk collected by the author have revealed two main textures to exist. In chalk of soft and medium hardness the coccospHERes composing the rock are usually complete and the texture is open. As a result the chalk is of relatively high porosity and seepage through the mass of the rock is possible. In the harder varieties of chalk complete coccospheres are uncommon and instead the rock is mainly composed of the separated coccoliths. As a consequence the texture is compact, the porosity is lower, and seepage through the mass of the rock is considerably reduced. The harder varieties of chalk are therefore likely to be characterised by fracture flow and the softer varieties by seepage flow.

The field investigations conducted by the author have revealed that the fracture spacing in the harder chalks is considerably wider than in the softer chalks. Springs are also known to sometimes emerge from these harder bands of chalk where they outcrop inland. The harder layers of chalk are therefore more likely to be subjected to high fracture water pressures.

The author's investigations suggest that the influence of ground water on the stability of slopes in chalk is limited. There is, for example, no evidence to indicate that high water pressures are an important contributory cause of the major types of slope failure.
which have been recognised in steep coastal chalk slopes.

Nevertheless the effects of ground water on stability are complex and further detailed work is required.

10.4 Fracture orientations

The orientation and nature of pre-existing fractures has been recognised as the main factor controlling the stability of steep slopes in chalk. The field measurements of the strike directions of fractures enabled definite fracture patterns to be discerned within the Chalk. That these fracture patterns may be determined regionally is believed to be of significance. If the fracture pattern may be recognised at a number of localities, and if the pattern remains reasonably constant over a given area, then predictions may be made of the fracture pattern likely to be present in an area where perhaps there are no exposures.

Measurement of fracture orientations and inspection of fracture surfaces using cores obtained from boreholes is extremely difficult because of the relatively small area of the fracture observed, and the problem of rotation of the core during extraction. Although large diameter boreholes do allow the rock to be inspected in situ, their use is justified only in major site investigation projects, and safety problems also exist. The present author considers that maximum use of exposures at the surface is essential. Extrapolation of this surface information downwards is possible provided the geological limitations are realised.
Although the regional fracture patterns recognised by the present author are considered to be related to the structural geology, the analysis of the fractures was undertaken to investigate their engineering geological significance rather than to interpret their structural geological significance. No detailed attempt was therefore made to relate these fracture patterns to the structural histories of the areas studied.

The distinction between areas of simple and those of more complex geology is extremely important when considering the orientations of fractures. The strike directions of the fracture sets are closely related to the fold axes present in the area under consideration. The dip angles of fractures have also been found to be reasonably consistent for each fracture set.

In the South Downs and Thanet 73-74% of the fractures measured were found to occur within $\pm 10^\circ$ of each of the mean strike directions of the fracture sets. Each fracture set may, however, contain fractures with a range of strike directions of as much as $30^\circ$. Fractures also occur that are apparently unrelated to the main fracture sets recognised. Thus, although regional fracture patterns may be discerned and dominant fracture directions recognised, there will nearly always exist some fractures that are randomly orientated and therefore not readily predictable.

In the areas of relatively simple structural geology where the bedding is inclined at less than $25^\circ$, 94-97% of joints were
found to have dip angles exceeding 50°. This is believed to be important since it indicates that in slopes inclined at that angle, or at some lesser angle, few pre-existing planes inclined unfavourably are likely to occur. Failure of such slopes along pre-existing fractures is therefore also unlikely. Further, an angle of 65° would seem to be the normal lower limit for major instability. This indicates that slopes of up to 50° and probably up to 65° are reasonably stable.

In the areas of more complex geology where the bedding is orientated at angles in excess of 25°, failure is possible along continuous bedding planes dipping at angles of 30°. In addition, in these areas the joint fractures are commonly inclined at angles much less than 90°. For example in the Isle of Wight many joint fractures dip at 50-60°. All slopes inclined at angles of more than 25-30° are therefore potentially unstable in these areas.

10.5 Fracture surfaces

In the field sliding has been observed along fractures inclined at angles as low as 30° from the horizontal. The results of the laboratory sliding tests showed that the angle of statical friction of smooth, dry chalk surface varied from 34° to 40°, with an average value of 37°. Sliding of a mass of chalk along an underlying dry fracture surface inclined at an angle of less than 34° would not therefore be expected to occur. Although many fractures in the Chalk have smooth or slickensided surfaces, numerous pre-existing fractures are discontinuous, and even continuous fractures tend
to have some irregularities and/or infillings of chalk fragments. The laboratory studies have indicated that these irregularities and fragments tend to increase the shear strength of the fractures. Sliding along such dry fracture surfaces inclined even at 40° would therefore appear unlikely.

Field and laboratory investigations have, however, indicated that the natural moisture content of chalk is normally at or close to the saturation value. Fracture surfaces are therefore usually moist. The laboratory sliding tests have indicated that for smooth, moist surfaces the static angle of friction is frequently reduced to 21°, although values as low as 13° may be recorded. These results indicate that in the field sliding is possible along moist fracture surfaces inclined at these very shallow angles. The possible effects of various factors on the static angle of friction are given in Figure 78. There is some evidence to indicate that where two very smooth fracture surfaces are in contact suction forces may operate producing an increase in the static angle of friction to values of as much as 50-60°. Flow of water along fractures may significantly alter their frictional characteristics by removing infillings and/or eroding or dissolving the surrounding chalk producing smooth or iron-stained surfaces.

The results of the laboratory sliding tests have indicated that sliding is possible along any fracture plane in the Chalk inclined in excess of an angle of 15-20° downwards towards the
<table>
<thead>
<tr>
<th>Factors Increasing Friction</th>
<th>Suggested Effect on Typical $\phi_s$ of 37$^\circ$</th>
<th>Factors Decreasing Friction</th>
<th>Suggested Effect on Typical $\phi_s$ of 37$^\circ$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry surface</td>
<td>0</td>
<td>Smooth surface (or surface with slickensides aligned in direction of possible movement)</td>
<td>0</td>
</tr>
<tr>
<td>Minor irregularities</td>
<td>+3$^\circ$</td>
<td>Infillings absent</td>
<td>0</td>
</tr>
<tr>
<td>Infillings of fragments</td>
<td>+3$^\circ$</td>
<td>Continuous fractures</td>
<td>0</td>
</tr>
<tr>
<td>Major irregularities</td>
<td>+10$^\circ$</td>
<td>No displacement of fracture</td>
<td>0</td>
</tr>
<tr>
<td>Discontinuous fracture</td>
<td>up to strength of intact chalk i.e. failure at 45$^\circ$ or $\phi_s$ + $\frac{4}{5}$</td>
<td>Moist surface</td>
<td>up to 12$^\circ$</td>
</tr>
<tr>
<td>Displacement by other fractures</td>
<td>up to strength of intact chalk i.e. failure at 45$^\circ$ or $\phi_s$ + $\frac{4}{5}$</td>
<td>Wet surface</td>
<td>up to 22$^\circ$</td>
</tr>
</tbody>
</table>

Fig. 78 Effects of various factors on the static angle of friction.
face of an excavation. There are, however, factors which tend to
increase the shear strength of such planes. As mentioned already
irregularities will tend to increase the frictional resistance
developed along the fracture surface.

In the areas studied the fracture pattern has been found to
be often composed of six well-defined sets of fractures. As these
sets of fractures are closely related to the structural geology
of each area, they were formed in a sequence related to the
structural history of the area. The author has suggested that
some sets are complimentary and were indeed formed simultaneously.
Sets of fractures formed at a later date will traverse existing
fractures and may displace them by small amounts. Some fractures
observed in the Chalk have been found to exhibit displacements of
up to 100 mm. Such fractures have similar orientations to other
fractures without displacements, and they have therefore been
referred to as joint fractures rather than as fault fractures.
The displacement of one set of fractures by another results in
the surface of the former becoming irregular (Fig. 79).

The author believes that this is of importance when considering
the effect of these fractures on stability. If the fracture surface
is to act as a failure plane then failure of intact rock will be
necessary before movement is possible. Such displaced fractures
will therefore have a considerably higher shear strength than that
of smooth continuous fractures. Effects such as this are clearly
on a large-scale within the rock mass and are not readily assessed.
Fracture, $F_1$, cut by fracture set, $F_2$, without displacement.

Fracture, $F_1$, cut by fracture set, $F_2$, with displacement.

Development of failure surface along displaced fracture, $F_1$.

Failure surface after small displacement by sliding.

Fig. 79. Effect of intersecting fractures on failure surfaces.
by laboratory tests. Field inspection of available rock exposures provides the best available method of assessment. The author suggests that more detailed investigations of the fracture pattern to determine the relative ages of the fracture sets and the extent of displacements of one fracture set by another would be very useful.

There is some evidence to indicate that the friction developed along the fracture surfaces is load dependent. The sliding apparatus used in the laboratory tests only allowed study of the behaviour of the fracture surfaces at low normal loads. Nevertheless after continued sliding at the highest normal load used the average statical angle of friction was $37^\circ$, $3^\circ$ less than the value obtained at the lowest normal load used.

Even if a mass of rock does overlie a plane which is inclined unfavourably towards a face, the mass must be limited in extent laterally for failure to occur. Field evidence shows that the lateral limitation may be along pre-existing fractures but is sometimes by fracture of the rock itself. In the latter case the stability of the potentially unstable mass will be increased significantly.

The average angle of statical friction of fractures measured in the dry sliding tests was $37^\circ$. This would seem to compare well with the maximum angle of $30^\circ$ for the long-term stability of natural inland chalk slopes obtained from field work.
The standard shear box used in the testing of soil samples is considered by the author to be unsuitable for the testing of fracture surfaces. The surface area of fracture tested is inadequate, and, in addition, samples are difficult to prepare. The author suggests that the simple sliding apparatus which has been developed provides a simple, inexpensive, rapid and flexible method for assessing fracture surfaces. This apparatus together with careful field inspection of the fractures is considered to provide a useful method of studying the frictional characteristics of fracture surfaces. The effect of irregularities and different types of infillings on sliding would be worthy of investigation.

10.6 Design and assessment of slopes

The work suggests that as many fractures in the Chalk are continuous and with smooth surfaces, the residual shear strength parameters with respect to effective stress, $\phi_r$ and $c_r$, should be used in the design of new excavations in chalk to be certain of achieving long-term stability. The laboratory sliding tests indicated that for dry chalk surfaces the value of $\phi_r$ is typically 35-37°, whereas for wet surfaces the value is typically 20-30°. If the surfaces of the fracture are smooth there will be no $c_r$ value caused by irregularities. That 30° is approximately the maximum angle for long-term stability of a slope with fractures unfavourably inclined is supported by the author's field observations of failures. As mentioned previously the minimum angle of inclination of a fracture along which planar sliding has occurred was 30°.
The author has found that many slopes are apparently stable at angles in excess of those that would be defined by the residual shear strength parameters for chalk. Many slopes will be stable at angles greater than $30^\circ$ where the planes are not inclined towards the face of the excavation, or where there is additional shear strength due to fracturing of surrounding rock being necessary to limit the failure laterally. As already mentioned, slopes inclined at angles of $60-65^\circ$ in areas of simple structural geology may also be stable because of the relatively infrequent occurrence of fractures dipping at angles of less than $65^\circ$, and the unlikely occurrence of failures involving shearing of intact chalk. Flow charts for the design or assessment of slopes in chalk are presented in Figures 80 and 81.

Although the modes of failure discussed in Chapter 5 have been recognised in steep chalk slopes, the author has observed similar modes of failure occurring in many other types of competent bedded sedimentary strata. As in the case of the Chalk, the fractures which control these failures are related to the structural geology, particularly the directions of the fold axes. The continuity of the fractures and the nature of the surfaces has, however, often been found to vary considerably from one rock type to another. The author believes that this regional study of the stability of chalk slopes has been valuable and such regional studies of rock slope stability might be usefully extended to other geological strata.
Fig. 80. Flow chart for design and assessment. A. 1. Where the dip of the bedding is known and does not exceed 25°.
Fig. 80 continued. Flow chart for design and assessment. A. 2. Where the dip of the bedding is unknown.
Fig. 81. Flow chart for design and assessment. B. Where the dip of the bedding is known and exceeds 25°.
The author suggests that as the behaviour of rock masses, and particularly the fractures in them, is not well understood at present, the use of field-based investigations like that described are essential if a full scientific explanation of their characteristics and behaviour is to be achieved. The techniques available for testing and assessing fractured rock are not generally considered to give meaningful results, and until more reliable methods are developed, the largely subjective assessment of rock in situ will remain extremely important. The author believes, however, that detailed field investigations supported by laboratory studies, such as the present investigation, do allow the factors controlling stability to be more fully understood, and criteria to be established which will permit practical and realistic assessments of the stability of specific rock slopes to be made.
11. MAIN CONCLUSIONS

1. Two main classes of slope instability affect chalk slopes:
   (a) slope degradation;
   (b) major slope failures.

(a) Slope degradation

(i) Slope degradation is associated with an outward movement of the face of the slope.
(ii) The movement of the slope face is caused by stress relief and weathering.
(iii) Frost action is mainly responsible for the weathering of chalk.
(iv) Slope degradation results in the development of a secondary fracture system which becomes superimposed on the primary fracture system, with consequent reduction in the size of individual blocks of chalk and minor rock fall from steep faces of rock.
(v) The fracture spacing within the Chalk may be assessed by geophysical seismic velocity determinations.

(b) Major slope failures

(i) The following types of major instability affecting steep slopes in chalk may be recognised:

   1. Planar failures
      A. Translational sliding
      B. Block sliding
      C. Inverted block removal
      D. Tension-shear
      E. Fracture controlled rock fall
      F. Irregular rock fall
2. Wedge failures
3. Complex failures
4. Superficial failures
5. Miscellaneous failures

(ii) The main factors determining the types of major slope failure are:

(a) the orientation of fractures with respect to that of the slope;
(b) the continuity of the fractures; and,
(c) the nature of the fracture surfaces.

(iii) Tension-shear failures involve fracture of the intact chalk and require different analysis to failures along pre-existing fractures.

2. Regional fracture patterns which are related to the structural geology may be recognised, and the constituent fracture sets are sufficiently consistent in dip and strike to be of engineering significance.

(a) When considering slope stability on a regional basis the distinction between areas of simple structural geology and those of more complex geology is extremely important.

(b) In areas of relatively simple structural geology where the bedding planes are inclined at angles of less than 25°, slopes inclined at angles of up to
60° are likely to be relatively stable, since the majority of joints have inclinations of over 60°, and failure of intact chalk is unlikely in slopes inclined at angles of less than 65°.

(c) In areas of more complex structural geology where the bedding planes are inclined at angles in excess of 25°, slopes inclined at angles greater than 30° are likely to be unstable since in these areas joints are frequently inclined at shallow angles and failure along the bedding planes is also possible.

3. Ground water may significantly reduce the stability of chalk slopes by:

(a) Changing the character of the fractures by removing infillings or tending to lubricate the surfaces of fractures which may be possible failure planes; and

(b) By freezing of water in fractures near the face causing a rise in the ground water level within the slope and thereby reducing its stability.

4. In assessing the stability of existing chalk slopes, or in the design of new excavations the residual shear strength parameters \( \phi' = 25-35°, c' = 0 \) should normally be adopted.

(a) An angle of slope greater than that achieved by applying the residual shear strength parameters may be justified, where adequate site information
is available to show that the shear strength of the rock mass forming the slope is sufficient to maintain that angle.

(b) Regional studies of the factors affecting stability provide valuable information enabling the achievement of a practical and rational approach to the design of new slopes in rock and the assessment of the stability of existing slopes.
12. SUGGESTIONS FOR FUTURE RESEARCH

1. Study of the stability of chalk slopes in areas not covered by the present research, such as the Yorkshire and Lincolnshire Wolds, Salisbury Plain and the Marlborough Downs.

2. The monitoring of movements of a steep slope in chalk subject to major failures, and the recognition of any pattern of movement prior to failure.

3. Field and laboratory investigations of slopes in various rock types to establish realistic criteria for use in analyses of stability.

4. Regional studies of slope stability related to structural geology, to determine the uniformity of the factors affecting stability.

5. Detailed studies of structural geology to determine, for example, the extent of displacements of one fracture set by another.

6. Further investigations of the use of geophysical seismic methods for the in situ determination of fracture spacing. Characteristics other than velocity, such as attenuation and impedance may prove useful indicators.

7. Further development of the simple sliding apparatus to assess fracture surfaces, and the comparison of results obtained from it with those obtained from shear box tests and in situ investigations.
8. Further studies of the effects of ground water on slope stability, such as changes in the nature of the fracture surfaces.

9. Historical surveys of reports of slope instability and investigation of their possible causes, for example, their relationship to climatic conditions.
The author wishes to express gratitude to his project supervisor, Dr. N.S. Farrar of the University of Surrey, for his valuable advice and numerous suggestions about the research project and the preparation of this thesis. Thanks are also due to Mrs. E. Tyrrell for typing the thesis.

The author is also grateful for the assistance given by the following individuals and organisations:

Messrs. J. Foster and D. Greenbank for help with some of the field measurements and preparation of some laboratory specimens;

Mr. Ashley Cooper, Hexton Manor, Hertfordshire, for granting permission for excavation of the trial pit at Pegsdon;

Barnet Lime Company (and latterly F. Greenall and Sons Ltd.), Castle Lime Works, South Mimms, Hertfordshire, for granting permission for excavation of the trial pit at South Mimms;

Lee Conservancy, Thames Conservancy and Great Ouse River Authority for providing records of ground water levels;

Lee Valley Water Company for providing temperature and rainfall records for North Mimms;

quarry owners and managers for granting permission to examine chalk faces and undertake geophysical work;

colleagues for their helpful advice; and

The Hatfield Polytechnic for some financial assistance.
14. REFERENCES


APPENDIX 1

TEMPERATURE AND RAINFALL RECORDS
FOR NORTH MIMS, HERTFORDSHIRE,

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