LONG-TERM STABILITY
OF SLOPES IN OVER-CONSOLIDATED CLAYS

VOLUME I

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ABSTRACT

Long-term stability of slopes in over-consolidated clays is considered in this thesis on an effective stress basis, and it is shown that this approach to the problem is a practical one. The study is carried out with special reference to London clay as it is a well known over-consolidated clay of wide occurrence, is relatively homogenous and there exist in it a considerable number of stability problems of economic importance.

The investigation is considered under three main divisions:

(a) A laboratory study of over-consolidated clay.

(b) A field investigation of the stability of natural slopes.

(c) A field study and an analysis of past failures of slopes and retaining walls.

In the laboratory study it is shown that heavily over-consolidated clays have a dilatant structure and the application of shearing stresses tends to set up negative porewater pressures which give the clay a high initial strength. In the course of time after construction this negative porewater pressure dissipates, with a consequent diminishing of the strength. A number of tests were carried out on undisturbed samples of London clay and these tests together with results from other sources give a clear indica-

tion of the drained shear strength parameters of the clay. It is shown that over-consolidated clay can fail at lower stresses than usual if it is subjected to alternating stresses. Indications are that the loss in strength is at least partially a reduction of cohesion in terms of effective stresses.

The field investigation of natural slopes in London clay indicates that the limiting inclination for long-term stability is very nearly 10^{o}. No failures have been found on flatter slopes and very few stable slopes which are steeper were found. Analysis indicates that the limiting angle of 10^{o} is consistent with the water table at the surface and the presence of only the frictional component of the shear strength. The indications then are that the cohesive portion of the shear strength is reduced to zero on a long-term basis.

The consideration of failures of retaining walls and man-made slopes shows that the most practical method of investigating stability for these structures in over-consolidated clay is on an effective stress basis. Results of the analyses indicate that cohesion diminishes with time after construction, and a probable rate for the reduction is suggested. It appears that the rate of reduction is influenced by the effective normal pressure on the potential failure surface.

Remedial measures and their effectiveness are considered for several cases and suggestions for design and remedial measures are considered.

CONTENTS

SECTION	TITLE	PAGE
	PART I GENERAL CONSIDERATIONS	
1	Introduction	1
2	Historical Background	3
3	Over-consolidated Clay	9
4	London Clay	11
5	Shear Strength	13
	PART II LABORATORY INVESTIGATION	
6	Introduction	20
7	Laboratory Equipment	22
8	Laboratory Procedure	25
9	Test Results	30
10	Volume Change	30
11	Undrained Test with P.W.P. Measured . . .	34
12	Constant σ_1 Tests	37
13	Drained Shear Strength Parameters for London Clay	39
14	Alternating Stresses	40
15	Constant Pressure, Volume Change Apparatus	41
16	Clay Used in Alternating Stress Tests . .	42
17	Procedure, Alternating Stress Test	43

18 Discussion of Alternating Stress Test . . 45

19 Conclusion from the Laboratory Investigation 47

PART III STABILITY OF NATURAL SLOPES

20 Introduction 50

21 Area of Field Investigations 52

22 Field Investigation 54

23 Unstable Slopes 55

24 Stable Slopes 57

25 Stability Analysis 58

26 Conclusions from Natural Slopes 61

PART IV EXAMINATION OF FAILURES

27 Introduction 63

28 Choice of Failure Surface 65

29 Water Forces 67

30 Analysis by Circular Arcs 70

31 Analysis by Planes 71

32 Factor of Safety, Analysis by Planes . . 77

Case Histories

33 Wembley Hill Retaining Wall 79

34 Park Village East Retaining Wall 87

35 Mill Lane Retaining Wall 93

36	Uxbridge Retaining Wall	102
37	Northolt Slope	111
38	Sudbury Hill Slope	121
39	Kensal Green Retaining Wall	126
40	Wood Green Retaining Wall	131
41	Upper Holloway Slope and Retaining Wall . .	136
42	Summary of Results from Examination of Failures	140
43	Variation of c' with Age and σ_n'	142
44	Remedial Measures	146
45	Suggested Remedial Measures	148
46	Suggestions for Design	151
47	Conclusions, Examination of Failures . . .	153

Appendix A	Corrections for Rubber Membrane and Drain	156
" B	Ø' and c' between London clay and a Rough Surface	159
" C	Drainage in Slopes by Counterforts	162

Acknowledgements	164
References	166

NOTATION

$\bar{B}$	Pore pressure coefficient
b	Width of slice
c'	Unit cohesion in terms of effective stresses
c'_m	Mobilized unit cohesion in terms of effective stresses
C'	Cohesion force in terms of effective stresses
C'_A	" " on base of active wedge
C'_B	" " on back of wall
C'_F	" " on front of wall
C'_P	" " on base of passive wedge
C'_W	" " on base of wall
d	Depth of slice
F	Factor of safety
h_w	Water head on failure surface
P_A	Active force in terms of effective stresses
P_P	Passive " " " " " "
P_W	Wall resistance " " " "
P'_P	Vector sum of P_P and U_F
R_A	Reaction at base of active wedge
R_P	" " " " passive wedge
R_W	" " " " wall
S	Shearing strength
S'	Shearing strength in terms of effective stresses
s_m	Mobilized unit shearing strength in terms of effective stresses

U_A Water force on base of active wedge

U_B " " at back of wall

U_F " " at front of wall

U_P " " on base of passive wedge

U_W " " on base of wall

u Porewater pressure

W Weight of slice

W_A Weight of the active wedge

W_P " " " passive wedge

W_W " " " wall

w Water content

X Vertical shear forces between slices

α Angle between base of slice and horizontal

$$\beta = \frac{\sec \alpha}{1 + \frac{\tan \phi'}{F} \tan \alpha}$$

γ Unit weight of soil (saturated)

γ_w " " " water

δ Angle of wall friction

σ_1 Maximum principal total stress

σ_3 Minor " " "

σ_n Total normal pressure on shear plane

σ_n' Normal pressure on shear plane in terms of effective stresses

τ Shearing stress

ϕ Angle of shearing resistance

ϕ' Angle of shearing resistance in terms of effective stresses

ϕ'_m Mobilized angle of shearing resistance in terms of effective stresses

ϕ'_f Mobilized angle of shearing resistance in terms of effective stresses at a particular value of F.

ARRANGEMENT

Volume I contains the complete text and all figures and plates except those for the case histories. The figures and plates for the case histories have been put in Volume II for convenience in handling but are numbered consecutively with those in Volume I.

PART I

GENERAL CONSIDERATIONS

INTRODUCTION

It has been found on many occasions that natural earth slopes, man-made slopes and earth retaining structures which have been apparently stable for many years do not always remain in that condition; failure sometimes occurring, for no obvious external reason, a considerable time after their construction or formation. The time to failure may vary from a few years in the case of a slope in a cutting to unknown thousands of years for a slope formed by geological processes. This phenomenon is most common in the type of clay first described by Terzaghi (1936) as "stiff, fissured clay" of the intensely overcompacted group of clays and now known as heavily over-consolidated clay.

Over-consolidated clays are of wide occurrence in Great Britain and western Europe and the problem of stability after construction, or "long-term stability", in these clays has been a concern of engineers for at least two centuries. However, it is only within the last 35 years that knowledge

of the mode of formation of these clays and techniques of handling the problems concerned were developed, and it is only within the last five years that the laboratory techniques and equipment capable of determining the relevant fundamental properties of the clay were established and the results applied to long-term stability problems.

Since the London clay is probably the best known of the over-consolidated clays, with regard to both its fundamental properties as determined in the laboratory and its action and properties in the field, this study has been concentrated on London clay. The problem is considered in three main divisions:

(a) Laboratory investigation of the clay

(b) Stability of natural slopes

(c) Stability of retaining walls and slopes in cuttings.

During the laboratory investigation the behaviour of the clay under stress is investigated and its shear strength parameters in terms of effective stresses determined. The application of these parameters to the stability of natural slopes in London clay observed in the field, indicates that the cohesion intercept (c') approaches zero on a geological time scale. By analysing past failures of retaining walls and slopes in cuttings a probable rate for the reduction in

c' is deduced. The variation in the factor of safety of these structures through the possible range of c' is determined and the effectiveness of remedial measures considered.

HISTORICAL BACKGROUND

Problems of slope stability are among the oldest in the field of soil mechanics and probably date from man's first attempts to protect his home with earthworks and to build burial mounds. Early written work dating from the 17th and 18th centuries are concerned for the most part with fortifications (see for instance Feld, 1948) as these were the major engineering problems of the time. Records of an observed slip surface behind a retaining wall date from as early as 1699 (Vauban as recorded by Collin 1846), and Coulomb (1776) noted that the failure surface behind a retaining wall was approximately a plane. Coulomb postulated the concept that shear strength in soils is made up of a cohesive component and a frictional component and used this concept to establish methods of determining lateral earth pressure. It is basically his methods that have been used to determine the forces on retaining walls in Part IV of this study.

The first detailed observation on slip surfaces in

slopes were made by Collin (1846) who collected information on several failures and observed that movements were rotational and the shape of the failure surface approximately cycloidal. The common idea at the time was that slips resulted from a pre-existing plane of weakness or a weak stratum. Collin however, concluded that inadequate shear strength was the cause of failure and that the failure surfaces which were observed were generally the result of the earth movement and not the cause of the movement. He was working with stiff fissured clay, and found that the percolation of water had a great deal to do with the shear strength and long-term stability in these slopes. At the same time he advanced the idea that all slopes would tend to return ultimately to the inclination at which they were found in nature. The first recorded shear tests on clay were carried out by Collin who noted that the strength of the clay varied with moisture content and with time to failure, but the tests were generally what would now be considered undrained tests.

In Great Britain the problems of slope stability became more pressing with the construction of the railroads, starting about 1825. Gregory (1844) gives several examples of failures in railway cuttings which took place some years after construction. He attributed most of these slips to

"statical" water forces and the softening of the clay due to the infiltration of water, and notes that drainage was a definite aid to stability as follows:

> "water in these and most other cases appeared to be the ultimate cause of all the slips; the drainage therefore of the slopes, as recommended by the author, in both cuttings and embankments, is a consideration of the utmost importance ....."

Gregory makes no mention of a study of the shape of the sliding surfaces nor does he attempt to determine shear strength and investigate methods of analysis as Collin was doing at this time.

The works of Collin and Gregory were almost completely ignored, but Coulomb's theory as well as Rankine's conjugate stress theory (1857) were applied to retaining wall design in the latter part of the 19th century. However as soil properties were not in general measured nor was the effect of water pressure understood, the results were far from satisfactory and numerous failures occurred. The situation was so unsatisfactory that Sir Benjamin Baker in 1881 remarks:

> "If an engineer has not had some failures with retaining walls it is merely evidence that his practice has not been sufficiently extensive ....."

In keeping with the trends of the time it appears

that although theories and mathematical approaches had been advanced for the design of earth retaining structures the investigation of soil properties and failures was seriously deficient. Baker (1881, p.182) refers to this trend, without doubt quite justifiably, in the following words:

> ".....it has often struck the author that the numerous published tables giving the calculated required thickness of retaining walls to three places of decimals stand really on the same scientific basis, and have the same practical value, as the weather forcast for the year in Old Moore's Almanack."

These remarks seemed still to pertain to engineering practice for earthworks during the first decade of this century and it is questionable if any really significant progress in the understanding of the basic problems had been made beyond the work of Coulomb and Collin.

The assumption of failure along a circular arc for the purpose of analysis was first introduced by Petterson and Hultin (1916) in considering the failure of a quay at Gotenburg. Further work in Sweden under Fellenius (1922) on the investigation of railway slips led to the use of the shear strength of the clay as a cohesive force only, neglecting frictional effects. Together these two procedures result in the $\emptyset = 0$ analysis as it is now used.

Terzaghi in 1936, examined the stability of natural slopes, considering stiff fissured clays as a particular class of clay, and advanced an explanation for its reduction in strength with time. He considered this to be due to expansion of the clay under reduced lateral restraint and softening of the clay surrounding the fissures by the infiltration of water.

Skempton (1948) considered the problem further and gave a tentative relationship between average shear strength along a failure surface and time. Further developments since then indicate that it is more logical to consider the long-term stability problem in terms of effective stress parameters determined from drained tests, rather than on a total stress ($\emptyset = 0$) basis.

A significant step towards the understanding of long-term stability in over-consolidated clays was made by Terzaghi in 1925 when he introduced the concept of effective stress, although this principle was not applied to the problem at that time.

Bishop and Henkel (1953a) showed that when a heavily over-consolidated clay was stressed, negative porewater pressure changes could be set up, and herein seems to lie one of the keys to the understanding of long-term stability in this class of clay. When the clay is stressed in the

field, generally by the removal of lateral support, internal negative porewater pressures are set up which give the clay a high initial strength. These negative pressures eventually dissipate causing a lowering of the shear strength and hence a less stable condition. It is this negative porewater pressure which gives over-consolidated clays the ability to remain at very steep slopes for short periods of time, a phenomenon that has baffled investigators and observers for many years. Cowper (1844) noted that, after the failure of a retaining wall in London clay near Euston Station, when the wall was removed for rebuilding the same clay which has caused the failure was able to remain almost vertical without support. The stability of the retained clay in a case such as this is probably due to the setting up of further negative porewater pressures under the new stress conditions, the dilatency of the clay having been incomplete in its former state. The same occurrence was noted by Baker (1881) and doubtless has been observed on many occasions both before and since.

The application of effective stress analysis to stability problems in stiff fissured clays has developed only within the past five years. Henkel and Skempton (1954) used drained shear strengths in the analysis of a landslide on a natural slope in over-consolidated clay and showed that this approach to the stability problem in over-consolidated

clays was the most feasible one. It was shown in that study that a good indication of stability is obtained if the cohesion portion of the drained shear strength is omitted and only the frictional part used. This implies that on a long-term stability basis c' is reduced to zero and it is a further consideration of this phenomenon that this thesis will consider, with special reference to London clay.

OVER-CONSOLIDATED CLAY

Over-consolidated clay may be defined as clay which was originally consolidated under a greater load than that under which it now exists. The original consolidation in this type of clay has generally been brought about by the weight of incumbent ice sheets or sediments which have since disappeared. In some places the over-consolidating loads may have been both ice and sediments with the meltwater from the ice contributing to the removal of the sediments.

Over-consolidated clays include relatively homogeneous sedimentary clays such as the London or Lias clays as well as boulder clays, which by the very nature of their formation under ice sheets are over-consolidated. The boulder clays are usually composed of ground-up debris with

great variation in particle size and usually have no well defined structure or stratification. Clays deposited as sediments and later over-consolidated are usually much finer in grain size and more homogeneous in character. Unlike the boulder clays they generally contain fissures which exist throughout the soil as a system of fine cracks. When the soil is in a relatively dry state approaching its plastic limit it breaks easily along these fissures into fragments, the size of which vary from clay to clay, and within the same clay, and have been observed to be as small as cubes of less than ½ in.

Fissuring is not usually found in samples either normally or over-consolidated in the laboratory but has been found to take place (Skempton and Northey, 1952) in London clay remoulded at the liquid limit and stored for a year. Many normally consolidated marine clays are also found to be fissured. It was suggested (ibid) "....that the fissures developed as a result of syneresis, a colloidal process whereby the particles draw themselves together under the action of attractive forces and expel some of the pore-water."

Because of the high loads once sustained by heavily over-consolidated clay, high lateral pressures were set up within the soil mass and with the removal of the

load some of this pressure has remained. The result is that the coefficient of earth pressure at rest (K_o) is proportionally high for these clays, commonly being greater than 1.0. When a cutting in stiff-fissured clay is opened the high lateral stress is relaxed, which gives the clay an opportunity to expand laterally and the fissures to open. Such expansion has been noted to a very marked degree in a stiff Devonian clay underlying glacial till by Terzaghi (1950) and has been observed, although to a lesser extent in London clay (e.g. at Northolt, see Part IV). This opening of the fissures in London clay apparently alters the permeability of the soil to a considerable degree, see Appendix C, and allows easier passage of the porewater, undoubtedly to the detriment of the clay's shear strength.

LONDON CLAY

London clay, one of the best known of the heavily over-consolidated clays, is a marine deposit of the Eocene period which was originally about 400 ft. thick and overlain by further sediments. The superimposed sediments have been completely removed by erosion in most of the London basin and generally the larger portion of the London clay has been

removed as well. Total thickness of deposits on what is now a typical point in London clay, at 100 ft. above its base has been estimated (Skempton and Henkel, 1957) to have been at least 500 ft. and probably nearer 700 ft. In some areas the London clay has been covered again, after the major portion was removed, by glacial and river gravels and alluvium. The depth of these deposits, however, very seldom reaches 50 ft.

The top 15 to 25 ft. of the clay where it is now exposed is usually found to be weathered and light brown in colour. Below this is the blue-grey unweathered clay. Usually both zones are found to be considerably fissured, but near the surface where the clay has a high moisture content the fissures may not be particularly evident. The average plastic limit is about 27 and the average liquid limit about 80 but there may be variations in these values, particularly the latter. Natural moisture content below the surface zone is usually near the plastic limit, indicating the high degree of consolidation to which the clay has been subjected. Both the moisture content and the Atterberg limits are usually found to be slightly higher in the weathered zone.

Although variations and very occasional fine sand and silt seams occur in the clay remarkably small differences have been found over zones many feet in thickness

(Cooling and Skempton, 1942). It has also been shown (ibid) that index values give a good indication of the other mechanical properties of the soil.

Sensitivity of the clay is very nearly one (Skempton and Henkel, 1957).

SHEAR STRENGTH

Coulomb (1776) first suggested that the shear strength of a soil was made up of a cohesive and a frictional component. This led to the equation for shear strength:

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

where S = the shear strength

C = the cohesion

σ_n = the normal pressure on the shear plane

ϕ = the angle of internal friction

The existence of internal porewater pressures in soils was shown by Terzaghi (1925) but was not applied directly to shear strength at that time. Rendulic (1937) showed that the strength and deformation properties of a soil were controlled by the intergranular forces and since

then this has been demonstrated to be so for a wide range of soil types. The intergranular or effective stress acting at any point within a soil mass may be expressed by:

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

where σ' = the effective stress

σ = the total stress

u = the porewater pressure.

The shear strength may then be expressed in terms of effective stresses as:

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

where S' = the shear strength

C' = the cohesion

σ_n' = the normal pressure on the shear plane

ϕ' = the angle of shearing resistance

all in terms of effective stresses.

When clays are consolidated and then tested under drained conditions no porewater pressures are set up during the testing and the shear strength in terms of effective stress is obtained directly. If a number of samples of the same clay are consolidated under different confining pressures and drained tests then carried out at the same pressures,

Mohr circles may be drawn which will yield a Mohr envelope for the clay in terms of effective stresses.

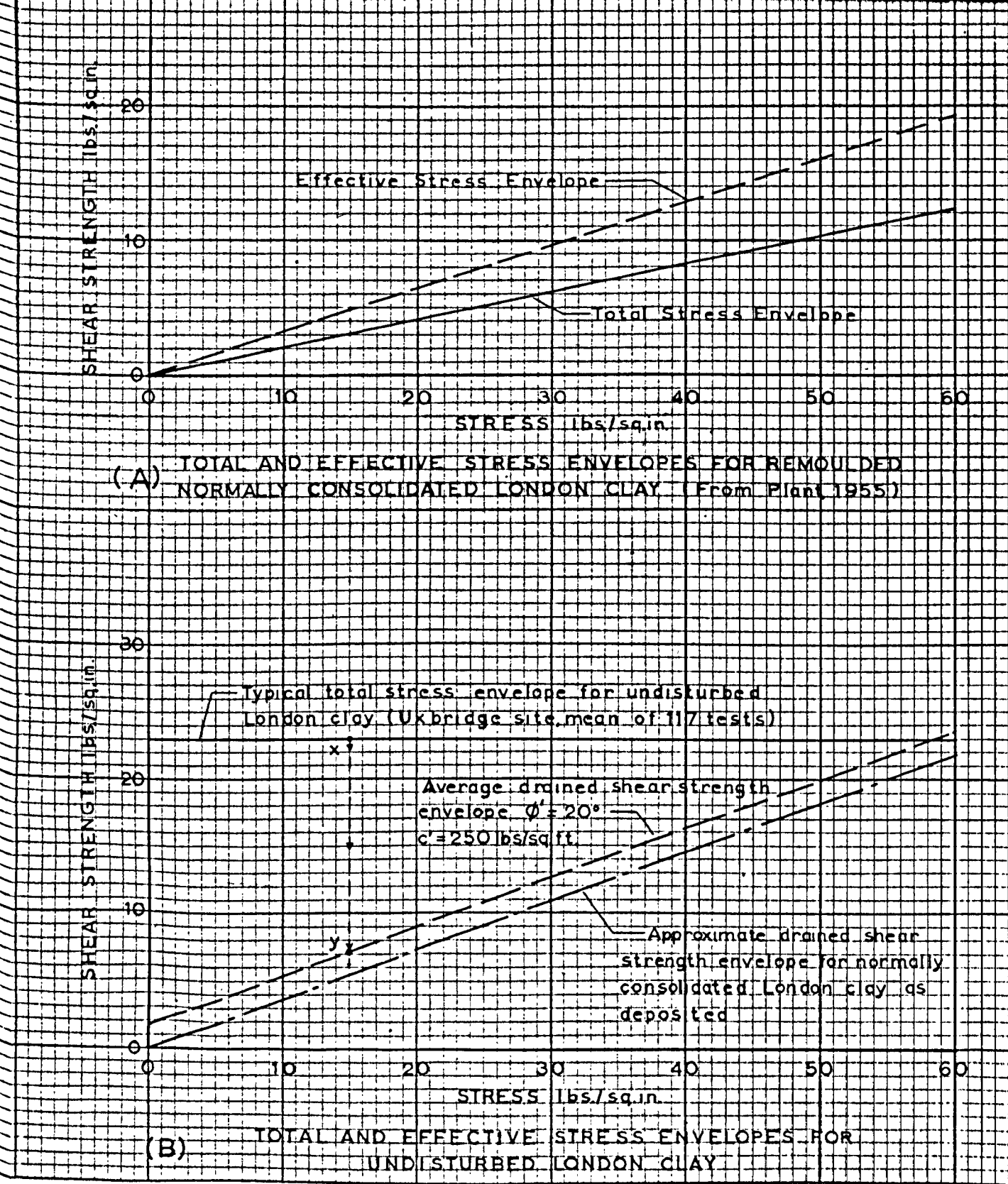
Similarly if a number of samples are consolidated at different pressures and then tested under undrained conditions with porewater pressures measured, their shear strength in terms of effective stress may be determined and a Mohr envelope, also in terms of effective stresses, obtained for the clay. However samples consolidated at the same pressure and then tested under undrained conditions yield the same shear strengths in terms of both total and effective stresses irrespective of the confining pressure during testing. The total shear strength for such samples may then be expressed in terms of cohesion only. This is also found to be the case for undisturbed field samples of saturated clay, see for example Golder and Skempton (1948).

Normally consolidated clays usually exhibit positive porewater pressure changes when tested under undrained conditions and their effective stress Mohr envelopes pass through the origin indicating an angle of shearing resistance but no cohesion. The same result, although usually a slightly different angle, is obtained from drained tests on normally consolidated clay. As the porewater pressure changes set up when this class of clay is stressed are positive, their dissipation must result in an increase of strength due to an increase of the inter-

granular forces. This is illustrated in Fig. 1A where the total and effective stress envelopes are given for remoulded, normally consolidated London clay. It is therefore apparent that the critical period for stability in normally consolidated clays is usually when they are first stressed. The shear strength is then lower than it is after the porewater pressures set up by the stressing have had an opportunity to dissipate.

Heavily over-consolidated clays usually show negative porewater pressure changes when stresses approaching failure stress are applied under undrained conditions. The Mohr envelope, see Fig. 1B, in terms of effective stresses from undrained tests indicates an angle of shearing resistance and the presence of cohesion. Drained tests give the same type of results except that the angle is slightly smaller and in some cases the effective cohesion somewhat lower. As the porewater pressure changes set up when heavily over-consolidated clay is stressed are negative, their dissipation must result in a lowering of the intergranular force and hence a lowering of the strength. That is the shear strength drops from point x, Fig. 1B to point y. This indicates that the critical period for stability in heavily over-consolidated clay is not when the clay is first stressed but after the negative porewater pressure

FIG 2



(A) TOTAL AND EFFECTIVE STRESS ENVELOPES FOR REMOULDED NORMALLY CONSOLIDATED LONDON CLAY (From Plant 1955)

(B) TOTAL AND EFFECTIVE STRESS ENVELOPES FOR UNDISTURBED LONDON CLAY

TYPICAL MOHR ENVELOPES FOR LONDON CLAY

FIG. 1

changes are dissipated and the drained strength applies.

It appears that there is a further reduction in shearing strength beyond that indicated by the drop from undrained to drained conditions. This lessening of the strength is apparently a loss of effective cohesion and it is the purpose of this thesis to show that this occurs and to consider reasons why this comes about, and make an estimate of the time taken for it to occur.

Research on clay that has been both normally consolidated and over-consolidated from a high moisture content remoulded state in the laboratory (Gilbert 1954, Plant 1956) indicates that there is relatively little difference in the shear strength characteristics of the clay until an over-consolidation ratio of about four is reached. At higher over-consolidation ratios however there is a pronounced difference, in the shear strengths, becoming greater at higher over-consolidation ratios. For the purpose of this thesis the term over-consolidated will be used to mean heavily over-consolidated clays, that is clays with a considerable over-consolidation ratio such that cohesion (c') is present and the shear strength in terms of effective stresses is greater than the normally consolidated shear strength in terms of effective stresses.

Until recently, common practice has been to use

immediate undrained strengths for the investigation of slope stability in all types of clay. These strengths were measured in the shear box, by unconfined compression or by triaxial tests, the latter method usually being used with fissured clays to prevent failure from taking place along the fissures. As has been discussed above, the immediate undrained strength is the critical one in normally consolidated clays but in over-consolidated clays the drained strength is the most critical, being in some cases only one quarter of the undrained strengths. Softening of the clay about the fissures as shown by Terzaghi (1936) could account for some of the loss in strength in a highly fissured over-consolidated clay. But so far as is known, softening has never been observed to occur extensively in a highly fissured clay to the point where the average shear strength on a potential failure plane is as low or lower than the drained strength. Therefore it appears that the only practical method is to use drained shear strengths for the consideration of long-term stability problems in over-consolidated clay. The first time that this approach for stability considerations in over-consolidated clay was used, with a full comprehension of the reasons for doing so, appears to have been in the analysis of the slide at Jackfield, Shropshire by Henkel and Skempton (1954).

Drained shear strengths will be used generally for the analysis of the failure cases in Parts III and IV of this thesis. These cases are all in London clay and are all failures that have occurred a considerable time after the stressing of the clay.

PART II

LABORATORY INVESTIGATION

INTRODUCTION

It is evident that a knowledge of the effective shear strength parameters, and the drained shear strength in particular, is necessary for the consideration of long-term stability in over-consolidated clays. To obtain significant shear strength parameters which can be applied to failures in the field, testing must be carried out on undisturbed samples of the clay, as remoulding destroys any c'. Also, it is usually found that the effective angles of shearing resistance are slightly lower in remoulded samples. Both these reductions in shear strength are probably due to some loss of structure within the clay itself.

For the reasons given, the laboratory investigation was carried out on undisturbed clay with only a few exceptions. Samples in an undisturbed state were not available in some cases and for the boulder clay it was very difficult to take good 1½ in. diameter undisturbed

samples because of the pebbles in the clay. Clay samples, even when taken from side by side, are seldom as uniform as remoulded samples, therefore the same amount of agreement cannot be expected in test results. Difference in structure and grain size accounts for some of the variation between samples and it appears that fissuring can cause large differences in test results.

As the largest number of examples of failures in over-consolidated clay most suitable for analysis are in London clay and as this clay was the most readily obtainable in the undisturbed state, the laboratory testing programme was concerned primarily with London clay. A few other over-consolidated clays, however, were tested so that a comparison of their properties and behaviour under shearing stresses could be made.

Samples are usually tested triaxially in the laboratory by increasing the vertical stress (σ_1) on the sample and keeping the lateral stress (σ_3) constant. However, in the case of clay under stress in a natural slope or the side of a cutting the load has generally been applied in the opposite manner. There, the vertical load stays constant or perhaps decreases somewhat while there is a considerable decrease in the lateral restraint. This might result in different shear strength parameters than those operative at failure in the usual type of laboratory test.

With this in mind tests were carried out under two different sets of stress conditions:

(A) Increasing σ_1 with constant σ_3 stresses,

(B) Constant σ_1 with decreasing σ_3 stresses.

The test results could then be compared and differences in shear strength, if any, determined.

LABORATORY EQUIPMENT

The greater part of the testing programme consisted of triaxial tests on samples of undisturbed over-consolidated clay. Types of test carried out were:

(1) Drained test with increasing σ_1 and constant σ_3 stresses.

(2) Drained test with constant σ_1 and decreasing σ_3 stresses.

(3) Undrained test with porewater pressure measurements, increasing σ_1, constant σ_3.

(4) Undrained test with porewater pressure measurements, constant σ_1, decreasing σ_3.

(5) Immediate undrained tests.

The majority of the triaxial tests were of types (1) and (2) the others being used primarily for purposes of comparison.

Samples tested were 1½ in. diameter by approximately 3 in. long and were tested in a triaxial cell of internal dimensions 3¾ in. diameter by 7¾ in. long. Cell pressure was maintained by a constant head mercury cylinder pressure device (Bishop and Henkel, 1953b).

In the increasing σ_1 type of test load was applied by means of a constant rate of strain loading frame as shown in Fig. 2 and Plate I. This machine was equipped with a multi-speed gear box so that a very wide selection of rates of strain was available. Porewater pressures were measured, without allowing volume change of the sample to take place, using the apparatus shown in Fig. 2. In operation, the lead from the sample to the top of the mercury column is completely filled with de-aired water and any change of porewater pressure in the sample tends to cause a change in the level of the mercury in the capillary tube. By adjusting the pressure at the base of the mercury column with the hand operated control ram, any change in the mercury level is prevented. The balancing pressure required, which can be read from the pressure gauge or manometer, then indicates the porewater pressure in the sample.

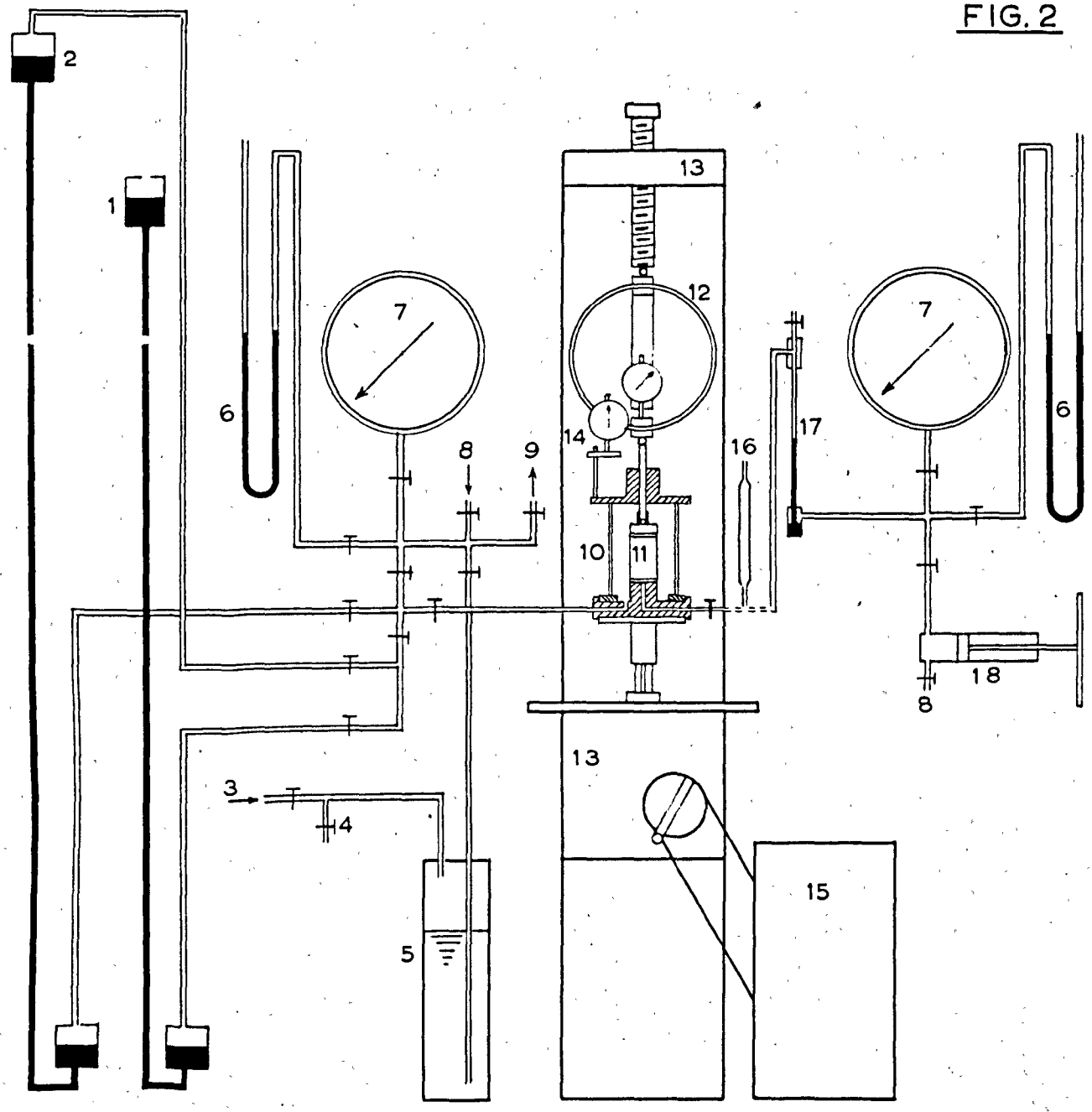
Constant σ_1 drained tests were carried out by applying a vertical load to the sample as the cell pressure was reduced. To do this aluminium alloy hangers were designed, see Fig. 3 and Plate III, to balance on the piston of the triaxial cell and suitable loads were added to the hangers as required.

In the case of a test in group (4) (constant σ_1, with p.w.p. measured) the constant rate of strain loading frame was used to apply a vertical load (σ_1) and the cell pressure was continuously adjusted by moving the mercury pot so that the total σ_1 on the sample remained constant.

Two tests were carried out so as to simulate the conditions of alternating stress to which clay in a slope is subjected by fluctuations of ground water level. To do this, the external loads on the sample must be kept constant while the porewater pressure is varied, drainage of the sample being allowed to take place.

A circular volume gauge was built for this purpose, see Fig. 9 and Plate V. With it, the volume change in the sample could be measured while a pre-set porewater pressure was maintained. The mode of operation of the gauge is indicated on Fig. 9.

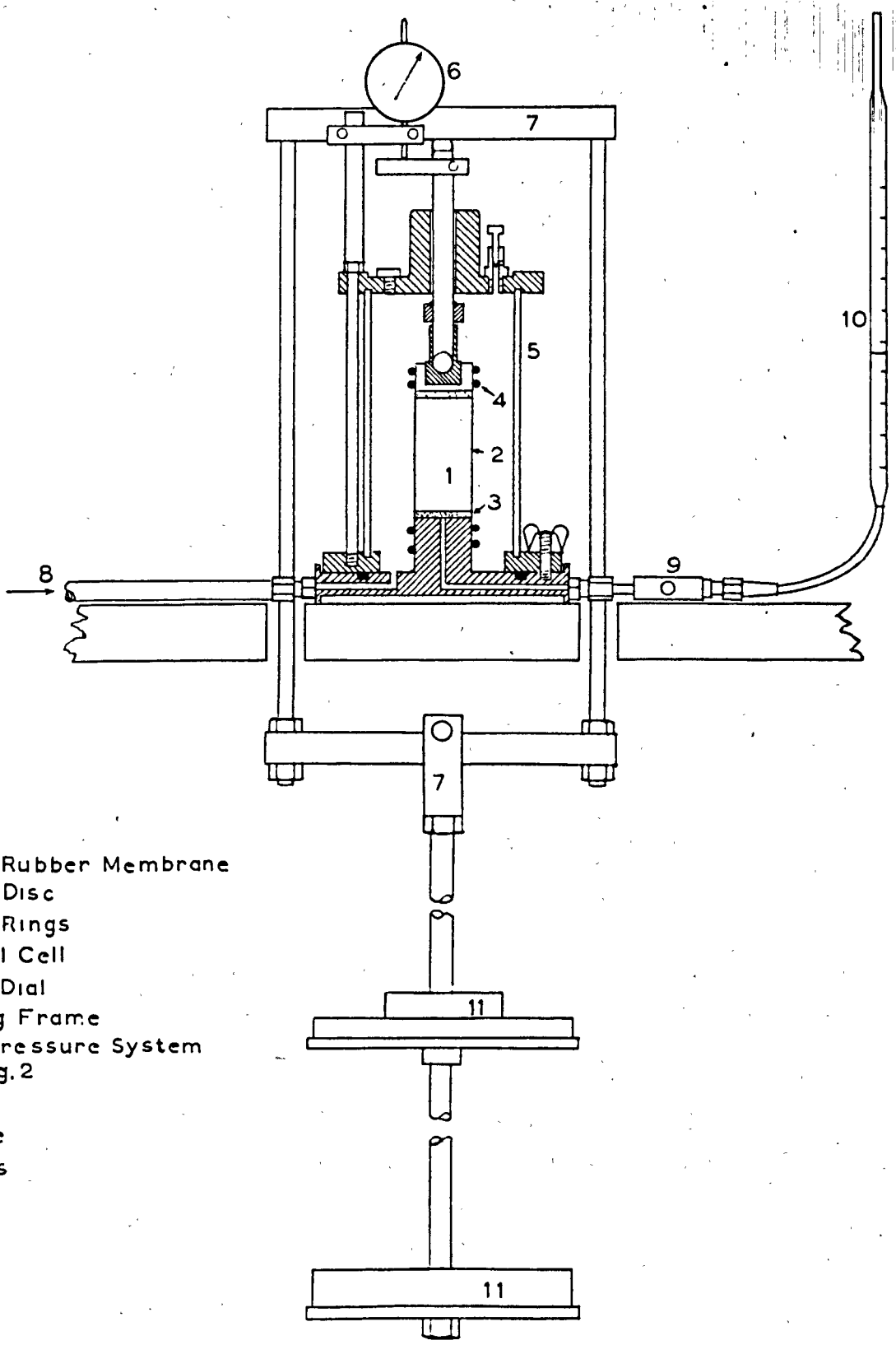
FIG. 2



1 Adjustable Mercury Cylinder
2 Fixed Mercury Cylinder
3 Compressed Air Supply
4 Exhaust Valve
5 Air / Water Reservoir
6 Mercury Manometer
7 Pressure Guage
8 Water Supply
9 Vacuum
10 Triaxial Cell
11 Sample
12 Proving Ring
13 Loading Frame
14 Strain Dial
15 Electric Motor & Gear Box
16 Burette for Drained Tests
17 Mercury Balancing Column
18 Hand Operated Ram

ARRANGEMENT FOR INCREASING σ_1 TRIAXIAL TESTS

FIG. 2



1 Sample
2 Drain & Rubber Membrane
3 Porous Disc
4 Sealing Rings
5 Triaxial Cell
6 Strain Dial
7 Loading Frame
8 From Pressure System as in Fig. 2
9 Valve
10 Burette
11 Weights

ARRANGEMENT FOR CONSTANT σ_1 TRIAXIAL TESTS

FIG. 3

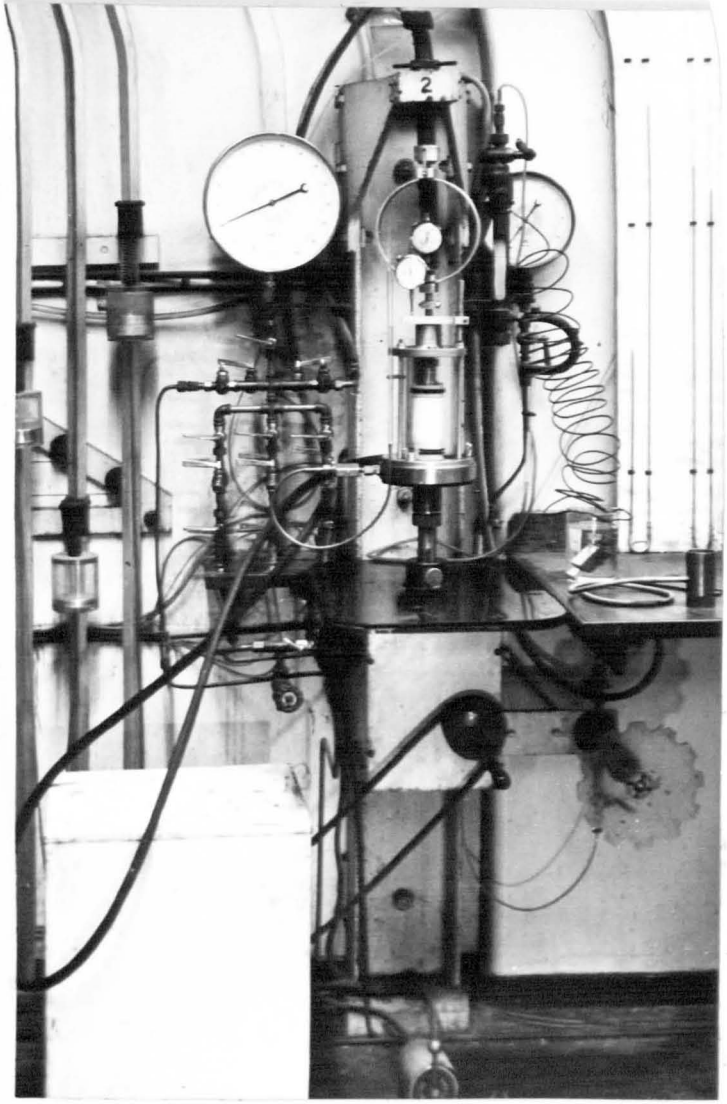


PLATE I. TRIAXIAL TEST IN CONSTANT RATE OF STRAIN COMPRESSION MACHINE.

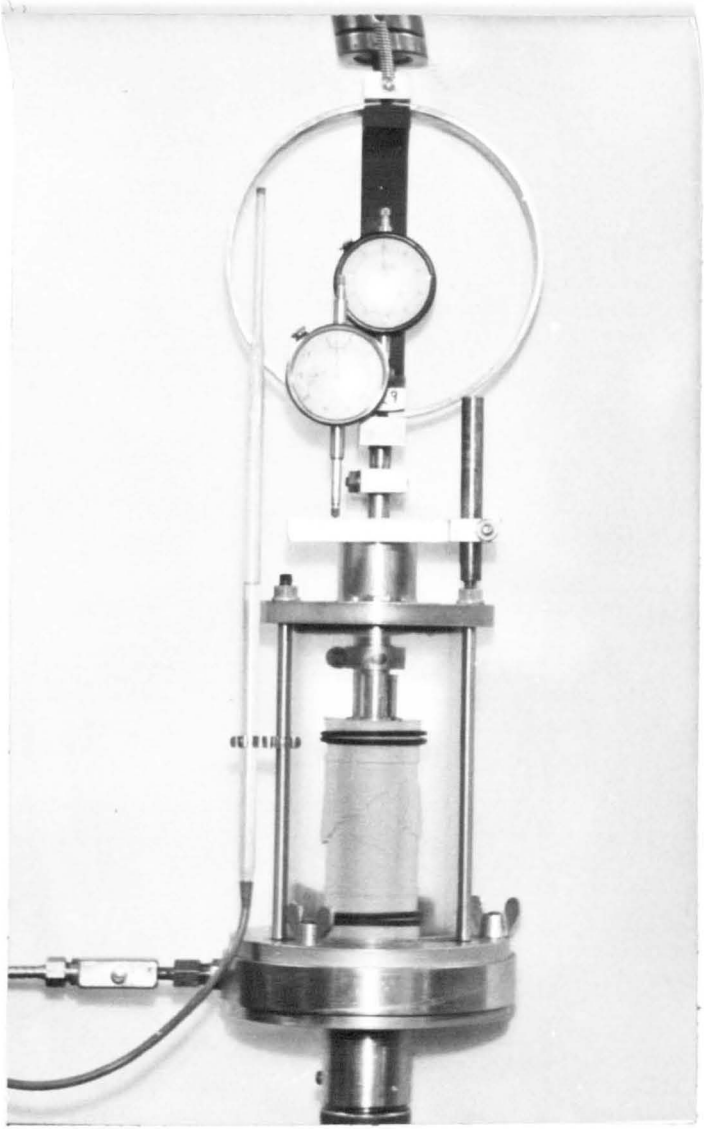


PLATE II. SAMPLE AT FAILURE IN DRAINED TEST UNDER INCREASING σ_1 STRESSES. AS IN PLATE I

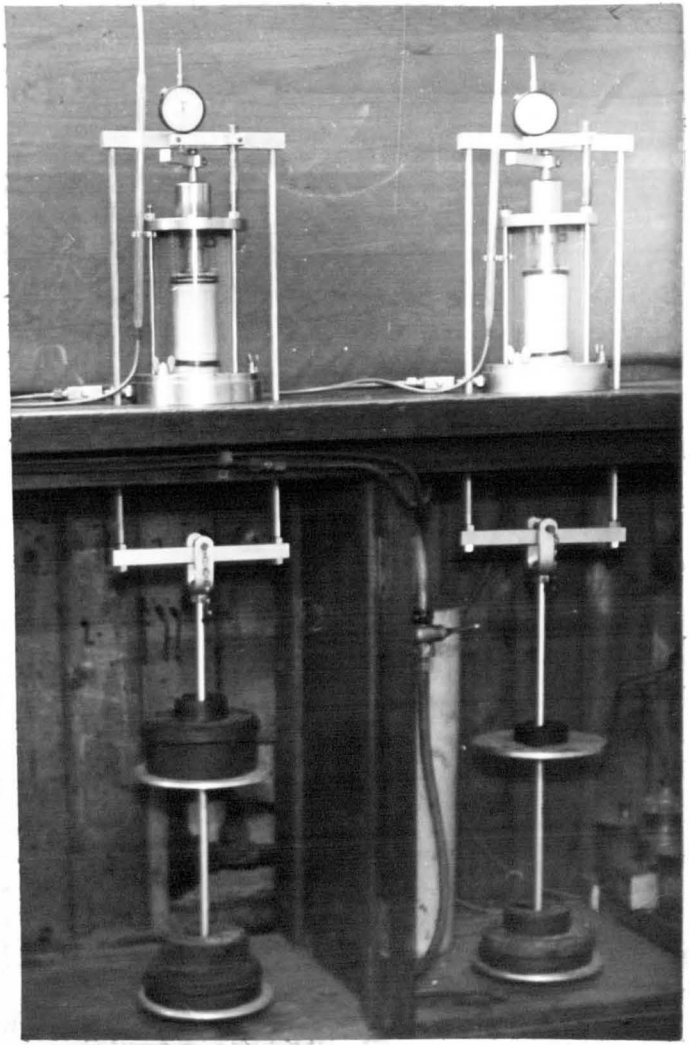


PLATE III. DRAINED TRIAXIAL TEST UNDER CONSTANT σ_1 STRESSES.

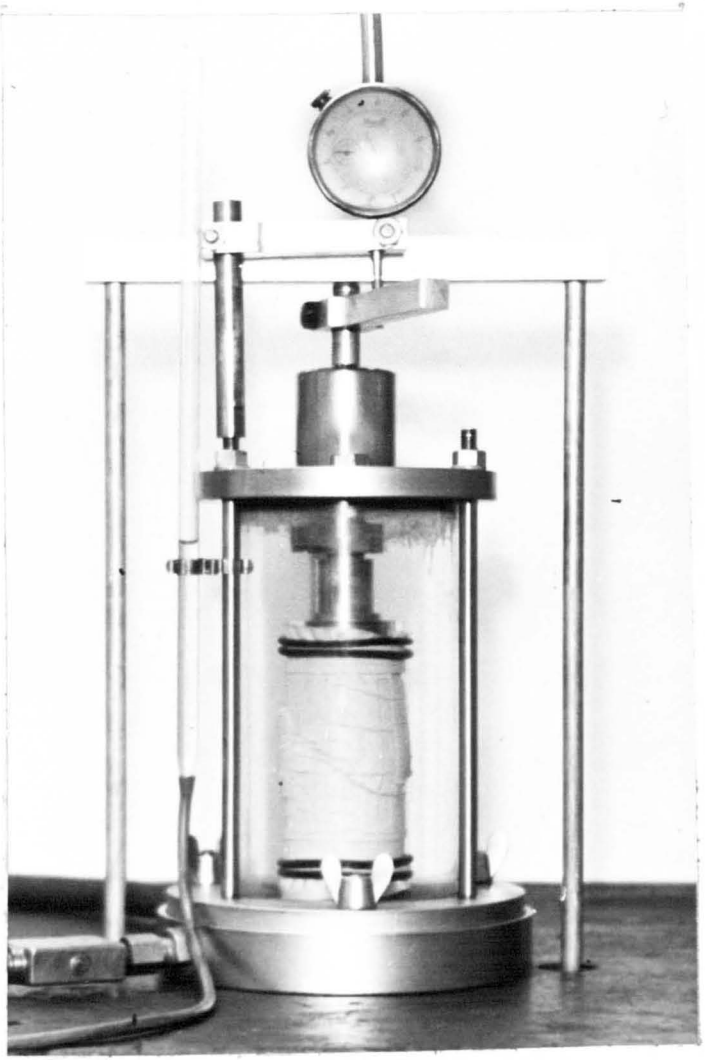


PLATE IV. SAMPLE AT FAILURE IN CONSTANT σ_1 DRAINED TEST AS IN PLATE III.

LABORATORY PROCEDURE

The majority of samples used in the tests were undisturbed samples and arrived at the laboratory in the 4 in. diameter sampling tubes in which they had been taken. To prepare the samples for testing a portion of the clay was extruded from the tube into a 4 in. diameter brass mould. Three 1½ in. diameter thin walled brass tubes with sharpened ends were then jacked into the undisturbed clay in the mould so that three 1½ in. diameter samples were taken. At least two moisture content samples were taken from the clay left in the mould and the remainder discarded. The clay in the 1½ in. diameter tube was next squared off at each end to 3 in. length, while still in the tube. Moistened filter paper discs were placed on each end of the sample and it was extruded, then weighed and the height measured.

The base of the cell had been de-aired by inverting it in an evaporating dish of de-aired water and sucking water through it by means of a vacuum line attached to the burette. After de-airing the base was placed upright and a porous disc, which had been de-aired by being kept in water that was boiling for at least 15 minutes, was placed on top of it. The sample was then positioned on the disc and a similar disc placed on its top. A filter paper drain, which had

been moistened, was then set in place and a rubber membrane put around the sample and sealed to the cell base by two rubber rings. Air between the membrane and the sample was removed by flooding the space with water from the burette and gently forcing any air bubbles to the top by a light touch. After the air had been removed, the top cap was placed on the sample and the two rubber rings to seal the membrane to the cap added. A negative head of about one foot of water was put on the sample, by lowering the burette, to drain off surplus water between the sample and the membrane. About a minute was usually sufficient to complete this drainage, the tap to the burette was then closed.

With the sample now ready, the cell top was put in position and the cell filled with de-aired water. Machine oil of medium density was added on top of the water, to complete the last $\frac{1}{8}$ to $\frac{1}{4}$ in. of filling the cell, so that leakage around the piston during the test would be minimized. Cell pressure was applied by means of the air/water cylinder and then maintained at the desired level by the mercury cylinder system which had been pre-set to give the required pressure.

The sample was now ready for consolidation under the cell pressure, and this was allowed to proceed by opening the valve to the burette. Readings of water movement

on the burette were taken on a $\sqrt[2]{t}$ basis and it was found that equilibrium was usually reached in less than 24 hours. Once equilibrium had been attained the sample was ready for testing.

Drained tests of the increasing σ_1 type were carried out in the constant rate of strain loading frame at a rate such that full drainage would occur, the necessary rate being calculated as given by Gibson and Henkel (1954). Average strain at failure in this type of test was about 5% and the rates of strain used were such that this strain was usually reached in two to three days. As it was necessary to keep the test going continuously the readings during the night had to be omitted. However, in some cases, the rate of strain was reduced overnight if it appeared failure would occur while the sample was unattended. Although complete stress-strain and volume change curves cannot be drawn with such a procedure, enough points are obtainable to draw the curves without appreciable error and by judicious timing the failure points can be reached while the test is attended.

Undrained tests with increasing σ_1 and porewater pressure measurements were carried out in the same manner except that instead of allowing drainage during the test the burette was removed and the sample connected to the porewater pressure measuring device. When the test continued

overnight it was necessary to close off the porewater pressure measurement, but since this operation resulted in no volume change and the pressure could be estimated closely and pre-set, there was no effect on the sample.

Drained tests under constant σ_1 with decreasing σ_3 stresses were carried out by using dead loading on the top of the sample to keep σ_1 constant as the cell pressure was decreased. The most practical method in a test of this type is to apply the loads in increments and adjust the cell pressure to the suitable value. The increments of loading were arranged so that each was about 10% of the estimated load at failure, up to 70 or 80% of the total, increments of about 5% of the load were then used until about 90% total load when the increments were cut down to 2 or 3%, (see Fig. 5). In this manner a relatively smooth stress strain curve was obtained. Usually equilibrium under an increment of loading was reached in three or four hours but this varied with different clays and with the position of the increment in the stress-strain curve, i.e. increments at higher loads usually take longer to come to equilibrium due to the increased strain they cause. Increments of load were never added at a rate of more than two per day and in some cases only one per day, hence the time to failure was usually one to two weeks which is sufficient to allow complete

drainage.

Only one test was done measuring porewater pressure under constant σ_1 conditions and it was carried out in a constant rate of strain loading frame, the cell pressure (σ_3) being lowered to keep σ_1 constant as the load measured by the proving ring increased. As a test of this sort requires constant attention in adjusting the cell pressure it was necessary to stop it overnight and maintain the vertical load by dead loading. As far as could be seen, no ill effects resulted from this procedure.

Upon completion of the tests the tap at the base of the cell connecting to the sample was closed to prevent moisture change, load was then removed, the cell pressure released and the cell fluid removed. After removal of the sample from the cell, the rubber membrance and filter drain were removed and the sample weighed. The angle of the shear plane, if one occurred, was measured and three moisture contents taken, one each from near the top, centre and bottom of the sample and, in addition, moisture contents of the clay on both sides of the shear plane were taken.

Two tests were also run on samples under constant σ_1 conditions and alternating stresses by changing the porewater pressure. The tests are considered separately

at the end of the general test results.

TEST RESULTS

A summary of the results of the tests is given in Table I for London clay and in Table II for other clays. Typical stress-strain and volume change curves are given for tests on the same soil under increasing σ_1 and constant σ_1 stresses in Fig. 4 and Fig. 5. As pointed out earlier, some scatter of test results was to be expected due to lack of uniformity in the undisturbed samples. However, in most cases it was possible to draw an envelope which was very nearly tangent to three Mohr circles. In a few cases an envelope could not be drawn as the circles were very erratic and it is notable that this occurred with the most intensely fissured samples.

VOLUME CHANGE

Research on saturated clays at Imperial College has shown that volume change during shear under drained conditions may be divided into two components:

LAB. No.	TYPE AND ORIGIN OF CLAY	DEPTH	L.L.	P.L.	P.I.	INITIAL M.C.	γ LBS/CU.FT.	TYPE OF TEST	CONSOL. PRESS.	σ_3' AT FAILURE	$(\sigma_1-\sigma_3)_F$ LBS./SQ.IN	FINAL AVERAGE W %	$\angle$ OF PLANE	W AT PLANE	ϕ'	c' LBS./SQ.FT.
1	LONDON CLAY, REMOULDED		83	26	57	56·2	104	DRAINED, CONSTANT σ_1	120/5	0·5		37·7	64°	–	17°-30'	340
2	" " "					54·3	105	" " "	120/15	3·3	10·4	35·9	45 & 60°			
3	" " "					54·8	106	" " "	120/30	11·5	15·8	33·5	55°			
4	" " "					55·7	104	" " "	120/60	28·5	28·0	31·3	53°			
5	BROWN L.CLAY, GRESHAM ST. B.H.4.	28'-0"	86	27	59	30·1	122	DRAINED, CONSTANT σ_1	30	12·6	14·8	30·3	53°		19°-30'	210
6	" " " "	"				30·1	122	" " "	15	4·3	9·4	33·1	47°			
7	" " " "	"				30·1	122	" " "	60	24·5	30·3	29·8	51°			
8	BROWN L. CLAY, GRESHAM ST. B.H.4.	27'-0"	86	27	59	29·0	123	DRAINED, INCREASING σ_1	5	5	12·5	32·2	40° & 51°		19°	350
9	" " " "	"				29·0	123	" " "	15	15	23·4	30·4	45° & 52°			
10	" " " "	"				29·0	123	" " "	30	30	36·2	29·2	54°			
11	BROWN L. CLAY, GRESHAM ST. B.H.4.	27'-6"	86	27	59	30·3	122	DRAINED, CONSTANT σ_1	15	5·2	8·6	33·1	47°	34·2		
12	" " " "	"				29·5	122	" " "	30	13·5	14·1	30·8	48°	31·6		
13	BROWN L. CLAY, UXBRIDGE No. 24	7'-6"	82	28	54	28·1	120	DRAINED, INCREASING σ_1	5	5	11·4	32·5	51°		20°	290
14	" " " "	"				28·1	119	" " "	15	15	20·7	30·5	49°			
15	" " " "	"				28·1	120	" " "	30	30	37·0	29·2	52°			
16	BLUE L. CLAY, UXBRIDGE No. 78	22'-6"	74	25	49	26·6	123	DRAINED, INCREASING σ_1	5	5	10·8	29·8	51° & 57°		20-30'	270
17	" " " "	"				26·6	123	" " "	15	15	23·3	27·6	47° & 49°	28·0		
18	" " " "	"				26·6	123	" " "	30	30	37·4	26·1	50° & 52°			
19	BLUE L. CLAY, UXBRIDGE No. 78	22'-0"	74	25	49	27·1	126·0	DRAINED, INCREASING σ_1	5	5	11·6	30·5	40° & 46°	32·1	NOT SIMILAR TESTS	
20	" " " "	"				27·1	126·1	DRAINED, CONSTANT σ_1	10	2·2	7·3	30·2	38° & 49°	30·7		
21	" " " "	"				27·1	125·2	DRAINED, ALTERNATING STRESSES	19·1	2·6, 4·8	6·2	33·0	45° & 47°	37·3		

LABORATORY TEST RESULTS FOR LONDON CLAY

TABLE I
SHEET I

LAB. No.	TYPE AND ORIGIN OF CLAY	DEPTH	L.L.	P. L.	P. I.	INITIAL M.C.	γ LBS/CU.FT.	TYPE OF TEST	CONSOL. PRESS.	σ_3' AT FAILURE	$(\sigma_1-\sigma_3)_F$ LBS./SQ.IN.	FINAL AVERAGE W %	$\angle$ OF PLANE	W AT PLANE	ϕ'	c' LBS/SQ.FT.
22	BROWN L. CLAY, UXBRIDGE No.24	7'-0"	82	28	54	27.6	124	DRAINED, CONSTANT σ_1	15	3.9	9.9	33.8	50°		21°-30'	280
23	" " " "	"				27.6	124	" "	35	11.6	20.6	31.7	50°	34.0		
24	" " " "	"				27.6	125	" "	70	27.8	39.0	28.0	CURVED	29.1		
25	BLUE L. CLAY, SHELL BLDG. No. 02	43'-6"	84	29	55	32.3	121	DRAINED, CONSTANT σ_1	10	11.9	7.7	34.4	45° & 49°	36.6	24°	210
26	" " " "	"				32.3	120	" "	20	5.9	12.8	32.9	42° & 46°	33.7		
27	" " " "	"				32.3	121	" "	40	13.9	24.3	31.3	—	31.6		
28	BLUE L. CLAY, SHELL BLDG. No. 02	43'-0"	84	29	55	31.8	121	DRAINED, INCREASING σ_1	2	2	7.4	33.5	47°	34.4	22°	260
29	" " " "	"				31.8	121	" "	6	6	14.1	33.2	45°	34.5		
30	" " " "	"				31.8	121	" "	16	16	24.6	31.4	45°	32.0		
31	BLUE L. CLAY, SHELL BLDG. No. 02	42'-6"	84	29	55	31.5	121	DRAINED, ALTERNATING STRESSES	16.7	2.0, 5.0	7.0	34.8	40° & 55°	40.0	NOT SIMILAR TESTS	
32	" " " "	"				31.5	121	UND. WITH P.W.P. MEAS'D, INCREASING σ_1	6	5.0	11.3	33.8	46°	36.8		
33	" " " "	"				31.5	121	UND. WITH P.W.P. MEAS'D, CONSTANT σ_1	20	12.7	19.6	32.0	45°	35.0		
34	BLUE L. CLAY, NORTHOLT	12'-0"	81	25	56	29.6	122	DRAINED, CONSTANT σ_1	10	0.4	9.8	34.2	IRREGULAR	—	CIRCLES VERY IRREGULAR	
35	(HIGHLY FISSURED)	"				29.6	123	" "	20	5.8	13.1	30.5	57°- 61°	32.8		
36	" "	"				29.6	123	" "	40	11.1	27.3	32.2	57°	33.0		
37	BROWN L. CLAY, SUDBURY HILL	6'-0"	82	29	53	32.2	121	DRAINED, CONSTANT σ_1	10	2.0	7.2	34.9	48° & 50°	38.1	23°	320
38	(HIGHLY FISSURED)	"				32.2	120	" "	20	4.9	14.0	34.7	45°- 60°	-		
39	" " "	"				32.2	120	" "	40	13.8	24.8	31.1	45° & 52°	31.1		

LABORATORY TEST RESULTS FOR LONDON CLAY

TABLE I SHEET II

TABLE I
SHEET II

LAB. No.	TYPE AND ORIGIN OF CLAY	DEPTH	L.L.	P.L.	P.I.	INITIAL w %	γ	TYPE OF TEST	CONSOL. PRESS. LBS/SQ. IN.	σ_3' AT FAILURE LBS/SQ.IN.	$\sigma_1-\sigma_3$ AT FAILURE LBS/SQ.IN.	FINAL w. %	∠ OF PLANE	w. AT PLANE	ϕ'	c' LBS/SQ.FT.
T1	BOULDER CLAY, TYNEMOUTH TUBE 2	—	30	15	15	15·4	136	DRAINED, INCREASING σ_1	5	5	9·5	16·2	NO PLANE	—	28°-30'	NIL
T2	" " REMOULDED	—				15·4	136	" "	30	30	55·4	13·4	"	—		
T3	" " " "	—				15·4	136	" "	15	15	29·7	14·2	"	—		
T4	BOULDER CLAY, TYNEMOUTH TUBE 2		30	15	15	14·1	137	DRAINED, CONSTANT σ_1	10	2·2	7·2	17·3	53°	—	30°-30'	NIL
T5	" " REMOULDED					14·1	138	" "	40	11·8	25·7	15·1	NO PLANE	—		
T6	" " "					14·1	137	" "	80	23·4	50·2	13·6	"	—		
T7	BOULDER CLAY, TYNEMOUTH TUBE 2		31	18	13	22·5	126	DRAINED, CONSTANT σ_1	15	3·7	10·4	21·0	NO PLANE	—	28°	75
T8	" " "					22·5	128	" "	35	11·6	22·8	22·1	49°	23·0		
T9	" " "					22·5	127	" "	70	22·8	44·9	17·9	50°	24·9		
G1	GAULT CLAY, LUTON B.H. 3, No.3.	7'-0"	53	18	35	19·4	132	DRAINED, INCREASING σ_1	5	5	12·5	23·5	45°	—	25°	190
G2	" " " "					19·4	132	" "	15	15	25·7	21·4	VARIOUS	—		
G3	" " " "					19·4	132	" "	30	30	49·5	20·5	VARIOUS	—		
G4	ABOVE 3 SAMPLES REMOULDED		53	18	35	38·1	115	DRAINED, INCREASING σ_1	5	5	7·1	32·2	NO PLANE	—	20°-30'	NIL
G5	" " "					38·1	114	" "	15	15	19·5	27·0	"	—		
G6	" " "					38·1	115	" "	30	30	33·0	24·5	"	—		
L1	LIAS CLAY, LONDON-YORKS. ROAD B.H.11.		56	24	32	44·5	110	DRAINED, INCREASING σ_1	5	5	8·7	34·7	NO PLANE	—	23°	NIL
L2	" " " REMOULDED					44·5	111	" "	15	15	20·8	29·5	"	—		
L3	" " " "					44·5	112	" "	30	30	40·3	26·2	"	—		

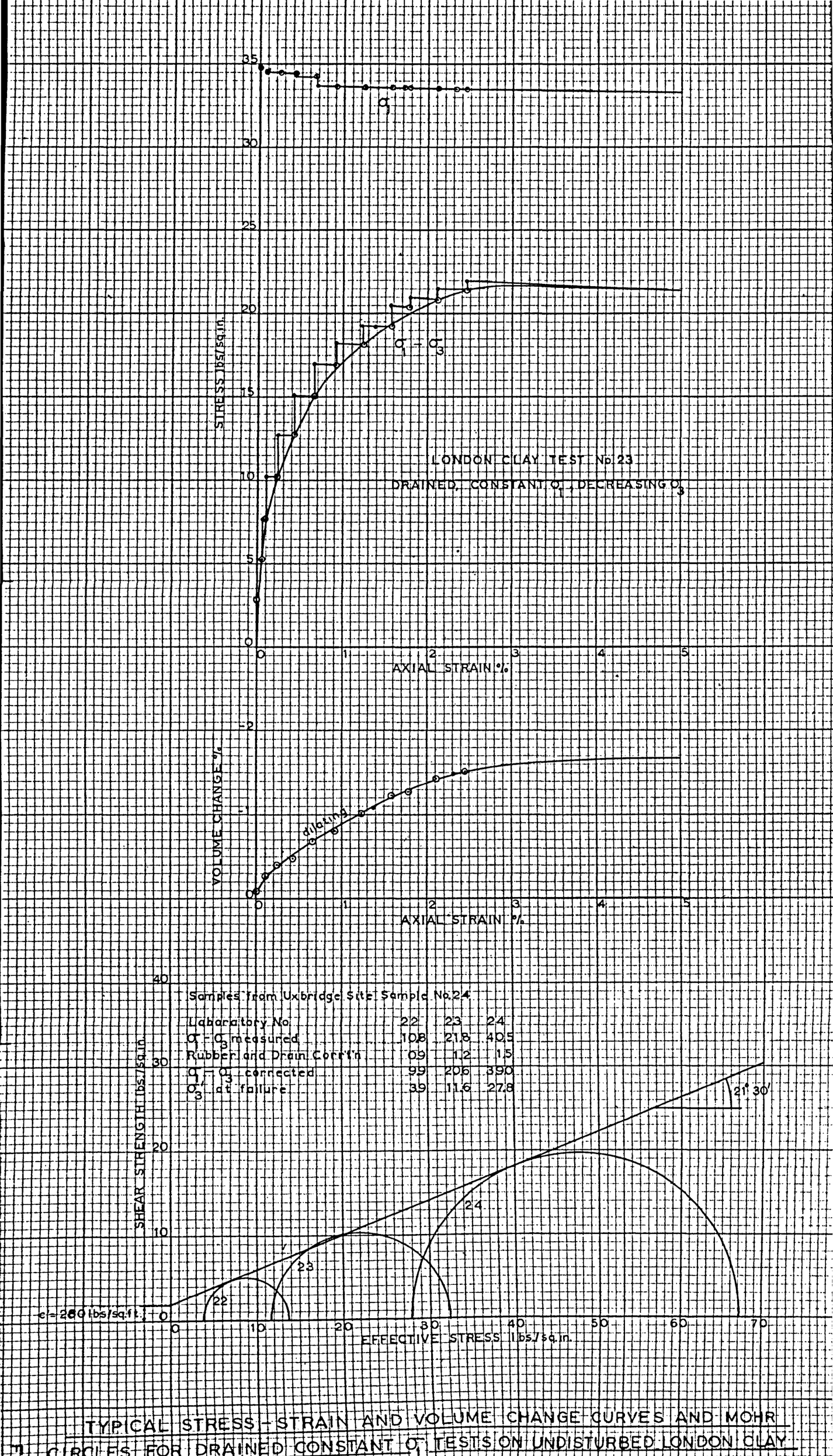
LABORATORY TEST RESULTS
ON CLAYS OTHER THAN LONDON CLAY

TABLE II

TYPICAL STRESS-STRAIN AND VOLUME CHANGE CURVES AND MOHR CIRCLES FOR DRAINED INCREASING σ_1 TESTS ON LONDON CLAY

FIG. 4

FIG. 4



TYPICAL STRESS - STRAIN AND VOLUME CHANGE CURVES AND MOHR CIRCLES FOR DRAINED CONSTANT σ_1 TESTS ON UNDISTURBED LONDON CLAY

FIG. 5

(1) a volume change associated with change in average effective stress in the sample;

(2) a volume change due to the dilatency characteristics of the clay.

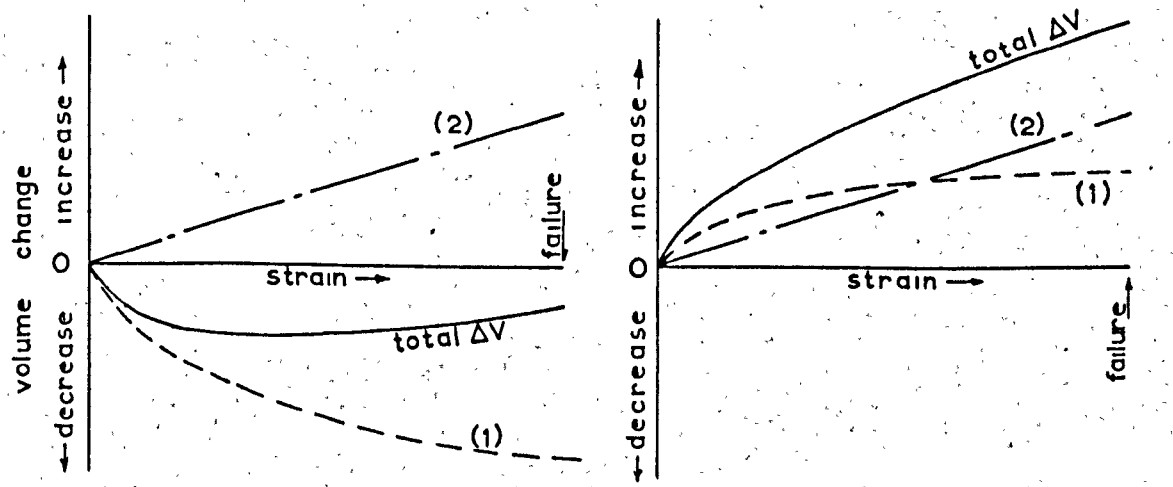


FIG. 6. VOLUME CHANGES IN HEAVILY OVER-CONSOLIDATED CLAY UNDER SHEARING STRESSES

FIG. 6

In heavily over-consolidated clays the dilatency component of volume change leads to an increase in the volume of the sample as shown by (2) in Fig. 6 A and B. When shear tests are carried out such that there is an increase in average effective stress the first component of volume change leads to a decrease in volume as shown by (1) in Fig. 6 A, and the total volume change is similar to that

shown. When the average effective stress decreases during a shear test it leads to an increase in the volume as shown by (1) in Fig. 6 B and the total volume change is an increase as shown.

In all tests listed in Table I the sample was found to be dilating at failure as indicated by the burette reading. Total volume change indicated an increase in volume for all tests under constant σ_1, with decreasing σ_3 conditions. The greater number of samples tested under increasing σ_1 with constant σ_3 stresses showed a decrease in total volume at failure; however, some of these tests at low cell pressures (i.e. high over-consolidation ratios) showed an increase in volume at failure. Weights of all samples, after completion of testing, indicated that volume had increased irrespective of how the stresses were applied and of the total change as shown by the burette. The discrepancy apparently comes about by leakage of moisture through the rubber membrane and the absorption of water during the setting up process, neither of which is indicated by the burette.

All tests on samples of undisturbed over-consolidated clay were found to fail by shear along one or more well-defined plane (see, for instance, Plate VI) as opposed to the bulging type of failure obtained with remoulded material

over-consolidated in the laboratory. Moisture contents about the shear planes were measured in a considerable number of samples, as given in Tables I and II, and in all cases found to be higher than the average moisture content in the remainder of the sample. This higher moisture content about the failure plane indicates that dilatancy is concentrated in a small portion of the sample adjacent to the plane. Observations in the field, see Figs. 27 and 30 , show that this is also the case in full scale failures. It is therefore apparent that volume measurements on this type of sample as a whole are not indicative of the true state of affairs within the sample, particularly at the shear plane. For, whereas the volume change of the sample as a whole may be shown to be positive by the burette readings, as it is in most increasing σ_1 tests, that at the portion under shear is always negative irrespective of the method in which stress is applied.

It is not possible to show whether the increase in moisture content about the shear surface is more for one type of test than the other due to the practical difficulty of taking truly representative samples. The usual procedure was to take a layer of clay 1/16 to 1/8 in. thick from each side of the surface and average the results.

Since the shear surface is never perfectly plane and it is never known how far from it the zone of dilatancy exists (at Northolt a field case, the zone was found in places to be on one side of the failure surface only) it cannot be certain that the moisture content measured about the plane is exactly correct. Any error, however, is most certainly tending to minimise the differences in moisture content.

UNDRAINED TESTS WITH P.W.P. MEASURED

Porewater pressures have been measured in clays over-consolidated in the laboratory and it has been found that negative porewater pressures are developed at high over-consolidation ratios (Gilbert 1954, Plant 1956). The values measured, however, are considerably smaller than those which must exist in the clay in an undisturbed state to give it the undrained strength it normally has. Tests of this type are usually performed by increasing σ_1 stresses, but clay in situ in a slope or behind a retaining wall is usually stressed under conditions closer to the constant σ_1 reducing σ_3 method.

To determine what difference would result from

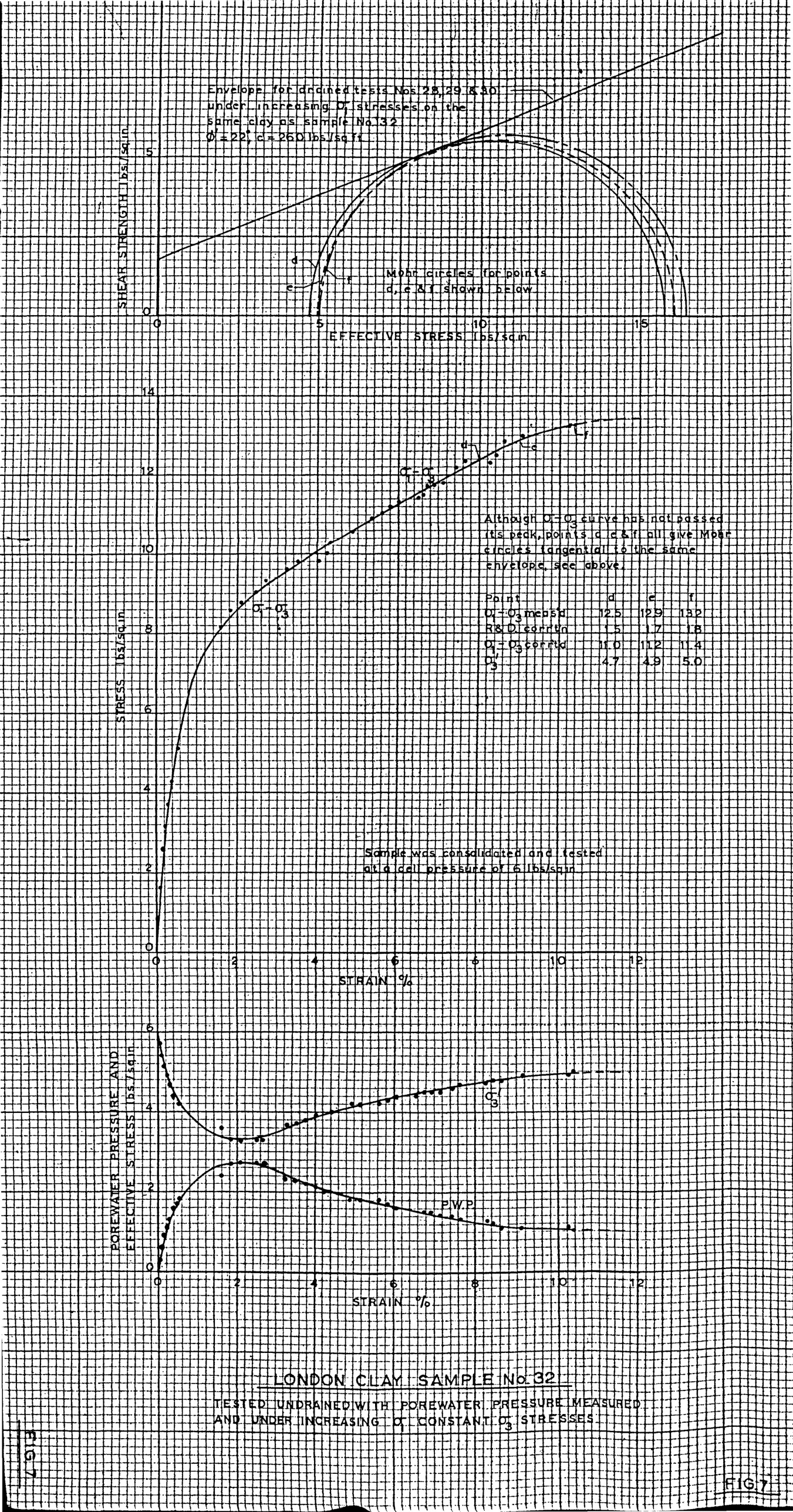
the method of testing, two tests were done under undrained conditions with porewater pressure measured, failure in one case being reached by increasing σ_1 in the other by decreasing σ_3 with σ_1 constant. Both tests were done at a rate of speed comparable to the drained tests. The increasing σ_1 test took five days to failure at 10% strain and the constant σ_1 test was conducted over the same period, but as it was done in the constant rate of strain compression machine the cell pressure required constant attention and the machine had to be stopped each night. While stopped, the latter test was dead loaded so that stresses remained constant. Total running time to failure at 10% strain was therefore 60 hours.

Stress-strain and porewater pressure change - strain curves for these two tests are given in Figs. 7 and 8. It will be noted that the porewater pressure change is positive in the increasing σ_1 test while it is negative in the constant σ_1 test. This would indicate that considerably greater negative porewater pressures are set up when heavily over-consolidated clay is stressed under constant σ_1 with decreasing σ_3 conditions, as it approximately is in the slope of a cutting or behind a retaining wall, than when the clay is stressed under increasing σ_1 with constant σ_3 conditions. Although

porewater pressure changes were of the opposite sign moisture content about the planes in both these tests was found to be considerably higher than that in the remainder of the sample indicating dilatancy at the planes. Increased moisture contents of this nature in a drained test can only come about at the expense of moisture in other parts of the sample. Therefore, there must be moisture migration within the sample and it is logical that undrained tests with porewater pressure measurement should be carried out at the same speed as drained tests to allow for the moisture movement.

Mohr circles in terms of effective stress for these two undrained tests are given in Fig. 10 A (circles 32 and 33) and indicate that the shear strength is of the same order as the drained tests, not higher, as normally obtained. Although conclusions of general validity cannot be drawn from the results of two tests, there is a distinct possibility that undrained tests carried out at the same speed as drained tests tend to give the same effective shear strength.

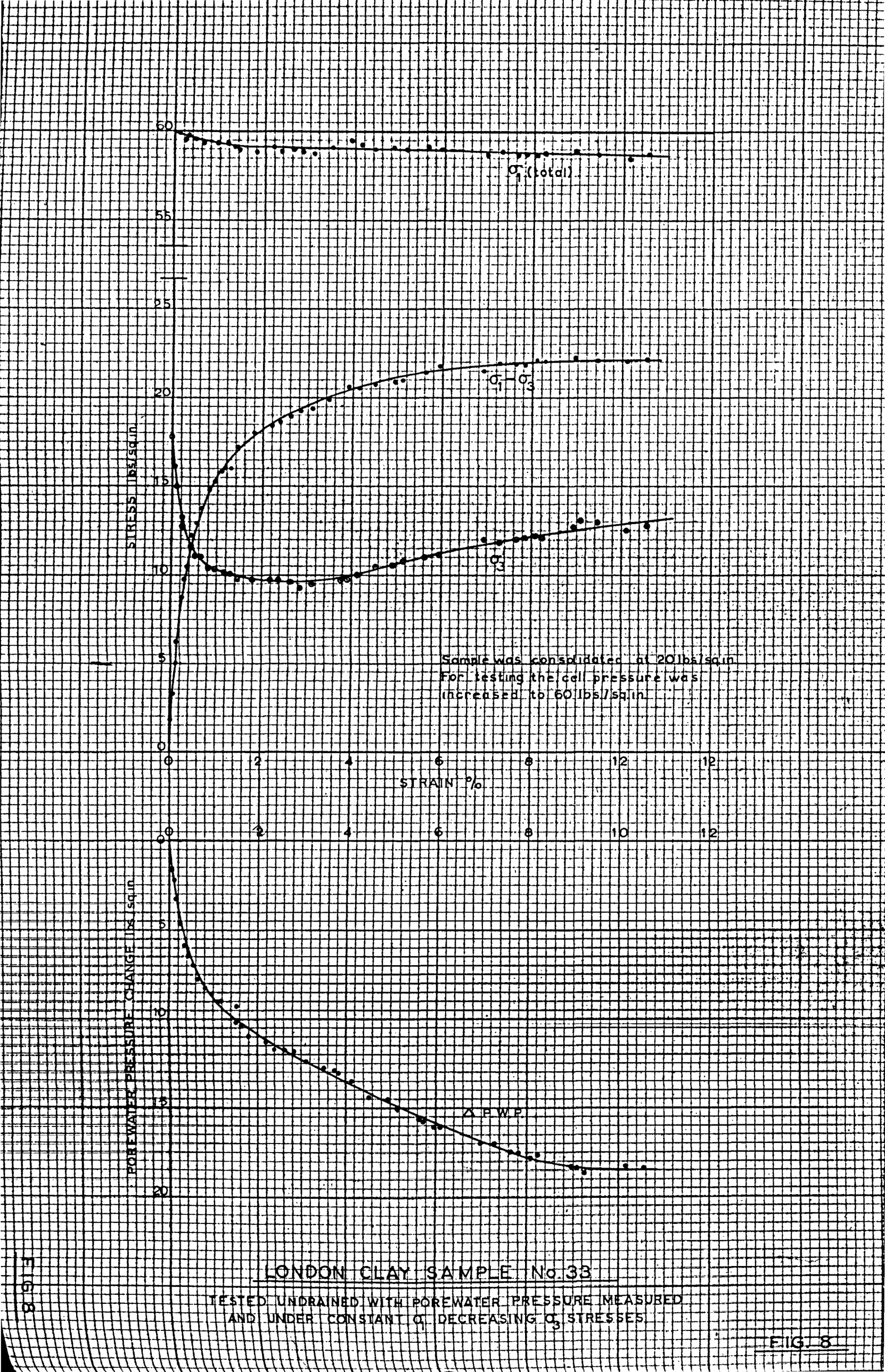
There can be no assurance that the porewater pressure measured at the base of the sample is that acting on the shear plane and it is this latter pressure which determines the shear strength of the sample when failure occurs on a plane. It has been shown (M.I.T., 1951)



LONDON CLAY SAMPLE No. 32

TESTED UNDRAINED WITH POREWATER PRESSURE MEASURED AND UNDER INCREASING σ_3' CONSTANT σ_3 STRESSES

FIG 7



LONDON CLAY SAMPLE No. 33

TESTED UNDRAINED WITH POREWATER PRESSURE MEASURED AND UNDER CONSTANT σ_1 DECREASING σ_3 STRESSES

FIG. 8

8.6.73

that considerably different porewater pressures can exist throughout a sample tested to failure at 10% strain in about one hour. However, the times to failure used in the two tests recorded here should tend to diminish these differences considerably.

CONSTANT σ_1 TESTS

Table III, page 38, gives a comparison of the shear strength parameters obtained with increasing σ_1 and constant σ_1 tests. The tests compared, were on material from the same sampling tube but of necessity from different levels in the tube, about 5 in. apart.

Indications are that $\emptyset'$ is from one to two degrees higher on the average and c' slightly lower for constant σ_1 tests. The same trend, although to a lesser extent, was found by Parry (1956), for remoulded London clay over-consolidated in the laboratory.

In consideration of these results it must be borne in mind that the loads are applied by different methods in the different tests. The increasing σ_1 loads are applied steadily in the compression machine while the constant σ_1 tests are loaded by increments.

CLAY	INCREASING σ_1		CONSTANT σ_1		DIFFERENCE OF CONSTANT σ_1	
	Ø'	c' lb./ sq.ft.	Ø'	c' lb./ sq.ft.	Ø'	c' lb./ sq.ft.
UNDISTURBED LONDON CLAY						
Gresham St.	19°	350	19½°	210	+½°	-140
Uxbridge No.24	20°	290	21½°	280	+1½°	-10
Shell Bldg. No.02	22°	260	24°	210	+2°	-50
TYNEMOUTH BOULDER CLAY, Remoulded	28½°	nil	30½°	nil	+2°	nil

TABLE III. COMPARISON OF SHEAR STRENGTH PARAMETERS FROM INCREASING σ_1 AND CONSTANT σ_1 TESTS.

Loads applied in unlike ways may cause different reactions in the sample which give a false indication of the parameters. Loading by increments can give an error of one half the increment causing the failure, which in the case of these tests could amount to 2% of the shear strength. This error would always be on the high side as the increment causing failure can only be larger than necessary. Friction in the triaxial cell is also probably greater in the case of constant

σ_1 tests since the plunger is allowed to come to rest between increments of load and is not kept continuously moving as in the increasing σ_1 tests. Both these errors tend to give a higher than true shear strength to the constant σ_1 tests.

Considering these points and the small size of the difference obtained no definite opinion can be formed from testing results to date whether or not constant σ_1 tests give a higher $\emptyset'$. It is logical therefore, at the present time to consider $\emptyset'$ as being independent of the method of application of the stresses and equal to the value obtained in the increasing σ_1 tests.

DRAINED SHEAR STRENGTH PARAMETERS FOR LONDON CLAY

Table IV contains the results of drained tests on undisturbed London clay samples, similar to that which might be encountered in a slope or retaining wall problem as well as results of drained tests on remoulded clay. In compiling the list the only known fully drained tests which have been omitted are on samples from deep borings at Paddington (Henkel, 1954) taken at a depth greater than has been encountered in any slope problem and the

DRAINED TESTS ON UNDISTURBED LONDON CLAY

SITE	TYPE OF CLAY	INITIAL WATER CONTENT	L.L.	P.L.	σ_1 INCREASING		σ_1 CONSTANT		AVERAGE FOR SITE		SOURCE OF DATA
					c' LBS/SQ.FT.	ϕ'	c' LBS/SQ.FT.	ϕ'	c' LBS/SQ.FT.	ϕ'	
SYDENHAM HILL	BROWN	32	83	26	210	19°	–	–	220	19½°	SKEMPTON & HENKEL 1955
"	BLUE	33	82	28	220	20°	–	–			" " "
QUEEN VICTORIA ST.	BROWN	33	89	26	220	22°	–	–	220	21°	HENKEL, 1953
"	BLUE	32	95	29	220	20°	–	–			" "
CHINGFORD	BROWN	34	82	29	230	19°	–	–	230	19°	HENKEL, 1953
GRESHAM ST.	BROWN	29	86	27	350	19°	210	19½	280	19°	TABLE I
UXBRIDGE (24)	BROWN	28	82	28	290	20°	280	21½°	280	20½°	TABLE I
" (78)	BLUE	27	74	25	270	20½°	–	–			
SHELL BLDG. (02)	BLUE	32	84	29	260	22°	210	24°	240	23°	TABLE I
OVERALL AVERAGE		31	84	27					250	20°	

UNDRAINED TEST WITH P.W.P. MEASURED, ON UNDISTURBED LONDON CLAY

CHINGFORD	BROWN	31	82	23	210	23			210	23°	HENKEL, 1953

-DRAINED TESTS ON REMOULDED LONDON CLAY

L.L.	P.L.	σ_1 INCREASING		σ_1 CONSTANT		SHEAR BOX		SOURCE OF DATA
		c'	ϕ'	c'	ϕ'	c'	ϕ'	
74	25	–	–	–	–	0	20½°	GIBSON, 1953
73	25	0	20°	–	–	–	–	HENKEL, 1954
73	25	–	–	–	–	0	20°	GIBSON, 1954
78	26	0	19°	–	–	–	–	PLANT, 1955
78	26	–	–	0	19°	–	–	PARRY, 1957
AVERAGE ϕ' = 20°								

TABLE IV

SUMMARY OF AVAILABLE TEST RESULTS IN TERMS OF EFFECTIVE STRESSES, FOR LONDON CLAY.

single result from Sudbury Hill (Table I). This latter sample was from intensely fissured, weathered London clay taken at a shallow depth, and as it was probably at times above the water table and suffered dessication, it is not representative.

For practical purposes therefore, the shear parameters can be taken as $\emptyset' = 20^{\circ}$ and $c' = 250$ lb./sq.ft. and it is these values which are used as a basis for the analysis of slope and retaining wall failures in Parts III and IV of this thesis.

ALTERNATING STRESSES

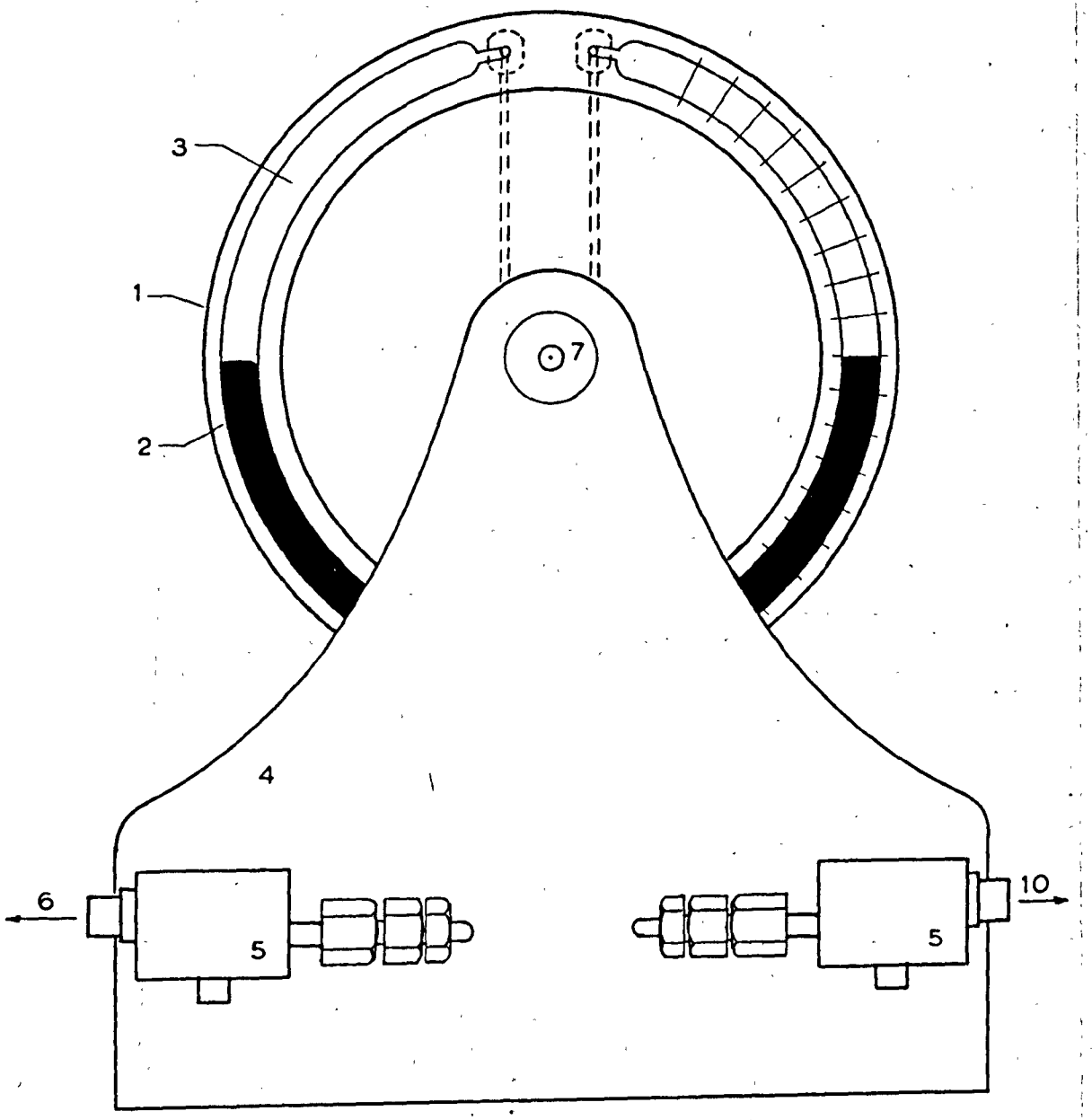
Observation has shown that the shear strength of London clay and some other heavily over-consolidated clays appears to diminish in time beyond the drained shear strength as shown by Fig. 1 B. This reduction is considered further in Parts III and IV and shown to be largely a lessening of the cohesion intercept. Although the reduction in shear strength is apparent in cases of failure in the field, samples tested in the laboratory, even though they be from the same site do not show any reduction and always indicate a finite c'.

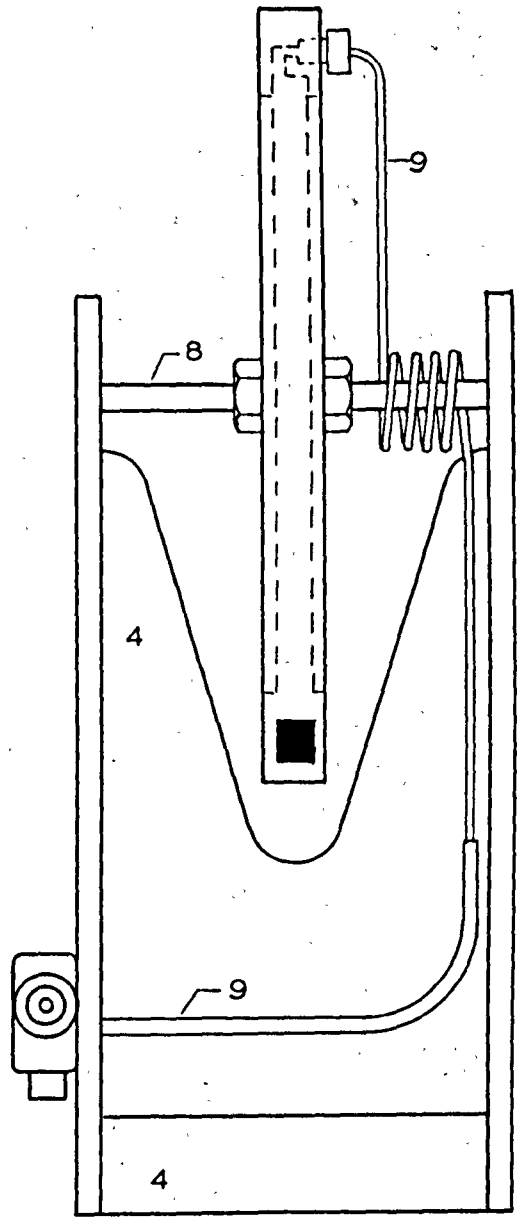
When in situ, for instance in a slope, soil is subjected to alternating stresses due to seasonal changes in the water levels and the drying out of the surface layer. As this type of stressing may cause alterations in the soil strength which are not found in the ordinary laboratory tests it was decided to subject samples to approximately the same conditions they would encounter in a slope or behind a retaining wall. This meant subjecting the sample to alternating stresses, by altering the porewater pressure, while the external loads remained constant and drainage was allowed to take place.

CONSTANT PRESSURE, VOLUME CHANGE APPARATUS

It was necessary to construct an apparatus which would allow drainage and measure its amount while still maintaining the porewater pressure in the sample at any desired value. The circular volume gauge designed for this purpose is shown in Fig. 9 and Plate V. In operation, a lead from one side of the wheel was attached to the connection on the base of the cell leading to the bottom of the sample. The lead from the other side of the wheel

FIG. 9





ARRANGEMENT OF

CIRCULAR VOLUME GUAGE

DRAWINGS ARE DIAGRAMATIC AT APPROXIMATELY 1/2 SIZE

1 Perspex Wheel
2 Mercury
3 De-aired Water
4 Stand
5 No Volume Change Tap
6 To Sample
7 Ball Bearing
8 Spindle
9 Plastic Tubing
10 To Mercury Cylinder Pressure System

In operation, pressure is transferred from the mercury cylinder system, through the fluid in the wheel to the sample. If volume change occurs in the sample, water moves from one side of the wheel. This tends to put the mercury in the wheel out of balance. To maintain balance the wheel rotates, the mercury remaining motionless. Volume change can then be read from the level of the mercury on a scale attached to and moving with the wheel.

FIG. 9



PLATE V. ARRANGEMENT FOR SUBJECTING SAMPLE TO ALTERNATING STRESSES.
CELL PRESSURE IS MAINTAINED CONSTANT THROUGH LEAD ON LEFT AND σ_1 IS MAINTAINED BY DEAD LOADING. PORE WATER PRESSURE IS ADJUSTED BY MERCURY POT SYSTEM ACTING THROUGH THE CIRCULAR VOLUME GAUGE TO PERMIT VOLUME CHANGE MEASUREMENTS.

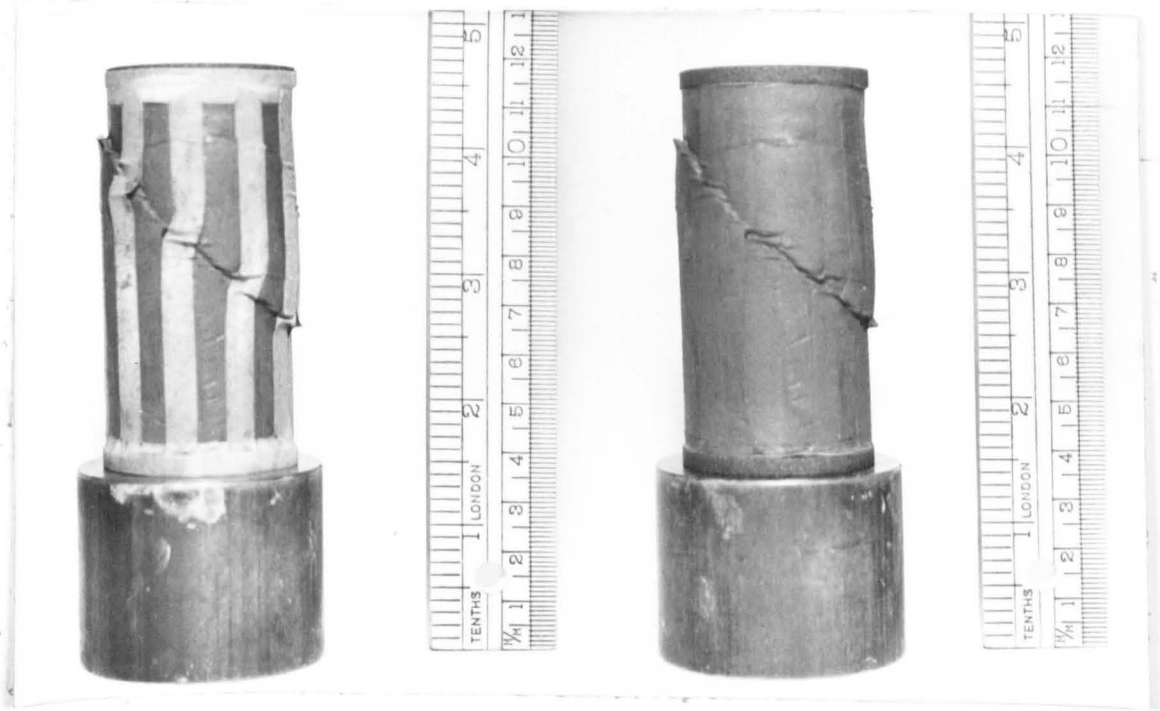


PLATE VI. TYPICAL FAILED SAMPLE OF LONDON CLAY (No. 29) AFTER REMOVAL FROM TRIAXIAL CELL.
THIS IS THE SAME SAMPLE BEFORE AND AFTER REMOVAL OF THE FILTER PAPER DRAIN. NOTE THAT THE DRAIN APPEARS TO HAVE BEEN COMPRESSED RATHER THAN SHEARED AND FAILURE HAS TAKEN PLACE PRINCIPALLY ALONG A SINGLE PLANE WITH A SLIGHT BULGING OF THE SAMPLE IN ADDITION.

was attached to the constant pressure mercury cylinder system which was adjusted to give the desired porewater pressure. This pressure was transmitted through the wheel of the volume gauge to the sample. If movement of water into or out of the sample occurred it tended to alter the level of the mercury on one side of the wheel thus putting it out of balance. The wheel rotates to maintain its balance thereby eliminating any difference in pressure through it. Volume change could be read by observing the elevation of the mercury on a scale attached to and rotating with the wheel.

CLAY USED IN ALTERNATING STRESS TESTS

As it was proposed to load the samples so that a considerable proportion of their shear strength was mobilized, only samples whose shear strength could be estimated closely were suitable. The first sample tested was No. 21, Table I. This test, however, ended in catastrophic failure due to accidental loss of cell pressure after 1½ months under alternating stresses. Strain under these stresses amounted to nearly 1% in that time. The second sample used was No. 31, Table I.

This sample was blue London clay from 42 ft. below the surface at the Shell Building site on the south bank of the Thames, see Skempton and Henkel (1957) for site location and details of boring. The clay in the tube was very uniform in appearance and yielded the same Atterberg limits from both ends of the tube. It was only moderately fissured.

Clay from the same tube, as the sample tested under alternating stresses, had been tested under both drained and undrained conditions. Mohr circles for these tests are given in Fig. 10 A.

PROCEDURE, ALTERNATING STRESS TEST

The sample was assumed to come from 20 ft. below the surface with a porewater pressure varying between 12 and 17 ft. head. It was set up and allowed to reach equilibrium under a cell pressure equivalent to the 20 ft. of surcharge and the porewater pressure of 17 ft. head. After equilibrium of the sample was reached (2 days) the cell pressure was lowered, dead load being added to keep σ_1 constant, so that the sample was loaded as indicated by the Mohr circle "c" in Fig. 10 B. This circle represents

60% of the failure load of the sample as indicated by the minimum Mohr envelope.

When equilibrium had been reached in both volume change and strain under these conditions the porewater pressure was altered so that the sample was loaded as indicated by Mohr circle "d" Fig. 10 B. This is the stress condition brought about by lowering the porewater pressure head on the sample from 17 to 12 ft. of water. These two positions were alternated when equilibrium was reached in each.

After ten such changes of stress it was found that there was no further movement and the sample was going through a cycle of consolidation and dilation with no resultant strain. Stress-strain and volume change, graphs are given in Fig. 11 where it will be noted that the strain reached under the 60% load was less than 1%.

As it appeared that further strain would not be reached at 60% of the probable failure load the cell pressure was reduced, keeping σ_1 constant, until approximately 85% of the probable failure load was applied. The stress on the sample is then that given by circle "e" Fig. 10 B. When equilibrium was reached the porewater pressure was changed so that the stress was that given by circle "f" Fig. 10 B. After equilibrium under either stress condition was reached, usually 24 hours was sufficient,

the other condition was substituted. The sample continued to strain as shown by the graph of Fig. 11 and after 26 changes of stress failed completely under the stress as given by circle "e". Total time to failure was 49 days with a total of 36 stress changes.

DISCUSSION OF ALTERNATING STRESS TEST

While results of general validity cannot possibly be drawn from two tests, the pattern of both showed they were acting in a similar manner. As with other drained tests stressed under constant σ_1 conditions there was dilation in the sample. This amounted to 3.7% volume change at the final strain of 10.6%. However, discrepancy between the initial weight plus the absorbed moisture and the final weight after the test indicates that volume change was probably over 5% increase. The additional water causing the discrepancy probably came from leakage through the rubber membrane as the test was of unusually long duration and some may have been absorbed by the sample in the setting up process.

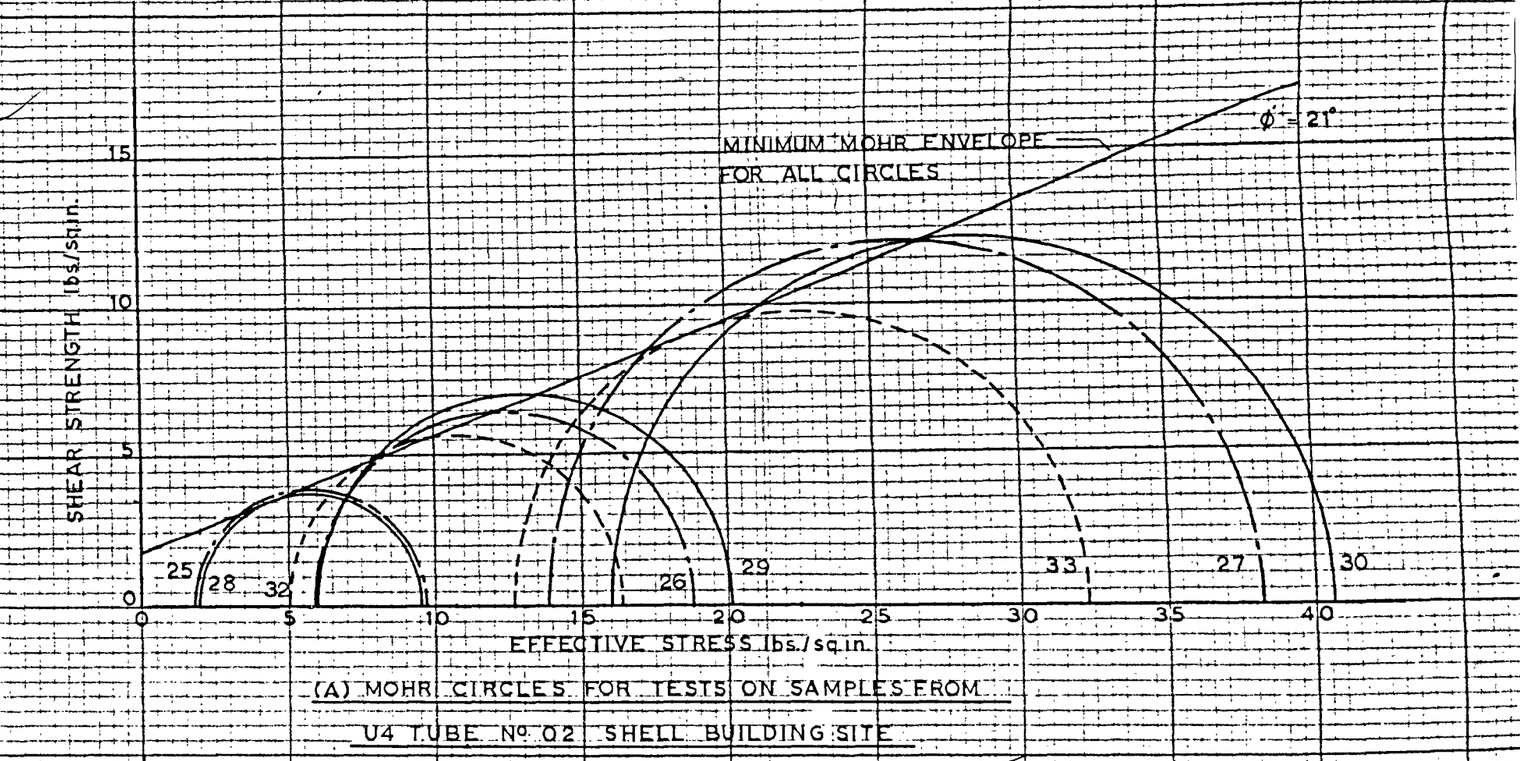
The major portion of the strain took place as can be seen in Fig. 11, while the porewater pressure was

at its low value, i.e. the sample was stressed as indicated by circle "f" Fig. 10 B. Strain at the lower porewater pressure is apparently due to consolidation under the increased effective ambient pressure, and the ratio of strain to volume change of about 0.3% strain per percent volume change is that usually obtained in consolidation of samples of London clay under ambient pressures. This process of consolidation and strain is apparently not reversible at 85% of failure stress although it seems to be nearly so at 60%. The result is that when the effective ambient pressure is lowered, i.e. stress is as shown by circle "e", the sample takes in as much or more water than was expelled but there is little or no recovery of strain. As the moisture content is then as high or higher than at the start of the previous cycle consolidation with accompanying strain can again take place when the porewater pressure is lowered. In this manner the strain under consolidation is cumulative and eventually there is an increase of moisture content due to the dilatency characteristics of the clay and the sample fails under the applied stresses. It is important to note in this regard that although most of the strain took place while the sample was stressed as indicated by circle "f" the actual non terminating failure took place while circle "e" applied.

Results of this one test cannot be considered of

Type of Test	Drained Constant σ_1			Drained Increasing σ_1			Undrained p.w.p. meas.d	
Lab. No.	25	26	27	28	29	30	32	33
$\sigma_1 - \sigma_3$ measured	8.1	13.6	25.5	7.9	15.4	26.8	13.2	22.3
R & D. correction	0.4	0.8	1.2	0.5	1.3	2.2	1.9	2.7
$\sigma_1 - \sigma_3$ corrected	7.7	12.8	24.3	7.4	14.1	24.6	11.3	19.6
σ_3' at failure	1.9	5.9	13.9	2.0	6.0	16.0	5.0	12.7

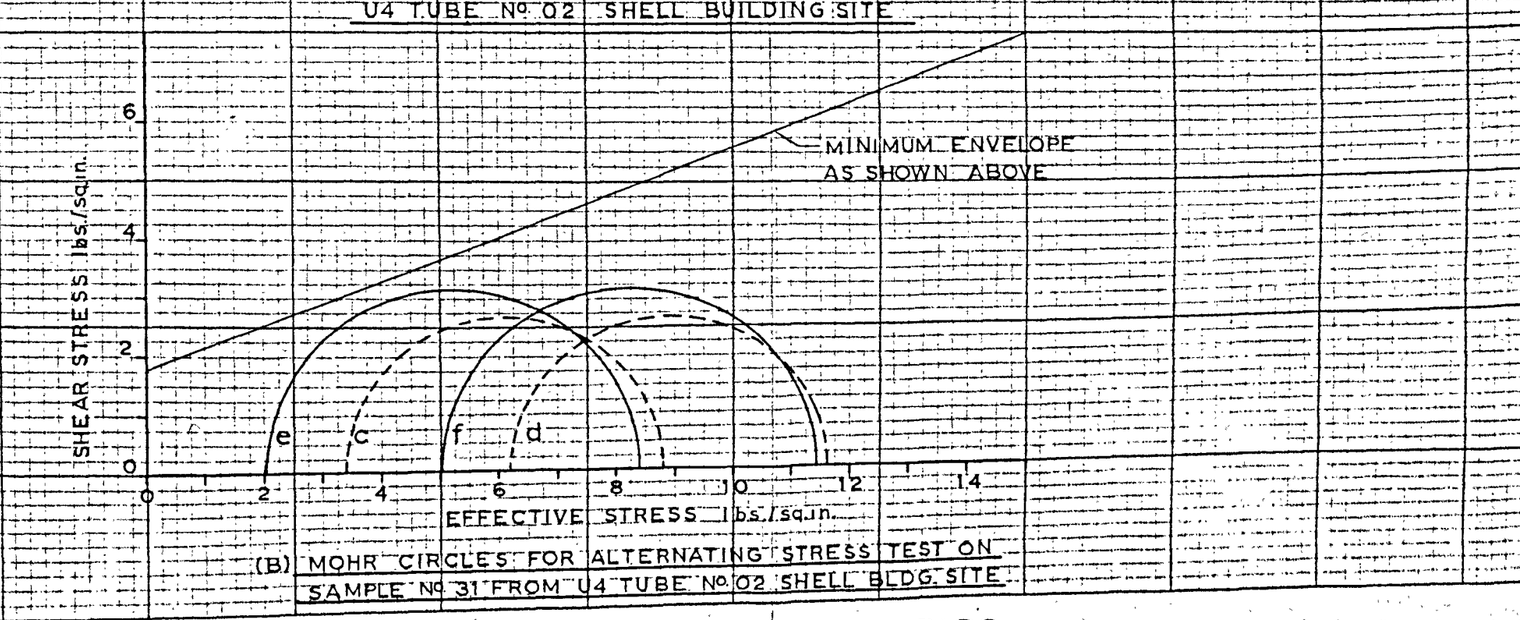
ALL SAMPLES ARE FROM U4 TUBE No. 02
FROM THE SHELL BUILDING SITE
LL = 84 PL = 29



(A) MOHR CIRCLES FOR TESTS ON SAMPLES FROM U4 TUBE No. 02 SHELL BUILDING SITE

SAMPLE No. 31	At 60% of S. Strength	At 85% of S. Strength
$\sigma_1 - \sigma_3$ measured	5.9	7.0
R & D correction	0.5	0.5
$\sigma_1 - \sigma_3$ corrected	5.4	6.5
σ_3'	3.4 & 6.2	2.0 & 5.0

SAMPLE No. 31 LL = 84
PL = 29
Clay Fraction = 61%
Silt " = 24%
Sand " = 5%



(B) MOHR CIRCLES FOR ALTERNATING STRESS TEST ON SAMPLE No. 31 FROM U4 TUBE No. 02 SHELL BLDG. SITE

MOHR CIRCLES FOR TESTS ON SHELL BLDG. SAMPLE No. 02 AND FOR ALTERNATING STRESS TESTS

FIG. 10

FIG 11

ALTERNATING STRESS TEST ON LONDON CLAY SAMPLE NO 31

PLOT OF STRESSES, PORE WATER PRESSURE AND VOLUME CHANGE FOR

FIG 11

general significance but it is notable that failure was caused at stresses smaller than would be necessary in a normal drained test. The possibility does exist that clay in situ subjected to alternating stresses in this manner would eventually increase in moisture content and hence diminish in strength. Whether or not this change in strength is in the c' portion of the shear strength cannot be established without more testing. But, as any envelope tangent to the Mohr circle for this sample at failure (i.e. circle "e") would obviously have a smaller c' intercept, it is highly probable that there is a definite reduction in c'.

CONCLUSION FROM THE LABORATORY INVESTIGATION

Samples of undisturbed over-consolidated clay tested under drained conditions show that the clay tends to dilate under stress. While this dilatancy may not be indicated by volume change of the sample as a whole under the usual method of measurement, due to practical difficulties of making true measurements, it is always indicated by increase in weight of the sample during testing and by the moisture content about the shear plane. Moisture

contents from the area about the shear plane indicate dilatancy in undrained tests carried out at slow rates as well as in drained tests.

Porewater pressure changes in samples tested under increasing σ_1 with constant σ_3 stresses are found to be small while quite large negative porewater pressure changes occur in samples tested under constant σ_1, with decreasing σ_3 stresses. However, dilatancy near the shear surface indicates that negative porewater pressures would be set up in all over-consolidated clays if the stressing is not at a rate sufficiently slow to allow moisture migration to take place. The presence of moisture migration towards the shear surface suggests that undrained tests with porewater pressure measurements, should be carried out at a rate comparable with drained tests to prevent the setting up of porewater pressure gradients with the sample.

Test results show that constant σ_1 tests give a slightly higher ϕ' than increasing σ_1 tests, but not enough difference to draw positive conclusions with the accuracy of the methods of testing used.

Results of tests done by the writer as well as results from other sources, indicate the average drained shear strength parameters of London clay to be very nearly $\phi' = 20^{\circ}$ and $c' = 250$ lb./sq.ft.

Failure under alternating stresses has been produced in London clay at 85% of the normal failure load. This failure at decreased stresses apparently comes about by the non-reversibility of consolidation strain and the build up of moisture content within the sample due to dilatancy under stress. Evidence is not sufficient to make it clear whether the strength reduction is a reduction in $\emptyset'$ or c' but indications point to it being a reduction in c'.

PART III

STABILITY OF NATURAL SLOPES

INTRODUCTION

Movement on natural slopes is of two main types, soil creep and failure due to insufficient shear strength. Most slopes in clay are subject to soil creep which, combined with the agents of erosion, ultimately tends to eliminate hills and convert slopes to flat ground. Creep may be caused by a variety of agents including water, frost and vegetation and results in a movement of the surface layers of soil down the slope, the movement lessening with depth. It is this non-uniform movement which distinguishes creep from slope failures due to inadequate shear strength. Failures in over-consolidated clay due to insufficient shear strength, although movement may be only a few feet per year, are usually considerably faster than soil creep which may be only a few inches per decade. Soil creep does occur in London clay but is usually so slow that problems concerning it very seldom arise and it is not generally

susceptible to analysis. Failures on natural slopes in London clay due to insufficient shear strength are also found and as they constitute a greater hazard and can be analysed, it is this type of failure which will be considered in this section.

Observers as early as Collin (1846) suggested that slopes of cuttings would tend to revert to the slopes found in nature. However very little has been written on the stability of natural slopes, particularly those in clay and what constitutes a stable natural slope has remained rather obscure. Many failures in natural slopes have occurred in various places and have been investigated but there appears to have been no overall comparison of stable and unstable slopes in the same clay. London clay presents an opportunity for such an investigation as its occurrance is widespread and its characteristics very uniform. The shear strength parameters have been shown in the previous section to be nearly constant throughout the clay at $\emptyset' = 20^{\circ}$ and $c' = 250$ lb./sq.ft. Both stable and unstable slopes are to be found in London clay as well as a considerable number of past failures.

Skempton (1945) noted that the limiting angle for stability in London clay slopes was about 10°. At

that time the significance of, or the reason for, this value was not apparent but it is now known to be controlled by the angle of internal friction when water level is at the surface. From the analysis of a failure of a natural slope in the stiff-fissured clays of the coal measures at Jackfield, Shropshire (Henkel & Skempton, 1954) it was concluded that the clay behaved as if it possessed internal friction only and the effective cohesion had dropped to zero.

An investigation of London clay slopes was therefore undertaken to find what inclination of slope was stable and if this value could be explained in terms of stability analysis.

AREA OF FIELD INVESTIGATIONS

A preliminary study of London clay slopes in the Sewardstone, Elstree and Potters Bar areas, all to the north of London, was done by A. C. Graves (1954). The work in these areas was covered in more detail for the present study and extended to further slopes in the same areas. Attention was directed to the slopes in the Childerditch, Essex area by Mr. W. H. Ward of the

Building Research Station. These slopes were examined and found to yield valuable data.

Investigations at Sydenham Hill in South London, (Skempton and Henkel, 1954) showed the higher levels there to be gravel and Claygate Beds with the lower part of the slope in London clay. Further reconnaissance south of the Thames, in areas indicated to be in London clay by the geological maps, located steep slopes at Telegraph Hill and Bunkers Hill near Chessington. However, samples taken from a hand auger borehole showed the major portion of these slopes to be in the Claygate Beds. One slope which failed, and is apparently in London clay, was found in Windsor Great Park, otherwise the area south of the Thames yielded no failures or slopes approaching the limiting stability inclination.

Preliminary reconnaissance indicates that there are some slopes warranting further consideration in the Isle of Sheppey but time was not available to make a complete study. In addition to stable slopes in that area, the London clay is being actively eroded along the coast. This permits a visual examination of large faces of London clay up to 100 ft. high in places and there are many examples of short-term failures. A coastline of this type, see Plate VII, shows very clearly the difference between

short and long-term stability. At places there, the London clay stands temporarily at 75 or 80° to a height of 50 ft. whereas slopes of 10° have been found unstable in other areas.

The reconnaissances and survey of slopes showed very clearly that in the area north of the river Thames the slopes were generally steeper and the countryside gave the appearance of being much younger. This change in topography is consistent with the extent of the Pleistocene glaciations which stopped just north of the Thames and no glacial deposits are to be found south of the river in the London clay areas. It appears that the valleys north of the Thames were deepened by the melt-water from the ice sheets. This water was carried away by the river leaving the area to the south unaffected. The slopes of the newly deepened valleys were considerably steepened and it is apparent they have not yet reached equilibrium. Deforestation of the slopes during the last millennium has probably contributed to the instability.

FIELD INVESTIGATION

Inclinations of slopes were measured by means

of a vernier clinometer with the observer sighting on another person at least 150 ft. down the slope. Slopes shorter than 150 ft. were disregarded as being not sufficiently large to present a true picture of the conditions. The probable level of the water table was observed where possible and any topographical features, such as ponds, which would indicate a high water table were noted.

On representative slopes hand auger holes were put down and samples taken for moisture content and Atterberg limit determinations. All the samples of London clay taken were found to be the brown, weathered clay and yielded the following average results:-

Water Content	-	33
Liquid Limit	-	80
Plastic Limit	-	29
Density	-	119 lb./cu.ft.

These average values are not far different from those obtained for London clay in the testing programme, therefore the average value of $\emptyset' = 20^{\circ}$ and $c' = 250$ lb./sq.ft. may be considered to apply equally to the clay in the natural slopes.

UNSTABLE SLOPES

During the investigation the eight slopes listed

TABLE V

Site & Slope No.	Inclination	W	L.L.	P.L.	Remarks
Windsor 1	12½°				Recent small slip
Childerditch 5	11¾°	36	78	33	Moderately large slip
" 9	10¾°				High G.W.L.
" 1	10½°	33	78	29	Active slip
Sewardstone 6a	10½°				Slip in 1913
Childerditch 2	10¼°				
Elstree 1	10°	31	80	28	G.W.L. at surface
Sewardstone 1	10°	36	84	28	G.W.L. at surface

TABLE V Unstable Slopes in London Clay,

in Table V above, were found to be unstable and slipping. At some of these sites there was definite evidence, such as free water on the surface or marsh grass, that ground water level was at or near the surface. In other cases, particularly that at Windsor, evidence points to a lower ground water level, but probably not more than 2 or 3 ft. below the surface in winter and likely less than this amount at some sites.

The movement at Childerditch Nos. 1 and 2 and at Sewardstone No. 1 is apparently nearly parallel to the surface

and extends over a fairly large area. Plate VIII shows Childerditch No. 1 and a definite ridge is noticeable around the base of the moving earth with a steep face remaining at its top. In the other cases of Table V movement has taken more the form of a slump with a proportionally larger steep surface above the slide and more bulging at the toe, see Plate IX as an example. At Windsor this type of movement is the most pronounced and the slip surface is apparently approaching the cycloidal shape commonly found in failure in cuttings.

In addition to the active and recent failures a number of sites indicated from their topography, that failures had occurred a considerable time ago and so could not be included in Table V because of the lack of positive data. On some slopes there is evidence of soil creep with perhaps some shear strength failure. Slopes in this class were considered borderline cases and not included in either the list of unstable or stable slopes.

STABLE SLOPES

Table VI gives details on 20 slopes which were found to be stable.

TABLE VI

SITE AND SLOPE No.	INCLINATION	W	L.L.	P.L.	REMARKS
CHILDERDITCH 6	11 1/2°				WOODED SLOPE
" 10	10 1/2°				
" 12	10 1/2°				HIGH G.W.L.
SEWARDSTONE 6c	10 1/4°				
" 7	10 1/4°				
" 5	10°				
" 4b	9 1/2°				PROBABLY LOW G.W.L.
" 2	9 1/4°	28	79	27	
" 8b	9 1/4°	30	81	29	
ELSTREE 4	9 1/4°				
" 3	9°				PROBABLY HIGH G.W.L.
POTTER'S BAR 2a	9°				
SYDENHAM A 5	8 1/4°	31	90	30	G.W.L. ABOUT 3 FT. DEEP
SEWARDSTONE 8a	8°				
POTTER'S BAR 5	8°				
" 2b	7 3/4°				
ELSTREE 2	7 3/4°				
POTTER'S BAR 1	7 1/2°				(THERE ARE A GREAT NUMBER OF STABLE SLOPES AT INCLINATIONS OF LESS THAN 7°)
" 3	7 1/4°				
" 4	7 1/4°				

STABLE SLOPES IN LONDON CLAY

TABLE VI

These slopes with the exception of Childerditch No. 6 are all in grass pasture land. The one exception is a wooded slope and it is probable that the ground-water level there never comes to the surface.

A typical stable slope is shown in Plate X. The profile of the stable slopes is usually almost straight over a considerable length with a comparatively short transition length at the top and bottom of the slope.

STABILITY ANALYSIS

In slips on natural slopes at inclination such as the ones observed in London clay the length of the moving mass of earth is usually large in comparison with its depth and in most cases the failure surface is parallel, or nearly so, to the ground surface. For the types of failure where the ground and failure surfaces are parallel and cover a considerable area, and effects may be neglected and the stability considered on the basis of the mass moving down the failure plane (see for instance Taylor, 1948; Henkel and Skempton, 1954).

Referring to Fig. 12, the shearing stress on the failure plane is:

$$\tau = z \gamma \sin\beta \cos\beta$$

and the normal effective pressure on the failure plane is:

$$\sigma_n' = Z\ (\gamma - m\gamma_w)\ \cos^2\beta$$

Stability is maintained when the shear strength is equal to or greater than the shearing stress, i.e.

$$c' + z\ (\gamma - m\gamma_w)\ \cos^2\beta\ \tan\phi' \geq z\gamma \sin\beta\ \cos\beta$$

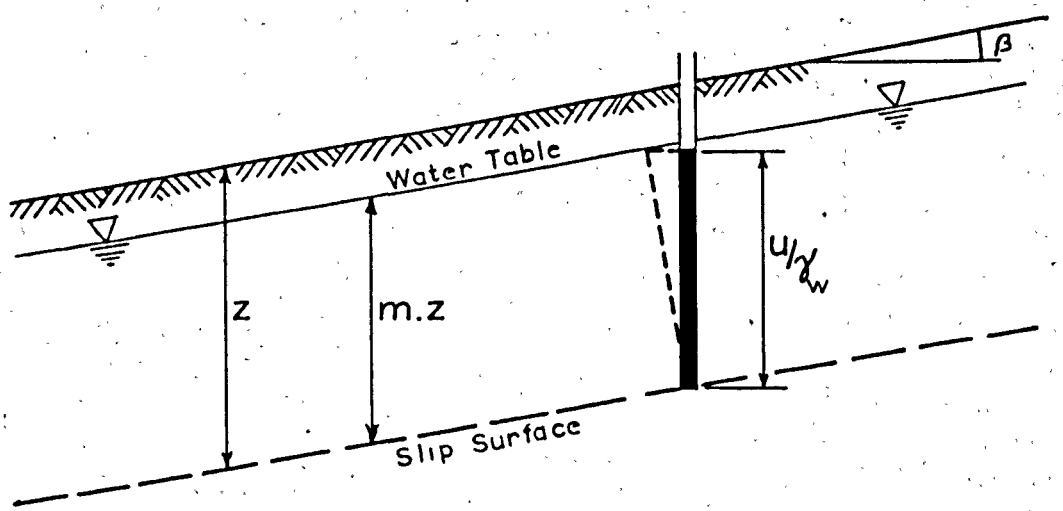


FIG. 12 STABILITY ANALYSIS FOR NATURAL SLOPES

There is substantial evidence from field cases indicating that c' ultimately approaches zero and therefore for long-term stability the frictional component of shear strength must be

equal to or greater than the shearing stress, i.e.

$$z(\gamma - m\gamma_w)\cos^2\beta \tan \phi' \geq z\gamma \sin\beta \cos\beta$$

therefore, resolving this equation the maximum value of β is given as

$$\beta_{max} = \tan^{-1}\left\{\frac{\gamma - m\gamma_w}{\gamma} \cdot \tan \phi'\right\}$$

In the case of London clay with the water level at the surface, $\phi' = 20^\circ$ and $c' = 0$ the limiting value of β is $9\frac{3}{4}^\circ$. When $m = 0$ the limiting value of β is obviously ϕ'. Between these two limits β is very nearly proportional to m.

It might be noted that since failures on natural slopes are generally not deep the frictional shear strength involved is low and even a small value of c' would alter considerably the limiting angle of stability. Consider for instance, a slope with water level at the surface and a potential failure plane at 10 ft. depth. A c' of 50 lb./sq.ft. would increase the limiting angle of stability for the slope from 10° to 12°. This same increase in the limiting angle would occur if the water level were two ft. below the surface. If the full value of c' = 250 lb./sq.ft. as obtained in the laboratory tests were present the limiting angle for stability

on such a failure surface would be approximately 22½°.

Field observations would indicate that the average depth to the failure surface in many cases may be less than 10 ft. If this is so the effect of c' on the limiting angle would be more pronounced.

CONCLUSIONS FROM NATURAL SLOPES

The information given in Tables V and VI is present graphically in Fig. 13. From the data the following information can be drawn:

(1) All slopes of less than 10° are stable.

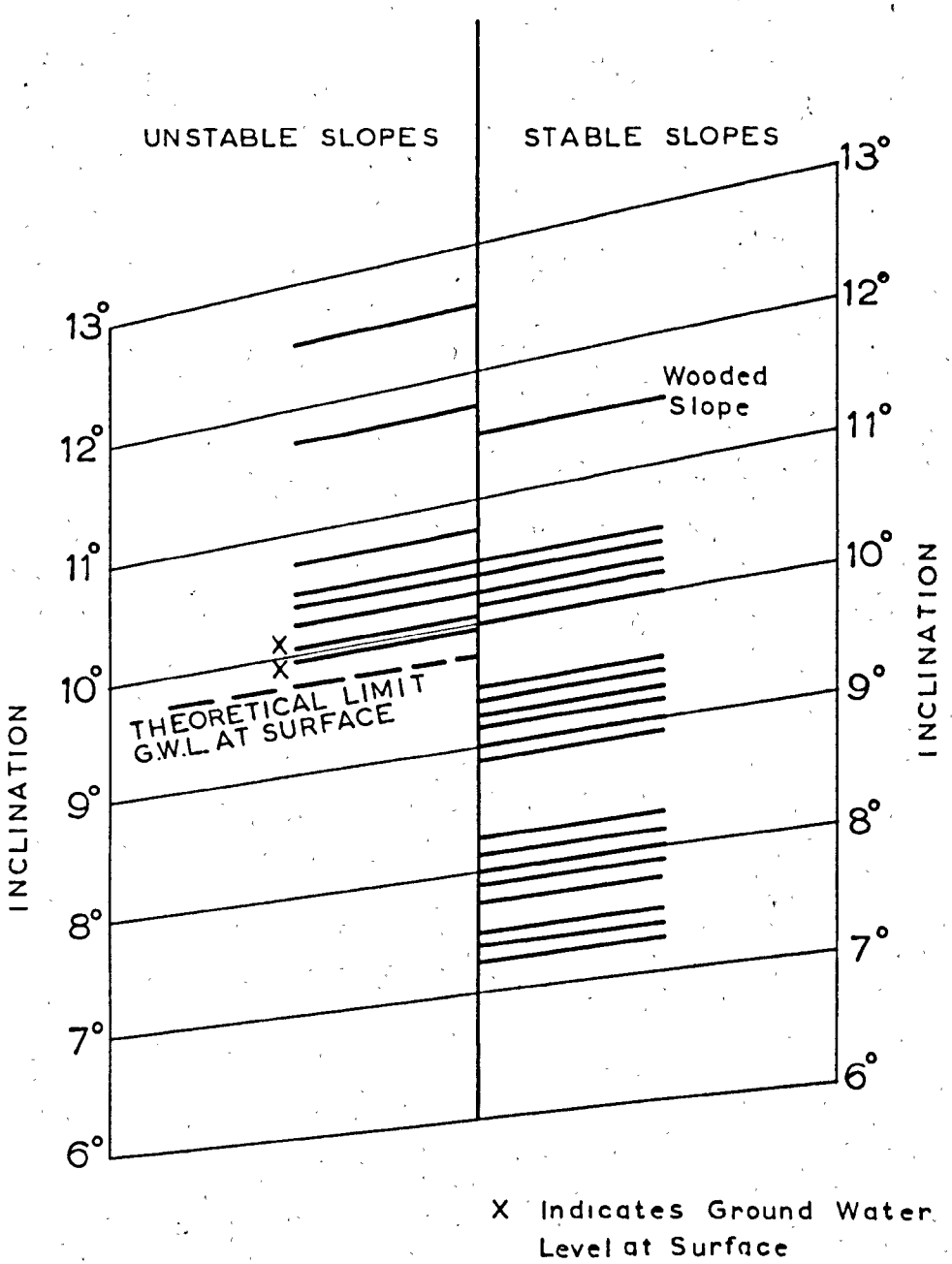
(2) All unstable slopes have inclinations between 10° and 12½°.

(3) No slopes steeper than 12½° have been found.

The results indicate that 10° is the critical inclination for natural slopes in London clay. This is virtually the same angle obtained in the stability analysis with the assumption that c' = 0 and hence indicates that c' does eventually reach zero in natural slopes in London clay. The same conclusion was reached by Henkel and Skempton, (1954), for the heavily over-consolidated clays of the coal

measures at Jackfield, Shropshire.

Although definite conclusions for all heavily over-consolidated clays cannot be drawn from these two sets of results, they are indications that c' may reach zero on a geological time in over-consolidated clays generally. A consideration of natural slopes in other types of clay will be necessary to insure that this result is generally valid.



NATURAL SLOPES IN LONDON CLAY

FIG. 13



PLATE VII. ACTIVELY ERODING COASTLINE IN LONDON CLAY. WARDEN, ISLE OF SHEPPEY.
THE STEEP SLOPES AND THEIR UNSTABLE NATURE ARE VERY NOTICEABLE COMPARED WITH THE SLOPE IN THE FIELD WHICH IS STABLE AT $8\frac{1}{2}^{\circ}$ INCLINATION.



PLATE VIII. ACTIVE SLIP ON A $10\frac{1}{2}^{\circ}$ SLOPE IN LONDON CLAY. CHILDERDITCH No. 1.
A SLIP ON THE STEEPER PART OF THE SLOPE, SEE ARROW, WAS REPAIRED c1920 WITH COUNTERFORT DRAINS.



PLATE IX. OLD SLIP IN LONDON CLAY ON 10½° SLOPE, SEWARDSTONE No. 6A. THIS FAILURE OCCURRED IN 1913. ALTHOUGH THERE HAVE BEEN ATTEMPTS TO FLATTEN THE GROUND, THE TOP OF THE SLIP AND THE HUMMOCKY NATURE OF THE SURFACE ARE STILL VISIBLE.



PLATE X. STABLE SLOPE IN LONDON CLAY AT 10° INCLINATION, SEWARDSTONE No. 5.

PART IV

EXAMINATION OF FAILURES

INTRODUCTION

The following sections are concerned with the examination of nine failures in cuttings in London clay. The cases comprise both slopes and retaining walls and in two examples a retaining wall with a proportionally large slope behind it. These cases are all in railway cuttings which vary from 18 to 60 feet in depth and vary in age at failure from 13 to 81 years. In most cases some remedial measures have been taken to stop failure, with varying success.

The purpose of this examination is to consider each failure in detail and, by analysis, to determine the value of c' necessary for stability and the range of factor of safety for the possible range of c'. That is, to find (a) to what extent had the c' been reduced through time at the site in question, (b) what was the probable factor of safety on an effective stress basis at construction and (c) what would the factor of safety be if c' had

decreased to zero. If the effective stress parameters necessary for stability have been determined the effectiveness of remedial measures may be calculated. Such calculations were carried out for the cases where the remedial measures were well known and their effect could be determined quantitatively.

When the physical dimensions of any case and the soil shear strength are known it is usually possible to determine the factor of safety of the structure, conversely if the dimensions and factor of safety are known it is usually possible to determine the shear strength. In consideration of the examples here the latter circumstances hold, the factor of safety at failure is unity, hence the corresponding shear strength may be determined. On an effective stress basis the shear strength is composed of an angle of shearing resistance and cohesion. It has been shown earlier that the drained angle of shearing resistance for London clay is very nearly constant at 20°, hence with other dimensions known the cohesion portion of the shear strength may be determined.

The determination of c' was generally accomplished by assuming failure to take place along circular arcs or along planes and resolving the forces involved as outlined in a following section. In addition use was made, in two

cases, of a failure surface of non-uniform curvature and of a circular arc and straight line combined. Determinations in all cases were made per unit length of the slope or wall. For these failures in London clay the length of the failure zone is usually considerably greater than its width, the ratio varying from 2 to 1 at Sudbury Hill to about 6 to 1 at Mill Lane. It has been shown (Kenney 1956) that end effects are relatively unimportant when the length of the slip is considerably larger than its width and slices of unit thickness may be used for calculation with little error.

CHOICE OF FAILURE SURFACE

To apply any form of analysis to a particular case some failure surface must be assumed. Failure may be taken as occurring along planes, circular arcs or other mathematical curves such as logarithmic spirals or a combination of these shapes. However, for each case some note must be taken of the probable mode of failure and the appropriate slip surface or surfaces then assumed. Thus, while failure along a plane parallel or nearly parallel to the surface can generally be assumed for long natural

slopes, such an assumption is usually not logical for the shorter slopes found in cuttings in over-consolidated clay. For the slopes of cuttings a rotational type of failure is more common, hence circular arcs are a more suitable assumption for a failure surface.

For retaining walls it has been found (see for instance the Mill Lane case) that the assumption of failure on planes is a reasonable approach to reality. Therefore, in the analyses which follow the procedure has generally been to consider failure in the first instance along planes for retaining walls and on circular arcs for slopes.

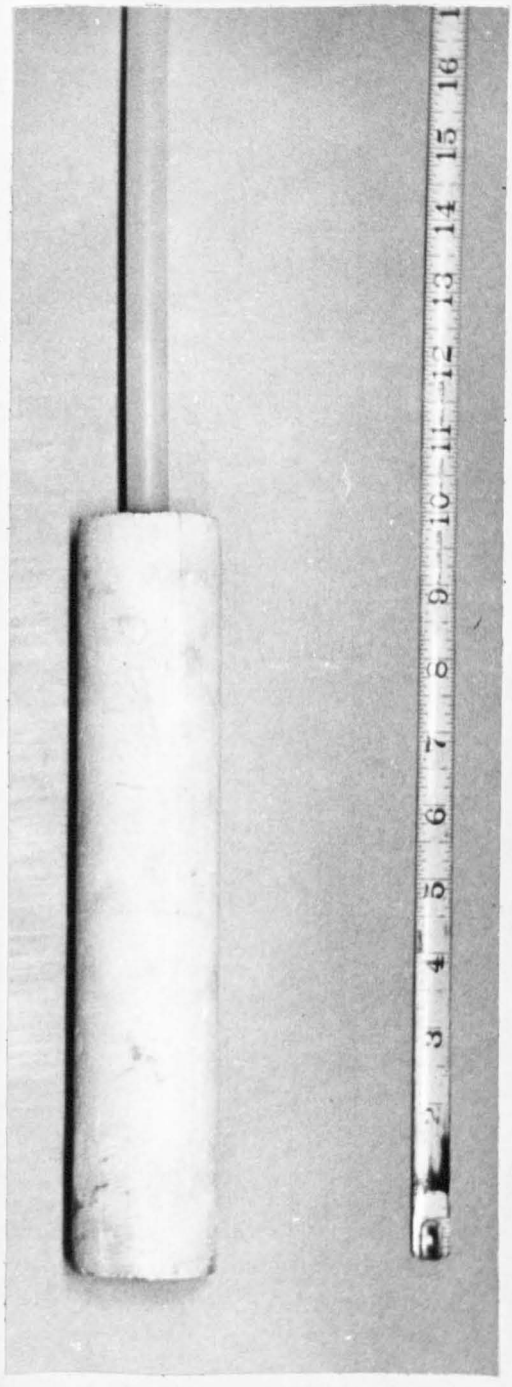
Actual failure surfaces have been observed in the case of one slope (Northolt) and were found to be approximated by a circular arc and straight line combined. For this surface a non-circular type of analysis was applied as given by Kenney (1956). However, the use of non-circular failure surfaces results in long calculations and as an infinite number of shapes and surfaces may be assumed such a procedure is not practical, from a design point of view, with the present knowledge of actual slip surfaces in over-consolidated clay. It is of course to be expected that as the shape of probable worst surfaces become better known the use of non-circular slip surfaces

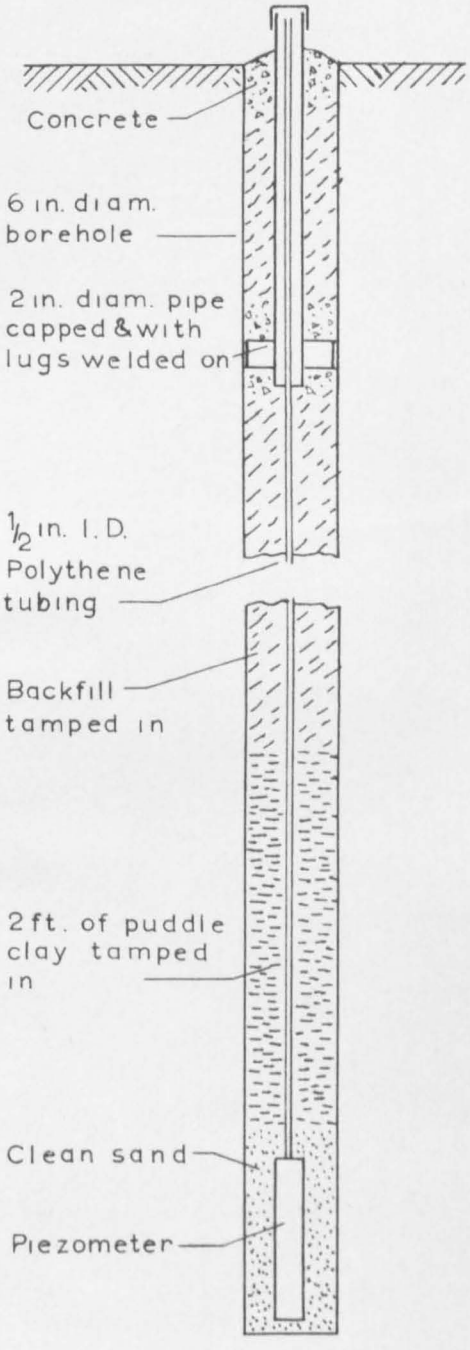
will receive increased attention for homogeneous soil as it is now receiving for earth dams of non-homogeneous section and foundation. The non-circular type of analysis was applied in the case of Northolt in order to compare the results on an actual failure surface with those obtained by the usual circular arc method of investigation.

Although failure along planes was assumed for retaining walls as a first analysis, several walls were also considered on the assumption of failure along a circular arc. The purpose of this was to compare the two methods and to determine if one could be used as a check on the other.

WATER FORCES

To use an analysis based on effective stresses it is essential that the water forces acting on the structure and on any assumed failure surface be known. To obtain this information piezometers of the Casagrande type (see Fig. 14) were installed at five sites. In one case (Wood Green) water levels had been observed over a considerable period of time in capped boreholes. In the other cases some information exists on water levels, but



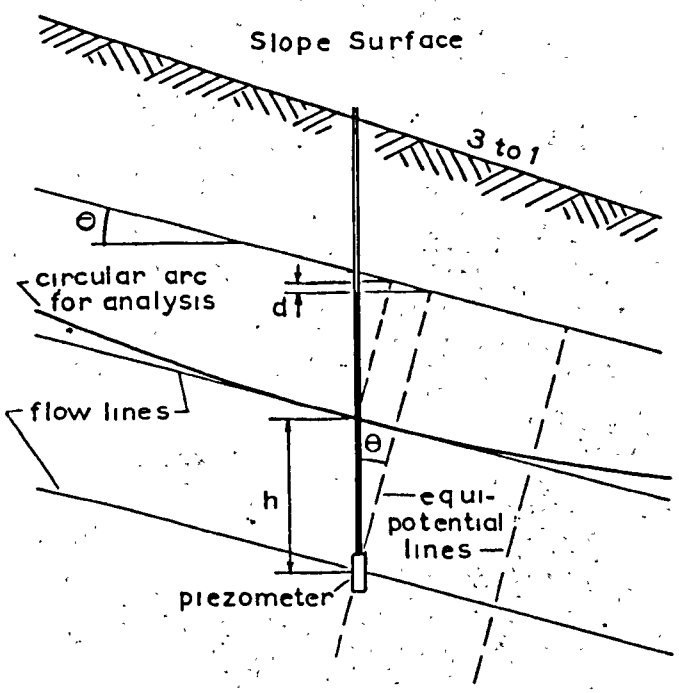


PIEZOMETER AND METHOD OF INSTALLATION

FIG. 14

it is by no means complete and remedial measures have changed the cross section to the extent that determination, at the present time, of porewater pressures that could be related to conditions at failure is not feasible. Even in the cases where water levels were determined, it is not certain that they are exactly the same as those operating at the time of failure, as the failure itself may change the drainage pattern of a slope. Where possible, water levels were measured outside as well as on the failed portion of the slope so that both together would serve as a guide in establishing a water level to be used in the analysis.

A flow pattern is usually set up in the slope of a cutting and therefore the assumption of a static water level to be used in determining the porewater pressure on slip surfaces at different depths is not rigorously correct as shown in the Fig. 15 on page 69. However, for conditions encountered in these slopes the difference between the water head measured at a piezometer and true porewater pressure on an assumed failure plane due to their different positions in the slope does not usually amount to more than 4 or 5 inches of head in the worst case and in most cases is less. While water levels can be measured to much smaller dimensions than this, the assumption that any measured water level was the one operating



$d = h \sin^2 \theta$

For a 3 to 1 slope
$\theta \doteq 15°$

When $h = 4$ ft.
$d = 3.2$ in.

FIG. 15 CORRECTION TO WATER LEVEL MEASURED WITH PIEZOMETER

FIG. 15

at failure cannot approach this tolerance, hence a static water level was used in determining the water force on assumed failure planes.

In three cases, measurements of water levels were made in open boreholes. The water level in boreholes usually takes a considerable time to come to equilibrium and in a few holes (at Mill Lane in 1942) equilibrium was not reached in seven weeks. Where equilibrium was reached in an open borehole it was found in the two places measured that the open borehole gave a higher water level than an adjoining piezometer by about two feet. Some difference

might be expected from the flow pattern set up, as mentioned above, but not a difference of this order. Evidence on this point is not enough to draw very definite conclusions but it does seem that there may be other factors involved such as the flowing of water into the hole from perched water near the surface.

ANALYSIS BY CIRCULAR ARCS

The method used for the determination of the effective cohesion at failure, when failure on a circular arc is assumed, is based on the equation for factor of safety as given by Bishop (1954)..

$$F = \frac{1}{\Sigma W \sin\alpha} \cdot \Sigma\left[\left\{c'b + \tan\phi'\left(W(1-\bar{B}) + (X_n - X_{n+1})\right)\right\} \cdot \frac{\sec\alpha}{1 + \frac{\tan\phi' \cdot \tan\alpha}{F}}\right]$$

For routine use of this equation the X forces are omitted and for use with measured water levels the $W(1-\bar{B})$ term is replaced by $(W-h_w\gamma_w\, b)$ where h_w is the observed head of water on the failure surface. The equation then becomes

$$F = \frac{1}{\Sigma W \sin\alpha} \cdot \Sigma\left[\left\{c'b + \tan\phi' \,(W-h_w\gamma_w\, b)\right\} \cdot \frac{\sec\alpha}{1 + \frac{\tan\phi' \cdot \tan\alpha}{F}}\right]$$

To determine the value of c' required for stability, F at failure is unity and the equation is rearranged as below

$$c' = \frac{\sum W \sin\alpha - \sum\beta\{\tan \phi'(W - h_w \gamma_w b)\}}{\sum\beta b}$$

where $$\beta = \frac{\sec\alpha}{1 + \frac{\tan\phi'}{F}\tan\alpha}$$

An example of the determination of c' by this method is given in Fig. 16.

ANALYSIS BY PLANES

Analysis by assuming failure on planes has been applied here only to retaining walls, as this type of analysis is not characteristic for slopes. When plane failure is assumed, the first determination is to find the c' required for stability by finding its value when the resistance to sliding offered by the wall plus the passive earth pressure equals the active earth pressure.

The active earth pressure is determined by assuming a failure plane and drawing a force polygon for

all the forces acting on the earth between the wall and the assumed plane. These force polygons are drawn using a $\emptyset'$ of 20° for London clay and for two values of c', generally c' = 0 and c' = 250 lb./sq.ft. Different planes of failure are assumed until the maximum active pressure is obtained for each of the above values of c'.

Passive earth pressure is determined by assuming failure on a plane from the front of the wall and resolving the force polygon for the wedge of earth involved. This procedure is followed for the two values of c' used above and for as many planes as necessary to find the minimum passive earth pressure.

Wall stability is next considered by drawing the force polygon for the wall using as one side the minimum passive force just obtained. The active force, however, is not included in the force polygon for the wall. Instead the closing side of the polygon is a force in a direction opposite to the active force and this closing side indicates the wall resistance to an active force in that direction. Wall resistance is determined for the two values of c' already used.

A plot is then made of the two forces, active earth pressure and wall resistance, obtained by the force polygons using effective cohesion as a base. It will be found that the two forces give a straight line plot

(geometry of the force polygons establish this) and intersect at some value of c' which is therefore that operative at failure, when wall resistance is just equal to the active force.

A step by step description of this procedure is given with Fig. 17 to be used with the explanatory analysis in that figure. In practice the force polygons are usually drawn partially superimposed upon each other as they have some sides in common, thus the complete analysis using four or five possible planes for the active and passive wedges can easily be done on a single drawing.

The shape of some of the wall cross sections is rather irregular and this causes difficulty in determining the probable type of failure at the wall itself. Generally it is necessary to assume the front and back of the wall are vertical, this being common practice even with cantilever type retaining walls (see for instance, Civil Engineering Code of Practice, No. 2 (1951) Earth Retaining Structures).

The active force is taken as acting at an angle of $\emptyset'$ to the vertical back of the wall. Tests (see appendix B) to determine the angle of friction between London clay and a rough surface of this type indicate that the full value of $\emptyset'$ is developed. It was also found in these tests that there was some c' between the clay and a rough surface,

therefore the c' between the clay and the wall was taken as being the same as in the clay at failure, generous allowance being made for a shrinkage crack at the back of the wall.

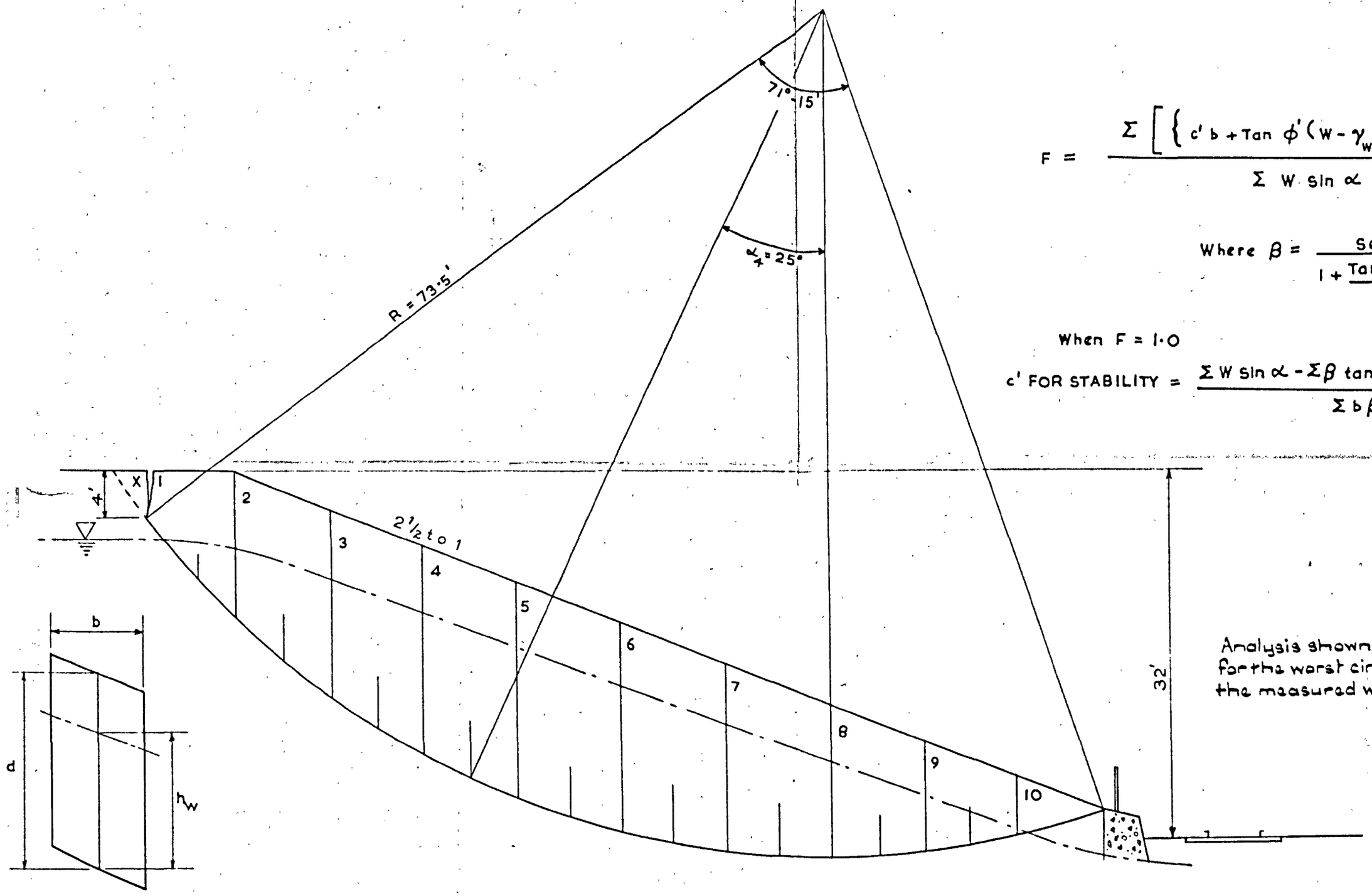
In the case of the passive force the angle of wall friction (δ) at the contact face can only equal ϕ' if the wall moves downwards relative to the soil as well as forwards. For the other extreme where the wall may move freely with the passive wedge (e.g. a sheet pile anchor wall) no frictional component is developed at the wall and $\delta = 0$. In the cases to be considered in the following sections the walls were observed to move horizontally in several instances and in no case was either upward or downward movement noted. Therefore, the angle of wall friction at the passive contact face must be between the two extremes noted above and for the analyses has been taken as $\phi'/2$, this being considered as close an estimate as possible to the actual δ. The presence of wall friction results in a curved surface of sliding but for low values of δ (i.e. 10°) the assumption of failure on planes yields the same results as the more rigorous solution for failure on curved surfaces as given by Caquot and Kerisel (1948) (see Tschebotarioff, 1951, p.244).

The effect of using $\delta = \phi'/2$ rather than $\delta = \phi'$,

on the final analysis is usually small, amounting to about 3% difference in the factor of safety. As the actual δ is undoubtedly nearer $\phi'/2$ any error from using this value cannot amount to more than about 1% on the factor of safety, a very tolerable margin considering the other uncertainties involved.

In the case of walls with a sloping base there is some question where the failure takes place. The Mill Lane wall is the one case where extensive observations of the slip surface exist and the failure surface there was found to be nearly horizontal below the wall, see Figs. 26 and 27. Although no other cases as well defined as this exist, it is worthy of note that if failure occurs along the base of a wall with a sloping base, forward movement of the wall would mean an accompanying upwards movement of the wall and such movement has not been observed in any of the failures considered. Analyses summarized in Fig. 18 show that the c' necessary for stability increases the deeper the failure below the wall. It will be noted that although the passive wedge and hence the passive earth force becomes larger, the lessening in the component of the reaction at the base of the wall opposing forward movement overbalances this increase, with the net effect of less wall resistance with

with deeper failure paths. However, there is a limit to the possible practical depth at which failure could occur. It is obvious that if the position of the wall reaction continues to make an angle $\emptyset'$ to the vertical on the failure surface, when the failure surface goes more than $\emptyset'$ below the horizontal the reaction has a component in the direction of movement. This means the wall actually contributes to its own failure, an obvious absurdity. Practical considerations of the displacement of earth along the failure surface make any path underneath the wall below the horizontal seem unlikely, this coupled with the observed slip surface at Mill Lane and the observed fact that no walls have been known to move downwards or upwards during failure points to a horizontal assumption of failure surface under the wall to be the only practical one. Accordingly, horizontal failure surfaces have been assumed for analysis under walls with sloping bases. In most cases the c' required, assuming failure along the sloping base, is given for comparison. The difference is more pronounced the larger the wall. In the smallest wall, Upper Holloway, where the weight of the active wedge is much greater than that of the wall both paths of failure give the same result.



$$F = \frac{\Sigma \left[\left\{ c' b + \tan \phi' (W - \gamma_w h_w b) \right\} \beta \right]}{\Sigma W \sin \alpha}$$

Where $\beta = \dfrac{\sec \alpha}{1 + \dfrac{\tan \phi'}{F} \tan \alpha}$

When F = 1·0

$$c' \text{ FOR STABILITY} = \frac{\Sigma W \sin \alpha - \Sigma \beta \tan \phi' (W - \gamma_w h_w b)}{\Sigma b \beta}$$

Analysis shown is that for the worst circle using the measured water level.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
No.	b	d	A	W	h_w	$h_w \gamma_w b$	$W - \gamma_w h_w b$	$\tan \phi' \times$ ⑧	α	sin α	W sin α	β	$\beta(\tan \phi' \times$ ⑧)	$b\beta$
1	7·3	8·4	61·3	7,360	2·9	1,090	6,270	2,280	47° - 30'	·738	5,430	1·060	2,420	7·75
2	8·3	14·1	117·0	14,040	8·2	4,240	9,800	3,570	39°- 50'	·641	9,000	1·000	3,570	8·30
3	8·0	16·9	135·2	16,230	11·2	5,600	10,630	3,870	32°	·530	8,610	·962	3,720	7·70
4	8·0	18·1	144·8	17,370	12·6	6,300	11,070	4,030	25°	·422	7,320	·945	3,810	7·56
5	9·0	18·1	162·9	19,540	12·7	7,130	12,410	4,530	18°	·309	6,050	·942	4,270	8·48
6	9·0	16·9	152·1	18,270	11·7	6,570	11,700	4,270	10°-30'	·182	3,330	·954	4,070	8·58
7	9·0	14·5	130·5	15,650	9·6	5,390	10,260	3,740	3°-30'	·061	1,010	·982	3,670	8·84
8	8·0	11·1	88·8	10,660	6·4	3,200	7,460	2,710	- 3°-10'	·055	-590	1·025	2,780	8·21
9	8·0	7·0	56·0	6,720	2·5	1,250	5,470	1,990	- 9°-30'	·165	-1,110	1·088	2,150	8·65
10	7·2	2·4	17·3	2,080	0	0	2.080	760	- 14°-30'	·250	-520	1·140	870	8·21
Σ											38,530		31,330	82·28

$$c' \text{ FOR STABILITY} = \frac{38,530 - 31,330}{82·28} = 88 \text{ LB/SQ. FT.}$$

(IF PORTION X, BEYOND THE TENSION CRACK, IS INCLUDED IN THE WEIGHTS, c' FOR STABILITY IS 89 LB/SQ. FT.)

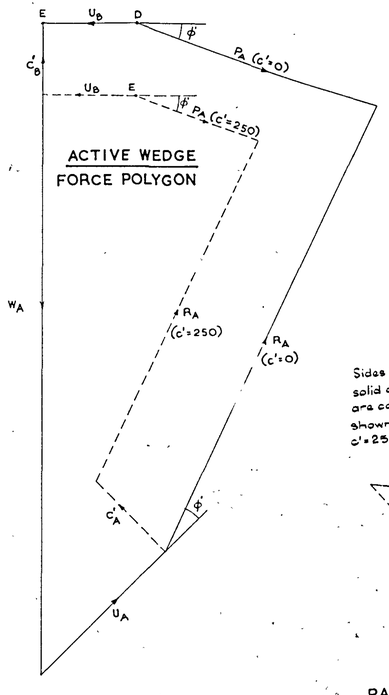
TYPICAL CIRCULAR ARC ANALYSIS

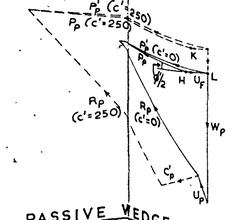
TO DETERMINE c' FOR STABILITY

AT NORTHOLT SLOPE

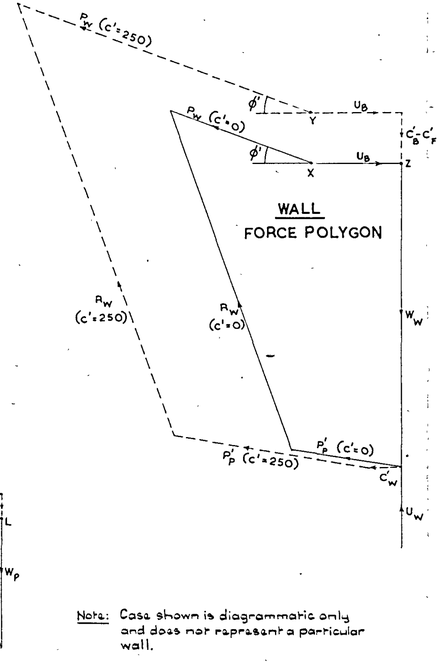
SCALE 1" = 10'

FIG. 16.

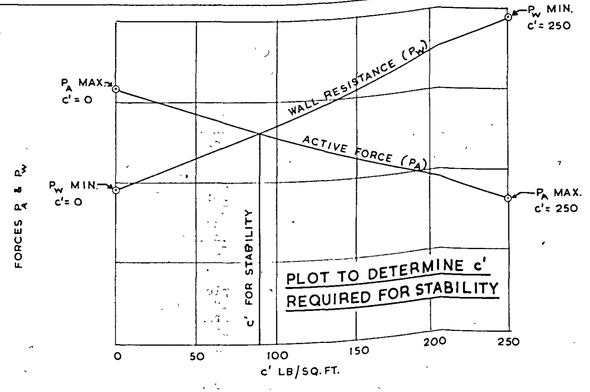


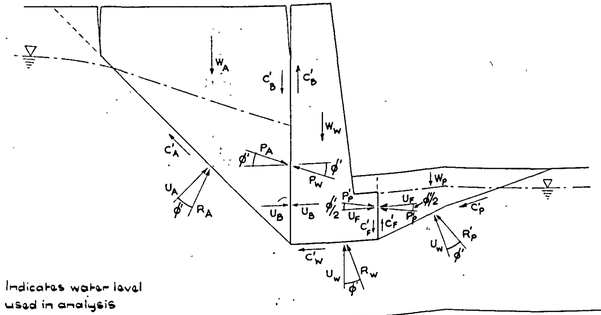


Sides in force polygons shown solid are for case when c'=0 or are common to both cases. Sides shown dotted are for case when c'=250 lb/sq.ft. only.



Note: Case shown is diagrammatic only and does not represent a particular wall.





Indicates water level used in analysis

PROCEDURE

Assumed planes of failure for the active and passive wedges are drawn as shown and the determinable forces on the wedges and the wall calculated. Analysis is then carried out, first on the assumption that there is no cohesion present, and then on the assumption that there is cohesion present, 250 lb./sq.ft. usually being taken as the amount.

Considering first the case of the active wedge with no cohesion: the origin of the force polygon is taken at point D. From D a line is drawn in the known direction of the active force P_A (at ϕ' to the back of the wall). The force polygon is then completed starting at point D and plotting the water force on the face of the wedge U_B, the weight W_A, the water force on the base U_A, and the reaction on the base R_A (at ϕ' to U_A) which is known in direction but not in magnitude. The intersection of the sides P_A and R_A then establishes the magnitude of these forces and the value of the active force P_A when c' = 0 is determined.

For the active force when cohesion is present, a point E is chosen as the origin by plotting backwards from point C, so that when the force polygon is drawn with origin at E the addition of forces U_B and the cohesion force between the back of the wall and the wedge (C_B) brings the force polygon to point G. From G the forces are plotted as before with the addition of the cohesion force on the base of the active wedge (C'_A). The magnitude of the active force P_A when c' = 250 is then determined as before by the intersection of the sides R_A and P_A.

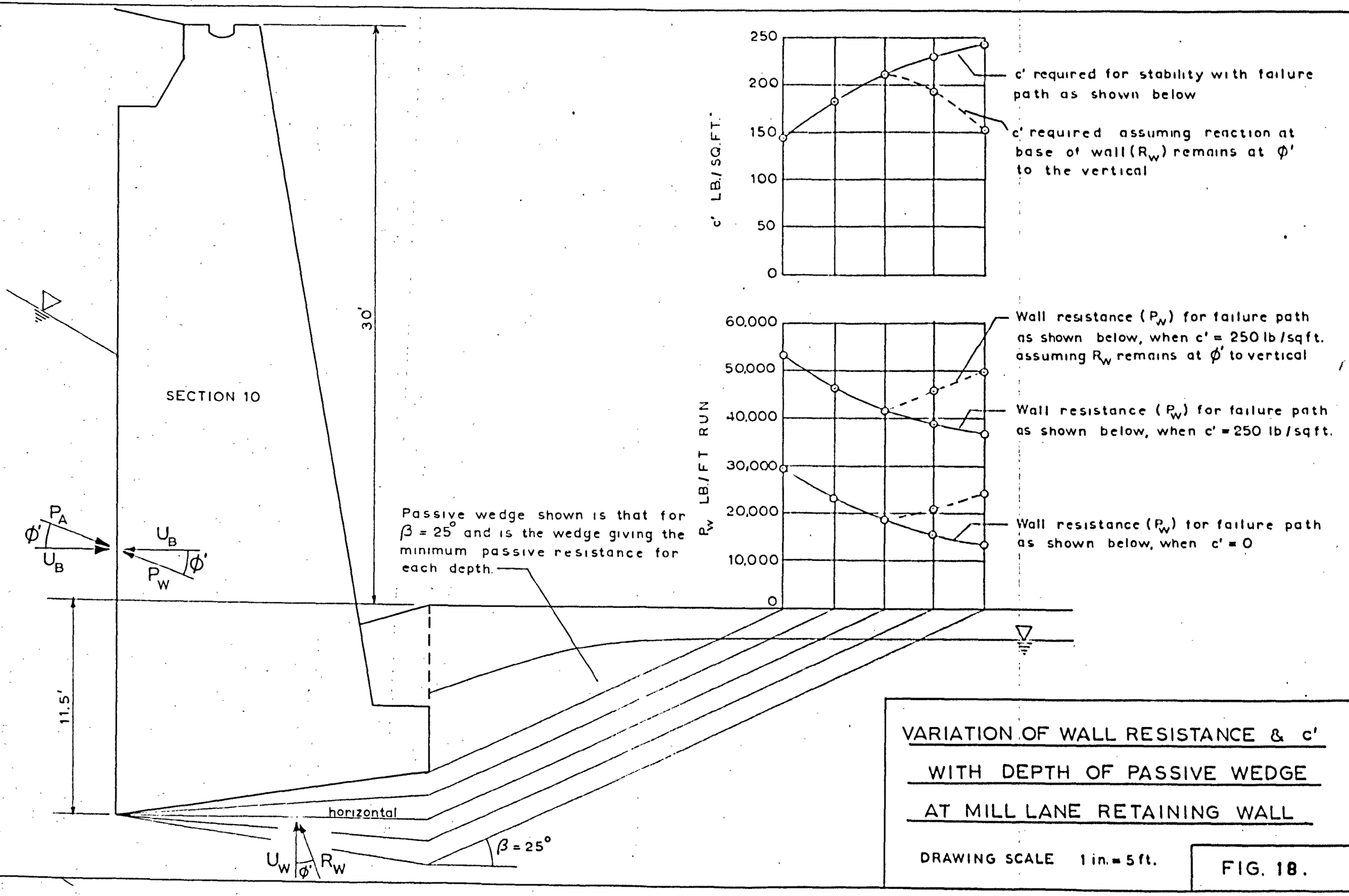
The passive force is determined in a similar manner starting at point H when c' = 0 and point K when c' = 250. After the passive force P_p has been determined the total force on the front of the wall P'_p is determined for convenience by drawing the triangle of forces with sides P_p, U_F and P'_p.

The forces on the wall when c' = 0 are next considered. With the origin at point X the line of action of the wall resistance (P_W) is drawn and the force polygon is plotted with the forces in the order shown, one side being the total force on the front of the wall just determined (P'_p). The intersection of the lines for R_W and P_W then establishes the magnitude of the wall resistance P_W. The force polygon for the wall when c' = 250 lb./sq.ft. is then drawn, with the origin at point Y arranged so that the plotting of the weight force will start from point Z as before. P_W when c' = 250 lb./sq.ft. is then determined.

Several cases are determined as outlined until the maximum active and the minimum passive forces are obtained. A plot is then made, as shown, of the maximum active force (P_A) and the minimum wall resistance (P_W) obtained by using the minimum passive force for c' = 0 and c' = 250 lb./sq.ft. The point where the plots of the two forces intersect gives the value of c' required for stability, i.e. when the wall resistance is just equal to the active force.

ILLUSTRATIVE PROCEDURE FOR ANALYSIS BY PLANES

FIG. 17



VARIATION OF WALL RESISTANCE & c'
WITH DEPTH OF PASSIVE WEDGE
AT MILL LANE RETAINING WALL

DRAWING SCALE 1 in. = 5 ft.

FIG. 18.

FACTOR OF SAFETY, ANALYSIS BY PLANES

The factor of safety (F) when the analysis is for assumed plane failure, may be determined by two methods:

(a) The ratio of the wall resistance to the active force.

(b) The ratio of the available shear strength of the soil to that required for stability.

Although method (a) is commonly employed to determine F (see for instance Peck, Hanson and Thornburn, 1952, p.363) such a procedure is not inherently logical. It implies that the force polygon for the wall resistance may be reduced in scale by the factor, at which point the wall resistance is just equal to the active force. By reducing the scale of the force polygon in this manner the weight and water force sides of the polygon are reduced and this is a basically incorrect procedure since these forces are always fully present.

The use of method (b) which is also used for the analysis by circular arcs, eliminates this inconsistency by applying the factor to the shear strength parameters, and is the procedure recommended by the Civil Engineering Code of Practice on Earth Retaining Structures. In this use of limit design methods, incipient failure is assumed and the shear strength necessary for stability is

compared with that available.

This may be expressed in equation form as:

$$s_m = \frac{1}{F}\left\{ c' + (\sigma_n - u) \tan \phi' \right\}$$

(Bishop 1954)

where s_m is the shear strength mobilized for stability.

The procedure used in applying this equation to the force polygon is shown in Fig. 32 and consists of assuming some value of factor of safety and replotting the force polygons using $c'_m = \frac{c'}{F}$ and $\phi'_m = \tan^{-1}(\frac{\tan \phi'}{F})$ as the shear strength parameters. The active force and wall resistance can then be determined at the factor of safety assumed. By choice of suitable further factors of safety, generally only two other than the values at F = 1 are necessary, a plot of active force and wall resistance against factor of safety may be made. The point at which these curves intersect is the factor of safety applicable to the case.

CASE HISTORIES

The following nine case histories are arranged in order of their age at failure. The figures and plates to accompany the histories are in Volume II.

WEMBLEY HILL RETAINING WALL

Reference Nos. 10 & 18

History

This cutting and wall which are shown in plan and section in Fig. 19 were constructed in 1905, the wall being built in trench. The cutting is one mile in length and with a maximum depth of 60 ft., is the deepest of the cases considered in this thesis. The wall was constructed of concrete faced with brick and was strengthened when built by concrete blocks at the toe, 10 feet long extending 6 feet below the wall and spaced at intervals of one chain along the front of the wall.

No trouble was experienced with the wall or slopes until February 1918, at which time movement of the track next to a portion of the wall was reported. From then on, the wall was watched carefully and it was observed that the movement was extending. Consideration was being given to placing struts from the wall in question to the smaller wall and the slope opposite, but, before anything was done, a portion of the wall nearly 500 feet long moved up to 20 feet forward. This movement, which completely disrupted the three railway lines next to the wall was accompanied by a movement of the slope behind the wall

over the same length and extending a short distance beyond the top of the cutting. The major portion of the movement took place in half an hour and was preceded by a loud report, apparently caused by the breaking of the wall.

The height of the portion of the wall that failed varied from 24 to 29 feet above rail level. For the accompanying analyses the section used is that given in "The Engineer" (20th December, 1918) and, being 26.5 feet high, represents approximately an average section.

Remedial measures consisted of cutting the slope back to 4 to 1, in place of the original 3 to 1, and the rebuilding of the wall. The new wall was lower than the original and founded deeper in the ground, in addition the base of the destroyed wall was left in place below track level to act as a buttress.

Site Investigation

There is no record of any investigation being carried out at the time of the failure, and the failure was said to be " ... due to the characteristics of the blue clay which in the Wembley cutting underlies the

yellow London Clay." (The Engineer, December 20th, 1918). However, in excavating to rebuild the wall " ... planes, the surfaces of which were greasy, were encountered at the bottom and below the old wall", it appears that the slip surface has been uncovered here and the characteristic zone of higher moisture content due to dilatency under shear stress observed.

Four piezometers, three in the slip area and one on the slope outside it were installed in April 1956 to determine the water levels in the slope. Readings of water levels have been taken from that time and are continuing.

Observation of the material removed from the boreholes made to install the piezometers, indicate that approximately the lower half of the cutting is in unweathered London clay, the upper portion being in weathered London clay which becomes slightly sandy near the top of the slope. A short distance beyond the top of the slope the London clay is overlain by pebble gravel. Atterberg limits on three samples from the boreholes for the piezometers give the following results:

Borehole	Depth to Sample	Liquid Limit	Plastic Limit	
A (1)	13' 4"	69	25	44
B (2)	9' 3"	81	31	50
C (3)	20' 0"	86	27	

Analysis

Although the final failure at this site took place in about half an hour, this unusual speed for a failure in London clay appears to have been caused by the sudden withdrawal of support occurring when the wall broke. The true failure must be considered the initial movement of the wall and slope which was taking place over a period of some months before the sudden catastrophic movement. Such a slow gradual movement is undoubtedly a drained failure and is here taken as such for purposes of analysis.

The slope was cut back in the failure area and behind the remainder of the wall as a remedial measure soon after the failure. Hence, the observed water levels could not be used directly to give the water levels in the slope at the time of failure. A compromise was made between the measured water levels and these levels adjusted for the change in ground level, as shown in Fig. 19.

The first analysis was by assuming failure on planes, see Figs. 20 and 21. The active force was determined in the usual manner and the wall resistance was determined, first, for the case of no concrete blocks at the toe and secondly with the concrete blocks in

position, assuming failure as shown in Fig. 21. This method of approach in using the same active force for each case implies that the toe blocks would have no effect on the shape of the active wedge. Although no proof for or against this premise can be offered it seems reasonable to suppose, considering the geometry of the case, that blocks extending 10 feet out of each 66 feet along the front of the wall would not have a great effect on the shape of the active wedge.

In the graph of Fig. 20 the value of the maximum active force when c' = 0 and when c' = 250 lb./sq.ft. has been plotted and the minimum wall resistance at each of these values plotted for each of the cases mentioned in the above paragraph. Undoubtedly the effect of the toe blocks is somewhat greater than their actual proportion to the length of the wall which is $10/66$. The exact effect cannot be determined, but it is estimated that the toe blocks increase the wall resistance to the higher value shown on the graph for one quarter of its length. Accordingly, a line has been drawn on the graph one quarter way between the two values for wall resistance and this line is considered to be the actual wall resistance. On this basis it is found that a c' of 190 lb./sq.ft. is required for stability.

The second analysis for this case was made by assuming failure on circular arcs. Values of c' required for stability were first determined for circles passing through a bottom corner of the wall assuming there were no toe blocks. Then using the same centres the values of c' were determined with the blocks in position. For the latter determination it was assumed, as with planes, that the portion of the failure surface behind the wall would be unaffected by the blocks at the toe. This assumption indicates a certain amount of discontinuity in the slip surface under the wall but is probably a reasonable approximation to the actual failure surface, since single arcs passing under the wall and toe block require a smaller c' for stability.

With no toe blocks in position a c' of 270 lb./sq.ft. would be required for stability, see Fig. 22, and with continuous toe blocks a c' of 150 lb./sq.ft. would be required. Following the same procedure as in the analysis by planes the actual c' required is estimated to be one quarter way between these values or 240 lb./sq.ft.

Factors of safety have been determined for this wall when c' = 0 and when c' = 250 lb./sq.ft. for both analysis on planes and by circles.

A summary of results is:

Type of Analysis	c' required for stability	Factor of Safety	
		c' = 0	c' = 250
By Planes	190	0.73	1.08
By Circular Arcs	240	0.73	1.02

Remedial Measures

An analysis by planes was carried out for the revised cross section existing after repairs, which consisted of a decrease in the height of the wall and a reduction in the inclination of the slope to 4 to 1. The measured water levels were used and the analysis was carried out with the assumption used previously that the toe blocks were effective for $\frac{1}{4}$ the length of the wall.

Results of the analysis are:

Type of Analysis	Factor of Safety	
	when c' = 190 (i.e. at construction)	when c' = 0 (i.e. ultimate)
By Planes	1.39	1.01

The above values obtained for the factor of safety must be considered as approximations since conditions

at the wall where the base of the old wall was left in position, make any straight forward analysis impossible.

PARK VILLAGE EAST RETAINING WALL

Reference Nos. 6 & 25

History

At this site there is a brick retaining wall on a concrete base founded on unweathered London clay and supporting a 34 ft. deep face of London clay the top 25 ft. of which is weathered. See Fig. 23 for plan and section. The wall was built in trench in 1901-02, when the original cutting which dates from 1838 was considerably widened. It is the retaining wall of the first cutting that is mentioned in Section 2 and the trouble with this 1838 wall was discussed in several early considerations of failure in London clay. (Cowper 1844, Baker 1881).

By 1920, 19 years after construction, serious movement of the 1901 wall occurred and six concrete counterforts, as shown in the plan and section, were installed. After this installation, movement was very slight but increased slowly so that by 1940 further remedial measures became necessary. Total movement up to this time amounted to 36 in. at the top of the wall and 18 in. at track level.

Remedial measures consisted of counterforts filling in the areas between the 1920 counterforts and

installing further counterforts so that the cross section of the wall was virtually doubled over the failed length. No further movements have been observed since this installation. While the trenches for the construction of the additional counterforts were open the forces in the struts supporting the exposed wall of earth were measured (Golder 1948).

Site Investigation

There is no record of any soil investigation being carried out in 1920. However, in 1940 a number of boreholes were put down, as shown in the plan, and samples taken. Using the results of shear and compression tests, analysis done at that time indicated a factor of safety of over two.

"Tube wells" were installed in the boreholes and sealed off from surface water with clay seals several feet thick. Water levels were measured in these wells from May to December, 1940. The levels in front of the wall were normal for water levels under tracks, being one to three feet below ground, and showed a seasonal variation reaching a peak in mid-November. The two

boreholes behind the wall, however, showed great disparity in water levels, hole A stayed continuously at about the same level as the water in front of the wall, while the water level in hole B was at times 30 ft. higher than this, reaching 7 ft. from the surface. No explanation can be offered for the difference as both boreholes were 4 ft. behind a failed portion of the wall and installed under similar circumstances. The water level of 7 ft. from the surface seems absurdly high since the area around the borehole is completely covered with pavement. On the other hand, water level behind the wall at the same elevation as in front is extraordinarily low. Because of the great difference in these levels little confidence can be placed in them and therefore an estimate of the water level behind the wall is the best that can be done under the circumstances.

To indicate the probable range of effective cohesion necessary for stability three water levels were taken and the analysis done for each level.

The levels, see Fig. 24, were taken with the following reasons in mind:

Lowest: Assuming the canal was the only source of ground water. This is the lowest possible level.

Intermediate: Since the sewer is a potential source of water and there is probably some seepage from the surface.

Highest: Assuming considerable seepage from the surface at some seasons and the possibility of leakage from the 5" service water main.

Analysis

Analysis was by assuming failure on planes and was done in the usual manner. This wall has a sloping base but for reasons explained in Section 31 failure was assumed to take place on a horizontal plane under the wall and the c' required for stability, and the possible range of factor of safety, were determined on this basis. For comparison c' was also determined assuming failure from the corner of the wall.

Results of the analyses are:-

No.	Water Level	Position of Passive	c' for Stability lb./sq.ft.	F	
				c' = 0	c' = 250
1	Lowest	To Horizontal	45		
2	"	To Corner	10		
3	Intermediate	To Horizontal	125	0.82	1.16
4	"	To Corner	92		
5	Highest	To Horizontal	182		
6	"	To Corner	152		

It will be observed that the passive to the corner of the wall requires a c' of about 30 lb./sq.ft. less, this indicates a difference of about 5% in factor of safety for the two methods.

As the exact water levels are not known for this wall the effective cohesion required for stability must be considered to be within a range of values rather than any particular amount. It would seem that the actual water level is probably about the intermediate or highest level shown on the section and therefore the range of c' required is considered to be between the values obtained at these two water levels. The actual age of this wall is 19 years. However, the canal cut was made in the 18th century and the first railroad cutting before 1838 so that here the clay has probably had more opportunity to expand laterally than at any other site. It is impossible to estimate the effect of these cuttings on the virtual age of the wall beyond saying it increases it. The values of c' therefore have been plotted at 19 years but this must be considered the minimum age at this site.

Remedial measures consisted of the installation of counterforts behind the wall. Assuming the counterfort to be continuous and failure to take place as shown in Fig. 24 the following results were obtained for an analysis with the intermediate water level.

Section	F	
	$c' = 125$	$c' = 0$
Wall + Counterfort	1.17	0.99

MILL LANE RETAINING WALL

Reference Nos. 9 & 11

History

The railway line through this cutting was opened in 1868, however the cutting was enlarged to more than double its original width in 1902 and it was then that the wall being considered was built. The portion of the wall where failure took place lies between two bridges which are 600 feet apart and is constructed of concrete with a thick facing of brick keyed into the concrete. Total height varies from 30 to 46 feet, the height above track level varying from 21 to 35 feet; (See Figs. 26 and 27 for plan and sections), and there is a small surcharge on the wall, the cutting being generally about 40 feet deep. The retained material is 20 feet of weathered London clay underlain by unweathered London clay which extends to a depth of 250 or 300 feet. Weep pipes of 3 in. diameter were placed down the back and through the wall above track level during construction, as shown in one section, but there is no record of the spacing of these pipes. The counterfort shown in the plan mid-way between the two bridges was, as far as is known, placed there when the wall was built.

Trouble was first recorded with this wall in 1930. Three houses nearest the wall were demolished at that time and after that numerous cracks appeared in the walk and pavements of the house at the opposite end of the wall. No major movement was recorded until the 27th January, 1943; at that time a 7" depression appeared in the roadway just behind the western bridge abutment where Mill Lane crosses the cutting. Four days later a crack became visible in the wall at Section 9, 180 feet from the bridge, and in the following two weeks the wall moved forward 3 feet at this point and the crack widened to 3½ in. Further movement was prevented by the removal of 12 feet of the retained clay from behind the wall.

From the sequence of events it seems the actual failure of the wall was in 1930, or possibly earlier as records do not exist for this portion of its history. After this original failure, considerable load was probably transferred to the bridge abutments as evidenced by the cracks appearing about them, indicating forward movement of the retained earth forcing the abutment to move. The restraining effects of the abutments, and perhaps to a certain extent that of the bridge, were apparently sufficient to give the wall the extra support it needed. Eventually the abutment and bridge moved, possibly because of overstressing due to

a further reduction in c' behind the wall and the triggering effect of considerable rainfall in the few previous days. It was only then that the wall itself was supporting the retained clay and it is important to note that the first sign of distress in the wall was four days after the movement of the abutments, a time in which negative porewater pressure set up by the sudden loading would partially dissipate.

Site Investigation

No detailed records exist of the happenings at the wall in 1930; however, in 1943 the Soil Mechanics Section of the Building Research Station conducted an extensive investigation into the failure, starting while the wall was still moving. Later in the same year the Midland Railway staff conducted a further investigation for their own purposes.

The investigation by the Building Research Station included a total of ten boreholes both in front of and behind the wall and a pit dug at the toe of the wall to observe the slip surface, to take samples and for loading tests. As the boreholes were made while the wall was still moving some of them were cut off by the movement

along the failure surface and this, combined with the observed position of the failure surface in samples taken and in the pit, gives a clear indication of the position of the slip surface (see sections in Fig. 26 and 27). A considerable number of samples were taken in the boreholes. But, as this was long before the advent of drained tests on this type of clay, the tests done were largely immediate triaxial compression tests. Analyses, both by circular arcs and by wedges, made at the time indicate a factor of safety of the order of four or five using the undrained strengths.

Some of the samples taken at this site included the slip surface and a profile of moisture content throughout the length of two samples taken in the pit has been determined and is included in the B.R.S. report and reproduced in Fig. 27. This is probably the first quantitative record of moisture contents across a slip plane in London clay showing the definite increase of moisture content on and about the plane. Although the dilatancy effect for drained shear in heavily over-consolidated clay was unknown at this time these moisture content profiles seem to be a very good example of the phenomenon. Since then the same effect has been observed at other London clay sites and is found in most laboratory drained shear tests on this

type of clay. Water levels were measured in the boreholes apparently after equilibrium had been reached although there is no record of the period of observations. These water levels are the ones shown in section 10, Fig. 26.

Additional borings were done by the Railway staff in November 1943, nine months after the major failure. Samples were taken and undrained tests done, yielding the same order of results as in the earlier investigation. Water levels were recorded in the boreholes but after seven weeks the level had not reached equilibrium in most holes and there is no record of further readings.

For purposes of analysis the B.R.S. observed water levels were used where possible and where no observations existed an estimate, based on water levels at other sections, was used.

Analysis

As will be seen from the cross sections in Fig. 26 and 27, there is considerable variation in height and size of the wall, sections 10 and DD being nearly the

two extremes in size. Because of this variation, analyses were done for three sections, the two extremes just mentioned and an intermediate section, so that the effect of the difference in size could be determined.

Observations at other sites indicate that the water levels measured in open boreholes are generally higher, by two or three feet, than those measured by sealed off piezometers at the bottom of holes of equal depth. Consequently, two water levels were taken on sections 9 and 10 in order to determine the effect of a lower water table on the c' required for stability.

The failure path for Mill Lane was found to be nearly horizontal below the wall, with the passive failure curving upwards from just in front of the wall. A horizontal position for the failure below the wall was assumed in most of the analyses done for this case, but the analysis was also repeated in three cases for failure assumed along the base of the wall to determine the effect of this variable.

Analysis in each case was made by assuming failure on planes. The analysis for the critical forces on section 10 is given in Fig. 28. A summary of results is shown overleaf.

No.	Section	Water level Used	Position of Passive	c' for stability	F when c'=0	F when c'=250
1	10	B.R.S.	To horizontal base	210	0.71	1.05
2	10	2' below B.R.S.	To horizontal base	190	0.75	1.09
3	10	B.R.S.	To corner of wall	145		
4	9	Estimated	To horizontal base	203	0.72	1.07
5	9	3' below above	To horizontal base	162		
6	9	As in No.4	To corner of wall	175		
7	DD	Estimated	To horizontal base	200		
8	DD	Estimated	To corner of wall	175		
After Remedial Measures						when c'=200
9	9	As in No.4	To horizontal base	140	0.76	1.08

From the analyses on different sections it is apparent that the difference in size of the wall has little effect on the c' required, largely owing to the fact that the depth of the wall below ground decreases proportionally with the decrease in height. Variation in the estimated position of the water level, however, does have a substantial effect on the result.

The position of the passive wedge causes much greater variations in c' required than change in size of the section, but for reasons given in the preliminary discussions on the analysis it seems most logical to use the passive from a horizontal base.

It is seen that the lowering of the water level by one foot in the active earth wedge diminishes the c' required by approximately 10 to 13 lb./sq.ft. depending on the size of the section.

The c' required generally is fairly high, in this case the upper limit being 210 lb./sq.ft. There are several points which all tend to make c' actually to be lower than analysis would suggest. They are:

(a) Water level in the open boreholes may be one or two feet higher than the head on the failure surface.

(b) The wall was supported to a certain extent by the bridge abutments at each end and there must have been some strength from beam action of the wall itself.

(c) A large counterfort existed behind the failed portion of the wall and this must have added to the stability, but as its size is not known it cannot be included in the quantitative analysis.

For reason (a) c' could decrease 20 lb./sq.ft. and it seems logical to allow a decrease in c' of nearly the same amount for reasons (b) and (c) together. Therefore, the possible range of c' would seem to be from 210 to 170 lb./sq.ft. This indicates a range of F of 5 or 6%, a not very large variation in a problem of this sort. The wall is therefore considered to have failed 28 years (1902 - 1930) after its construction and the c' required for stability to be in the range 170 to 210 lb./sq.ft.

UXBRIDGE RETAINING WALL

Reference Nos. 48 & 53

History

This wall, which is shown in plan and section in Figs. 29 and 30, was built in 1937 when the original cutting, dating from 1902, was widened. The site is in London clay capped by a gravel layer up to 5 ft. thick. Stiff fissured brown clay exists to a depth of about 24 ft., two thirds the depth of the cutting; below this is stiff fissured blue London clay. At the base of the slope constructed in 1902 there was a small retaining wall varying in height up to 8 ft.

In 1914 a slip, severe enough to stop traffic, occurred on the north or opposite side of the cutting, but no further trouble in the cutting is on record until 1935. However, during the 1914-35 interval some hardcore groynes were constructed on the part of the slope near the wall, so it is probable there was some trouble which has not been recorded. In 1935 several cracks were noted in the two houses nearest to the top of the slope. These cracks were pointed and later found to have opened again, but apparently no further action was taken.

Widening of the cutting took place in 1937 to

create a new approach for Uxbridge Station and the wall being considered was built at that time. To construct the new wall a trench was dug behind the smaller wall mentioned above. During the excavation a failure occurred filling most of the partially completed trench with clay. Although the failure took place at night and there were no eye witnesses it appears to have been almost instantaneous. The loud report accompanying the failure indicates that the sudden collapse of the perhaps ill-designed and over stressed timbering was probably a major contributing factor to this unusually rapid failure in London clay. Because of the speed it can hardly be considered a completely drained failure. However, the dissipation of negative porewater pressure set up by stress in the clay must have been progressing since the trench was opened.

After the re-excavation and re-timbering of the trench the wall was completed without further incident in 1937. The wall is slightly curved in plan and the height varies, over the failed section, from 8 to 26 ft. It is constructed of concrete which was apparently, for the lower half of the wall, poured against the formwork placed on the side of the trench and left there. The formwork was left for the upper

portion of the wall at the section investigated as well, but since it is stepped in, there must have been some backfill placed.

Cracks in the footpath and road at the top of the slope were reported on 21st September, 1954. By the 10th of November subsidence up to 4 in. had occurred in the footpath and by 22nd November it was found that there was difficulty in maintaining track superelevation and alignment below the wall. It was then obvious that the wall was moving forward and a failure of considerable size was taking place.

Remedial measures were immediately started to prevent further movement. Soil was removed from behind the wall to lessen the active force, concrete struts were put in below track level from the front of the wall to a small wall opposite and the track level was raised by the addition of about 4 ft. depth of ballast to increase the passive resistance. Movement of the wall stopped very shortly after the concrete struts were put in. Subsidence at the top of the slope was nearly 3 ft. by this time, see Plate XVI, and the wall had moved forward about 18 in. and was cracked in several places.

Site Investigation

Soil mechanics consultants were called in by the owners to find the reason for the failure and to estimate the efficiency of the remedial measures. A trench was dug to a depth of 29 ft. immediately behind the wall and stepped back in four steps to a depth of 16 ft. at the top of the slope. It was not possible to go deeper behind the wall as water entered the pit in considerable quantity from along its back. The elevation of the bottom of the pit was then about the same as the drain in the cess on the track side of the wall and 7 ft. above the base of the wall.

Boreholes were put down an average of 12 ft. below the bottom of each step in the trench, nearly continuous 4 in. diameter samples being taken. A considerable number of slip planes were discovered in the trench but no single continuous slip plane was encountered. Generally the smaller slip planes when projected, intersected the wall above its base. From their direction it is probable that many of these small slip planes were the result of the 1937 slip. Further evidence pointing to the same conclusion is the layer of blue-grey material or sewage that had seeped in along a plane which was once open and does not seem to be compatible with an active slip surface.

In the examination of the samples from the boreholes a very noticable softened zone was found in one sample (No. 125), and the position of the sample was consistent with a slip plane going to the base of the wall. A moisture content profile for the portion of this sample about the softened zone is given in Fig. 30. Although no slip plane could be seen in the sample the moisture content is indicative of dilatancy that has occurred under shearing stress.

Piezometers were installed and water levels measured at four points behind the wall as shown on the plan Fig. 29.

Testing Data

The soil mechanics consultants took 155 samples in the investigation trench and from the boreholes below the trench. The majority of these samples were 4 in. diameter undisturbed samples. A programme of testing was carried out, the average results being:

Atterberg Limits

	Average of 24 determinations	Range Min.	Max.
Liquid Limit	85	72	99
Plastic Limit	24	22	31
Plasticity Index	61	48	75

Particle Size Distribution

Average of 4 determinations: 56% clay, 42% silt, 2% sand.

Immediate Undrained Triaxial Tests

(cell pressures of 10, 30 & 60 lb./sq.in.)

Average shear strength (117 tests) 3310 lb./sq.ft.

Range 980 to 7500 lb./sq.ft.

In addition to the above tests carried out by the consultants, drained shear tests were carried out at Imperial College. The results are summarized below; for a more complete record see the summary of test data on London clay Table I.

Sample No.	Method of Testing	ø'	c' lb./sq.ft.
13,14,15	Triaxial, Increasing σ_1	20°	290
16,17,18	Triaxial, Increasing σ_1	20½°	270
22,23,24	Triaxial, Constant σ_1	21½°	280

Analysis

For analysis a typical section was taken, the one chosen being that through the investigation trench. This section is approximately the average height for the wall and has the added advantage that the size of the wall

is known exactly at this point, whereas at other sections some doubt exists concerning the exact shape and depth. Failure on planes was assumed and the analysis for the passive to the horizontal line below the wall is shown in Fig. 31. Factors of safety for the same case were determined on a limit design basis as shown in Fig. 32 for $c' = 250$ lb./sq.ft. and $c' = 0$. The results of the analyses are:

Type of Analysis	c' for stability	F	
		c' = 250	c' = 0
Planes, passive to horizontal	138	1.17	0.79
Planes, passive to corner of wall	97		

Because of the rather complicated history of this case, which has already been outlined, it is difficult to give an exact time to failure. Certainly the effective time to failure must be considered greater than the age of the wall which is 17 years (1937 - 1954). At the other extreme the age of the cutting is 52 years (1902 - 1954). From a consideration of the sections and the fact that the upper part of the slope was changed little when the 1937 wall was built it appears logical to take the time to failure as the age of the wall plus one third of the interval between the time of the original cutting and the wall

construction in 1937. This amounts to a time to failure of 29 years.

It is possible that the slip into the construction trench in 1937 may have had some effect on the strength of the clay involved. However, the 1937 slip took place considerably above the probable plane of failure in the 1954 slip and covered only about one fourth the area, therefore any effect from the 1937 slip is not likely to be large.

Effectiveness of Remedial Measures

Wall stability at this site was increased by three measures:

(1) The placing of struts below track level from the toe of the main wall to a small wall opposite,

(2) Raising the track level 3 to 4 ft. by the addition of ballast,

(3) Removing a portion of the clay retained by the wall.

Remedial measures were almost immediately effective, movement of the wall stopping within a day or two of the installation of the first set of struts. As each of the above steps is quantitatively well known and their effect quite positive, this is a useful case on which to determine the effect of each remedial measure and

their combination. Accordingly, this was done first for $c' = 138$ lb./sq.ft. which would give the effectiveness of the operation when carried out, and secondly for $c' = 0$ to obtain the long-term effectiveness.

The factors of safety were determined on a limit design basis in the first instance, and for comparison the factors of safety based on $F = \frac{\text{Wall Resistance}}{\text{Active Force}}$ were also calculated. Results of the determinations are given in Fig. 33. It is apparent that the most effective single measure is the raising of the track level. The calculations show that this operation is very nearly sufficient for stability when $c' = 0$ even without the other measures, and if either of the other are done in addition, stability on a long-term basis is achieved. The conclusion to be reached here is that it was probably unnecessary to remove soil from behind the wall. However, the factor of safety for the total of the three measures is not unduly large on a long-term basis.

The most objectionable remedial measure from an operational point of view was the raising of the track since it increases grades and interferes with train traffic; but it is necessary to insure long-term stability if c' decreases to zero.

NORTHOLT SLOPE

Reference No. 34

History

At this site there is a cutting averaging 32 ft. deep in London clay (Fig. 34). The first cutting was made in 1903 and was for two steam railway lines. In 1936 this cutting was widened 25 ft. to take two electric lines as well, and it was then that the slope now under consideration was uncovered. At that time slag and stone drains nominally 4 ft. 6 in. deep were placed at 33 ft. centres running from nearly the top to the bottom of the slope. In addition smaller drains running half way up the slope were placed between the full length drains. A small wall 3 ft. high retains the toe of the slope.

The surface layer of clay to a depth of about 22 ft. is weathered, and below this the clay is very firm and highly fissured. Although there are some isolated pieces of unweathered clay in the weathered zone the line of demarcation between the zones is nearly horizontal and fairly well defined. Negligible weathering has occurred in the lower part of the slope since the cutting was opened.

Movement first became noticable on the slope in January, 1955, and consisted of a slumping of the bank at

the top of the slope accompanied by a bulging of the lower portion. The small wall at the toe was unaffected by the slope movement. By October, 1955, the toe of the slope was piled up 4 ft. high against the signal cables and lines which ran on supports along the top of the toewall, and remedial measures became necessary. Restoration consisted of placing deep counterforts 6 ft. wide averaging 25 ft. apart along the slope. In addition the slope was trimmed to 1 in 3 instead of the 1 in 2½ of the original surface. A drain 6 ft. deep was placed along the top of the slope discharging into the counterforts, each of which had a concrete base with an open joint tile drain on it discharging to a collection drain running along just behind the wall. Plate XVII shows the appearance of the slope after the completion of the repairs.

Site Investigation

As the owners wished the counterforts to go deeper than the failure, the trench for each counterfort was inspected closely on both sides to locate the slip surface. The upper portion of the trenches was excavated by a dragline and the lower portion by hand, but no slip surface could be found during the digging, however one or two days after the excavating the failure plane showed up

in nearly every trench. Evidently the plane was obscured by the digging and some movement was necessary to make it appear. Work on the slip proceded from west to east and in most cases the slip surface showed up only on the east side of the trench, although it did show up on the west side in a few cases. The general lack of visible slip surfaces on the west sides of the trenches indicates that movement must stop quite soon after the new counterforts are installed. This is probably not surprising as the clay is nearly in a state of equilibrium and the lowering of the porewater pressures by drainage into the trench is probably enough to stop movement without any action of the counterforts.

The recorded slip surfaces at this site are probably the most complete so far recorded for a failure in London clay. Generally the upper half of the surface is nearly circular and the lower half approaching a straight line. In some cases the lower portion of the surface is along the demarcation line between the weathered and the unweathered clay but it is also found in the unweathered zone. Examples of the actual slip surfaces recorded are given in Fig. 34 and photographs of the slip surface as it appeared in the trench and a portion of the slip surface are shown in Plates XVIII and XIX.

A few 1½ in. diameter samples were taken across

the slip plane where it was in weathered clay. This was not a very satisfactory procedure as there was considerable difficulty recovering the sample and once recovered no shear plane could be seen. However, a softened zone about ½" thick, which was probably the region about the failure surface, could be noticed by prodding of the sample. This softened zone had a higher moisture content than the surrounding soil indicating dilatancy.

Where the slip surface was in unweathered clay some blocks about 8" cube, which included the slip surface were taken and broughtto the laboratory for observation. Several moisture content profiles were carefully taken and they indicated a region of higher than average moisture content about the slip surface. A typical example of the moisture content profile from one of these blocks is shown in Fig. 34.

Piezometers were installed at this site to determine porewater pressures for use in the analysis. Three piezometers were installed outside the failure area at the top, centre and bottom of the slope and three were installed in similar positions at the centre of the failure. Readings of water levels have been taken at regular intervals since their installation in November, 1955, and are continuing. Continuity of the records for the piezometers on the active

part of the slope was hampered by the remedial works and one piezometer was lost. After completion of the works four more piezometers were installed to determine the change in water levels. The recorded levels show that the counterforts act as drains quite soon and the standing water level in the slope is at about the bottom of the counterforts.

Testing Data

A 4 in. diameter tube of clay was taken from below the slip plane in the trench near Section 7. Drained triaxial shear tests were done on 1½" diameter samples from this tube under constant σ_1, reducing σ_3 conditions. Results however were very erratic and it was not possible to get a definite $\emptyset'$ or c' from them. More complete details are given in Table I. When checking $\emptyset'$ and c' of clay against concrete (Appendix B) samples of undisturbed material from this tube were used and yielded a $\emptyset'$ of 21½° and a c' of 150 lb./sq.ft.

Atterberg limits give the following average results:

	Liquid Limit	Plastic Limit	Plasticity Index
Weathered Clay	80	29	51
Firm Fissured Clay	78	27	51

Results of immediate undrained shear tests are:

Description of Sample	w %	Cell Press. lb./in.^{2}	c lb./ft.^{2}
Brown weathered clay from trench at Section A - A			
From just above the shear plane	31.0 29.8	30 30	2100 2800
Softer clay from above shear plane	36.4	Lab. Vane Tests	600
Very soft clay from above shear plane and 3 ft. from the surface	45.7 43.7		280 260
Firm fissured clay taken in 8 in. cube across the shear plane at Section B - B, 46 ft. from wall.			
From above shear plane	29.5 29.9	20 20	2260 2200
From below shear plane	29.0 27.8	20 20	2220 2000

Analysis

As the actual failure surface was known in several places at this slip it is logical to attempt to find c' acting on that surface at failure. This was determined for two sections. In the first case, Section A - A, Fig. 34, a circular arc fits in with the observed surface and the profile of the slope at the top, and was used to determine the value of c' necessary for stability, which was found to be 66 lb./sq.ft. In the second case, Section B - B, Fig. 34, the failure surface was very nearly a combination of a circular arc and a straight line and this shape was used for the analysis. Calculations as outlined by Kenney (1956) indicate that c' necessary for stability along this surface is 25 lb./sq.ft.

Although the above values are probably indicative of the true state of affairs within the slope, such a procedure has no predictive value as the failure surface would never be known in advance. Therefore for comparison with the other slopes and retaining walls the highest value of c' by the usual circular arc method of analysis must be determined. Accordingly this case was examined by the assumption of circular arcs for the slip surface.

For the first case the circular arcs were all taken passing through the top corner of the wall (i.e. toe

circles) using the measured water levels available at that time. The use of toe circles results in a fairly deep circle as the critical one, see Fig. 35, and a portion of it is considerably below the level of the top of the wall. Clay at this depth would not have the same opportunity to expand laterally and soften as would the more unconfined clay above the level of the wall. To obtain an indication of the difference in c' necessary with shallower circles the analysis was repeated for circular arcs tangential to the horizontal line through the top of the wall. Results of these analyses are given in Fig. 36.

Water levels used for the above analyses were those obtained in the three months following December, 1955. As this space of time had less rainfall than the recorded average for the same period it is highly probable that the water level at the start of the failure was higher than that measured. Accordingly, a water level at 4 ft. from the surface, which is about the base of the original counterfort drains, was assumed to be effective at failure and the two circular arc analyses were repeated for this water level. Results are shown on the same figure as the previous analysis. Observations of water levels which have continued since the analysis have resulted in higher levels than the measured levels first

used. At the bottom of the slope levels of just over 2 ft. below the surface and levels of 5 ft. below the surface at the centre of the slope have been noted, so that the assumption of a water level at 4 ft. below the surface does not seem to be improbable.

The results obtained for the worst toe circles are given below.

Water Level	c' for stability	F	
		c' = 250	c' = 0
Measured prior to February, 1956	88	1.37	0.79
Assumed 4' from surface	116	1.30	0.72

A summary of results for all the analyses for this case is given in Fig. 37.

This slope had been uncovered 19 years (1936-55) when failure occurred but the original cutting dates from 1903 so that the practical age of the slope is greater. The exact effect of the earlier cutting on this slope cannot be calculated but a logical estimate seems to be to add $\frac{1}{4}$ of the age of the original cutting (1903-36) to the actual age of the slope. This gives its practical age as 35 years (19 plus 16).

The c' necessary for stability at this site must

be considered to be in the range of 88 to 116 lb./sq.ft. for comparison with other cases, and this value has been plotted at the age of 35 years on the c' : age plot.

SUDBURY HILL SLOPE

Reference No. 10

History

The cutting at this site, which is wholly in London clay, was opened in 1900. It is just over ¼ mile in length and varies in depth up to 36 ft. at which point the railway lines go into a tunnel. (See Fig. 38 for a plan of the cutting). Considerable trouble has been encountered with the slopes on both sides of this cutting over the last 10 years and includes the destruction of a small retaining wall. In addition, there are excavations on one side of the cutting indicating a probable earlier slope failure and removal of the slipped material. Old counterfort drains are found in other places along the slopes indicating earlier troubles of which no records exist.

The portion of the retaining wall destroyed, was removed in 1948 and relatively shallow counterforts placed in the slope, the top 2½ ft. of which had been removed and replaced by clinker. This section, see Plate XXI, again failed about 1953 and is still moving slowly downwards.

Within the past four years two other failures

in the slopes of this cutting have been repaired, with some success, by the insertion of counterfort drains of moderate depth.

The slope selected for analysis is directly opposite the site of the damaged wall and is completely in weathered London clay. Movement has been going on since 1949 and by 1955 had amounted to a drop of 7 ft. at the top of the slope with accompanying bulging at the toe. (See Plate XX and the cross section on Fig. 39). A small amount of trimming has been done at the toe of the slope to keep it from encroaching on the tracks of a siding, otherwise there has been no attempt to remedy the situation.

Site Investigation

A total of twelve piezometers were installed in the slopes of this cutting: four at the section used for the analysis, four on the slope opposite where the wall failure had occurred and four at the entrance to the tunnel where a slip had been repaired by the installation of counterfort drains. These piezometers were installed in January, 1956, and readings of water levels were taken until March, 1957.

Observation of the clay removed from all bore-

holes showed it to be weathered London clay, and Atterberg limits on samples from two of the boreholes made to install the piezometers gave the following results:

Borehole	Depth from Surface	Plastic Limit	Liquid Limit
1	6' - 0"	29	82
11	12' - 0"	27	83

A 4 in. diameter sample was taken in a tube 6 ft. from the surface at piezometer No. 1, and was found to be extremely fissured weathered London clay. Three 1½ in. diameter samples from this tube were tested under constant σ_1, reducing σ_3 conditions but the circles obtained were rather erratic so that parameters can only be estimated at c' = 320 lb./sq.ft. and $\phi' = 23^\circ$, for more details see Table I. Clay from the same tube was used in the tests for ϕ' and c' between clay and a rough surface, see Appendix B; and c' there was found to be 120 lb./sq.ft. and ϕ' to be 20°, the test points lying exactly on a straight line.

Analysis

There is no exact record of the original cross-section at this site and failure has progressed to the point where it cannot be determined. Observation at other

slopes in the cutting, that are apparently the original ones, indicate a gradient of nearly 3 to 1 and as this fits in fairly well with the existing cross-section at the failure, it has been used as the cross-section for analysis.

Water levels measured were from the failed surface and had obviously been lowered by the movement at the top of the slope so that they were adjusted for the original ground level as shown in Fig. 39.

Analysis was by assuming failure on circular arcs passing through the toe of the slope and the results are

c' for Stability lb./sq.ft.	F	
	c' = 0	c' = 250
50	0.83	1.60

Circular arcs passing beyond the toe of the slope were also tried but none yielded a c' required as great as the maximum for the toe circles.

The c' of 50 lb./sq.ft. for stability at this site is lower than would be expected considering the time to failure of 49 years. The low value may be partially accounted for by the fact that the failure was completely in weathered clay. There is in addition the possibility

that the water level at the initial failure was higher than that used and the slope slightly steeper, both of which would tend to increase the c' necessary for stability. However, the answer seems to be in keeping with the result shown by the Northolt slope and indicates that the c' available in the case of slopes is lower than that for walls.

KENSAL GREEN RETAINING WALL

Reference No. 7

History

At Kensal Green there is a brick retaining wall on a concrete base supporting a 12 ft. face of weathered London clay and a further slope of 9 ft. depth at 4 to 1 above it. (See Fig. 41 for a plan giving the location and a cross-section). The original cutting at this site dates from c. 1834 and was 2 tracks in width. Two widenings, one of which is thought to have been on the side to be considered, took place between 1872 and 1878. The final widening of the cutting took place in 1912 and it was then that the wall was built. It was constructed in trench and must have been placed as close to the earth face as possible since apparently no backfill was used.

Drainage of the slope was accomplished by a rubble drain at the toe communicating with chases down the back of the wall at 9 ft. centres. Two drains were installed 4 ft. below track level but the one nearest the wall was found to be blocked when uncovered in 1941.

Observations of movement on the wall started in 1929 but at that time no distress was visible, the observations being an extension of those on another wall behind

the station which was seriously cracked in 1927. The wall movements, which are given in Fig. 41 for the section of maximum movement, show the displacement increased at an accelerating rate. By 1938, 26 years after construction, the rate became considerably faster and by 1940 remedial measures were being considered.

In April, 1941, a tension crack appeared in the clay 20 ft. behind the wall. The top of the wall had moved forward 12 in. by this time but the wall was still intact. Later in the same year the wall cracked and movement became so much faster that remedial action was immediately necessary. This consisted of removing a considerable portion of the retained clay from behind the wall, and was successful in stopping further movement. The void left by the removal of the clay was later filled with ashes.

Site Investigation

A number of boreholes were put down, both behind and in front of the wall, early in 1942 and samples taken for testing. A pit was also sunk behind the wall to below its base in order to observe the slip surface and clay but the slip surface could not be clearly identified.

The profile of the clay is usual for London clay and consists of a soft surface layer of 4 to 5 ft. subject

to seasonal variations, below this the clay is weathered and highly fissured with moisture content decreasing slightly with depth. Firm unweathered clay is reached at just below the base of the wall, 28 ft. from the surface. In places a considerable softening due to higher moisture content was noticed about the fissures. This softening was also noticed along fissures just below the base of the wall in the unweathered clay. Samples taken from along the base of the wall near its heel gave moisture contents of 37 and 39% which is considerably higher than the average of 34% found in the weathered clay.

Water level was recorded in a borehole at the top of the slope but there are no records of water levels directly behind the wall except the observation that "the borings behind the wall filled up with water more rapidly than did those in front of the wall", (Ref. 7). Personal observations in 1956 indicate that there is seepage through the wall, where the retained clay has not been removed, at about the mid-point of the exposed face.

Atterberg limits determined in 1942 gave the following results:

Plastic Limit		Liquid Limit	
Mean	Range	Mean	Range
29	25 - 31	82	76 - 92

Analysis

As the exact water levels are not known it is necessary to make an estimate of their range. The lowest probable would be to the level of the drains through the wall and the highest probable a level at the same distance below the slope as the level measured at the top of the slope. An estimate of the water level in front of the wall can be made from the elevation of the drains and in any case is not nearly so critical as the level behind the wall.

Analyses were done for both water levels behind the wall, assuming failure on planes. As the wall has a sloping base, failure was assumed to take place horizontally below the wall in the first instance and repeated for failure along the base for comparison. The analysis for the higher water level and horizontal base is given in Fig. 42.

Results of the analyses are:

Water Level	Position of Passive	c' for Stability	F	
			c' = 0	c' = 250
Higher	To Horizontal	102	.74	1.40
Higher	To Corner	92		
Lower	To Horizontal	40	.92	1.51
Lower	To Corner	25		

The actual time to failure of this wall was 29 years (1912 - 1941). However, the part of the slope above the wall had been excavated c. 1875 and not a great deal of clay had been removed from in front of the wall when it was constructed in 1912. Therefore, it is necessary to add to the actual age of the wall a portion of the age of the cutting and this portion is estimated to be $\frac{3}{4}$ of the 1875 - 1912 interval or 24 years. The practical time to failure for this case is then 29 plus 24, that is 53 years. As the water levels are not known exactly the c' required for stability must be considered to be in the range 40 to 102 lb./sq.ft. Seepage through the wall, however, indicates that the water level at failure was probably such that the actual c' required was nearer the higher value.

WOOD GREEN RETAINING WALL

Reference No. 5

History

This wall, which is shown in plan and section in Fig. 43 and pictured in Plates XXII and XXIII, is founded on London clay and supports a 16 ft. high face of the clay which rises a further 21 ft. above the wall at a slope of 3 to 1. The wall was built of concrete in 1893 and gave satisfactory service until November, 1948. At that time forward movement of the wall was noticed with accompanying heave of the tracks in front of the wall and slumping at the top of the slope. Records of wall movement were taken and it was found that from November, 1948, to March, 1949, the top of the wall moved forward 19 in. and the bottom 14 in. at the section of maximum movement.

Remedial measures which were completed by August, 1949, consisted of driving steel sheet piling at the toe of the wall, the placing of small buttresses to the front of the wall and struts from the toe to a small concrete anchor running along between the two nearest railway lines. In addition five mass concrete counter-weights were placed behind the wall and attached to it and a drain was installed in front of the wall at about

the level of its base, but apparently this drain was never provided with an outlet.

These works were thought to have stopped the movement but observations started in October, 1949, showed the wall to be still moving, although more slowly. This slow movement was still continuing in 1953 when a firm of soil mechanics consultants were called in. They recommended the installation of horizontal boreholes through the wall to drain water-logged portions of the slope, tracing and intercepting the source of water at the top of the slope and filling the drain installed in 1949 with grout. As a result of these recommendations two horizontal boreholes were drilled through the wall to connect with vertical holes at the centre of the slope. These two holes apparently did not lower the water levels in surrounding vertical boreholes and no further drains were installed.

Movement of the wall continued slowly so that by December, 1954, the total movement was 30 in. at the top of the wall and 20 in. at track level.

As a further remedial measure cement grout was injected into the slope early in 1956. The results of this operation are not known at the present time as movement is usually in the winter and the effectiveness cannot be judged until after the winter of 1956-57.

Site Investigation

In 1953 the Railway staff put down four boreholes in the slope and took samples. The samples were all weathered London clay and gave an average unconfined compression strength of 1500 lb./sq.ft. and had an average moisture content of 34%. Atterberg limits were found to be L.L. = 78 and P.L. = 30. Later in the same year the firm of consultants put down 10 further boreholes and the two horizontal drains mentioned earlier.

A pipe was placed down each of the 10 boreholes and they were filled with gravel, sealed off at the top and a cover fitted. Water levels were observed in the boreholes over the six month period December, 1953, to June, 1954. The highest levels are shown on the cross-section of Fig. 43, and have been used in the analysis.

Analysis

The first analysis, see Fig. 44, for this case was carried out by assuming failure on planes and using the measured water levels. Failure on a circular arc was then assumed and the analysis repeated. In each case the factor of safety when c' = 0 and c' = 250 lb./sq.ft. was also determined.

A "non-circular" analysis was also done at this

wall in order to compare its results with the other analyses.

A summary of results follows.

Type of Analysis	c' for Stability	F	
		c' = 0	c' = 250
(1) Planes	116	.77	1.28
(2) Circles	159	.72	1.15
(3) Non-Circular	185		

The value of c' necessary for stability seems to be rather high considering the wall's age of 55 years. Undoubtedly the high water level, which was measured, is a major contributing factor to this. The water levels were measured in virtually open boreholes and as it has been found that such levels are generally higher than levels from sealed-in piezometers the true value of c' is probably lower than that indicated by the analysis. However, since the exact amount is not known the values shown must be taken as a reasonable estimate.

Effectiveness of Remedial Measures

Although a considerable amount of effort was expended on remedial measures in 1949 it is obvious that they were of little use. This is perhaps not surprising as buttressing systems and struts which transfer the load

from the wall to the passive earth wedge which is moving with the wall, and drains without outlets are not particularly significant engineering operations. The installation of mass concrete blocks at the back of a wall does not seem to increase the overall stability of the wall appreciably unless the blocks go considerably deeper than the walls, as at Park Village East. The sheet piling driven in front of the wall to depths of 15 and 25 ft. seems to have had little effect on the movement, and as the piling was not anchored in any way considerable movement may be necessary to mobilize its full resistance. The effect of the grouting cannot be determined at present but will eventually be forthcoming.

UPPER HOLLOWAY SLOPE AND RETAINING WALL

Reference No. 8

History

At Upper Holloway there is a cutting 18 feet deep, all in weathered London clay. The lower 4 feet of the slope is held by a brick retaining wall and the upper portion runs back at 2½ to 1 from the wall (see Fig. 46 and Plate XXIV). The railroad line was opened in October, 1870, and it appears that the wall was built just prior to this.

No trouble was recorded in this cutting until 1951. At that time a portion of the wall was forced forward and damaged to the extent that it had to be replaced but the amount of movement has not been recorded. This movement was accompanied by a downward movement of the top of the slope.

The new wall was built slightly larger than the old one but by 1953 it also started to move forward. From July, 1953, to April, 1955, the forward movement was 11½ in. at the worst point. During this period five timber struts 16" x 8" were placed from the wall to the opposite side of the cutting. These struts have apparently stopped the movement, and appear to be taking a considerable load since

some of them are noticably bowed. Personal observations indicate the total movement of the top of the wall to be about 26 in at the maximum point. No records have been kept as to whether the wall moved forward equally at both top and bottom or not. In 1955 as an attempt further to stabilize this slope and wall a row of Larsen No. 2 steel sheet piling alternately 20 and 30 ft. long was driven about 10 feet behind the wall.

Site Investigation

During an investigation in 1955 the owners put down eight boreholes, on the slope, varying in depth up to 24 feet. Samples 1½ in. diameter were taken in four of these holes and unconfined compression tests carried out. The average unconfined compression strength was 2290 lb./sq.ft. (mean of 21 tests, range 400 to 3390 lb./sq.ft.) and the average moisture content 32.6% (mean of 20 determinations, range 28.1 to 46.0%).

Four Casagrande type piezometers were installed at this site, two on a section near the centre of the failure and two on a section outside the failure area and water levels have been observed since mid-April, 1956. At this site the water level at the tracks can be observed in catch basins between the two lines and next to the wall

and it varies from 10 in. to 20 in. below the base of rail. Pumping of clay up through the ballast occurs in several places along this cutting and the action is doubtless aided by the unusually high water table in the track bed.

Atterberg limits on material from the boring for piezometer A are L.L. 86% and P.L. 28%.

Analysis

This case was first examined by using plane surfaces of failure, see Fig. 47 and the effective cohesion required for stability was found to be 38 lb./sq.ft. In a further examination, slip circles were assumed as the failure surface and it was found that the worst circle gave a c' required of 68 lb./sq.ft. (Fig. 48). Factors of safety were determined for this case when c' = 250 lb./sq.ft. and when c' = 0, (see Fig. 49 for F on circular arcs.) A summary of results is:

Type of Failure Surface	c' required for stability	Factor of Safety	
		when c' = 250	when c' = 0
Planes	38	1.95	.86
Circular Arc	68	1.62	.73

Effectiveness of Remedial Measures

It is quite obvious that the slightly larger

retaining wall built in 1951 was uneffective as a remedial measure. Once a slip has started in London clay it appears that the c' is further lowered by the remoulding about the slip surface. It therefore follows that once a failure has started more force is required to hold it in check than was required to hold the original wall or slope. The failure of the 1951 wall bears out this fact and it appears that the increment in wall size was not enough to compensate for the reduction in c'.

Movement of the wall had apparently stopped before the installation of the sheet piling and it appears from their bowed shape that the struts are taking a considerable portion of the load, and are highly stressed. Whether or not the sheet piling is taking any load is unknown, but it is probable that appreciable movement would have to occur to mobilize its strength. Even then it is open to question, if the strength could be fully used and if so, would it be sufficient. The design of the sheet piling was apparently empirical and the present conditions at the wall defy the determination of any quantitative factor of safety. However, in the light of present knowledge it would seem that additional and stronger struts would serve the wall as well, if not better, than the sheet piling and probably be less expensive.

SUMMARY OF RESULTS FROM EXAMINATION OF FAILURES

The results of the analyses of the nine failures considered are summarized in Table VII. Cohesion, in terms of effective stresses, (c') required for stability has been shown in most cases as a range rather than a single value. The different values given exist for two reasons:

(a) Sufficient data was not available to establish accurately the water table in some cases, and, in other cases, it could not be obtained at the present time due to changes at the site.

(b) Different methods of analysis give different answers, all other things being equal.

In consideration of the differences shown by the range of values of c' it should be borne in mind that the cohesion is on the average only about 20% of the shear strength of the clay involved in the failure. Therefore, if the difference in values of c' is 30% it indicates an average difference of 6% in the factor of safety, a not unreasonable value for most problems in clay.

Results show that the choice of assumed failure surface can have a considerable effect on the value of c' required for stability. The assumption of failure along a

a circular arc yielded in every case a higher value of c' necessary for stability than when failure on planes was assumed. Actual failure surfaces have been found to be nearly planes in the case of walls without a slope above them (e.g. Mill Lane), but in the case of a wall with a proportionally large slope behind it (e.g. Upper Holloway) the failure surface probably tends to be more the shape of that found in a slope. Hence, the range shown for the value of c' in the case of combined walls and slopes must cover the actual value of c' required.

Table VII also shows values of the factor of safety of each structure when c' = 250 lb./sq.ft., that is at construction, and when c' = 0. The latter figure is of course hypothetical, since factors of safety less than one cannot exist, but it serves to show the possible range of factor of safety with the possible range of c' and indicates what proportion of the shear strength was contributed by each of its components at the time of failure. It will be noted that on the average c' amounted to approximately 20% of the shear strength at failure, the frictional component contributing the other 80%.

As shear strength on an effective stress basis is made up of two components, a constant one, and one which varies with the effective normal pressure on the shear plane, the ratio of the components will vary with the

CASE	AGE AT FAILURE	TYPE OF ANALYSIS AND DETAILS	c' FOR STABILITY LB/SQ.FT.	F	
				WHEN c' = 250	WHEN c' = 0
WEMBLEY HILL	13	PLANES, MEASURED WATER LEVEL	190	1·08	0·73
		CIRCLES, " " "	240	1·02	0·73
PARK VILLAGE EAST	19	PLANES, HIGHEST W.L.	182		
		PLANES, INTERMEDIATE W.L.	125	1·16	0·82
MILL LANE	28	PLANES, B.R.S. SECTION, MEAS'D W.L.	210	1·05	0·71
		" " " 2' LOWER W.L.	190	1·09	0·75
UXBRIDGE	29	PLANES, MEASURED W.L.	138	1·17	0·79
NORTHOLT	35	TOE CIRCLES, MEASURED W.L.	88	1·37	0·79
		" " , W.L. 4' FROM SURFACE	116	1·30	0·72
SUDBURY HILL	49	CIRCLES, MEASURED W.L.	50	1·60	0·83
KENSAL GREEN	53	PLANES, HIGHEST PROBABLE W.L.	102	1.40	0.74
		" , LOWEST " "	40	1.51	0.92
WOOD GREEN	55	PLANES, MEASURED W.L.	116	1·28	0·77
		CIRCLES, " "	159	1·15	0·72
UPPER HOLLOWAY	81	PLANES, MEASURED W.L.	38	1·95	0·86
		CIRCLES, " "	68	1·62	0·73

TABLE VII

TABLE VII SUMMARY OF RESULTS OF ANALYSES

effective normal pressure on the failure surface. Also, for the same reason, the range of the factor of safety with the possible range of c' will vary with the depth to the failure surface. This is shown very distinctly by the calculations, the smallest range of factor of safety being at Wembley Hill, and the largest range being at Upper Holloway.

It is significant that in all cases the c' required for stability was within the possible range of this value in London clay. This fact in itself clearly shows the possibility of using effective stress analyses for problems in over-consolidated clay. The ranges of factor of safety show that the maximum possible error obtainable with this type of analysis, even if nothing is known about the value of c', will under no circumstances approach the error in factor of safety commonly obtained by using total stress ($\emptyset = 0$) analysis for this type of clay.

VARIATION OF c' WITH AGE AND σ_n'

The major purpose of the analyses was to determine if a rate of decrease could be obtained for the cohesive portion of the shear strength. Values of the c' required for stability have been plotted against the practical age at failure for each case in Fig. 50. The

practical age in some cases, as detailed under the case histories, is greater than the actual age of the structure since the construction of the slope or retaining wall under study was the result of widening an existing cutting.

Values of c' plotted in Fig. 50 show a definite trend to be smaller with age and a curve has been drawn giving an approximate rate of decrease for c'.

Reduction in c' can only take place if there has been some reduction in the lateral earth pressure. A reduction in the lateral earth pressure must take place whenever a cutting is opened, but the reduction at any potential failure surface will in all probability depend on the distance to that surface. This would indicate that the reduction in c' probably varies with the effective normal force on the failure plane, being greatest in the shallow failures and least in the deeper ones. There should therefore, probably be a group of curves on Fig. 50 each for a different σ_n'. These curves would all start at c' = 250 lb./sq.ft. and reduce by different amounts with age, the greatest reduction being with the smallest values of σ_n'.

There are indications from the analyses that slopes tend to give lower values of c' required for stability than retaining walls. As slopes would have more

opportunity to expand laterally than the clay behind a retaining wall this does not seem unreasonable. The consideration of σ_n' on the failure surface does not entirely eliminate this inequality since in the case of retaining walls any failure surface used in the analysis is not very far from the actual one, as shown by several of the cases, while in the case of slopes the worst circle found is usually considerably deeper than the actual failure surface.

Average effective normal pressure, on the critical failure surface found by the analysis, has been plotted against time to failure for the nine cases analysed in Fig. 51. There, it is apparent that the oldest cases have the least σ_n' on the failure surface. However, as the effective cohesion is a function of both age and σ_n' on the failure surface, a complete picture can only be obtained by plotting all three on an interrelating graph.

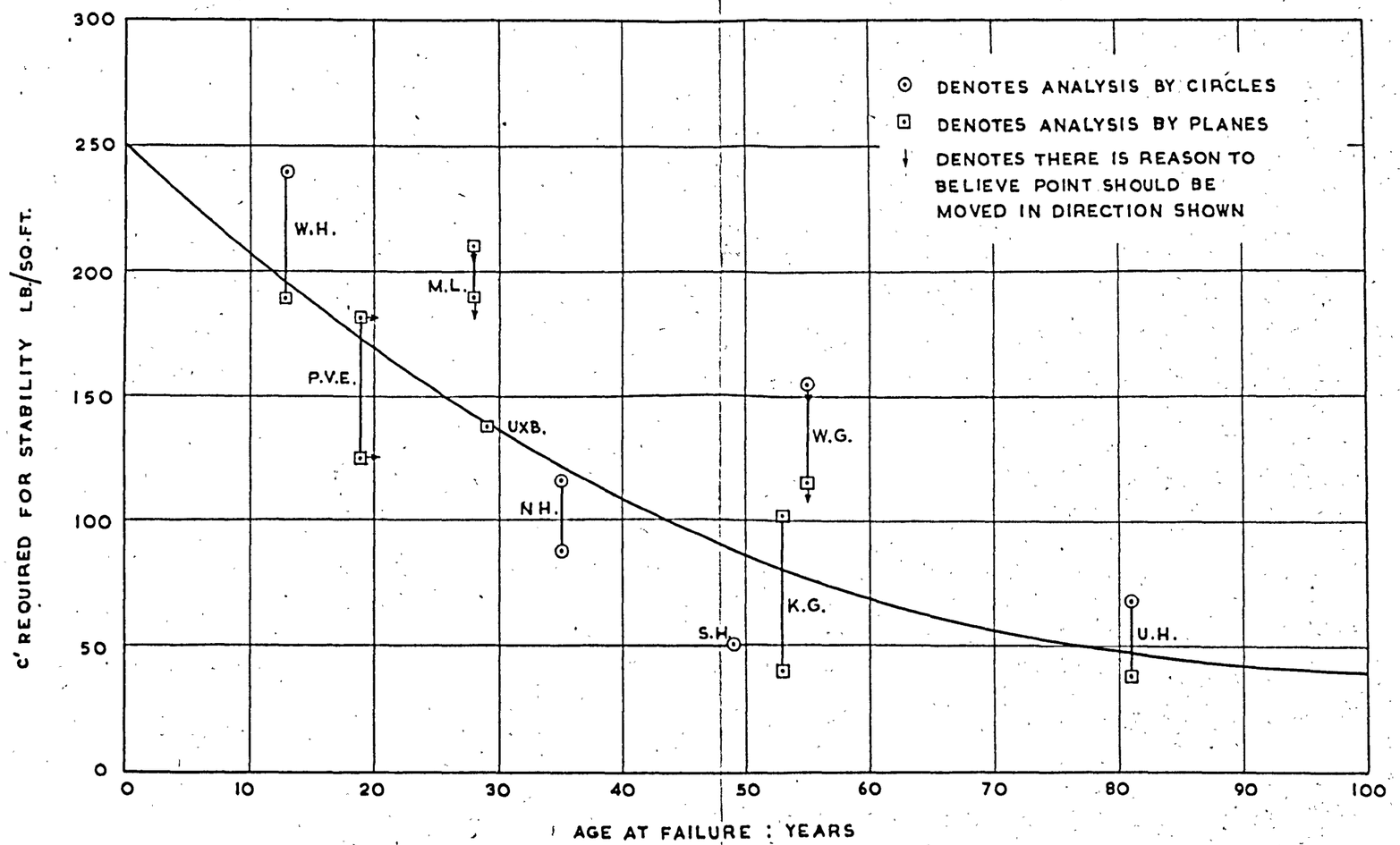
It is found that if c' for stability is noted at each of the points in Fig. 51 radiating lines may be drawn from the origin for different values of c' with some degree of agreement. Fig. 52 shows these lines drawn in a somewhat idealised form on the plot of Fig. 51. Data is actually insufficient to draw true curves for c', as it was to draw various curves of c' versus age at different values of σ_n', but the plot of Fig. 52 is put forward as being as interesting

approach to the problem. Further data may lead to alterations in the plot but it is felt that the basic idea of the plot is correct and it has considerable potential value.

As the plot of Fig. 52 interrelates c', age at failure and σ_n', values from it should lead to a series of curves each for different values of σ_n' on a plot similar to Fig. 50. The idealized values given by the lines on Fig. 52 have been replotted in Fig. 53 for different values of σ_n', using c' and age as axes. The result is a group of curves showing a greater decrease in c' with time at lower values of σ_n'. These curves however must not be considered final but are put forward for the same reason as Fig. 52. It will be noted that the average curve of Fig. 50 when plotted on Fig. 53 does not give the average of the different curves. This comes about as the smaller cases are the older ones, and indicates that the average curve of Fig. 50 may not represent an exact overall average. However, data available at the present time is not sufficient to indicate definitely that this is so, or to plot a more representative average curve.

A plot such as Fig. 52 would be used to check a design in the following manner. The effective normal pressure on the potential failure surface and the c'

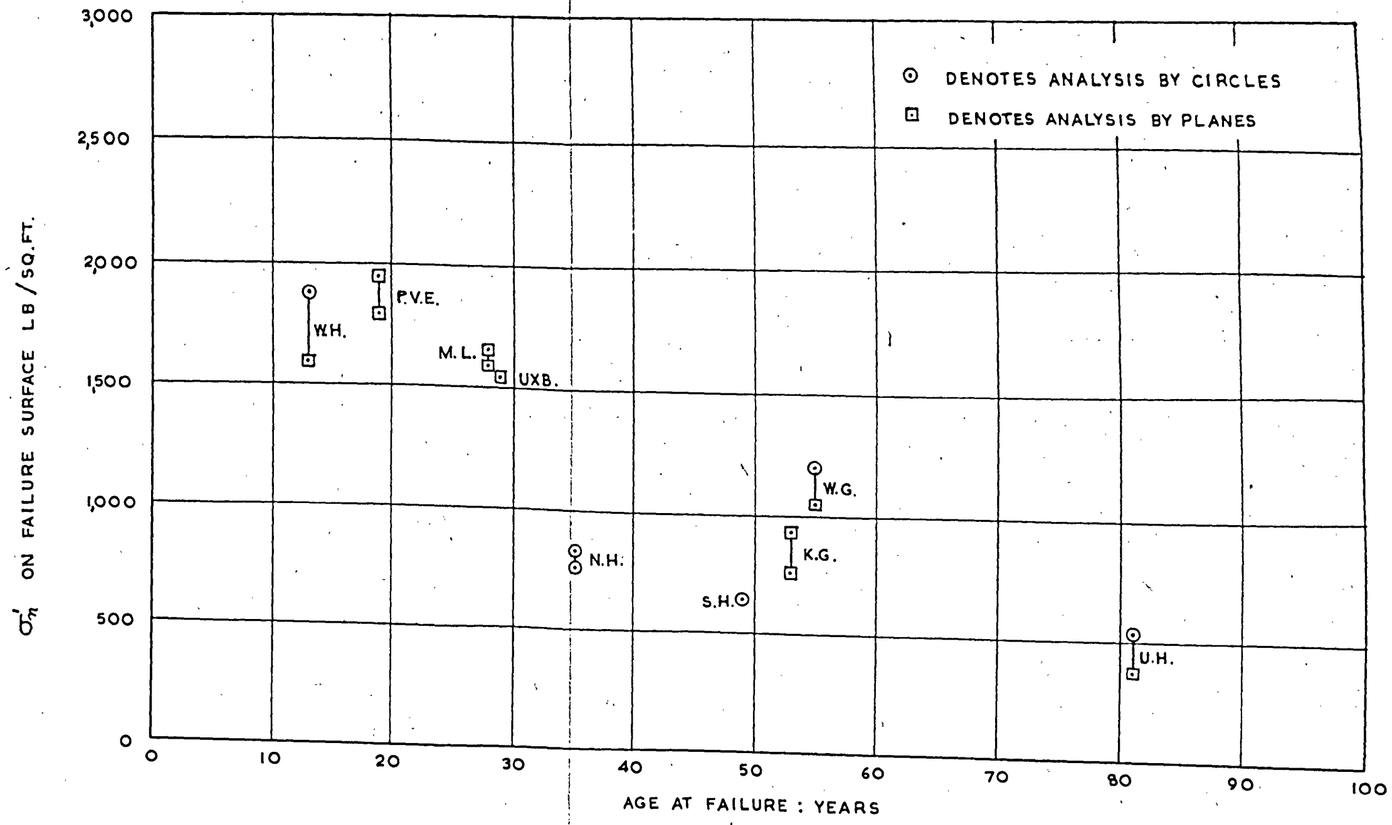
CASE	AGE AT FAILURE	c' FOR STABILITY LB./SQ.FT.
W.H. WEMBLEY HILL	13	190 - 240
P.V.E. PARK VILLAGE EAST	19	125 - 182
M.L. MILL LANE	28	190 - 210
UXB. UXBRIDGE	29	138
NH. NORTHOLT	35	88 - 116
S.H. SUDBURY HILL	49	50
K.G. KENSAL GREEN	53	40 - 102
W.G. WOOD GREEN	55	116 - 155
U.H. UPPER HOLLOWAY	81	38 - 68



AVERAGE VARIATION OF c' REQUIRED FOR STABILITY WITH AGE AT FAILURE, FOR FIELD CASES

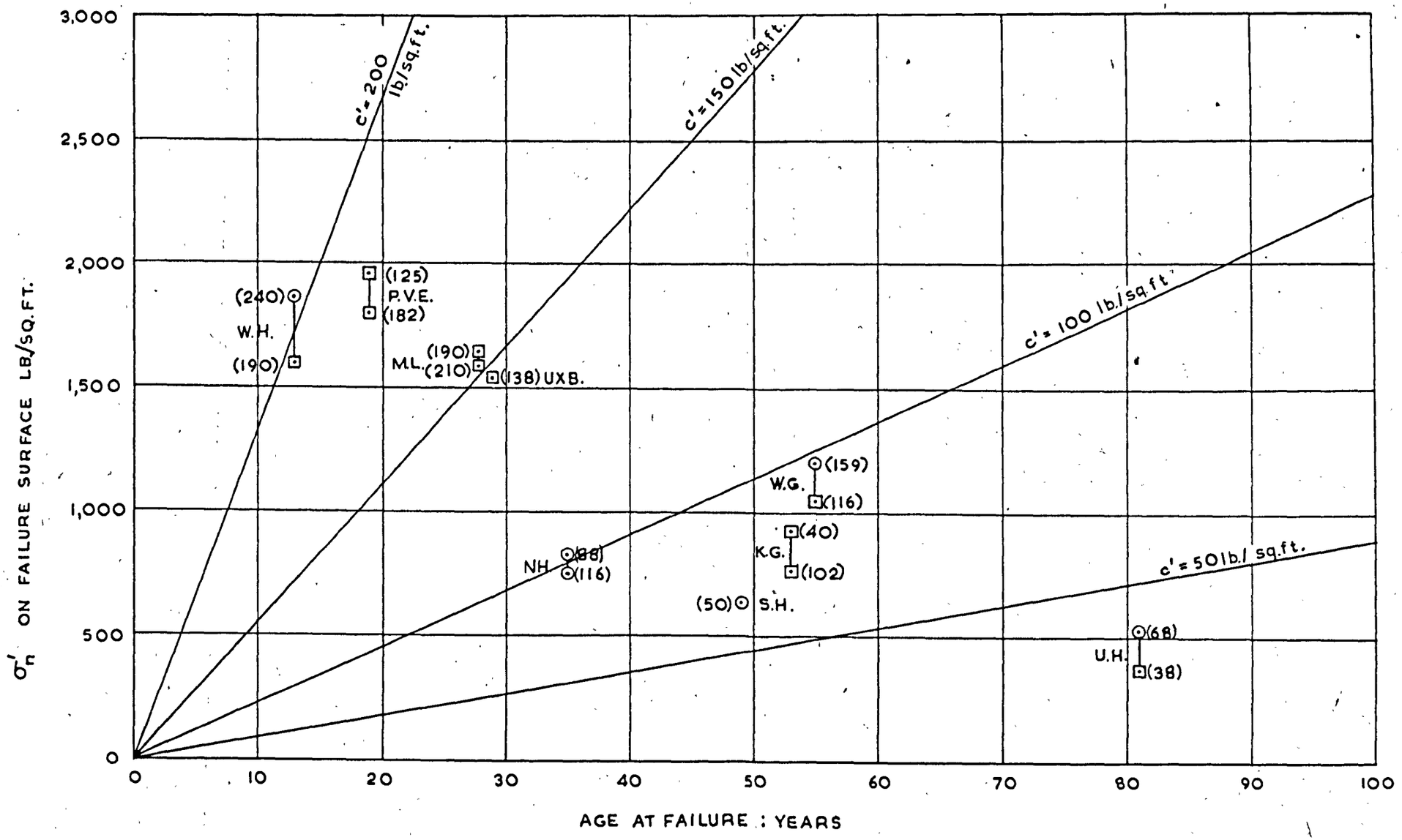
FIG. 50.

CASE	AGE AT FAILURE	c' FOR STABILITY LB/SQ.FT.	σ_n' ON FAILURE SURFACE (lb./sqft)	REMARKS
W.H. WEMBLEY HILL	13	190 240	1600 1870	ON PLANES ON CIRCLES
P.V.E. PARK VILLAGE EAST	19	125 182	1960 1800	INTERMEDIATE W.L. HIGHEST W.L.
M.L. MILL LANE	28	210 190	1590 1650	B.R.S. SECTION, MEAS'D W.L. " " 2' LOWER W.L.
UXB. UXBRIDGE	29	138	1550	ON PLANES
N.H. NORTHOLT	35	88 116	830 760	CIRCLE, MEAS'D W.L. " W.L. 4' DOWN
S.H. SUDBURY HILL	49	50	640	ON CIRCLE
K.G. KENSAL GREEN	53	40 102	930 760	LOWEST WATER LEVEL HIGHEST " "
W.G. WOOD GREEN	55	116 159	1050 1200	ON PLANES ON CIRCLE
U.H. UPPER HOLLOWAY	81	38 68	370 530	ON PLANES ON CIRCLE



VARIATION OF EFFECTIVE NORMAL PRESSURE ON THE FAILURE SURFACE (σ_n') WITH AGE AT FAILURE FOR FIELD CASES

FIG. 51



SUGGESTED RELATIONSHIP BETWEEN σ_n', AGE AT FAILURE AND c'

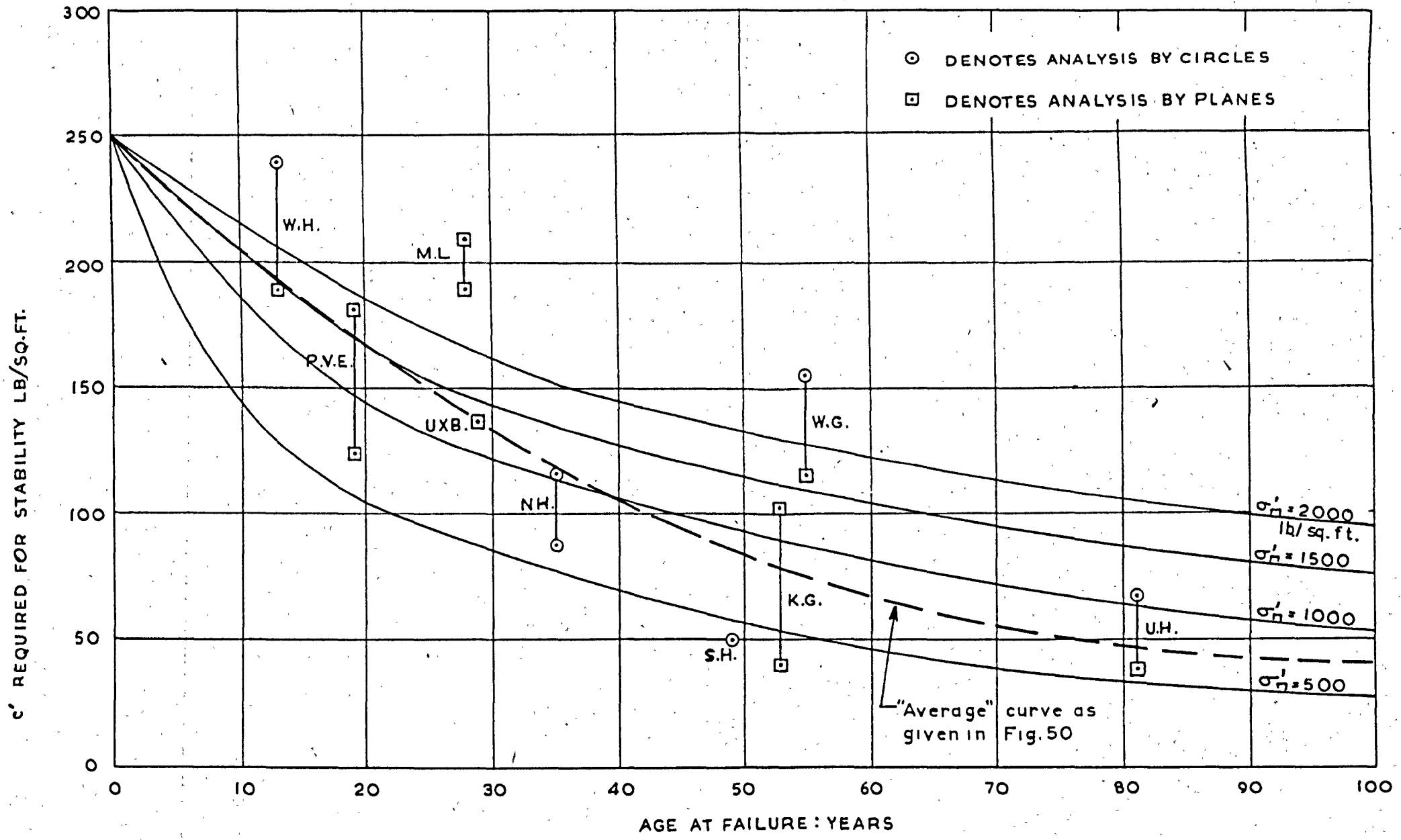
⊙ DENOTES ANALYSIS BY CIRCLES

⊡ DENOTES ANALYSIS BY PLANES

DATA PLOTTED ON THIS SHEET IS GIVEN IN FIG. 51.

FIGURES IN BRACKETS INDICATE c' FOR STABILITY AT POINT

FIG. 52.



SUGGESTED REDUCTION IN c' WITH AGE FOR VARIOUS VALUES OF σ'_n

CURVES SHOWN FOR THE DIFFERENT VALUES OF σ'_n ARE BASED ON THE LINES FOR c' IN FIG. 52.

FIG. 53.

required for stability would be determined. The graph would then be entered at the value of σ_n' involved and moving across the sheet, the position of the intersection with the relevant c' line determined. A vertical line from this point would then give the probable life expectancy of the structure. If the horizontal line at the σ_n' involved fails to intersect the appropriate c' line the probable life expectancy will be beyond that shown by the graph, but present knowledge cannot estimate how far beyond.

REMEDIAL MEASURES

A summary of the calculated factors of safety for the remedial measures carried out at five sites is given in Table VIII. Other cases either had no remedial measures (e.g. Sudbury Hill) or those carried out are not susceptible to ordinary analysis and are perhaps not completely effective (e.g. Upper Holloway, Wood Green).

The factor of safety was first determined for conditions when the remedial measures were carried out using the c' indicated to be present by the analysis. F was then also determined on the assumption that c' would decrease to zero.

Values obtained show that in general the remedial measures considerably increased the factor of

safety. In all cases except Mill Lane it is probable that the structure would still be stable if c' decreases to zero. At Mill Lane the remedial measures were not extensive and consisted of removing some of the retained clay.

Indications at Northolt are that the effectiveness of deep counterforts is largely as drains rather than as buttresses. In practice the drainage effect stops any movement before the shear strength of the counterfort is mobilised and it is questionable if counterfort strength could ever be mobilised, for the method of installation precludes any great effect from frictional properties of the material. The counterfort is commonly arranged to go about one foot deeper than the slip surface. Therefore, a failure surface passing completely under the counterforts has a factor of safety greater by only a few per cent than the failure being repaired and a slip could take place along such a surface before the additional strength of the counterfort was mobilised.

Observations at Northolt and Sydenham Hill indicate that the rubble drains commonly used in London clay are quite efficient. At both these sites water levels were found to be at about the bottom of the drain not long after the installation and to remain at that level.

For retaining walls, mass concrete blocks as counter-weights behind the wall are not indicated by the analysis to be particularly effective remedial measures. This

CASE	TYPE OF ANALYSIS AND WATER LEVEL USED	REMEDIAL MEASURES	c' AT FAILURE LB./SQ.FT.	F	
				WHEN INSTALLED	WHEN c'= 0
WEMBLEY HILL	PLANES MEAS'D W.L.	INCLINATION OF SLOPE REDUCED, WALL REBUILT DEEPER.	190	1.39	1.01
PARK VILLAGE EAST	PLANES INTERMEDIATE W.L.	CONTINUOUS COUNTERFORT EXTENDING 6' BELOW THE WALL INSTALLED	125	1.17	0.99
MILL LANE	PLANES SECTION 9, W.L. AS AT FAILURE	CLAY REMOVED FROM BEHIND WALL	203	1.08	0.76
UXBRIDGE	PLANES MEAS'D W.L.	TRACK RAISED STRUTS PLACED CLAY REMOVED	138	1.35	1.07
NORTHOLT	CIRCLES W.L. MEAS'D AFTER C'FORTS INSTALLED	COUNTERFORTS INSTALLED SLOPE CUT BACK	88	1.34	1.14

TABLE VIII

SUMMARY OF EFFECTIVENESS OF REMEDIAL MEASURES

has been demonstrated in practice by more than one case (e.g. Wood Green) where the addition of such blocks has made little or no difference. However, if the blocks project below the wall as at Park Village East they are quite effective.

Struts to the base of the wall from the opposite side of the cutting are generally an effective measure. Removal of some of the retained clay is of course also effective in stopping wall movement but often cannot be carried out due to limiations of space.

Unanchored sheet piling has not proved of great value in the two cases where it was used (Wood Green, Upper Holloway). Considerable movement is probably necessary to mobilise its full strength and even then it is questionable if the strength would be sufficient to stop movement.

The use of cement grout injected into the retained clay has as yet been unproven and may, in some cases, initially cause additional movement due to the volume change occasioned by its insertion.

SUGGESTED REMEDIAL MEASURES

The most obvious remedial measure in the case of retaining walls seems to be the insertion of struts to the opposite side of the cutting or to a suitable barrier wall

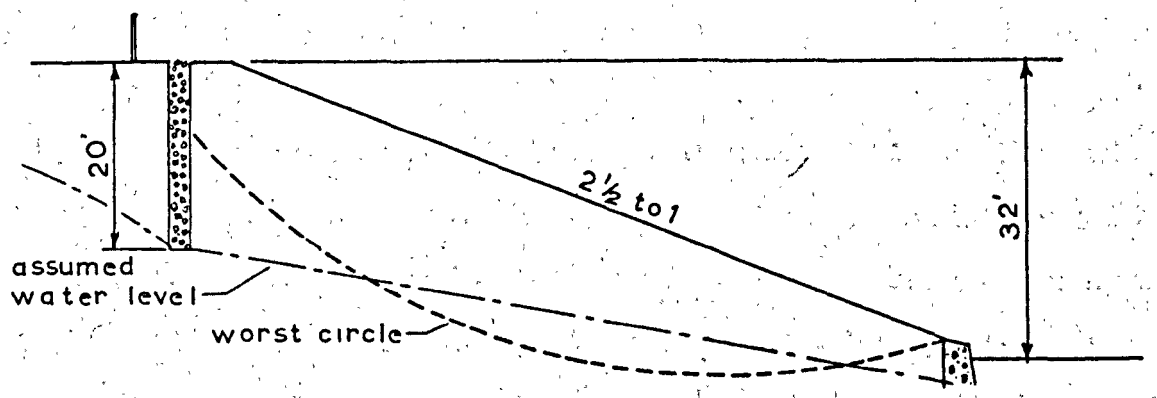
below ground level outside the passive failure area. This obvious solution does not seem to be utilized to its full advantage in many cases where it could easily be installed (e.g. Upper Holloway) and other less effective and more expensive remedial measures are substituted. In the case of high walls opposite each other, struts above train level combined with struts at track level are undoubtedly very effective and this procedure has been adopted for a number of failures in London clay.

The most direct approach in remedying a failure is to go to the source of the trouble and decrease the forces causing the failure rather than increase the resisting forces. Decreasing the active force may be accomplished by removing some of the retained clay or by lowering the water forces, thus decreasing the direct pressure on the wall and increasing the effective shear strength of the soil. As the first method of lowering the active force is obvious but cannot always be carried out due to limitations of space, the latter method of lowering the water forces is perhaps of more general use.

Counterforts for the repair of slips are commonly built up to 20 ft. deep and 6 to 8 ft. wide. Since their chief function is apparently drainage it is only logical that their width be kept to a minimum and a drain 2 ft. wide would probably be as useful. The installation of counterforts seems

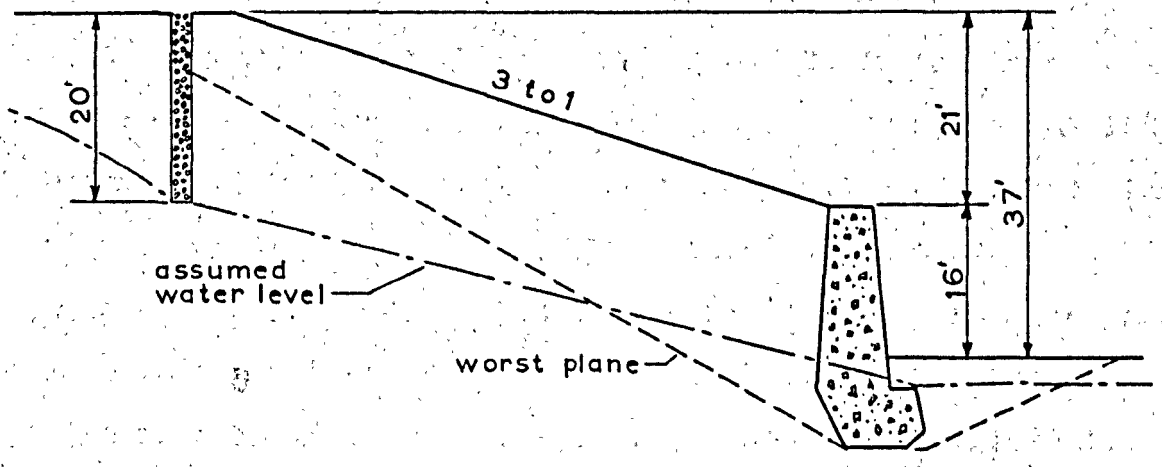
to increase the permeability of the clay, probably due to the relaxation of horizontal forces along the strike of the slope, opening up drainage paths along the fissures. It has been observed that there is some movement of the clay into the trenches during the construction of counterforts, and observations on old counterforts, built without filter zones, show that the clay moves into the counterforts up to 6 in. on each side over a period of several years.

Since the major function of counterforts seems to be drainage it is logical to consider them as drains and position them so that they intercept the flow of water towards a slope rather than attempt to remove it after it has arrived. The water table in a short steep slope, as is usual in a cutting, is apparently brought about by the inflow of water from the area beyond the top of the slope, and due to the steepness of the slope, run-off on it is proportionally high. Therefore, if a deep drain were installed at the top of the slope it would lower the water forces by conducting away ground water before it reached the slope. Such a system of drainage could be constructed as shown in Fig. 54. The major advantage of the system is that it would be much less expensive than the counterfort method now in use and achieve the same end. Considerable saving in excavation and backfill would be made and the top barrier drain could be excavated by machine rather than by hand as is necessary with counterforts.



NORTHOLT SLOPE

TYPE OF ANALYSIS	FACTOR OF SAFETY	
	WHEN C = 88 (i.e. at installation)	WHEN C = 0 (i.e. ultimate)
CIRCLES	1.34	1.12



WOOD GREEN RETAINING WALL

TYPE OF ANALYSIS	FACTOR OF SAFETY	
	WHEN C = 116 (i.e. at installation)	WHEN C = 0 (i.e. ultimate)
PLANES	1.22	0.96

EXAMPLES OF 20FT. DEEP BARRIER DRAINS AT NORTHOLT SLOPE AND WOOD GREEN RETAINING WALL

SCALE 1" = 20'

FIG. 54

With a barrier drain of this sort installed at the Northolt site the water table would be lowered to approximately that shown in Fig. 54 and the factor of safety increased accordingly.

The same method of drainage may be applied to retaining walls and is shown on a cross section of the Wood Green site in Fig. 54. The water table there would probably be lowered to that shown, which gives the calculated factor of safety of 1.22. This is an effect that the many remedial measures carried out at this site have been unable to produce.

Horizontal bore holes lined with drainage tile have been used with considerable success to drain slopes (Porter 1940) but as far as is known they have not been used in London clay on a large site. The expansion of London clay and presumably other heavily over-consolidated clays into the counterfort trenches materially assists the drainage, and since this could not occur to the same extent with bore holes there is reason to suspect they would not be as efficient. The final judgement on their efficiency, however, can only come after they have been tried, and as their installation would probably not be exceedingly expensive such a trial is probably justified at a suitable site.

SUGGESTIONS FOR DESIGN

The cost of remedial measures in the case of a failure such as that at Uxbridge is of the same order of

amount as the lifetime earnings of the man who designed the wall. It is therefore apparent that considerable care and thought is justified in the design of retaining walls.

The analyses make it apparent that earth retaining structures in London clay and probably most over-consolidated clays should be designed on an effective stress basis. The controlling factor in the design should be to insure that the structure has a factor of safety of at least 1.0 when $c' = 0$ and with the highest probable water level. If this procedure results in an excessively expensive structure small values of c' may be used as indicated by Fig. 52; however, the use of any amount of c' must be approached with extreme caution.

Factors of safety on an effective stress basis are generally not high at construction, for the walls considered, as evidenced by the summary in Table VII. In only two cases, and they are small walls, does F exceed 1.5 at construction and with most cases it is below 1.20. The design of these walls in regard to base pressure is generally not good. Analysis on the assumption of active pressure acting on the back of the wall at one third the height, a conservative estimate, shows that in most cases the reaction at the base is outside the middle third but inside the base.

In the design of any retaining wall it is probably good practice to take advantage of the opportunity during

construction of installing as much drainage as possible. Chases down the back and weep holes through the wall even when placed at 9 ft. spacing have proved themselves inadequate. The best solution seems to be a complete drainage blanket behind the wall. For clay the blanket should be a graded filter to insure that it will not become blocked. In addition, a barrier drain as shown in Fig. 54 could be installed.

With slopes the basic problem is also one of drainage. The 2½ to 1 slope of Northolt shows that a fairly low water level must be maintained when $c' = 0$ but not lower than that which can be obtained by the use of counterfort drains or a barrier drain. The case of Sudbury Hill indicates that stability would be reached on a 3 to 1 slope of that size when $c' = 0$ with a water level approximately 6 ft. below the surface on the average. The choice of the inclination for a slope is therefore one of balancing the cost of extra land and excavation against the cost of providing suitable drainage.

CONCLUSIONS, EXAMINATION OF FAILURES

The examination of failures in London clay has shown that the effective stress method of analysis is most suitable for long-term stability considerations in clay of this type.

Results indicate that the cohesion in terms of

effective stresses decreases with time, and a plot (Fig. 50) has been given to show the probable average rate of this decrease. The reduction in c' is also related to the effective normal pressure on the potential failure surface, decreasing more quickly with shallow surfaces. Data is insufficient to obtain full details of the decrease but a tentative plot is put forward (Fig. 52) relating c', effective normal pressure on the potential failure surface and age, for earth retaining structures in London clay.

Water forces are found to be among the main contributory forces causing failure. Therefore, the prevention of large water forces by drainage is important to promote stability and is the most effective remedial measure, short of removing the retained clay.

Counterforts in slopes apparently act more as drains than as buttresses but even so, are an effective remedial measure. However, it is probably more efficient to prevent water from reaching the slope rather than remove it once it is there, and for this purpose a deep barrier drain at the top of the slope is suggested. A barrier drain of the same type would also be suitable for an existing retaining wall.

Design of slopes and earth retaining structures for long-term stability should be based on the frictional

portion of the clay shear strength and c' assumed to be zero. It is essential in all designs that maximum possible water levels be considered and steps should be taken to insure that the maximum amount of drainage consistent with the economics of design be used.

APPENDIX A

CORRECTION FOR RUBBER MEMBRANE AND DRAIN

Undisturbed over-consolidated clays usually fail on one or more definite shear planes as shown in Plate VI and the bulging type of failure found with most remoulded clays is uncommon. Failure in drained tests on over-consolidated clay normally takes place at 2 to 5% strain, depending on the method of applying the stresses, whereas the bulging type of failure commonly occurs at 15 to 20% strain. Corrections for the rubber membrane and drain with the latter type of failure have been determined (Henkel and Gilbert, 1952).

It is quite evident in testing, that failures along planes cause considerably more deformation of the rubber membrane at the same strains than do bulging failures. Therefore, since the strain at failure is very different and the rubber is deformed in a different manner it was necessary to determine what corrections should be applied to failure on planes and a series of tests were carried out for this purpose.

Samples of undisturbed clay are variable to the point that the difference between strengths of supposedly similar samples can be more than the membrane and drain

corrections. Hence as any corrections could be lost in the variations and the planes of failure in undisturbed samples are unpredictable, determinations had to be made on an artificial sample rather than on the clay itself.

A sample shape of perspex, (see Fig. 55), 1½ in. diameter by 3 in. long was made for the purpose. This shape was cut across its centre at 45° so that when compressed it failed along the plane to simulate failure in the clay. A series of grooves were cut on the lower part of the plane and connected by a passage through the sample to the base of the cell so that pressures could not be built up about the plane. During a test, the two planes were lubricated to allow easy movement, heavy machine grease was found to be most satisfactory for this purpose.

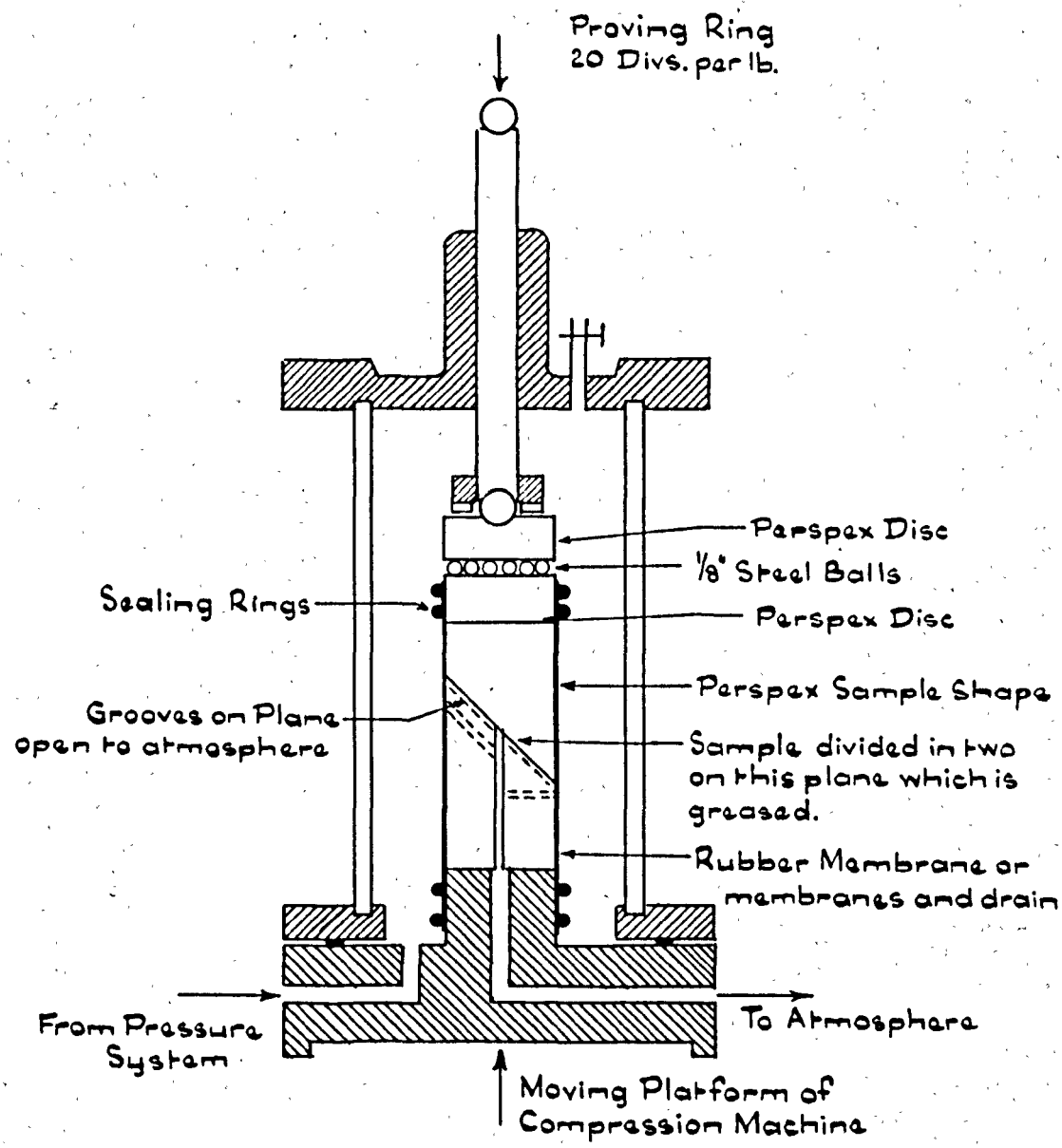
Test procedure was to set the sample up as shown in Fig. 55 with one rubber membrane and note the load required for deformation. After unloading, the sample was returned to its original position and a second rubber membrane applied, the inner membrane having been pierced in several places to prevent pressure between the membranes when the cell pressure was applied. The sample shape enclosed by the two membranes was then loaded and a failure simulated. The correction for a rubber membrane was taken as the difference between the first and second loadings. This procedure was carried out

cell pressures of 5, 10 and 20 lb./sq.in., several runs being made at each cell pressure and the results averaged. Corrections for one rubber membrane were obtained as shown in Fig. 56.

The procedure was repeated at 10 lb./sq.in. cell pressure with a filter paper drain being added before the second membrane. Results of this series of tests are also shown in Fig. 56.

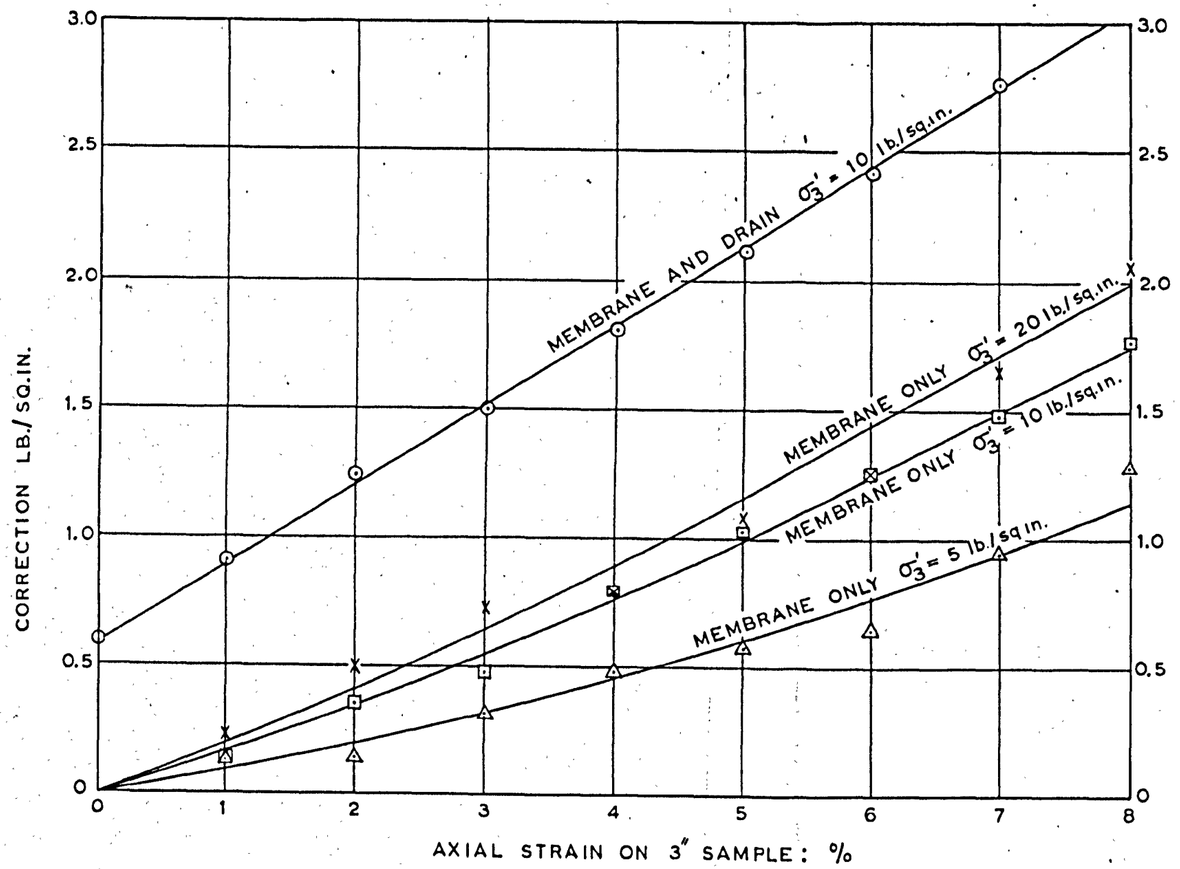
Distortion of the membrane is rather more severe with the perspex sample than with a clay sample since it allows no deformation at the edges, whereas in a clay sample there is deformation of the edges of the sample due to the strength of the rubber. The extra effect caused by the solid perspex shape cannot be determined but it is estimated that the true correction should be $\frac{2}{3}$ of that obtained. This difference would apply only to the rubber membrane portion of the correction, as the drain which appears to act somewhat like a strut would not be affected.

Curves shown in Fig. 56 have been drawn on this basis for the corrections to be applied when failure occurs on planes at cell pressures of 5, 10 and 20 lb./sq.in. These curves have been used to correct the test data for undisturbed over-consolidated clay presented in Part II.

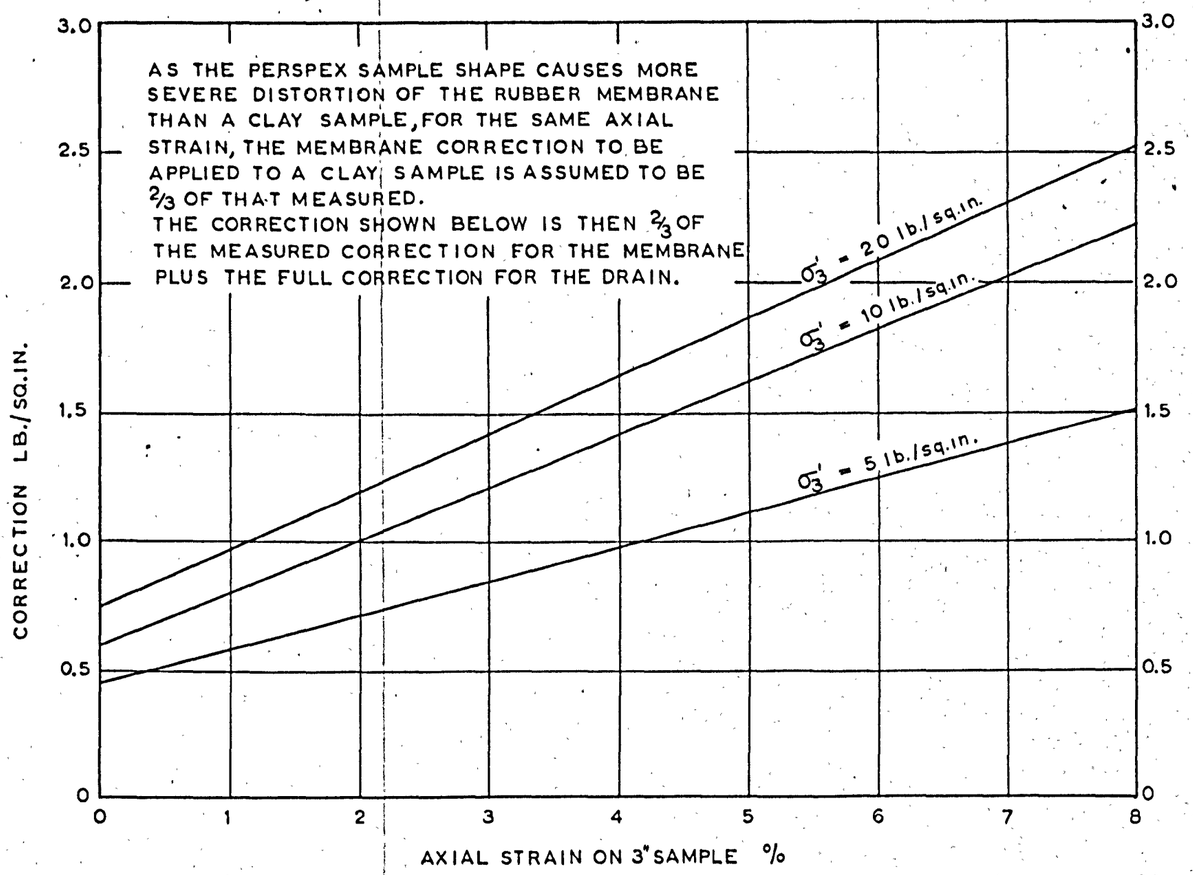


ARRANGEMENT FOR DETERMINING RUBBER AND DRAIN CORRECTION WITH FAILURE ON A PLANE

FIG. 55.



MEASURED STRENGTH OF RUBBER MEMBRANE AND DRAIN WITH 1½"ϕ BY 3" LONG PERSPEX SAMPLE SHAPE



CORRECTION TO DEVIATOR STRESS FOR RUBBER MEMBRANE AND DRAIN FOR 1½"ϕ BY 3" SAMPLE, WHEN FAILURE IS ON A PLANE

FIG. 56

APPENDIX B

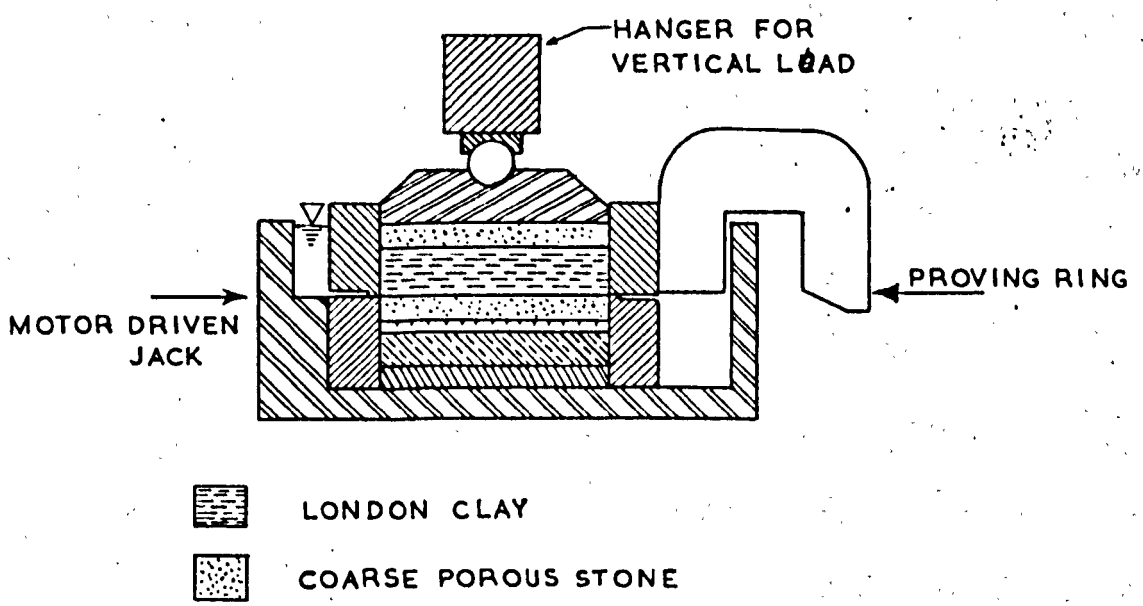
Ø' AND c' BETWEEN LONDON CLAY AND A ROUGH SURFACE

The angle of reaction between any material retained by a wall and the substance of the wall as well as the cohesion between the two materials has a considerable effect on the stability of that wall. Therefore, for the analysis of stability of walls in London clay it is necessary to know the parameters which apply to these inter-material surfaces. General data exists for the angle of reaction between various types of soils and various types of surfaces; however, as far as is known, no specific data exists for London clay and concrete or brick surfaces. Since c' is a product of over-consolidation and usually this has never occurred between the clay and wall surfaces its presence or absence at the interfaces must be proved. In order to ascertain Ø' and c' at the interfaces a series of tests was carried out.

The tests were done in a shear box arranged so that the shear plane through the centre of the box was also the plane dividing the sample from a rough surface. This was accomplished, see Fig. 57, by building up the stone in the bottom half of the shear box so that it was flush with the surface. The stones used were rougher than the normal

ones being bonded quartz sand of about 46 mesh size. Roughness of the surface of these stones was of the same order as unfinished concrete or unglazed brick.

The shear box and sample were set up as shown in Fig. 57 and allowed to come to equilibrium under the applied vertical loads. After equilibrium the sample was moved across the stone by operating the shear box in the usual manner, the speed used being the slowest available (.000077 in./min.) so that full drainage could take place. The samples failed under unusually small strains (0.025 in. average) with an average time to failure of 8½ hours.



ARRANGEMENT OF SHEAR BOX TO DETERMINE ϕ' & c' BETWEEN LONDON CLAY AND A ROUGH SURFACE

FIG.57

Four sets of shear tests were carried out with London clay. In three cases the sample was undisturbed clay placed in the shear box in the usual manner; in the other case the clay was remoulded into the shear box at its natural moisture content of 31.5%. Results of the tests are given in Table B1.

Sample Nos.	Type and Origin of Clay	Ø'	c' lb./sq.ft.
40,41,42	Brown L. clay, Uxbridge No. 24	21½°	75
43,44,45	Blue L. clay, Northolt	21½°	150
46,47,48	Brown L. clay, Sewardstone (remoulded)	19°	75
49,50,51	Brown L. clay, Sudbury Hill	20°	120

TABLE B1: Results of shear tests between London clay and stone.

From the results it is apparent that the Ø' mobilized between London clay and a rough surface is the full Ø' of the clay and that c' is approximately one half that found in the undisturbed clay. The existence of c' on such a failure plane probably results from some of the shear surface going through the body of the clay due to its small particle size rather than occurring between the clay and stone.

A P P E N D I X C

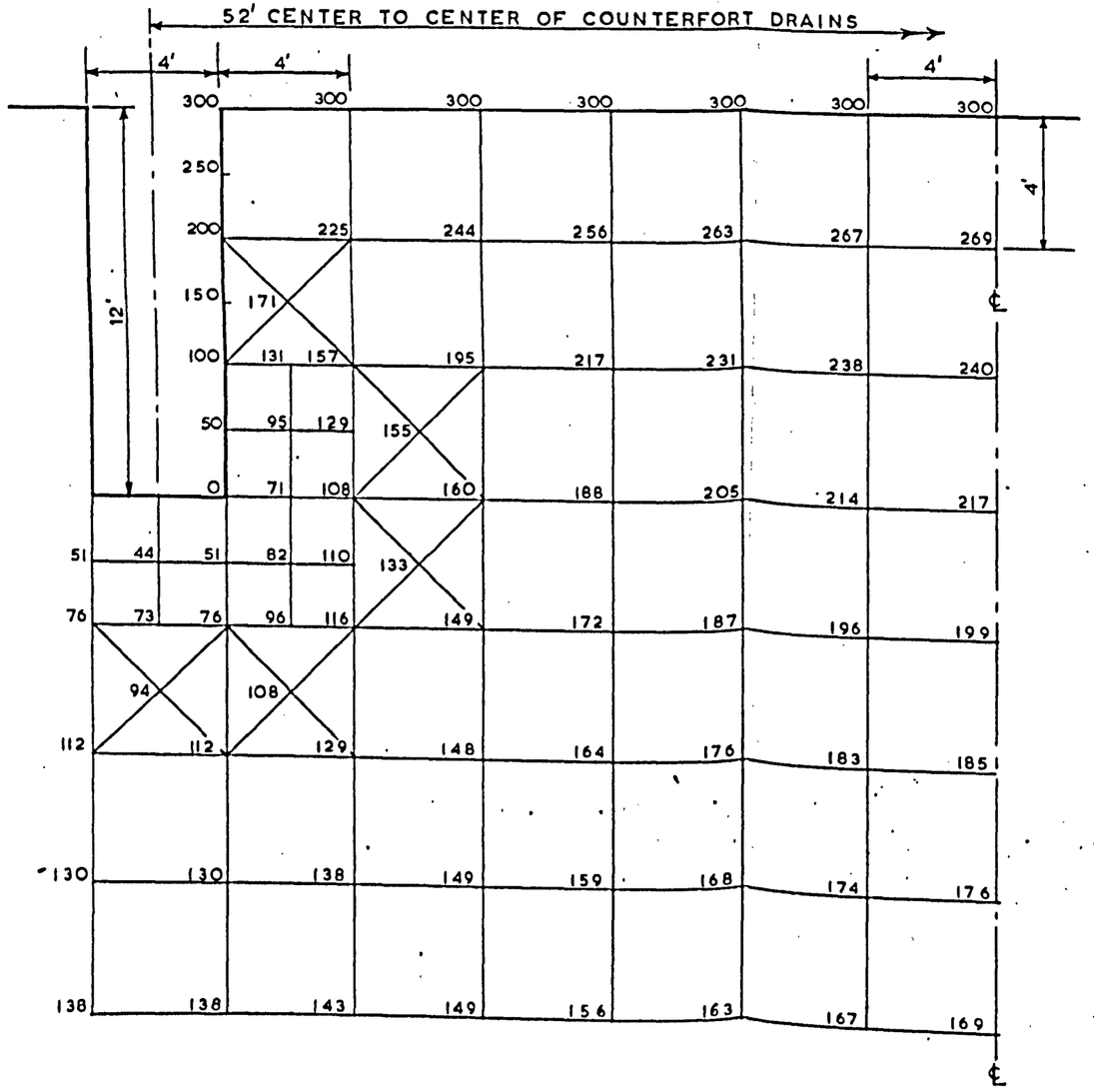
DRAINAGE IN SLOPES BY COUNTERFORTS

The usual method of stabilizing slopes is by the installation of counterfort drains. Counterforts in railway cuttings are commonly considered to provide stability through the greater shear strength of the material composing the counterfort and drainage due to the counterfort is considered relatively unimportant. The fact that there is little of no change in the moisture content of the clay near counterforts after their installation is considered evidence that the drainage effect is small. This observation is made without fully realizing that clays usually remain saturated above the water table and heavily over-consolidated clays are on the rebound portion of their consolidation curve and therefore large changes in effective stress are necessary to make appreciable changes in moisture content.

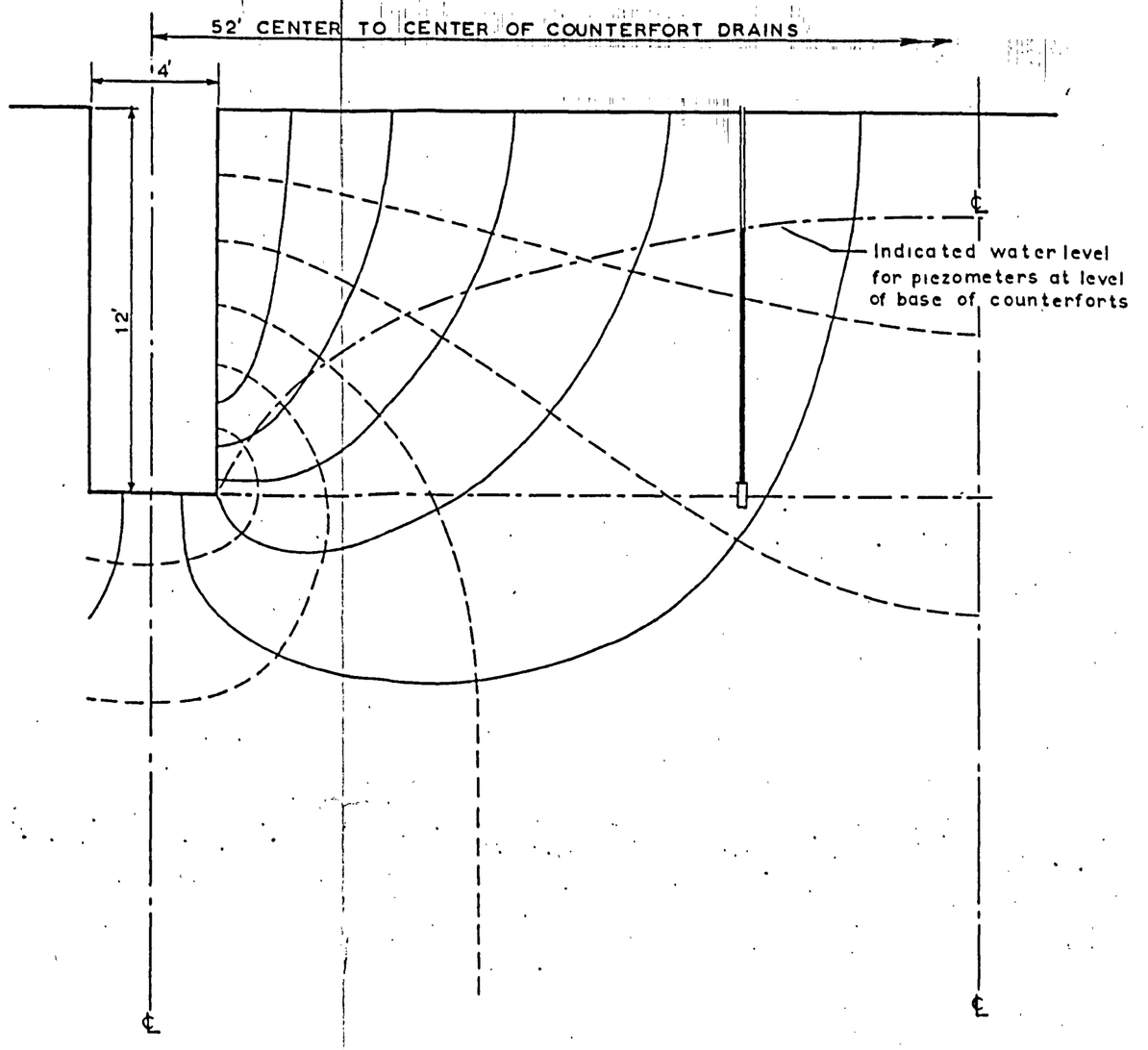
If the inflow of water in a slope is from surface moisture it is possible to obtain by relaxation methods the flow pattern for the area between drains. A relaxation solution was carried out for a section along the strike of a slope between counterforts on the assumption of two dimensional flow. Results of the relaxation are given in

Fig. 58. The position of the anticipated water table is shown in that figure for the level of the base of the counterfort drain.

In order to obtain field data on changes in the water table with counterforts such as these, a group of piezometers were installed at the Sydenham Hill site (Skempton and Henkel, 1955). Nine piezometers were placed so that in indication of the water table at different levels would be obtained. Observed water levels show that the water table was at about the base of the counterforts when the piezometers were installed about three months after the installation of the counterforts and readings over the following nine months show it has remained there. A similar occurrence was found at Northolt where piezometers installed soon after the counterforts showed the water level to be at the base of the counterforts. The lack of a water table above the base of the counterforts in both these cases leads to the conclusion that the major portion of the water in a London clay slope is caused by the inflow of water from further up the slope, or the area above the slope, and not by seepage downward from the surface. The observed water levels also show that counterforts are highly efficient as drains, and as they lower the water table considerably so that $\emptyset'$ is seldom fully mobilized, their effectiveness must be largely as drains.



RESULTS OF RELAXATION SOLUTION FOR POTENTIALS ABOUT COUNTERFORT DRAINS ASSUMING A CONTINUOUS SUPPLY OF WATER AT THE GROUND SURFACE



EQUIPOTENTIAL AND FLOW LINES AS DETERMINED BY THE RELAXATION SOLUTION SHOWN ON THE LEFT

FIG. 58

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