FIELD AND LABORATORY STUDIES
OF 'SHORT-TERM' EARTHWORKS FAILURES
INVOLVING THE GAULT CLAY
IN WEST KENT

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Abstract

An investigation into the factors involved in the pore water pressure and stability of earthworks in the Gault is presented. The study is based on field measurements in cuttings and natural slopes, and on instrumented full scale embankment and cutting trials.

Many of the characteristics displayed by the Gault is attributed to its sedimentary and post-depositional history and so an outline of the geologic history of the strata and its engineering significance is presented. Details are given of the nature and widespread occurrence of Pleistocene periglacial discontinuities and their implications on the stability of earthworks are discussed.

The trial embankment data includes the pore water pressure details before and during construction and the results of insitu permeability tests. The embankment failed during construction as expected and the contribution towards instability made by pre-existing slip surfaces in the foundation are elucidated.

The cutting trial data includes the pore water pressure and ground displacement measurements made before, during and after the excavation of the cutting. Details are given of the stability of the steeply inclined section of the slope which failed 15 weeks after construction. Examples of cutting failures which occurred after a much longer period are presented in the discussion of the Maidstone By-Pass investigations. The analyses of these failures and the recorded lateral slope displacements give an insight into the significance of progressive failure in the stability of the slopes. Post-failure movement of the slipped masses provide some data on the shear strength mobilized along slip surfaces under low effective normal stresses.
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Dedication

To my mother and grandfather whose efforts
I can never hope to repay.
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In common with other sedimentary stiff fissured clays, the Gault contains numerous discontinuities which adversely affect its engineering properties. Terzaghi (1936) recognised that fissures, apart from being planes of weakness, can also lead to progressive softening of a clay mass. Shear zones often containing discrete principal slip surfaces were found at the site of the Mangla Dam in Pakistan, Skempton (1966), Binnie et al (1967), which significantly affected the design of the dam and the associated works.

During the construction of the Sevenoaks By-Pass in Kent, a major disruption of the construction programme occurred due to embankment and cutting failures along the southern face of the Hythe escarpment. The area is generally prone to landslipping, and in the past, considerable damage has been done to private property and roads by this activity. In an attempt to solve these problems, very detailed and extensive investigations were instituted, Skempton and Petley (1967), Weeks (1970), which established the widespread existence of superficial deposits and slip surface discontinuities on ground sloping at angles of between 3 and 7°. The slackest slopes in which slip surfaces were found are considerably
flatter than their angle of ultimate stability against landsliding, and show no morphological expressions of recent movement. The occurrence of slip surfaces in a wide area of Kent and in Oxfordshire has been reported by Weeks (1969) and, in the Lias Clay by Chandler (1970a). The origin of these slip surface discontinuities is not clear, but they have been ascribed to Pleistocene periglacial activity by Weeks (1969), (1970) and Chandler (1970a).

The difficulties created by the existence of slip surfaces during the construction of the Sevenoaks By-Pass, were overcome by the use of extensive drainage measures. Counterforts were excavated at close intervals at cutting sites and sand drains had to be installed at an embankment site where it was necessary for construction to be completed within a stated period. The coefficient of consolidation calculated from the observed rate of dissipation in the foundation material (Weald Clay) of the completed embankment was found to be only 45% of the value assumed in design. Fortunately, however, in the foundation material only approached a value of 0.5 during construction as opposed to the value of 1.0 assumed in the design.

The Sevenoaks By-Pass investigations highlighted the fact that the stability of an area is related to its surface topography, the shearing
resistances along potential failure surfaces, and the pore water pressure distribution on potential failure surfaces. Perched water tables were found to exist in some areas which did not show any consistency with water levels recorded in shallow open tubes.

After the completion of the Sevenoaks By-Pass, a greater emphasis was placed on establishing the existence of slip surface discontinuities along proposed motorway routes, at the site investigation stage, Weeks (1969). Along the route of the Tonbridge By-Pass which is the southward continuation of the Sevenoaks By-Pass, slip surfaces were found in low-angle slopes in the Weald Clay. A trial excavation along a 5 - 7° slope in the London Clay at Boughton Hill, during the site investigation for the Boughton By-Pass, revealed a slip surface at the base of a mantle of granular Head deposit, running almost parallel to the ground surface. In the underlying London Clay, a series of steeply inclined slip surfaces were encountered. The Ditton By-Pass is mainly routed across the Folkestone Beds, except for a short section of its length at Wrotham Heath, which is routed on embankment over the Gault. In a trial pit along a 3° slope in the Gault at Wrotham Heath, a slip surface was also encountered at the base of a mantle of granular Head deposit.
A trial pit excavated on a $3\frac{1}{2}^\circ$ Gault clay slope during the site investigation for the M.40 in Oxfordshire encountered an essentially clayey mantle of solifluxion material overlying Gault. The geometry of the slip surface associated with the solifluxion mantle, however, differed from those previously described as it was seen to have a curved outline, tending to rise towards the ground surface. It was also noted that the slip surface extended below the solifluxion material into the underlying Gault.

In all these projects, the problem created by the existence of slip surfaces at embankment sites was overcome by digging out the foundation material and replacing it either with granular fill material or with the remoulded excavated material.

The whole of the proposed M.25 motorway in Kent is routed along the Gault outcrop. A notable feature of the Gault outcrop is that it is mainly inclined at very flat slopes predominantly at angles of less than $3^\circ$. This is in contrast to the Sevenoaks and Tonbridge areas investigated by Weeks (1969), where the slope angles are generally about $3^\circ$ and steeper. In addition, the deposits forming the low angle slopes in the Sevenoaks and Tonbridge areas consist of strongly laminated and shaley clays with many silt partings. These features are also frequently aligned in a direction almost parallel to the ground surface. The Gault (and
London Clay) on the other hand is structurally, relatively more uniform, hardly exhibiting a laminar structure or containing silt partings.

Despite these differences, the significance of slip surfaces which had been so clearly demonstrated during the construction of the Sevenoaks By-Pass, was considered to be very relevant to the Gault. It was therefore decided at the outset of the M.25 programme, that the extent and nature of slip surfaces in the Gault should be investigated, so that the likely construction problems could be foreseen in advance of site work.

The Maidstone By-Pass, which was completed in 1959, formed the only previous experience of major road construction in the Gault in Kent. All the cuttings in the Gault along this road, ranging in depth from 5 to 16 m, and originally excavated at a slope of 1 (v) on 3 (h), were showing signs of failure only six years after the end of construction. The cutting failures were investigated as a supplement to the M.25 programme, with considerable emphasis being placed on the location of failure surfaces, and the measurement of pore water pressure on them, and in the surrounding material.

The data obtained from the M.25 and Maidstone By-Pass investigations relating to slip surfaces discontinuities, showed that there are basically two types of slip surfaces in the Gault. In some areas, continuous slip surfaces were encountered running almost parallel
to the ground slope, and in others, the slip surfaces were not continuous but were separated by unsheared fissured material. There was a tendency for continuous slip surfaces to be associated with superficial deposits on the steeper slopes, whereas the non-continuous slip surfaces tended to occur mainly in the flatter slopes.

The piezometric data revealed the existence of perched water tables in natural slopes and cuttings and analysis of response time in piezometers resulted in field permeabilities in the fissured Gault that were one to three hundred times greater than those measured in laboratory tests on 100 x 200 mm samples. Similar ratios of field to laboratory permeability have been observed by Skempton and Henkel (1960) for the London Clay, and in various other deposits by Rowe (1968).

At the earthworks design stage, major problems relating to slip surfaces, pore water pressure response, and the relevance of laboratory parameters in earthworks design had to be resolved, and these are discussed in the following sections.

a) As is to be expected, the high embankments along the proposed M.25 motorway occur across valleys which separate higher ground; and, it is along these valleys that very flat slopes tend to occur, and where non-continuous slip surfaces are invariably found. It was necessary to predict the extent to which these slip surfaces would affect the stability of embankments. A
potential failure surface in a zone of non-continuous slip surfaces would utilise slip surfaces along part of its length, with the remainder occurring through unsheared fissured material which separate the slip surfaces, and which are capable of mobilizing peak strength. Failure to allow for this would obviously lead to a conservative design of side slopes.

b) It was also necessary to determine the pore water pressure response to loading in the Gault in view of the observation at Sevenoaks, that \( \mu \) in the foundation material (Weald Clay) was generally less than 0.5 throughout the construction of a 16 m high embankment.

c) Due to the relative lack of data on the insitu shearing characteristics of the Gault, the relevance of laboratory parameters in earthworks design could not be ascertained. Although the major part of the testing programme was carried out using 100 x 200 mm samples, it was recognised that this size of sample may not adequately reflect the insitu characteristics of the Gault. Such estimates can only be made from the analysis of slides in unsheared material, in which the pore water pressure on the failure surface is known with reasonable accuracy.
The only detailed study of the stability of Gault clay slopes in Kent, known to the author, involves coastal landslides at the Warren in Folkestone. Failure at the Warren has involved some 150 m (500 ft.) of Chalk and about 46 m (150 ft.) of Gault, and has utilised a shear surface of residual strength at the base of the Gault, Hutchinson (1969). The large scale and deep seated nature of some of the slides render the shear parameters calculated from analyses of the slides, insensitive to quite significant variations in pore water pressure and topography. The range of variation noted by Hutchinson (1969) is large enough to have a significant effect on the scale of earthworks proposed along the M.25 motorway. The conditions at the Warren are therefore largely atypical of those that will be encountered along the proposed M.25 motorway.

An observational study of railway cuttings in the Gault in Kent and Surrey, all of which were originally excavated at slopes of between 1 (v) on 2 (h) and 1 (v) on 3 (h) some 30 to 150 years ago, revealed widespread incidence of instability. Failure has not been restricted to cuttings, and in a section of the Lenham to Charing railway in Kent, routed across the Gault, records kept by British Railways Southern Region show that all the embankments have been affected by failure at some stage in their
history. The observational study did not reveal any obvious relationship between height and angle of cutting or embankment slope, and even if a relationship had emerged, the fact that it was persistently observed that long cuttings and embankments of uniform inclination were nearly always only partially affected by landslipping, would impose severe limitations on such a correlation. This observation is thought to indicate a non-uniformity in the factors that give rise to instability at any given site on the Gault.

As the problems encountered during the earthworks design stage of the M.25 programme could not be solved by recourse to published data or past experience, they helped to reinforce the need for a trial embankment which had arisen out of the fact that the bulk of the Gault that will be excavated along the M.25 motorway will fail to satisfy Ministry of Environment requirements for fill material. A considerable saving in cost would, therefore, be effected if it was established that all the Gault could be used as fill material. Consequently, a trial was planned, primarily to ascertain the conditions under which 'unsuitable' Gault could be used as fill material. This decision presented the opportunity to study the likely contribution that non-continuous slip surfaces would make towards the
failure of an embankment.

Prior to this experiment, the problems raised by non-continuous slip surfaces below embankments were overcome by digging out the foundation material and replacing it either with imported granular fill or by replacing the remoulded excavated material. Although this method has proved effective, it is undoubtedly conservative as it fails to take advantage of the unsheared fissured material which separate non-continuous slip surfaces. In order that this factor may be taken into account in the earthworks design for the M.25 motorway, it was necessary to instrument the foundation adequately with piezometers, and to design the embankment to fail during construction.

It was also decided to incorporate a cutting trial at the site of the excavation for the trial embankment fill material to study the 'short-term' failure characteristics of the Gault, and to give a comprehensive picture of the pore water pressure distribution and deformation at all stages during and after construction.

Most of the factors that affect the engineering properties of the Gault, stem from its geological history. The sedimentary and post-depositional history of the Gault is therefore reviewed first of all in Chapter 2, before details of the trial embankment and trial cutting are presented in Chapters 3 and 4 respectively. The investigation of slope failures along the Maidstone
By-Pass is also included in Chapter 4. The main conclusions of the study are presented in Chapter 5.

Details of the standard procedures adopted during the investigation are summarised in Appendix 1 (p.388). The Appendix covers details of boring and sample description, trial pit excavation, piezometer installation (open-tube and close-circuit), inclinometer installation, index tests and clay fraction determination, and a brief discussion of the Janbu method of stability analysis which is used throughout the project.
CHAPTER 2.

OUTLINE OF THE GEOLOGY OF THE WEALDEN DISTRICT
WITH SPECIAL REFERENCE TO THE GAULT

2.1. General Account.

The Wealden District is the oval shaped tract bounded by the inward facing scarps of the Chalk rim. It embraces the major part of Kent, Surrey, Sussex and Hampshire and is truncated on the east by the English Channel. The intervening areas between the Chalk consist of sands and clays of Cretaceous age, which give rise to a rhythmic repetition of ridges and troughs running almost parallel to each other, Fig. 2.1.

The central area of the Weald comprises of gently rounded ridges predominantly of Hastings Beds, circumvented by the Weald Clay, which forms a wide strike vale mainly of very gentle relief, but, particularly in the west, thin seams of sandstone and limestone (Sussex Marble) give rise to slight features. Rising above this belt of heavy clay is the escarpment formed by the Lower Greensand Beds, which reflects in its minor relief features and changes of vegetation, the varied nature of these beds.

At the foot of the dipslope of the Lower Greensand series, another strike vale occurs predominantly on the Gault outcrop but often extending
on to the Folkestone Beds, the highest stratum of the Lower Greensand series. In the western Weald the strike vale of the Gault is partly replaced by a broad platform formed in a resistant facies of the Gault – the Upper Greensand. This feature is however impersistent, being developed only when these beds are cemented. Towards the east, where the Upper Greensand passes laterally into the more argillaceous facies of the Gault, the feature is missing, and the Lower Chalk forms the lower part of the Chalk escarpment.

The Chalk rises above the foot of the Gault vale to form an escarpment, much more uniform in height than that of the Lower Greensand. The basal bed of the Chalk, the Chalk Marl, being almost of clayey consistency, allows a very gentle transition of ground slope between the Chalk escarpment and the Gault outcrop. In the western Weald, however, the Upper Greensand feature prevents this gradual transition and so gives rise to steeper ground slopes along the Gault outcrop.

Structurally the Weald is an anticline, composed of a large number of east-west folds, arranged en echelon and usually asymmetrical, with steeper northward dipping limbs. The fold-pattern is complicated by strike faulting and less numerous dip-faults.
In the central Weald the drainage is essentially longitudinal; elsewhere it is not only transverse to, but in the southern Weald, is superimposed on, the Cretaceous rocks. Wooldridge and Linton (1955) have shown that this variation in drainage pattern is due to a partial submergence of the area in Pliocene times. This submergence also resulted in the 'bevel', thinly covered with sandy deposits, which is traceable immediately behind the crest-line of the North Downs.

2.2. Outline of the Palaeogeographical History of S.E. England.

The stratigraphic range of the surface outcrop in S.E. England is almost wholly restricted to Cretaceous and Tertiary rocks. The complete sequence of Mesozoic strata overlying Palaeozoic rock has, however, been established from borehole records, (Gallois, 1965).

It was recognised, since the earliest accounts were presented by Ramsay (1894), that the distribution of land and sea in the whole of Europe was altered towards the end of the Palaeozoic Era. North-Western Europe became part of the Hercynian Continent, and strata of continental facies were laid down over a wide range of areas during the succeeding Permian and
Triassic periods, marking the start of the Mesozoic Era. The predominantly continental facies were succeeded by the marine Jurassic and Cretaceous systems. Ramsay (1894) recognised that the marine Oolitic series is separated from the marine Lower Cretaceous series in S.E. England by a thick succession of terrestrial and fluviatile deposits, the Purbeck and Wealden Beds. Towards the end of Wealden times there was renewed submergence of the area by the sea, accompanied by the deposition of the purely marine series of Lower Greensand, Gault, Upper Greensand and Chalk.

In S.E. England, the Lower Greensand series is overlain by the Gault, elsewhere, the Gault extends beyond the limits of the Lower Greensand Beds and rests on Jurassic and older strata. Because of this, and partly also for palaeontological reasons, Ramsay suggested that the Wealden and Lower Greensand Beds be grouped as Lower Cretaceous and that the overlying deposits including the whole of the Chalk, be classified as Upper Cretaceous.

After the deposition of the Gault, the Upper Greensand was laid down. It is often difficult to separate the Upper Greensand lithologically from the Gault and in many areas the deposits are separated by a series of passage Beds having alternately Gault and Upper Greensand characteristics. While the Upper Greensand always follows the Gault, both are variants
of the same depositional sequence, the Upper Greensand beginning in the west before the Gault deposition had ceased in the east.

Following the deposition of the Upper Greensand, the Chalk was laid down. The Chalk sea deposited nearly 500 m (1500 ft.) of sediments before it receded in response to increased uplift activity, which raised the central Weald above sea level. This activity continued throughout the succeeding Eocene transgression and as a result, deposits of this age are largely absent from the Weald. However, the occurrence of chert pebbles, originating from the Lower Greensand series, in some Eocene deposits, Gallois (1965), is an indication of submergence, even if temporary, and of sub-aerial denudation having progressed through the Chalk cover down to the Lower Greensand Beds.

Crustal movements of early Tertiary times culminated in the Alpine Orogeny, which reached its peak during the late Oligocene or early Miocene period. Major palaeogeographical changes took place, and the deposits laid down since the start of the Mesozoic on a slowly subsiding Palaeozoic sub-floor were uplifted and folded. Much of the area was above sea level for the first time and a period of non-deposition, which lasted for about 20 million years, was instituted.
The British Isles lay on the fringes of the main areas of crustal movement, and in S.E. England only gentle folds with an east-west trend were produced. Much of the present physiography and drainage pattern of the Weald began to develop during this period.

Sediments of the succeeding Miocene are absent from Britain. However, in S.E. England, the period is represented by peneplains along the ridges formed during the late Oligocene. Denudation continued into the Pliocene period and, after its cessation, sedimentation returned to the area with the deposition of the Lenham Beds of East Kent, and other similar outliers along the North and South Downs. Tectonic folding had virtually ceased during this period, but changes in land levels relative to sea level probably brought into being a neck of land bridging England and France.

The deterioration in climate during the late Pliocene culminated in the Quaternary, for which four main periods of glaciation can be distinguished in the British Isles, Table 2.1. The climate was not uniformly severe and cold spells (glacials) alternated with warmer and more humid periods (interglacials and interstadials). Eustatic changes resulted in relative changes between land and sea levels. These changes, resulting from climate variations, were superimposed on tectonic,
isostatic and temperature changes which are also capable of producing significant changes in sea level. However, a gradual lowering of sea level could be inferred from the Thames Terraces. These show a drop from the Boyen Hill and Swanscombe terraces, formed during the Hoxnian interglacial stage at about 30 m (100 ft.) O.D., to the Ipswichian Taplow terrace at about 17 m (50 ft.) O.D., to the more recent Flood Plain Terrace.

The major part of S.E. England is generally believed to have been outside the limits of Quaternary glaciation; however, unglaciated areas such as the Weald were within the zone of influence of the ice sheets. The unglaciated regions, in which the cooling effect of the ice sheets produced a frost climate, have been described by Zemmer (1959) as the periglacial zone. Frost action (cryoturbation) in the periglacial zone resulted in extensive weathering and denudation, and the materials so formed, were transported downslope as a saturated mass during warmer and wetter periods. The process whereby saturated surface layers of soil are displaced down gentle slopes has been described as solifluxion (soil flow) by Anderson (1906), and the materials displaced are called solifluxion deposits. As a result of this activity the Gault outcrop is mantled by solifluxion deposits, Smart et al (1966), Dines et al (1969), with
constituents originating from the Chalk and Lower Greensand escarpments, as well as from the Gault itself. Another feature of the periglacial areas, particularly around the escarpments of S.E. England, is the widespread development of landslipping, cambering and the associated valley bulging, Hollingworth et al (1944).

The first part of the Quaternary, the Pleistocene, was succeeded by the Holocene period, which marks the final retreat stages of the last ice sheet and the start of present-day climatic conditions. In North-Western Europe, the Holocene is represented by eight vegetational zones which are based on pollen analysis. A detailed account of the British Flora is given by Godwin (1956). Though defined by fossils, the pollen zones are not truly time stratigraphic, a true time correlation is being made possible through the use of radio carbon ($^{14}$C) dating.

Large quantities of water were released by the melting of the ice-sheets which caused the sea level to rise everywhere. Isostatic compensation also began to take effect as the ice load was reduced. The slow but steady submergence of land in S.E. England produced submerged forests, drowned river valleys and buried channels which occur just below the present-day flood plain deposits of rivers. Britain was severed from the European continent during this period.
2.3. Detailed Geology of the Gault.

a) General Consideration.

The Gault was deposited as a result of a widespread marine incursion which spanned the Middle and Upper Albian stages. The transgression engulfed the Anglo-Belgian ridge so that the Gault, which in parts of the Weald rests on Folkestone Beds, outcrops on Jurassic and Palaeozoic rocks towards the east and north respectively, Fig. 2.2. According to Jukes-Browne (1901) this overlap could only have resulted from a general and rapid subsidence of the region which greatly extended the Gault sea. The strata was originally deposited with a south-easterly dip but this has subsequently been altered by post-depositional flexures trending east-west and giving rise to local dips to the north and south.

The variation in the depth of the Gault sea is reflected in the thickness of the strata which shows a westward increase across the Weald. At Folkestone the Gault is between 40 m and 50 m thick (120 - 160 ft.) and it increases to a maximum of between 90 - 110 m (280 - 350 ft.) in the Sevenoaks area and in much of south and west Sussex. The variation in thickness of the Gault across the Weald is given in Table 2.2.

The conditions were not uniform throughout the period of deposition and so several beds of differing lithology were laid down. In eastern Kent the Gault is mainly argillaceous whereas in the west
starting from the Sevenoaks area apparently arenaceous deposits predominate. When first recognised the arenaceous sediments were thought to be stratigraphically more recent than the Gault and were consequently named the Greensand. These beds were often confused with the sandy beds below the Gault and so an Upper and a Lower Greensand separated by the Gault, came to be distinguished.

It has been established that the Gault and Upper Greensand are lithological variants of the same sequence. The Upper Greensand was probably deposited in shallow current swept conditions near the shore-line whilst argillaceous facies represent a shallow but quieter condition of deposition.

Detailed work on the lithology of the Gault at Folkestone, Price (1879), Casey (1950) have established the existence of thirteen Ammonite zones Beds I-XIII with an even larger number of sub-zones. A number of these zones have been established throughout the Weald, Owens (1971).

The variation in lithology is also reflected in the mineralogy of the strata. Large numbers of clay size mica crystals often found in the formation were attributed to the erosion of granites mica schists and Triassic sands by Seeley (1892). He also suggested that the clay facies were derived from crystals altered during decomposition of partly weathered mica. The
sandy facies though containing less altered material were also considered to be of similar origin. Quantitative information on the mineralogy of the Gault for example by Farrar and Coleman (1967), Hutchinson (1969) suggest that illite is the dominant clay mineral comprising some 50-75%. The kaolinite content is on average about 15% although greater proportions often occur. Montmorillonite may be present in quantities of about 10% but may be local to some beds of the strata where it can account for up to 30% or more of the clay mineral content. Calcite is also commonly found but only in minor quantities.

The thirteen beds noted at the type section at Folkestone cannot always be easily identified elsewhere along the outcrop; however, it is stated that an Upper and a Lower Gault could everywhere be distinguished, Table 2.2.

The Lower Gault is usually a dark grey clay with a westward increase in thickness from about 10 m (30 ft.) at Folkestone to 47 m (155 ft.) in Upper Beeding, Sussex. The Lower Gault is separated from the Folkestone Beds by a band of pyrites nodules and a matrix of dark grey clay and dark green glauconitic sand. Other lines of phosphatic nodules occur throughout, and a hard ragstone band is found capping Bed VI at the type section at Folkestone.
The interface with the Upper Gault is marked by a fairly persistent band of phosphatic nodules at the base of Bed VIII.

The Upper Gault in contrast is a paler grey clay with marly and calcareous facies. It contains several lines of phosphatic nodules and in some areas a band of glauconitic clay, Bed XIII, occurs at the top of the sequence. Although the conditions of sedimentation during the Middle and Upper Albian stages were quite different, the resulting deposits can only be distinguished visually where the Upper Greensand with its silt-size particles represents the Upper Gault i.e. in the western Weald.

The simplest form of visual classification to which the Gault lends itself is that based on the degree of weathering and decomposition. This system is adopted throughout this investigation and it is supplemented with details of the lithology whenever these are ascertainable.

In the Sevenoaks area mainly, the particles of the Gault appear to be of silt-size. Index tests on samples from the area, however, give results that are inconsistent with this apparent particle size; for example, liquid limits exceeding 75% are usually obtained. As will be seen from Table 2 given overleaf, the range of index parameters for the Gault in the Sevenoaks area, is similar to those obtained
over a wide area of the outcrop. The range of measured clay content of the Gault in the Sevenoaks area is, however, lower than is typical along the rest of the outcrop between Folkestone and Westerham.

**TABLE 2.3**

<table>
<thead>
<tr>
<th>Location</th>
<th>Liquid Limit %</th>
<th>Plastic Limit %</th>
<th>Clay Fraction &lt;2μ %</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ashford</td>
<td>67 - 105</td>
<td>23 - 25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maidstone</td>
<td>70 - 92</td>
<td>23 - 33</td>
<td>51 - 62</td>
<td></td>
</tr>
<tr>
<td>Westerham</td>
<td>68 - 102</td>
<td>24 - 33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sevenoaks Area (Otford - Dunton Green)</td>
<td>77 - 95</td>
<td>25 - 36</td>
<td>42 - 50</td>
<td>Trial Site</td>
</tr>
<tr>
<td></td>
<td>77 - 107</td>
<td>25 - 38</td>
<td>38 - 45</td>
<td></td>
</tr>
<tr>
<td>Westerham</td>
<td>65 - 100</td>
<td>22 - 30</td>
<td>52 - 60</td>
<td></td>
</tr>
</tbody>
</table>

Investigations of the mineralogy of the Keuper Marl by Dumbleton (1965), Davis (1967), Chandler (1969), have established inconsistencies between index parameters and measured clay content (<2μ %) which were ascribed to some cementing or aggregating agent. Davis (1967) observed that the activity (Skempton 1953), of the Keuper Marl he investigated increased with aggregation ratio (defined as the ratio of % clay mineral content, to the % of particles less than 2μ).

As cementing is usually considered to result in some loss of plasticity, the observation by Davis appears anomalous. However, Lambe and Martin (1956) have demonstrated that plasticity is not necessarily...
destroyed by aggregation. This could account for the lack of any significant difference between the plasticities of the Gault with apparent silt-sizes and those obtained for the more 'normal' Gault with apparent clay-sized particles, Table 2.3. In addition, Sherwood (1967) observed that the degree to which aggregation is reflected in either the index parameters or the measured clay content is controlled by the cementing agent. In those aggregated deposits in which the cementing can be destroyed by the mixing process in index tests, and not by the pretreatment during the clay content determination, the effect of aggregation will not be reflected in the index parameters, whereas it will be in the measured clay content.

An average increase in clay content of the Otford samples of 4% was recorded after the samples had been remoulded for the liquid limit test, further suggesting possible aggregation of the Gault in the Sevenoaks area. To distinguish these apparently silty facies of the Gault from the more 'normal' Gault, it is described as 'aggregated clay' and materials with intermediate properties are described as 'clay with some aggregation'. The Gault is also classified according to its degree of weathering and disintegration, and the classification of deposits as clays or silts is based on the Casagrande Table
given below:

<table>
<thead>
<tr>
<th>Material</th>
<th>Liquid Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clays</td>
<td>&gt; 50%</td>
</tr>
<tr>
<td>Silty Clays</td>
<td>35 - 50%</td>
</tr>
<tr>
<td>Silts</td>
<td>&lt; 35%</td>
</tr>
</tbody>
</table>

b) Unweathered Gault.

When freshly exposed and unweathered, the Gault is essentially a stiff, fissured, dark grey, plastic clay. Locally some pale or bluish grey bands and laminar layers may occur in the strata, and individual fissure lumps also show a laminar structure. The clay exhibits varying amounts of sub-horizontal and vertical fissuring with the strike direction of the fissure faces showing little or no preferred orientation, Fookes and Denness (1969). The mean size of the fissure lumps hardly exceeds 100 mm, however, there is a tendency for the mean size to decrease as the ground surface is approached, and consequently, the number of fissures per unit volume to increase towards the ground surface. This suggests that stress release and weathering are two of the many factors that may be involved in the formation of fissures. Fookes and Denness (1969) have established that bedding is the dominant factor influencing the fissure pattern of the Gault. Factors such as lithology, stress release and tectonism were found to have only a minor effect on
fissure pattern. A marked change of lithology, for example, between Chalk and Gault, was found to influence fissure size rather than fissure intensity. The changes in lithology that occur within the Gault strata have a less significant effect on the size of the fissure lumps.

Some fissures in the Gault display a high degree of polish and slickensiding, possibly indicating that small relative movements were involved in their formation. The majority of fissure faces are, however, planar with a matt surface texture; curvilinear and conchoidal features are also encountered, giving rise to closely interlocking fissure lumps; these seldom possess any marked degree of polish or striation.

Fissure lumps in the unweathered Gault may be separated by thin, and sometimes persistent, layers of soft, finely fissured or remoulded material with a much higher moisture content than the fissure lumps. These lenses of soft material are often found around bedding planes in exposures in the field, and may have resulted from tectonic activity.

c) Weathered Gault.

The weathered clay, usually located within the top 9 m or so of the formation, is normally less stiff than the underlying Gault. It is predominantly
brownish grey in colour, and only differs structurally from the unweathered Gault by its scale of fissuring, which is likely to be variable. On average the fissure lumps are between 50 - 100 mm in size and irregular fissuring is more common than in the unweathered Gault.

Basically, the weathered zone represents the extent of superficial alteration in the formation, resulting mainly from ground water percolation. In other words, its lower horizon marks the extent of chemical weathering in the strata.

The extent of the zone of weathering is affected by landform, which is also a reflection of the character of the deposits overlying the clay. The Gault is usually more extensively weathered at hill tops, though this is subject to the nature and thickness of the superficial cover. Hardly any weathering occurs where the clay underlies Chalk.

d) Frost Shattered (Cryoturbated) Gault.

This zone of the Gault represents the layer that has been subjected to both chemical weathering and severe mechanical weathering by frost action. It may be regarded as a zone of advanced disintegration. The physical breakdown of material in this layer has resulted in a most striking feature, namely, that the fissuring is on a much smaller scale, usually less than 25 mm, with generally irregular orientation. The
layer is normally soft and mottled greenish grey and brown in colour. The mottled effect arises from the grey colour of the individual fissure faces which are separated by soft remoulded greenish brown clay with gypsum crystals. The interior of the fissure lumps is brownish in colour, often also showing a laminar structure; this is in contrast to the Weathered clay which is essentially a grey clay with brown fissure faces. The colour of the cryoturbated clay is thought to reflect the repeated cycles of oxidation and reduction to which the layer was subjected during the cyclic freeze and thaw episodes of the Pleistocene period.

The layer is of variable thickness and does not occur everywhere along the outcrop. It is thickest at hill tops and its absence is normally restricted to valleys along the outcrop. In general, the layer seldom extends below about 6 m from the ground surface.

e) **Solifluxion Deposits.**

Almost the entire Gault outcrop is mantled by solifluxion deposits. The process by which these deposits originate, i.e. the displacement down gentle slopes, of the saturated surface layers of soil, has been described as solifluxion by Anderson (1906).
The solifluxion process has come to be closely associated with frost action although Anderson's definition does not restrict the phenomenon to cold climates. In this thesis, solifluxion and solifluxion deposits refer to the specific case where the process is thought to have occurred under periglacial conditions. Where the constituents of the solifluxion material are locally derived it is termed 'Head' and details of many of these deposits are discussed by Dines et al (1940), Smart et al (1966), Dines et al (1969).

Three main varieties of solifluxion material are generally distinguished along the Gault outcrop. The first (HH¹), consists of locally derived angular flints, up to 150 mm in size, with Chalk pellets and, towards the southern end of the outcrop, chert from the Lower Greensand escarpment, in a matrix of silty clay. This material is mainly found capping hilltops and spurs, and although it is of variable thickness, there is a tendency for the thicker deposits to be found on the higher spurs.

The second category of Head material is mainly confined to the low lying areas. It consists of two sub-groups, the first of which, HV¹, is a soft whitish grey matrix of chalk mud and angular flints up to 150 mm in size, flint fragments and, locally, may contain some chert. It is often overlain by
HV₂, which is a reddish brown deposit of very clayey silt with fragments of Chalk, flint and chert. These deposits are usually associated with coombe valleys in the Chalk escarpment.

Both categories of Head material described may be found overlying another superficial deposit in which the clayey constituents have been recognised as originating from the Gault. This deposit is described as soliflucted Gault. It represents a zone of almost complete disintegration at the top of the Gault, which has also been displaced by the solifluxion process. The layer usually consists of remoulded Gault, Chalk pellets, tufa nodules and coarse gravel-size to 150 mm flint fragments.

The soliflucted Gault is seldom more than about 1 m thick and may be absent in some areas, usually in valleys along the outcrop.

f) Dating of Head Deposits.

Investigation of Head deposits along valleys in the Gault outcrop near Brook in Kent, Kerney et al (1964) established the existence of a marker horizon representing Allerod Oscillation (Zone 11) at the base of a chalky Head deposit. Radio carbon dating of the
organic deposit forming the marker horizon gave a
dating of 9950 ± 160 B.C. (i.e. approximately 12,000 yr.
B.P.). The granular debris overlying the organic deposit
was found to be of Zone III age, while the underlying
deposit consisting of predominantly Lower Chalk and
Gault was considered to have been formed during the
main Devensian stage being probably of Zone 1 age.

Kerney et al (1964) stated that a
deposit comparable to the marker horizon organic
deposit in Brook, can be traced over a wide area of
South East England. Although no organic deposits have
been found in the sites investigated, the similarities
of the deposits and the topographic features with which
they are associated suggest that a general correlation
might exist.

The three varieties of Head material
found in Gault vales emanating from the Chalk escarpment
may, therefore, correspond to the following vegetational
zones:

Reddish brown deposit of Silt, Flints Chert (HV₂) Zone III
Base of Chalk mud with flint layer (HV₁) Zone 11
Soliflucted Gault Zone 1?

Main Devensian.

The ridges which these valleys separate
are capped by another category of Head deposits (HH₁)
which probably relate to the Wolstonian glaciation.

This tentative zonal classification presented for the Head deposits found on the Gault outcrop also correlates with the work done in the Weald by Skempton and Petley (1967).

2.4. Engineering Significance of the Geology.

A qualitative assessment of the engineering properties of the Gault which arise from its geological history are discussed in this section.

a) Effects of Sedimentary and Post-Depositional History.

The variable nature of the deposits arising from its sedimentary history suggest that considerable variations in engineering properties are likely to be exhibited by the Gault strata. Lithological variations are not simply a reflection of time of deposition, as strata deposited contemporaneously may possess significant differences in structure and mineralogy which may be reflected in the engineering parameters.

A marked degree of structural anisotrophy probably existed in the strata at the sedimentary stage due to bedding and syneresis cracks, which have now been assimilated in the pattern of macrostructural discontinuities formed by post-depositional factors such as stress release, tectonism and weathering. The orientation of these discontinuities especially where they
exhibit a pronounced preferred orientation, and the degree to which they are represented in 'undisturbed' samples tested in the laboratory, will largely pre-determine the results obtained from such tests.

The one-dimensional consolidation resulting from the weight of the Chalk altered the insitu effective stress-void ratio relationship; in addition it induced large horizontal stresses in the Gault strata. Investigations by Langer (1936), Skempton (1961), Bishop et al (1965), have shown that the effective horizontal stresses in overconsolidated clays may exceed the effective overburden pressure by 50% or more. Bishop and Henkel (1965), Brooker and Ireland (1965) have also shown that under conditions of no lateral strain in laboratory tests, the ratio of lateral to axial effective stress, $K_0$, increases with the degree of overconsolidation. Bishop et al (1965) also noted that the $K_0$ values for the London Clay at Ashford Common were greater, for a given depth, than those determined by Skempton (1961) for the London Clay at Bradwell. The differences are however, consistent with the higher preconsolidation stress estimated for the Ashford Common site.

As the Gault has been subjected to a maximum overburden stress of between about 1.5 and 3.0 times greater than that of the London Clay (Bishop et al 1965, Skempton and Hutchinson 1969), the horizontal stresses in the Gault may be even higher than those determined
for the London Clay. This deduction, however, ignores the fact that $K_0$ has been found to decrease with increasing plasticity for the heavily over-consolidated clays, Brooker (1967).

Bjerrum (1966) postulates that strong diagenetic bonds were developed in the Gault during its consolidation and that the strength of these bonds are related to consolidation pressure, its duration, the pore fluid, mineralogy and temperature of the clay. Bonds of varying strengths would, therefore, have developed in Gault as a result of its variable mineralogy and lithology.

Subsequent to the period of consolidation the Gault was unloaded as a result of erosion of the Chalk down to the Pliocene platform. The erosional cycle like the loading sequence was intermittent and the effects of the resulting overconsolidation cannot be adequately investigated in the laboratory. This is because the number of loading and unloading cycles, each of which modify the effective stress void ratio relationship, are unknown. Due to the strong bonds developed in the strata, little of the strain energy accumulated during the consolidation stage was released on unloading. The Gault will, therefore, exhibit a considerable expansion potential when its latent strain energy is dissipated, for example by weathering. In the absence of weathering, dissipation of latent strain energy will arise from the differential expansion
potential which exists within the strata as a result of its variable lithology.

Brooker and Ireland (1965) have found that while $K_0$ was essentially constant at the loading stage, it increased significantly on unloading. Brooker (1967) also observed that the strain energy stored after rebound, in the clays he investigated, increased consistently with plasticity. These observations support the hypothesis by Bjerrum (1966) that bonds may play an important part in the ability of plastic clays to expand.

The Gault outcrops everywhere below the Chalk and has consequently been subjected to leaching by ground water saturated with calcium bicarbonate. The mechanical properties of the clay may, therefore, show some dependence on the stability afforded by the 'cementing' carbonate, Bjerrum (1967).

b) Tectonic History.

It has been stated that the late Oligocene folding in southern England was not intense. The major structures produced, however, were of considerable extent, and consist of several minor elements which give rise to local complexities. The Weald, Fig. 2.3, comprises of several anticlinal axis concentrated in the central and southern areas.
The effects of tectonism on the Gault were determined by three main factors namely:

a) the depth of the competent Palaeozoic sub-floor.

b) the variable plasticity of the Gault resulting from its lithology.

c) the position of the Gault strata in the stratigraphical sequence between the competent Folkestone Beds and Chalk.

The absence of significant tectonic structures in the northern and eastern Weald noted in Fig. 2.3 derives from the occurrence at relatively shallow depth, of the Palaeozoic rocks in those areas.

The Alpine uplift induced large stresses across the Gault strata which were probably dissipated by plastic deformation and formation of fissures. The variable lithology of the strata probably also resulted in differential deformation. Such movements are likely to have occurred along boundaries of marked lithological change such as the Chalk-Gault interface, at the junction between Upper and Lower Gault and at the basal interface with the Folkestone Beds.

c) Pleistocene History.

Weeks (1969), (1970), Chandler (1970), have described a number of sites, where slip surfaces have been found on slopes inclined at angles generally exceeding 3°. These low-angle slopes are considerably
flatter than their angle of ultimate stability against landsliding, and show no topographic expression of instability.

Slip surfaces have also been found on similar low-angle slopes on the Gault outcrop as well as on slopes flatter than 3°, and including horizontal ground. Over fifty trial excavations have been made mainly in Kent, and in Oxfordshire, and these have revealed two main types of slip surfaces which can be roughly related to the natural slope topography. It should be noted that all the excavations in the Gault have been made on slopes with inclinations of about 5° or less. Conditions corresponding to, for example, the upper solifluxion (lobate) sheet at Sevenoaks which show signs of recent instability, Skempton and Petley (1967), Weeks (1969), have not been encountered.

The two types of slip surfaces distinguished in trial excavations in the Gault are referred to as principal slip surfaces and non-continuous slip surfaces.

1) Principal slip surfaces are commonly found around the lower horizon of solifluxion mantles on slopes inclined at angles of about 3° or more. These slip surfaces run almost parallel to the ground slope, and are striated in a direction close to that of the maximum ground slope. Examples of principal slip surfaces on 5° and 4° slopes are given in Figs. 2.4a, b, c.
Slip surfaces are also found on flatter slopes, but then the surface of the discontinuity tends to become less planar as the slope angle decreases. This point is illustrated by Figs. 2.4 a to g which covers slopes ranging from $5^\circ$ to $1^\circ$ inclination. The most planar slip surfaces occur on the $5^\circ$ slopes, and as the slope angle reduces, there is a general degradation in the smoothness of the slip surface associated with the solifluxion mantle. Two examples of excavation along $2^\circ$ slopes are given from which it can be seen that a principal slip surface was exposed in Pit No.1936, Fig. 2.4e, whereas none was encountered around the base of the solifluxion mantle in Pit 2400, Fig. 2.4f. It should, however, be noted that Pit No.1936 was excavated obliquely to the direction of maximum ground slopes with a slope of $4^\circ$, and that Pit No.2400 was excavated in the direction of maximum ground slope. A very irregular slip surface was exposed at the base of the solifluxion layer in the excavation along a $1^\circ$ slope, Fig. 2.4g, and although this slip surface was exposed across the length of the pit, the impression gained was that the surface exposed was actually a composite one, consisting of at least four intersecting discontinuities. The constitution of the hard, soliflucted Gault layer precluded a complete exposure of these discontinuities. It therefore appears as if the flatest slope on which principal slip surfaces occur
along the area studied in Kent, would in most cases lie between 2° and 3°. It should be pointed out that slip surfaces have been found on horizontal ground but such occurrences are rare, and are discussed later in section (iii).

ii) It has already been shown that as slope angle decreases below about 3°, the solifluxion mantle - Gault interface becomes irregular. In addition, the Head deposits become more granular, (compare for example the Head in the 5° slope, Fig. 2.4a, with that on horizontal ground at Pit No.550, Fig. 2.4j), and as horizontal ground is approached principal slip surfaces are gradually replaced by non-continuous slip surfaces. These slip surfaces usually have a curved outline when exposed on a large scale, as will be seen from Figs. 2.4h to j. Their occurrence is, however, not restricted to any range of slope angle, and they may be found in the Gault underlying principal slip surfaces along 'steep' slopes as well as within the solifluxion mantle, where these are essentially clayey deposits Figs. 2.4b, c, d, f.

The excavation along a 5° slope given in Fig. 2.4b shows a slip surface dipping at 35° to the horizontal in the cryoturbated Gault underlying a principal slip surface at the base of the solifluxion layer. The striations on the inclined slip surface were, however, aligned in a direction almost at right
angles to the direction of ground slope. Steeply inclined slip surfaces which are characteristic of non-continuous slip surfaces, were not encountered in the cryoturbated Gault in the 4° slope, Fig. 2.4c. In a borehole adjacent to the trial pit, however, slip surfaces ranging in inclination from 25° to 50° were encountered in the cryoturbated Gault, to a depth of about 5.4 m below ground level.

Trial excavations in horizontal ground at Lobb Farm near Tetsworth in Oxfordshire are shown in Figs. 2.5a, b. The site of the excavations is a valley at the foot of high ground which occurs towards the north-west, and which is capped with granular Head material. The general direction of natural ground slope in the area is from north-west to south-east and this, together with the orientation of the trial pits, is shown below.

Trial Pit GP3, Fig. 2.5a was excavated perpendicularly to the general direction of ground slope in the area, and encountered a solifluxion mantle overlying cryoturbated Gault. The interface between these layers was not marked by a slip surface. Instead, an inclined slip surface tending to rise towards the ground surface and, tending towards the horizontal at its lower end, was exposed within the cryoturbated Gault. A
dragged root was uncovered along the slip surface which together with the striations on the slip surface, was orientated in the direction of face AB of the trial pit i.e. perpendicularly to the general slope direction in the area. Limited sections of other slip surfaces of apparently similar geometry, were also uncovered in the trial excavation.

Trial Pit GP9, Fig. 2.5b was excavated in a north-south direction, obliquely to GP3 and the general direction of ground slope in the area, to establish the geometry of the main slip surface exposed in GP3. The slightly curved outline of the slip surface CY, was encountered along the face of the excavation, and was exposed for about 2 m into the face of the trial pit. The exposed surface had an apparently radial distribution of striations. Adjoining this slip surface was a more steeply inclined discontinuity, YZ, along which the striations were orientated in the north-south direction. This slip surface is considered to be part of another discontinuity in the cryoturbated Gault.

(iii) It has previously been noted that principal slip surfaces are occasionally found along horizontal ground, and two such cases have been encountered during this study, Figs. 2.4k, l. In both cases the valleys in which the slip surfaces were found, were comparatively narrow, and were associated with marked indentations and extensive coombe deposits along the Chalk escarpment.
d) **Classification and Origin of Movement along Slopes.**

The foregoing discussion of slip surfaces and their associated deposits encountered in the Gault outcrop, and published data on the subject, suggest that some form of classification is possible and tentative proposals are presented below.

a) The upper solifluxion (lobate) sheet at Sevenoaks, Weeks (1969), and the slab slide in the Lias near Uppingham, Chandler (1970b), both of which involve essentially clayey deposits, may be grouped into the first category of slope displacement. These slopes are inclined at angles near to their ultimate angle of stability against landsliding, and may become unstable under present day conditions. These conditions have not been encountered in the sections of the Gault outcrop that have been studied.

b) The displacement of essentially granular material on low-angle slopes may be regarded as a second group. Typical examples of this type of movement are given in Figs. 2.4a-f, and by Weeks (1969), from which it can be seen that a principal slip surface may only be associated with the displaced mass along slopes steeper than about $2^\circ$. This category of movement is however, mainly restricted to slopes flatter than their angle of ultimate stability against landsliding.
c) The third category of movement involves essentially clayey deposits along slopes whose angles are less than the ultimate angle of stability against landsliding. Two sub-groups can however, be distinguished:—

ci) Movement along this category of slope in the Weald Clay, discussed by Weeks (1969), (1970), is associated with a principal slip surface running almost parallel to the ground surface.

cii) For the Gault, however, and the London Clay at Boughton Hill, discussed by Weeks (1969), the associated slip surfaces have a curved outline, part of which rises steeply towards the ground surface, with the lower section tending towards the horizontal.

The difference between ci) and cii) lies in the structure and its orientation, in these deposits. The case histories in the Weald Clay involve strongly laminated and shaly clays with many silt partings, which are all orientated almost parallel to the ground surface. The Gault and London Clay are, in contrast, structurally more uniform, seldom exhibiting structures with any preferred orientation.
The origin of slip surfaces in slopes inclined at angles equal or steeper than their angle of ultimate stability against landsliding may be ascribed to previous landslip activity. Along flatter slopes, Weeks (1969), Chandler (1970a), have suggested that slip surfaces may have resulted from failure under artesian pore water pressure conditions during the Pleistocene. The strongly laminated and shaly development, and the silt partings, all of which are orientated parallel to ground surface, would make the Weald Clay discussed by Weeks (1969) 'ideal' for the development of artesian pore water pressures. Failure in such a situation would result in a slip surface running almost parallel to the ground surface, as was indeed observed. Where the structures are at a divergence with the ground surface, as in the Gault and London Clay which do not show any strongly laminar structures, then the characteristic curved outline of non-continuous slip surfaces are to be expected.

Artesian pore water pressure cannot be regarded as responsible for the displacement which formed the slip surfaces in horizontal ground. The suggestion by Weeks (1970) that these slip surfaces may have resulted from ice-heave and thrust is to some extent corroborated by the observations made in the trial pits at Lobb Farm in Oxfordshire, Fig. 2.5a, b.
Principal slip surfaces in the Gault along horizontal ground probably resulted from the drag force induced during the displacement of solifluxion materials, in a manner similar to glacial shear, Knill (1968), Higginbottom and Fookes (1970). Due to the comparatively smaller width of the valleys in which these slip surfaces have been found, and the greater extent of the coombe deposits with which they are associated, in comparative terms, a large amount of solifluxion material would have been displaced on a small area, probably giving rise to sufficient shearing stress to induce failure in the underlying Gault.
<table>
<thead>
<tr>
<th>EPOCH</th>
<th>STAGE</th>
<th>BRITISH</th>
<th>ALPINE</th>
<th>CLIMATE</th>
<th>DATE B.P.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Thousand Yr.</td>
</tr>
<tr>
<td>Holocene</td>
<td>Flandrian</td>
<td></td>
<td></td>
<td></td>
<td>10</td>
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<tr>
<td></td>
<td>Devensian</td>
<td>Würm</td>
<td></td>
<td></td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Ipswichian</td>
<td>Riss/Würm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Nettletonian</td>
<td>Riss</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Hoxnian</td>
<td>Mindel/Riss</td>
<td></td>
<td></td>
<td>-500</td>
</tr>
<tr>
<td>Pleistocene</td>
<td>Anglian</td>
<td>Mindel</td>
<td></td>
<td></td>
<td>-1000</td>
</tr>
<tr>
<td></td>
<td>Cromerian</td>
<td>Gunz/Mindel</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Beestonian</td>
<td></td>
<td></td>
<td></td>
<td>1000</td>
</tr>
<tr>
<td></td>
<td>Pastonian</td>
<td>Gunz</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bovianian</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Antranian</td>
<td>Donau/Gunz</td>
<td></td>
<td></td>
<td>-1500</td>
</tr>
<tr>
<td></td>
<td>Thurnian</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ludhamian</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Walthamian</td>
<td>Donau</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Villafranchian</td>
<td></td>
<td></td>
<td></td>
<td>-2000</td>
</tr>
</tbody>
</table>

(Based on Holmes 1965)
### TABLE 2.2.

**VARIATION IN THICKNESS OF GAULT ACROSS THE WEALD**

<table>
<thead>
<tr>
<th>Location</th>
<th>Grid Reference</th>
<th>Thickness (m)</th>
<th>References</th>
<th>Remarks</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td>L.G.</td>
<td>U.G.</td>
<td>Total</td>
</tr>
<tr>
<td>Aycliff</td>
<td>TR 294 395</td>
<td>14</td>
<td>445</td>
<td>58.5</td>
</tr>
<tr>
<td>Folkestone Warren</td>
<td>TR 250 375</td>
<td>10</td>
<td>40-50</td>
<td>50-60</td>
</tr>
<tr>
<td>Copt Point</td>
<td>TR 243 365</td>
<td>10</td>
<td>22.0</td>
<td>32.0</td>
</tr>
<tr>
<td>Wye</td>
<td>TR 055 469</td>
<td>8.5</td>
<td>48.0</td>
<td>56.5</td>
</tr>
<tr>
<td>Charing</td>
<td>TQ 956 500</td>
<td></td>
<td></td>
<td>60.0</td>
</tr>
<tr>
<td>Maidstone</td>
<td>TQ 778 575</td>
<td>8.5</td>
<td>52.5</td>
<td>61.0</td>
</tr>
<tr>
<td>Boxley</td>
<td>TQ 774 595</td>
<td></td>
<td></td>
<td>62.0</td>
</tr>
<tr>
<td>Gossington</td>
<td>TQ 747 599</td>
<td></td>
<td></td>
<td>68.5</td>
</tr>
<tr>
<td>Trottiscliffe</td>
<td>TQ 636 521</td>
<td>12.0</td>
<td>44.0</td>
<td>&gt;56.0</td>
</tr>
<tr>
<td>Kemsing</td>
<td>TQ 555 595</td>
<td>12.5</td>
<td>85.5</td>
<td>98.0</td>
</tr>
<tr>
<td>Otford</td>
<td>TQ 524 594</td>
<td>13.0</td>
<td>84.5</td>
<td>97.5</td>
</tr>
<tr>
<td>Brasted</td>
<td>TQ 478 568</td>
<td>20.0</td>
<td>56.2</td>
<td>76.2</td>
</tr>
<tr>
<td>Limpsfield</td>
<td>TQ 410 550</td>
<td></td>
<td></td>
<td>82.6</td>
</tr>
<tr>
<td>Clandon</td>
<td>TQ 041 510</td>
<td>30</td>
<td>88</td>
<td>118</td>
</tr>
<tr>
<td>Upper Beeding</td>
<td>TQ 207 123</td>
<td>47.5</td>
<td>59.5</td>
<td>107</td>
</tr>
</tbody>
</table>

**References:**
The Probable Palaeogeography of Gault, Upper Greensand, and Times
(Middle–Upper Albion, Isopachytes based on Wooldridge: Linton 1955)

Fig 22
Oligocene Fold Structure of SE England Fig 2.3

After Benham and Wright 1969

Ankland Area
Tertiary
Chalk
Tertiary
Chalk
Tertiary
Chalk
1 HEAD  Dense brown clayey Silt with flint fragments

1\ HEAD  Soft remoulded grey CLAY with flint fragments

1\ HEAD  Soft reddish brown clayey Silt with clay patches

1\ HEAD  Matrix of chalkwash, flints and grey brown clay

2 SOLIFLUCTED GAULT CLAY  Soft finely fissured grey brown CLAY with flints

3 CRYOTURBATED GAULT CLAY  Firm finely fissured grey brown CLAY

\[\text{Level at which water was encountered during excavation.}\]

\[\text{Slip surface.}\]

CH. 736+50  PIT No. 10600
1. HEAD  
   Very soft finely fissured brown CLAY with flints remoulded and sheared.

2. SOLIFLUCTED CHALK MARL  
   Soft finely fissured grey brown CLAY with chalk pellets remoulded and sheared.

3. CRYOTURBATED GAULT CLAY  
   Soft finely fissured grey brown CLAY.

   Level at which water was encountered during excavation.
   
   Slip surface.

CH.697+00  PIT No.6680
1 HEAD
Very stiff finely fissured reddish brown CLAY with flints.

2 SOLIFLUCTED GAULT CLAY
Stiff finely fissured grey brown CLAY with flints and chalk pellets.

2a SOLIFLUCTED GAULT CLAY
Stiff finely fissured grey brown CLAY with calcareous concretions generally sheared.

3
Brown SILT and SILTSTONE band with phosphatic nodules.

4 CRYOTURBATED GAULT CLAY
Very stiff fissured gray brown CLAY.

5
Red SILTSTONE band.

Pit remained dry during excavation.

Slip surface.

CH. 755 + 60
PIT No 12510
1 **HEAD**

Matrix of cobble size angular flints and brown clayey SILT.

2 **SOLIFLUCTED GAULT CLAY**

Soft finely fissured grey brown CLAY with flints generally sheared.

3 **CRYOTURBATED GAULT CLAY**

Firm finely fissured grey brown CLAY.

Pit remained dry during excavation.

---

**Slip surface**

---

CH. 608+00

**PIT No. 1994/2**
1 HEAD

Soft finely fissured brown CLAY with flints.

1st HEAD

Matrix of flint gravel and pale grey CLAY.

2 SOLIFLUCTED GAULT CLAY

Soft finely fissured grey brown CLAY with chalk pellets.

3 CRYOTURBATED GAULT CLAY

Firm finely fissured grey brown CLAY.

Pit remained dry during excavation.

Slip surface.

CH. 590+00

PIT No. 1936
1 HEAD  Soft finely fissured grey brown CLAY with flints.

2 SOLIFLUCTED Soft finely fissured whitish grey CLAY sheared.
GAULT CLAY

3 CRYOTURBATED Soft finely fissured whitish grey CLAY.
GAULT CLAY

3a CRYOTURBATED Soft fissured grey brown CLAY.
GAULT CLAY

Level at which water was encountered during excavation.

Slip surface.

CH. 654 + 10

PIT No. 2400

Fig 24
1 HEAD Soft finely fissured reddish brown CLAY with flints.

1A HEAD Matrix of flints and reddish brown CLAY.

2 SOLIFLUCTED GAULT CLAY Hard finely fissured grey brown CLAY with flints chalk pellets and tufo nodules.

3 CRYOTURBATED GAULT CLAY Stiff finely fissured grey CLAY with calcareous concretions.

5A CRYOTURBATED GAULT CLAY Stiff finely fissured grey brown CLAY.

Pit remained dry during excavation.

Slip surface.

CH.695+00 PIT No.6600/1
1 Head: Firm to stiff medium brown silty CLAY grey mottling. Some flints up to 25 mm.

IA Head: Matrix of mottled brown CLAY Mass flints and pockets of reddish brown sand

2 SGH: Firm remoulded grey brown CLAY Generally sheared tufa and carbonate patches

3 Gypswalled Stiff finely fissured grey yellow brown CLAY Some polished Gault fissure faces selenite crystals and carbonate crystals

3A " Stiff finely fissured grey yellow brown CLAY Some polished fissure faces selenite crystals and carbonate patches Fissured on 25 mm to 100 mm scale becoming hard fissured on 150 mm scale with depth

Slip surface
Pit remained dry during excavation

CH.  Pit No. 698
1 HEAD
   Soft finely fissured brown CLAY with flints.

1a HEAD
   Matrix of soft finely fissured brown CLAY and flints.

1b
   Layer of chalk pellets and tuba nodules.

2 SOLIFLUCTED
   GAULT CLAY
   Firm finely fissured grey brown CLAY generally sheared.

3 CRYOTURBATED
   GAULT CLAY
   Stiff fissured grey brown CLAY.
   Pit remained dry during excavation.
   Slip surface.
1 HEAD
Soft finely fissured mottled greenish grey and brown CLAY with boulder size flints remoulded and sheared.

2 SOLIFLUCTED
GAULT CLAY
Soft finely fissured whitish grey CLAY with angular flints sheared.

3 CRYOTURBATED
GAULT CLAY
Firm finely fissured grey brown CLAY.

4 WEATHERED
GAULT CLAY
Firm fissured grey brown CLAY.

5 GAULT CLAY
Stiff dark grey CLAY.

Pit remained dry during excavation

Slip surface

CH. 636+40

PIT No. 650
1 HEAD
Matrix of reddish brown Silt Chalkwash and flints.

1a HEAD
Soft whitish grey chalkwash and flints.

1b HEAD
Matrix of reddish brown Silt chalkwash grey clay and flints.

2 SOLIFLUCTED GAULT CLAY
Very soft finely fissured grey brown CLAY and flints remoulded and sheared.

3 CRYOTURBATED GAULT CLAY
Soft finely fissured grey brown CLAY.

6
10mm band of pale grey very soft CLAY.

4 WEATHERED GAULT CLAY
Soft finely fissured blue grey and brown CLAY with pockets of glauconite, some mudstone.

▽
Level at which water was encountered during excavation.

- - -
Slip surface.

CH. 581+ 10 PIT No. 1906
1 HEAD Soft finely fissured reddish brown CLAY with flints sheared.

1A HEAD Matrix of flint gravel chalkwash and reddish brown CLAY.

2 SOLIFLUCTED Soft finely fissured grey brown CLAY with chalk pellets Gault CLAY sheared.

3 CRYOTURBATED Firm finely fissured grey brown CLAY carbonate patches Gault CLAY

   Pit remained dry during excavation.

   Slip surface.

CH. 718+10 PIT No. 8800

Fig. 2.44
1. **HEAD**  Soft finely fissured brown CLAY with flints

2. **HEAD**  Matrix of coarse gravel size Flints, Sand, Silt and Clay

3. **CETYTURBATED**  Soft to firm finely fissured grey-brown CLAY Gault

   Scale of fissuring increasing with depth.

---

**Lobb Farm Trial Pit GP3**

Fig. 2.5a
1: HEAD  Soft finely fissured brown CLAY with flints

2: HEAD  Matrix of coarse gravel size Flints, Sand, Silt, and Clay

3: CRYOTURBATED  Soft to firm finely fissured grey brown CLAY GAULT  Scale of fissuring increasing with depth

---

Lobb Farm Trial Pit GP9
CHAPTER 3

STABILITY OF THE OTFORD TRIAL EMBANKMENT

3.1 Background to the Investigation

a) Introduction.

The construction of the proposed M.25 motorway in Kent will involve the excavation of considerable quantities of Gault clay. The index properties of the clay were found to be extremely variable both vertically and laterally, and the various facies of the Gault cannot be classified with regard to their suitability as fill material, without resorting to a complex system based on palaeontology. Also, where the Gault complies with Ministry of Transport (Environment) specification for fill material, it is in general only marginally suitable.

Another factor that emerged during the site investigation is that certain facies of the Gault with apparent silt size particles show all the characteristics of a clay when subjected to moderate disturbance. Index tests on samples from these facies of the Gault generally give liquid limit values in excess of 75%. Thus what appears on visual examination to be a silt is in reality silt-size aggregation of clay particles. As some disaggregation was apparently achieved by the mixing process in the liquid limit test, it was necessary to establish how this material would behave under normal site handling conditions.
The trial pit programme carried out as part of the M.25 investigation, established the existence and form of slip surfaces occurring on slopes generally less than 5° in inclination, and including horizontal ground along the Gault outcrop. Details of these slip surfaces have been discussed in Chapter 2. Basically, two main types of slip surfaces (principal and non-continuous) were encountered. On ground sloping at angles of 2.5° or more, principal slip surfaces are generally associated with the base of the solifluxion mantle, running almost parallel to the ground surface. Non-continuous slip surfaces, as the name implies, are separated by unsheared material, and they do not appear to be restricted to any range of slope inclination. They may be found in the Gault below principal slip surfaces or within the solifluxion mantle where this is essentially a clayey deposit. They do, however, tend to be prevalent along the flatter slopes of less than about 2.5° inclination.

It is now well established that the shear strength along discontinuities is much lower than those obtained from tests on unsheared material, Skempton and Petley (1967). The shear strength that would be mobilized below an embankment constructed on flat ground in which non-continuous slip surfaces exist would, however, depend on the proportion of the failure path that utilises pre-existing slip surfaces.
A trial embankment was therefore designed to determine:

a) the suitability of the aggregated facies of the Gault (which was considered from the constructional point of view, to be most critical) as fill material,

b) shear parameters for the design of embankments where the potential failure path is likely to utilise pre-existing non-continuous slip surfaces.

b) Choice of the Trial Site.

The choice of the trial site, Fig. 3.1, was dictated by factors discussed previously, i.e. that the fill material should be of the aggregated variety and that the area should contain non-continuous slip surfaces. These prerequisites were established by a site investigation programme consisting of boreholes and trial pits at the trial embankment site and at the site of the excavation for fill material. The geotechnical properties of the Gault at the trial site and at the site of the excavation for fill material are given in the table overleaf:
### Geotechnical Properties of the Gault at the Trial Site.

<table>
<thead>
<tr>
<th>Site</th>
<th>Soliflucted Gault</th>
<th>Groturubated Gault</th>
<th>Weathered Gault</th>
<th>Unweathered Gault</th>
</tr>
</thead>
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<tr>
<td></td>
<td>L.L.</td>
<td>P.L.</td>
<td>m/c</td>
<td>L.L.</td>
</tr>
<tr>
<td>Trial Embankment</td>
<td>37 - 122</td>
<td>25 - 31</td>
<td>30 - 34</td>
<td>77 - 95</td>
</tr>
<tr>
<td>Fill</td>
<td>33 - 90</td>
<td>30 - 31</td>
<td>32 - 34</td>
<td>89</td>
</tr>
</tbody>
</table>
The observations made in the trial excavations at the trial embankment site are shown in Figs. 3.2a, b, from which the characteristic curved outline of non-continuous slip surfaces, tending to rise towards the ground surface, may be seen.

The Gault has a maximum thickness of about 100 m in the Otford area, Dines et al (1969); however, the thickness at the trial site is estimated to be about 61 m, of which the lower 15 m or so comprises the Lower Gault, Owens (1971). The Gault involved in the trial area is thus located within the middle zone of the Upper Gault in the area.

c) Design of the Trial Embankment.

For the objectives outlined to be completely fulfilled, it was necessary to design an embankment that would fail during construction. The height of the embankment was fixed by the maximum height proposed along the M.25 motorway, which is about 10.5 m. The side slope of the embankment was determined from stability analysis in which the following assumptions were made:

i) principal slip surfaces exist to a depth of 4.55 m below ground level along which the shear strength mobilized corresponds to $C' = 0, \phi' = 14^\circ$. 
ii) the parameter $\beta$ equals 0.5. This value of $\beta$ was chosen intuitively from the analysis of construction pore water pressure below an embankment on the Weald clay, along the Sevenoaks By-Pass, Weeks (1970).

iii) Peak effective stress parameters of $c' = 10 \text{ kN/m}^2$, $\phi' = 23^\circ$ determined from triaxial tests on 38 mm diameter samples, were applied to the section of the failure path through the fill material.

Under the assumed conditions a side slope of 1 (vertical) on 2 (horizontal) resulted in a minimum factor of safety of 0.7; this side slope was adopted to ensure failure during construction.

Due to the shortage of Gault clay fill, it was decided to use the bulk of the fill in the main body of the embankment, and to use the granular Head material from the excavation site in the ramps at each end of the Gault embankment, Fig. 3.3. This measure resulted in a 50 m (164 ft.) length of Gault clay embankment, which is considered long enough to be free of interference from the granular material at the ends, and also, to be amenable to two-dimensional analysis.
Details of the piezometer installation in the foundation material are given in Table 3.1 and in Fig. 3.4. The methods of installation and measurement are described in Appendix 1d (p370).

3.2 Initial and Construction Pore Water Pressure

a) Initial Pore Water Pressure.

The pore water pressure, $H$, recorded in all the piezometers, at the start of construction, are given in Table 3.1, and they are shown plotted against depth of piezometer below ground level in Fig. 3.5. The figure clearly shows the existence of two perched water tables pertaining to the solifluxion and cryoturbated Gault layers. The pore water pressure distribution in the weathered and unweathered Gault layers corresponds to a water table at about 0.9 m below ground level, and the corresponding levels in the soliflucted and cryoturbated layers are about 0.9 m and 1.45 m respectively. All the piezometers in the Gault were below their respective water tables, at the start of construction.

b) Construction Pore Water Pressure.

1) General.

The pore water pressure response to loading in each piezometer was measured twice daily, at the start and end of each day's loading, until failure occurred 34 days after the start of construction. The pore water pressure recorded for each piezometer has
been plotted, and typical examples are given in Figs. 3.6. The excess pore water pressures recorded in the foundation along Section B, for four loading stages corresponding to the 18, 25, 31, 34 days after the start of construction are shown in Fig. 3.7. The end of each loading stage corresponds to the addition of about 2.5 m of fill, and the attainment of the final height of fill material above successive piezometer groups in the foundation.

It can be seen from Fig. 3.7 that the rise in excess pore water pressure below the embankment slope continued after the final level of fill had been attained above each piezometer. In addition, the pore water pressure response to loading below the solifluxion layer increases with depth and, although the rate of increase varies, it is almost uniform within each layer of the Gault at the site. The trend indicated for the cryoturbated Gault has been used to predict the excess pore water pressure distribution in pz. Group 3, in which there was a faulty piezometer installation (B3, Table 3.1) at the top of the cryoturbated layer. The data for the solifluxion layer is insufficient to determine a general trend, if any, in the layer. The apparently inconsistent nature of the pore water pressure response is considered a reflection of the variable composition of the solifluxion layer.
The variation in pore water pressure response to loading in the Gault is consistent with the existence of perched water tables at the site. This fact, however, cannot be regarded as wholly responsible for the observed variation, as different responses were noted in the weathered and unweathered zones, both of which were subject to the same pore water regime at the start of construction. The length of the drainage path is another factor capable of introducing some variation in the pore water pressure response to loading.

The pore water pressure response to loading is further illustrated in Figs. 3.8, in which the excess pore water pressure rise with stress increment is shown for piezometers at approximately 2.4 m and 4.5 m below ground level, and for all the piezometers below the shoulder of the embankment. The increase in vertical stress, after the final height of fill had been attained above piezometers below the slope, especially those situated near to the toe (Group 1), is quite small. The observed increase in pore water pressure may, therefore, be due to the horizontal stresses induced by loading at the shoulder. This implies that pore water pressures would have developed outside the toe of the embankment, where the principal stresses are likely to have been more nearly horizontal than vertical. Such observations have

*The vertical stress increments for the four loading stages discussed previously, have been calculated as suggested by Mirata (1969). Details of the method are given in Appendix 3a. . . .
been reported in a lightly overconsolidated clay by Parry (1967), (1968) and in many investigations involving soft clays, for example by Morgason and Symmons (1969), Rowe (1967), (1968).

The rise in pore water pressure occurred gradually up to the period preceding failure when a rapid increase was observed in all piezometers except B6 and B4, Figs. 3.6a, b. The post-failure investigations established that these piezometers were located on the potential failure surface. The onset of this rapid rise in pore water pressure below the embankment shoulder, Fig. 3.8c, also coincided with the appearance of cracks along the lower section of the embankment slope. This sharp rise in pore water pressure just before failure was unexpected in a stiff clay such as the Gault, as this phenomenon is generally associated with local structural collapse in soft normally consolidated deposits, Morgensten (1967), Al-Dhahir et al (1969), Rowe (1969), James (1970). Its occurrence at Otford, probably marks the onset of plasticity in the Gault. During the post-failure investigation, the moisture content immediately adjacent to the failure surface was consistently higher than that in the surrounding soil. This is consistent with the observed pore water pressure gradients in the foundation which would have caused a migration of pore fluid to the failure surface. Henkel (1956) also observed that post-peak displacements in a retaining wall
failure in the London Clay were associated with a migration of water to the slip surface. Similar observations can be made in shear box tests, see for example, Schofield and Wroth (1968).

3.3 Comparison of Predicted and Observed Pore Water Pressure

In problems relating to the stability of foundations under imposed loads the concept of pore pressure parameters, Skempton (1954), provides a basis for estimating the magnitude of the pore water pressure that will be generated by an applied load. According to Skempton (loc. cit.) when a saturated element of soil is subjected to an increase in stress, an excess pore water pressure $\Delta U$ is set up which can be expressed as:

$$
\Delta U = B \left[ \Delta \sigma_h + A \left( \Delta \sigma_v - \Delta \sigma_h \right) \right]
$$

where $\Delta \sigma_v$ denotes major principal stress increment, $\Delta \sigma_h$ minor. $A$ and $B$ are pore pressure parameters.

The parameter $B$ depends on the compressibility of the soil structure as a whole, relative to that of the fluids in the voids. In a saturated soil loaded sufficiently quickly so that no drainage occurs during the loading, deformation will occur without any change in volume and any vertical compression will be accompanied by a lateral expansion. Under these
conditions the parameter B will be equal to unity. At the other extreme, a perfectly dry soil will result in a B value of zero due to the high compressibility of air.

The value of A depends largely on whether the soil is normally consolidated or over-consolidated, the proportion of the failure stress applied and conditions of testing i.e. whether by compression or extension. In addition the value of A is liable to show marked sensitivity to sample disturbance. The earlier estimates of the coefficient, Bishop and Henkel (1965) from tests on remoulded samples, indicated a range of variation from +1 for normally consolidated clays to about -0.5 for over-consolidated samples. Recent estimates based on undisturbed samples, for example by Simons and Som (1969), however, indicate that in compression, the A values for the London Clay decrease with increasing stress-level from about 0.6 to 0.4.

The expression given in Eq. 1 neglects the effect of the intermediate principal stress on pore water pressure. The conditions existing in most stability problems, however, approximate to plane strain in which the change in intermediate principal stress does not equal the change in minor principal stress as in the standard triaxial tests. Studies by Skempton (1948), Hansen and Gibson (1949) indicate that the form of the equation remains unaltered but that values of A in the field may be higher than those determined from
laboratory tests. The laboratory determinations of A referred to were, however, made using remoulded samples. In addition, no account is taken of the probable reorientation of the direction of principal stresses which may occur. The significance of this limitation, in the absence of tests on which the principal stresses can be rotated (true triaxial test), can only be assessed from any discrepancy which may occur between observed and predicted values of pore water pressure in the field.

The parameters A and B may be combined to give a simpler expression viz:

\[ \bar{E} = \frac{\Delta U}{\Delta \sigma_v} \text{ where } \bar{E} = B \left[ 1 - (1 - A)(1 - \frac{\Delta \sigma_h}{\Delta \sigma_v}) \right] \] .... (2)

The expression for \( \bar{E} \) involves both 'A' and the stress ratio \( \frac{\Delta \sigma_h}{\Delta \sigma_v} \) which are also inter-dependent, Simons and Som (1969). The latter ratio also depends on the degree of lateral confinement, and in a saturated soil \( \bar{E} \) will only be equal to 1.0 when \( \Delta \sigma_h = \Delta \sigma_v \), and under \( K_o \) conditions. These conditions are usually achieved during the application of an all round pressure in the triaxial test and under the rigid confinement in the oedometer test. The influence of principal stress ratio on \( \bar{E} \) has been demonstrated by Bishop and Bjerrum (1960). According to equation (2), \( \bar{E} \) will exceed unity when \( \Delta \sigma_h \) exceeds \( \Delta \sigma_v \), provided the value of A is less than unity. Such conditions may be attained in the foundation material
around the toe of high embankments. The pore pressure parameters as previously stated, are, however, only relevant to those cases where the intermediate and minor principal stress increments are equal.

In a situation where the macro fabric allows drainage to occur during the initial loading stage, possibly as a result of the high permeability of the soil under low stresses, the initial $B$ will be less than unity. On further loading however, $B$ may increase correspondingly with the reduction in permeability under increased stresses, Gibson and Marsland (1960), Penman and Watson (1963), Rowe (1968).

b) **Flexibility of Measuring System.**

The measurement of pore water pressure in the field is affected by the flexibility of the measuring system. As no pore water pressure measuring system is absolutely rigid, some 'drainage' is bound to occur due to differences in pressure between the soil and the measuring system during the moment of loading. Some flow of water into or out of the measuring system will occur before equilibrium is reached, and if the measuring system has a high flexibility, then the time-lag may be large because of the delay in achieving equilibrium between the two systems. During the period of delay, some dissipation of pore water pressure may occur, so that the pore water pressure that will be measured eventually will
be less than the actual value generated during the load application. The importance of time-lag has been discussed by Hvorslev (1951), Whitman et al (1961), Gibson (1965), Bishop and Henkel (1965). These studies have recognised that the principal causes of time-lag are the flexibility of the measuring system, the filter and piezometer tip dimensions and permeability of the soil. The permeability of the soil controls the rate of flow into or out of the piezometer, and the size of filter and piezometer tip affect the effective intake area. The flexibility of the measuring system is determined by the compressibility of the fluid filling the device and the volume expansibility of the device itself. The size of tubing in the measuring system also determine the volume of flow required for response.

At the trial embankment site, steps were taken to minimise the factors that can give rise to time-lag in pore water pressure response to loading. High air-entry porous ceramic piezometer tips were used in conjunction with double-walled polythene piezometer leads of 1.5 mm (\(\frac{1}{16}\) in.) internal diameter. The piezometer tips were surrounded with a sand filter to increase their intake area, and so reduce their response time. The close-circuit twin-lead system enabled de-aired water to be circulated through each system to flush out gas bubbles. The presence of gas bubbles would cause an increase in time-lag by diminishing the permeability of the soil or piezometer tip. Also, the change in volume
of gas in response to loading, will add to the volume of flow required to produce piezometric response, thereby increasing the time-lag. The hut containing the measuring panels was located downslope, as near to the toe of the embankment as was possible, so as to keep the length of the piezometer leads to a minimum. The maximum length of lead used was 50 m (165 ft.).

The process of piezometer installation unavoidably produces a temporary change in pore water pressure, giving rise to stress-adjustment time-lag. Boring at the trial site was by water flush rotary rig which is generally considered to minimise this effect. The time-lag introduced by boring and installation however, only affects the readings taken shortly after installation. Construction at the trial embankment site did not start until after equilibrium conditions had been attained in the piezometers.

The response time has been estimated by the method suggested by Hvorslev (1951) in which the compressibility of the soil skeleton is ignored. This assumption can lead to errors in silts and clay. However, Penman (1960) has shown that although Hvorslev's method generally under-estimates the degree of pore water pressure equalization it can be used to obtain the response time in hydraulic piezometers for nearly complete equalization of pore water pressure.

*The more generally accurate method by Gibson (1963) does not take into account the flexibility of (polythene) piezometer leads in a close-circuit installation. These have been considered by Penman (1960) also using the method by Hvorslev (1951).
The quantity of water Q which flows into a
tip under a pressure difference p in time t could be
expressed as:

\[ Q = \frac{F \cdot pt}{\delta_v} \] ........................(5)

where F defines the intake factor which depends on the
shape and dimensions of the filter tip. For a cylindrical
tip of length L and diameter D:

\[ F = \frac{(2\pi L)}{\left[2.3 \log_{10} \left(\frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right)\right]} \text{(Cm)} \]

For the trial the filter tip dimensions were:

\[ L = 30 \text{ Cm}, \quad D = 10 \text{ Cm} \quad \text{and} \quad F = 104.2 \text{ Cm}. \]

K is the permeability of the soil and a value of \(4 \times 10^{-8}\)
Cm/Sec. commonly obtained from response time in standpipe
piezometers may be assumed.*

If the pressure recorded by a piezometer system
at time \(t = 0\) is out-of-balance by an amount \(P_0\) from the
true steady pressure in the ground and after a time
interval the out-of-balance pressure reduces to \(P\), then
the degree of equalization may be expressed as \((P_0 - P)/P_0\).
Values of a dimensionless group of variables \(\frac{\kappa \cdot t}{\sqrt{\psi_v}}\) for
various percentages of \((P_0 - P)/P_0\) are quoted by Penman
(1960).

*Higher values of K have been obtained from insitu constant
head tests discussed later. These higher values of K
cannot be attributed to improper sealing of piezometers
as the boreholes were generally backfilled with a bentonite-
cement grout. Vaughan (1969) has shown that the permeab-
ility of a grout seal must be considerably higher than the
ground permeability before a significant error results in
piezometer readings. The assumed K value is in addition
consistent with observed dissipation rates below motorway
embankments on the Gault in Surrey.
For a degree of equalization of 99.9%,

\[ \frac{EFK}{\gamma W} = 6.91 \] where \( \gamma W \) is the specific weight of water, and \( \gamma \) is a volume factor; the relationship between volume change and pressure change in the piezometer system is assumed to be linear and for unit pressure increase the volume of water entering the system defines the volume factor. Penman (1960) conducted experiments to determine the values of \( \gamma \) for single walled 980 ft. length polythene and 340 ft. length nylon tubes of 3.0 and 2.8 mm bore respectively. The value of \( \gamma \) decreased from 58.7 x 10^{-4} \text{cm}^3/\text{gm} for the polythene tube to 4.96 x 10^{-4} \text{cm}^3/\text{gm} for the nylon tube. It would seem from these results that a value of \( \gamma \) of say 10 x 10^{-4} \text{cm}^3/\text{gm} could be assumed for the double-walled polythene tube with a maximum length of 50 m (165 ft.) and 1.5 mm bore, used in the trial.

The response time \( t \) for 99.9% equalization of pore water pressure can be expressed as:

\[ t = \frac{EFK}{6.91 \gamma W} \]

\[ (4) \]

It can be seen that \( EFK \) needs to be large compared to \( \gamma \) for equilibrium to be established quickly. Substituting the relevant values in Eq. (4) gives:

\[
\begin{align*}
\gamma \min &= \frac{6.91 \times 10 \times 10^{-4} \times 1}{4 \times 10^{-8} \times 104.2 \times 60} \\
&= \frac{28 \text{ mins}}{38 \text{ mins}}
\end{align*}
\]
This response time is consistent with those that were commonly observed for manometer readings (of pore water pressure) to return to their original values after constant head permeability tests. Although the value of volume factor assumed in the calculation may be in error, the likely maximum error would be accounted for by the higher insitu permeability which, as will be seen later, is up to 100 times greater than the value assumed in the calculation.

It therefore appears that the flexibility of the system is unlikely to introduce significant errors in the measured pore water pressures during construction.

c) Construction $B$ Values in the Foundation.

Values of $B$ corresponding to the initial loading stage when 2.75 m of fill was placed above all the piezometers, and at the end of construction, are given in Table 3.2 Cols. 1 and 2. The measured $B$ values are seen to increase with depth in each piezometer group, and generally during the construction of the embankment.

For the cryoturbated Gault, $B$ generally increases with distance from the centre of the embankment. The high end of construction $B$ of 1.73 in pz. B6 Table 3.2, may be partly due to its location, which was subsequently established to be below the
failure surface. However, the end of construction pore water pressure in pz. B6 is approximately at the same level as those in pzs. B8 and B4 also located near to the base of the cryoturbated Gault, Fig. 3.9. This suggests that some lateral migration of pore water pressure might have occurred. The high $B$ value in pz. B6 is due to the high excess pressures and the low vertical stress increments near the toe of the embankment. The magnitude of $B$ outside the embankment shoulder, especially near to the toe probably also reflects the influence of the principal incremental stress ratio.

The average $B$ values below the shoulder of the embankment which are summarised below, show significantly greater values of $B$ at the end of construction than at the initial stage, except in the unweathered Gault in which maximum $B$ values were attained. The average values of the ratio

<table>
<thead>
<tr>
<th>Zone</th>
<th>Initial Stage $B$ Fill = 2.75 m (1)</th>
<th>End of Construction $B$ Fill = 10.65 m (2)</th>
<th>$(2)/(1)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solifluxion Deposits</td>
<td>0.18 (0.88)</td>
<td>0.33 (0.63)</td>
<td>1.64</td>
</tr>
<tr>
<td>Cryoturbated Gault</td>
<td>0.25 (0.80)</td>
<td>0.36 (0.53)</td>
<td>1.43</td>
</tr>
<tr>
<td>Weathered Gault</td>
<td>0.30 (0.72)</td>
<td>0.45 (0.46)</td>
<td>1.49</td>
</tr>
<tr>
<td>Unweathered Gault</td>
<td>0.52 (0.62)</td>
<td>0.56 (0.40)</td>
<td>1.08</td>
</tr>
</tbody>
</table>

$\Delta \sigma_h / \Delta \sigma_v$ in each zone of soil is shown bracketed. These observations appear anomalous as, because of symmetry, the minor and intermediate principal stress...
increments below the shoulder should be approximately the same, and as the variation in the laboratory values of A for the corresponding stress increments is not very significant, then according to Eq. (2), B should decrease as the ratio \( \Delta \sigma_h/\Delta \sigma_v \) falls below unity.

For both the initial and final loading stages considered, the average rate of construction was about 0.5 m/day. A much faster rate of construction of just over 1 m/day was achieved during the placing of the final 2.5 m of fill material. The B values for this loading stage are given in Col. 3 Table 3.2. For piezometers in Groups 1 and 2, nearest to the toe of the embankment, B generally exceeds unity. Although there was an increase in pore water pressure in pz. Bl, there was no corresponding increase in the vertical stress increment and B approached \( \infty \). This clearly demonstrates the limitation of the parameter B in predicting pore water pressure below the embankment slope.

The observed average B values below the embankment shoulder are given below and for comparison, the average end of construction B are also given:

<table>
<thead>
<tr>
<th>Zone</th>
<th>Final Loading B</th>
<th>Construction B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solifluxion Deposits</td>
<td>0.50</td>
<td>0.33</td>
</tr>
<tr>
<td>Cryoturbated Gault</td>
<td>0.72</td>
<td>0.36</td>
</tr>
<tr>
<td>Weathered Gault</td>
<td>0.80</td>
<td>0.45</td>
</tr>
<tr>
<td>Unweathered Gault</td>
<td>0.86</td>
<td>0.56</td>
</tr>
</tbody>
</table>
As all the piezometers in the Gault were below their respective water tables at the start of construction, the impression might be gained from the final loading stage $\bar{B}$ values, that the rate of construction during the early stages was not sufficiently rapid to cause an undrained pore water pressure response. This would also appear to be consistent with the observed anomaly between the ratio of the principal stress increments and the $\bar{B}$ values which were noted earlier. Whereas this may have been the case, it should be noted that the macro-permeability of the Gault is probably high under low stresses but decreases as the stress level is increased. Open fissures may be closed and any permeable seams may become constricted, so that even under constant rate of loading $\bar{B}$ may increase purely as a result of the decrease in permeability. The influence of stress level on permeability has been discussed by Rowe (1967), Bishop and Al-Dhalîzir (1969), Weeks (1970).

One of the features of all the graphs of piezometer readings during construction, Figs. 3.6, is the reduction in pore water pressure that occurred overnight, especially during the early stages of construction. Fig. 3.9 shows the piezometric lines at the base of the Cryoturbated Gault after 18 days of construction, and at the end of construction. The smoothness of these lines at both stages of construction,
to some extent reflect the apparent construction dissipation which would have contributed towards a smoothing out of pressure gradients. Included in Fig. 3.9 are also the distributions of 'undrained' increments of pore water pressure at the base of the cryoturbated Gault. These distributions have been obtained by adding the 'undrained' increments during the placement periods; this however, does not produce the same values of pore water pressure that would have resulted under undrained conditions, indicating that some drainage probably also occurred during the load application. The absence of any significant pressure gradients in the 'undrained' distributions may also be regarded as a further indication of drainage, induced during the application of the load. It can also be seen from Fig. 3.9 that the apparent construction dissipation decreased from about 50% at the 18 day stage, to about 25% at the end of construction stage. This reduction is a reflection of the variation in pore water pressure during construction. It can be seen from the piezometric graphs, Figs. 3.6, that the overnight dissipation of pore pressure which was so prevalent during the early stages, virtually ceased towards the end of construction. It was during this latter period that the incremental $\tilde{B}$ values also approached unity below the embankment shoulder. The existence of perched water tables at the site and the influence of the length of the drainage path may also
be reflected in the calculated $B$ values.

It may be concluded that if conditions at the trial site are typical of the Gault, then for the range of depths likely to be involved in the stability of motorway embankment, the average value of $B$ during construction is unlikely to exceed 0.5. This conclusion is supported by the observed pore water pressures in the foundation of motorway embankments on the Gault in Surrey. Data by Rowe (1972) also suggest a $B$ value of generally less than 0.5 in the London Clay foundation of the Ardleigh dam.

d) Construction $A$ values in the Foundation.

The parameter $A$ should be much more suitable for predicting pore water pressure as it takes into account both the vertical and the horizontal stress increments. As the parameter only applies to conditions in which the minor and intermediate principal stress increments are equal, only the piezometers below the embankment shoulder are considered.

The values of the parameter $A$ determined from several stages of consolidated undrained test on 100 mm diameter samples are shown plotted in Fig. 3.10 from which it can be seen that $A$ decreases with stress level from 0.50 to 0.26. For the range of stresses involved in the foundation material little error will arise from the assumption of a constant $A$ value of say 0.4. The calculated and observed pore water pressure
for the initial and end of construction stages, corresponding to the addition of 2.75 m and 10.65 m of fill material, are given in Table 3.4. Details of the horizontal and vertical stress increments are summarised in Appendix 3a.

The average ratios of observed to predicted pore water pressure for the various zones of soil at the site are summarised below. It can be seen that for both the stages considered the ratio is generally less than unity and increases with depth below ground level. The increase in the ratio with depth may be a reflection of the existence of perched water tables in the solifluxion layer and in the cryoturbated Gault, and of the increase in the length of the drainage path with depth. The increase in the ratio of observed to predicted pore water pressure during construction is consistent with reduction in apparent construction.

<table>
<thead>
<tr>
<th>Zone</th>
<th>Depth (m)</th>
<th>Initial Stage</th>
<th>End of Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solifluxion</td>
<td>0 - 1.85</td>
<td>0.18</td>
<td>0.43</td>
</tr>
<tr>
<td>Deposits</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cryoturbated Gault</td>
<td>1.85 - 4.84</td>
<td>0.28</td>
<td>0.49</td>
</tr>
<tr>
<td>Weathered Gault</td>
<td>4.84 - 7.30</td>
<td>0.35</td>
<td>0.66</td>
</tr>
<tr>
<td>Unweathered Gault</td>
<td>7.30 - 9.70</td>
<td>0.69</td>
<td>0.88</td>
</tr>
</tbody>
</table>

*It is interesting to note that for pz. B6 near to the toe of the embankment, and for which $\delta = 1.73$, the value of $A$ corresponding to the observed end of construction excess pore water pressure is equal to 9.0. The ratio of observed to predicted ($A = 0.4$) pore water pressure at the end of construction is 2.0.
dissipation noted earlier, and considered to be possibly related to the reduction in permeability with increasing stress level.

The end of construction pore water pressure predicted for the unweathered Gault gives the best approximation to the observed value. For the range of depths likely to be encountered in stability problems below motorway embankments, however, it appears that the pore water pressure during construction would only be about 50% of that predicted using the pore pressure parameters A and B.

The relevance of these parameters is however, restricted by the extent to which field conditions are reflected in the laboratory test from which the values of A are determined, and the correctness of the assumption that 'B' equals unity. The latter assumption would seem reasonable as all the piezometers in the Gault were below their respective water tables at the start of construction and the soil around them could therefore, be regarded as being saturated at the start of construction. The evidence that the parameter B in a saturated soil approximates to unity is, however, based on laboratory triaxial tests.

The conditions existing in most practical problems approximate more closely to plane strain than to axial symmetry used in the triaxial tests. This difference would undoubtedly lead to corresponding differences between the laboratory values of the pore
pressure parameters and those actually operative in the field. There is little evidence of the possible magnitude of this difference, which is only one of the many factors influencing the relationship between field and laboratory values of undrained pore water pressure.

The influence of possible rotation in direction of principal stresses in the field is also not taken into account by the laboratory values of pore pressure parameters. The limitation is usually ignored but will undoubtedly affect the accuracy of predicted pore water pressures. It should also be noted that the pore pressures set up in a soil sample are determined by the change in stress imposed under undrained conditions in the test. The initial state of stress before this change must be correctly reflected in the test, if the relevant pore water pressures (and strength values) are to be measured. The results of triaxial tests in which consolidation is under an equal all-round pressure cannot therefore be expected to apply directly to conditions in the field without an allowance being made for the differences in stress ratio.

3.4 Comparisons of In situ and Laboratory Determinations of Permeability and Coefficient of Consolidation

a) Introduction.

It is commonly observed that overconsolidated clays generally give a lower magnitude of settlement and a faster rate of settlement than that predicted from
The study of the consolidation characteristics of these soils is often complicated by the lack of homogeneity especially with regard to fissuring, three-dimensional drainage, and anisotropic stress conditions insitu.

Schiffman and Gibson (1964) were able to show from a consideration based on one-dimensional theory that, for the London Clay, the variations in permeability and coefficient of volume compressibility alone can account for the faster rate of consolidation commonly observed in the field. The volumetric compressibility of the London Clay under a fairly wide range of stresses has been shown by Simons and Som (1969) to be mainly dependent on the effective vertical stress; the vertical strain, however, was shown to be influenced by the ratio of vertical to lateral stress increment during consolidation. As a result they noted that the use of oedometer test results cannot be expected to give accurate predictions of settlement even if due consideration is given to the differences between the excess pore water pressures in the field and in the oedometer test.

The limitations and uncertainties of the oedometer test are further discussed by Tan (1967), and Som (1968). The more serious of these are that there is no control over drainage which starts immediately as the load is applied, and consequently, before peak pore water pressures are attained in the soil. The load
application subjects the sample to shock, and there is no means of measuring pore water pressure or drainage from the sample.

In this section the field measurements of permeability and coefficient of consolidation below the embankment shoulder, \( p_z \), Group 4, are compared with the results of laboratory oedometer tests. Rowe (1968) has pointed out that laboratory tests on small samples give misleading values of coefficient of consolidation and permeability except for uniform clays. These conclusions have been found to apply to the Gault at the trial embankment site.

Measurements of permeability were made at the trial site before, and during construction of the embankment using a constant head testing technique. If a constant out-of-balance head is applied to a piezometer, consolidation or swelling occurs around it. Gibson (1963) has presented an analytical solution to the problem for the case of spherical tip which Al-Dhahir (1967) and Wilkinson (1968) have shown to be applicable to cylindrical tips, which are normally used in practice, provided the ratio of length to diameter does not exceed 4.0. This condition is satisfied by the piezometers at the trial site.

The solution presented by Gibson (1963) shows that if sufficient time is allowed to elapse for steady state conditions to be approached in a piezometer under a pressure imbalance, the permeability, \( K \), of the soil
can be evaluated from the expression given below, which is the same as that obtained by Hvorslev (1951).

\[ K = \frac{q}{P \Delta h} \]  \hspace{1cm} (5)

where \( q \) denotes the rate of flow.
\( \Delta h \) denotes the applied change in pressure head.
\( P \) denotes the intake factor, which is \( 4\pi r \)
for a spherical tip of radius \( r \) in an isotropic permeable medium.

It can be seen from Eq. (5) that the time required for steady state conditions to be attained increases with the intake factor \( P \), which is in complete contrast to the response time in piezometers when used to measure pore water pressure. Hvorslev (1951) has tabulated formulae for the calculation of intake factor for several shapes of tip under various soil conditions. For the case of a cylindrical tip in a uniform semi-infinite soil, \( P \) is calculated from the expression given by Dachler (1936) viz :-

\[ K = \frac{2\pi L}{2.3 \log_{10} \left[ N + \sqrt{1 + N^2} \right]} \]  \hspace{1cm} (6)

where \( L \) = length of tip.
\( N \) = length of tip/diameter of tip.

Al-Dhahir (1967) has shown that the expression by Dachler is only valid when \( N \) is approximately unity, and suggests the following expression for intake factor \( P \) :-

\[ F = \left( 4\pi \times \text{surface area of cylindrical tip} \right)^\frac{1}{2} \]  \hspace{1cm} (7)
Dachler’s expression generally results in an underestimate of F derived from equation (7) of about 7%.

According to Gibson (1963), if the discharge \( q \) at time \( t \) is plotted against \( \frac{1}{\sqrt{t}} \) a linear relationship of gradient \( n' \) is to be expected, with the intercept at \( t_\infty \) corresponding to \( q_\infty \) and hence \( K \) from Eq. (5). The coefficient of consolidation or swelling of the soil could according to the theory also be determined from the \( q \) vs \( \frac{1}{\sqrt{t}} \) plot using the relationship:

\[
q = q_\infty \frac{2r^2}{\pi n_0^2}
\]  

\[
...............(8)
\]

Several instances have been reported of departure from the behaviour predicted by the theory e.g. Wilkinson (1968), Bishop and Al-Dhahir (1969). These have been ascribed to errors which may arise in the test from such factors as smear, and head loss in the piezometer tips, and anisotropic permeability. Modifications to Gibson’s theory have been proposed to account for these effects, Al-Dhahir (1967), Wilkinson (1968). More recently, Gibson (1969), (1970) has, by taking into account the inelastic behaviour of real soils, shown that the piezometer compressibility, the magnitude of the pore pressure parameter \( A \) of the soil, and the direction of flow all have an influence on the relationship between flow rate and time. Whereas reasonable estimates of

\*Since the discharge \( q \) is normally measured over a finite time interval, the average value of \( q \) should be plotted against \( \sqrt{\frac{t_2 - t_1}{2}} \) where \( t_1 \) and \( t_2 \) are the beginning and end of the time interval used for determining \( K \).
permeability can still be made from short cylinders in soils of isotropic permeability from the theory proposed by Gibson (1963), the coefficient of consolidation or swelling may be grossly over-estimated.

b) **Method of Field Measurement.**

A new apparatus was devised for the measurement of field permeability. It essentially involved measuring the rate of flow of mercury under constant head, along a horizontal tube filled with water, and connected to the piezometer. Details of the apparatus are given in Fig. 3.11. A constant pressure is ensured by the use of a self-compensating pot which supplies mercury under a pre-determined pressure imbalance through a horizontal tube set against a scale, to the piezometer.

To operate the apparatus one section of the piezometer is isolated from its manometer and connected through the bus-bar to the self-compensating manometer. The difference in mercury levels should be equal to that recorded on the manometer unit. Both sections of the piezometer are now cut off from the system through valves A and B and a pressure imbalance \( \Delta H \) (positive or negative) is then induced in the self-compensating manometer using the (foot) pump in the deairing unit. The range of pressure was restricted to \( \pm 13.8 \text{kN/m}^2 \) (2 p.s.i.) and the majority of the tests were carried out under an excess pressure within the range of \( \pm 6.89 \text{kN/m}^2 \) (1 p.s.i.). The compen-

*The need to use low excess pore water pressures if hydraulic fracturing is to be prevented has been discussed by Bjerrum et al. (1972).
sating pot is then lifted or lowered until the mercury in the right hand limb is at the zero mark when water is to be pushed out of the piezometer \( (C_s) \) or at the end of the horizontal tube when water is expected to flow into the piezometer \( (C_p) \). The deairing unit is then closed and the self-compensating manometer is connected to one section of the piezometer under test through the appropriate busbar and valves. Measurements are made of the distance travelled by the mercury along the horizontal tube.

The main advantage of this system is that it does not require a lot of space and it is comparatively simple to operate. The horizontal tube could be set up against the piezometer units and round the sides of the instrument hut if this is required for the attainment of steady state conditions.

c) *Insitu Permeability.*

Constant head tests were performed at the start of construction and after 3.2 m of fill had been placed and typical examples of the test results are given in Figs. 3.12. The majority of the tests display a linear feature; however a strong downward curvature was obtained in some tests. The calculated values of insitu permeability are given in Table 3.4 from which the general tendency for permeability to decrease with depth may be seen. This is consistent with the fact that the fissure spacing of the
Gault increases with depth. Also fissures tend to be open under low-effective stresses, as is commonly observed in freshly exposed faces of trial pits. Free moisture is also, sometimes evident on the fissures in these pits, suggesting that they may act as channels for ground water flow. Minor variations in the soil, such as the 'silt vein' observed in trial pit 3800/1 Fig. 3.2a will undoubtedly also be reflected in the insitu permeability. Such variations may account for the range of variation of permeability measured within the various zones of soil at the site.

The average values of permeability within the various zones of soil at the trial site are summarised below. It is clear that the insitu permeabilities of the Gault are considerably greater than

<table>
<thead>
<tr>
<th>Zone</th>
<th>Average Insitu Permeability K under height of fill H(m)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H = 0</td>
<td>H = 3.2</td>
</tr>
<tr>
<td>Solifluxion Deposits</td>
<td>0.35 m/y</td>
<td>0.18 m/y</td>
</tr>
<tr>
<td>Cryoturbated Gault</td>
<td>0.27</td>
<td>0.09</td>
</tr>
<tr>
<td>Weathered Gault</td>
<td>0.34</td>
<td>0.098</td>
</tr>
<tr>
<td>Unweathered Gault</td>
<td>0.15</td>
<td>0.054</td>
</tr>
</tbody>
</table>

is commonly expected from this type of clay, see for example Bishop and Bjerrum (1960), Table 4. The
measured $K$ values of the Gault may be compared with
the value of $0.012 \text{ m/y (} 4 \times 10^{-3}\text{ cm/sec)}$ used in the
calculation of pore water pressure equalization time
in the piezometers, and which is typical of the values
commonly obtained from the analysis of the response
time in open-standpipe piezometers.

The influence of stress level on the
permeability within each zone of the Gault is also
evident from the summary, from which it can be seen
that $K$ decreases with increasing load. Although the
effect of stress level may also be reflected in the
variation of permeability with depth under any given
stress condition, the sharp drop in $K$ within the
unweathered Gault may in addition to the influence
of fissure spacing, also reflect the relative absence
of permeable fabric in the layer. Rowe (1972) found
that whilst isolated samples of the blue London Clay
at Croydon were without any significant permeable
fabric, there was a profusion of interpedal silt
and sand in a sample of the weathered brown finely
fissured London Clay from a depth of 2 m below ground
level. Samples of the brown London Clay used in a
centrifugal model study of short-term failure in a
cutting, showed a failure time corresponding to
12 - 24 weeks in the field. The undrained strength
at failure was 60% of that obtained from 38 mm samples,
and as the fissure lumps were between 5 and 10 mm in
size, the effect of sample size on the measured undrained strength would be small. In addition, failure was accompanied by an increase in water content on the slip surface. Rowe (1972) has noted that the model behaviour is consistent with the incidence of a permeable fabric.

The interesting deduction that can be made is that in those zones of the Gault exhibiting a high mass permeability, some water content change may occur during typical construction periods especially under low levels of stress. The pore water pressure variation during the construction of the trial embankment shows some consistency with this expectation.

Very little data has been published on the insitu permeability of stiff fissured clays. A comparison of the coefficient of consolidation determined from the insitu permeabilities and by other methods, with published data for similar soils is presented in the subsequent section. It will suffice at this stage to mention that Rowe (1968), (1972) observed that the field performance of the carboniferous shale and mudstone foundation of the Staunton Harold dam and the London Clay foundation of the Ardleigh dam was consistent with the result of insitu constant head permeability tests. The unweathered London Clay at Ardleigh was found to display a permeable fabric and the insitu K decreased from about 0.13 m/y at 40 kN/m² to about 0.032 m/y at 200 kN/m². A 'short-term' failure at
the base of the cofferdam at the Hythe End reservoir also in the London Clay has been partly ascribed by Rowe (1972) to the high insitu permeability allowing seepage of water and rapid softening of the clay.

In contrast to these sites the blue London Clay at Wraysbury has been demonstrated by Skempton et al (1969) to be free of permeable fabric. Constant head insitu permeability tests by Garga (1970) on samples from this site resulted in $K$ values varying from 0.013 m/y at about 100 kN/m$^2$ effective overburden pressure to 0.00077 m/y under about 200 kN/m$^2$ effective overburden pressure. These values of $K$ are between 10 and 50 times those at the corresponding stress level obtained by Rowe (1972) for the Ardleigh London Clay. They are still, however, higher than the laboratory values commonly obtained from tests on small samples.

d) Coefficient of Consolidation $C_v$.

In this section the values of $C_v$ calculated from insitu permeability $K$, and 3 in. oedometer compressibility $\bar{M}_v$, ($C_v = K / \gamma_w \bar{M}_v$), are compared with $C_v$ determined from the observed dissipation during a 7½ day 'rest period' shortly after the start of construction, and the $C_v$ determined from the apparent dissipation at the end of construction.

1) Calculation of $C_v$ from insitu $K$ and Laboratory $M_v$.

The laboratory value of $M_v$ is largely insensitive to sample size, the presence of permeable fabric and test
conditions, Simons and Som (1969), Garga (1970), Rowe (1972). This method of calculating $C_v$ is therefore unlikely to involve significant errors. The calculation of $C_v$ for the various zones of soil at the trial site under insitu overburden pressure, and after 3.2 m of fill had been placed, are shown in Table 3.5a. The calculated $C_v$ values are 200 - 1500 times higher than the 3 m oedometer values under the low insitu overburden pressure. The range decreases to 200 - 500 times after the addition of 3.2 m of fill material.

ii) Calculation of $C_v$ From Observed Rate of Pore Water Pressure Dissipation.

The values of $C_v$ relate to the observed dissipation over a 7½ days break in construction due to the wet weather, which occurred after 0.83 m of fill material had been placed, Figs. 3,6. The rainfall does not appear to have had any significant effect on the pore water pressure dissipation as could be seen from Pz B 31.593.69 located in the solifluxion layer. Pzs. B8s and B7s below the shoulder were also located in the solifluxion mantle which has been found under natural conditions, to show an almost immediate response to rainfall. Pz B8s showed a marked response during loading, but the pore water pressure began to dissipate soon after construction stopped. At the end of 7½ days, the pore water pressure in the piezometer was slightly lower than the initial level at the start of
construction. Pz E7s only showed an insignificant response to the placing of 0.83 m of fill material and throughout the 7½ days the pore water pressure was virtually the same as the initial value. The variation in response reflects the variability of the solifluxion mantle, however, in both cases the indication is that the rainfall had no significant effect on the pore water pressure variation during the 7½ days of cessation of construction.

The percentage dissipation $V_a\%$ is shown plotted with depth of piezometer in Fig. 3.13a and the calculation for average $C_v$ in each zone of soil at the trial site is given in Table 3.5b. Values of time factor $T$ have been obtained from Taylor (1948). The applied pressure after the addition of 0.83 m of fill does not differ significantly from the insitu overburden pressure, and the ratio of observed $C_v$ to oedometer $C_v$ similarly varies between 200 and 1500.

iii) Calculation of $C_v$ From Apparent End of Construction Dissipation.

It has been noted previously that a certain amount of dissipation occurred in all the piezometers during construction, Figs. 3.6. The apparent construction dissipation has been calculated from the difference between the sum of the 'undrained' increments of pore water pressure and the observed pore water pressure at the end of construction.
The apparent end of construction dissipation \( U_p \) is expressed as:–

\[
U_p = \left( \frac{\Delta \bar{U} - \Delta U}{\Delta \bar{U}} \right) \times 100
\]

where \( \Delta \bar{U} \) is the summated 'undrained' pore water pressure at the end of construction.

\( \Delta U \) is the observed pore water pressure at the end of construction.

The values of \( U \) so calculated, are shown plotted against depth in Fig. 3.13b, and the calculation for \( C_v \) is given in Table 3.5c. The calculated values of \( C_v \) would not reflect any drainage induced during the actual load application, and may as a result underestimate the true end of construction \( C_v \) values.

The high in situ permeability in the solifluxion layer is not reflected in the variation of apparent dissipation with depth Fig. 3.13b. This may be due to the fact that the top 1 m layer consisting of a granular Head material, probably allowed a very rapid dissipation of pore water pressure in the shallow layers of soil at the site. The comparatively lower apparent construction dissipation in the shallow layers may therefore be reflecting the greater proportion of drainage in these layers during the actual load application. The calculated average \( C_v \) value for the solifluxion layer in particular, is likely to underestimate the true field value.

iv) **Comparison of Calculated \( C_v \) Values.**

A summary of calculated \( C_v \) values is shown
in Fig. 3.13c from which it can be seen that the $C_v$ values for all zones of the Gault are several hundred times higher than the typical laboratory value of about $1 \text{ m}^2/\text{yr}$ under low effective stresses. With increasing load however, there is a sharp decrease in the $C_v$ values which then tend to converge to values between 10 and 100 times the typical laboratory values. The variation of $C_v$ for the London Clay at Ardleigh, Rowe (1972), and Wraysbury, Garga (1970), based on insitu constant head permeability tests and laboratory compressibility, and tests on large diameter samples are also shown in the figure. Mirata (1965), from a consideration of the time-settlement curve of the Chingford and Hanningfield reservoir on the London Clay estimated the field $C_v$ values to be 5 $\text{m}^2/\text{y}$ and 6.5 $\text{m}^2/\text{y}$ respectively. These values are between 10 and 20 times the oedometer values and are roughly consistent with the convergence of $C_v$ under high stresses noted by Garga (1970) and Rowe (1972), Fig. 3.13c.

The results obtained by Weeks (1970) from the field dissipation of pore water pressure in the Weald Clay foundation of embankments on the Sevenoaks By-Pass are also shown in Fig. 3.13c. The $C_v$ value at 70 kN/m$^2$ was determined from pore water pressure measurements in the foundation of a 3 m high trial embankment and that at 300 kN/m$^2$ was calculated for
the foundation of a 16 m high embankment. The $C_v$ value from the trial was 12.4 times the corresponding laboratory value. In the design of the 16 m high embankment, this ratio was applied to the appropriate oedometer $C_v$, and resulted in a value of 30 m$^2$/y which was adopted in the design. It can be seen from Fig. 3.13c that the actual $C_v$ calculated from the dissipation of pore water pressure below the embankment, was only 15 m$^2$/y; fortunately, however, the $B$ value during construction was generally less than 0.5, whereas a value of 1.0 had been assumed in the design.

The influence of permeable fabric on the values of $C_v$ determined from insitu measurement and tests on large sample is clearly demonstrated in Fig. 3.13c. The Gault at the trial embankment site, the London Clay at Ardleigh and Weald Clay at Sevenoaks all of which contain a permeable fabric display very high $C_v$ values under low levels of stress. At the trial embankment site, the permeable fabric was observed from trial excavations and borings to consist of silt veins, bands of shattered silt-stone phosphatic nodules and pyrites nodules, selenite crystals and occasionally, thin seams of silt-size and sand-size particles between fissure faces. The $C_v$ values for the London Clay at Wraysbury where the absence of a permeable fabric has been noted by Skempton et al (1969), is in complete contrast to those at Ardleigh. The values of $C_v$ for the London Clay at Wraysbury though determined by indirect
methods, Garga (1970), are consistent with the field performance of embankments at the site, (Priv. Comm. Mr. Pawsey, Metropolitan Water Board) which indicates $C_v$ values of between 40 m$^2$/yr under low stresses, and 5 m$^2$/yr under the completed embankment.

The ultimate values of $C_v$ generally obtained under high levels of stress both in areas with and without a permeable fabric show a tendency to converge towards a value much closer to the laboratory value, indicating that the influence of permeable fabric predominates at the lower levels of stress. This is consistent with the fact that under increasing load, open fissures and permeable seams are expected to be closed or constricted. The implication of this is that on the Gault, low embankments probably up to between 6 and 7 m in height can be constructed safely, without special drainage measures where the foundation material displays a permeable fabric. For higher embankments at such sites, it would still be possible for special drainage measures to be omitted, if a controlled rate of placing is specified after construction of the first 6 m or so of embankment. This procedure has been successfully adopted during the construction of embankments on the Weald Clay along the Tonbridge By-Pass.
Comparisons have been made between the field values of $C_v$ and those determined from laboratory oedometer tests. This comparison is, however, not strictly valid as the field values relate to three dimensional consolidation whilst the oedometer tests give the one-dimensional $C_v$ values. Som (1968), from a consideration of a 14 m (45 ft.) thick layer of soil with a $C_v$ of 1.2 m$^2$/y (11 ft.$^2$/year), showed that under one dimensional consolidation, 80% consolidation would occur in 50 years. The corresponding figure for three dimensional consolidation is only 15 years. The difference between field $C_v$ and the laboratory values even at the higher levels of stress may therefore be a reflection of the divergence between the laboratory test conditions and field conditions. A better estimate of 'ultimate' field $C_v$ even in soils which display a permeable fabric, would only result from tests in which the field conditions are simulated. Triaxial dissipation tests on small samples may therefore give a better reflection of the field $C_v$ values under high levels of stress, compared to those that would be obtained from oedometer tests. For soils exhibiting anisotropic permeability, both horizontal and vertical samples should be tested.

3.5 Stability of the Trial Embankment

a) Mode of Failure

Failure occurred along the whole length of the embankment, 34 days after the start of construction.
The maximum height at the centre line was 10.65 m and allowing for about 10 days soon after the start of construction when no fill was placed because of wet weather, this corresponds to a rate of construction of 0.45 m/day (1.5 ft./day). The events leading up to failure are listed below.

5th October, 1970: A bulge was clearly visible in the lower section of the embankment slope, which became more marked in the succeeding days.

9th October, 1970: Cracks observed along the lower section of the embankment slope. The width of the uppermost crack was monitored and the following data was obtained:

<table>
<thead>
<tr>
<th>Time</th>
<th>Observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.00 a.m.</td>
<td>thin hair line crack</td>
</tr>
<tr>
<td>2.00 p.m.</td>
<td>width of crack 25 mm (1 in.)</td>
</tr>
<tr>
<td>4.00 p.m.</td>
<td>&quot; &quot; &quot; 50 mm (2 in.)</td>
</tr>
<tr>
<td>6.00 p.m.</td>
<td>&quot; &quot; &quot; 60 mm (2½ in.)</td>
</tr>
</tbody>
</table>

During the period of observation the area affected by ground heave extended for a maximum distance of 9 m (30 ft.) beyond the toe of the embankment. A maximum heave of 0.3 m (1 ft.) was noted.

10th October, 1970: Ground heave attained a maximum level of 1.3 m (4 ft.) above original ground level; there was no further southward extension of the affected area.

A network of tension cracks was in existence running along the embankment slope, but which only extended to about half-way up the slope. There were also vertical cracks running across the slope associated with the differential displacement of the slope. Towards the evening, the lower section of the slope consisted of a
number of displaced blocks bounded by tension cracks in the fill. Despite these movements, there was no sign of failure in the top section of the embankment slope.

10th October, 1970:
10.00 p.m. Failure occurred along the whole embankment, Plates 3.1.

The lower section of the slope having been displaced as a series of wedges, the top section of the slope then slumped behind the displaced wedges leaving a polished and striated rear scarp.

b) 'Conventional' Stability Investigation.

In this section, the embankment failure is examined using conventional methods. For failure occurring during or at the end of construction, the total stress method of analysis is regarded as appropriate, and the failure surface is assumed to be a circular arc. The shear parameters used in the analysis are the undrained strengths which are either determined from laboratory tests on undisturbed samples or field vane tests where conditions are suitable. For a saturated soil, the undrained strength is equal to one half the undrained compression strength, and as this value may vary with depth, results are required for samples from a wide range of depths, so that the appropriate values may be used along trial failure surfaces.

The broad applicability of total stress method of analysis to stiff clays has been established both from
(a) Tension crack in fill at initiation of failure

(b) Tension crack in fill at failure

(c) Slumping of upper slope behind laterally displaced lower embankment slope

(d) General view of embankment at failure
theoretical considerations and from field data. Skempton and Sowa (1963) found that the undrained strength of an undisturbed isotropically consolidated specimen only depends on the water content. The collation of data on loading failures involving stiff fissured clays by Bishop and Bjerrum (1960) indicates that using undrained strength of 38 mm diameter samples, the total stress method of analysis generally results in factors of safety of between 1.0 and 2.0. Similar observations have also been made by Parry (1971) for embankments on normally consolidated, and lightly overconsolidated clays.

The undrained strength of stiff fissured clay is usually determined from triaxial tests, plate loading tests or direct shear tests in the field. Most of the available data comparing the results from these tests have been obtained for the London Clay.

It has been recognised that even where undrained conditions may be entirely satisfied in the field, the radical difference between the laboratory conditions of the conventional undrained test, and those insitu may result in large errors in stability analysis. The discrepancies between field and laboratory values of undrained strength are known to arise from a number of factors, the effects of some of which cannot at present, be discerned individually.
Factors Affecting the Measurement of Undrained Shear Strength.

It is known that factors affecting the measurement of undrained strength include:

i) Disturbance of soil during insitu testing or during 'undisturbed' sampling, and sample preparation in the laboratory test, such as obtaining smaller 'undisturbed' specimens from 'undisturbed' U.4 samples.

ii) Structural anisotropy arising from one-dimensional consolidation which may result in particle orientation and high residual strain energy; other sedimentary and post-depositional features such as fissures, joints and bedding may introduce a variation in undrained strength with orientation of sample if they show a preferred orientation.

iii) The size of sample in the laboratory test, which must be large enough to be fully representative of the insitu structure of the clay.

iv) The difference between the stress system in the laboratory test and those insitu, both before and during shear.

v) The influence of strain rate during the measurement of undrained strength. The time to failure in the conventional test of between 5 and 15 mins is hardly, if ever, achieved in the field.

vi) Stress redistribution in the embankment during construction, which may result in high lateral stresses in the embankment and initiate failure.
vii) Pore water pressure migration, which may cause swelling and local overstress in the foundation. This phenomenon is however likely to be critical in soft clays and highly structured deposits.

viii) Progressive failure of the slope, which would result in a variation of the strength mobilized along a potential failure surface, depending on the variation in strain along the surface.

ix) The correctness of assuming that the failure surface is a circular arc, and the likelihood of the total stress method of analyses resulting in the most critical failure surface.

x) The time interval from the opening up of a borehole or test pit, to the start of the in situ test. This is more critical in soils with a high mass permeability in which some drainage may occur before the test is performed.

Some of these factors are enumerated in the ensuing section.

**Sample Disturbance.**

The effects of sample disturbance on the London Clay has been investigated by Ward et al (1959) and Ward et al (1965). They found that on average, strengths from block samples were about 30% greater than conventional borehole samples. Also, strengths from samples obtained by rotary coring were close to the block sample strengths. All the in situ total stresses are removed during sampling and a negative pore water
pressure is set up in the samples, Skempton and Sowa (1963). According to the concept of stress path testing, Lambe (1967), Simons and Som (1969), the insitu conditions must be restored, before shearing in the laboratory. However, reconsolidation of the sample may lead to lower void ratios and water contents than in the field, and to higher estimates of undrained strength, Skempton and Sowa (1963). Whereas the stress path procedure is undoubtedly more relevant to the field condition, its adoption in this case would probably result in a greater over-estimate of stability. This point is elaborated later, in the section dealing with insitu and laboratory stress conditions.

**Structural Anisotropy.**

Bishop (1966) considering the undrained strength of the London Clay from the Ashford Common shaft, showed that strength is influenced by the inclination of the failure plane. The strength obtained from samples in which the major principal stress $\sigma_1$ was applied horizontally, was found to be 1.46 times the strength in the conventional test, with $\sigma_1$ vertical. For the shallow samples of London Clay from Wraysbury, Argaiwal (1967), Simons (1967), it was found that the horizontal undrained strength was only slightly greater than the vertical undrained strength. Samples from both sites, tested with the failure plane arranged to coincide with the bedding plane, were, however, found to give strengths of about 75% of those
determined from vertical* samples. The ratio was found to be 86% for the shallow depths of the London Clay at Maldon, Bishop and Little (1967). Tests on other materials, Duncan and Seed (1966), Lo (1965) similarly also show that undrained strength may vary with orientation.

The strong particle orientation parallel to bedding in intact samples of London Clay, Gault and other deposits, has been discussed by Tchalenko (1968). This development was attributed to consolidation of the sediments under overburden pressure, and was found to persist except where the clay has been very severely weathered. The lower undrained strengths along the bedding plane may be reflecting the preferred particle orientation in the London Clay.

It follows from the foregoing, that the shear strength along a potential failure surface in the field may vary purely as a result of differences in its orientation. According to Lo (1965), a drop in undrained strength of 50% in the horizontal direction, leads in a circular arc analysis of a typical slope, to a reduction in factor of safety of only 15 - 30%. This observation coupled with the fact that some reduction in strength always accompanies sampling, led Bishop (1966) to suggest that the total stress method of analysis has probably appeared more accurate than theoretically it should be.

*The term 'vertical' means that the axis of the specimen was parallel to the direction of major principal stress in the test.
Sample Size.

Studies of the effect of sample size on the undrained strength of London Clay have been made by Bishop (1966), Bishop and Little (1967), Marsland (1967), and Simons (1967). Bishop and Little (1967) noted that for the shallow depths of the brown London Clay investigated at Maldon, the average insitu undrained strength in a 0.6 x 0.6 m (2 ft x 2 ft) shear box test was only 55% of that obtained from vertical 38 mm diameter specimen, obtained from U4 samples. There was, however, no discernible differences between the undrained strength of 38 mm and 100 mm diameter samples. The low insitu result is partly due to orientation effects on the failure plane, however, laboratory tests on 38 mm diameter samples resulted in undrained strength along the bedding equal to 86% of that determined for vertical samples. The insitu test was sheared at a rate 12 times slower than in the laboratory test, but as will be shown later, this variation in testing rate is unlikely to be responsible for any significant part of the difference in the undrained strengths. A major part of the reduction from 86 - 55% may therefore be ascribed to size effects. It should be noted, however, that the degree to which anisotropy is reflected may be more marked in the large insitu shear box sample than in the small laboratory sample, due to the insitu sample being more representative.

The undrained strength determined from large bases, 0.3m (12 ins) to 1 m (36 ins), were about 10% higher than the insitu shear box values. This departure
may well be due to the difference in orientation of the failure planes in these tests.

From the analysis of failure conditions in bored piles in the London Clay, Whitaker and Cooke (1966) noted that the undrained strength was only about 75% of that indicated by 38 mm x 76 mm (1½ in x 3 in) specimens obtained from U4 samples. This conclusion is supported by the results of Burland et al (1966) who found the range to be from 67% to 115% and by Hooper and Butler (1966) who also found that the insitu strength of London Clay determined from plate loading tests was 72% to 110% of that indicated by 38 mm x 76 mm specimens obtained from U4 samples.

Simons (1967), from a consideration of insitu shear box tests and triaxial undrained tests on the London Clay at Wraysbury, concluded that 100 x 200 mm (4 in x 8 in) samples were large enough to adequately reflect the insitu fissure structure of the clay. The strength obtained from the analysis of a 'short-term' slip was about 76% of that measured on vertical 100 mm x 200 mm samples. The discrepancy was ascribed to differences in the orientation of the failure planes in the field which was mainly horizontal and in the laboratory test on vertical samples and the differing times to failure. Bishop and Little (1967) also found that a longer rate of loading led to lower estimates of undrained strength.
Triaxial undrained tests on 38 mm, 76 mm and 125 mm (1⁄4 in, 3 in and 5 in) diameter specimens, cut from block samples of the London Clay from Wraysbury, Marsland (1967), resulted in the following strength ratios when compared to the strength of the smallest sample, 100:77:66 at 8 m (26 ft) below the surface of the clay where the fissure lumps varied from 50 - 150 mm (2 - 6 in) and 100:77:59 at 31 m (103 ft) below the surface of the clay, where the fissure lumps were 75 - 410 mm (3 - 16 in) in size. Large diameter plate loading tests also resulted in undrained strengths only about 75% of those determined from vertical 38 mm diameter samples.

Marsland and Butler (1967) also found that for the stiff fissured Barton Clay, 38 mm diameter samples gave a higher value of undrained strength than 75 mm and 125 mm diameter samples. The effect of sample size however became smaller at shallow depths below the ground surface.

The Insitu and Laboratory Stress Systems.

The effect of sampling on the insitu stress system has already been mentioned and the need for a testing procedure which first brings the sample to a state of stress similar to those insitu before shearing has also been noted. The problem of determining the insitu stress system can, however, be a very difficult one. There is evidence from laboratory work that the effective horizontal stress in overconsolidated clays
may exceed 2 times the value of the effective vertical stress, Skempton (1961), Bishop et al (1965).

Recently the results of Bishop et al (1965) have been applied in the prediction of long term deformation of deep basements in the London Clay by Ward (1971), Cole and Burland (1972), and was found to give good agreement with pile loading tests and full scale observations. The method of calculating values of insitu $K_0$ which was first proposed by Skempton (1961), therefore appears to be reliable.

Skempton and Hutchinson (1969) have suggested that the horizontal undrained strength of the London Clay probably reflects the high lateral insitu consolidation pressure in the clay. The $K_0$ values for the Ashford Common shaft, Bishop et al (1965), are greater, for a given depth, than those at Bradwell, Skempton (1961). The difference is, however, consistent with the higher estimated preconsolidation load at the site of the Ashford Common shaft. The preconsolidation loads at Ashford Common and at Wraysbury are likely to have been similar; the absence of any significant difference between the ratio of horizontal undrained strength to vertical undrained strength at Wraysbury, Aggarwal (1967), probably reflects the fact that in the shallow layers, unloading and release of lateral support by excavation has resulted in dissipation of lateral insitu stresses. Such a behaviour would be consistent with the assessment by Bjerrum (1966), that the London Clay has weak bonds which can be easily destroyed.
In the conventional laboratory test, samples are not first of all subjected to the stress system that was operative in the field, and the applied stress path during shear may not follow that in the field. Even if sampling does not involve any disturbance or volume change on removal of the insitu total stresses, a small residual shear strain will occur due to the removal of the stress difference \((1 - K_o)\sigma'_v\) which would exist in the natural strata when \(K_o\) is not equal to unity. As the stress-strain behaviour of soils is not truly elastic, and is as a result, not fully reversible, the insitu stress state and void ratio will not be completely attained even if samples were subjected to the insitu stress state before being sheared. The error resulting from this will be indistinguishable from that due to structural disturbance, which will also affect the reversibility of strains; the overall effect may be that the insitu undrained strength is overestimated. Laboratory tests in which attempts are made to simulate sampling and other stress-states insitu, do not, however, result in significantly different parameters from those obtained by conventional methods, Skempton and Sowa (1963), Simons and Som (1969).

The rotation of principal stresses which may occur in the field are also neglected in the laboratory test. In the field, the directions of major and minor principal stresses are rotated during shear, except along

\[\sigma'_v\] is the insitu effective vertical stress.
the centre line under a symmetrical embankment, Parry (1971). Despite these disadvantages, the triaxial compression and extension tests probably approximate to the limits of undrained strength that is likely to be encountered in the field.

Ladd (1967), considers the triaxial test on saturated soils, to result in an under-estimate of the insitu strength due to sample disturbance, but that the insitu undrained strength for most modes of failure in the field is likely to be less than the conventional triaxial undrained strength because of changes in the intermediate principal stress, and differences in the orientation of the principal stress direction. Thus, it is suggested that the broad applicability of the total stress method of analysis, may be due in part, to the fact that sample disturbance, and the differences between the stress systems in the field and in the triaxial test, may be compensating errors.

For the Gault, the strong diagenetic bonds developed during its consolidation, Bjerrum (1966), may have subsequently been destroyed in the shallow layers, by the strains which accompanied cyclic freezing and thawing, drying and wetting, and oxidation and reduction of the mineral constituents. Only small differences may now exist between horizontal undrained strength and vertical undrained strength in these layers, which are likely to be mainly involved in foundation failures below
motorway embankments. In any case the section of a circular arc failure surface, in a typical slope, along which the horizontal strength would be appropriate, would generally be small. This factor, together with the possible compensating errors in the triaxial test, tends to reduce the apparent error involved in the use of the laboratory undrained strength in the analysis of end of construction failures.

**Time Dependent Behaviour.**

The influence of strain rate on undrained strength, progressive failure, and the time interval involved in insitu testing are discussed.

Apart from the need to determine undrained strength from representative samples, the problem of deciding on a value of insitu strength is complicated by its variation with time. The conventional undrained test is visually completed within 15 mins, whereas the rate of loading in most constructional operations is measured in terms of weeks or months, a loading rate many times smaller than in the routine test. In contrast, the shear stresses in the subgrade of roads and runways is applied at a much faster rate than that used in the laboratory test.

In the investigation of cutting failures in the London Clay at Bradwell, Skempton and La Rochelle (1965) observed that the undrained strength of vertical
38 mm diameter specimens cut from the same block sample, decreased with increasing time to failure. The results suggest that a failure occurring in one day would develop an undrained strength 10% less than the conventional 15 min test value, and for a time to failure of 1 month the reduction in strength would be about 20%. This decrease in strength was associated with an increase in the moisture content in the shear zone of samples, although there was no overall change in the moisture content of the samples. The Cambridge Clay studied by Casagrande and Wilson (1951) showed a drop in conventional undrained strength of about 18% for a failure time of 1 month. In contrast the Fornebu Clay, Bjerrum et al (1958) showed a considerable drop of about 50% when the failure time was increased from 15 mins to 1 month. This decrease was accompanied by an increase in pore water pressure at failure which, however, was not sufficient to account for the whole decrease in undrained strength with time.

The undrained shear strength mobilized along failure surfaces at Bradwell was only about 55% of the conventional value. It was suggested that this discrepancy was due to the differing times to failure in the laboratory and in the field, size effects arising from testing small samples that are unrepresentative of the insitu fissure structure of the clay, with progressive failure playing a very minor role.
The experiments of Skempton and La Rochelle also show that the drop in undrained strength in the 'long term' agrees roughly with the post-peak drop-off in strength noted in the conventional test. A similar average drop of about 20% was recorded in the large shear box tests at Maldon reported by Bishop and Little (1967), over an average displacement of 30 mm (1.18 ins). Recently, Bishop (1971) has reported undrained ring shear tests with a time to failure of 1½ mins, on remoulded blue London Clay from Wraysbury, showing 43% post-peak drop-off in shear strength after large strains.

The undrained strength along a failure surface in the field may therefore vary with the strains along it. The Brittleness Index Ip displayed in laboratory test may decrease with increasing time to failure, and with increasing stress in the laboratory test. Bishop (1971) considers that even under low stress levels, negative pore water pressures are unlikely to be maintained across a rupture surface, and that the undrained strength would adjust itself to the post-peak 'effective' angle of shearing resistance \( \phi' \) multiplied by the applied normal stress \( \sigma' \). A rapid post-peak drop-off in shear strength is therefore likely during short-term failures in cuttings, where

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*The post-peak drop-off in undrained strength has been designated, Brittle Index Ip by Bishop (1971).*
the reduced stresses may cause negative pore pressures and hence high peak undrained strength. The formation of a failure surface would then cause a sharp drop in a value consistent with $\sigma_{\tan \theta'}$. Under high normal stresses such as in confined penetration tests, Bishop (1971) considers that a fissured clay may behave as a work-hardening plastic material. This assessment is supported by the results of Marsland (1971) on the London Clay which show an increase in undrained strength of about 50% in confined plate loading tests over those obtained in unconfined tests. Progressive failure thus seems unlikely to be of significance in end-of-construction failures below high embankments.

Insitu tests are sensitive to softening which may occur due to water seepage into shafts and test pits. The risk of softening is however related to the rate at which water is fed to the fissures, the differential pressure head created by the excavation and the magnitude and direction of any changes in effective stress that occur, Rowe (1972). As most insitu shear box tests are carried out during the dry summer months, and at shallow depths, then, provided reloading takes place soon after excavation, very little error would arise from drainage. In deep shafts, however, such as at Ashford Common and in bored piles, conditions are more favourable for some drainage to occur in a short period of time.
Insitu loading tests in the London Clay in which the start of testing is delayed after the completion of the excavation, may result in lower ultimate bearing pressures due to opening of fissures and moisture migration, Ward et al (1965), Palmer and Holland (1966). Insitu tests were not carried out at the trial site, and so this problem does not arise.

Stress Redistribution in Embankment.

The differential settlements which can occur during the construction of an embankment can give rise to high lateral stresses which would in turn reduce stability. These stresses are usually ignored in embankment design although an indirect allowance is sometimes made by ignoring the shear strength of the fill in stability analysis. This procedure will be reasonable in cases where a tension crack develops in the fill.

The calculation of foundation stresses by a method based on the theory of elasticity, has shown that high lateral stresses were induced during construction which exceeded the vertical stresses around the toe of embankment. A large horizontal force would therefore, have been exerted along the lower part of the embankment, thus increasing the risk of failure. A tensile strength component would also have been generated by this lateral force. After the formation of tension cracks, which is likely to be during the early stages of movement, lateral stresses cease to
be of significance. Failure would, however, only continue if the shear strength of the soil is decreased by disturbance, or, high pore water pressures are developed on the slip surface. The latter condition was observed to precede failure of the trial embankment, after tension cracks had appeared lower down the embankment slope.

Conclusions.

From the foregoing discussion, it is possible to suggest some tentative corrections to the conventional undrained strength. All the corrections would involve a reduction in factor of safety.

Structural Anisotropy: (i) The ratio of horizontal undrained strength to vertical undrained strength in the shallow layers that will be involved in most embankment failures may be small, and could be neglected without serious loss of accuracy in stability calculations.

(ii) The undrained strength in the direction of bedding may be as low as 75% of that determined from vertical samples. A greater proportion of anisotropy may be reflected on a larger failure surface in the field than in the laboratory samples. As only a small proportion of a typical circular arc failure surface would involve failure in the plane of bedding, this factor would only have a small affect on stability calculations.
Size Effects: Where the fissure spacing is generally less than 100 mm, only insignificant reductions in undrained strength seems likely from the use of samples larger than 100 mm in diameter in laboratory test. This condition applies to the shallow layers of the Gault which will in general be involved in most embankment failures. Smaller specimens (38 mm diameter) obtained from U4 samples are likely to over-estimate the undrained strength, determined from 100 mm samples, by about 30%. Due to the fact that failure in the field may occur through a larger and more representative area, the insitu strength may be lower than that estimated from large laboratory samples. The insitu tests at Maldon suggest, taking an optimistic view, that a 16% error may result from this.

Strain Rate: For the time to failure involved in the trial embankment (34 days) a 20% error is likely to result in stability calculations, from the use of conventional undrained strength.

*A reduction from 86% to 55% was observed; allowing 5% for shear box error, Bishop (1966), and 10% for larger anisotropy results in a size error of 71 - 55 = 16%. The effect of anisotropy after the size correction is not any more significant than has been observed in the London Clay.*
iii) Laboratory Undrained Strength.

Measurements of undrained strength have been made on 100 mm x 200 mm samples of the Gault to reduce size effects. The samples were obtained by percussion boring and the tests were carried out with a time to failure of about 15 mins. The results are summarised below:

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth (m)</th>
<th>Undrained Strength $C_u$ (KN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solifluxion Deposits</td>
<td>1.0 - 2.0</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>2.0 - 3.6</td>
<td>62</td>
</tr>
<tr>
<td></td>
<td>3.6 - 4.5</td>
<td>76</td>
</tr>
<tr>
<td>Cryoturbated Gault</td>
<td>4.5 - 5.7</td>
<td>62</td>
</tr>
<tr>
<td></td>
<td>5.7 - 7.5</td>
<td>60</td>
</tr>
<tr>
<td>Weathered Gault</td>
<td>7.5 - 8.5</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>8.5 - 10.0</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td>10.0 - 12.05</td>
<td>62</td>
</tr>
<tr>
<td></td>
<td>12.05 - 13.6</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>13.6 - 15.35</td>
<td>70</td>
</tr>
</tbody>
</table>

The undrained strength of the fill material was determined from 38 mm samples, tested so that failure occurred in about 15 mins, was:

$$C_u = 110 \text{ kN/m}^2 \quad \phi_u = 0$$

There is little scatter in the undrained strength values of the undisturbed Gault though minor variations occur. This may be due to the use of 100 mm diameter samples in the laboratory tests.
The Gault was reduced to a crumbled mass of small fissure lumps by the excavation and compaction processes. The use of 38 mm samples for measuring the undrained strength of the fill is unlikely to involve any significant error arising from the size of the laboratory sample.

iv) Total Stress Analysis.

Analyses have been made to find the most critical failure surface consistent with the observed slip surface at the rear scarp and beyond the toe after the general collapse of the embankment. Further analyses established that this failure surface was also the most critical for the embankment. The results of stability analysis in which the laboratory undrained strength-depth variation reported previously was applied along the failure surface, are given below:

1) Circular arc through observed slip surface at rear and beyond toe, ab, Fig. 3.14,

Factor of Safety \( F_a = 1.43 \)

Ignoring the strength of the fill,

Factor of Safety \( F_b = 0.97 \)

ii) Circular arc through observed slip surface at rear and passing through toe of embankment, ad,

\( F_a = 1.88 \)

ignoring fill strength \( F_b = 0.79 \)
Ignoring the strength of the fill material results in a factor of safety of less than unity. This seems unlikely in view of the fact that no corrections have been applied to the undrained strengths for errors arising from testing rate, size effects, and anisotropy. Although this method, together with an adequate factor of safety, would apparently result in a safe design, it clearly does not reflect the observed mode of failure. As previously stated, the final collapse of the embankment involved shearing in the fill material which left the exposed rear scarp with a high degree of polish and slickensiding. It would have been appropriate to neglect the strength of the fill material in the analyses, if a tension crack had formed in the fill before the general collapse of the embankment; the observations before and after failure do not justify such an assumption.

The factor of safety for the most critical failure surface has been reassessed, making corrections for testing rate and anisotropy to the laboratory undrained strengths.

a) The undrained strengths have been reduced by 20% to allow for slower rate of loading in the field.

b) The undrained strength along those parts of the slip surface inclined at ±5° have been assumed to be 75% of the vertical undrained strength.
Applying correction b) lowers the factor of safety from 1.43 to 1.39 and a combination of a) and b) results in:

\[ F = 1.12 \]

The error in the calculated factor of safety lies within the limits suggested for loading failures by Bishop and Bjerrum (1960). However, in the failures considered by Bishop and Bjerrum, no corrections were applied to the undrained strength used in the analyses, for size, time or orientation effects. The errors indicated by these analyses should strictly be compared with \( F \) of 1.43 obtained for the trial embankment, using uncorrected undrained strengths.

v) Effective Stress Analysis.

In this section, the embankment failure is analysed assuming a circular arc failure surface, and an assumed pore water pressure distribution.

(i) Laboratory Effective Stress Parameters.

The results of consolidated undrained tests with measurement of pore water pressure on vertical 100 mm diameter samples of undisturbed Gault and 38 mm diameter samples of the Gault Clay fill, are given overleaf. The results show that at the site of the trial, weathering and frost disturbance have resulted in a reduction in effective cohesion \( c^t \). A similar observation has also been made for the London Clay, Lias Clay Keuper Marl by Chandler (1969), (1972).
<table>
<thead>
<tr>
<th>Zone</th>
<th>Depth below Ground Level</th>
<th>Effective Stress Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cryoturbated Gault</td>
<td>2.00 - 4.85 m</td>
<td>C' = 10 kN/m², ϕ' = 23.0°</td>
</tr>
<tr>
<td>Weathered Gault</td>
<td>4.85 - 7.20</td>
<td>C' = 13 &quot;</td>
</tr>
<tr>
<td>Unweathered Gault</td>
<td>7.20 -</td>
<td>C' = 25 &quot;</td>
</tr>
<tr>
<td>Gault Clay Fill</td>
<td></td>
<td>C' = 10 &quot;</td>
</tr>
</tbody>
</table>

(ii) Factors Affecting Effective Stress Parameters.

Very few studies have been made into the effects of sample size and orientation on peak effective stress parameters. Despite the variation in undrained strength with orientation of samples from the Ashford Common shaft and Wraysbury noted earlier, there was no significant difference between the effective stress parameters determined from 38 mm vertical, horizontal and inclined samples, Bishop et al (1965), Agarwal (1967). Marsland (1971), however, observed from tests on 98 mm (≈ 4 in) diameter samples of London Clay from Wraysbury, taken so that failure occurred along the bedding, that at low normal stresses, below about 100 kN/m² (13.5 p.s.i.), which are commonly encountered in stability problems, the shear strengths were appreciably lower than the comparable values determined from 38 mm samples by Agarwal (1967). The tests by Marsland (1971) also show that at the low stress levels, the failure envelope displays a marked curvature and passes through the origin. This is in conflict with the observation made from tests on 38 mm samples by Bishop et al (1965), from which it was concluded that no tendency to show a C' = 0 is displayed at the very low normal stresses.
Effective stress parameters for vertical 38 mm samples of the Barton Clay, Marsland and Butler (1967) were also found to be higher than those measured on 75 mm (3 in) and 125 mm (5 in) diameter samples of similar orientation. Insitu shear box tests on a square 610 mm (2 ft) sample, in which failure occurred in the horizontal plane, similarly also resulted in lower parameters than were measured on vertical 38 mm samples.

Far more data than is at present available, is required before the influence of structure on the shear strength of stiff fissured clays can be satisfactorily understood. The data available at present do, however, clearly demonstrate the need for shear tests on these soils to be performed on large samples.

(iii) Stability Analysis.

Analyses have been carried out assuming that the pore water pressure parameter $S$ is equal to 0.5 in. the foundation material, and in the fill material, at failure. In the foundation, the end of construction pore water pressure $U$ is thus, given by:

$$U = U_0 + 0.5 \Delta \sigma_v$$

where $U_0$ is the initial pore water pressure, and $\Delta \sigma_v$ is the increase in total vertical stress.

The variation of effective stress parameters with depth noted earlier, was used along the length of the slip surface in the foundation.

The most critical failure surface $ae$, Fig.3.14, gives a factor of safety $F = 1.19$. 
If the strength of the fill material is ignored, the factor of safety reduces to $F = 1.11$.

For a circular arc failure surface passing through the observed slip surface at the rear, and at the toe, $ab$, Fig. 3.14, the factor of safety is equal to $F = 1.26$.

Ignoring the strength of the fill material in this case results in a factor of safety $F = 1.20$.

Compared to the minimum factor of safety $F$ of 1.43 obtained from conventional total stress analysis, the effective stress method of analysis seems to give a more accurate estimate of factor of safety. Some of the factors affecting the total-stress analysis have a smaller effect on the effective stress method. In particular strain rate affects do not arise in the measurement of effective stress parameters as very slow rates of strain are employed.

The major difficulty with the effective stress method of analysis is that its accuracy depends mainly on the reliability of the pore water pressure distribution used in the analysis. A considerable discrepancy between field and calculated values of $F$ may arise where failure is non-circular, with the failure path occurring mainly along bedding. The influence of pore water pressure on the value of $F$ has been investigated by considering the failure pore water pressure distribution to follow the profile of the embankment slope. This distribution gives considerably higher
pore water pressures than the predicted distribution, yet, stability analyses show that the most critical failure surface, ae, results in a factor of safety $F = 0.92$ compared to $1.19$ obtained using predicted pore water pressures. For failure surface, ab, the value of $F$ is $1.03$ for the higher distribution compared to $1.26$ obtained using predicted pore water pressures.

The effective stress method of analysis, involving comparatively less uncertainties than the total stress method, appears to be a more reliable method of estimating construction stability even though the assumed pore water pressures may involve quite significant errors.

c) Investigation of the Embankment Failure.

(i) Site Investigation.

The post-failure profile was surveyed and the section through the centre of the embankment is shown in Fig. 3.16. The failure surface was established by two boreholes B.10 and B.11 and a trial pit excavated at the toe of the slipped mass. The trial pit, Fig. 3.17, was carefully logged and sampled, and a number of features noted are discussed below.

The trial pit was excavated on the 15th October, four days after the complete collapse of the embankment. The main slip surface was exposed and some slight movements were still observed to be taking place along it.
A relative displacement of 51 mm (2 in) occurred overnight across the slip surface on the northern face of the trial pit. The failure surface exposed was found to be highly polished and striated. It consisted of a slightly curved surface inclined at an average angle of 30°, which formed the main toe of the slide. This section of the failure surface occurred across the solifluxion layer. The adjoining section was situated in the cryoturbated Gault and displayed an irregular profile with an average dip of 10° into the embankment. Several slip surfaces with curved outlines which tended to rise steeply towards the ground surface, and become sub-horizontal with depth can also be seen in the drawing of the pit, Fig. 3.17. These slip surfaces are characteristic of non-continuous slip surfaces, which have been shown to be a feature of the layer of the Gault underlying the solifluxion mantle. One of these surfaces YZ, Fig. 3.16, was seen to have formed part of the failure path associated with a displaced block which had resulted in a tension crack XY, in the fill material. This type of displacement involving the formation of a tension crack in the fill material has been referred to as 'wedge' failure. It is possible that the location of 'wedge' failures and hence tension cracks in the fill material, were determined by the points of emergence at ground surface of non-continuous slip surfaces in the foundation.
(ii) Shear Strength Determination - Standard Tests.

A block sample was cut across the slip surface in the trial pit from which two sets of 3 No. 38 mm diameter samples containing the slip surface, were taken. Each set was taken such that the slip surface was inclined at 40° to the vertical axis of the samples. One set of these samples was tested under drained conditions in the triaxial apparatus, and the other set, under undrained conditions with a time to failure of about 15 mins. Standard consolidated undrained tests with measurement of pore water pressure and undrained tests were carried out on samples of the Gault clay fill and the cryoturbated Gault. The results of these, and other standard tests are given in Table 3.6.

(iii) Consolidated Undrained Test on Partly Saturated Gault Clay Fill.

In addition to the standard tests, 100 mm samples of the Gault clay fill obtained from the embankment, were tested under consolidated undrained conditions, without any prior saturation under back pressure.

The effective stress in a saturated soil (single pore fluid) could be defined by the difference between the total stress $\sigma$ and the pore water pressure $U$ without any serious loss of accuracy, Skempton (1960). For unsaturated soils, however, the pore space contains
two fluids, air and water. An expression defining effective stress in these soils was suggested by Bishop (1955) which has subsequently been justified by Skempton (1960), Jennings (1960), Bishop et al (1960), Coleman (1960). The expression defines the effective stress in partly saturated soil as follows:

\[ \sigma' = \sigma - U_a + \chi (U_a - U_w) \]  \hspace{1cm} (9)

where \( \sigma \) = total stress.
\( U_a \) = pore air pressure.
\( U_w \) = pore water pressure.
\( \chi \) is a parameter related to the degree of saturation, equalling unity when the soil is saturated, and zero for dry soils.

Bishop and Henkel (1965) advocate caution in the application of Eq. (9), as the principle of effective stress for saturated soils considers that changes in total stress and in the pore water pressure have similar effects on deformation and shearing characteristics. Donald (1961) has shown that the rate of volume change in a partly saturated soil varies with the degree of saturation, and the effects of saturation on volume change characteristics have also been noted by Burland (1961). Thus, changes in the degree of saturation in a partly saturated soil may alter the mechanical properties of the soil. It would appear from the data, however, that such changes mainly affect deformation characteristics.
For soils having a relatively high degree of saturation, a reasonably accurate evaluation of the effective stress does not require the measurement of pore air pressure. Tests by Blight (1961), Bishop and Henkel (1965) suggest that a unique relationship between $k$ and the degree of saturation $S$, may exist for different types of soils. At the higher levels of saturation, however, $(S > 90\%)$, $k$ was generally close to unity, and the effective stress expression, Eq. (9), approximates to:

$$\sigma' = \sigma - U_w \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (10)$$

The relationship between dry density and moisture content obtained in the British Standard (B.S.) compaction test are given in Fig. 3.15a. The mean result from core samples taken daily throughout construction are also shown in the figure, and it can be seen that the fill was placed generally about 5% above the optimum moisture content, corresponding to a degree of saturation of about 96%. Eq. (10) could, therefore, be used to estimate the effective stress of samples of the fill, without any significant loss of accuracy.

The difficulties of measuring pore water pressure in partly saturated samples have been discussed by Bishop (1960), Bishop and Henkel (1965),

*Data published by Bishop (1960) suggests that the assumption that $k = 1$ leads to a lower estimate of $C'$; the error in $C'$ however, becomes insignificant for high degrees of saturation.*
Bishop (1970). It has been shown that accurate measurement of effective stress parameters in partly saturated soil, compacted at or below optimum moisture content requires the use of high air entry porous discs in the triaxial test. The use of a coarse disc may result in the measurement of pore air pressure leading to high estimates of effective cohesion $c'$. Slow testing rates are also recommended to avoid pressure gradients in the pore water and air.

As the initial pore water pressure in a specimen is always negative, water will be drawn from the porous disc as soon as the sample is placed in contact with it. According to Bishop and Henkel (1965), a balance can be achieved if three conditions are met:

1) the initial pore water pressure is not lower than -1 atmosphere, (about -13 lb/sq in (90 kN/m$^2$)) as cavitation then occurs in the measuring system, allowing water to be drawn from it through the porous disc, into the sample.

2) the initial negative pore water pressure is not higher than the air entry value of the porous disc as air is then drawn into the porous disc displacing water into the sample.

3) after the sample is subjected to applied stress, the pressure difference ($U_a - U_w$) does not exceed the air entry value of the porous disc.
Due to the high degree of saturation in the Gault Clay fill, these difficulties could be overcome by the use of a porous ceramic disc with a high air entry value. In a partly saturated soil a change in volume occurs during undrained shear due to the compressibility and solubility in water of air and other gases in the pore space. This change can be measured from the amount of water entering or leaving the cell, by noting the displacement of the boundary between water and mercury in a manometer introduced between the cell and the pressure system in the standard triaxial apparatus, Fig. 3.15b.

The results obtained from tests on partly saturated samples of the Gault Clay fill are summarised in Figs. 3.15c, d. A typical consolidation curve is shown in Fig. 3.15e, from which the tendency for a marked volume change to occur in the first minute or

*For cases where condition 1) is not satisfied Bishop and Henkel (1965) recommend that the pore air pressure in the sample be artificially increased so as to raise the pore water pressure, which differs from the pore air pressure by \(U_a - U_w\), into the range which can be measured without cavitation.
so of consolidation can be seen. This behaviour results in a very rapid rate of shearing which is considered uncharacteristic of the actual failure. The large initial volume change is thought to correspond to the displacement of air in the sample. The shearing rates were determined from a corrected consolidation curve obtained by subtracting the initial volume change from all other subsequent readings. This procedure resulted in slower testing rates ranging from 0.00114 cm/min at the lowest cell pressure of 20 kN/m$^2$ (2.9 p.s.i.) to 0.11 cm/min at the maximum cell pressure of 275 kN/m$^2$ (40 p.s.i.).

The test results shown in Fig. 3.15d clearly reflect the sensitivity of the parameter $\bar{B}$ to moisture content around the optimum value. Even though the samples were in general 96% saturated, with an average moisture content of 32%, the value of $\bar{B}$ was less than 0.5, within the range of pressures acting along the slip surface in the fill. Only in the test carried out with a cell pressure of 275 kN/m$^2$ (40 p.s.i.), did $\bar{B}$ slightly exceed the value of 0.5. In contrast, $\bar{B}$ of approximate unity was obtained on saturated samples of the Gault Clay fill for which the average moisture content was only 2% higher than the unsaturated fill material. Similar observations have been reported by Elsden et al (1958).

The presence of air in samples of the Gault clay fill results in a modest increase in cohesion.
C' of about 7 kN/m² (1 p.s.i.) over the saturated samples, Fig. 3.15c. The value of φ' of 14° is, however, lower than that for the saturated samples, so that there is no significant overall increase in shear strength in the partly saturated samples which, as previously stated, were about 96% saturated with an air content of only about 2%. Elsden et al (1958), Bishop (1960), found that C' is quite sensitive to small changes in the degree of saturation, around the optimum moisture content. At the higher degrees of saturation, the changes in C' with moisture content becomes insignificant. The value of φ' in contrast only shows a small variation at all levels of saturation provided a slow testing rate is adopted at the shearing stage.

Test results for clays from the Sasumua dam in Kenya, and the Silvan dam in Australia reported by Bishop (1970) suggest that low φ' values are associated with the measurement of pore air pressure arising from the use of a porous disc of inadequate air entry value. The value of C' is then also significantly greater than that based on pore water pressure. The lower φ' value obtained for the partly saturated Gault clay fill would tend to cast some doubts on the pore pressure measurements, however, this would not be entirely consistent with the small increase in C' and the high degree of saturation of the samples. It has previously been noted that in highly saturated soils, the difference between pore
water pressure and pore air pressure is generally quite small. There is, in fact, essentially little difference between the parameters of the saturated and partly saturated samples of the Gault clay fill. The mineral halloysite predominates in the clays from the Sasumua dam and the Silvan dam, and this mineral is known to result in some apparent inconsistencies in the engineering properties of these clays.

(iv) Total Stress Analysis on Observed Failure Surface.

Total stress analyses have been made for the observed failure surface with the relevant undrained strengths corrected for time and orientation effect, which it has been shown, can introduce errors of 20% and 25% respectively. The following results were obtained from stability calculations:

i) for undrained strength $C_u$ in fill material
   $= 110 \text{kN/m}^2$ (determined from vertical samples)
   and the undrained strength in the foundation equal to the value determined from vertical samples of the Gault, at the levels of the slip surface.

   \[ F = 1.95 \]

ii) for $C_u$ in fill material $= 110 \text{kN/m}^2$ and
    $C_u$ in the foundation $= 17.2 \text{kN/m}^2$ (determined from samples taken across the slip surface in the field)

   \[ F = 1.38 \]

Applying corrections for time and orientation effects to i)

   \[ F = 1.57 \]

Applying correction for time effects to ii)

   \[ F = 1.10 \]

*Orientation effects only apply to the foundation $C_u$. 
The conventional analysis based on undrained strengths on vertical samples of the unsheared material results in a significant over-estimate of the factor of safety. The error is, however, reduced when the undrained strength determined for samples containing the slip surface is used in the analysis, and becomes insignificant after corrections have been made for time effects. The significance of this result is however not clear as the shear strength across the slip surface is so small that it may be due to membrane restraint. It also suggests that the stability of the embankment depends mainly on the strength of the fill material. Secondly, the site investigation established that the failure path in the foundation was situated in a zone of non-continuous slip surfaces. As such, a proportion of its length would have followed pre-existing slip surfaces with the remainder occurring through fissured, but previously unsheared material. The factor of safety of 1.10 obtained from ii) above, after correction for time effects, therefore, appears fortuitous. If it is assumed that say 70% of the failure surface occurred along pre-existing slip surfaces, with the remaining 30% occurring through fissured material, the factor of safety, after correction for time and orientation effects, is 1.30. For proportions of 80% and 20% for pre-existing slip surfaces and fissured material respectively, $F = 1.22$. 
Thus, even after corrections have been applied for time and orientation effects, the total stress analysis pertaining to the observed mode of failure, results in a significant over-estimate of factor of safety, considering that a factor of safety of 1.5 is generally adopted in design.

(v) Effective Stress Analysis on Observed Failure Surface.

General Collapse.

In view of the observed dissipation during construction, this method is more appropriate to the analysis of the failure. Stability calculations have been carried out for the observed failure surface, using a pore water pressure distribution based on the measured pore water pressures in pz B4 and B8 which were located on the failure surface. The pore water pressure distribution on the slip surface in the foundation shown in Fig. 3.16 has been completed on the assumption that the piezometric line continued beyond the toe of the embankment. A value of $\bar{B}$ of 0.5 was assumed in the fill material. As failure only involved a relatively small section of the fill it will be seen that the value of $\bar{B}$ in the fill is not critical. Analyses were carried out using the following assumptions and shear parameters:

(i) The failure surface was a continuous pre-existing slip surface with shear parameters of $C^t = 0$, $\phi^t = 14^\circ$. 
(ii) The saturated effective stress parameters of the Gault clay fill of $C^* = 14 \text{kN/m}^2$, $\phi^* = 23^\circ$ were mobilized along the length of the failure surface in the fill material.

For pore water pressure parameter in the fill:

<table>
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<th>$B$</th>
<th>Factor of Safety F</th>
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</thead>
<tbody>
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<td>0.5</td>
<td>0.69</td>
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<tr>
<td>0</td>
<td>0.79</td>
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<tr>
<td>1.0</td>
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</table>

The value of $B$ in the fill, thus has an insignificant effect on the calculated factor of safety for the embankment. The discrepancy in the calculated factor of safety may be attributed to:

(i) The linear approximation of the failure envelope for samples containing the slip surface, Fig. 3.15f, introduces some error in the $\phi^*$ value especially at the low effective stress range.

(ii) The assumption that failure occurred along a continuous pre-existing slip surface in the foundation is not valid.

(iii) The shear parameters in the fill are at variance with those obtained from tests on saturated samples. Little error is likely to arise from this as there is essentially no significant difference between the saturated and unsaturated effective strengths.

The first source of error could be investigated by using the actual failure envelope in
the analysis. The envelope in the lower stress range has been completed by extrapolation to the origin, Fig. 3.15f.

To isolate the effect of this factor, analysis has been carried out using the saturated parameters and a $\beta$ of 0.5 in the Gault clay fill. The analysis resulted in a factor of safety of 0.72; under the same condition a factor of safety of 0.68 was obtained using the linear approximation of $\sigma^f = 0$, $\phi^f = 14^\circ$. Thus, the use of the curved envelope only has a marginal effect on the factor of safety, which is still insufficient to account for observed discrepancy in the calculated value of factor of safety.

The second source of error arises from the fact that only a proportion of the length of the failure path in the cryoturbated Gault would have utilised suitably orientated sections of non-continuous slip surfaces in the layer. It has previously been noted that these discontinuities are separated by unsheared fissured clay which are capable of mobilising peak parameters. The effective shear strength along the failure surface in the cryoturbated Gault, would therefore, have been partly peak and partly residual. This deduction, however, neglects the fact that failure was observed to start at the toe of the embankment and move progressively towards the top of the slope. The failure surface is likely to have been subjected to non-uniform displacements,
and as a result, a variable shear strength would have been mobilized along pre-existing slip surfaces, and across previously unsheared material. It has been shown, however, that progressive failure is unlikely to be of significance in end-of-construction failures of high embankments.

The embankment failure has been analysed using the observed pore water pressure on the slip surface in the foundation, and under the following conditions:

\[ E = 0.5 \text{ in fill material.} \]

\[ C' = 20.7 \text{ kN/m}^2, \phi' = 14^\circ \text{ in fill material (unsaturated parameters).} \]

\[ C' = 0, \quad \phi' = 14^\circ \text{ along pre-existing slip surfaces.} \]

\[ C' = 10 \text{ kN/m}^2, \phi' = 24^\circ \text{ in the fissured cryoturbated Gault.} \]

Under the above conditions, it is required that 67.5\% of the length of the failure path in the foundation should utilise pre-existing slip surfaces, with the remaining length occurring through previously unsheared cryoturbated Gault, for a factor of safety \( F \) of 1 to be obtained in stability calculation. The factor of safety drops to 0.975 if the saturated parameters of the Gault clay fill are used in the analysis.

\'Wedge\' Type Failures.

The above analysis was mainly concerned with the stability of the whole embankment, and fails to take into account the observed mode of failure which has been described. Failure initially involved the lateral displacement of a block around the toe of the
embankment XYZB, Fig. 3.16, which left a vertical tension crack in the fill material. Part of the failure surface associated with the displacement of this block was seen in the trial excavation to have utilised the curved outline YZ, Fig. 3.16, of a non-continuous slip surface. This initial 'wedge' failure was followed by a series of other displacements extending to about midway along the embankment slope. A vertical tension crack IM was also associated with the uppermost wedge which has also been assumed to have utilised the outline of non-continuous slip surface MN, at failure.

The initial and final wedge failures have been analysed using the pore water pressure distribution determined for the final collapse. The wedge failures preceded the general collapse and the pore water pressure assumption slightly over-estimates those that actually existed during the wedge failures. The tensile strength developed across the fill material has been determined from the effective stress envelope for the partly saturated fill material, to be equal to 83 kN/m², Fig. 3.15c.

For the initial wedge failure XYZB, Fig. 3.16, the whole of the slip surface in the foundation is required to have occurred along pre-existing slip surfaces for a factor of safety F of 1.0 to be obtained in stability calculations.
For the uppermost wedge failure LMB, 90% of the failure path in the foundation is required to have occurred along pre-existing slip surfaces, with the remainder occurring through fissured, unsheared material, for a factor of safety $F$ of 1 to be obtained in stability calculations.

In both analyses the measured shear parameters of $C' = 0$, $\phi' = 14^\circ$ were applied to slip surfaces and $C' = 10 \text{ kN/m}^2$, $\phi' = 24^\circ$ were applied to the fissured cryoturbated Gault.

d) **Conclusions.**

A good estimate of the stability of the whole embankment was obtained from conventional analysis, based on the total stress method and the assumption that failure occurred on a circular arc, after corrections were applied to the laboratory undrained strength for time and orientation effects; the laboratory undrained strengths in the Gault foundation having been determined from 100 mm diameter samples. The neglect of the strength of the fill material was found to lead to an inaccurate assessment of stability.

In the less conventional analysis, in which the total stress method was applied to the observed non-circular failure surface, a factor of safety of 1.57 was obtained when the corrected fissured undrained strength was used throughout in the foundation. A much closer estimate was achieved after allowance was
made for the fact that in the foundation, failure occurred partly through unsheared fissured material and partly along pre-existing slip surfaces. The assumption that 70% pre-existing slip surfaces and 30% unsheared fissured material were involved along the observed failure path in the foundation resulted, after correction for time and orientation effects, in a factor of safety of 1.30.

The total stress method of analysis assumes that there is zero dissipation of pore water pressure during loading. Although the correction for the discrepancy between the loading rates in the field, and those in the laboratory, has been mainly responsible for the reasonable estimates of factor of safety obtained from the total stress method, the effect of time on the pore water pressure distribution is completely neglected in this form of analysis. In view of the high values of coefficient of consolidation determined for the Gault foundation, from observed dissipation rates and insitu tests, some drainage is likely to have occurred during construction, especially under low levels of stress. Some dissipation of pore water pressure was indeed recorded overnight in all the piezometers in the foundation, during the early stages of construction. The calculated values of the pore pressure parameters $A$ and $B$ are also only about half the predicted values. The good estimates of factor of safety obtained from the total stress
method of analysis may therefore be fortuitous.

The effective stress method of analysis is more appropriate to the embankment failure in view of the dissipation of pore water pressure during construction. Such an analysis in which failure was assumed to occur along a circular arc, and in which $\beta$ was assumed to be about 0.5, resulted in a minimum factor of safety of 1.19. The effective stress parameters in the foundation were determined from vertical 100 mm diameter samples, and the influence of any orientation effects was ignored in the analysis. The post-failure investigations, however, established that the failure surface was mainly non-circular, with the major part of its length in the foundation, occurring almost parallel to bedding. The investigations also established that the failure path in the foundation was located in a zone of non-continuous slip surfaces which would have been partly utilised at failure. The remainder of the failure surface would have occurred through the fissured Gault, separating the non-continuous slip surfaces. Stability calculations showed that at failure, 67.5% of the failure path in the foundation utilised pre-existing slip surfaces.

The observed mode of failure and subsequent calculations indicate that the stability of embankments on the Gault would depend on the stability around the toe. As the failure path in these areas is likely to be situated in the shallow layers, which also have
the most intensive distribution of slip surfaces, failure in the foundation would inevitably utilise pre-existing slip surfaces along the major part of its length. Stability could, therefore, be enhanced by increasing the shear strength capable of being mobilized around the toe. This objective could be achieved by digging out the slip surfaces around the toe, in the layer of the foundation in which the failure path is likely to be located, the top 3 m or so, and replacing it with granular fill material, or the remoulded, excavated Gault.

The use of a granular backfill would reduce the excess pore water pressure on the potential failure surface, but the remoulded Gault backfill would result in higher shear strength than a granular $\phi' = 30^\circ$ fill. The factor of safety $F$ of the whole embankment has been calculated with the foundation of the initial wedge, between B and Y, Fig. 3.16, replaced with a $\phi' = 30^\circ$ fill, and alternatively, with remoulded Gault clay. Assuming that the $\phi' = 30^\circ$ fill only lowers the pore water pressure to original ground level over the section BY, $F$ was calculated to equal 1.12. For the Gault backfill, with the pore water pressure distribution corresponding to the maximum observed, $F$ was calculated to be 1.20.

It would therefore be more economical to reuse the remoulded, excavated Gault as backfill, for limited 'dig-out' below embankment slopes. Adequate
construction stability could in this case have been ensured, by the use of flatter side slope and, or, by digging out and replacing a greater proportion of the failure path in the foundation.

The procedure up till now in Kent, has been to replace the foundation material below the full length of the embankment slope. With the exception of high embankments, and instances where a restriction is placed on side slopes, the remoulded, excavated material has been re-used as backfill. In view of the results of the trial, replacement will now only be necessary over a limited section of the foundation below the toe areas of most embankments, if the typical side slopes previously used in embankment construction are adhered to.
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<th>Geology at Pz. Tip</th>
<th>Initial Pore Water Pressure Head H (m)</th>
<th>Final Pore Water Pressure Head H_T (m)</th>
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Faulty Installation
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<th>Geology at Pz. Tip</th>
<th>Initial Pore Water Pressure Head H (m)</th>
<th>Final Pore Water Pressure Head H_T (m)</th>
<th>Final Height of Fill Above Pz. (m)</th>
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**H** = Head  
**WG** = Weathered Gault  
**SG** = Soliflucted Gault  
**UG** = Unweathered Gault  
**CG** = Cryoturbated Gault
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<td>0.68</td>
</tr>
<tr>
<td>(E3d)</td>
<td>UG</td>
<td>0.53</td>
<td>0.57</td>
<td>0.98</td>
</tr>
<tr>
<td>(E2d)</td>
<td>UG</td>
<td>0.49</td>
<td>0.56</td>
<td>0.88</td>
</tr>
<tr>
<td>(E1d)</td>
<td>UG</td>
<td>0.54</td>
<td>0.57</td>
<td>0.88</td>
</tr>
</tbody>
</table>

**Key to Geology**
- SD: Solifluxion Deposit
- CG: Gyturbated Gault
- WG: Weathered Gault
- UG: Unweathered Gault

**Col.1:** Stage 1, $\bar{E}$, height of fill = 2.75 m.
**Col.2:** End of Construction $\bar{E}$, height of fill = 0.65 m.
**Col.3:** Stage 4, $\bar{E}$, height of fill increased from 7.90 to 10.65 m.
### TABLE 3.3
Oxford Trial Embankment.

Observed and Predicted Construction Pore Water Pressure \((A = 0.40)\)

<table>
<thead>
<tr>
<th>Pz. No.</th>
<th>Excess Pore Water Pressure (\text{KN/M}^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial Stage, Fill = 2.75m</td>
</tr>
<tr>
<td></td>
<td>Observed</td>
</tr>
<tr>
<td>E8s</td>
<td>10.3</td>
</tr>
<tr>
<td>E7s</td>
<td>8.4</td>
</tr>
<tr>
<td>E6s</td>
<td>7.9</td>
</tr>
<tr>
<td>E5s</td>
<td>12.8</td>
</tr>
<tr>
<td>E4s</td>
<td>13.7</td>
</tr>
<tr>
<td>E3s</td>
<td>10.8</td>
</tr>
<tr>
<td>E2s</td>
<td>10.3</td>
</tr>
<tr>
<td>Els</td>
<td>15.7</td>
</tr>
<tr>
<td>E8D</td>
<td>8.8</td>
</tr>
<tr>
<td>E7D</td>
<td>14.2</td>
</tr>
<tr>
<td>E6D</td>
<td>10.8</td>
</tr>
<tr>
<td>E5D</td>
<td>25.0</td>
</tr>
<tr>
<td>E4D</td>
<td>26.4</td>
</tr>
<tr>
<td>E3D</td>
<td>26.0</td>
</tr>
<tr>
<td>E2D</td>
<td>26.0</td>
</tr>
<tr>
<td>E1D</td>
<td>26.5</td>
</tr>
</tbody>
</table>

- Solifluxion Deposits
- Cryturbated Gault
- Weathered Gault
- Unweathered Gault
### TABLE 3.4

Field Permeability Test Results Below Embankment Shoulder.

<table>
<thead>
<tr>
<th>E2 No.</th>
<th>Depth (m)</th>
<th>Geology</th>
<th>$H = 0$</th>
<th>$H = 3.20$ m.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\Delta \mu$</td>
<td>$\mu$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$K_{H/m^2}$</td>
<td>$K_{H/m^2}$</td>
</tr>
<tr>
<td>E8s</td>
<td>0.88</td>
<td>SD</td>
<td>17.0</td>
<td>6.89</td>
</tr>
<tr>
<td>E8s</td>
<td>1.25</td>
<td>SD</td>
<td>20.0</td>
<td>&quot;</td>
</tr>
<tr>
<td>E6s</td>
<td>1.85</td>
<td>SD</td>
<td>30.6</td>
<td>&quot;</td>
</tr>
<tr>
<td>E5s</td>
<td>2.45</td>
<td>CG</td>
<td>35.5</td>
<td>&quot;</td>
</tr>
<tr>
<td>E4s</td>
<td>3.02</td>
<td>CG</td>
<td>43.5</td>
<td>&quot;</td>
</tr>
<tr>
<td>E3s</td>
<td>3.61</td>
<td>CG</td>
<td>48.0</td>
<td>&quot;</td>
</tr>
<tr>
<td>E2s</td>
<td>4.05</td>
<td>CG</td>
<td>62.2</td>
<td>&quot;</td>
</tr>
<tr>
<td>E1s</td>
<td>4.85</td>
<td>CG</td>
<td>56.4</td>
<td>&quot;</td>
</tr>
<tr>
<td>E2D</td>
<td>5.50</td>
<td>WG</td>
<td>65.5</td>
<td>&quot;</td>
</tr>
<tr>
<td>E7D</td>
<td>6.10</td>
<td>WG</td>
<td>67.5</td>
<td>&quot;</td>
</tr>
<tr>
<td>E6D</td>
<td>6.70</td>
<td>WG</td>
<td>73.0</td>
<td>&quot;</td>
</tr>
<tr>
<td>E5D</td>
<td>7.30</td>
<td>WG</td>
<td>76.0</td>
<td>&quot;</td>
</tr>
<tr>
<td>E4D</td>
<td>7.85</td>
<td>UG</td>
<td>81.5</td>
<td>&quot;</td>
</tr>
<tr>
<td>E3D</td>
<td>8.55</td>
<td>UG</td>
<td>89.5</td>
<td>&quot;</td>
</tr>
<tr>
<td>E2D</td>
<td>9.15</td>
<td>UG</td>
<td>96.0</td>
<td>&quot;</td>
</tr>
<tr>
<td>E1D</td>
<td>9.70</td>
<td>UG</td>
<td>100.4</td>
<td>&quot;</td>
</tr>
</tbody>
</table>

UG: Unweathered Gault. *: Curved $\psi$ vs $\sqrt{t}$ plot.
## TABLE 3.5a

Calculation of Coefficient of Consolidation $C_v$
From Insitu Permeability $K$ and Laboratory Compressibility $M_v$

1) **Initial Conditions: Insitu Overburden Pressure.**

<table>
<thead>
<tr>
<th>Zone</th>
<th>Depth* (m)</th>
<th>Effective Overburden Pressure (kPa)</th>
<th>$K$ (m/yr)</th>
<th>$M_v$ (m$^2$/MN)</th>
<th>$C_v = K/M_v$</th>
<th>Cac. $C_v$ (m$^2$/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD</td>
<td>1.5</td>
<td>20</td>
<td>0.35</td>
<td>0.050</td>
<td>650</td>
<td>2.5</td>
</tr>
<tr>
<td>CG</td>
<td>3.5</td>
<td>40</td>
<td>0.27</td>
<td>0.050</td>
<td>520</td>
<td>1.0</td>
</tr>
<tr>
<td>WG</td>
<td>6.5</td>
<td>68</td>
<td>0.34</td>
<td>0.050</td>
<td>630</td>
<td>2.5</td>
</tr>
<tr>
<td>UG</td>
<td>8.5</td>
<td>80</td>
<td>0.15</td>
<td>0.030</td>
<td>470</td>
<td>0.31</td>
</tr>
</tbody>
</table>

11) **After Placing 3.20m of Fill**

<table>
<thead>
<tr>
<th>Zone</th>
<th>Depth* (m)</th>
<th>Effective Overburden Pressure (kPa)</th>
<th>$K$ (m/yr)</th>
<th>$M_v$ (m$^2$/MN)</th>
<th>$C_v = K/M_v$</th>
<th>Cac. $C_v$ (m$^2$/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD</td>
<td>1.5</td>
<td>76</td>
<td>0.180</td>
<td>0.045</td>
<td>372</td>
<td>1.50</td>
</tr>
<tr>
<td>CG</td>
<td>3.5</td>
<td>120</td>
<td>0.090</td>
<td>0.040</td>
<td>210</td>
<td>0.54</td>
</tr>
<tr>
<td>WG</td>
<td>6.5</td>
<td>142</td>
<td>0.098</td>
<td>0.035</td>
<td>260</td>
<td>2.50</td>
</tr>
<tr>
<td>UG</td>
<td>8.5</td>
<td>155</td>
<td>0.054</td>
<td>0.030</td>
<td>167</td>
<td>0.31</td>
</tr>
</tbody>
</table>

*: Average depth in each zone.

SD: Solifluxion Deposits.
CG: Cryturbated Gault.
WG: Weathered Gault
UG: Unweathered Gault.
**Table 3.5b**

Field Dissipation $C_v$ ($t = 7\frac{1}{4}$ days, Ht. of fill = 0.83m)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Depth* $Z$ (m)</th>
<th>$\sigma'$ $\frac{KN}{m^2}$</th>
<th>$\frac{Z}{H}$</th>
<th>$U_0%$</th>
<th>$T$</th>
<th>$C_v = \frac{H^2 T}{t}$ $\frac{m^2}{y}$</th>
<th>$Oed. C_v$ $\frac{m^3}{y}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD</td>
<td>1.5</td>
<td>35</td>
<td>0.125</td>
<td>80</td>
<td>0.075</td>
<td>525</td>
<td>2.5</td>
</tr>
<tr>
<td>CG</td>
<td>3.5</td>
<td>65</td>
<td>0.285</td>
<td>60</td>
<td>0.060</td>
<td>420</td>
<td>1.0</td>
</tr>
<tr>
<td>WG</td>
<td>6.5</td>
<td>90</td>
<td>0.540</td>
<td>39</td>
<td>0.080</td>
<td>560</td>
<td>2.5</td>
</tr>
<tr>
<td>UG</td>
<td>8.5</td>
<td>100</td>
<td>0.71</td>
<td>25</td>
<td>0.070</td>
<td>490</td>
<td>0.31</td>
</tr>
</tbody>
</table>

$H = 12.0$ (Fig. 3.13a)

---

**Table 3.5c**

Apparent End of Construction $C_v$ ($t = 34$ days)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Depth* $Z$ (m)</th>
<th>$\sigma'$ $\frac{KN}{m^2}$</th>
<th>$U_0%$</th>
<th>$T$</th>
<th>$C_v = \frac{H^2 T}{t}$ $\frac{m^2}{y}$</th>
<th>$Oed. C_v$ $\frac{m^3}{y}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD</td>
<td>1.5</td>
<td>140</td>
<td>12.5</td>
<td>0.062</td>
<td>75</td>
<td>0.50</td>
</tr>
<tr>
<td>CG</td>
<td>3.5</td>
<td>192</td>
<td>26.0</td>
<td>0.105</td>
<td>127</td>
<td>0.52</td>
</tr>
<tr>
<td>WG</td>
<td>6.5</td>
<td>220</td>
<td>22.0</td>
<td>0.080</td>
<td>97</td>
<td>2.50</td>
</tr>
<tr>
<td>UG</td>
<td>8.5</td>
<td>250</td>
<td>12.0</td>
<td>0.055</td>
<td>67</td>
<td>0.31</td>
</tr>
</tbody>
</table>

$H = 10.6$m (fig 3.13b)

*: Average depth in each zone

$\sigma'$ = Effective Pressure

SD: Solifluxion Deposits.
CG: Cryoturbated Gault
WG: Weathered Gault
UG: Unweathered Gault.
<table>
<thead>
<tr>
<th>Location</th>
<th>Sample Diameter</th>
<th>Type of Test</th>
<th>$C^t$ or $C_u$ (\text{KN/m}^2)</th>
<th>$\phi^t$ or $\phi_u$ (\text{deg.})</th>
<th>$\gamma$ (\text{kg/m}^3)</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>Clay Fraction ((&gt;\phi_u))</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slip Surface</td>
<td>38</td>
<td>Drained Quick</td>
<td>$C_u = 17.2$</td>
<td>$\phi_u = 0$</td>
<td>1930</td>
<td>89</td>
<td>28</td>
<td>61</td>
<td>53</td>
<td>linear approximation</td>
</tr>
<tr>
<td>Gyraturbated Gault</td>
<td>100</td>
<td>Consol. Und. Quick</td>
<td>$C^t = 10$ (\phi^t = 24)</td>
<td>$\gamma = 2000$</td>
<td>78</td>
<td>34</td>
<td>44</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weathered Gault</td>
<td>100</td>
<td>&quot;</td>
<td>$C^t = 13$ (\phi^t = 13)</td>
<td>2010</td>
<td>78</td>
<td>25</td>
<td>57</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unweathered Gault</td>
<td>100</td>
<td>&quot;</td>
<td>$C^t = 25$ (\phi^t = 16.5)</td>
<td>2003</td>
<td>87</td>
<td>32</td>
<td>55</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gault Clay Fill</td>
<td>38</td>
<td>Consol. Und. Quick</td>
<td>$C^t = 14$ (\phi^t = 23)</td>
<td>1870</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Saturated samples</td>
<td></td>
</tr>
<tr>
<td>Gault Clay Fill</td>
<td>100</td>
<td>Consol. Und.</td>
<td>$C^t = 20.7$</td>
<td>$\phi^t = 14$</td>
<td>1850</td>
<td></td>
<td></td>
<td></td>
<td>Unsaturated samples</td>
<td></td>
</tr>
</tbody>
</table>
1 HEAD. Firm brown CLAY with flints generally remoulded and sheared.

2 SOLIFLUCTED Firm grey brown CLAY with flints remoulded and sheared.
GAULT CLAY with phosphatic nodules chalk pellets and tufa nodules.

3 CRYOTURBATED Stiff fissured grey CLAY with carbonate patches.
GAULT CLAY

7 Vein of brown Silt in a matrix of grey clay flints chalk pellets and tufa nodules.

Level at which water was encountered during excavation.

Slip surface.

(Orientation of pit shown in Fig 3.1)
1 HEAD
Soft mottled reddish brown CLAY with flints generally sheared and remoulded.

2 SOLIFLUCTED
GAULT CLAY
Soft finely fissured whitish grey CLAY with chalk pellets and tufa nodules.

3 CRYOTURBATED
GAULT CLAY
Firm finely fissured blue grey CLAY with traces of carbonate.

3a CRYOTURBATED
GAULT CLAY
Firm finely fissured whitish grey CLAY with high carbonate contents.

Sides of pit remained stable for 24 hours
Water seepage at bottom of pit.

Slip surface.

(Orientation of pit shown in Fig 3.1)
PLAN OF TRIAL EMBANKMENT
SITE SHOWING PEZOMETER INSTALLATION
(Scale 1 in 500)
Initial Foe Water Pressure Distribution at Trial Embankment Site

Initial P.W.P Head \( H_{m} \) (m)

Granular Head

Slightly Fluctuated Gault

Cryoturbated Gault

Weathered Gault

Unweathered Gault

Depth of Tip Below Surface (m)

@ point along Section A:

\( \Delta \) " " " B

\( \times \) " " " C

" Below Shoulder

Fig 3.5
M 25 TRIAL EMBANKMENT POROWATER PRESSURE DISTRIBUTION B.H. NO. E/1s

HEIGHT OF FILL ABOVE PEIZOMETER

ORIGINAL GROUND LEVEL

PEIZOMETER LEVEL

SEPTEMBER 1970

OCTOBER 1970
Total Vertical Stress Increment vs. Zm

Observed Rise in Excess Pore Water Pressure

Final Level of Overburden Pressure

Swell

PZ on slip surface

Vertical chain lines
Observed as in Excess Pressure Pressure below Shoulder
In situ Permeability Test Results
Prior to the start of construction

\[ q = \frac{D_{\text{h}} \times \pi \cdot d^2}{4} = K \cdot D \cdot \text{cm}^2/\text{min} \]

- \( d = \) internal diameter of horizontal tube
- \( = 0.38 \text{ cm} (0.15 \text{ m}) \)
- \( n = \) gradient \( \times K \)

![Graph showing permeability test results](image)
In situ permeability test results

(3.20 m height of fill above pz)

\[ q_{\mu} = \frac{D_{60} \times \pi c^2}{4} = \frac{D_{60} \times K \times \text{cm}^3}{\text{min}} \]

\[ c = \text{internal dia. of horizontal tube,} \]
\[ = 0.33 \text{cm (0.15 in)} \]

\[ n = \text{gradient} \times K \]

Fig 3.12b
Apparent Construction Dissipation below Embankment Shoulder

- Soil fluxion deposits
- Cored turbated Gault
- Weathered Gault
- Unweathered Gault

Fig 3.13b
Consolidation of the Gault Foundation at the Trial Embankment Site

Mean Effective Pressure \( \sigma' \) KN/m\(^2\)

- Sollifected Gault
- Cryptoturbated Gault
- Weathered Gault
- Unweathered Gault


Weald Clay
Sevenoaks Bypass Weeks (1970)

London Clay, Arpley Dam Rowe (1972)

Coefficient of Consolidation \( c' \), m/yr
**B.S. Compaction Test - Otford Trial**

**Gault Clay Fill**

- Specific Gravity of Sample: 2.70
- Maximum Dry Density: 91.3 lb/ft³ or 1.46 Mg/m³
- Optimum Moisture Content: 28.5%
- Air Voids at Optimum Moisture Content: 5%

![Graph showing compaction test results](Fig3.15a)
Modification to the Triaxial Apparatus for the Measurement of Consolidated Undrained Strength on Partly Saturated Soil (after Bishop and Henkel 1965)
CONSOLIDATED UNDRAINED TEST WITH P.W.P. MEASUREMENT
ON SATURATED GAULT CLAY FILL
(38x76mm Samples)

Av. m/c before test 31%
Av. m/c after test 34%

\[ C' = 14.0 \text{ kN/m}^2 \quad \phi' = 23^\circ \]

CONSOLIDATED UNDRAINED TEST WITH P.W.P. MEASUREMENT
ON PARTLY SATURATED GAULT CLAY FILL
(104x208mm Sample)

Range of m/c before and after test: 31.7 - 32.7%

\[ C = 20.7 \text{ kN/m}^2 \quad \phi' = 14^\circ \]

\[ \text{Tensile Strength of Fill} = \frac{C}{\tan 14^\circ} = 83 \text{ kN/m}^2 \]

Shear Stress kN/m²

Effective Normal Stress kN/m²
Results of Consolidated Undrained Test on Natural Fault Fill

Stress-Strain Curves

Pore Water Pressure Variation

Volume Change During Shear
Effective Normal Stress \( \text{kN/m}^2 \)

- Observed Residual Envelope
- Observed Peak Envelope
- Linear Approximation of
- Observed Peak Envelope

Shear Stress \( \text{kN/m}^2 \)

100

200

400

Continuing the Failure Surface

Mohr Envelope for Drained Triaxial Tests on 38mm Samples

Fig. 3.15.4
CHAPTER 4.

STABILITY OF CUTTING SLOPES

The Otford Trial Cutting Investigation.

4.1 Background to the Investigation

a) Introduction.

A cutting trial was incorporated into the excavation for fill material during the construction of the trial embankment at Otford. The trial was designed to give a comprehensive picture of the pore water pressure distribution at all stages of construction, and in addition, data was sought on the insitu shear parameters of the Gault. A section of the cutting face was, therefore, designed with steeply inclined slopes at which it was anticipated that failure would occur during or soon after construction. Other slopes were excavated at slacker angles expected to remain stable during or some time after construction, to facilitate a study of time-effects on the stress-strain characteristics of the Gault.

b) Description of the Site and Geology.

The trial site is situated on a moderate spur about \( \frac{1}{4} \) mile east of the Darent valley, Fig. 4.1. The natural ground slopes at an angle of about 3.5° along the axis of the spur which runs in a NE-SW direction.
The gentle southerly slope is maintained until it meets the flood plain of a tributary of the river Darent.

The geological sequence at the site was determined from BH.3800/1 in which continuous samples were taken to a depth of 7.62 m. The succession established from the description of the samples is summarised below:

- 0 - 0.38 Top Soil
- 0.38 - 1.90 Granular Head Material
- 1.90 - 2.46 Soliflucted Gault
- 2.46 - 3.51 Cryoturbated Gault
- 3.51 - 4.62 Weathered Gault
- 4.62 - 7.61 Unweathered Gault

The thickness of the Gault at the site i.e. the depth to the underlying Folkestone Beds is estimated from the Sevenoaks Area Memoirs (Geological Survey) to be about 61 m, the lower 15 m of which comprises the Lower Gault.

c) Design of the Experiment.

The layout of the trial site is shown in Fig. 4.2. The cutting trial experiment was restricted to the northern half of the site along which the cutting face was excavated at various inclinations. The excavation generally extended to a depth of about 7.9 m below original ground level and the slope angles varied from 1 (vertical) on 3 (horizontal) at CS. 3 + 00 to about 1 on ½ at CS. 6 + 00. The longitudinal section
along the centre line of the site is shown in Fig. 4.3.

Three cross sections 3N, 4N and 5N at CS. 3 + 10, 4 + 10 and 5 + 10 respectively were instrumented with piezometers and slope indicators in advance of site work. The instrumentation programme was restricted by the fact that a minimum distance had to be maintained between piezometers and slope indicators within the actual cutting area so as to allow access by a scraper comprising a 'Caterpillar D8' tractor and a 20 cu.yd. box. Because of this restriction, slope indicators were given priority and all of them were installed before the start of site work. The majority of the piezometers shown in Fig. 4.2 were, however, also installed in advance of site work.

Slope indicators were installed generally to a depth of about 5 m below the cutting floor level and the location of piezometer tips was based mainly on the likely failure paths determined from previous studies of cutting slope failures in the Gault. The location of piezometer tips and slope indicators relative to the geological sequence established at the site may be seen from Figs. 4.16 and are also summarised in Table 4.1.

d) Excavation of Slopes.

The site work was carried out in a period of eight weeks between the 17th August and 13th October, 1970. Regular surveillance during the excavation provided valuable information on the macro-fabric and the geology.
of the site. Some of these observations have been used together with those from the initial site investigation, and borings for the installation of piezometers and slope indicators, in the preparation of cross-sections shown in Figs. 4.16. Details of the macro-fabric are discussed in the relevant sections of the Chapter.

4.2 Initial and Construction Pore Water Pressures

a) Initial Pore Water Pressure

The piezometers at the trial site were installed between eight and twelve weeks prior to the start of construction in August 1970, with the exception of pz. 3800/1 which was installed during the initial site investigation in December 1969. Piezometer 3800/1 approached equilibrium conditions within six weeks after its installation, Figs. 4.4a, b. The factors affecting time-lag in piezometers have already been discussed in Chapter 3. The permeability corresponding to the observed pore water pressure rise in pz. 3800/1, has been calculated to equal $7.1 \times 10^{-7}$ cm/sec, (0.224 m/y), which is about 100 times the corresponding laboratory value determined from 100 mm diameter samples, Table 4.2. The time-lag has been calculated according to Gibson (1963), to equal 25 mins, which will be seen to be consistent with observations made during construction.

In contrast to piezometer 3800/1 for which boring was by shell and auger method, the rest of the
piezometers at the trial site which were sunk by flight auger and a rotary coring machine, showed a faster rate of pore water pressure equilization, with equilibrium conditions attained in most cases in two weeks and generally within three weeks. A typical example of the initial pore water pressure response in these boreholes can be seen in Fig. 4.4c. The slower response noted in pz. 3800/1 is considered a reflection of the greater degree of disturbance and stress adjustment time-lag, associated with shell and auger boring. It also follows that the insitu permeability of the Gault may be as high as 20 x 10^-7 cm/sec (0.66 m/y) which is about 300 times the laboratory value.

Although the water levels in the piezometers prior to the start of construction do not correspond to the maximum values, steady state conditions had, however, been attained. The distribution of pore water pressure with depth below ground surface is shown in Fig. 4.5. The figure shows two hydrostatic distributions corresponding to piezometers on the northern slope (north of centre line, Fig. 4.2), and southern slope. A closer examination of the data, shows that this distinction stems from the method of presentation; the pore water pressure at the site, at the start of construction, is best represented by a flow net, Fig. 4.6.

The geological sequence at the site, when superimposed on the pore water pressure – depth relationships, Fig. 4.5, shows that the initial pore pressure
regime only relates to the weathered and unweathered Gault at the site. All the piezometers in the soliflucted and cryoturbated Gault layers were dry at the start of construction.

b) Construction Pore Water Pressure.

(i) Observed Piezometric Behaviour.

The pore water pressure variation during construction for piezometers along section 511 given in Figs. 4.4c, d, are typical of those that were observed along the other sections. They show a general tendency for pore water pressure to fall commensurately with the excavation of the cuttings. Although the most significant reduction in pore water pressure occurred in piezometers directly affected by excavation, there was also a lowering of the ground water level at the rear of the cutting slopes.

A continued drawdown was also observed in all piezometers after excavation had ceased directly above them, but was in progress lower down the slope. The greater part of the drawdown in piezometers located near the top of the cuttings occurred under this condition. This suggests that there was either a time-lag in pore water pressure response to excavation or that drawdown is a function of both the changes in vertical and horizontal stresses consequent on the excavation of the cuttings.

The drawdown was however, in every case seen to cease within a very short period after the completion
of excavation, and even during a short period of cessation of excavation during dry weather in September, due to the unsatisfactory performance of the compacting roller at the trial embankment site, pore water pressure equilibrium was almost immediately attained. A slight recovery in pore water pressure also occurred in some piezometers during this period.

The continued drawdown after the attainment of the final ground level can therefore only be attributed to the lateral release of pressure in front of the piezometers and changes in the magnitude of the principal stresses arising from their probable reorientation.

The drawdown along the cutting slopes has in consequence been considered in two parts relating firstly to the attainment of the final level above each piezometer $H_2$, and the end of construction $H_3$, Table 4.1. This two-fold division stems from the fact that if each piezometer along a trial section is considered to be located below the toe of a slope where the finished level is equal to the final level above that piezometer (along the trial section), then the end of construction drawdown would be equal to $H_2$.

A summary of events relevant to the construction pore water pressure data is given below:

17th August 1970: Start of Excavation — removal of Granular Head everywhere along the cutting site.

28th " " Excavation of Granular Head completed on northern slopes.
30th August 1970: Excavation of Granular Head completed on southern slopes.

1st September 1970: Excavation suspended because of unsatisfactory compacting roller at embankment site.

6th " " Excavation restarted.

9th-15th" " Cessation of excavation due to heavy rainfall; water accumulated in trial pit and was later pumped out.

17th " " Excavation restarted - confined to northern slopes and lower part of southern slopes.

23rd " " Excavation extended to cover whole site.


Total period of construction: 58 days (8 weeks, 2 days).

The observed drawdown during construction in the piezometers along the northern and southern slopes are given in Table 4.1 and are plotted in Fig. 4.7a, b, respectively. It can be seen from the figures that drawdown is directly related to the inclination of cutting slope, with the maximum drawdown associated with the steepest slope.

The water levels recorded in pz. 3800/1 have been shown to be consistent with those measured generally at the trial site before the start of construction, Fig. 4.5. The pore water pressure in the piezometer was monitored throughout the construction period and the relevant details are given in Table 4.1. The drop in water level in pz. 3800/1 during construction
of about 0.5 m, is typical of those commonly observed during the trial period in piezometers located in the unweathered Gault, at sites of similar geological setting to the trial area. This suggests that PZ 3800/1 was outside the domain of construction drawdown, and as such, could be regarded as a control piezometer. The horizontal axis of the drawdown curves given in Figs. 4.7 should, therefore, be lowered by 0.5 m to allow for the natural lowering of the water table during construction. This measure shows that there was no significant drawdown at the rear of the southern slopes, Fig. 4.7b, where the direction of cutting slope is opposite to that of the natural ground slope.

Attention has been drawn to the fact that there was continued drawdown in piezometers along the cutting slopes after the ground surface above them had reached their final level. This is regarded as an indication of the dependence of drawdown on both the vertical and horizontal stress changes consequent on the excavation of the cuttings. It has previously also been noted that the pore water pressure response to excavation was almost immediate and that drawdown could be considered in two component parts pertaining to the attainment of the final level above each piezometer $H_2$, and the completion of excavation $H_3$. The drawdown corresponding to the final level above each piezometer ($H_1 - H_2$) is subsequently referred to as the 'immediate' drawdown.
The variation of immediate drawdown with thickness of excavation material, for the northern slope, Fig. 4.8a, is seen to be independent of slope angle. A small correction has been applied for the natural lowering of the water table, which is assumed to have increased uniformly from 0 to 0.50 m during the period of construction. The variation of immediate drawdown with thickness of excavated material, Fig. 4.8a, is seen to be non-linear for depths of excavated material involving only the soliflucted and cryoturbated layers. The water table established at the site, was below these layers at the start of construction, Fig. 4.5. It is interesting to note that the succeeding linear section of the drawdown plot has a gradient of about unity, which corresponds to a $\bar{B}$ of 0.5 if the drawdown is related to the thickness of weathered and unweathered Gault excavated.

The end of construction drawdown relationship with thickness of excavated material, in contrast, shows a tendency to increase with slope angle, and unlike the immediate drawdown case, the curves do not pass through the origin. This is a consequence of stress readjustment extending to the rear of the northern slopes even though these areas were not directly affected by excavation. It points to the fact that the parameter $\bar{B}$ may not, generally, be suitable for representing the pore water pressure changes along the cutting sections during excavation.
Along the southern slope which is inclined in a direction opposite to the natural ground slope, there was no significant drawdown at the rear of the sections, Fig. 4.7b, presumably because these areas were not affected by any stress readjustment during the excavation of the slope. There is also no significant difference between the immediate and total drawdown relationships with thickness of excavated material, Fig. 4.8b, which are seen to be linear, and tending towards the origin. The average gradient is about unity, which corresponds to a $B$ of 0.5.

(ii) **Pore Water Pressure Prediction.**

Few investigations have been made of the stress redistribution that accompanies excavation of cutting slopes in overconsolidated stiff fissured clays. One of the reasons for this being the limitation introduced by the inelastic behaviour of real soils. Analyses based on elastic methods have shown that stresses around slopes may be large enough to cause local failure even when the overall factor of safety against complete failure of the slope is large, Bishop (1967), Dunlop and Duncan (1970). Once failure has occurred along a significant part of the potential failure surface in a slope, it would be expected that the stress distribution may differ considerably from the elastic stress distribution. Another difficulty arises from the fact that high initial horizontal stresses in over-consolidated clays, Skempton (1961),
may considerably influence the magnitudes of shear stresses after the excavation of a slope, Duncan and Dunlop (1969).

Estimates of the stress conditions in steep slopes have been made by La Rochelle (1960) by means of photo-elastic tests on gelatine models, in which the excavation procedure simulated that at a site where cutting failures had occurred soon after the end of construction. Contours were produced for various slopes including a 1 on 1 slope, relating to the principal stress difference including the influence of gravity, the principal stresses, and the inclination of the major principal stress. The stresses estimated for the 1 on 1 slope, Figs. 4.10, have been used to determine the pore pressure changes during excavation, along section 5N (1 on 1 slope) using the expression by Skempton (1954):

$$\Delta u = B\left[\Delta \sigma_h + A(\Delta \sigma_v - \Delta \sigma_h)\right] \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots
from which it can be seen that a reduction in mean principal stresses will cause a drop in pore water pressure, and the associated shear stress will also lead to a reduction in pore pressure unless $A$ exceeds 0.5. The relationship between the triaxial test $A$ values and principal stress ratio for all zones of the Gault given in Fig. 4.9, shows that the value of $A$ decreases from about 0.5 at low stress levels to 0.26 at failure. It will be seen that for the range of stresses involved in the trial cutting section 5N, only small errors would be involved in assuming a constant value for $A$ of 0.4.

The calculation for change in pore water pressure during excavation is given in Table 4.3. The predicted pore water pressure changes in all cases exceed the observed values with the ratio of observed $\Delta U$ to predicted $\Delta U$ increasing from about 1.6% farthest away from the cutting to about 75% in the vicinity of the cutting slope. This discrepancy may be attributed to the limitations of Eq. (1), and to the calculation of stresses by an elastic method in which the possible existence of high initial horizontal stresses exceeding the initial vertical stresses, are ignored. However, all the indications are that in a plastic analysis, in which the initial lateral stresses are considered, high initial lateral stresses will result in higher values of shear stresses, La Rochelle (1960), Dunlop and Duncan (1970). As a
result a greater drop in pore water pressure during construction, will be predicted by a plastic analysis for the assumed A value of 0.4. The greater discrepancy that is likely to result from the more appropriate plastic analysis points to the probability of the occurrence of some drainage during construction. Some drainage would have occurred during construction provided the insitu Gault has a sufficiently large mass permeability, in which case, the amount of drainage would have varied from a maximum farthest away from the cutting to a minimum at the toe of the cutting. The observed variation in the ratio of observed to predicted pore water pressure, Table 4.3, is consistent with this expected drainage effect.

It has previously been noted that there was hardly any time-lag in piezometric response to excavation, and that during short periods of cessation of site work during dry weather, there was some recovery of pore water pressure at the site. A recovery of about 25% of the total observed drop in pore water pressure, occurred during dry weather in pz. 5N/10, Fig. 4.4d, between 1st and 6th September when there was a general cessation of site work. The occurrence of significant drainage during such a short period clearly points to a sufficiently high mass permeability of the Gault at the site, which would have allowed some drainage during the 8 weeks construction period.
4.3 Long Term Pore Water Pressure

a) Pore Water Pressure in the Weathered and Unweathered Gault.

The maximum water levels recorded in the winter succeeding the trial are shown in Figs. 4.11a, b for the northern and southern slopes respectively. The pore water pressure variation in the northern slope, Fig. 4.11a, is such that at a given distance back from the top of the cutting slopes, the water table is lowest at the rear of the steepest section. However, the water levels ultimately converge on a line about 0.5 m below the ground surface. Along the cutting slopes, the maximum water levels are coincident with the lower slope profile of 1 on 2 and 1 on 2½ slopes, for the steeper 1 on 1½ and 1 on 1 slopes, the water levels are generally below the surface of the slopes.

For the southern slope excavated at 1 on 5, the water levels at the rear of the sections rose to a level about 0.6 m below the ground surface, Fig. 4.11b. This is in contrast to the northern slopes where the effect of slope inclination was reflected in pore pressures for some distance back from the top of the cutting slopes. The water levels along the southern cutting slopes, however, show a similar distribution to that noted on the flatter northern slopes.

b) Maximum Pore Water Pressure in the Soliflucted and Cryoturbated Gault.

The piezometers in these zones of the Gault are mainly confined to the rear of the northern sections.
The maximum water levels recorded in these piezometers are shown in Fig. 4.11a, from which the tendency for the water levels in these zones of the Gault to correspond to separate distributions is clearly discernible. The water levels in the soliflucted Gault are generally higher than those in the weathered and unweathered Gault. Those in the cryoturbated Gault, however, correspond to the lowest water level distribution at the site. By projecting the trends indicated in the soliflucted and cryoturbated layers beyond the likely limit of the residual drawdown zone, the naturally occurring pore water distribution can be determined, Fig. 4.11a.

c) Discussion of Observations.

During the period when maximum pore water pressures were recorded in the trial area, the water level in pz. 3800/1 located in the unweathered Gault but, outside the zone of construction drawdown, attained a level 0.56 m below ground level. This is in good agreement with the level, (0.62 m below ground surface), recorded in the winter preceding the trial. As the water levels measured in pz. 3800/1 were consistent with those in the trial area prior to excavation, it may, therefore, be concluded that the maximum water level of about 0.5 m generally recorded at the rear of the trial sections indicates that steady state conditions were attained in the first winter after the
trial. This is also consistent with the observed maximum water level of 0.6 m below ground level in the piezometers at the rear of the southern slope, where no significant drawdown was recorded during construction. This observation has subsequently also been confirmed by the water level measurements made in the second winter after the trial, which showed no discernible difference from the first winter readings.

The attainment of steady state conditions in one season is contrary to the current supposition that this condition is approached gradually over several seasons. The rapid rise to the long term equilibrium state can therefore, only be attributed to a considerably greater mass permeability of the Gault than is commonly envisaged. Bishop and Bjerrum (1960) stated that the reduction in stress during the excavation in a fissured clay will cause fissures to open up so that the pore water pressure rise to long-term equilibrium conditions will occur more rapidly. Morgenstern (1967) remarked that the presence of a secondary structure in a fissured clay would permit, on unloading, a more rapid distribution of pore water pressure. Duncan and Dunlop (1969) recognised that fissures may help to reduce the effective drainage path within a slope. For, whereas the drainage path in a slope excavated in an intact clay would be of the order of the slope height, in a fissured clay filled
with free water, the length of the drainage path may be reduced to the order of the spacing between fissures. Skempton and Hutchinson (1969) also noted that the interval of time between 'short-term' and 'long-term' conditions, depends on the permeability of the soil.

In addition to the fissure permeability, a number of silt-filled wedges and veins were observed during the excavation of the cuttings, which occurred down to the top 2 m of the unweathered Gault. The influence of such permeable fabric on mass permeability has been discussed by Rowe (1968), (1972), and in Chapter 3 for the adjoining trial embankment site for which the values of field $C_v$ were estimated to range from 600 m$^2$/yr at low effective stresses to about 100 m$^2$/yr under an effective stress of 200 kN/m$^2$ (28.5 p.s.i.). For an effective stress range of up to 125 kN/m$^2$, which is typical of the upper limit of insitu effective pressure involved in the trial cutting, the field $C_v$ determined at the adjoining site was about 220 m$^2$/yr (Fig. 3.13c p. 123). Using this value of $C_v$, and a length of drainage path $H$ equal to the cutting depth ($\approx 8$ m), a calculation based on Terzahi's one-dimensional consolidation theory ($T_v = C_v t/H^2$) shows that 95% equalization (rise to steady state) would occur in 3 months and 10 days. Steady state conditions were actually attained in 3 months and 6 days. It is also interesting to note that a field $C_v$ of 205 m$^2$/yr was
obtained from the field permeability of 0.66 m/yr determined from the response time in piezometers installed in boreholes formed by a rotary method, and the oedometer compressibility $K_v$ of 0.03 m$^2$/MN.

In contrast to these observations, Hutchinson (1971), (1972), concluded from water level observations in deep piezometers in the Cucaracha Formation in the Panama Canal, that the attainment of steady seepage conditions in these deposits is likely to occur over a much longer period than may be imagined from the consolidation characteristics of conventional small laboratory samples.

The equilibrium values of pore water pressure along the trial sections are best determined from the flow net corresponding to steady seepage along each section. The steady state ground water flow patterns in the weathered and unweathered Gault along the northern slopes are shown in Figs. 4.12. A rather surprising observation that can be made from Figs. 4.12 is that the end of construction water levels along each section also closely approximate to a steady state flow line. The divergence from steady state conditions mainly occurs near the toe of each section. The uniformity of the pore water pressure recovery at the rear of the sections after the end of construction ($H_5 - H_2$) may be seen in Table 4.1.

The ground water flow pattern in the northern
slopes was, therefore, approaching a 'drained state' by the time the excavation had reached its final level. The only indications of drainage during construction stemmed from the observation that small quantities of water seepage occurred overnight around the base of the excavation, and the pore water pressure recovery noted in dry weather, during a short cessation of site work. The moisture content of samples taken daily during construction were also consistently about 5% higher than those obtained for samples from BH.3800/1. The borehole was completed within one working day in December 1969 and did not encounter any water. The operation was, therefore, completed in a considerably shorter period than the excavation of the cuttings, and the lower moisture content of the borehole samples suggests that some drainage must have occurred during the excavation of the cuttings. There were also pools of water at the toe on the northern slopes by the end of construction even though there was no rainfall during this period.

It is interesting to speculate if pore water pressure equilibrium existed at the end of construction between the water in the fissures and that within the intact lumps of the clay. This would obviously depend on the micro and macro permeability of the clay. All the available evidence suggests that the macro permeability greatly exceeds the micro permeability. It is, therefore, possible for an out-of-phase condition
to exist within the slopes after pore water pressure equilibrium has been established in the ground water between fissures.

The likelihood of such an occurrence would have a significant effect on the pore water pressures developed in a 'short-term' failure. Where the potential failure path follows the outline of fissures then the fissure permeability will have the dominant effect on the pore water pressure distribution. In the more typical case where failure occurs through fissures and through intact lumps of the clay, intermediate conditions will prevail. In both cases, however, it is unlikely that a truly undrained failure would occur due to the large mass permeability of the Gault.

d) Other General Observations.

The water level in the soliflucted and cryoturbated Gault layers show considerable amplitude of variation which occur more in response to rainfall than with changes in the season. The water level variations in the weathered and unweathered Gault in contrast display far less amplitude with changes occurring gradually with the seasons rather than with rainfall. Similar observations have been noted by Weeks (1970).

There was a general drop in water levels between August - October 1971 i.e. the corresponding period during which the cuttings were excavated in 1970. The water levels in the unweathered Gault beyond the zone of residual drawdown at the rear of the sections
showed a drop of about 0.6 m during this period. This suggests that the natural lowering of 0.5 m assumed in the analysis of the construction pore water pressure data was reasonable.

4.4 'Conventional' Investigation of Cutting Failure

In this section, the cutting failure is investigated on the basis that the pore water pressure and failure surface are unknown. This is normally the case in most of the published case histories on slope failures.

a) History of Instability.

At the end of construction on the 13th October 1970, none of the slopes showed any sign of instability. The first slip occurred after a period of moderate rainfall on the 8th November, i.e. 25 days after the end of construction \( t = 25 \) days; it consisted of a minor surface failure just east of CS. 6 + 00 which was excavated at 1(v) on \( \frac{1}{2}(h) \), Fig. 4.2, Plate 4.1a. Slight movements were observed in the area between the 21st and 23rd November, 1970.

During the next wet spell a new slip occurred on the 8th January, 1971 \( t = 87 \) days which was centred around CS. 6 + 00, Plate 4.1a. This movement was later followed by a major slip which started on the 22nd January \( t = 101 \) days and continued until the 24th. The slide area extended from section 6 + 00 to 4N, Fig. 4.2,
Right: Minor surface failure 8th Nov. 1970

Left: First major slip 8th Jan. 1971

(b)
General view of main slip 22-24 Jan. 1971

(c)
Serrated rear scarp and tension crack in granular Head
Plate 4.1b, involving slopes excavated at between $1(v)$ on $\frac{1}{2}(h)$ and $1(v)$ on $1.5(h)$.

*No major slides have occurred since although there is continued recession of the rear scarp and minor movements occur in the slipped mass.*

**b) Tension Cracks.**

*After the major slip in January a vertical tension crack was exposed extending across the full depth of the granular Head material. In some areas the tension crack was followed by a steeply inclined and slightly curved surface in the soliflucted and cryoturbated Gault. The exposed surfaces in the clay were highly polished and had a radial distribution of striations.*

*The rear scarp of the slip had a serrated outline formed by a series of recesions in the Head material, Plate 4.1c. Each recession formed an almost semi-circular one at the original ground surface which converged radially towards the base of the cryoturbated Gault.*

*The location of tension cracks and the nature of their outline has apparently resulted from the distribution of non-continuous slip surfaces which have been found to display the features described, and a number of which were exposed during construction. It appears that these discontinuities have been utilised by the failure path in the soliflucted and cryoturbated layers of the Gault.*
The shear strength mobilised in the unsheared fissured Gault at failure would depend on the stage at which tension cracks were formed at the rear of the slopes. A likely method of formation of tension cracks is discussed later in the light of the slope indicator data. It will suffice at this stage to state that the removal of top-soil at the rear of the sections after the major slip had occurred, revealed a number of tension cracks located within an area about 1 m back from the edge of the slip. The surfaces of the cracks were separated but did not show any differential vertical displacement. One of the tension cracks exposed was later involved in a retrogressive movement which dislodged slope indicator SI. 5N/5 at the rear of section 5N, Fig. 4.12c. It appears from these observations that tension cracks are formed and remain in existence without the occurrence of failure. Once formed, they will allow a quicker flow of ground water to the Gault and so, probably accelerate the time to failure.

c) **Laboratory Investigations.**

(1) **Index Tests and Moisture Content.**

The results of index tests and moisture content determinations performed during the initial site investigation and during construction are given in the Table overleaf. It can be seen from the Table that the moisture content of the borehole samples decreases with depth below the Head material. The moisture content of samples
Moisture Content and Index Parameters of the Gault at the Trial Cutting Site

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Liquid Limit %</th>
<th>Plastic Limit %</th>
<th>Plasticity Index</th>
<th>B.H. m/c %</th>
<th>Excavation m/c</th>
</tr>
</thead>
<tbody>
<tr>
<td>Head</td>
<td>46</td>
<td>16</td>
<td>30</td>
<td>12 - 20</td>
<td>-</td>
</tr>
<tr>
<td>Soliflucted Gault</td>
<td>87</td>
<td>31</td>
<td>56</td>
<td>32 - 34</td>
<td>37 - 40</td>
</tr>
<tr>
<td>Cryoturbated Gault</td>
<td>89</td>
<td>36</td>
<td>53</td>
<td>34 - 31</td>
<td>35 - 38</td>
</tr>
<tr>
<td>Weathered Gault</td>
<td>84</td>
<td>28</td>
<td>56</td>
<td>31 - 29</td>
<td>33 - 38</td>
</tr>
<tr>
<td>Unweathered Gault</td>
<td>87</td>
<td>32</td>
<td>55</td>
<td>29 - 26</td>
<td>29 - 33</td>
</tr>
</tbody>
</table>

taken during the excavation of the cuttings are about 5% higher than those of the borehole samples. This increase in moisture content has been considered an indication that some drainage occurred during construction.

(ii) Shear Strength Tests.

The peak effective strength parameters for various zones of the Gault were determined from 100 mm diameter samples, tested under consolidated undrained conditions, with measurement of pore water pressure. The test results, which are summarised below, are based on 3 No. samples tested under cell pressures of about 70, 140 and 240 kN/m².

**Effective Stress Parameters**

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>$c'$ kN/m²</th>
<th>$\phi^o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cryoturbated Gault</td>
<td>10</td>
<td>23</td>
</tr>
<tr>
<td>Weathered Gault</td>
<td>13</td>
<td>24.5</td>
</tr>
<tr>
<td>Unweathered Gault</td>
<td>25</td>
<td>18.5</td>
</tr>
</tbody>
</table>

The failure envelope for the unweathered Gault, which is mainly involved in the cutting failure, is shown
in Fig. 4.13a. Further details of the test are also given in Table 4.2, from which it can be seen that the permeabilities of the Gault, determined from the consolidation stage of the tests, vary from $6.8 \times 10^{-11}$ m/s ($2.15 \times 10^{-3}$ m/y) under a cell pressure of 70 kN/m$^2$ to $1.8 \times 10^{-11}$ m/s ($0.57 \times 10^{-3}$ m/y) under a cell pressure of 240 kN/m$^2$. The value of field permeability estimated from the rise in water level in the piezometers installed in rotary cored boreholes, has been shown to be between 200 and 300 times the corresponding laboratory values.

The undrained strength of the Gault was also determined from 100 mm diameter samples, tested so that failure occurred within fifteen minutes. The variation of undrained strength with depth is shown in Fig. 4.14.

All the samples used in the triaxial tests were borehole samples obtained by percussion boring.

d) Total Stress Analysis.

Total stress analyses have been performed on various circular arc failure surfaces for section 5N which was excavated at a slope of 1 on 1. The undrained strength $C_u$ determined from 100 mm diameter samples varied between 78 and 96 kN/m$^2$, and its variation with

*In view of the fissure spacing at the trial site being generally less than 100 mm, only small 'size' errors are likely to result from the use of 100 mm diameter sample for the measurement of shear strength. This point has been discussed in greater detail in Chapter 3.
depth is shown in Fig. 4.14. In the first set of analyses, it was assumed that a tension crack existed through the granular Head layer as was observed, and the following results were obtained:

1) **most critical circle** at:

\[ F = 2.48 \]

allowing a 20% reduction in undrained strength for time effect, and 25% for orientation effect along those parts of the failure surface inclined at \( +5^\circ \)

\[ F = 1.95 \]

assuming that tension crack in granular Head (depth = 25% depth of cut) is filled with water, and allowing for time and orientation effects

\[ F = 1.61 \]

2) **critical circle** through observed points of failure at the toe and at the rear of the slope:

\[ F = 3.38 \]

allowing for time and orientation effects as above

\[ F = 2.70 \]

assuming that tension crack in granular Head (25%) is filled with water and allowing for time and orientation effects

\[ F = 2.00 \]

The depth of tension crack was assumed to equal 50% of the depth of cutting, and the whole procedure was repeated.
iii) Most critical failure surface f.u.b: $F = 2.27$
allowing for time and orientation effects as before: $F = 1.76$
allowing for time and orientation effects as above, and assuming that the tension crack is filled with free water: $F = 0.93$

iv) Critical circle through observed slip surface a.k.b: $F = 2.33$
allowing for time and orientation effects
$F = 1.84$
allowing for time and orientation effects, and assuming that the tension crack is filled with free water: $F = 0.96$

v) The factor of safety is not reduced, simply by increasing the depth of tension crack at the rear of the section, unless it is also assumed that the cracks are filled with free water. Only this condition has been further investigated. The most critical conditions involve a tension crack through point x, at the top of the cutting slope, Fig. 4.14. The following results were obtained:

- Failure surface xy.p.b. (tension crack = 75% depth of cut)
  - allowing for time and orientation effects, with tension crack filled with free water: $F = 0.65$
  - allowing for time and orientation effects, with tension crack half filled with free water: $F = 1.05$
failure surface \( x,y,p,r,b \) (tension crack = 88% depth of cut) allowing for time and orientation effects, with tension crack filled with free water: \( F = 0.59 \)
allowing for time and orientation effects, with tension crack half filled with free water: \( F = 1.23 \)
allowing for time and orientation effects, with tension crack filled with free water to base of granular head: \( F = 0.98 \)

failure surface a,k,l,s,b (tension crack = 88% depth of cut and passing through the observed failure points), allowing for time and orientation effects, with tension crack filled with free water: \( F = 0.75 \)
allowing for time and orientation effects, with tension crack half filled with free water: \( F = 1.42 \)
allowing for time and orientation effects, with tension crack filled with free water to base of granular head: \( F = 1.15 \)

vi) **Summary.**

As was expected, the most critical failure surface lay further back within the slope than the circular arc through the observed failure path. Even after allowing for time and orientation effects, the average shear strength mobilized along the most critical failure surface was only about 55% of the conventional laboratory undrained strength. The scale of fissuring at the trial site was generally less than 100 mm so that this error is unlikely to
be due to size effects as 100 mm diameter samples were used in the laboratory tests. By assuming also that the tension crack was filled with free water, the ratio of field to laboratory undrained strength was still only about 65%.

In contrast to the above results, the discrepancies between field and laboratory undrained strength for 'short-term' failures in deep cuttings in the London Clay at Bradwell, have been attributed to size and time effects, Skempton and La Rochelle (1965), Skempton and Hutchinson (1969). These investigations are discussed in greater detail, later in the Chapter.

The Bradwell analyses were also based on inferred failure surfaces passing through the observed points of failure at the toe, and at the rear of the slopes. In the case of the cutting trial, an even greater discrepancy between the field and laboratory undrained strength results from the analysis of a circular arc through the observed points of failure at the toe and at the top of the cutting slope. After allowing for time and orientation effects, and assuming the tension crack to be filled with free water, the ratio of field to conventional laboratory undrained strength of 100 mm diameter samples, was only about 50%.

It is interesting to note that according to Peck and Lo (1960), under steady seepage conditions,
conventional total stress analysis of an unstable slope in a fissured clay, with a liquidity index of between $\pm 0.1$, which is typical of the Gault at the trial site, would result in a minimum factor of safety of about 3.0. This figure is in close agreement with that obtained for the trial cutting failure.

For a factor of safety of unity to be obtained in stability calculations after allowing for time and orientation effects, it was found necessary to assume a depth of tension crack equal to 50% of the depth of cutting, and which is also filled with free water. Alternatively, the tension should extend to 88% of the depth of cutting which corresponds to about $2L^*/\theta$, and should be filled with free water up to the base of the granular Head.

The location of the tension crack which results in the minimum factor of safety does not, however, coincide with the observed failure path through the granular Head.

e) **Effective Stress Analysis.**

Stability analyses have been carried out for the end of construction condition based on predicted pore water pressures and circular arc failure surfaces. The end of construction pore water pressure

*Laboratory undrained strength corrected for time effect.*
distribution has been obtained from Eq. (1), assuming that the drawdown during construction $\Delta U$ was only 75% of the values resulting from the use of the elastic stresses given in Fig. 4.10*. A ratio of about 75% was obtained from the comparison of the observed and predicted end of construction pore water pressures along the slope of section 5N, Table 4.3.

Using the effective stress parameters obtained for the various zones of the Gault and the calculated pore water pressures, Fig. 4.15, the following results were obtained:

1) **Most critical failure surface c.b (with tension crack across granular Head)**

   Estimated end of construction pore water pressure.

   a) peak effective stress parameters in Gault $F = 1.32$

   b) as for a) above, with tension crack filled with water: $F = 1.08$

   c) triaxial residual strength of $C' = 0$, $\phi' = 13^\circ$ in the cryoturbated Gault and peak effective strength in the other zones $F = 1.20$

   d) as for c) above, with tension crack filled with water: $F = 1.02$

2) **Estimated steady seepage conditions.**

   e) peak effective stress parameters in the Gault: $F = 1.16$

   f) triaxial residual parameters of $C' = 0$, $\phi' = 13^\circ$ in the cryoturbated Gault and peak effective stress parameters in the other zones: $F = 1.08$

\[ \Delta U = \frac{75}{100} \left[ \Delta \sigma_h + 0.4 \left( \Delta \sigma_v - \Delta \sigma_h \right) \right] \]
11) Failure surface a, b (through observed rear scarp of failure)

Estimated end of construction pore water pressure.

- a) Peak effective stress parameters in the Gault \( F = 1.42 \)
- b) As for a) above, with tension crack filled with water: \( F = 1.10 \)
- c) Triaxial residual parameters of \( C' = 0, \phi' = 13^\circ \) in the cryoturbated Gault and peak effective stress parameters in the other zones \( F = 1.35 \)
- d) As for c) above, with tension crack filled with free water: \( F = 1.03 \)

Estimated steady seepage conditions.

- e) Peak effective stress parameters in the Gault \( F = 1.18 \)
- f) Triaxial residual parameters of \( C' = 0, \phi' = 13^\circ \) in the cryoturbated Gault and peak effective stress parameters in the other zones \( F = 1.11 \)

Summary.

The effective stress method of analysis for the 1 on 1 slope gives a better estimate of stability over a wide range of pore water pressure distributions. The minimum factor of safety using peak parameters decreased from 1.32 under zero pore water pressure at the end of construction to 1.16 under assumed steady seepage conditions. Allowing for the fact that part of the failure is considered to have occurred along a pre-existing 'non-continuous' slip surface in the cryoturbated Gault, the value of \( F \) then decreases from 1.20 at the estimated end of construction conditions to 1.08 under the assumed steady seepage conditions. Good estimates of stability also generally result from the assumption that at the end of construction stage...
the tension crack in the granular Head was filled with free water. The nature of the granular Head however, was such that it is unlikely that it would have retained any free water.

The pore water pressure distribution thus, has a relatively minor effect on the calculated factor of safety for this geometry of slope, excavated mainly in previously unsheared, fissured material. A similar deduction can also be made from the effective stress analysis of slopes of similar inclination in the London Clay at Bradwell, La Rochelle (1960), James (1970). La Rochelle (1960) found that using peak parameters*, steady seepage pore water pressure conditions were required for failure to occur in the Turbine House excavation with a slope of $\frac{1}{4}$ on 1. If this cutting had not been backfilled, failure might have occurred during the wet winter period. If this cutting had not been backfilled, the slope remained stable for 4 months but was backfilled just before the onset of the wet winter period. If this cutting had not been backfilled, failure might have occurred during the wet winter period provided steady seepage pore water pressure conditions were approached, a possibility which was recognised by La Rochelle (1960).

*The peak effective stress parameters were measured on 38 mm diameter triaxial and 6 cm shear box samples.
4.5 Detailed Investigation of Cutting Failure

a) Location of Failure Surface.

The failure path along three sections 5 + 00, 5N and 6 + 00 inclined at 1 on 1.2, 1 on 1 and 1 on \( \frac{1}{2} \) respectively, Figs. 4.2, 4.16, were located using a Mackintosh probe. This equipment is not normally suitable for use in stiff clays, however, the original structure of the Gault was completely destroyed during failure. The contorted and softened slipped mass, allowed the slip surface at its lower horizon, which was also the boundary with the underlying stiff insitu clay, to be located without difficulty. The slip surface was also located from hand auger holes, sunk for the installation of piezometers within, and below the slipped mass.

b) Total Stress Analysis.

Analyses have been carried out for the three sections shown in Figs. 4.16, using the undrained strengths determined from conventional tests on 100 mm diameter samples, which are given in Fig. 4.14. In the analyses, the tension crack was only considered to extend across the granular Head at the moment of failure as was observed. The results show that the strength mobilized at failure was only about 30\% of the average laboratory value. Only time effect is likely to introduce any significant errors into the calculations, and even after allowing for this the ratio of field to laboratory undrained strength is still
only about 42°. These results are similar to those previously obtained from circular arc failure surfaces.

(c) Effective Stress Analysis.

1) Pore Water Pressure on the Failure Surface.

The pore water pressures measured in the piezometers along the northern slopes during the period immediately preceding the major slip are given in Table 4.1. They show that there was a general decrease in pore water pressure during this period, although the subsequent failure did not extend beyond section 4N, Fig. 4.2.

The dissipation was most marked in pz. 5N/6 and 5N/8 located near to the failure path. At the rear of the sections the dissipation showed a tendency to decrease with distance back from the top of the cutting. In pz. 4N/1 and 5N/1 located furthest away from the slopes and which are also amongst the deepest piezometers at the site, a slight increase in pore water pressure was recorded just before failure. The pore water pressure gradients would have caused a migration of water to the slip surface.

The dissipation of pore water pressure was temporary and maximum water levels were generally recorded immediately after the failure. The pore water pressure developed on the slip surface at failure is likely to have been within the range of readings taken just before and after failure. The effect of this range of variation of pore water pressure is considered
at the analysis stage.

The pore water pressure acting on the slip surface along section 5N at failure could be determined from the flow net for the section, Fig. 4.12c. Along sections 5 + 00 and 6 + 00 an assessment of the pore water pressure on the slip surface could be made from the fact that the line of seepage varies uniformly across the instrumented sections. For example, at a horizontal distance of 1 m downslope from the top of the cuttings, the level of the line of steady seepage (post failure p.w.p.) along the instrumented sections, Figs. 4.12, and the calculated levels along sections 5 + 00, 5N and 6 + 00, are as follows:

<table>
<thead>
<tr>
<th>Section</th>
<th>3N</th>
<th>4 + 00</th>
<th>4N</th>
<th>5 + 00</th>
<th>5N</th>
<th>6 + 00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Line of seepage O.D. level (m)</td>
<td>73.75</td>
<td>73.35</td>
<td>72.95</td>
<td>72.53</td>
<td>72.10</td>
<td>71.67</td>
</tr>
</tbody>
</table>

By repeating the procedure at other locations along the slopes the line of seepage along sections 5 + 00 and 6 + 00 could be completed.

ii) Effective Stress Analysis (Pre-failure profile).

Analyses have been carried out for sections 5, 5N and 6, Figs. 4.2, 4.16 using the peak parameters determined for the weathered and unweathered Gault and parameters of $c' = 0$, $\phi' = 13^\circ$, in the soliflucted and cryoturbated Gault. It has been stated that the failure path appears to have utilised non-continuous slip surfaces.
existing in these layers. Analyses have been carried out under the following pore water pressure conditions:

(i) p.w.p. determined from seepage line corresponding to water levels recorded 3 days before failure.

(ii) p.w.p. determined from seepage line corresponding to maximum water level recorded after failure.

The results of the analyses are given below:

<table>
<thead>
<tr>
<th>Section</th>
<th>Factor of Safety F</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>p.w.p. Condition (i)</td>
<td>p.w.p. Condition (ii)</td>
</tr>
<tr>
<td>5 + 00</td>
<td>F1 = 1.29</td>
<td>F2 = 1.21</td>
</tr>
<tr>
<td>5N</td>
<td>1.25</td>
<td>1.17</td>
</tr>
<tr>
<td>6 + 00</td>
<td>1.00</td>
<td>0.96</td>
</tr>
</tbody>
</table>

Mass Factor of Safety, $F_{mass}$.

The factor of safety of the whole slide has been determined using the method of end areas.

SECTION ACROSS SLIPPED MASS
The calculation is tabulated below:

<table>
<thead>
<tr>
<th>Section</th>
<th>Area A(m²)</th>
<th>F₁</th>
<th>F₂</th>
<th>F₁ x A</th>
<th>F₂ x A</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 + 00</td>
<td>40.99</td>
<td>1.29</td>
<td>1.21</td>
<td>53.00</td>
<td>49.60</td>
</tr>
<tr>
<td>5N</td>
<td>24.50</td>
<td>1.25</td>
<td>1.17</td>
<td>30.30</td>
<td>28.70</td>
</tr>
<tr>
<td>84.15</td>
<td></td>
<td></td>
<td></td>
<td>102.35</td>
<td>96.20</td>
</tr>
</tbody>
</table>

\[ F_{m1} = \frac{102.35}{84.15} = 1.21 \]

\[ F_{m2} = \frac{96.20}{84.15} = 1.14 \]

The application of peak parameters determined from vertical 100 mm diameter samples along the whole failure surface results in an over-estimate of the factor of safety of 14 to 21%. An examination of the failure surface along the sections analysed, Figs. 4.16, will show that a total of about 34% of the failure path is inclined at angles of between 10 and 25 degrees to the horizontal. A representative shear strength along these sections of the failure surface would be that determined from triaxial test on samples taken such that failure occurs at the appropriate angle across the bedding plane.

After failure, 100 mm diameter samples were taken at 15° to the horizontal at the toe of the stable section of the cutting face. The samples were tested
under consolidated undrained conditions with measurement of pore water pressure, with cell pressures of 35, 70, 140 kN/m² (5, 10, 20 p.s.i.). The results are summarised in Fig. 4.13b from which it can be seen that the shear parameters correspond to:

\[ C' = 11.5 \text{ kN/m}^2 \quad (1.75 \text{ p.s.i.}), \quad \varphi' = 25^\circ \]

The Mohr's circles obtained from 100 mm diameter vertical samples of the unweathered Gault at cell pressures of 70, 140 and 280 kN/m² (i.e. \( C' = 25 \text{ kN/m}^2 \), \( \varphi' = 18.5^\circ \), Fig. 4.13a) are also shown in Fig. 4.13b.

It can be seen from Fig. 4.13b, that within the range of effective normal pressures common to both tests, i.e. less than about 100 kN/m², the 'inclined' samples are of a lower shear strength than the vertical samples. At higher stress levels the reverse appears to be the case. This deduction is however, largely based on the extrapolation of the Mohrs envelope for the 'inclined' samples.

Tests on inclined (45°) 98 mm diameter samples from shallow depths of the blue London Clay at Wraysbury, Marsland (1971), resulted in a curved failure envelope passing through the origin with \( C' = 0 \) and \( \varphi' = 29^\circ \) at very low effective normal stresses, while for normal stresses of around 1000 kN/m², \( \varphi' \) dropped to 14°. The tendency displayed by the London Clay samples at very low normal stresses may well be a feature of other stiff fissured clays;
reference to Fig. 4.13b will show that it is quite possible to draw a failure envelope for the 'inclined' Gault samples which passes through the origin, with an initial inclination of 35° to the horizontal. These observations are in conflict with the results of Bishop et al (1965) based on 38 mm diameter samples of the London Clay from the Ashford Common shaft, from which it was deduced that the samples showed no tendency to behave as a cohesionless material at very low effective stresses. The strength of inclined (45°) 38 mm diameter samples of the London Clay from Wray'sbury, Agarwal (1967) are also considerably greater than the average strength at effective normal stresses below 100 kN/m², obtained from inclined 98 mm diameter samples by Marsland (1971). A proportion of this difference may be due to size effects. The results of Bishop et al (1965), show that the marked change in curvature in passing from the low stress to the high stress range is not exclusive to samples taken at any particular orientation to the bedding.

Marsland concluded from his results that the absence of any cohesion component at the low effective normal stress, commonly involved in natural slope and cutting slope stability problems, implies that any reduction in stability with time will be due entirely to the gradual decrease in the value of $\phi'$; the cohesion value $c'$, which is due to overconsolidation, will then only be of significance in so far as it will
affect the clay between fissures. The results of tests on inclined samples of the Gault from a depth of 8 m below original ground level, Fig. 4.13b, suggest that although a marked curvature of the failure envelope may occur at the very low effective normal stresses, the tendency to show a $C' = 0$, may well be a feature local to the immediate vicinity of the origin.

The lower shear strength resulting from the inclined Gault samples may be partly due to pore water pressure effects. It can be seen from the pore water pressure variations for the vertical and inclined samples, Figs. 4.13a, b, that for the same cell pressures, lower increases in pore water pressure occurred during shearing of the inclined samples. The values of $A'$ given below show that the pore water pressure changes in the inclined samples were only about half of those recorded for the vertical samples under similar cell pressures.

<table>
<thead>
<tr>
<th>Cell Pressure kN/m²</th>
<th>Inclined $A'$</th>
<th>Vertical $A'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>0.144</td>
<td>-</td>
</tr>
<tr>
<td>70</td>
<td>0.157</td>
<td>0.34</td>
</tr>
<tr>
<td>140</td>
<td>0.335</td>
<td>0.60</td>
</tr>
<tr>
<td>280</td>
<td>-</td>
<td>0.80</td>
</tr>
</tbody>
</table>

Only section 5N has been reanalysed, as its factor of safety was previously seen to almost equal that of the mass. The 'inclined' parameters were applied to the sections of the failure surface sloping at between 0 and 25° to the horizontal, and the vertical
parameters were used over the remainder of the failure surface. The following results were obtained:

pore water pressure distribution 3 days before failure:

\[ F = 1.11 \]

pore water pressure distribution just after failure:

\[ F = 1.00 \]

**Average:** \[ F = 1.05 \]

The small error in the average factor of safety could be attributed to the over-estimate of the mass strength by 100 mm x 200 mm samples.

d) **Conclusions.**

Conventional total stress analysis of the failure results in a significant over-estimate of the factor both for the case of the most critical circular arc failure surface and the observed failure surface. Although the large errors were considerably reduced after allowing for time and orientation effects the minimum factor of safety still significantly exceeded unity.

Effective stress analysis based on assumed circular arc failure surfaces and a wide range of pore water pressure distributions, resulted in good estimates of the factor of safety. The 'large' values of the effective cohesion component \( c' \) determined from 100 mm diameter samples of the Gault, made the effective stress analysis insensitive to significant pore water pressure changes. For example, the end of construction condition
was estimated to correspond to zero pore water pressure on the critical failure surface, and the factor of safety $F$ was determined to equal 1.20. Under assumed steady seepage conditions, the factor of safety $F$ only dropped to 1.08. Although the most critical circular arc failure surface lay further back into the slope than the observed failure surface, there is little doubt that an effective stress analysis using parameters determined from 100 mm diameter samples, would have given a good indication of stability, even though significant errors may have been involved in the assumed pore water pressure distribution.

A detailed study of the failure was carried out based on the actual failure path and the pore water pressures measured just before, and just after failure. In view of the fact that some drainage occurred during construction, and the attainment of steady seepage conditions in the winter succeeding the trial period when failure also occurred, the analyses were carried out in terms of effective stress, which is more relevant to the cutting failure, Bishop and Bjerrum (1960). The average factor of safety $F$ of the mass was calculated as 1.18, but after allowing for the orientation effects, established at the appropriate stress level, the value of $F$ dropped to 1.05. The accuracy of the calculated factor of safety confirms the reliability of the effective stress method of analysis provided that reliable shear strength parameters are used in the analysis.
The excavation of a cutting in a clay is generally considered to result in an undrained, out-of-phase pore water pressure condition, Bishop and Bjerrum (1960), Skempton and Hutchinson (1969). However, this was not the case at the end of construction of the trial cutting, where the end of construction pore water pressure distribution was found to correspond to a flow line below the steady seepage flow line, Figs. 4.12. The pore water pressure distribution was, therefore, in a 'drained' state at the end of construction.

These observations are in general agreement with those made for an instrumented vertical 12 m trial cutting in fissured Lacustrine clay, Kwan (1971), which failed under 'drained' pore water pressure conditions 4 days after construction. It was also concluded from the study that the effective stress analysis would have given a good indication of incipient failure conditions at the site. A vertical 10 m cut in overconsolidated fissured Boom clay, De Beer (1969), remained stable for about 7 weeks while studies of a model cutting in the London Clay by Lyndon and Schofield (1970) indicated a time to failure of 12 to 24 weeks. Failure of the model was also accompanied by an increase in moisture content along the slip surface. Rowe (1972) has reported that after the rapid emptying of a flooded Gault clay pit, the sideslope failure fitted an effective
stress analysis with steady seepage pore water pressure conditions and that failure was reactivated later by rainfall. These observations raise some doubts as to the general applicability of the total stress method of analysis to 'short-term' failures. The 'short-term' failures at Bradwell which have been shown to conform to the total stress concept are discussed in the following section.

In the investigation of deep cutting failures in the London Clay at Bradwell, Skempton and La Rochelle (1965) found that the strength mobilized in the field was between 52% and 58% of the average strength determined from 38 mm diameter samples in the conventional undrained test with a time to failure of about 15 mins. The London Clay at the site was excavated at a slope of \( \frac{1}{4} \) on 1 on 1. The slides occurred in an area where a bank had been constructed, just behind the top of the excavation, to a height of between 8 ft (2.44 m) and 11 ft (3.3 m) above original ground level.

The discrepancy arising from the use of the laboratory undrained strength in stability analysis, has been ascribed to size and time effects by Skempton and Hutchinson (1969). However, these factors do not explain the stability of the major part of the excavation which had similar slope inclinations. The only apparent difference between the stable and the unstable areas was the bank which had been constructed just behind the
slopes which were later involved in the failure. This operation would have effectively reduced the pore water drop during excavation, thus giving rise to a more critical distribution of pore water pressure in the adjoining slopes. It is likely that these higher pore water pressures played a part in the initiation of failure. For the complex stress system that would have been involved, an effective stress analysis would seem more appropriate.

Several other deep cuts at Bradwell inclined at \( \frac{1}{3} \) on 1 and 1 on 1 remained stable for at least four months before they were backfilled prior to the onset of the succeeding wet winter period. 

- **Brees (1837)** observed that deep cuttings also excavated at a slope of \( \frac{1}{3} \) on 1 in the London Clay at Euston remained stable for long periods before the construction of retaining walls. These observations further point to the uncharacteristic nature of the failure at Bradwell.

- Effective stress analyses of the slopes at Bradwell using the laboratory peak parameters determined from 38 mm diameter samples and assuming a wide range of pore water pressure distribution, resulted in good estimates of stability, La Rochelle (1960), James (1970). These results suggest that size effects are not involved in the effective stress parameters which were determined from 38 mm samples.

The agreement between field and laboratory undrained strength (and hence the relevance of the total
stress method of analysis) was established for the Bradwell slides after corrections had been applied to the laboratory strength for time and size effects. It was assumed that the mass undrained strength was only 70% of the values determined from 38 mm diameter samples. Ward et al (1965) observed that the undrained strength of 38 mm specimens obtained from block samples of the London Clay from the Ashford Common Shaft were about 30% higher than those from borehole samples. However, at level B where the clay was more highly fissured the ratio of block to borehole strength was approximately unity. At Bradwell there was also no significant difference between the undrained strengths of specimens cut from open drive samples or from block samples, and between 38 mm diameter samples generally and 100 mm samples within the range of depths involved in the failure, Skempton and La Rochelle (1965). This suggests that the brown London Clay which was mainly involved in the slides at Bradwell was closely fissured.

The figure of 70% for the ratio of laboratory to field undrained strength used by Skempton and Hutchinson (1969), is based on published data for tests almost exclusively on the blue London Clay. The results of fissure studies in the London Clay, Skempton et al (1969), suggest that the fissure lumps in the blue London Clay at Wraysbury have a mean size of 56 cm (2.25 in), with a significant number of fissure lumps exceeding the mean size. For the brown London Clay
at Edenware and Apex Corner, however, the fissures were predominantly less than 50 cm in size, with a mean size of about 40 cm. The fissure spacing of the London Clay also increases with depth, and the effect of this variation on the undrained strength determined from various sizes of sample has been demonstrated by Marsland (1967). The sizes of the specimens of the blue London Clay tested by Marsland were 38 mm, 76 mm and 125 mm (1½ in, 3 in, 5 in respectively) in diameter, and the following corresponding strength ratios were obtained at the depths investigated:

7.9 m (26 ft): 100:77:69, fissure spacing 50 - 150 cm
31.4 m (103 ft): 100:77:59, fissure spacing 75 - 406 cm

Similar variations in undrained strength with depth have been published for the London Clay at Hendon by Marsland (1971). Data by Simons (1967), Bishop (1971), for the blue London Clay at Wraysbury, suggest that 100 mm (4 in) diameter samples are large enough to be fully representative of the fissure structure of the clay. It is also clear that the ratio of field to laboratory undrained strength, is a function of fissure spacing, and due to the general reduction in fissure spacing as ground surface is approached, the ratio for the brown London Clay is likely to be greater than that for the blue London Clay. In other words, the
error in mass undrained strength that would result from testing 38 mm diameter samples of the brown London Clay is likely to be less than for the blue London Clay.

Tests on shallow samples of the brown London Clay at Maldon, Bishop and Little (1967), tend to suggest that a size error of about 30% may arise from tests on 38 mm samples, however certain points about these tests need to be considered. In common with Bradwell, there was no significant difference between the undrained strength of 38 mm and 100 mm diameter samples. Large insitu shear box tests, 610 mm sq (2 ft sq) in size, resulted in undrained strengths only 55% of the strength of vertical 38 mm samples, although 38 mm samples taken so that failure in the triaxial test occurred along the horizontal plane, as in the insitu shear box, had an undrained strength of 86% of the vertical 38 mm samples. The difference between the insitu test results and those from 38 mm samples can be attributed to size and orientation effects. A greater degree of anisotropy may well be reflected in the larger insitu test, Bishop (1966), however, the proportion of the drop from 86% to 55% which is attributable to this factor is open to conjecture. The tests at Maldon were carried out at low confining pressures of about half the overburden pressure i.e. about 76 kN/m², Bishop (1971). The
results of tests on jointed rock models by Walker (1972) show that anisotropy is a function of stress level, being more marked under the low normal stresses that are commonly involved in most slope stability problems. The effective stress envelopes for the London Clay, Marsland (1971), and the Gault, Fig. 4.13b, are also considered instructive in this regard, as they show that anisotropy is more marked at the lower levels of stress, and the larger 98 mm inclined samples of the London Clay only had a lower shear strength than those obtained from similarly inclined 38 mm samples, at the low levels of effective normal stress. Significant orientation effects are therefore, likely to be involved in the insitu shear tests at Maldon.

The available evidence at the present time, therefore, suggests that less errors would be involved in the undrained strength of the brown London Clay determined from 38 mm samples, than would be the case for the blue London Clay. The very good agreement between field conditions and the corrected laboratory undrained strength for the Bradwell slides, may be largely fortuitous, as it is based on the apparently erroneous assumption that there is a uniform size effect error in the brown and blue London Clay.
4.6 Post-Failure Stability

a) General.

After failure, hand auger holes were sunk in the slipped mass, along the line of the three sections 5 + 00, 5N, 6 + 00, previously considered. Piezometers were installed in the failure surface and in the underlying material, Figs. 4.16d-f. The softened and saturated state at the rear of the slip masses, aided by a constant supply of water from the soliflucted Gault at the rear scarp, made it impossible for piezometers to be installed towards the rear of the sections.

Movements have occurred in the slipped masses after the piezometers were installed, which in some cases, were induced by the dislodged masses during localised recessions of the rear scarp. Stability analysis could therefore be carried out to determine the average shear parameters along the post-failure slip surface.

b) Pore Water Pressure.

The highest water levels recorded on the failure surface and in the underlying Gault are given Figs. 4.16d-f. It can be seen from the figures that separate water tables existed in the slipped mass and in the Gault below the original cutting floor which the slipped mass now overrides. Despite the several pools of standing water on the surface of the slipped
mass, the water levels in the layer were generally below the surface.

c) Stability Analysis.

Minor movements have occurred during periods of heavy rainfall and so the slipped mass could be regarded as being in a state of limiting stability. The slipped mass along sections 5 + 00, 5N and 6 + 00, Figs. 4.16d–f, have been analysed using the maximum recorded water levels and assuming $C' = 0$ along the failure surface, and the effective angle of shearing resistance $\phi'$, required for a factor of safety of 1.0 are tabulated below:

<table>
<thead>
<tr>
<th>Section</th>
<th>$\phi'$ required</th>
<th>Ht. of rear scarp h</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 + 00</td>
<td>20.3</td>
<td>0.294</td>
<td></td>
</tr>
<tr>
<td>5N</td>
<td>20.0</td>
<td>0.197</td>
<td>$\phi'$ mass = 19.6°</td>
</tr>
<tr>
<td>6 + 00</td>
<td>17.7</td>
<td>0.510</td>
<td></td>
</tr>
</tbody>
</table>

The average shear strength mobilized during post-failure movement is higher than the typical value commonly indicated by linear approximation of the residual envelope for shear box and triaxial tests carried out under relatively high effective normal pressures. It should be noted that the average effective normal stress on the post-failure surface is about 11.2 kN/m² (1.65 lb/in²). The influence of displacement on the
shear strength mobilized under constant stress is also reflected in the results of the analysis. Along section 6 + 00 where failure initially occurred, and which has in consequence been subjected to the greatest lateral displacement, the $\phi'$ required for the reactivation of movement is less than those required along the other sections analysed.

4.7 Slope Deformation

a) General.

In both the effective and total stress analyses on the pre-failure profile, the tacit assumption that a uniform shear strength was mobilized along the failure surface, was made. As a result of the existence of steady state conditions at failure the effective stress method is more appropriate to the analysis of the failure. However, the shear strength mobilized at failure under these conditions would depend on the magnitude of the strains to which various points on the slip surface had been subjected.

A temporary dissipation of pore water pressure was observed in the period immediately preceding failure. The pore water pressure gradients were such that they would have caused a migration of water to the slip surface. Such occurrences in laboratory tests are normally associated with post-peak displacements. Evidence of the migration of water to the slip surface in the field is
also provided by Henkel (1956). However, there were no visible signs of failure in any of the slopes subsequently involved in the slip when the dissipation of pore water pressure was recorded. This raises the important question of the magnitude of the field strains which preceded the final collapse of the slope and how they compare with and relate to those measured in conventional laboratory tests.

The effect of strain on the shear strength mobilised at failure is also reflected in the analysis of the post-failure profiles. The results of these analyses which assume that a uniform shear strength is mobilised on the failure surface, show that lower than average shear strength pertains along section 6 + 00, Fig. 4.16f, which experienced the greatest displacement due to failure.

The slope indicators installed along the northern slopes Figs. 4.2, 4.12, were monitored regularly during and after construction, and the results are discussed.

b) Slope Indicator Data.

The data obtained from the slope indicators are plotted in Figs. 4.17. As no lateral displacement of the natural ground slope was recorded prior to construction, the observed movements are wholly attributable to the excavation of the cutting slopes.

The figures show that the excavation of the cuttings resulted in a horizontal displacement varying from a maximum near the toe of each section to a minimum at the rear. The extent of the zone in which the
excavation of the cuttings resulted in a lateral displacement increased with inclination of slope. For example, a small horizontal displacement was recorded during construction in S1.5N/5 and 5N/2, Fig. 4.17b,c, located at the rear of the 1:1 slope; however, no discernible displacements were recorded during this period in 3L.3N/8, Fig. 4.17e, near the top of the 1:2\(\frac{1}{2}\) slope or in any of the inclinometers at the rear of sections 3N and 4N, Figs. 4.17d,e.

During the period of dry weather following the end of construction, no significant increase in displacement was recorded in any of the inclinometers. Renewed displacements were observed at the onset of wet weather in the winter succeeding the trial. This trend was maintained until failure occurred involving the whole of section 5N and the toe area of section 4N. The rate of deformation along the slopes during the post construction stage continued to vary directly with their inclination.

It can be seen from Figs. 4.17 that there was an acceleration of pre-failure movements just before failure. Records of similar observations in other stiff fissured clay are summarised by Skempton and Hutchinson (1969). The last set of pre-failure slope displacement measurements at the trial made eleven days before the start of failure, show an increased rate of deformation. An even faster rate could be deduced to have occurred just before failure from the measurements made three days after
failure. However, these readings may also reflect the lateral release of pressure which possibly resulted from failure.

The maximum displacements recorded just before failure along section 5N are shown in Fig. 4.16b. They show that the horizontal displacement at each inclinometer location decreased with depth below the existing ground surface. In S1.5N/9 a maximum displacement of 72.5 mm was recorded at the surface of the slope, but the displacement decreased to 20 mm at the level of the potential failure surface. Similar observations can also be made from the data for sections 4N and 3N, along which the displacement was limited to the top 1-2 m layer. Thus the slope displacement not only varied from a maximum at the toe to a minimum at the rear but also with depth at each location on the slope, and this non-uniform displacement would have given rise to a progressive type failure in the slopes. This point is elaborated, later on in the section.

The renewed displacement at the start of the winter of 1970/71 ceased immediately after the occurrence of failure. No significant increase in displacement was measured in any of the slope indicators up till June 1971 after which most of the slope indicators were destroyed by children who found the site an irresistible attraction. The slope indicators still useable continued to show no further displacement when the latest readings were
taken in November 1971.

The downslope displacement up till November, 1971 could, therefore, be summarised as follows:

i) 25-30 mm displacement during construction followed by

ii) a period of dry weather when no further increase occurred

iii) renewed displacement at the onset of wet weather during the winter succeeding the trial; an acceleration of movement preceded the failure along some sections of the slope.

iv) in the succeeding drier period no significant increase in slope deformation occurred.

The above sequence indicates a tendency for post-construction slope deformation to occur in the wetter months of the year. This may be a reflection of the considerable expansion potential of the Gault under reduced stresses in the presence of excess water. The slope deformation would be affected by the release of latent strain energy, and in the unweathered Gault, this would depend on the rate at which diagenetic bonds are destroyed, Bjerrum (1966). It is possible that the destruction of these bonds is accelerated in an environment of excess water.
c) **Significance of Slope Deformation Data.**

(i) **Formation of Tension Cracks.**

The lateral deformation of the slopes would have induced tension forces in the clay varying from a maximum at the toe to a minimum at the rear of the sections. It is unlikely that such forces can be adequately transmitted across the inclined sections of non-continuous slip surfaces which occur across the soliflucted and cryoturbated layers. As a result, the major part of the stresses induced at these discontinuities, would have been transferred to the overlying granular Head material. About 60% of samples of this material was retained on the No. 25 B.S. sieve so that its ability to withstand tensile forces must be comparatively small. The application of tensile stresses to the granular Head would have resulted in a brittle failure (tension crack) following the outline of non-continuous slip surfaces, thus, giving rise to the serrated rear scarp of the slip.

The existence of tension cracks in areas beyond the limit of failure at the rear of the slopes suggests that they were formed prior to the occurrence of failure.

(ii) **Stability Analysis.**

The variation of lateral displacement recorded along section 5N immediately after failure is shown in Fig. 4.18. The lateral deformation at failure in slope indicator 5N/9 which was lost in the slip has been estimated on the assumption that its rate of deformation continued to be at least as fast as that in 51.5N/5. Fig. 4.18 shows that the displacement along the failure
surface in the unsheared fissured Gault varied from a maximum of about 90 mm at the toe to a minimum of about 10 mm at the top of the layer.

A variable shear strength would therefore have been mobilized at failure due to the non-uniform strains along the failure surface. However, the application of laboratory parameters to the analysis of the failure, requires a correlation of the field strains with those measured in the laboratory. An attempt is made to relate the observed field strains with laboratory shearing characteristics of the Gault.

Shear box tests by Morgenstern and Tchalenko (1967) show that the formation of a rupture surface only requires a displacement of about 8 mm. If this is generally the case, then the ring shear test results of Bishop et al (1971) suggest that the shear strength along a rupture surface decreases with increasing displacement. In the ring shear tests on brown London Clay and Weald Clay, the residual strength $\phi'$ decreased with increasing normal stress $\sigma_n'$ and at the low normal stresses commonly involved in slope stability problems in these soils, the ring shear residual strengths were fortuitously similar to the triaxial and shear box values determined at stress levels generally higher than those in the field. No variation in $\phi'$ with stress level was however, noted in the ring shear tests in blue London Clay. The brown London Clay was found to have a higher montmorillonite content than the blue London Clay, and according to
Kenney (1967) a variation in $\phi'$ with stress level is to be expected in soils with a high montmorillonite content, as opposed to soils containing heavy clay minerals. The significance of the higher montmorillonite content in the brown London Clay, is however, speculative.

A method of analysis which attempts to account for the variation in strains along a potential failure surface has been suggested by Bishop (1971b). The method is based on the application of a local residual factor $R_L$ which reflects the proportional drop between peak and residual strength of points along a failure surface. The shear stress $\tau$ at any point is then defined as:

$$\tau = \tau_f - R_L (\tau_f - \tau_r)$$

where $\tau_f$ is the peak effective shear stress and $\tau_r$ is the residual shear stress.

The method, however, requires a proper definition of the peak effective stress and residual envelopes for the range of stresses involved in the failure.

Along section 5N Fig. 416b, the effective normal stress involved in the failure is generally less than 70 KN/M$^2$ (10ps.i.). The peak effective stress envelope for vertical 100 mm diameter samples of the Gault were determined stress levels exceeding 50 KN/M$^2$. The results of tests on 98 mm samples of the London Clay by Marsland (1971) which have already been discussed, suggest that at the very low normal effective stresses the failure envelope in stiff fissured clays may show a
marked curvature, and pass through the origin. The peak effective stress failure envelope of the Gault may therefore be assumed to vary according to the linear relationship representing $C' = 25 \text{ kN/m}^2$ $\phi' = 18.5^\circ$ (see fig. 4.17a) or the envelope may be assumed to curve sharply at the low effective stress levels, to pass through the origin, Fig. 4.19a.

The results of ring shear tests on remoulded Gault clay from the trial site are shown in Figs. 4.19a,b. The tests were carried out in an apparatus manufactured by Armfield Engineering Ltd., and involved shearing the same sample at a series of normal stresses. The apparatus is described by Rowe (1971), and is designed for an annular soil sample confined between two pairs of upper and lower confining rings with an internal radius of 50 mm and an external radius of 75 mm. A 30 mm thick sample is loaded normally through a lever system and sheared by slowly rotating the lower half, while the top section reacts against a tangential load proving ring. The apparatus has been modified to take a second tangential load proving ring. The main disadvantage of the Armfield apparatus, compared to the more refined design by Bishop et al (1971) is that there is no proper confining ring gap control mechanism.

Assuming a uniform distribution of normal stress $\sigma_n$ and shear stress $\tau$ over the slip surface, (this assumption is considered realistic by Bishop et al (1971)):

$$\text{The normal load } N = \sigma_n \pi (R_0^2 - R_i^2)$$
where \( R_0 \) and \( R_1 \) are the external and internal radii.

The torque \( T = \int_{r=R_0}^{R_1} 2\pi r dr = \frac{2\pi \gamma (R_0^2 - R_1^2)}{3} \)

\[ = \frac{3T}{2\pi \gamma (R_0^2 - R_1^2)} \]

\[ \tan \phi' = \frac{3T}{2\pi \gamma (R_0^2 + R_1^2 R_0 + R_0^2)} \]

\[ T = FDA \]

\[ \tan \phi' = \frac{1.59FD}{N} \]

\( F \) = proving ring calibration factor.

\( D \) = proving ring division recorded.

\( a \) = distance proving ring reaction from the axis of the apparatus.

\( a = 102 \text{ mm} \quad R_0 = 75 \text{ mm} \quad R_1 = 50 \text{ mm} \).

Although the test results obtained, Fig. 4.19b, are to some extent consistent with those of Bishop et al. (1971), a more extensive testing programme is required before the reliability of the apparatus can be established. The limited data obtained so far shows that the residual shear strength of the remoulded unweathered Gault varies with stress level.

Section 5\( \mathcal{N} \) has been reanalysed using the observed failure water pressure, and an assumed \( R_L \) distribution based on the inclinometer displacement along the failure path, Fig. 4.18. The displacement on the failure surface is shown plotted with distance from the toe, along the failure surface in Fig. 4.19a. It is assumed that \( R_L \) varies according to the displacement curve from a minimum of zero at the base of the cryo-turbated Gault, to a maximum at the toe of the cutting.
The value of maximum $R_L$ was determined by trial and error until a value was found which resulted in a factor of safety of unity, for any given set of conditions.

Using the ring shear residual data, and the curved peak effective stress envelope passing through the origin, $R_L$ is required to vary between 0 and 0.36 for a factor of safety of unity to be obtained in stability calculations. For the linear peak effective stress envelope ($c' = 25\text{KN/M}^2$, $\phi' = 18.5^\circ$) and the ring shear residual data, $R_L$ is required to vary from 0 to 0.25 for $F$ of unity.

These results suggest that residual conditions were not attained anywhere along the failure surface and that only comparatively small strains were involved in the failure. Assuming uniform conditions at failure, the shear strength mobilized was about 88% of the measured peak strength of vertical 100 mm diameter samples. Allowing for the neglect of orientation and stress level effects, the difference between the measured shear strength and mobilized shear strength does not differ significantly from the average residual factor on the failure surface. By all accounts, therefore, it would appear that the strains involved in the failure were quite small.

The three main criticisms that can be made against this approximate analysis are discussed:

i) The peak parameters measured in the laboratory may be in error. However, a significant increase in
local residual factor $R_L$ requires a higher peak shear strength than that measured in the laboratory. This is contrary to the current supposition that the insitu strength is likely to be lower than the laboratory strength due to the fissure structure of the clay.

ii) The second point is considered in two parts:

a) The existence of a tension crack at the rear of the potential failure path may have caused a propagation of failure in the unweathered Gault from the top of the slope, and

b) A more rapid drop-off in post peak strength ($R_L$ distribution) could have been assumed.

Both these factors would, however, result in a lower estimate of the maximum $R_L$, for a factor of safety of 1.0 to be obtained in the analysis.

iii) The third point stems from the fact that the shear strength was assumed to vary between peak and residual values at the moment of failure. At the point of limiting equilibrium the shear strengths mobilized along the failure surface varied from the pre-peak to the residual value. However, if the insitu stress-strain characteristics of the Gault display a rapid post-peak drop off in strength then the conditions at the moment of collapse would probably have been between the post-peak and residual states. This raises the question of how the point of failure is defined in
a work-softening material such as the Gault. Laboratory investigations quoted by Peck (1967) suggests that the difference between the limiting equilibrium state and point of collapse is insignificant. On the basis of this evidence the assumption that the strength at the actual moment of failure varied between peak and residual values seems reasonable.

It may, therefore, be concluded that the strains measured along section 5N were small compared to those necessary to establish residual shear strength conditions in laboratory and ring shear tests. It is interesting to note that it is estimated from Fig. 4.18, that only about 17% of the failure surface along section 5N was subjected to a displacement exceeding one traverse of the shear box i.e. 60 mm.

4.8 Maidstone By-Pass Cutting Failures

a) Introduction.

The Maidstone By-Pass A.20(M) runs south-east to north-west for approximately five miles between Hollingbourne and Preston Hall, Fig. 4.20. The southern section of the By-Pass crosses an undulating Gault outcrop along which the motorway is routed in a series of embankments and cuttings.

Cuttings up to 15 m (50 ft) in depth were excavated in the Gault generally at an inclination of about 1 (vertical) to 3 (horizontal). In the deep cuttings, a 3 m (10 ft) berm was provided at 6.2 m (20 ft)
above the finished road level. Open ditches with unpaved inverts were excavated along the up-slope side of all cuttings.

The earthworks were completed in 1960 without any major incident during construction. However, some three or four minor slips are reported to have occurred before seeding of the slopes had started. No further movements occurred after the cuttings were trimmed back to approximately their original inclination. The exact locations of these slips were not recorded.

In the winter of 1965, five years after the end of construction, varying degrees of failure occurred in nearly all the Gault cuttings. In the subsequent years movements both of a progressive and retrogressive nature have continually occurred in all the cuttings in the Gault. In all cases failure was observed to start at the toe of the cuttings and progress towards the rear of the sections.

Details of investigations into two cutting failures in the Gault are presented. These are:

1) Gore Wood cutting which is 6.6 m (21.5 ft) deep and represents a typical depth of cut in the Gault, and

2) Longham Wood cutting which is 16.2 m (53 ft) at its deepest section and is also the deepest cutting in the Gault.
b) Gore Wood Cutting (Grid Ref. 805 567).

Gore Wood forms a marked spur through which the By-Pass is routed in 6.2 m of cut. The axis of the spur runs north-east to south-west, almost perpendicularly to the line of the motorway. The natural ground slopes at about $1^\circ$ in this direction although steeper slopes of $3^\circ$ and $4^\circ$ occur towards the north-west and south-east. The valleys adjacent to the spur extend back to Coombe Valleys on the face of the Chalk escarpment.

The first major failure along the By-Pass occurred in the eastern section of the northern slope of this cutting in October 1967, Fig. 4.21. There are strong reasons for believing that one of the end of construction failures mentioned previously also occurred along the eastern part of this cutting. Minor movements have continually occurred in the slipped mass of 1967 and the area of the slip has extended eastwards during subsequent winter periods. In December 1968 a bulge was observed about 1 m above the toe of the southern slope (facing north) and again movement was confined to the eastern end of the cutting. Failure finally occurred along the slope in March 1970 when the toe of the slipped mass was seen to run obliquely across the cutting face. By April 1970 failure had extended to virtually the whole of the south cutting face. A similar westward extension has not, however, occurred in the northern face.
The changes in the profile of cross-section 389B across the northern slope between 1965 and 1970 are shown in Fig. 4.22. The average inclination of the slipped mass is seen to have decreased from 16° in 1965, to 14.5° in 1967, and 12° in 1970. There has also been a corresponding lengthening of the zone around the toe, where a flatter than average slope of about 9° inclination occurs. The profile of the slipped mass also becomes smoother and more regular with time. Similar observations have also been made for natural slopes in the London Clay, Hutchinson (1967), and in the Lias Clay by Chandler (1970c).

1) Investigation of Failure.

The northern face of the cutting has been investigated by means of trial pits Fig. 4.21, in which the failure surface was exposed, plotted, and sampled for laboratory testing. Hand boreholes were also sunk for the installation of piezometers on the failure surface. The main observations from each trial pit are discussed.

Trial Pit (T.P.) 389/1, Fig. 4.23.

This pit was excavated through the 1967 failure, and the whole of the failure surface was exposed. The excavation revealed the existence of a significant thickness of soliflucted Gault extending to a level only about 2 m above the cutting floor level. This corresponds to an original thickness of the soliflucted Gault of 3.5 m (11.5 ft) at the section. The layer consisted of
a very soft remoulded greyish brown clay with chalk pellets and angular flint fragments. It was followed by a layer of firm finely fissured mottled grey brown clay with isolated fissure lumps up to 155 mm (6 in) in size. This layer was recognised as the cryoturbated Gault and its lower horizon was marked by a highly polished and striated sub-horizontal slip surface along which failure had occurred. The slip surface was found to extend beyond the failure zone further back into the slope indicating that it may have pre-existed the failure. A steeply inclined slip surface formed the head slope of the slip thus giving rise to a wedge-shaped failure. Several changes of slopes were observed throughout the length of the failure path.

The soil underlying the sub-horizontal slip surface, was a firm finely fissured blue grey clay with only a slight degree of weathering near the top of the layer.

Trial Pit 389/2, Fig. 4.24.

This pit was excavated 27.5 m (92 ft) west of T.P.389/1, in a stable section of the cutting, Fig. 4.21. The deposits exposed were similar to those in T.P. 389/1 although an expansion in the thickness of the soliflucted Gault of about 2.0 m was observed. The most striking observation made in this pit was that although the toe of the cutting had risen only by about 0.5 m (1.6 ft) above the level at T.P. 389/1, the slip surface was 1.8 m
below the cutting floor level. It also occurred at the base of the cryoturbated layer and as before hardly any weathering was observed in the underlying Gault.

**Trial Pit 389/3, Fig. 4.25.**

The excavation for this pit was made 25 m (82 ft) west of T.P. 389/2 to further investigate the geometry of the slip surface found at the base of the cryoturbated layer in the two previous pits. The base of the cryoturbated layer in this pit was located just below the cutting floor at a level about 2 m (6 ft) higher than that which was observed in T.P. 389/2. The base of the layer was not marked by a continuous slip surface; however, a more distinct slip surface than had hitherto been observed, was exposed at the junction of the solifluxion and cryoturbated layer. A further expansion in the thickness of the soliflucted Gault amounting to 0.50 m was also observed in the pit.

**Trial Pit 389/4, Fig. 4.26.**

The pit was located in the 1968 eastward extension of the failure, Fig. 4.21. The excavation revealed that a shallow mantle slide had occurred along this section of the cutting. The soil mainly involved in the failure was a softened remoulded pale grey brown clay which was seen to be moving on a slip surface running almost parallel to the cutting slope. The
failure was also observed to be utilising non-continuous slip surfaces in the cryoturbated Gault. A total movement into the pit of 15.5 mm (6 in) was recorded at the failure surface in a period of two hours. The pit remained dry for four days except for small seepage emanating from the failure surface.

**Trial Pit 389/5, Fig. 4.27.**

The trial pit was located in the slipped mass of the 1967 failure 13.5 m (44 ft) east of T.P. 389/1. The observations made in this pit are in agreement with those made in T.P. 389/1; in addition some of the features of the structural discontinuities noted previously were seen to occur on an amplified scale. The sub-horizontal discontinuity at the base of the cryoturbated layer displayed more marked changes of slope strongly suggesting that it might be a composite surface of non-continuous slip surfaces. Other non-continuous slip surfaces were exposed within the cryoturbated layer, and the depth of cryoturbation showed considerable variation. A zone of unweathered fissured Gault was located between non-continuous slip surfaces between the northern and eastern faces of the pit.

ii) **Summary of Trial Pit Investigation and Conclusion.**

The investigation of the failure is summarised in the longitudinal section shown in Fig. 4.28, which has been drawn on an exaggerated scale (vertical = 10 x
horizontal) to highlight certain aspects of the investigation which are now discussed.

It can be seen from the figure that stability along the northern cutting slope has been determined by the geometry of a pre-existing discontinuity at the base of the cryoturbated Gault layer. Failure has occurred along the section of the slope situated within a zone in which the intensity of non-continuous slip surfaces has apparently given rise to a continuous rupture surface. The surface is seen to extend over a limited part of the stable area where it is located well below the cutting floor level. Although the zone of cryoturbation extends further along the slope, the excavation located well into the stable area did not reveal any discontinuities in the cryoturbated Gault.

Non-continuous slip surfaces were exposed within the cryoturbated Gault layer in the slipped section of the cutting slope, where they were found to be separated by unsheared, fissured Gault. Failure across this layer of the Gault is therefore, likely to have involved suitably orientated pre-existing discontinuities and previously unsheared material.

The slip surface at the junction of the soliflucted and cryoturbated Gault is likely to be continuous across the whole face; however, a significant exposure was only possible in T.P. 389/3. This was prevented elsewhere by the occurrence of flint fragments,
chalk pellets and tufa nodules which were wedged between the surfaces of the discontinuity. The stability of the cutting face has not been affected by this slip surface; this is partly accounted for by the occurrence of more resistant materials between the surfaces of the discontinuity. Triaxial tests on samples containing the slip surface reported later in the section (p.298) resulted in relatively high shear parameters.

Part of the unstable section of the cutting is associated with the displacement of a softened and remoulded soil mantle. The layer which has lost all trace of the original structure of the clay is based by a failure surface running almost parallel to the cutting slope. Although the failure path has partly utilised pre-existing discontinuities of suitable geometry, the slip surface is considered to be essentially the result of failure of the softened layer. The observed seepage along the slip surface, even though none occurred in the rest of the pit, indicates the possible existence of a perched water table in the layer.

iii) Laboratory Tests.

The geotechnical and shear strength properties determined for samples obtained from the trial pits are summarised in Table 4.4.
iv) Pre-Failure Pore Water Pressure.

The pre-failure pore water pressure data on the potential failure path along section 389B is not accurately known. After the initial movement in 1965, two piezometers, 389/1A and 389/5A, Fig. 4.29, Table 4.5, were installed. Pz. 389/1A was located below the failure surface within the area affected by the 1965 slide. The other piezometer 389/5A was situated outside the slide area and by coincidence it was located close to the potential failure path of the 1967 slide.

A pore water pressure of 0.75 m was recorded in pz. 389/5A, Fig. 4.30a, just before failure in 1967; the maximum pore water pressure on the whole slip surface consistent with this recorded value has been drawn and used in the analysis. A temporary dissipation of pore water pressure occurred prior to the failure which was also recorded in pz. 389/5, Fig. 4.30b, located below the failure surface.

The water levels recorded in pz. 389A/1 and 389A/2 which were also installed before failure in 1967, but in the stable section of the cutting, are shown in Figs. 4.30c, d. By comparing these graphs with those shown in Figs. 4.30a, b, for piezometers located within the area of failure, it will be seen that the piezometer readings for the unweathered Gault show far less amplitude of variation than those in the cryoturbated
Gault. The piezometers in the latter layer were also observed to show a much greater response to rainfall.

These observations suggest that separate water tables exist in each of these zones of the Gault at the cutting site.

v) Post-Failure Pore Water Pressure.

A number of piezometers were installed on the failure surface, Fig. 4.29, and details of the installation are given in Table 4.5. The maximum post-failure piezometric line on the slip surface is shown in Fig. 4.29. It can be seen from the figure that the maximum piezometric level in pz. 389/6 located in the cryoturbated Gault outside the failure area, is inconsistent with the piezometric distribution within the slipped mass.

It appears, therefore, that after failure, a new pore water pressure regime was established in the slipped mass. A similar observation has been made in the discussion of the Otford trial cutting data. It is probable that the formation of a continuous rupture surface at failure which introduces some directional properties in the permeability along the slip surface, effectively results in an isolation of the ground water conditions in the slipped mass.

vi) Pre-Failure Stability.

Cross-section 389B, Fig. 4.29, has been analysed using the following measured parameters:
\[ C^' = 0, \quad \phi^' = 12.7^\circ \text{ for pre-existing slip surfaces.} \]
\[ C^' = 13.8 \text{ kN/m}^2, \quad \phi^' = 24^\circ \text{ for unsheared, fissured cryoturbated Gault.} \]

It should be noted that the latter parameters were determined from samples of the cryoturbated Gault taken outside the unstable area, after failure, and as a result, reflect the effect of softening on the peak strength. Under these conditions a factor of safety \( F = 1.0 \) is obtained if 61.4% of the head slope, which is located in the cryoturbated Gault, utilises pre-existing discontinuities. This proportion of pre-existing slip surfaces in the cryoturbated Gault compares well with the 67.5% calculated for the foundation of the Otford trial embankment.

Assuming that a uniform shear strength is mobilized on the failure surface and that \( C^' = 0 \), the required average \( \phi^' \) for \( F = 1 \) is 24.0\(^\circ\). This result indicates erroneously that the fully softened strength, Skempton (1970), applies to the initial failure condition; it further demonstrates that factors involved in the stability of a cutting can be concealed by the use of average parameters. The fully softened strength concept is discussed later in the Chapter.

vii) Post-Failure Stability.

After the failure of 1967 no further major reactivitation of the slip has occurred; minor movements have, however, been observed indicating that the
slipped mass may be of marginal stability. An analysis has been carried out assuming that the length of the headslope (38.5%) along which peak parameters were mobilised in the pre-failure analysis now mobilises shear parameters of:

$$c' = 0 \text{ and } \phi' = 24^\circ$$

Using the maximum recorded pore water pressure on the failure path, the factor of safety of the slipped mass was calculated to be equal to 0.97. The average shear parameters along the failure surface for F of 1.0, correspond to $$c' = 0, \phi' = 17^\circ$$.

c) Longham Wood Cutting.

i) Introduction.

Longham Wood forms a very marked spur, Fig. 4.31, extending across virtually the whole of the Gault outcrop. It is separated from the more moderate Gore Wood spur by a flat bottomed valley emanating from a Coombe Valley in the Chalk escarpment. The axis of the spur runs perpendicularly to the line of the motorway and natural ground slopes of between 1 and 2 degrees occur in this direction. Locally, dips of up to 5 degrees occur to the north-west and south-east.

The By-Pass is routed in a deep cutting through the spur with a maximum depth of 16.2 m (53 ft). Failure was first observed at the toe of the cutting in the winter of 1966 and successive slips have subsequently occurred further up the slope giving rise to step-like
features. There was widespread failure in December 1968, involving the whole of the lower section of the cutting. Failure was also observed in a side road cutting at Water Lane, Fig. 4.31, which runs at the foot of the spur. There has not been any major movement since 1968 although minor movements continually occur in the slipped mass. A slight bulging of the toe of the south cutting face was observed in April 1970, but had not shown any significant increase up till January 1972.

ii) Investigation of the Failure.

After the major movement along the northern slope in December 1968 two trial pits 39/1 and 39/2 were excavated at the toe of the slipped mass, Fig. 4.32. The observations made from these excavations and the subsequent investigation instituted are discussed.

**Trial Pit 39/1, 39/2, Figs. 4.33, 34.**

A softened remoulded brown clay showing no trace of the original structure of the Gault from which it was derived, was found mantling the slope. The base of the layer was marked by a highly polished and striated slip surface on which some movement was taking place. The failure surface was inclined at an angle almost parallel to the cutting slope.

This layer was followed by a stiff fissured medium grey clay with heavy iron-staining and scattered shell fragments recognised as weathered Gault. Its
lower level was marked by a poorly polished and striated discontinuity which was utilised by the failure of the softened Gault layer around the toe of the cutting. The discontinuity was almost horizontal in T.P. 39/2, Fig. 4.34, whereas it had a dip of 6½ degrees towards the north-west in T.P. 39/1, Fig. 4.33. Its level was seen to increase by about 0.5 m in the 46 m distance from T.P. 39/2 to T.P. 39/1.

A change in colour and in the degree of weathering of the clay was observed across the discontinuity. The underlying zone consisted of a hard dark grey clay with slight ironstaining and many shell fragments.

In both excavations failure was restricted to the softened Gault layer at the base of which small amounts of seepage were observed. The pits, however, remained dry for seven days after the excavation in June 1969.

After the trial pit investigation it was decided to further explore the extent of the softened layer and to locate the level of the discontinuity in the weathered Gault further along the upper parts of the slope. A number of boreholes were sunk along and at the rear of the cutting slope, Fig. 4.32, from some of which continuous samples were taken for inspection.

Piezometers were installed in all the boreholes and, in addition, two Soils Instrument Ltd., (SII) slope indicators were installed along the slope and a third
was located at the rear of the section. Details of the piezometer and slope indicator installations are given in Table 4.6 and in Fig. 4.35.

The inspection of borehole samples revealed the existence of a softened layer along the whole length of the slope. The extent of the layer is shown in Fig. 4.35 from which it can be seen that separate failures have occurred along the slopes on either side of the berm. The slipped mass from the upper slope appears to have overridden the top section of the lower slope. The extent of softening is greater in the upper slope presumably because the already partly remoulded soliflucted and cryoturbated Gault is more susceptible to breakdown.

A succession of cryoturbated, weathered and unweathered Gault was established along the slope, the extent of each of which is shown in Fig. 4.35. A layer of solifluxion material was also found at the rear of the slope. Steeply inclined slip surfaces, characteristic of non-continuous slip surfaces, were generally found in the cryoturbated layer, and poorly developed slip surfaces marked the boundaries of the soliflucted Gault layer.

The discontinuity in the weathered Gault was found extending for at least 34 m beyond the top of the cutting. It was at all times located in the vicinity of the colour change from medium to dark grey clay which also coincided with a change in the degree of weathering.
Locating the discontinuity became more difficult on moving upwards along the slope and, whereas it was easily located in the trial pits and boreholes near the toe of the cutting, at the rear, the surfaces of the discontinuity were tightly locked against each other.

Studies of the ammonite subzones exposed during the excavation of this cutting, Owens (1960), indicate that the boundary between the Lower and Upper Gault occurs around the toe of the cutting. The Lower Gault is usually a dark grey clay whilst the Upper Gault is pale grey in colour, and one of the most marked lithological changes in the Gault strata occurs at their interface. It is around such boundaries that tectonic discontinuities would have formed during the Oligocene movement. In addition, differential release of lateral stresses would also tend to occur at such boundaries when several beds of the Gault are exposed, for example, along the limbs of a marked spur. It is possible that this discontinuity is of tectonic origin and that its distinctiveness around the toe of the cutting may be due to the differential release of horizontal stresses across it.

iii) Pore Water Pressure.

The pore water pressure data obtained from all the piezometers are summarised in Table 4.6 and the variation of maximum pore water pressure with depth at
various locations on the slope are shown in Fig. 4.35. The perched nature of the water tables in the softened Gault and in the solifluxion and cryoturbated layers at the site is clearly discernible.

The water levels in piezometers in the softened Gault and in the solifluxion deposits generally show marked amplitudes of variation which were observed to correlate with wet and dry weather. Typical examples of water levels in these layers are given in Figs. 4.36a, b. The cryoturbated Gault shows a less variable pore water pressure pattern, and although the amplitude of the variation is quite large, changes seem to occur more gradually, Fig. 4.36c.

Along the slope, the pore water pressure in the weathered Gault decreases with depth down to the level of the discontinuity. Below this level, the pore water pressure shows a tendency to increase with depth although the increase does not occur immediately below the discontinuity. It seems, therefore, that the discontinuity in the weathered Gault is acting as a drainage path within the cutting.

A typical example of the pore water pressure variation in the weathered Gault at the rear of slope is shown in Fig. 4.36d. After the attainment of equilibrium conditions, which occurred in about 6 months, the water level has remained almost constant. There is evidence, however, suggesting that the first
set of piezometers installed at the rear of the cutting, pz. 39/13, 14 and 15, Table 4.6, were interfered with before they, and all other piezometers subsequently installed at the rear of the cutting slope, were concealed by fitting them with covers. The maximum water levels recorded in the three piezometers in the weathered and unweathered Gault are, as a result, not considered reliable.

iv) Stability of the Cutting: General Consideration.

Measurements of horizontal displacement in the slope indicators, Fig. 4.35, indicate that failure has been limited to the softened Gault layer, which mantles the cutting slope. Maximum displacements at the ground surface of 13.6 mm and 6.0 mm have been recorded in SI.38/19 and SI.39/17 respectively. The corresponding displacement across the slip surface at the base of the layer has been very small. The slope indicator measurements confirm the observation made during the trial pit investigation that the deep discontinuity in weathered Gault is only involved in the failure in so far as it is being utilised by the mantle slide around the toe of the cutting.

Laboratory Tests.

Only index tests and clay fraction determinations have been carried out on samples from the cutting. The results, given in Table 4.7, are generally similar to those obtained for the Gore Wood cutting samples, Table 4.4.
Pre-Failure Stability.

Stability analysis is limited by the lack of pre-failure pore water pressure data. This is crucial in shallow slides which generally show marked sensitivity to pore water pressure variation. The analysis carried out for the 'upper slope' is complicated by being located in the soliflucted and cryoturbated layers, as a result of which failure would have occurred partly along pre-existing slip surfaces. The analysis of the Gore Wood cutting failure has shown that pre-existing slip surfaces are likely to form about 60% of the length of the failure path in the cryoturbated Gault.

Stability analyses have been carried out on the assumption that the piezometric line coincided with the original slope profile.

**Lower Slope:** A factor of safety $F = 1.0$ requires average shear parameters along the failure surface of:

\[
\begin{align*}
C' &= 1.1 \text{ kN/m}^2 (23 \text{ lb/ft}^2), \quad \phi' = 24^\circ \\
S' &= 2.7 \text{ kN/m}^2 (56.5 \text{ lb/ft}^2), \quad \phi' = 24^\circ
\end{align*}
\]

**Upper Slope:** A factor of safety of 1.0 requires average shear parameters along the failure path of:

\[
\begin{align*}
C' &= 2.7 \text{ kN/m}^2 (56.5 \text{ lb/ft}^2), \quad \phi' = 24^\circ \\
S' &= 2.7 \text{ kN/m}^2 (56.5 \text{ lb/ft}^2), \quad \phi' = 24^\circ
\end{align*}
\]

This analysis does not take into account the pre-existing slip surfaces in the upper slope. Assuming that these slip surfaces have shear strength
parameters of $C^i = 0$, $\phi^i = 12.7^\circ$ and that they account
for 61.4% of the failure path, with peak effective
strength parameters of $C^i = 13.8 \text{kN/m}^2$, $\phi^i = 24^\circ$,
mobilised along the remainder of its length, stability
analysis results in a factor of safety $F = 0.96$.

v) Post-Failure Profile.

The post-failure masses have been considered
to be moving as a single mass as it is no longer
possible to distinguish between the slipped masses
along the slope. Using the observed maximum pore
water pressure on the slip surface and assuming $C^i = 0$,
then the average $\phi^i$ required for a factor of safety of
1 is $22.5^\circ$.

d) Summary of Maidstone By-Pass Investigation.

The assumption that a uniform shear strength
is mobilized along the failure surfaces of the slips
analysed leads to the erroneous conclusion that the
strength at failure corresponds closely to the fully
softened value. The analysis of the Gore Wood cutting
and the upper slope mantle slide in the Longham Wood
cutting, both of which partly involve pre-existing
slip surfaces, show that factors affecting stability
can be concealed by the use of average parameters.

In the lower mantle slide failure at Longham
Wood excess pore water pressure induced by the over-
riding slipped mass from the upper slope are likely
to have been involved in the failure. That such a
situation can arise has been demonstrated by Hutchinson et al (1971). As a result of these excess pore pressures the cohesion mobilized at failure in the lower slope would have been much greater than the calculated value. Failure in this section of the slope is also considered to be affected by stress release which is also thought to account for the initiation of failure at the toe of the cutting.

4.9 Conclusions

a) Pore Water Pressure.

The pore water pressure measurements at the trial cutting site and along the Maidstone By-Pass have shown that there are perched water tables in the soliflucted and cryoturbated Gault layers. The water levels recorded in piezometers in these layers show wide amplitudes of variation which occur mainly in response to rainfall rather than with changes in the seasons of the year. The changes in the cryoturbated layer, however, occur more gradually.

The water table in the 'undisturbed' Gault i.e. in the weathered and unweathered Gault, shows comparatively less amplitude of variation. The changes in level of the water table in these layers, occur more in accordance with changes in the seasons of the year rather than with rainfall.

There was a lowering of pore water pressure commensurate with the excavation of the trial cutting;
however, at the end of construction the pore water pressure distribution corresponded to a steady state flow line. It may therefore be concluded that an out-of-phase or undrained state did not exist at the end of construction. The fully drained, steady seepage state had not been attained either, although equilibrium (drained) conditions were in existence. Steady seepage conditions were subsequently established in the first winter after the trial. The attainment of this condition in one season is consistent with the high insitu permeability of the Gault which was found to be considerably higher than is commonly envisaged. Part of the large insitu permeability is considered to be due to the opening of fissures during the excavation of the cutting, and to the existence of a permeable fabric in the form of silt filled veins and wedges in the Gault at the trial site.

Failure along the cutting slopes was generally preceded by a temporary dissipation of pore water pressure which was most marked along the potential failure surface. The pore pressure dissipation was such that a migration of pore water would have occurred towards the potential failure surface.

After failure a new pore water pressure regime was established in the slipped mass which was found to be independent of all the pre-existing distributions. The ground water in the slipped mass behaved in a similar manner to that in the solifluxion layer, showing marked response to rainfall.
b) **Initial Slope Failure.**

The stability of slides in over-consolidated fissured clays involving previously unsheared material, i.e. 'first-time' slides, Skempton and Hutchinson (1969), has been considered by Skempton (1970), and found to accord well with the critical state concept of Schofield and Wroth (1968). Skempton argues that it follows from the softening process of Terzaghi (1936), that over-consolidated fissured clays will ultimately be reduced, essentially, to a remoulded normally consolidated, fully softened state. As only comparatively small magnitudes of strain were considered to accompany such slides in the London Clay, the progressive failure mechanism was not considered to be of significance in such slides. It is suggested that the fully softened strength which approximates to $C_s' = 0$ and $\phi_s' = \text{peak } \phi'$ can be regarded as the limit of strength reduction in 'first-time' slides. As the concept has evolved exclusively from a consideration of London Clay slopes, Skempton (1970) advocates caution in its application to other fissured clays.

An alternative view has been put forward by Bishop (1971) that 'first-time' slides are examples of progressive failure. It is argued that at the moment of limiting equilibrium the strength along a failure surface will vary from the peak value to a value at or approaching the residual value at the point where local overstress initiates failure. According to Bishop (1971),
instead of considering the average condition (Skempton 1964), the proportional drop in strength from the peak to the residual strength at any point along the failure surface may be represented by a local residual factor $R_L$. There is at present no reliable method of estimating the $R_L$ distribution in the field. However, from the stress-strain behaviour observed in ring shear tests and published data by De Beer (1969), Bishop was able to establish reasonable distributions of $R_L$ for the 'first-time' slides analysed by Skempton (1964), (1970), which gave results in good agreement with those obtained by the fully softened strength concept.

The analyses of slides in the Gault show that the average parameters mobilized at failure vary between the peak and the fully softened value depending, amongst other things, on the geometry of the slope.

The results of the analyses for the Otford trial sections which were inclined at between 1(v) on 0.6(h) to 1(v) on 1.2(h) show that peak effective stress parameters determined from vertical 100 mm samples result in an average over-estimate of factor of safety of 18%. By applying the relevant shear parameters to the section of the failure path inclined at less than 25° to the horizontal, the error in the calculated factor of safety is reduced to only 5%.

The excavation of the cuttings did not take place under undrained conditions, and conventional total stress analysis, as is to be expected, results in a considerable over-estimate of the factor of safety.
The investigation of cutting failures along the Maidstone By-Pass, originally inclined at 1(v) on 3(h), shows that instability arises from either or a combination of softening, and the existence of slip surfaces in the soliflucted and cryoturbated layers. Analyses of the cuttings assuming a uniform mobilization of shear strength at failure indicate erroneously that the average shear strength corresponds to the fully softened value. The investigation of the failures established that a proportion of the failure surface in the soliflucted and cryoturbated layers comprises of non-continuous slip surfaces which are separated by unsheared fissured material. Analyses which take these factors into account, show that about 60% of the length of failure surfaces in these layers utilise pre-existing slip surfaces if the shear strength along these discontinuities is assumed to correspond to the triaxial residual value. The remaining 40% or so of the failure path mobilized peak effective stress parameters of the softened Gault, which was found by triaxial tests to be higher than the fully softened value.

The effect of progressive failure is not accounted for in the discussion of the analyses of the Otford and Maidstone By-Pass cuttings. The slope indicator data for the Otford cuttings established the existence of non-uniform strains varying with depth at a given location, and decreasing from a maximum at
the toe, to a minimum at the rear of the sections. The shear strength at failure would, therefore, have varied in accordance with the stress-strain characteristics of the Gault. There is at present no quantitative method of relating field and laboratory strains; however, the magnitude of the error in stability analysis based on peak effective stress parameters, suggest that only small strains were involved in the failure.

This observation is generally supported by an approximate analysis which attempts to allow for the progressive nature of the failure. The results of analyses of the Maidstone By-Pass cuttings in which the effect of progressive failure is neglected also suggest that the mechanism could not have played a significant part in the failures.

Thus the shear strength mobilized by 'first-time' slides in the Gault varies with the geometry of the slope, the pore water pressure distribution on the failure surface and the rate of softening of the Gault. For steep slopes the shear strength across unsheared fissured material lies close to the 'unsoftened' peak, and for the slacker 1:3 slopes the value is about 65% of the 'unsoftened' value for the cases investigated.

The observed reduction in strength of the Gault has been attributed to softening; however, the softening process involves changes in volume (deform-
ation) which may not occur uniformly within the slope. The softening process cannot therefore be considered independent of the progressive failure mechanism. It appears that softening only involves comparatively small strains. Bishop and Lovenbury (1969) have shown that strains in clays are significantly time dependent, and hence the point of limiting equilibrium will also be time-dependent. The significance of progressive failure may, therefore, increase with the age of the slide.

c) Stability of Post Failure Profile.

A common feature of all the slipped masses analysed involving previously unsheared material in the initial failure, is that the average effective normal stress on the failure surface is generally less than 14 kN/m² (2 p.s.i.). The average shear strength mobilized during movement of the slipped masses along the slip surfaces formed during the initial failure, was consistently found to be higher than typical values indicated by linear approximations of failure envelopes determined from shear box and triaxial tests carried out under stresses in excess of those acting on the slip surfaces in the field.

*Nearly all the slides in the London Clay considered by Skempton (1970), Bishop (1971), were over 15 years in age.*
Bishop et al. (1971) have shown that residual parameters may vary with stress level, and the curvature often displayed around the origin by the failure envelope of triaxial and shear box residual tests, may be a reflection of this factor. The average shear strength mobilized during the reinitiation of movement of the slipped masses was about 1.25 times the ring shear residual strength at the appropriate stress level, Fig. 4.19b. This is probably a reflection of the fact that either residual conditions had not generally been attained along the failure surface or that irregularities occur along the failure surface in the field unlike that in the ring shear test. Weeks (1972, Forthcoming Publication) has also found a ratio of about 1.25 between field residual strength and the laboratory ring shear values. It is interesting to note that the case histories considered by Weeks involve slip surfaces formed, probably, between 500 and 60,000 years ago.
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<th>DISTANCE OF FZ. FROM TOP OF CUTTING</th>
<th>GEOLOGY AT FZ. TIP</th>
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**SECTION 3N**

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| 13D | 74.561 74.231 1.27 | 6.070 3.91 | UG 3.64 3.30 1.42 0.34 2.20 1.12 0.153 3.65 3.50 4.45 3.03 |
| 14 | 74.250 74.270 2.98 | 1.640 9.67 | UG 1.55 3.30 |
| 14D | 74.100 74.260 3.84 | 8.331 4.19 | UG 6.25 4.20 2.20 2.00 4.05 3.58 0.494 3.90 3.45 4.29 2.09 |
| 15S | 73.840 68.100 5.74 | 1.640 18.02 | UG 1.49 2.84 |
| 15D | 73.750 67.380 6.39 | 7.850 1.505 19.73 | UG 1.51 |
| 11 | 73.426 65.490 7.94 | 12.141 4.215 25.84 | UG 10.67 4.02 4.02 6.65 6.65 6.15 1.000 4.20 4.20 4.20 0 |

**SECTION 4**

| 4/15 | 7.82 0 1.870 -0.30 | CG 0.80 0.80 |
| 1D | 7.35 0 2.080 | UG 1.71 1.71 |
| 4/26 | 75.752 0.973 | 2.000 -2.40 | UG 1.47 1.47 |
| 4/4D | 71.251 2.650 | 1.620 6.04 | UG 4.10 4.10 |
| 4/3D | 69.857 4.440 | 1.500 10.43 | UG 1.60 1.60 |
| 4/4D | 67.861 6.240 | 1.615 14.34 | UG 4.54 4.54 |

Initial Site F.Z. (Control)

H1 = Initial P.N.P.
H2 = P.N.P. on attainment of final level above FZ.
H3 = P.N.P. at end of construction

H4 = Max. P.N.P. before failure
H5 = Max. P.N.P. after failure
H6, H7, H8 = P.N.P. 4 days before failure

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H₁ = Initial P.W.P.  
H₂ = P.W.P. on attainment of final level above PZ.  
H₃ = P.W.P. at end of construction  
H₄ = Max. P.W.P. before failure (F)  
H₅ = Max. P.W.P. after failure (F)  
H₆ = Max. P.W.P. before failure (F + 11 days)  
H₇ = Max. P.W.P. after failure (F + 11 days)  
H₈ = Max. P.W.P. before failure (F + 11 days)  
H₉ = Max. P.W.P. after failure (F + 11 days)  
H₁₀ = Max. P.W.P. before failure (F + 11 days)  
H₁₁ = Max. P.W.P. after failure (F + 11 days)  
H₁₂ = Max. P.W.P. before failure (F + 11 days)  
H₁₃ = Max. P.W.P. after failure (F + 11 days)  

Table 4.1
<table>
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<th>Section</th>
<th>Initial P.W.P</th>
<th>Final P.W.P</th>
<th>P.W.P. at End of Construction</th>
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<td>4.13 0.28</td>
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<td>2.92</td>
<td>4.59 1.47</td>
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**Table 4.1**

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<th>Borehole</th>
<th>Initial P.W.P.</th>
<th>Final P.W.P.</th>
<th>P.W.P. at End of Construction</th>
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<tr>
<td>48</td>
<td>12.1 0.90</td>
<td>5.5 1.25</td>
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<td>2.8</td>
<td>4.13 0.28</td>
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<td>1.52 1.52</td>
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<tr>
<td>52</td>
<td>2.29</td>
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</tr>
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**Notes:**
- **H**<sub>i</sub> = Initial P.W.P.
- **H**<sub>2</sub> = P.W.P. on attainment of final level above P.W.
- **H**<sub>3</sub> = P.W.P. at end of construction
- **H**<sub>4</sub> = Max. P.W.P. before failure
- **H**<sub>5</sub> = Max. P.W.P. after failure
- **H**<sub>6</sub> = P.W.P. 11 days before failure
- **H**<sub>7</sub> = P.W.P. 11 days after failure
- **H**<sub>8</sub> = Max. P.W.P. after 11 days

**Table 4.1** (continued)
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<th>Stage</th>
<th>Specimen</th>
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<th>B</th>
<th>C</th>
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<td><strong>Initial</strong></td>
<td>Moisture Content %</td>
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<td>27</td>
<td>27</td>
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<tr>
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<td>Bulk Density Tonne/m³</td>
<td>2.00</td>
<td>1.99</td>
<td>1.96</td>
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<tr>
<td><strong>Saturation</strong></td>
<td>B Value</td>
<td>0.971</td>
<td>0.971</td>
<td>0.980</td>
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<tr>
<td><strong>Consolidation</strong></td>
<td>Effective pressure</td>
<td>70</td>
<td>140</td>
<td>260</td>
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<tr>
<td></td>
<td>Coefficient of Consolidation ( C ) m²/yr</td>
<td>8.920</td>
<td>4.230</td>
<td>3.916</td>
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<tr>
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<td>&quot; Volume Compressibility ( N_v ) m²/KN</td>
<td>0.0245</td>
<td>0.0201</td>
<td>0.0147</td>
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<tr>
<td></td>
<td>Permeability (calculated) m/s</td>
<td>( 6.8 \times 10^{-11} )</td>
<td>( 2.6 \times 10^{-11} )</td>
<td>( 1.8 \times 10^{-11} )</td>
</tr>
<tr>
<td><strong>After</strong></td>
<td>Moisture Content %</td>
<td>25</td>
<td>25</td>
<td>24</td>
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<tr>
<td><strong>Consolidation</strong></td>
<td>Density Tonne/m³</td>
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<td>2.02</td>
<td>2.01</td>
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<td>Rate of Strain % per hr.</td>
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<td>0.1</td>
<td>0.1</td>
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<td>Failure Strain %</td>
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<td>4.3</td>
<td>5.6</td>
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<tr>
<td></td>
<td>&quot; Major Effective Principal Stress ( \sigma_1 ) KN/m²</td>
<td>148</td>
<td>190</td>
<td>311</td>
</tr>
<tr>
<td></td>
<td>&quot; Minor &quot; ( \sigma_2 ) KN/m²</td>
<td>38</td>
<td>65</td>
<td>130</td>
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<tr>
<td></td>
<td>&quot; ( (\sigma_1 - \sigma_2') ) KN/m²</td>
<td>110</td>
<td>127</td>
<td>181</td>
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<tr>
<td></td>
<td>Pore Water Pressure Increase ( \Delta W ) KN/m²</td>
<td>32</td>
<td>77</td>
<td>150</td>
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<tr>
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<td>Failure A value ( \Delta u/(\sigma_1 - \sigma_2') )</td>
<td>0.290</td>
<td>0.605</td>
<td>0.830</td>
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</table>

**TABLE 4.2**

RESULTS OF CONSOLIDATED UNDRAINED TESTS ON THE GAULT FROM TRIAL CUTTING SITE

Sample Location: B.H. 3200/1 5 - 6 m below ground level.
Unweathered stiff grey fissured Gault.
Size : 105 mm x 210 mm (vertical)
<table>
<thead>
<tr>
<th>No.</th>
<th>Initial $\sigma_v$ (kN/m²)</th>
<th>$\Delta \sigma_v$</th>
<th>End of Construction $\sigma_v$ (kN/m²)</th>
<th>$\Delta \sigma_h$</th>
<th>$\Delta (\sigma_v - \sigma_h)$</th>
<th>Pred. $\Delta u$ (kN/m²)</th>
<th>Obs. $\Delta u$ (kN/m²)</th>
<th>Remarks</th>
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</thead>
<tbody>
<tr>
<td>5N/1</td>
<td>250</td>
<td>-50</td>
<td>200</td>
<td>-117</td>
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<td>90.2</td>
<td>14.2</td>
<td>0.157</td>
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<tr>
<td>3</td>
<td>165</td>
<td>-18</td>
<td>147</td>
<td>-87</td>
<td>27.5</td>
<td>59.5</td>
<td>18.5</td>
<td>0.310</td>
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<tr>
<td>4</td>
<td>108</td>
<td>-45</td>
<td>93</td>
<td>93</td>
<td>19.2</td>
<td>73.8</td>
<td>22.0</td>
<td>0.297</td>
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<tr>
<td>6</td>
<td>76</td>
<td>-31</td>
<td>65</td>
<td>34</td>
<td>13.5</td>
<td>&gt;11.6</td>
<td>&gt;0.378</td>
<td>Water level below pz. level at end of construction.</td>
</tr>
<tr>
<td>7</td>
<td>150</td>
<td>-63</td>
<td>116</td>
<td>63</td>
<td>21.2</td>
<td>45.8</td>
<td>35.0</td>
<td>0.76</td>
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<tr>
<td>10</td>
<td>200</td>
<td>-49</td>
<td>151</td>
<td>79</td>
<td>31.6</td>
<td>86.4</td>
<td>62.5</td>
<td>0.72</td>
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Note: Pz. 5N/8 installed after the end of construction.
<table>
<thead>
<tr>
<th>Trial Pit No.</th>
<th>Description of Sample Location</th>
<th>m/c%</th>
<th>Index Properties</th>
<th>Clay Fraction</th>
<th>Specific Gravity S.G.</th>
<th>Shear Parameters</th>
<th>Remarks</th>
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</thead>
<tbody>
<tr>
<td>389/1</td>
<td>Soliflucted Gault</td>
<td>32</td>
<td>92 29 63</td>
<td>61</td>
<td>2.65</td>
<td>$C'_s = 13.8$</td>
<td>38 mm Txl. Test</td>
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<td>Cryoturbated &quot;</td>
<td>32</td>
<td>70 23 47</td>
<td>51</td>
<td>2.61</td>
<td>$\theta'_s = 24$</td>
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<tr>
<td></td>
<td>Unweathered &quot;</td>
<td>31</td>
<td>72 24 56</td>
<td>60</td>
<td>2.64</td>
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<tr>
<td></td>
<td>0.3 m above slip surface</td>
<td>42</td>
<td>42</td>
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<td>Slip surface</td>
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<td>389/3</td>
<td>0.3 m below slip surface</td>
<td>33</td>
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<td>Unweathered Gault</td>
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<td>Gault Clay with slip surface</td>
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<td>38 mm Txl. Test</td>
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<th>Index Properties</th>
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<th>Specific Gravity S.G.</th>
<th>Shear Parameters</th>
<th>Remarks</th>
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<td>Geology At Pz. Tip</td>
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**R3G**: Remoulded, Softened Gault  
**VG**: Weathered Gault  
**SG**: Soliflucted  
**U3**: Unweathered  
**C3**: Cryoturbated  
**H**: Head
<table>
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<th>Liquid Limit %</th>
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<th>Plasticity Index %</th>
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Initial Pore Water Pressure Distribution
At Trial Cutting Site

Pore Water Pressure Head $H_i(m)$

- Granitic Head
- Soil-Placed Gault
- Cydrated Gault
- Weathered Gault
- Unweathered Gault

- Section 3N (1:26) Northern SLOpes
- 4N (1:18) Northern SLOpes
- 5N (1:1) Northern SLOpes
- Section 4A,B,C Southern SLOpes
- 5S Southern SLOpes (1:5)

Depth of P below G.L (m)

Fig 4.5
Fig. 4.3

End of construction/Dehydration of the_uninhabited_Captain_Northern Slopes (in"of the" limit)
Relationship between Immediate and Total Drawdown in the Unweathered Gault and Thickness of Excavated Material (Northern Slopes)

Immediate and Total Drawdown (m)

Total Drawdown

- Section 3N
- 4N
- 5N

Immediate Drawdown

+ Section 3N (1:2½)
- 4N (1:1½)
X 5N (1:1)

Thickness of Excavated Material ΔH(m)
Relationship between Immediate and Total Drawdown in the Unweathered Gault and Thickness of Excavated Material

(Southern Slopes)
Slope 1m: 5m

![Graph showing the relationship between Immediate and Total Drawdown against Thickness of Excavated Material AH (m)].

- Immediate Drawdown
- Total Drawdown
- Sections AA, BB, CC: Immediate Drawdown
- Total Drawdown

Fig 4. 2b
Effective Stress Ratio \(\frac{\gamma'}{\gamma}\) vs. Parameter A

- Laboratory Complex
- Underwater Complex

\[ \frac{\gamma'}{\gamma} \] values:
- 2.15
- 1.9
- 1.6
- 1.2
- 0.8
- 0.6
- 0.4
- 0.2
- 0.1

Variation of Embayment Pressure Parameters with Stress Ratio

(For 100mm diameter samples)
Contours of \((\delta \nu - \delta_n) / \delta H\) Determined from Photo-Elastic Tests with Gelatine Model

(From La Rochelle 1960)

(Scale 1:100)
CONTOURS OF $6v/8H$ (From La Rochelle 1960)
(Scale 1:100)
Maximum Water Levels Along the Northern Slopes

Distance of % from top of the cutting slope line

Ground Level

Sedimentary Gault (sa)

Cretaceous Gault (co)

Weathered Gault (wa)

Unweathered Gault (ua)

Pore Water Pressure Distribution
at 7.7m along the rear of the cuttings

Fig 4.11b
Piezometric Data Otford Trial Cutting

Section 4100

Fig 4.12d

- Head H
- Softbacked Gault SS
- Gypsum Halved Gault CG
- Weathered Gault W
- Unweathered Gault UG

- Max. Water Level
  (All piezometers installed after end of construction)
Stability of Otford Trial Cutting
Section 5+00

Fig 4.16a

Piezo line in Cryotorbated Gault
Piezo line in Solifluxeated Gault

Head H

Solifluxeated Gault SG
Cryotorbated Gault CG
Weathered Gault WG

Unweathered Gault UG

Max piezo line on Slip Surface
Piezo line on Slip Surface 3 days before failure

Slip Surface
Stability of Otford Trial Cutting

Section 5N

Fig 4.16b

Key to Slope Deflection
Stability of Otford Trial Cutting

Section 6400

Fig 4.16c

Head H
Soliflucated Gault SG
Gyrotrated Gault CG
Weathered Gault WG
Unweathered Gault UG

Piezo line in Gyroturbated Gault
Piezo line in Soliflucated Gault

Max piezo line on Slip Surface
300 pump on Slip Surface
3 days before failure

Slip Surface
Final Slope Level attained at SI

End of Construction

Partial Failure Surface

Failure (22/1/71)

HORIZONTAL DISPLACEMENT OF S15N/9

Time - Since Start of Construction (Weeks)

August
September
October
November
December
January 1971

Final Slope Level

HORIZONTAL DISPLACEMENT (mm)
Horizontal Displacement (mm)

Time - Since Start of Construction (Weeks)

End of Construction

Failure (22/1/71)

Fig 4.17c
VARIATION OF LOCAL RESIDUAL FACTOR ON SLIP SURFACE
ALONG SECTION 5N AND SHEAR STRENGTH OF THE GAVLTT

[Graph showing variation of residual factor and shear strength along section 5N]
Residual (b. c.)

Fig. 4-195

From Trial Cutting Site At Oregon

On Fronted Aggregate Unweathered Fault

Reynolds Jumbo Shear Test Results
PLAN OF MAIDSTONE BYPASS CORE WOOD CUTTING
SHOWING TRIAL PITS AND PIEZOMETER LOCATIONS

Fig 4.21
MAIDSTONE BYPASS

VARIATION IN SLOPE PROFILE AT CROSS-SECTION 389

Fig 4.22

(Note: after each slide toe of slipped mass was cut to back of verge)
MAIDSTONE BYPASS

LONGITUDINAL SECTION THROUGH GORE WOOD

SHOWING THE PROJECTED OUTLINE OF TRIAL PITS AND SLIPS

Fig 4.28

(Note: vertical scale = 10 x horizontal scale)
MAIDSTONE BY-PASS PIEZOMETER READINGS

Pz. No. 39/2
d
(Softened Gravel)
5.1 Engineering Significance of the Geology of the Gault

The engineering properties of the Gault are dependent on a number of inter-related factors arising out of its sedimentary and post depositional history. The most important of these include:

a) the variable lithology and mineralogy of the strata,

b) the directional properties arising from bedding, one-dimensional consolidation by the Chalk, tectonism, and macrostructural discontinuities such as fissures and any permeable fabric in the mass,

c) the considerable expansion potential under reduced stresses and in the presence of excess water,

d) the influence of Pleistocene periglacial conditions in the shallow layers.

The geotechnical properties are largely determined by a), b), and c), and as such, they are liable to show considerable variation. The relevance of laboratory parameters in design would depend on the extent to which the samples tested, and the conditions
of the test, reflect the conditions obtaining in the field.

Perhaps the most important single factor affecting the engineering properties of the Gault, is the influence of Pleistocene periglacial conditions, under which slip surfaces discontinuities are thought to have been formed in the shallow layers of the strata, which are likely to be mainly involved in most Civil Engineering projects. Two main types of slip surfaces, principal and non-continuous, have been encountered, associated with three main categories of movement along slopes.

i) The first category of movement mainly involves essentially clayey deposits in slopes inclined at, or near to their angle of ultimate stability against landsliding. For these slopes, pore water pressures no higher than those occurring with a water table at ground surface, are necessary for the re-initiation of movement on the principal slip surfaces which are usually almost planar, running almost parallel to the ground surface.

ii) Principal slip surface discontinuities have also been found associated with sheet movements composed mainly of granular material on slopes flatter than their angle of ultimate stability. Their origin has been ascribed to the development of artesian pore water pressures during the Pleistocene period.
iii) The third category of movement involves essentially clayey deposits in slopes flatter than their angle of ultimate stability against landsliding. It is necessary to distinguish between two classes of movements. Class a) involves the nearly planar slip surfaces running almost parallel to the ground surface found in laminar clays in which the macro-structures are almost orientated parallel to the ground surface; and b) comprises the non-continuous slip surfaces separated by unsheared fissured material found in clays in which the orientation of 'structures' are not related to the ground surface.

5.2 Pore Water Pressure Distribution

a) Natural Slopes.

In addition to the water table in the 'undisturbed' Gault i.e. weathered and unweathered zones, perched water table conditions may develop in the soli-flucted and cryoturbated layers. The perched water tables display considerable amplitudes of variation, with changes occurring more in response to rainfall than with changes in the seasons of the year. The changes in level of the water table in the undisturbed Gault show comparatively less amplitude of variation, and tend to occur gradually with the seasons of the year. Perched water table conditions have also been recognised in similar circumstances by Weeks (1970) who also demonstrated that failure to recognise this
condition in unstable solifluxion mantles can lead to incorrect estimates of shear strength, mobilized along pre-existing slip surfaces.

b) Cutting Slopes.

The incidence of a permeable fabric in the Gault, together with the probable opening of fissures during the excavation of the cuttings resulted in a high mass permeability, which allowed some drainage to occur during the construction of the trial cutting. Estimates of the field permeability based on the equilization time in rotary drilled boreholes, were about three hundred times the laboratory value. The percussive method of boring resulted in a much longer pore water pressure equilization period in piezometers due probably to the greater degree of disturbance and stress adjustment time-lag involved in this method of boring. The estimated permeability was still about one hundred times the laboratory value.

The high insitu permeability of Gault also resulted in the establishment of an almost 'drained' state by the end of the eight week period of construction, and the attainment of steady seepage conditions in about fifteen weeks after construction, in the winter succeeding the trial. Bishop and Bjerrum (1960) have suggested that the reduction in stresses during the excavation of a cutting in a fissured clay, will result in the
opening of fissures, and hence, increase the mass permeability, thereby allowing the pore water pressure rise to the long term equilibrium conditions to occur more rapidly. From the pore water pressure measurements made during the excavation of a test trench in the stiff fissured crust of a marine clay, Dibiagio and Bjerrum (1957) observed that undrained conditions were in existence for only a very short period after excavation. De Beer (1969) concluded from studies of steep slopes in overconsolidated Boom clay that lateral deformation of the slopes would have opened up fissures, thereby increasing the mass permeability and thus, the rate of pore water pressure readjustment to steady seepage conditions. For steep slopes excavated in previously unsheared material De Beer (loc. cit.), also concluded that estimates of stability should be made assuming steady seepage pore water pressure conditions and peak effective stress parameters. The observed and suggested period of pore water pressure readjustment in these cases is considerably shorter than is implied by Bishop and Bjerrum (1960).

In contrast to these observations, Muir Wood (1971), Hutchinson (1971), (1972), have considered the incidence of pore water pressures below sea level in piezometers at Folkestone Warren, and in the Culebra Cut of the Panama Canal, as indicating that the period of pore water pressure redistribution on a potential
failure surface may be considerably longer than has been implied, (Bishop and Bjerrum 1960), while for insitu material at great depth, the period of pore pressure readjustment may be measured in hundreds or even thousands of years. The coefficient of swelling for the lower part of the Guacaracha Formation is reported by Hutchinson (1972), to be about 0.1 m²/y, compared to a value of about 200 m²/y estimated for the Gault at the trial site. Thus, the insitu permeability of a clay is the dominant factor controlling the period of pore water pressure readjustment in a cutting.

Bishop and Bjerrum (1960) recognised that the main difference between the shearing characteristics of sand and clay lies in the very wide difference, about one million times, in permeability. They further stated unless construction in a clay is very slow or the clay contains permeable layers, little dissipation of pore water pressure will occur during the period of construction. Skempton and Hutchinson (1969) stated that in the more permeable soils such as sands and gravels, the period of pore water pressure readjustment will be very short and, except under conditions of transient loading, stability problems will fall into the long-term category. They also stated that in clays, however, particularly if they are intact, the mass permeability is so low that the period of pore pressure readjustment may last for months or years after the excavation of the cutting, and that in such soils,
it will be necessary to decide whether to categorise slope stability problems as long-term, short-term or intermediate, depending on the stage in the period of the pore water pressure readjustment. Thus, while the role of permeability in pore water pressure readjustment is fully appreciated, its measurement has unfortunately been based mainly on laboratory tests on small samples, which can lead to a gross underestimate of field values. Rowe (1968) observed that the real drainage behaviour of a deposit depends on the geological details of its formation, and the presence of a permeable fabric in the form of silt veins and other inclusions can transform the mass permeability compared to that of small samples.

The long-term pore water pressure distribution at Otford was seen to approach the slope profile thus, resulting in an ideal condition for softening and swelling to occur in the surface layers where the confining stresses are low. The surface layers of the trial cutting slopes completely lost all trace of their original fissured structure within three months after the onset of the wet weather which led to the establishment of steady state conditions. The fissured structure was reduced to a crumbled mass which however, retained the original colour of each zone exposed on the cutting face.

Along the Maidstone By-Pass with cutting slope inclination of 1(v) on 3(h), instability is associated in part, with a shallow layer, showing a more advanced
stage of softening. The thickness of the layer is generally greater than at Otford, and in addition, there has been a change in the origin colour of the soils exposed along the cutting face. A perched water table condition also develops in the layer during periods of rainfall when movement may also be initiated.

It appears from the foregoing observations, together with the results of observational studies of railway cuttings, that slopes in the Gault steeper than about 1(v) on 4(h) will be liable to a rapid deterioration of the surface layers unless extensive drainage measures are undertaken. This would require that the maximum water levels be kept below the surface layer; it would also be necessary to limit the ingress of surface water.

5.3 Stability of Earthworks

a) General Consideration.

In a classical paper on the total stress method of analysis, Skempton (1948b) in considering the limitations of the method stated that "\( \phi_u = 0 \) only when there is no water content change. The rate of consolidation of clays is so small that in most cases the change in water content during construction is negligible. Therefore the \( \phi_u = 0 \) analysis applies from the point of view almost exactly to the conditions obtaining immediately after construction". From a reconsideration of several case histories, Bishop and Bjerrum (1960) drew attention to the significance of stress path and distinguished
between an increase in stress level such as is involved in footings and embankments, from excavations which involve a reduction in stress. A distinction was also made between intact clays, for which the total stress method was found to apply to end of construction failures generally, and fissured clays, which did not show such a similarly consistent picture. For fissured clays, the total stress method was found to be adequate for analysing end of construction failures involving loading, while a considerable over-estimate of safety factor may result in the case of cutting failures. Subsequent to the review by Bishop and Bjerrum (1960), several case histories have been published dealing with embankments on soft clays where the conventional undrained strength, results in a significant over-estimate of safety factor, for example Parry (1968), (1971). Bjerrum et al (1972) found that the difference between conventional undrained strength and field strength increases with the plasticity of the clay and attributed the discrepancy to the effect of anisotropy and rate of loading. Rowe (1967) considers it necessary to further distinguish between essentially homogeneous clays which exhibit no mass permeability, from those which have a macroscopic fabric which increases their mass permeability by 10 to 10,000 times higher than the laboratory value measured on small samples. This recommendation is supported by the observations at the trial sites. It is also not altogether surprising that in cuttings, where the decrease in load can lead to an expansion of any permeable seams, and so allow a greater
volume of water content change to occur during construction, the total stress method may over-estimate the end of construction safety factor, while during loading permeable seams are closed or become constricted thus producing the opposite effect to the case of unloading.

5.4 Trial Embankment

a) Pore Water Pressure.

Some drainage occurred during the construction of the embankment and thus was shown to be consistent with the high insitu permeability of the Gault foundation. Estimates of insitu consolidation characteristics based on insitu permeability tests, and laboratory compressibility, and on observed dissipation rates, showed a decrease from about 600 m$^2$/y under the insitu effective overburden stress, to about 100 m$^2$/y under the completed embankment of about 11 m height. The high insitu coefficient of consolidation, especially at the low stress levels, is thought to be due to the presence of a permeable fabric in the Gault foundation. The average ratio of observed to predicted pore water pressures using the pore water pressure parameters A and B and elastic estimates of stress increments in the foundation, was generally less than 0.5 throughout construction.
b) **Total Stress Analysis.**

The main observations from total stress stability calculations are listed below:

i) It follows from the foregoing comments that the failure state of the embankment did not conform to the requirements of the total stress method of analysis. Use of the method in the analysis of circular arc failure surfaces however, resulted in a reasonable estimate of the factor of safety after correcting the conventional 100 mm diameter laboratory undrained strength for time and orientation effects.

ii) This agreement is considered largely fortuitous as, apart from the fact that the most critical circular arc did not coincide with the actual failure surface which was non-circular, the analysis also neglected the observed mode of failure.

iii) Using the vertical 100 mm undrained strength, corrected for time and orientation effects, analysis for the actual failure surface resulted in a safety factor of 1.57, compared to 1.12 obtained for the most critical circular arc.

iv) Both the pre-construction and post-failure investigations established the existence of non-continuous slip surfaces separated by unsheared fissured material in the Gault foundation. Failure would, therefore, have partly utilised pre-existing non-continuous slip surfaces for part of its length in the foundation with the remainder occurring through unsheared fissured material.
v) Assuming that for the actual failure surface, 70% of the failure path in the foundation utilised pre-existing slip surfaces, resulted in a factor of safety of 1.30 after the undrained strengths had been corrected for time and orientation effects. It will be seen later that this undrained factor of safety is more consistent with the observed mode of failure.

c) Effective Stress Analysis.

i) The effective stress method is more appropriate for the analysis of the embankment failure due to the occurrence of drainage during construction.

ii) Analyses based on circular arc failure surfaces and an assumed $\bar{F}$ of 0.5 in the fill and foundation materials, and using the peak effective strength of vertical 100 mm laboratory samples, resulted in a minimum safety factor of 1.19.

iii) Again this reasonable agreement has been obtained by neglecting the observed mode of failure and the effect of anistropy on the effective strength.

d) Detailed Investigation of Failure.

i) Apart from the existence of non-continuous slip surfaces in the foundation material, failure was first observed around the toe of the embankment, involving the displacement of a 'wedge' of soil which gave rise to a vertical tension crack in the fill material.
Subsequently, more 'wedges' were displaced leaving vertical tension cracks in the fill which extended to about the centre of the slope, before the upper section of the embankment slumped behind the displaced blocks, leaving a polished and striated rear scarp.

ii) Effective stress analysis for the observed initial failure path showed that the initiation of failure around the toe resulted from the fact that pre-existing slip surfaces were utilised along the entire length of the slip surface in the foundation.

iii) The uppermost 'wedge' displaced was also found to have utilised pre-existing slip surfaces along 90% of its length in the foundation material.

iv) For the whole embankment, the failure path in the foundation was found to consist of about 67% pre-existing slip surfaces with the remainder occurring through previously unsheared fissured cryoturbated Gault mobilizing peak effective strength.

v) Stability of embankments could therefore be ensured by reducing the risk of failure around the toe. This could be achieved by destroying the slip surfaces in the foundation which are likely to be utilised at failure, by digging out the top 3 m layer or so, and backfilling with the remoulded excavated material.

vi) The procedure up till now in Kent has been to replace the foundation material below the full length of the embankment slope. With the exception
of high embankments and cases where a restriction is placed on side slopes, the remoulded excavated material has been reused as backfill.

vii) In the light of the trial results, replacement will now only be necessary over a limited length of foundation in the vicinity of the toe of most embankments, provided typical side slopes previously recommended are adhered to.

5.5 Trial Cutting and Maidstone By-pass Investigations

a) Pore Water Pressure.

It has been shown that some drainage occurred during the excavation of the trial cutting due to the large insitu permeability of the Gault at the site, and that by the end of construction an almost 'drained' state had been established. Long-term equilibrium conditions were also attained in the winter succeeding the trial. The rise to steady seepage state is normally considered to occur over several seasons, and in some cases may take place over thousands of years depending on the insitu permeability of the strata, Bishop and Bjerrum (1960), Skempton and Hutchinson (1969), Wood (1971), Hutchinson (1971), (1972). Its attainment in one season at Otford is consistent with the estimated permeability of the Gault at the site which is considerably larger than is commonly envisaged from laboratory tests on small samples, due to the presence of a permeable fabric in the mass.
b) Total Stress Analysis.

i) The shear strength mobilized at failure along the most critical circular arc failure surface was only between 55 and 60% of the measured 100 mm diameter undrained strength, after corrections had been applied for time and orientation effects.

ii) For a safety factor of unity to be obtained in analysis, it was necessary to assume a depth of tension crack filled with free water and extending to 50% of the depth of cutting. Alternatively it was required that the tension crack should extend to $2C^*/\gamma$, (where $C^*$ is the undrained strength corrected for time effect), and should be filled with free water up to the base of the granular Head. The failure paths corresponding to a safety factor of unity do not however bear any relationship to that which was observed.

iii) Total stress analyses on observed failure surfaces showed that the undrained strength mobilized at failure was only about 50% of the laboratory 100 mm undrained strength, after corrections for time and orientation effects.

c) Effective Stress Analysis.

i) Analyses of circular arc failure surfaces using the peak effective strengths determined from 100 mm diameter samples, gave a better reflection of stability over a wide range of assumed pore water pressure distribution.
ii) The minimum safety factor decreased from 1.32 for zero pore water pressure to 1.16 under assumed steady seepage conditions.

iii) Allowing for the fact that failure is considered to have occurred along pre-existing non-continuous slip surfaces in the soliflucted and cryoturbated Gault resulted in a safety factor of 1.20 under zero pore pressure and 1.08 under assumed steady seepage conditions.

iv) It appears that for slopes steeper than about 1 on 1, excavated in unsheared fissured Gault, estimates of stability are not significantly affected by large variations in pore water pressure. The effective stress method could, therefore, be used to obtain reasonable estimates of construction stability in slopes with such geometry.

d) Detailed Investigation of Trial Cutting and Maidstone By-Pass Failures.

Trial Cutting.

i) As long term steady seepage conditions had been attained at failure, the effective stress method of analysis is appropriate for the analysis of the slides.

ii) Analyses based on observed failure surfaces which were found to be non-circular, and the measured peak effective strength determined from vertical 100 mm diameter samples resulted in an average mass safety factor of 1.18.
iii) Failure around the toe of the cutting involved low effective normal stresses on a failure surface inclined at between about 0 and 25° to bedding. The average shear strength of 100 mm diameter samples taken such that failure in the triaxial test occurred at about 15° to the bedding, was lower than that for the vertical samples under the low effective normal stresses which occur around the toe of the cutting. After allowing for this, the factor of safety was reduced from 1.18 to 1.05.

iv) The magnitude of the error in the calculated safety factor, coupled with the results of an approximate analysis in which the measured non-uniform strains along the failure surface were taken into account, suggest that progressive failure could not have played a significant part in the failure.

v) This observation together with those of Bishop and Lovenbury (1969) suggest that the significance of progressive failure in slopes may be a direct function of age of slope.

Maidstone By-Pass.

vi) In contrast to the Otford cuttings which were inclined at between about 1 on 1 and 1(v) on 1/3(h), the Maidstone By-Pass cutting failures involved slopes of a much flatter inclination of 1(v) on 3(h).
vii) Failure in these slopes was found to have arisen from either, or a combination of, softening and the occurrence of pre-existing slip surfaces in the soliflucted and cryoturbated Gault layers.

viii) The first major slip occurred in a part of a cutting where pre-existing slip surfaces had formed a continuous rupture surface outcropping along the toe of the slope.

ix) Elsewhere along the same cutting and at other sites along the By-Pass, failure was associated with a softened layer which had lost all trace of the original colour and structure of the Gault from which it was derived. The softened mass was based by a slip surface running almost parallel to the cutting slope.

x) Analyses of the cutting failures, assuming a uniform mobilization of shear strength along the failure surface, indicated erroneously that the shear strength corresponded to the fully softened value.

xi) Failure in the soliflucted and cryoturbated layers is likely to have utilised pre-existing slip surfaces along part of its length and, taking this factor into account, analyses showed that about 60% of the failure path is required to have occurred along pre-existing slip surfaces, with the remaining 40% or so occurring through softened but previously unsheared material.
xii) The proportions of pre-existing slip surfaces and softened unsheared material are in reasonable agreement with those determined from the trial embankment failure. Factors involved in the stability of slopes can therefore be concealed by the use of average parameters along the failure surface.

e) **General Comments.**

The shear strength mobilized by 'first-time' slides in the Gault, therefore, varies with the geometry of the slope, the pore water pressure distribution, the distribution of pre-existing slip surfaces, and the rate of softening of the Gault. Peak effective strength will be mobilized in steep slopes excavated in previously unsheared material, while in flatter slopes the unsheared material may suffer a reduction in strength due to softening. Failure in most slopes is likely to be a combination of shearing through pre-existing slip surfaces and through fissured material and the neglect of this factor can lead to inaccurate estimates of mobilized shear strength.

The softening process is considered a part of the progressive failure mechanism but which, however, only requires small strains. It appears that softening is mainly responsible for the occurrence of failure in Gault slopes steeper than about 1(v) on 4(h) when the failure occurs.
within a period of about 10 years. In such cases, failure has not been deep seated, but has been found to be restricted to the surface layers.

5.6 Post-Failure Movement

A feature common to both the Otford and Maidstone By-Pass slipped masses is that the average effective normal stress on the failure surfaces is generally less than 14 kN/m$^2$ (2 p.s.i.). Analyses assuming zero cohesion, showed that for a reinitiation of movement, the shear strengths mobilised along the failure surfaces considerably exceeded the typical triaxial and shear box residual values for the Gault. It should be noted, however, that triaxial and shear box tests to determine the residual strength, have mainly been carried out at stress levels greatly in excess of those involved in the post-failure movement. The preliminary results of ring shear tests on remoulded Gault from the trial site, tested at the appropriate stress level indicate shear strengths of between 70% and 80% of those obtained from stability calculations. The shear strength capable of being mobilized along a rupture surface, is a function of the relative displacement that has occurred along its surface, Bishop et al (1971), with the actual displacement defining the residual, depending on the post-peak shearing characteristics of the soil. The higher shear strengths occurring in the field probably, therefore, reflect
that residual conditions have not generally been attained or the irregular nature of the slip surface in the field compared to the ring shear slip surface.

5.7 General Conclusions

It is concluded, in agreement with Rowe (1972), that the classification of a soil as a stiff fissured clay is insufficient to describe the conditions at any particular site and that strata which may exhibit a low permeability at depth under high effective stresses "are by no means free of the general question which arises with most other deposits, as to the meaning of the term - undrained strength in the mass...especially when measured on small specimens from shallow depths and at low effective stresses in cuts and borings".

The existence of a permeable fabric affects the period of pore pressure readjustment and the rate of swelling and strength change. The application of conventional classifications of failure as 'short-term' or 'long-term' to earthworks in the Gault can therefore be very misleading as the distinction is mainly based on time of occurrence of failure. Clays are generally considered relatively impermeable, and are not, as a result, expected to show any significant change in water content during a construction period of a few weeks or months, and total stress method of analysis is conventionally applied to construction or 'short-term' failures. On the other hand, when steady seepage conditions are
achieved, this is called 'long-term', for which effective stress method of analysis is considered applicable. However, the 'short-term' and 'long-term' states refer strictly to the undrained and drained steady seepage conditions respectively, and the periods during and after construction, which can appropriately be referred to as 'short-term' or 'long-term' are dependent on the permeability of the clay, which in turn depends on the fabric of the deposit.

It is therefore, very important that the limitations of the total stress method of analysis should be borne in mind in clays which exhibit a permeable macro-fabric, and that the mass permeability should be determined from insitu tests, or laboratory tests on large samples before assuming a no water content change conditions in design.
Standard Procedures Adopted During The Investigation

a) Boring and Description of Samples.

Boring was by the percussion method and continuous U4 (100 mm diameter) samples were taken in each borehole; except at sites of detailed pore water pressure investigation where several additional boreholes were sunk purely for the installation of piezometers.

All the 'undisturbed' samples obtained from borings were carefully examined and described except those that were held for testing. After extrusion, the remoulded material surrounding each sample was carefully scraped off to reveal the in-situ structure of the soil. The existence of slip surfaces could then be established by careful visual inspection. This method allows subsequent measurement of shear strength across any slip surfaces observed. Shear box samples were taken with the slip surface in the horizontal plane and triaxial samples such that the discontinuity made an angle \( \theta = 45 + \phi \cdot r/2 \) with the horizontal. At the start of the project \( \theta \) of about 52° was adopted, subsequently this value has been reduced to 50° in the light of recent published data on residual strength determination.

b) Trial Pits.

Trial excavations were made using a 'JCB 30' excavator for excavations up to 5 m in depth, deeper
pits extending to a maximum depth of 6 m below natural ground level were excavated by a 'Hymac' excavator or a 'Priestman Beaver' with extension arm fitted. It is generally recognised that trial pits can be dangerous and a common procedure is that they are logged by observation from the top of the pit. The need for extreme caution is endorsed but, however, as a result of the method of excavation adopted, it has been possible to enter all the trial pits excavated during this study. This has enabled slip surfaces to be exposed, plotted and sampled for laboratory testing. The method involved excavating each pit to a width of at least 2-3 m i.e. two or three bucket widths of most excavators, with steps on one side of the pit to allow easy exit. Failure of the sides of trial pits could in most cases be pre-empted by the dislocation of fissure lumps and the cracking sound produced by the severence of tree roots or the formation of tension cracks.

c) Pore Water Pressure: Installation and Measurement.

With the exception of piezometers installed at the trial embankment site, 'Open-tube' Casagrande type piezometers were used for monitoring field pore water pressures. The piezometers used consisted of 75 mm (3/4") internal diameter alkathene tubing to which was attached a porous ceramic pot. The ceramic tip was lowered to the desired level of observation,
surrounded and covered by a 150 mm (6") thick layer of sand and sealed with a 0.3 m (12") layer of a stiff mixture of bentonite and water. When two or three piezometers are installed in the same borehole, or where the piezometer tip is to be located above the base of borehole, a bentonite sand mixture was used to backfill the borehole to about 150 mm (6") below the desired level of installation. The piezometer tip is lowered and surrounded with sand and sealed with a bentonite paste as already described. The operation requires that the sides of the borehole at the desired levels of installation should be kept free of bentonite and this is normally achieved by casing the borehole or introducing the bentonite backfill into the borehole through a 50 mm internal diameter flexible tubing extending below the lowest level above the base at which a piezometer is to be installed.

The water levels in the piezometers were measured using a battery operated dip-meter, and the data obtained has been plotted for each piezometer.

d) Close-circuit Piezometer Installation and Measurement. (Trial Embankment Site)

The Soils Instrument Ltd. (SIL) recording unit and piezometers were used for the measurement of pore water pressure. The piezometers consisted of a
cylindrical porous ceramic pot with an internal diameter of 51 mm (2 in.) and a length of 133 mm (5½ in.) which were capped at both ends by a 6.4 mm (¼ in.) thick plastic disc. One of these discs had two openings to which brass connectors were fitted. Double-walled polythene tubes were attached to each connector before the piezometer pot was lowered to the required depth in the borehole; it was then surrounded, and covered with a 150 mm (6 in.) layer of sand and sealed with 0.3 m (12 in.) thick layer of concrete and bentonite grout before being backfilled. The polythene tubes from the piezometers were buried in a shallow trench backfilled with sand and run to the instrument hut containing the recording panels. The latter consisted of two mercury manometers to which the pair of leads from each piezometer were connected. A back pressure was applied to the manometers through a constant water header tank. The system was filled with deaired water by means of a deairing unit which allowed pressure to be applied down one of each piezometer lead and suction to the other. The minimum possible pressure and suction were applied to effect a circulation of water which was allowed to continue until the system was free of air bubbles.

Regular readings of the difference in level of the mercury in each manometer were taken and during construction, readings were taken twice daily, before the start and at the end of each day's work.
e) **Inclinometer Installation.**

Inclinometers were installed at some of the sites investigated for the precise measurement of horizontal ground movement. The 'Soils Instrument Ltd.' SIL, device was used for this study; it consists of a watertight brass torpedo fitted internally with four resistance strain gauges connected to a bridge circuit which enables output to be measured on a voltmeter. The voltmeter is calibrated in two ranges $\pm 5^\circ$ and $\pm 25^\circ$. A precise calibration frame is provided which enables the span calibration over $0 - 5^\circ$ and $0 - 20^\circ$ to be checked. An adjustable potentiometer is fitted on the instrument box which allows span errors to be removed.

The SIL casing is of extruded PVC and is supplied in 10 ft. (3 m) lengths, consisting of four internal keyways with $90^\circ$ spacing which determine the direction of measurement when installed. The casing lengths are cemented together with quick hardening cement using a cruciform mandrel to obtain proper alignment at joints. The casings were installed with one pair of keyways orientated along the direction of expected movement; (the other pair of keyways is thus set at right angles to the direction of expected movement). The method of manufacture of the P.V.C. casings results in twisting of the keyway alignment and to minimise this effect each casing used was hand chosen and restricted to a maximum twist of $2^\circ$. As all the
inclinometer installations in the project extend to less than a 100 ft. (30 m) the total twist likely to occur will only result in a small error in the measured displacements.

The borehole and casing were filled with water during installation, and were backfilled with a bentonite-cement grout pumped under low pressure 2.0-3.0 p.s.i. (14 - 20 kN/m²) through a flexible hose to the bottom of the borehole. The grout mix consisted of a 1 to 1 bentonite-cement ratio and a 4 to 1 water-solids ratio.

The instrument shows significant sensitivity to temperature and so about half an hour was always allowed for the torpedo to equalise with the temperature inside the casing before the first set of readings were taken. The calibration of the voltmeter was then checked using the calibration frame and the torpedo lowered into the casing with the fixed runner aligned in turn along each of the keyways i.e. N, S, E or W. Readings were taken at 2 ft. (0.61 m) intervals from the bottom of each casing, and the face error i.e. differences between readings on opposite keyways, was restricted to a maximum of 0.2°. Each division of the voltmeter was divided into 10 parts by eye which combined with the limited range of face errors allowed would result in only very small and insignificant errors over the range of depth for which measurements were made.
Since the angular changes are small and the casing is nearly vertical the horizontal displacement of an element $dh$ may be calculated from the equation:

$$dh = d\theta L$$

where $d\theta$ = change in slope (radians)

$L$ = interval of readings = 2 ft. (0.61 m)

Assuming that the casing is of sufficient depth such that the bottom remains fixed or only experiences a small displacement than the total displacement at any level could be calculated relative to the base of the casing by the equation:

$$dh = d\theta L$$

where the instrument factor $F = \frac{2\pi \times 24 \times 25.4}{360}$

(displacement in mm)

$= 10.65$

The expression for $F$ assumes that every angular increment between 0 and $5^\circ$ (the range of scale normally used) is accurately recorded on the instrument box.

Tests on 3 No. SII instruments (Private Conn Green, I.C.) have shown that in the $0 - 5^\circ$ range the ratio of the average recorded incremental angle to the true change in angle varies between 1.02 and 0.92 which if typical would correspond to an $F$ of 10.55 and 11.16 respectively.
Only small errors are therefore introduced by assuming an $F$ of 10.65.

In the N-S direction the slope at any point $\theta = (N-S)/2$ so that the face error cancels out. Restricting the face error to a small value serves as a check on the calibration of the instrument. By subtracting the initial slope from the value of $\theta$ obtained, the change in slope of a 2 ft. (0.61) length of casing is obtained. The slope changes are added starting from the bottom and the displacements are calculated by multiplying by 10.65.

f) **Index Parameters and Clay Fraction Determination.**

These parameters have been obtained by the methods detailed in BS.1377 (1961). For the liquid limit test, however, the standard technique has been modified. A representative sample of the soil was chosen without resorting to sieving and oven drying. A limited study of the effect of dessication on liquid limit shows that a reduction of 10-15% in liquid limit is likely to result.

g) **Stability Analysis.**

The investigations into the stability of earthworks carried out during this project revealed that failure surfaces were generally non-circular. Published data by Skempton and Hutchinson (1969) suggest that the Janbu method and the method proposed
by Morgenstern and Price are suitable for the analysis of such slides.

The method proposed by Janbu in 1954 has been chosen primarily because analysis could be done by hand calculation, whereas the Morgenstern and Price method (Morgenstern and Price 1965) requires the use of a computer. In the latter method it is assumed that the horizontal and vertical interslice forces 'E' and 'X' respectively acting on each slice are related by the expression:

\[ \frac{X}{E} = \lambda f(X) \]

where \( \lambda \) is a scale factor which is determined from the solution and \( f(X) \) is theoretically an arbitrary function concerning the distribution of internal stresses. In practice, however, only a limited range of variation is required, Hutchinson (1969).

The Janbu method also tries to account for the internal stresses but by a much simplified assumption that:

\[ E = \text{Constant} \times X \]

The method, (as well as that by Morgenstern and Price), like the Bishop method of slices (1955) for a circular slip surface, defines the factor of safety \( F \) as the ratio of shear strength to shear stress which results in the following expression:

\[ F = \frac{1}{\sum ptan\alpha} \sum \left[ \frac{c' + (p-u)tan\phi'}{N_{\infty}} \right] \times fo \text{ for equal slice widths.} \]

Where \( N_{\infty} = \frac{1 + tan\alpha tan\phi'}{F} \times \frac{1 + tan^2\alpha}{F} \)

and \( fo \) is a correction factor for the assumption of zero resultant vertical shear forces between slices.
The value of $f_0$ depends on the shape factor and the shear parameters of the soil. Graphs have been produced by Janbu for the evaluation of $N_{\alpha}$ and $f_0$. $C'$ and $\phi'$ are the peak effective stress parameter and $p$ and $u$ are respectively the normal stress and pore water pressure acting on the slip surface. The angle $\alpha$ is the inclination of the failure surface to the horizontal along the line of the centre of gravity of each slice.

Data by Morgenstern and Price (1965) suggest that the Janbu method can over-estimate the factor of safety by 8% or less.
The Determination of Stress Increments Below the Embankment.

The simple method devised by Mirata (1969) has been adopted in the evaluation of the stresses below the trial embankment. The method is based on the observation from Bishop's relaxation method that the stresses applied at a point above the ground surface are related to the stresses at a geometrical similar point acting at the ground surface.

\[
\frac{\delta v}{H-d} = \frac{\delta v}{H}, \quad \cdots (1) \quad \text{and} \quad \frac{\delta h}{H-d} = \frac{R \delta h}{H}, \quad \cdots (2)
\]

where \( R \) is a factor varying from 1.00 at the base of the embankment to 0.67 for \( d/b > 0.25 \).

\( d \) is effective height of layer under consideration above base.

\( b \) is width of embankment at \( d/2 \) above base.
The variation of stress increment with depth caused by each layer of the embankment is accounted for by varying the effective level of the layer according to the expression:

\[ d = \left( \frac{2H - \bar{Z}}{2H} \right) \times d_0 \quad \ldots \ldots \ldots \ldots (3) \]

where \( d_0 \) = height above centre gravity of the layer above the base of the embankment.

\( \bar{Z} \) = depth below base of embankment.

\( H \) = height of the triangle to which the embankment can be approximated.

Eq. (3) is valid in the range of depth between \( 0 \leq Z \leq 2H \), which covers the range involved in most stability problems. At depths exceeding \( 2H \) the elevation of the layer has an insignificant effect on the stress increments.

In semi-infinite foundations the stresses induced by the embankment are independent of Poisson's ratio and Jurgenson's (1934) Tables for a uniform strip load acting on the surface are used to determine stress influence coefficients which take into account the elevation of each loading strip. The values of stress influence coefficients are given for a grid interval of \( B/4 \) below the strip where \( B \) is the width at the centre of gravity of the strip. Thus the depth below the equivalent level of the strip for which the stresses are calculated is given by:

\[ a = \bar{Z} + d \quad \ldots \ldots \ldots \ldots (4) \]

where \( a = 0, B/4, B/2, \frac{3}{4}B, B \) etc.

From Eq.s (3) and (4):

\[ \bar{Z} = X[Y^2 + d_0(a - d_0)\frac{H}{d_0}] \ldots \ldots (5) \]

where \( X \) and \( Y \) are constants given by:

\[ X = 2H/d_0, \quad Y = H - d_0 \]
Eq. (5) is valid for 'a' values ranging between $d < a < 2H$. The values of $Z$ calculated from Eq. (5) determine the vertical spacing of the grid lines, the horizontal spacing being $B/4$. The corresponding stress influence coefficients are then entered for each node and by multiplying these by the intensity of the strip loading the stress increments can be determined. Horizontal stress increments are multiplied by a factor $R$ which varies with the ratio of $d/b$. A bulb of vertical and horizontal stress increments is drawn for each layer from which the load increments at any position and depth below the embankment could be determined.

The horizontal and vertical stresses calculated for the piezometers in Groups 1 to 4 are tabulated in the following pages. The notations used in the table are defined below:

- $H = \text{height of fill at shoulder of embankment.}$
- $\Delta H = \text{thickness of layer.}$
- $U_0 = \text{initial pore water pressure.}$
- $\Delta U = \text{observed excess pore water pressure.}$
- $\Delta U = \text{'Summated' undrained pore water pressure.}$
- $\Delta U = \text{total vertical stress increment.}$
- $\Delta U = \text{horizontal stress increment.}$
### TABLES 3a

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* These readings do not correspond to a layer, they are included to illustrate P,N,P behaviour during the 2 days preceding failure.
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*These readings do not correspond to a layer; they are included to illustrate P.I.P. behaviour during the 2 days preceding failure.*
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<th>DH (m)</th>
<th>AU (KN/m²)</th>
<th>DU (KN/m²)</th>
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*Note: These readings do not correspond to a layer, they are included to illustrate ramp behaviour during the 2 days preceding failure.*
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* These readings do not correspond to a layer, they are included to illustrate P. N. P. behaviour during the 2 days preceding failure.
**Pz No E73 Depth 1.25 (m)  Pz location: Soi liquated Gault**

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**Pz No E73 Depth 6.10 (m)  Pz location: Weathered Gault**

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<th>U%</th>
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**Pz No E83 Depth 0.88 (m)  Pz location: Granular Head**

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**Pz No E83 Depth 5.50(m)  Pz location: Weathered Gault**

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<th>U%</th>
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*If these readings do not correspond to a layer, they are included to illustrate p.w.p. behaviour during the 2 days preceding failure.*
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