

RESIDUAL SHEAR STRENGTH UNDER LARGE STRAINS
AND THE EFFECT OF SAMPLE SIZE ON THE CONSOLIDATION
OF FISSURED CLAY.

A thesis submitted to the
University of London
(Imperial College of Science and Technology)

for the degree of
Doctor of Philosophy in the Faculty of Engineering

by

VINOD KUMAR GARGA, B.Sc.[ENG.](E.A.), M.Sc., D.I.C.

March, 1970

ABSTRACT

This thesis is composed of two separate parts. Part A deals with the investigation of the residual shear strength of stiff clays under very large displacement in a new ring shear apparatus. Residual shear strength from these tests was found to be lower than that obtained from conventional residual shear tests (e.g. the reversal direct shear box tests). It was found that some clays exhibit marked variation of residual shear strength, with the magnitude of effective normal stress, and that the residual strength is independent of stress history. Some optical studies on thin sections of shear zones produced in the ring shear apparatus have also been presented. A drained tension test apparatus was developed and it has been shown that the intact London clay has a tensile strength which is in good agreement with the modified Griffith failure criteria.

The effect of sample size on the consolidation characteristics of fissured blue London clay has been discussed in part B of the thesis. The results of conventional oedometer tests, dissipation tests on $1\frac{1}{2}$ in., 4 in. and 12 in. diameter undisturbed samples and in-situ constant head permeability tests have been compared. It has been shown that the field permeability (and the field rate of consolidation) at shallow depths, where the degree of fissuring is high, can be seriously underestimated by one dimensional consolidation tests in the laboratory.

CONTENTS

	Page
ABSTRACT	1
<u>Part A</u>	
Chapt. 1. Introduction to residual shear strength; some factors affecting it; and previous attempts to measure it.	
1.1 Introduction	7
1.2 Historical development	7
1.3 Some factors affecting residual shear strength	10
1.4 Tests to determine residual shear strength	14
Chapt. 2. The new ring shear apparatus	
2.1 Principles of the apparatus	25
2.2 Detailed description	26
2.3 Calibration of the apparatus	31
Chapt. 3. Sample preparation and test procedures	
3.1.1 Remoulded samples	36
3.1.2 Undisturbed samples	37
3.2 Assembly of the apparatus	39
3.3 Dismantling of the apparatus	41
3.4 Testing procedure	42
3.5 Calculation of results	44
3.6 Graphical presentation of results	45

3.7	Accuracy of results	46
Chapt. 4. Ring shear tests on stiff clays		
4.1	Tests on stiff clays from U.K.	50
4.1.1	Brief review of previous work	50
4.1.2	Geology at Wraybury	55
4.1.3	Tests on blue London clay	56
4.1.4	Tests on Weald clay	66
4.1.5	Tests on brown London clay	70
4.1.6	The significance of the variation of ϕ'_r with normal stress	74
4.1.7	Soil structure in shear zone	85
4.2	Tests on highly plastic clay-shales from North America	89
4.2.1	Introduction and review	89
4.2.2	Clays from Panama Canal	89
4.2.3	Bearpaw shale	93
4.2.4	Pierre shale	97
4.2.5	Pepper shale	97
4.2.6	Test results	100
4.2.6.1	Tests on remoulded Cucaracha clay shale	100
4.2.6.2	Tests on Dawson clay-shale	101
4.3	Conclusions	104
Chapt. 5. Microstructure of sheared clays		
5.1	Introduction	108
5.2.1	Preparation of thin sections for optical study	112
5.2.2	Preparation of samples for scanning electron microscope	116

	Page
5.3 Observations	
5.3.1 Blue London clay	117
5.3.2 Natural shear zone in Weald clay	120
5.3.3 Weald clay sheared in the ring shear apparatus	122
5.4 Summary of observations	124
 Chapt. 6. Tensile strength of clays	
6.1 Introduction	127
6.1.1 Basic concepts of Griffith's theory	128
6.2 Strength of rocks in tension	129
6.3 Strength of soils in tension	130
6.4 Drained tension tests	
6.4.1 Theory	133
6.4.2 Description of the apparatus	135
6.5 Results and discussion	137
6.6 Conclusions	140
 Chapt. 7. Concluding remarks	 141

Part B

Chapt. 8. Introduction and previous studies on consolidation of London clay	
8.1 Introduction	151
8.2 Brief outline of work on non-linear consolidation	152
8.3 Previous study on London clay	153

Chapt. 9. Constant head tests in the field

9.1	General considerations	157
9.1.1	Intake factors for cylindrical tips	158
9.1.2	Effect on anisotropy and three dimensional flow	162
9.1.3	Sealing of piezometers	163
9.2	In-situ C.H. tests in London clay	
9.2.1	Installation of piezometers	164
9.2.2	Constant head test apparatus	166
9.2.3	Test procedure	167
9.2.4	Results and discussion	169

Chapt. 10. Laboratory tests on London clay

10.1	Details of experimental work	175
10.1.1	1½ in. and 4 in. dia. tests	175
10.1.2	Test procedure	176
10.1.3	Description of 12 in. dia. dissipation cell	177
10.1.4	Oedometer tests	181
10.2	Results and discussion	
10.2.1	Dissipation tests	182
10.2.2	Oedometer tests	184
10.2.3	Coefficient of consolidation	185
10.2.4	Water contents	191
10.2.5	Coefficient of compressibility	195
10.2.6	Coefficient of permeability	198

	Page
10.2.7 Rate of consolidation	200
10.3 Conclusions	204
10.4 Some suggestions for future research	205
Chapt. 11 Tests on boulder clay	
11.1 Introduction	207
11.2 Layout of apparatus	209
11.3 Test procedure	209
11.4 Results and discussion	211
Appendix A	215
Appendix B	216
Acknowledgements	220
References	222

PART A

CHAPTER 1

Introduction to residual shear strength; some factors affecting it; and previous attempts to measure it.

1.1 Introduction

When a soil is subjected to a shearing force, the shear strength of the material progressively increases until the "maximum" or "peak" strength is reached. If the process of shearing is continued after the peak strength has been surpassed then a process comparable to strain softening takes place. During this stage considerable changes in soil structure occurs and the shear strength drops, until after large displacements a state is achieved when further straining would result in no decrease in shear strength. This strength of the soil is referred to as the "ultimate" or "residual" strength of the soil. The drop in strength from peak to residual is often more marked in clays than in granular soils, and therefore has important implications on long term stability of clay slopes.

1.2 Historical development.

The concept of residual shear strength of clays to explain long term stability of slopes is not a very recent one. Tiedemann (1937), Hvorslev (1937) and Haefeli (1938) attempted to measure residual shear strength of clays in the different torsion shear apparatus that they designed. Terzaghi (1937), described the mechanism of progressive failure in stiff fissured clays. He ascribed the softening

of clays with time to the presence of small fissures and cracks, which allow the water to seep through to the more intact zone causing local swelling. This in turn leads to the development of further cracks and fissures.

In an outstanding paper in 1950, Haefeli* discussed the concept of residual shear strength of clays in extremely precise terms. He defined 'residual shearing stress' as "the specific friction, which remains in the sliding surface after complete retarding of the shearing operation." He further wrote: "The practical importance of the residual shearing stress lies in its adoption when criticising questions of stability. The relation between the residual shearing stress and the shearing strength is then of special interest:

$$\tan \phi_R = \frac{S_R}{\sigma_n}; \quad \lambda_R = \frac{S_R}{S} = \frac{\tan \phi_R}{\tan \phi_S}, \quad (1.1)$$

(where λ_R denotes residual coefficient,

and S and S_R denote shear strength at peak and residual respectively.)

"Observations on landslides and avalanches have led to the conclusion that rupture never occurs simultaneously at all points of a surface of rupture, but takes place successively (progressive rupture). Once rupture has started at the weakest spot, it spreads along the most unfavourable shearing surface at a certain speed which is dependant on the mechanical properties of the material in question."

* Haefeli had in fact introduced the same concepts as early as 1938, (see Haefeli, 1938).

"In soils where slides have occurred before, so called slicksides are often found, which shows clearly that sliding surfaces often result in a residual weakness. Therefore, also when judging relieved soils, the greatest care should be taken and any cleavage should be considered. If there is much cleavage it is, strictly speaking, only possible to calculate with the residual shearing stress. The difference between residual shearing stress and shearing strength will be greater, the more the material has been preloaded."

It is clear from the above extract that Maefeli presented some important concepts about residual shear strength very clearly. His use of the 'residual coefficient', λ_R , gives a quantitative characteristic of the clay with respect to its drop in strength with time. He also clearly advocated that in soils where large movements have taken place in the past, only the residual strength should be deemed to operate. It is therefore very surprising that even after the above publication in 1950, numerous papers were published (e.g. Peterson, 1954; Van Auker, 1963; Peterson et. al., 1960) which continued to express concern at the observation that a number of slides could not be explained in terms of effective peak shear strength parameters!

Borowicka (1963) discussed the decrease of shear strength with displacement and described such clays as "slippage-prone clays". He suggested that the lowest value should be used for ultimate stability analysis, since the danger of progressive rupture was always present in

such clays, and recommended the use of direct shear reversal tests.

Skempton (1964), in his Rankine Lecture demonstrated the concept of residual strength in relation to long term stability of clay slopes. He showed that the average shear strength along a slip surface in overconsolidated plastic clays was nearer to the residual strength rather than the peak. He further postulated that the residual shear strength (as expressed by the parameter ϕ'_r) was independent of water content, liquidity index and stress history of the soil.

A number of papers have since been published which appear to confirm the validity of the residual shear strength concept.

1.3 Some factors controlling residual shear strength

For the purpose of this discussion it will be assumed that for clays, the peak strength can be denoted on the Mohr-Coulomb failure criteria by the parameters ϕ'_p and C'_p , while the residual shear strength is denoted by ϕ'_r and $C'_r = 0$. That the effective cohesion intercept reduces to zero in the residual state is not an unreasonable assumption for it can be seen from numerous tests that beyond peak strength the decrease in C' is more rapid than that in ϕ' (Henkel and Skempton, 1957; Skempton and Petley, 1967). It appears that the drop from peak to residual is more marked if the clay is overconsolidated. Most overconsolidated clays and clay shales have fissures, cracks and

joints, which are probably formed as a release of horizontal stresses. Bjerrum (1967), stipulated that clays with strong "diagenetic bonds" are most likely to achieve low residual strengths compared to the peak. When the diagenetic bonds are broken and weakened, by the release of locked-in horizontal stresses, (present in most overconsolidated clays) either by weathering processes or externally applied stresses, then the clay would ultimately achieve ϕ'_r which would be substantially different from ϕ'_p . This certainly appears to be true in a number of cases discussed by Skempton (1964), Petley (1966), Bjerrum (1967), and others. It is worth noting here that the "peak" strength of a mass of overconsolidated clay may itself be much lower than the peak strength of intact clay present in the mass (Bishop, 1967).

The value of ϕ'_r depends on the amount of movement that has taken place on the slip surface. The amount of displacement required to reduce shear strength to the residual is in fact a function of the structure and fabric of clay at the slip surface. In an unsheared mass of stiff clay the clay particles - thin platelets in shape surrounded by adsorbed water layers, (Mitchell, 1956; Rosequist, 1959;) - may perhaps be only moderately orientated in the direction of displacement. However, under a shearing stress, the clay particles adopt a parallel face to face structure orientated in the direction of the applied shearing force so that less resistance is offered to sliding. The degree of orientation (or the degree of parallelism) would obviously depend on the amount of displacement, the nature of pore

fluid and adsorbed water layer and the clay mineral type. In the present project, some clays have been subjected to large displacement in a new ring shear apparatus to investigate whether the values of ϕ'_r as obtained by limited and interrupted travel in other shear tests differ from those obtained in the ring shear apparatus. Some studies on the microstructure of clays both before and after shearing in the ring shear apparatus have also been made and are described in chapter 5.

Very little work appears to have been done to study the displacements required to reduce the shear strength values to residual for different type of clays. A very useful study on rocks has been made by Krsmanovic (1967) who tested large size samples of rocks of varying hardness in a specially designed direct shear machine. The samples were 40 sq. cm in plan and 20 cm high. The results indicate that the residual strength for rocks of smaller hardness would be obtained much earlier than those with greater hardness. Skempton (1964) indicated that ϕ'_r was a function of particle size and that ϕ'_r decreased as clay fraction (less than 0.002 mm) increased. It was also suggested that in a limiting case, if the soil consisted entirely of clay particles, then ϕ'_r would be the same as the value of interparticle friction, ϕ'_{μ} , as measured by Horn and Deere (1962), for various layer lattice minerals. This may perhaps be not strictly correct as recent elegant tests by Skinner (1969) have shown that on wetting sliding friction can change into much higher rolling friction, in which case ϕ'_r would not be directly related to ϕ'_{μ} directly. However Skinner's results may not be applicable to platy clay minerals.

Kenney (1967) conducted residual tests on wide range of soils, from massive minerals to pure clays minerals, and found that ϕ'_r infact did not correlate uniquely with Atterberg limits or even their clay fractions ($<2\mu$) alone. It was suggested that ϕ'_r was a function of clay mineral type and clay mineral fraction and not clay fraction based on particle size. This distinction is very important and only clay fraction based on clay mineral analysis would explain why some soils with low 'clay fraction' based on particle size show low residual values. For example the Pierre shale and the Pepper shale (van Auken 1963) have a clay particle size fraction of about 57% which should correspond to an average of about 15° from Skempton's curve (1964); infact the residual value is about 6° . In general, high ϕ'_r were obtained for soils containing large amounts of massive minerals (e.g. quartz, feldspar or chlorite) and those which contained clay minerals of the mica group, while low ϕ'_r were found for soils with high montmorillonite content. The two layer lattice minerals from the mica group - muscovite and hydrous mica showed ϕ'_r ranging from 17° to 26° , and for these materials, ϕ'_r seems to increase because of the following:

- a) increase in the valency cations; hence calcium samples show higher ϕ'_r than sodium or potassium samples:
- b) increase in salt concentration in the pore fluid;
- c) decrease of grain size.

The factors mentioned above and others like the influence of normal stress and rate of strain will be dis-

cussed more fully in subsequent chapters.

1.4 Tests to determine residual shear strength

A number of tests have been utilized in the field of Soil Mechanics for the determination of shear strength parameters, e.g., direct shear test, triaxial test, torsion test etc., and the suitability of type of test obviously depends on the objectives of the test. It has been shown previously that most clay soils and especially stiff soils (Terzaghi 1936) are subject to decrease in shear strength after failure (as defined by the peak strength) and often a large amount of displacement is required to achieve the ultimate or residual shear strength. It is also useful to know the volume change characteristics of the soil when subjected to a shearing force. To meet the above objectives, it should be possible to control stresses and strains at the failure surface of the sample and, ideally, these should be uniformly distributed throughout the specimen. Of course, other factors such as simplicity of the apparatus and speed and accuracy of the apparatus should also be taken into account.

1.4.1 Triaxial shear test

Although the triaxial test (Bishop and Henkel, 1962) is perhaps the best test to measure the peak shear strength parameters, it is of little use to study the post-failure shear strength behaviour. This is mainly because of limited displacement on the shear plane and also that large and irregular changes in cross-section take place just before and after failure thus causing a non-uniform distribution

of shear stress on the failure zone. More recently (Bishop et. al., 1965; Chandler, 1966; and La Rochelle, 1967) techniques to measure shear strength along a pre-formed, polished, inclined cut plane have been developed. However, these are often still subject to the limitation of inadequate displacement.

1.4.2 Direct shear test

The basic direct shear test apparatus, developed in its present form, successively by Krey, Terzaghi, Casagrande and Bishop, is perhaps the most common method of determining residual shear strength of clays under drained conditions. The sample, usually about 6 cm square and about 2.5 cm thick is held in upper and lower frame, located between porous stones at top and bottom. Under the action of a shearing force, the soil fails along the plane of separation between the upper and lower frame. The advantages and limitations of this test have been discussed by Hvorslev, (1939); Burmister, (1953); Sowers, (1963) and Petley (1966). Briefly, the advantages are the simplicity of the apparatus and testing operation and good drainage facilities which enable a drained test to be carried out in reasonably short time. A number of tests have been carried out on large, undisturbed in-situ samples. Thorfinson, (1954) carried out in-situ direct shear tests on bentonite seam on a sample as large as 2.5 ft x 2.5 ft x 18.8 ft long. Schultze, (1957) measured shear strength of soft rocks with a plane of shear of 1 sq. metre, and more recently tests on slip surface in London clay on 2 ft square shear boxes have been carried out by Bishop and Little, (1967). Borowicka, (1963);

Skempton, (1964); Petley (1966), and others have suggested the use of multiple reversal technique to achieve large displacement in which the upper half of shear box is brought back to the original position after the end of its travel, and the sample is sheared again. It will be shown subsequently in more detail in another chapter that the method seldom furnishes sufficient displacement to reach residual strength and very often disturbs the orientation of particles on the slip plane to such an extent that the shear strength on a subsequent travel is higher than in the previous one. Also non-uniform shear stresses are developed due to the rotation of principal stresses (Hansen 1961) and the effective area of cross-section of the shear plane decreases as shearing proceeds. A correction may be applied to account this decrease in area (see Petley 1966), but according to Hvorslev and Kaufman (1952) "such corrections become increasingly uncertain with increasing deformations, and the test permits only a limited investigation of the shearing resistance characteristics after initial failure". A fuller discussion on the limitation of direct shear test will be made in chapter 8.4.

In order to avoid the non-homogeneity of shear strain that develop in a direct shear box, Roscoe (1953) has designed a shear box with rigid but hinged sides so that the sides tilt as shearing proceeds, thus maintaining a constant length of sample in the direction parallel to the direction of shear. Although uniform shear strains are thus imposed, even more complicated non-uniform shear stresses are induced at the sides of the box in contact

with the soil which would tend to elongate as shearing proceeds.

A shear box developed by the Royal Swedish Geotechnical Institute and its advantages over conventional direct shear box tests have been described by Kjellman, (1951). The short cylindrical specimen is laterally confined by a rubber membrane reinforced by aluminium rings. Briefly, the advantages of this arrangement are that the area of potential surface of sliding remains constant; the strains are uniformly distributed over the sample and side wall friction effects are eliminated. However, the lateral deformation of the sample cannot be completely avoided and the possibility of the compression of rubber membrane between the rings always exists. This would, in turn, induce non-homogeneous lateral strains on the sample. The stresses on top and bottom of circular sample are also non-uniform and perhaps more complicated than for direct shear box. The above limitations and the inability of the apparatus to produce large displacements renders it of little use for determining residual shear strength of stiff clays.

1.4.3 Torsion shear tests

In general, the two obvious advantages of any type of torsion shear test are that there is no change in area of cross-section of shear plane and that the sample can be sheared through any uninterrupted displacement that may be desired. The latter, in particular, renders it extremely suitable for determining residual shear strength of stiff clays.

The earliest recorded torsion test apparatus was developed by the 'A.S.C.E. Special Committee to codify present practice on the bearing value of soils for foundations' in 1917. This consisted, very simply, of a laterally confined cylinder of soil which was loaded directly through a top platen. The sample rested on a base plate which carried a rotating base plate at its centre. The disc, whose diameter was half the diameter of the sample, had radial fins to engage the soil. A similar apparatus, but with connected loading platen and disc, was developed at the Institute of Technology in Hanover, and reported by Franzius, Streck and Hinderks, (1929). Both the above simple apparatus have the serious limitation that the normal load on the actual failure plane is unknown because of arching which develops as only part of the sample rotates. The failure plane would not necessarily be in a well defined horizontal plane and is more likely to be located at a short distance above the ribs of the rotating disc.

A refined torsion apparatus was designed by Langer (1938) in which a cylindrical sample, 10 cm in diameter and 6 cm high, was confined between two porous stones and upper and lower confining rings. An important and advanced feature was that after consolidation, and just prior to shearing, the upper confining ring could be lifted to maintain a gap between the two rings so that all normal load on the sample was conveyed to the failure zone through the soil-soil interface. This facility also ensures that failure takes place along the plane of separation of the upper and lower confining rings. The lower ring and the base plate were held stationary while the upper ring rotated; the

torque was transmitted to the sample by means of dentated porous stones in the walls of the upper ring. The torsion load was applied by a wheel to which the upper ring was fastened. Langer obtained some extremely interesting results on a number of clays, including the London clay, for which a value of ϕ_r of 8.75° was obtained. These will be discussed in chapter 4. Another example of a disc shaped sample (2.5 in. diameter and $3/8$ in. thick) laterally confined in split confining rings has been mentioned by Sembenelli and Ramirez (1969).

It should be noted that in case of torsion shear tests on solid cylindrical samples, the shearing strains are non-uniform, varying from zero at the centre to maximum at the edges. The shear stress can become non-uniform under certain conditions. These effects can be avoided to a marked extent by using ring shaped samples with suitable dimensions so that a more uniform distribution of angular displacement and hence more uniform shear stress and strain develop along the whole width of the sample. Generally, the ratio of internal to external diameter varies between 0.5 and 0.8 (Hvorslev 1939).

Hvorslev (1939) designed a refined ring shear apparatus at Vienna and later at the Waterways Experiment Station at Vicksburg in 1947. A section of his apparatus is shown in fig. 1-1. The sample, 11.95 cm external diameter, 5.95 cm internal diameter and 2 cm high is enclosed in top and bottom confining rings and ring porous stones. The stones have machined teeth to engage the soil. The upper rings can be raised by means of elevator screws to

maintain a gap. The vertical load is transmitted from a lever system to the torque wheel and annular top plate by means of a loading rod and self aligning thrust bearing. The two horizontal dial gauges measure the rotation of the torque wheel, which can be held stationary or rotated by means of two very flexible wire ropes which are connected to the wheel by adjustable clamps. The rotation of the base and volume changes of the sample can be measured by the total twist dial and the vertical dial respectively.

Hvorslev and Kaufman (1952) have described all the calibrations of the various components of the apparatus in detail. The effect of friction in rotating and stationary parts seems to have been taken into account in these calibrations. However, the influence of friction between soil and confining rings cannot be measured directly in this apparatus.

Haefeli (1938) (reported by Hvorslev, 1939) designed a ring shear apparatus in Zurich which could shear a sample of external diameter of 24 cm. and internal diameter of 16 cm. The sample is enclosed in upper and inner confining rings, but unlike other apparatus, the plane of separation between the two rings is at the base of the sample. This is an obvious drawback because the sample is more likely to fail along its base. Also, no facility seems to have been provided to maintain a clearance between the upper and lower confining rings.

Tiedemann (1937) improved on his earlier version of his 1933 apparatus, in which a ring sample (external dia. 14 cm; internal dia. 8 cm and 2 cm high) is sheared by

keeping the upper, outer and inner confining rings and the upper annular loading piston stationary while the base is rotated. A facility for maintaining a gap has been provided in this apparatus.

A summary of different tests on clays performed by Hvorslev, Tiedemann and Langer has been presented in Table 1.1. The tests on Vienna clay (also known as the Wiener Tegel Clay) showed that the residual strength for overconsolidated clay was slightly lower than that for normally remoulded consolidated clay (Hvorslev 1960). The Vienna clay also displayed remarkable gain in strength after the test had been stopped on reaching residual state. After a rest period of 10 minutes, the shear strength had increased from residual strength to 99% of peak strength. Tests on Little Belt Clay (which contained 77% clay fraction, as compared to 23% for Wiener Tegel Clay) showed very low residual shear strength (see Table 1-1). The residual values for overconsolidated and remoulded samples were 30% and 40% of the peak shear strength values and were achieved at a displacement of about 3 and 8 cm respectively. Tiedemann (1937) reported a number of residual tests on wide range of soils in remoulded and undisturbed state which have been included in Table 1-1. The interesting point to note are the results of highly plastic clay from Weser Elbe Canal. Unlike tests on Wiener Tegel Clay by Hvorslev, Tiedemann found little difference in residual shear strength between undisturbed and remoulded soils at high normal stress (about 4 kg/cm^2). However, the undisturbed clay displayed marked curvature in the shear strength envelope and the residual strength was clearly a function of the normal

stress while the remoulded sample appeared to be unaffected. The value of ϕ_r^1 for the undisturbed sample ranged from 8.3° at normal stress of 4.09 kg/cm^2 to 12.3° under normal stress of 1.0 kg/cm^2 . It would therefore appear that in this case, remoulding had altered the structure of the clay.

It should be noticed again, that in none of the apparatus described above, the effect of friction between soil and the upper confining rings can be evaluated directly.

Cooling and Smith (1937) sought to eliminate this problem by shearing an unconfined ring shaped sample (external dia. 4 in., internal dia. 3.25 in., and height of 1 - 1.5 in.). These tests were, in effect, undrained tests under zero normal stress. The torque was increased at a uniform rate until the peak value was obtained. The results for a number of English clays seem to compare well with strengths obtained from unconfined, undrained compression tests.

Ghani (1968) describes a torsion shear apparatus in which a disc shaped sample is used. The stresses on the upper surface of the sample are measured on a dentated ring plate called "measuring annulus" over a very small width of the sample. This, in effect, corresponds to using a ring shaped sample. The vertical normal load on the annulus is measured by strain gauges mounted on the double ring plates and the steel sheet which deflects under a normal load. The torque on the sample is measured by inductive strain gauge mounted on the holder. The normal stress can be measured

accurately on top of the sample; it may not, however, be the same stress which is actually present on the failure surface due to friction between soil and the outer confining ring.

Some attempts have been made in chemical engineering industry to design rotary shear cells for measuring shear strength of 'powders' and only two apparatus will be mentioned below for completeness of this review. Carr and Walker (1967) described an annular shear cell (external dia. of 10 in. and internal dia. of 6 in.) for investigating shear strength under normal pressure of 1 psi. Since the sample is confined in an annular trough, the slip surface is not necessarily in a horizontal plane at the mid-height of the sample. An annular shear cell in which the specimen is further divided into three zones by means of three concentric, brass annulii which can be constrained individually has been described by Scarlett and Todd (1968). The transmission of shear from one annulus to the next is prevented by a race of $1/8$ in. ball bearings. Normal stress is applied to each of the top caps of the annulii separately by means of weights suspended on three concentric tubes such that the stress on each annulus is proportional to its area - thus ensuring a uniform stress distribution on top of the sample. The shear strength is measured by recording the force required to constraint the middle annulus only.

1.4.4 Field torsion shear tests

In theory, there is no limitation to the use of torsional shear box in field. The author is not aware of any field torsion test apparatus with which insitu drained tests may be conducted and in which "side-wall" friction effects can be accurately assessed. Further, the field equipment uses relatively crude stress and torque measuring systems (see for example, Payne and Fountaine, 1952; Marsal et. al., 1965; and Helenlund 1966).

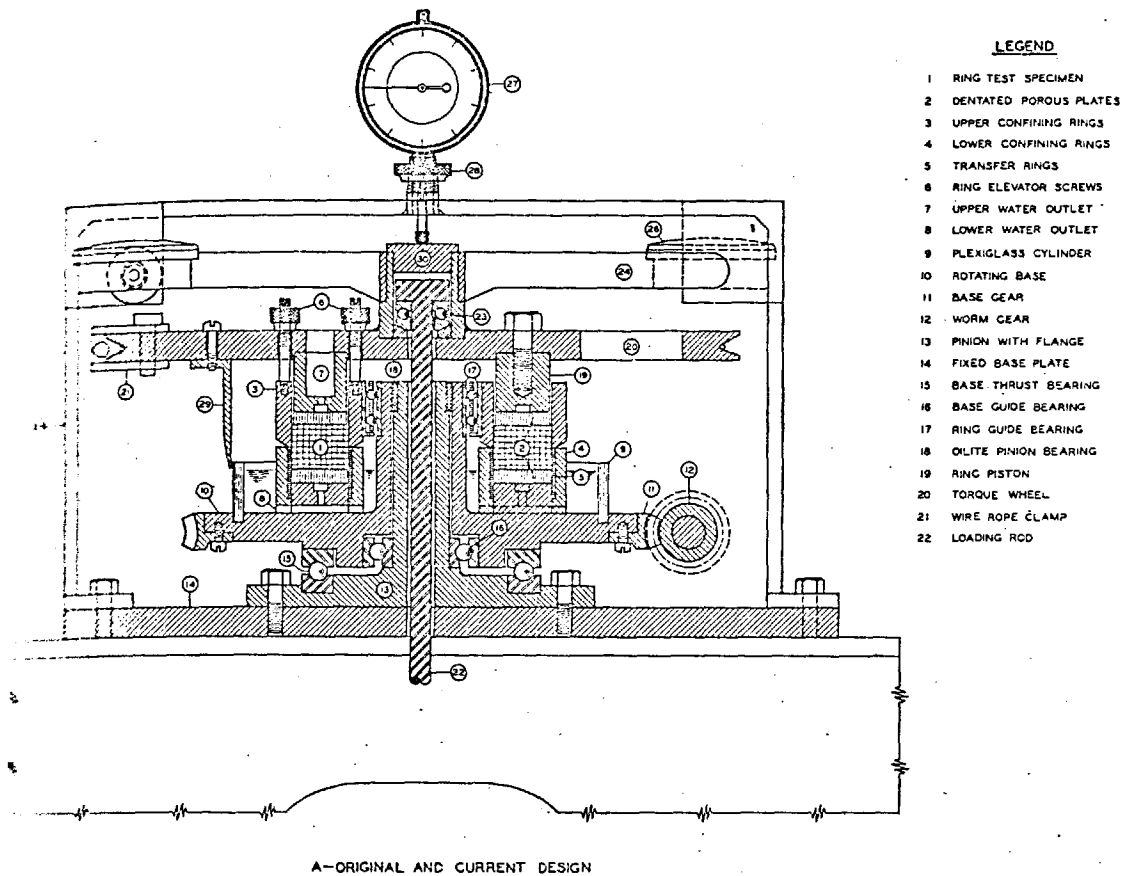
Table 1.1

Some results from early ring shear tests.

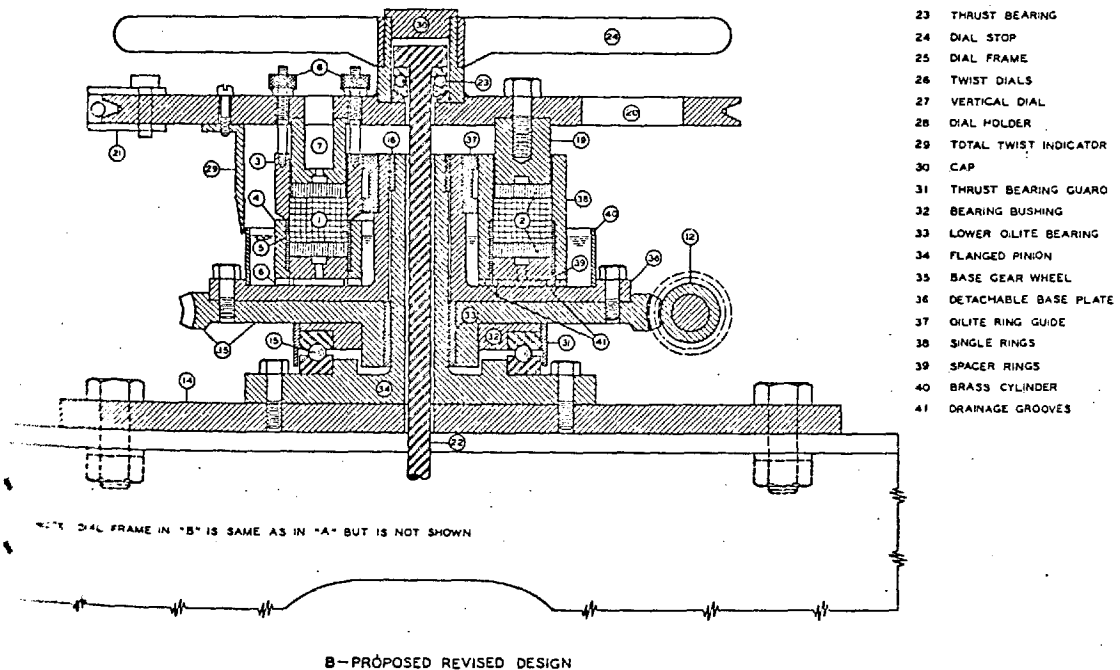
Soil sample	Description	LL %	PL %	PI	% clay fraction < 2 μ	σ'_n peak kg/cm ²	τ peak kg/cm ²
Wiener Tegel clay (Viennan clay)	O.C.	47	21	26	23	1.0	-
	N.C.	47	21	26	23	2.0	-
	O.C.	127	36	91	77	1.0	-
	N.C.	127	36	91	77	5.0	-
Fat clay from Weser Elbe Canal	Undisturbed	67	23	44	61	4.09	1.375
						3.09	1.125
						2.09	0.97
						1.09	0.825
	Remoulded	"	"	"	"	4.09	1.32
						3.09	1.0
						2.09	0.71
						1.09	0.6
	Undisturbed	68	22	46	62	3.09	0.73
						2.09	0.65
						1.09	0.6
						0.6	0.6
Lean clay	Undisturbed	44	20	24	32	4.09	2.0
						1.09	1.73
Brown Coal Soil from middle of R. Oder (silty clay with some sand)	Undisturbed	79	38	41	38	4.09	3.1
						2.09	1.95
Sandy soil from Kurzawka	Undisturbed	-	-	-	3	2.09	1.68
London clay	Remoulded	78	29	49		5.92	1.68
						4.44	1.31
						2.96	0.975
Atlantic Muck (soft clays, silts, sands and organic matter).	Remoulded C.C.	145	-	69	-	2.0	1.27
		163		106			
	N.C.					0.5	0.3
	N.C.					5.0	1.77
Lake Provvidene clay	O.C.	104	35	69	-	2.0	1.088

$\bar{\tau}_r$ kg/cm ²	Displacement (cm)		$\frac{\tau_r}{\tau_p}$	ϕ_r degree ($\sigma'_1=0$)	Brittle- ness factor $\frac{\tau_p - \tau_r}{\tau_p}$	Remarks	Reference
	to τ_p	to τ_r					
-	0.1	0.3	0.72	17.5°		Probably not comp- letely drained	Hvorslev, (1939, 1960)
-	0.7	2.0	0.31				
-	0.15	3.0	0.3				
-	0.4	3.0	0.4				
0.6	0.01	11	0.43	8.3°	56.4	Time to failure at peak strength = 16 days -drained tests.	Tiedemann (1937)
0.5	0.015	10	0.44	9.2°	55.5		
0.39	0.02	11	0.40	10.6°	59.8		
0.24	0.03	12	0.29	12.3°	70.9		
0.59	0.03	13	0.44	8.2°	55.3		
0.45	0.02	13	0.45	8.3°	55.0		
0.32	0.03	12	0.45	8.7°	55.0		
0.42	0.01	9.0	0.57	7.75°	42.5		
0.31	0.005	9.5	0.47	8.4°	52.3		
0.225	0.003	9.3	0.37	11.7°	62.5		
0.7	0.025	11.2	0.35	9.7°	65.0		
0.33	0.023	8.7	0.19	16.8°	80.9		
1.12	0.3	19	0.36	15.3°	63.9		
0.7	0.2	25	0.35	18.5°	64.1		
1.47	0.2	4	0.375	35°	12.5		
0.916			0.545	8.8°	45.5	Drained Probably undrain- ed	Langer (1938)
0.808			0.617	10.3°	38.3		
0.593			0.608	11.3°	39.2		
0.4	0.2	12	0.32	11.3°	68.5	*1	Hvorslev and Kaufmann (1952)
0.26	0.3	2.5	0.87	27.5°	13.3	*2	
0.32	0.3	13	0.46	9.3°	54.0	*3	
0.373	0.15	1.2	0.34	10.5°	65.7	*1	

*1) Displ. rate=0.0243 in/min. *3) Displ. rate: 0.0243 in/min
 *2) Displ. rate=0.1217 in/min. before p; 0.1217 in/min after p



- LEGEND**
- 1 RING TEST SPECIMEN
 - 2 DENTATED POROUS PLATES
 - 3 UPPER CONFINING RINGS
 - 4 LOWER CONFINING RINGS
 - 5 TRANSFER RINGS
 - 6 RING ELEVATOR SCREWS
 - 7 UPPER WATER OUTLET
 - 8 LOWER WATER OUTLET
 - 9 PLEXIGLASS CYLINDER
 - 10 ROTATING BASE
 - 11 BASE GEAR
 - 12 WORM GEAR
 - 13 PINION WITH FLANGE
 - 14 FIXED BASE PLATE
 - 15 BASE THRUST BEARING
 - 16 BASE GUIDE BEARING
 - 17 RING GUIDE BEARING
 - 18 OLITE PINION BEARING
 - 19 RING PISTON
 - 20 TORQUE WHEEL
 - 21 WIRE ROPE CLAMP
 - 22 LOADING ROD



- 23 THRUST BEARING
- 24 DIAL STOP
- 25 DIAL FRAME
- 26 TWIST DIALS
- 27 VERTICAL DIAL
- 28 DIAL HOLDER
- 29 TOTAL TWIST INDICATOR
- 30 CAP
- 31 THRUST BEARING GUARD
- 32 BEARING BUSHING
- 33 LOWER OLITE BEARING
- 34 FLANGED PINION
- 35 BASE GEAR WHEEL
- 36 DETACHABLE BASE PLATE
- 37 OLITE RING GUIDE
- 38 SINGLE RINGS
- 39 SPACER RINGS
- 40 BRASS CYLINDER
- 41 DRAINAGE GROOVES

NOTE: DIAL FRAME IN "B" IS SAME AS IN "A" BUT IS NOT SHOWN

SCALE-INCHES
0 1 2 3 4 5 6 7 8 9 10 11 12 13

Figure 1.1 Composite section through torsion shear apparatus

(HVORSLEV & KAUFMAN,) 1952.

CHAPTER 2

2.0 The new ring shear apparatus

The design of the apparatus is a result of collaboration between the Soil Mechanics Department of Imperial College and the Norwegian Geotechnical Institute. The apparatus was machined and fabricated by 'Geonor', Oslo. The apparatus will be described in some detail as this has not yet been done elsewhere.

2.1 Principles of the apparatus

The principles of this apparatus have been outlined in fig. 2-1. The sample, 6 in. O.D., 4 in. I.D. and 0.75 in. high is confined between a set of upper and lower confining rings. The lower set of confining rings is fixed to the rotating table which can be turned about the central vertical axis by means of worm drive. The sample is loaded by placing dead loads on the lever loading system under the base of the apparatus. The load is conveyed through the main shaft to the loading yoke which rests on the loading platten resting on top of the sample. A special feature of this apparatus is the ability to measure vertical frictional force between the sample and the upper confining rings. The upper confining ring can be lifted by raising the linking yoke by means of a differential screw at the top of the apparatus and the magnitude of the frictional force can be measured by the upper proving ring as shown in fig. 2-1.

As the rotating table turns, the lower confining rings and the lower half of the specimen rotate with it while the upper half is restrained from doing so by the torque arm which bears against the two side proving rings, from which the torque acting on the sample may be computed.

2.2 Detailed description

The apparatus can be described in detail by subdividing it into four components:

- a) Sample assembly,
- b) vertical loading system,
- c) the lifting mechanism,
- d) the torque measuring system.

2.2.1 General layout

The general layout of the apparatus is shown in fig. 2-2 and plate 2-1. The base is rigid aluminium casting, 100 cm long and 50 cm broad. Two vertical rigid steel columns are mounted on the base and provide reactions for the torque arm, as well as carrying the crosshead. The crosshead supports, and provides a reaction for, the upper confining ring lifting mechanism. A horizontal worm gear which rotates the sample assembly is mounted in the worm gear housing on the base. The worm gear is provided with a dog clutch (shown as 'drive to worm') which can engage a sprocket wheel moved by a chain drive from a motor, and variable speed gearbox. A principal element of the general

arrangement is the central main shaft not shown in fig. 2-2, but readily apparent in fig. 2-4 and 2-5. This carries the torque arm which also transmits the vertical load to the platten. The shaft is designed to provide the necessary vertical alignment for these load assemblies and the upper confining ring lifting mechanism.

The vertical loading on the sample is provided by dead weights hanging from the lever loading arm. The vertical deformation of the sample is measured by the two sample thickness dial gauges mounted on the dial gauge bridge as shown in fig. 2-2. The vertical force required to lift the upper confining rings in order to measure side-wall friction between the soil and the rings is measured by the upper proving ring and the dial gauge. The width of the gap between the upper and lower confining rings is measured by the two confining ring gap dial gauges which are also mounted on the dial gauge bridge. The torque loads are measured by two tangential load proving rings mounted on the rigid columns while the angular rotation of the sample is read on a circular scale with a vernier which enables the rotation to be measured to the nearest 0.1° . All dial gauges are capable of reading to 0.0001 of an inch.

2.2.2 Sample Assembly

The sample assembly is shown in detail in fig. 2-3 and 2-4. The concentric sample is confined between two sets of confining rings namely: the upper, outer and inner, confining rings and the lower, outer and inner, confining

rings, and the upper and lower annular porous ceramic discs. The confining rings are made of chromium plated brass alloy while the ceramic discs are made of moulded epoxy bonded carborundum. In order to overcome the risk of slipping between the ceramic and the soil, 12 radial bronze fins, 0.01 in. thick, 0.080 in. high and extending along the full width of the sample, are embedded into each of the ceramic discs. Both upper and lower ceramic discs are firmly screwed to the upper and lower annular loading plattens which are also made of chromium plated brass alloy. The lower outer and inner confining rings and the lower loading platten with the ceramic disc are fixed to the rotating table by means of eight fastening screws and two dowel pins as shown in fig. 2-4. The upper confining rings are aligned with the lower rings by a guide ring and locking screws. This guide ring is lowered during the shearing stage. A perspex cylinder sealed by an O-ring seal enables the sample to be submerged in a water bath.

2.2.3 Vertical loading system

The vertical loading system shown in figs. 2-2, 2-4 and 2-5, provides the normal stress on the horizontal shear plane of the sample. Dead weights are placed on a hanger at the end of the lever loading arm which provides a lever ratio of 10:1. The load is transferred to the main shaft through the thrust ball bearings which allow the rotation of the shaft and provide an axial load even when the lever loading arm is not in the normal horizontal position. A handle fitted to the torque free swivel (fig. 2-5) is provided to permit it to be given a small oscillatory motion

to eliminate any error in the torque readings.

An essential part of the loading arm mechanism is the system of counterweight and adjustments which are located under the base of the apparatus and shown in fig. 2-2. In order to accomodate vertical movements on the main shaft due to the variation in height of the sample, the lever loading arm is pivoted on an adjustable fulcrum. This can be moved up or down by turning the levelling screw and involves no change in the balance of forces. The vertical loading system can in this way be lifted or lowered approximately 0.5 in. from the middle position. Fig. 2-2 also shows a movable counterweight operated by a screw control at the side of the apparatus which is provided to balance the weights of the lever loading arm, including the dead weight hanger, the main shaft and the torque arm which act on the sample. A spirit level adjacent to the counterweight adjusting screw is provided to assist in keeping the lever loading arm horizontal.

The transfer of the vertical load from the main shaft to the sample is illustrated in fig. 2-4. The load from the main shaft is transferred to the torque arm by a nut through the spherical seating. The spherical seating at the top of the main shaft provides the torque arm with the limited ability to tilt if the sample height varies differentially during a test. The load is finally conveyed to the sample through the curved segmental spacer blocks screwed to the torque arm and which rest on the top loading platten of the sample assembly. Four guide pins are provided for alignment of the spacer blocks with the top loading platten.

2.2.4 The lifting mechanism

The components for the upper confining ring lifting mechanism are shown in figs. 2-2 and 2-5. A cast aluminium linking yoke is mounted on the main shaft with a ball bearing guide bushing to accommodate linear and rotary motion. Four pairs of fastening screws connect the upper, outer and inner, confining rings to the linking yoke. The confining rings can now be lifted by turning the differential screw mounted on the crosshead of the apparatus. Two guide pins align the linking yoke with the upper loading platten to ensure that during shearing, the upper half of the sample, the upper confining rings and the top loading platten move as a unit relative to the lower half of the sample. The lifting force is measured on the proving ring and the upper dial gauge whose spindle passes through the hollow centre of the differential screw (fig. 2-2).

2.2.5 Torque measuring system

The sample is subjected to torsion by transmitting a moment from the rotating table to the torque arm as shown in fig. 2-4. The driving torque, is provided by an A.C. motor with a 'Kopp' speed variator. A chain and sprocket connect the external drive to a dog-clutch on the input shaft of the worm gear to transmit the torque to the rotating table which is positioned by two radial deep groove ball bearings mounted on machined surfaces of the worm gear housing.

An oil filler plug, drain plug, and dip-stick oil level indicator are provided for gear lubrication (fig. 2-5).

Neoprene sealing rings are provided to prevent dirt and water from entering the moving parts.

The torque in the rotating table to which the base plate is fixed is then transmitted through the lower half of the sample to the upper loading platten and spacer block and the upper confining rings, and finally to the torque arm. The reaction for the torque arm is provided by two proving rings, mounted on the rigid columns. The proving rings are fitted with small wheels which can roll on the hardened steel discs at the ends of the torque arm, and which transmit an axial force to the rings without introducing any vertical friction component which may develop due to vertical movement of the torque arm, as the sample dilates or consolidates.

2.3 Calibration of the apparatus

2.3.1 Calibration of proving rings

The top friction measuring proving ring has to be calibrated in both tension and compression while the two torque force measuring rings need to be calibrated in compression only. The apparatus for calibration is very simple and consists of a 4 ft. length of rectangular steel box and a rectangular yoke which can straddle the box and which may be loaded at the bottom.

2.3.1.1 Calibration in compression

The set-up is illustrated in plate 2-2 (a). A smooth faced brass fitting is fitted in a $1\frac{1}{2}$ in. diameter hole through the channel. The shaft of the proving ring is next fed through this hole so that the proving ring sits firmly on the brass fitting. In the case of the friction proving ring, it was found convenient to apply the load through the bell-shaped casting of the linking yoke as shown in the illustration, plate 2-2 (a). The side proving rings could be set up in a similar way except that the rectangular loading yoke was placed directly on the loading wheel of these rings.

2.3.1.2 Calibration in tension

The proving ring is now suspended from the rectangular channel by means of a locking nut as shown in plate 2-2 (b). A short length of $\frac{1}{2}$ in. thick brass strip with a $\frac{1}{2}$ in. diameter tapped hole in the middle is positioned in the slot in the aluminium casting underneath the proving ring. A loading hanger can now be screwed in the brass strip and dead loads applied.

2.3.2 Static friction in the lever loading system

The lever loading system (including the weight of the hanger and the weight of the shaft) is carefully balanced by means of the counterweight. Very small weights are next

placed on the hanger until the lever loading arm moves continuously. It was found that only about 10 gms. on the hanger was required to overcome static friction in the system. This corresponds to a force of 100 gms. on the specimen and can be neglected.

2.3.3 Friction in the vertical loading system and the examination of the lever arm ratio

It is important to investigate the above effects to ascertain how much of the load on the hanger is actually conveyed to the sample. This can easily be done by coupling the vertical loading system to the lifting mechanism so that the load acting on the sample can be read off the upper proving ring. This can be done conveniently by removing the segmental spacer blocks from the torque arm and allowing it rest on a $\frac{1}{2}$ in. thick, steel ring, 6 in. I.D. 8 in. O.D., which is placed on top of the upper confining ring. (The top loading platten is removed during this stage.) A diagrammatic sketch of the layout is shown in fig. 2-6, from which it can be seen that as the lever loading system pulls the torque arm downwards the load is transferred to the linking yoke through the spacer ring and its value can be obtained from the upper proving ring. After the vertical loading system is balanced by the counterweight, dead weights are added on the hanger and corresponding loads on the upper proving ring are noted. Assuming that the loading arm has a lever ratio of 10:1, the loss in load was again found to be negligible. For hanger loads of

more than 3 lbs., a maximum loss of 0.3% of the total load acting on the sample, with no repeatable trends was noticed, and this accuracy is well within the accuracy of the proving ring measurements. Hence no appreciable inaccuracy would be introduced by assuming a lever arm ratio of 10:1, and neglecting friction in the linear bearings.

2.3.4 Effect of any horizontal thrust in the linear bearings

To investigate the effect of any possible development of horizontal thrust in the linear bearings during shearing stage, the apparatus was assembled as for (2.3.3), and a loop of cord was fed around the main shaft fig. 2.6, so that a horizontal force could be directly applied by hanging loads on this cord passing over a pulley as shown by dotted lines in fig. 2-6.

No change in upper proving ring dial gage could be detected when horizontal loads of up to 60 lbs. were applied.

Hence, it can be concluded that provided the vertical loading system is balanced carefully by the counterweight, no appreciable error (less than 0.3%) would be caused by any friction losses provided the load on the hanger is greater than 3 lbs.

2.3.5 Calibration of vertical deformation of apparatus under dead loading

The vertical compression of the apparatus has to be investigated so that its effect may be taken into account from the readings of height change dial gauges. This is done by assembling the apparatus with a dummy brass ring 'sample'. However, in order to avoid damage to the radial fins, the top and bottom annular ceramic with fins are replaced by annular ceramic without radial fins. A calibration graph of load on the hanger against vertical deformation is plotted as shown in Appendix A.

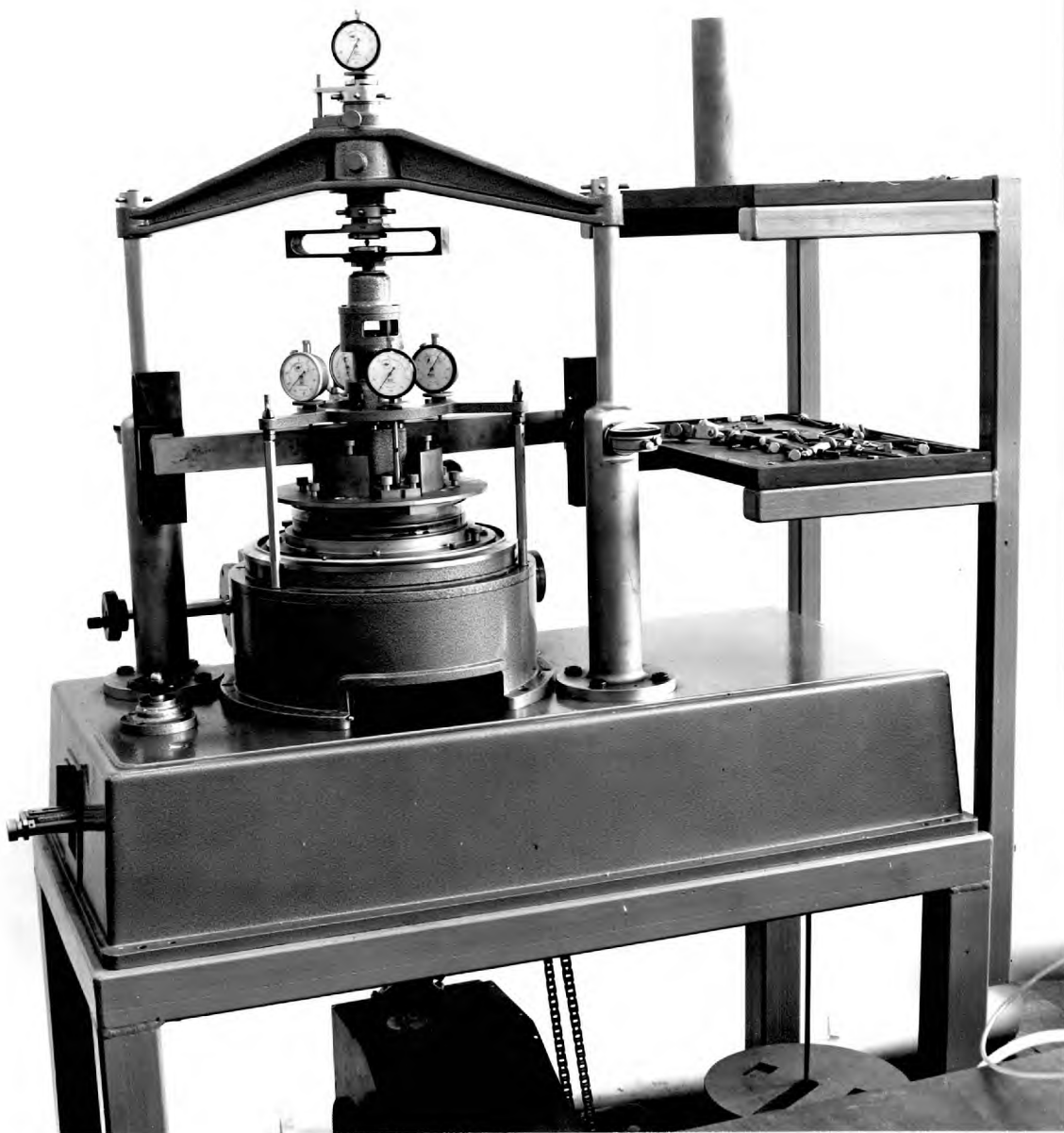


PLATE 2.1

Ring shear apparatus.

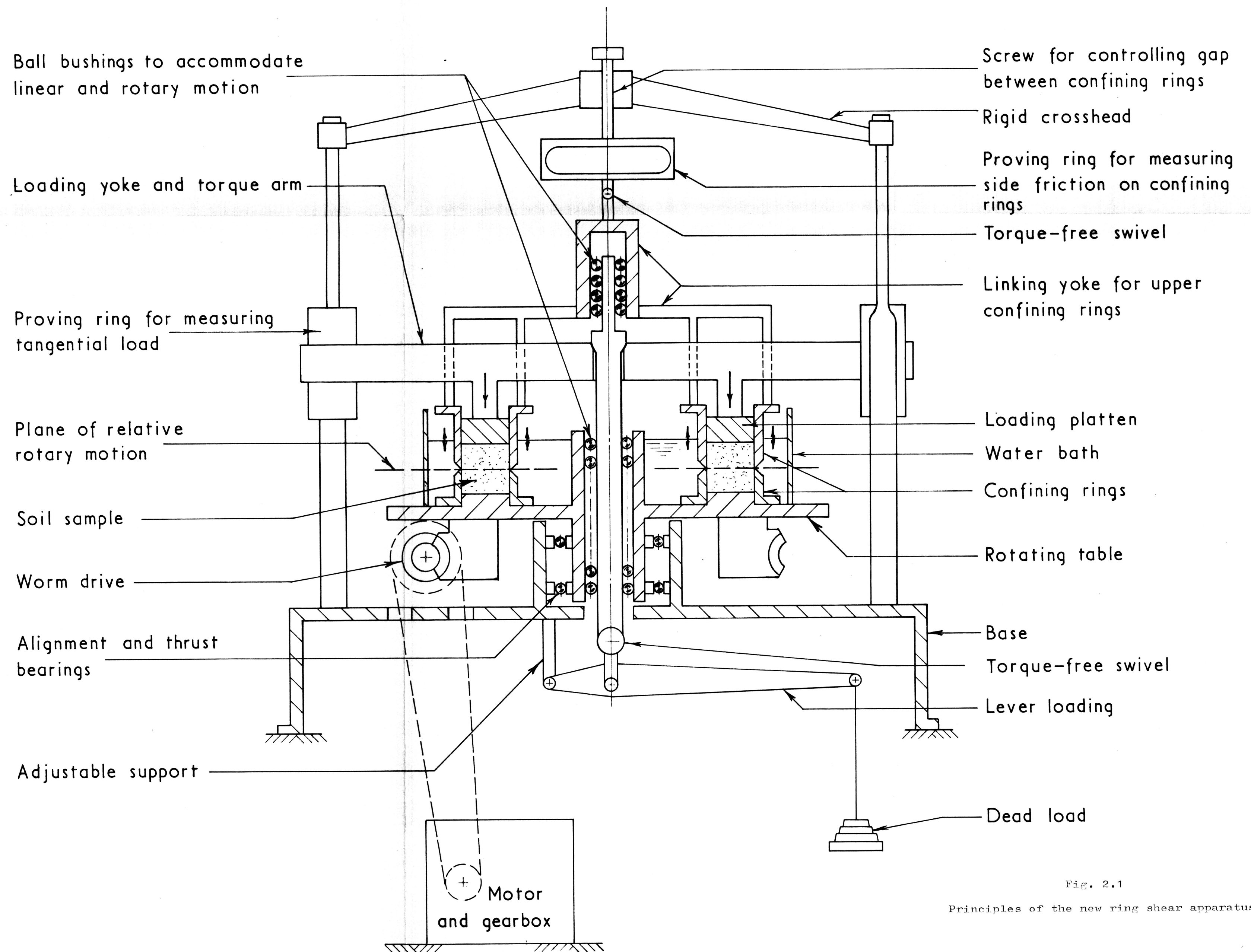


Fig. 2.1

Principles of the new ring shear apparatus.

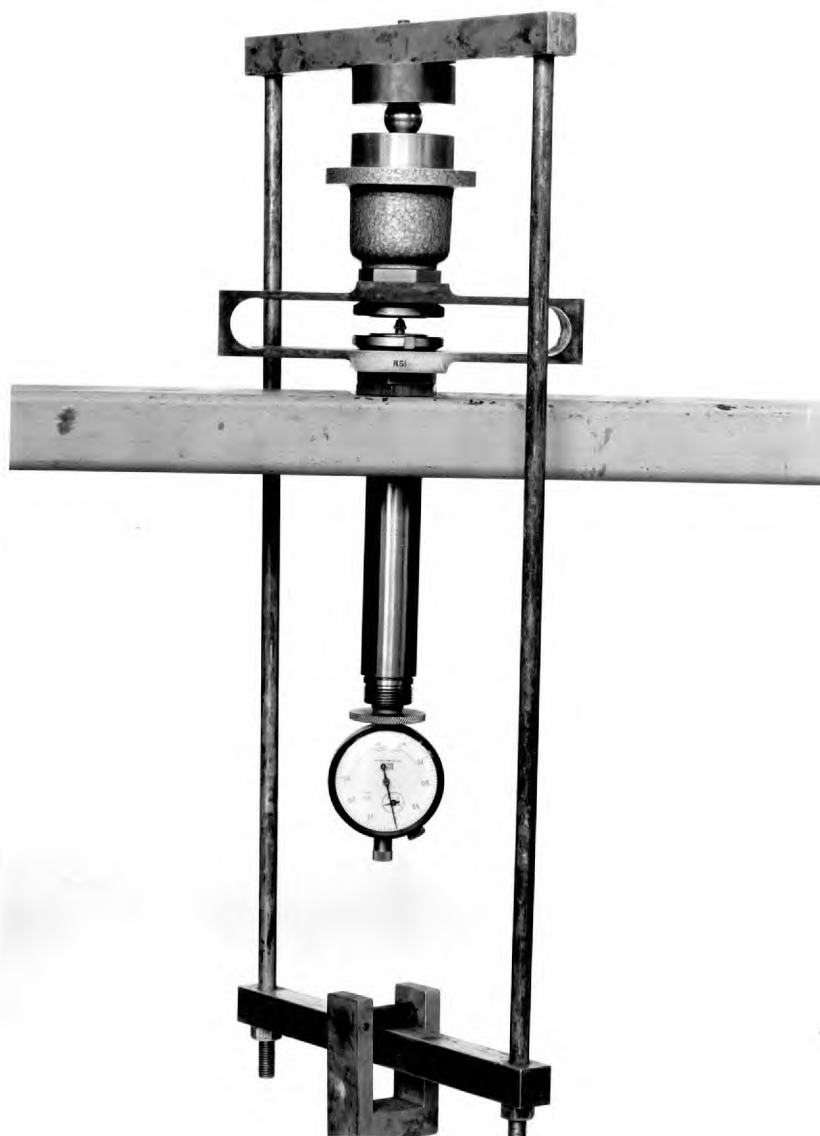


PLATE 2.2 (a)

Calibration of proving rings in
compression.

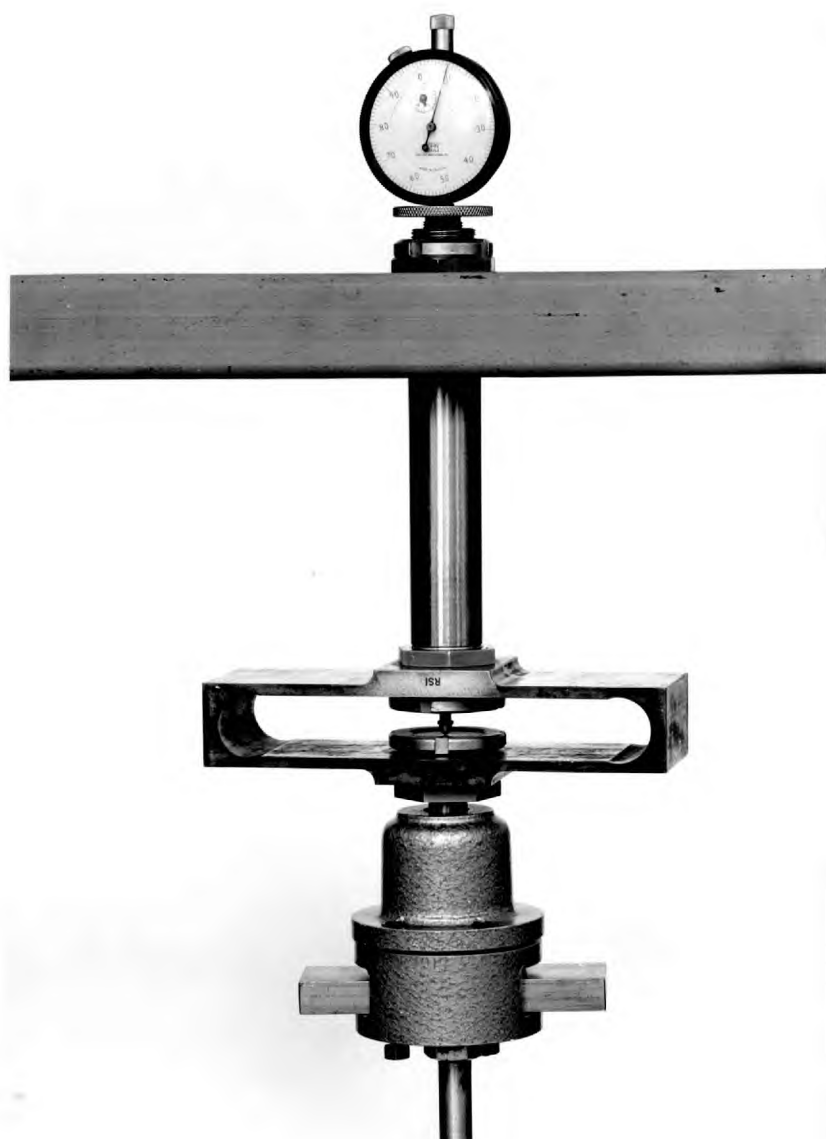


PLATE 2.2 (b)

Calibration of proving ring in tension.

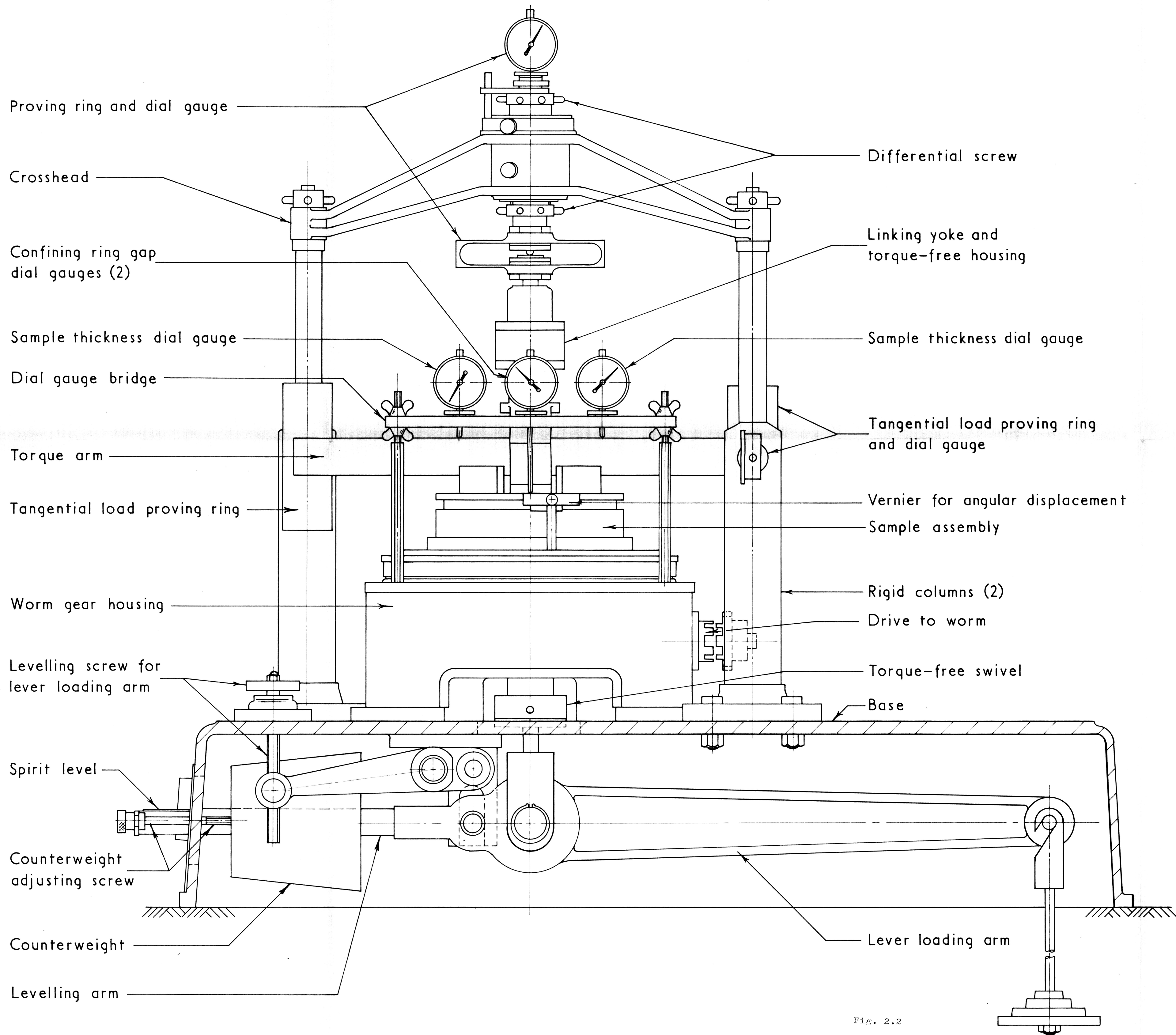


Fig. 2.2
General layout.

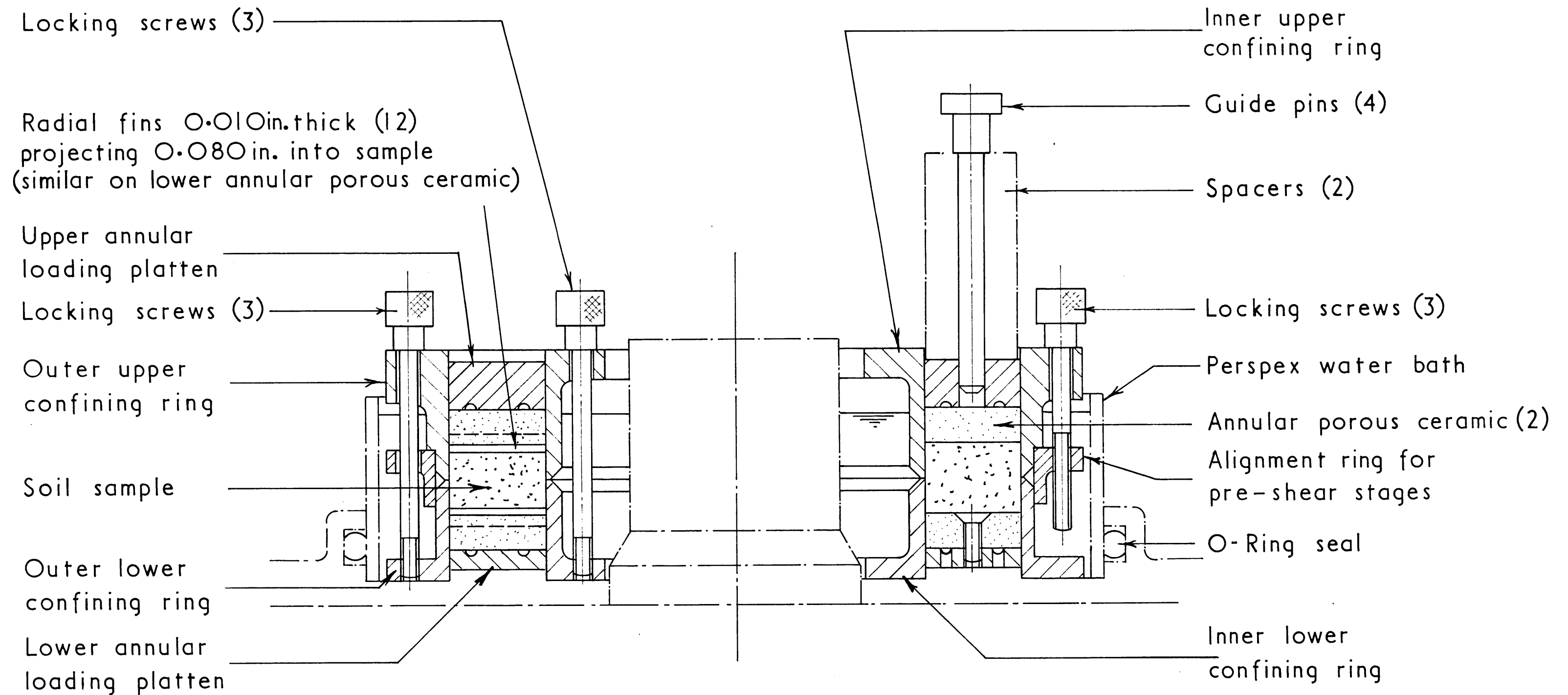


Fig. 2.3

Sample assembly.

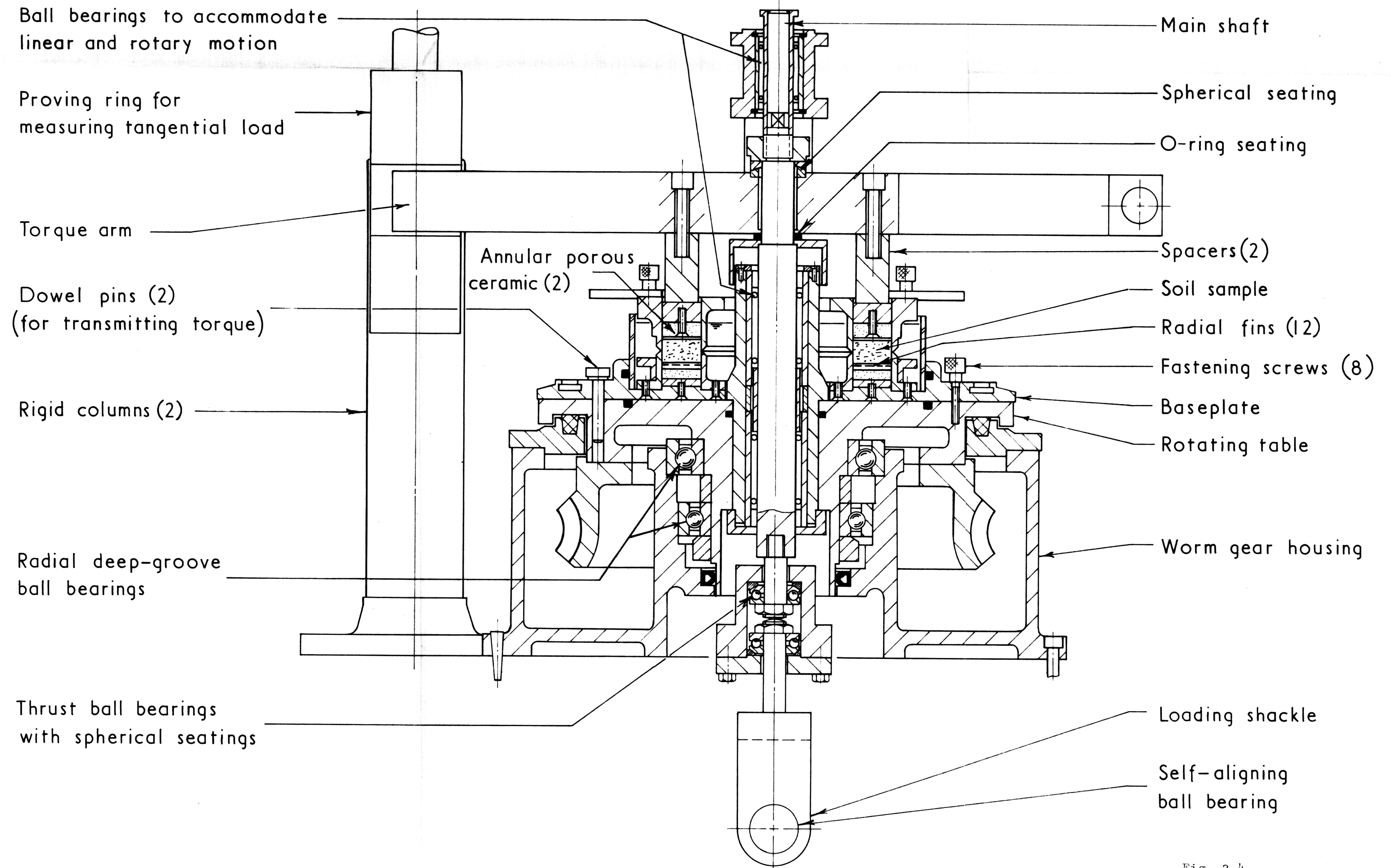


Fig. 2.4

Longitudinal section showing loading yoke and torque arm.

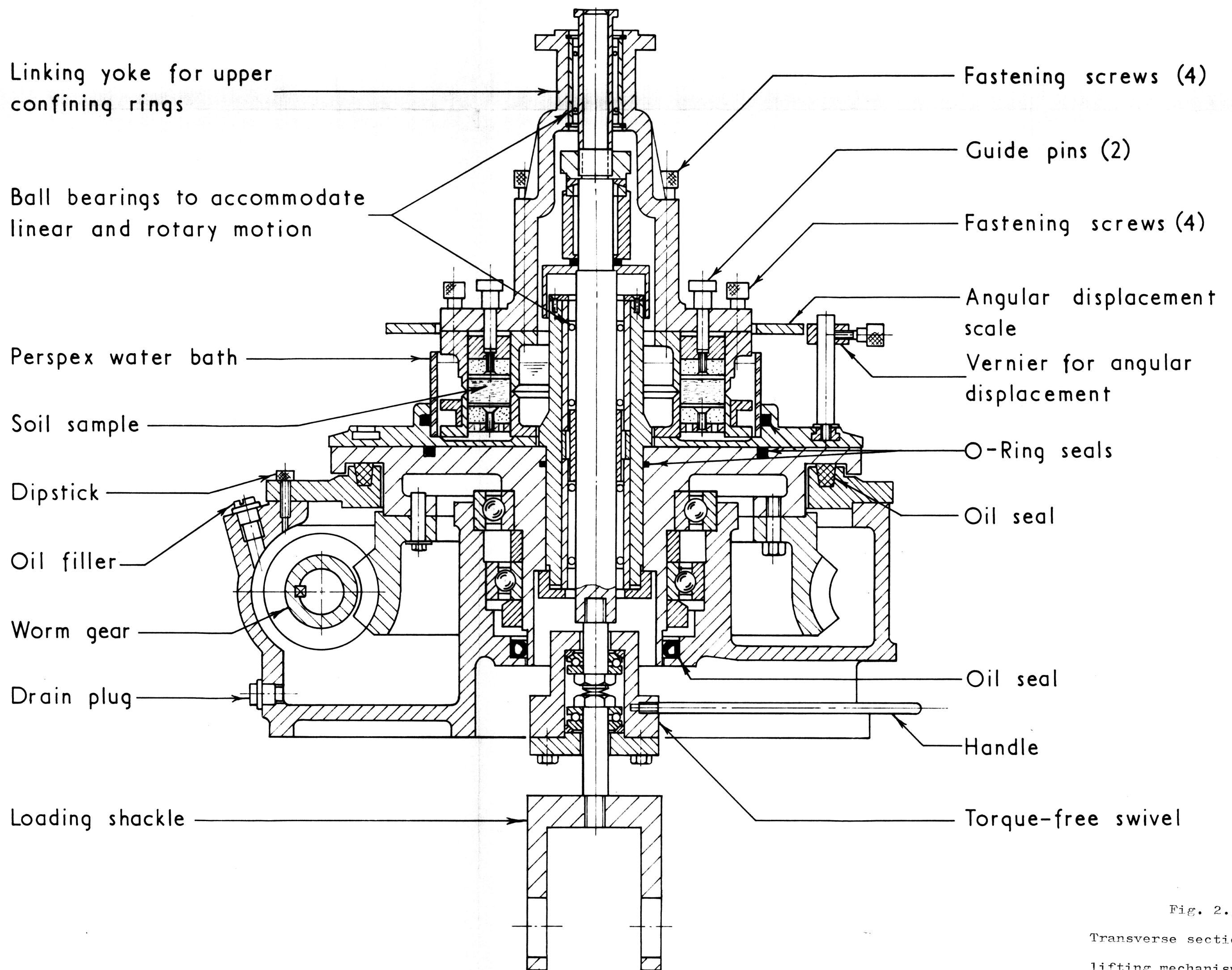
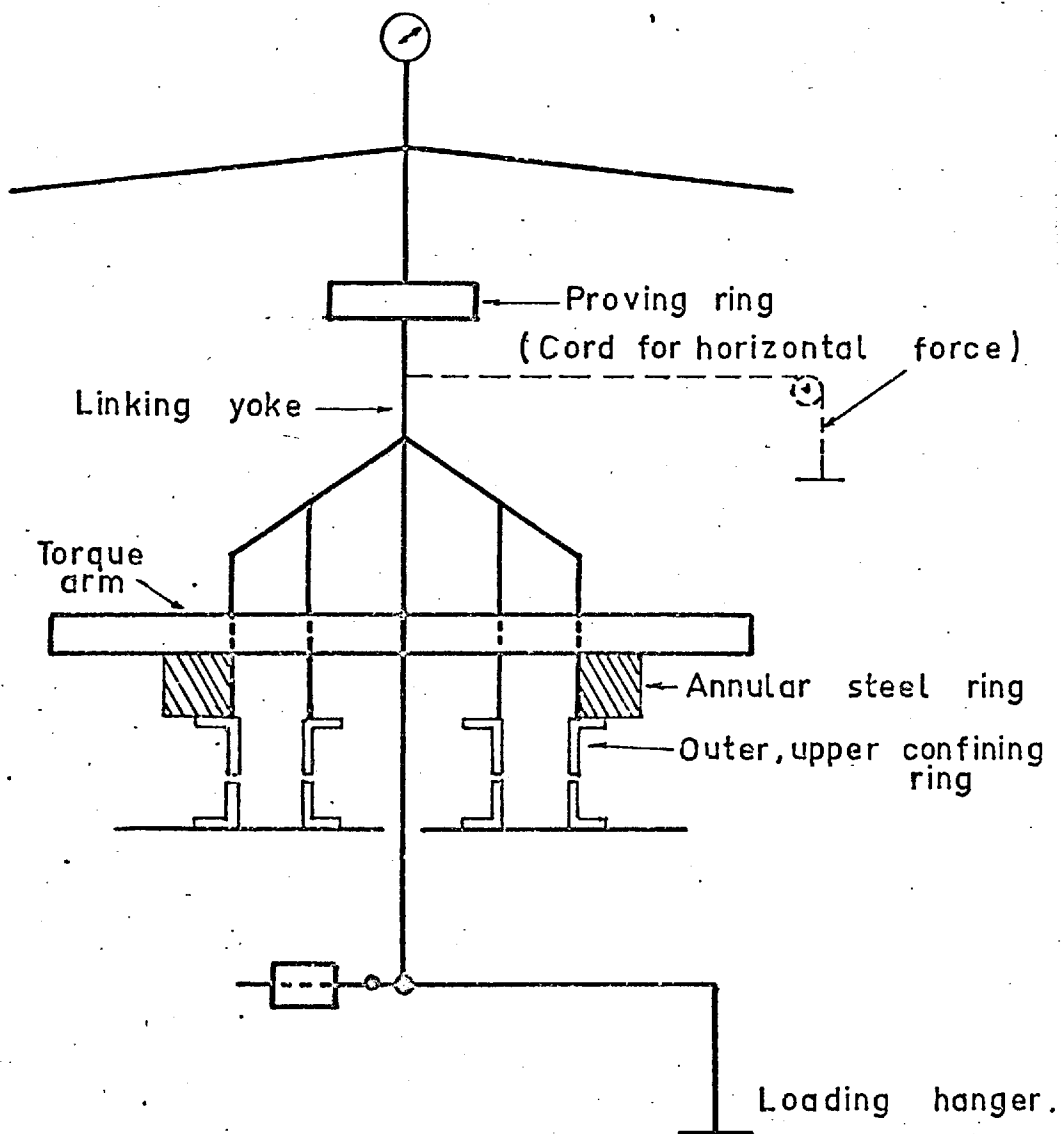


Fig. 2.5
Transverse section showing
lifting mechanism.



DIAGRAMATICAL SKETCH TO DETERMINE THE EFFECT OF LEVER ARM RATIO AND FRICTION IN LINEAR BEARINGS.

CHAPTER 3

Sample preparation; and test procedure

Two types of tests were carried out - tests on remoulded, and on 'undisturbed' samples.

3.1.1. Remoulded samples

Sample preparation of remoulded clay materials presented no difficulty. Clay was first air-dried in the laboratory, after which it was placed in a rotary grinder and ground for about 60 minutes. The ground material is then sieved through B.S. Sieve 36. Only material passing through this sieve size is used in the preparation of remoulded samples. After the Atterberg limits have been established some distilled water is added to the remainder of the dried clay and the mass is constantly stirred with a spatula to achieve a uniform water content throughout the sample. The water content was generally brought up to half of that at the liquid limit. The sample is now ready for placement in the apparatus.

The upper and lower, outer and inner, confining rings are then firmly screwed together to the base plate. The lower ceramic annulus, which has already been deaired in boiling water is also firmly screwed in the base plate. Wet clay, in 1" wide, 1/4" thick strips, is then carefully kneaded in; taking great care that no air voids are entrapped. After the sample has been built well over 3/4" thickness, a special scraping tool is then passed over the

soil to level it to exactly $3/4$ " thickness. The tool, which is made of special $1/8$ " thick hard gauge metal has a tip 1 in. wide and runs along the tops of both inner and outer upper confining rings. Its depth is such that it scrapes off any soil lying above the $3/4$ " height level and also removes any soil that may be lying around the sides of the confining rings. This also makes a clean passage for the upper ceramic annulus and top ring to be placed in.

3.1.2. Undisturbed samples

The 'undisturbed' samples were usually brought from the field in 9"x9"x12" high boxes. Soil was covered by aluminium foil over which molten paraffin had been poured to ensure no loss of moisture during storage.

The placement of undisturbed soil in the apparatus is achieved by using different cutting rings and wooden blocks as shown in fig. 3-1, a-f. A sharp, steel cutting ring is pushed in the block of clay (fig. 3.1 (a)). This ring is 6.0 in. internal diameter, $3/4$ in. high and 0.045 in. thick. Because the cutting ring has a small wall thickness (for least disturbance while cutting), it has a tendency to distort as it is pushed into the soil. To prevent this, a stiffening ring (0.08 in. thick, 6.0" in. internal diameter) is mounted around the cutting ring to provide lateral rigidity. The cutting ring coupled with the stiffening ring is, then pushed evenly into the block of clay, which is then cut by means of a wire saw.

The stiffening ring is removed and the clay on the upper end of the cutting ring is carefully planed with a

straight edge (fig. 3.1 (b)).

The outer upper and lower confining rings are screwed together and placed on a smooth working top. A wooden spacer nominally 6.0 in. in diameter with small clearance gap and 1.7 in. high is then placed inside the confining ring assembly. (Note that diameters of all wooden spacer blocks mentioned hereafter are nominal values and in practice a small clearance gap is maintained for ease of operation. Further, all the wooden blocks used were varnished to prevent any seasonal variation in dimensions and the top surfaces, which come into contact with soil, have a formica finish.) The upper confining ring has a small recess into which the cutting ring is now located. Soil is pushed down from the cutting ring on to the wooden spacer by using a heavy wooden block, 6.0" diameter and 2.7 in. thick, as a piston mounted in drill press as guide, (fig. 3.1 (c) and plate 3.1).

The cutting ring is now removed and the soil is planed with a straight edge such that the plane surface is flush with the top surface of the upper confining ring. The sample is now $3/4$ in. thick.

The 1.7 in. wooden spacer is now removed and a two piece wooden spacer 0.92" high and 6.0 in. diameter is substituted in its place. This spacer, consists of an annular ring, 6.0 in. O.D., 4.0 in. I.D., and a disc, also 0.92 in. high and 4.0 in. diameter, which fits inside the annulus. The sample is pushed down to rest on this spacer, and the whole assembly is carefully centred and clamped on

the working surface of a drill press (fig. 3.1 d). A thin walled steel core cutter is then positioned centrally in the drill-chuck. The cutter, 4.0 in. outside diameter and 2.25 in. high is made of hard steel, and has a wall thickness of only 0.003 in., to minimize any disturbance to the soil. It is firmly pushed into the sample, by lowering the drill head (fig. 3.1.e) and plate 3.2). This leaves behind a clay sample of required dimensions. The wooden spacer is removed very carefully and the sample assembly is placed on the base plate on which the inner lower confining ring and the lower annular loading platten with ceramic annulus (which has previously been deaired in boiling water) have already been preassembled. The clay is next embedded on to the radial fins of the lower ceramic and the upper, inner, confining ring is inserted (fig. 3.1 f). The sample assembly may now be fastened to the rotating table and the apparatus assembled. A V-notch is provided on all components of the sample assembly for subsequent correct positioning of the lifting yoke.

3.2 Assembly of the apparatus

The base plate with the sample assembly is aligned with the rotating table (a V-notch is marked on the components for easy alignment) and is fixed by means of two Dowel pins and eight fastening screws. The perspex cylinder which forms the water bath is next fitted in the base plate. The upper annular loading platten is aligned with the V-notch and placed gently over the soil sample. Care should be taken to ensure that the radial fins on the

underside of the upper loading platten do not embed deeply into the soil at this stage. The torque arm with the segmented spacer block (see fig. 2-4) is next carefully placed on top of the sample assembly and balanced by means of the counterweight. The sample assembly is then aligned with the torque arm such that four guide pins from the spacer blocks can be fed in their slots in the upper annular loading platten. To facilitate this, it may be necessary to rotate the whole sample assembly slightly by means of the rotating handle mounted on the horizontal worm gear drive. A nut which rests on the spherical seating in the middle of the torque arm is next tightened around the main shaft (fig. 2-4).

After the upper proving ring has been zeroed to balance the weight of the linking yoke, the cross-head carrying the linking yoke is mounted on the rigid columns. The guide pins and fastening screws are inserted so that the upper, outer and inner, confining rings are firmly attached to the linking yoke (fig. 2-5). The dial gauge bridge is now quickly assembled and zero readings of gap change and height-change dial gauges are noted after the lever loading arm has been levelled by means of the levelling screw and the spirit level. Dead weights are next placed on the hanger for consolidation to proceed and the water bath is completely filled with water. Finally the circular scale and the vernier are mounted to complete the assembly of the apparatus.

3.3 Dismantling of the apparatus

The dismantling operation takes place in reverse order of the assembling operation. After the final readings are noted, the motor is stopped and the drive to worm gear is disengaged.

After the water in the water bath has been siphoned off, the load on the hanger is removed. (The water is first removed so that the swelling of the soil on removal of load from the hanger may be minimised). Next the circular scale, the dial gauge yoke; the crosshead and the linking yoke; and the torque arm are successively and quickly removed. The base plate with the sample assembly can now be unfastened from the rotating table.

The problem now is to remove the sheared soil sample from the confining rings without disturbing it. To do this, the base plate (with the sample assembly) is lifted off the rotating table and is placed inverted on a 3 in. high metal ring (5.6 in. O.D.; 4.4 in. I.D.) with the annular upper loading platten resting on this ring. The sample assembly is next unscrewed from the base plate which is very carefully removed. The sample assembly is then placed on the metal ring such that it is now resting with the lower annular loading platten on the ring. The lower outer and inner confining rings are removed by pushing downwards while the upper pair are removed by sliding these upwards and applying a gentle pressure by thumbs on the upper loading platten. The soil sample is now confined by the upper and lower loading plattens only. Extreme care must

now be exercised to remove the upper loading platten so that the sample is not separated along its shear plane.

The determination of the final moisture contents of the sample can now be made. For the determination of water content profile along the height of the sample. A special thin-walled, $\frac{1}{4}$ in. diameter brass piston sampler was used. The determination of the final moisture content profile was made using $1/16$ in. thick soil discs weighed on aluminium foil on an accurate balance (accurate to 0.001 gm.).

3.4 Testing procedure

The sample is first allowed to consolidate (or swell) under a constant load on the hanger. After the consolidation is complete (as observed on the height change dial gauges), the confining guide ring is lowered and outer and inner locking screws, 8 in number, are removed. The original gap dial gauge reading is noted and the gap between the upper and lower confining rings is opened (initially by about 0.001 - 0.002 in.) by lifting the upper outer and inner confining rings by means of the differential screw. The dog clutch is next engaged and the motor is started. A speed is selected to give a constant rate of strain which permits adequate dissipation of pore pressure.

The speed was usually increased 5 to 25 times the original slow speed after peak failure in order to obtain

large deformations more quickly. The gap was usually closed during this stage. The shearing rate was then reduced to the original value, the gap reopened and the readings were taken after a few hours had elapsed to confirm whether a steady state had been achieved.

The same sample was used in a number of tests to measure residual shear strength under different normal stresses. When the residual condition had been established under a constant load on the hanger, the motor was stopped; the gap between confining rings was closed and the new load was placed on the hanger. The sample was allowed to consolidate (or swell) under the new normal stress, and at the completion of this stage (indicated by height change dial gauges), the gap was reopened and shearing commenced. Hence the subsequent shearing amounted to doing a test on a 'slip surface'. This method has also the advantage that residual strength under different normal stresses may be investigated by using only one sample. (It was, of course, necessary to determine that previous stress history had no significant effect on the value of ϕ_r' at any particular value of normal stress.) The following set of readings is taken at suitable intervals:

- a) Angular displacement in degrees as measured on the horizontal circular scale.
- b) Torque force readings on the two side torque proving rings.

- c) Friction force as registered on the friction proving ring.
- d) Sample height change readings.
- e) Gap width readings on the gap dial gauges.

3.5 Calculation of results

The average shearing stress, τ_a , is calculated on the assumption that normal stress and the shearing stress are uniformly distributed across the width of the failure surface in a horizontal plane. Then:

$$\tau_a = 3M/2\pi (R_2^3 - R_1^3) \quad - - - \quad (3-1)$$

where M is the torsional moment calculated from the readings on torque force proving rings;

and R_2 , R_1 are the external and internal radii of the sample respectively (= 3.0 and 2.0 in.).

$$\text{The normal stress, } \sigma'_n = P'/\pi (R_2^2 - R_1^2) \quad - - - \quad (3-2)$$

where P' is the net effective normal force acting on the sample.

The average displacement, in inches, is calculated on a mean radius of 2.5 in. It should be noted that because the apparatus produces a plane of failure perpendicular to the axis of the sample, a complicated stress condition is established, and it is no longer a case of pure torsion and pure shear (Hvorslev 1939). Hence equation

(3-1) is not strictly correct. Hvorslev (loc. cit.) therefore modified the Duguet - Ludwig - Prandtl solution for short thick cylinder (see Nadai 1931) to obtain stress strain curve by graphical differentiation of the moment - twist curve. However, work done at the Norwegian Geotechnical Institute has conclusively shown that the curves obtained from equation (3-1), and Hvorslev's method (which is tedious to perform) are virtually coincident (with, of course, no difference at residual) (see N.G.I. Internal Report F 346). Hence, in this project, all shear stress calculations have been based on equation (3-1).

3.6 Graphical presentation of results

Two main types of graphs have been used to present the results. The first is a graph of τ / σ'_n against displacement. It should be noted that since the friction value term in normal load changes as the test proceeds, (and hence the normal stress, σ'_n), it is advisable to plot values of the ratio τ / σ'_n at any particular point. The tangent of τ / σ'_n gives the value of ϕ' (origin ϕ'), the angle of friction directly.

The residual shear strength, by definition, operates when there is no further decrease in shear strength with displacement,

$$\text{i.e. when } \frac{d(\tau / \sigma'_n)}{d(\text{displacement})} = 0 \quad (3-3).$$

When the above condition is satisfied, the curve in the standard τ/σ'_n against displacement plot becomes horizontal. However, in practice, it is often difficult to visually judge from the above plot whether the soil has reached the residual state. Hence, another graph of τ/σ'_n against logarithm of displacement is also presented. It is suggested that in such a plot, the above condition (equation 3-3) will be satisfied when:

$$\frac{d(\tau/\sigma'_n)}{d(\log \text{ displacement})} = 0 \quad (3-4).$$

but with the advantage that the slope of the curve is now greatly exaggerated, thus making it easier to ascertain whether residual state has been reached. In subsequent chapter, a number of tests on direct shear apparatus have been presented on a shear stress against displacement plot where the slope of the curve appears to become zero. However, the same results when plotted against log. displacement demonstrate quite remarkably that, invariably, the residual state has not been achieved in direct shear tests.

3.7 Accuracy of results

The calibration of the apparatus has already been described in chapter 2, where it was mentioned that there appeared to be virtually no loss of load due to friction in the bearings. However, the following points have now to be considered to assess the accuracy of results.

- i) The effect of non uniform stress distribution on the failure plane, and,
 - ii) the effect of soil squeezing through the gap between the upper and lower confining rings:
- i) the calculation of average shearing stress, τ_a , using equation (3-1) implies that the normal stress acts uniformly over the failure surface. This may not necessarily be so as is seen by the small difference in loads on the proving rings at the end of the torque arm. It may also be non-uniform as a result of non-homogeneity of the sample and due to squeeze of soil on the failure surface through the confining ring gap.

The variation in ϕ'_r due to seven different stress distributions is shown in fig. 3.2, and the results are given in greater details in Appendix E. It will be seen from this figure that maximum variations in ϕ'_r (assuming uniformly distributed load) at assumed $\phi'_r = 10^\circ$, due to any non-uniform stress distribution that has been considered is about 0.85° or -0.6° . An examination of the slip surface across the width of the sample at the end of the test showed that the slip surface was concave downwards and suggests a parabolic symmetrical distribution of normal stress (cases paraboloid I and II). The following Table gives an estimate of the variation in ϕ'_r .

True ϕ'_r	Maximum variation	Most probable variation
10°	+0.85° to -0.6°	+ 0.1°
15°	+1.22° to -0.85°	+ 0.12°
20°	+1.6° to -0.12°	+ 0.13°

ii) The influence of squeeze through the gap

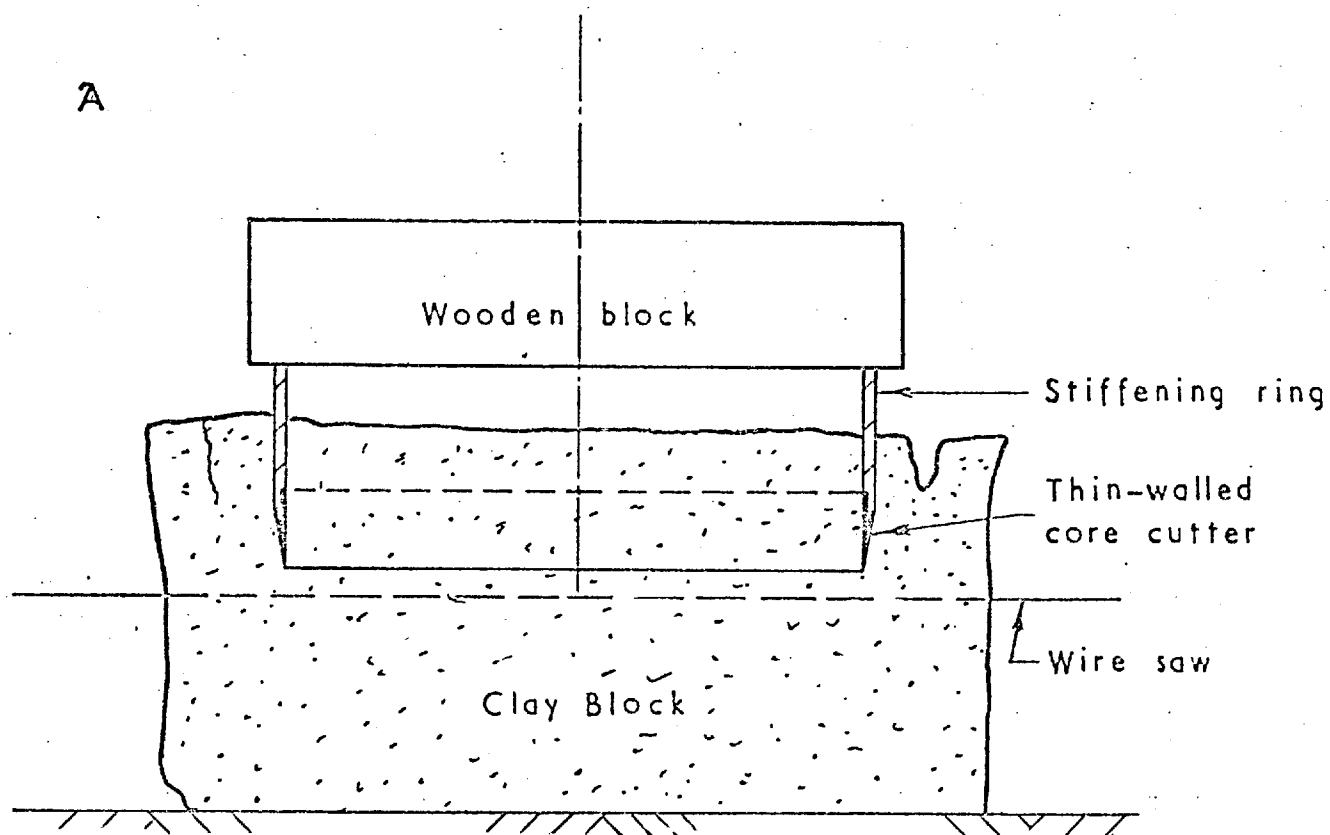
After initial consolidation of sample under a normal load, the upper confining rings are lifted by about 0.001 - 0.002 in. just prior to shearing the sample. This is done to ensure that net vertical effective load acts on the failure surface through the soil - soil interface. However, in doing so, the sample tends to squeeze out through the gap. This squeeze is seldom observed at peak strength, but it may be more marked at large displacements. The shearing resistance thus obtained may therefore be slightly larger than the true residual shear strength. An estimate of the maximum increase may be obtained by considering that squeeze takes place along a lip 1/32" wide. This, in effect, increases the external diameter from 6.0 in. to 6.031 in., and decreases the internal diameter from 4.0 in. to 3.969 in.. Hence the ratio between the apparent and true shearing stress is given by:

$$(6.031^3 - 3.969^3) / (6^3 - 4^3) = 1.03;$$

in other words, squeezing increases the value of

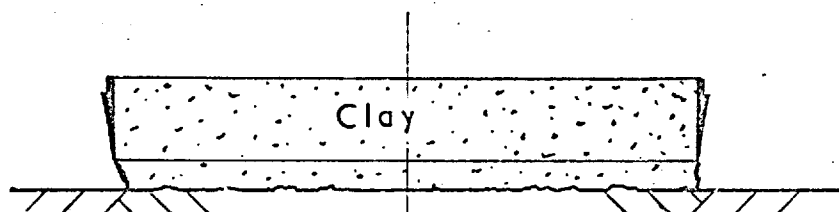
average shear resistance, τ_a , by 3%. The consequent increase in ϕ'_r would depend on the magnitude of normal stress. As an example, a clay which has a 'true' ϕ'_r of 9° at an effective normal stress of 30 psi. would show an apparent $\phi'_r = 9.3^\circ$. The actual increase would probably be much smaller than 3% because the coefficient of friction for soil on metal is less than for soil and soil and the vertical stress on the squeezed out soil in the gap would very likely be less than on the failure surface itself. Also, it has been noted in the tests carried out that often squeezing is not continuous and takes place only in limited sections of the gap. It should be mentioned that the direction of movement of squeezed material with respect to direction of shear will also determine the apparent increase or decrease in shear stress.

The squeezing of soil through the gap also implies that the decrease in height of the sample as registered on the height-change dial gauges does not accurately reflect the behaviour of the soil on shearing.



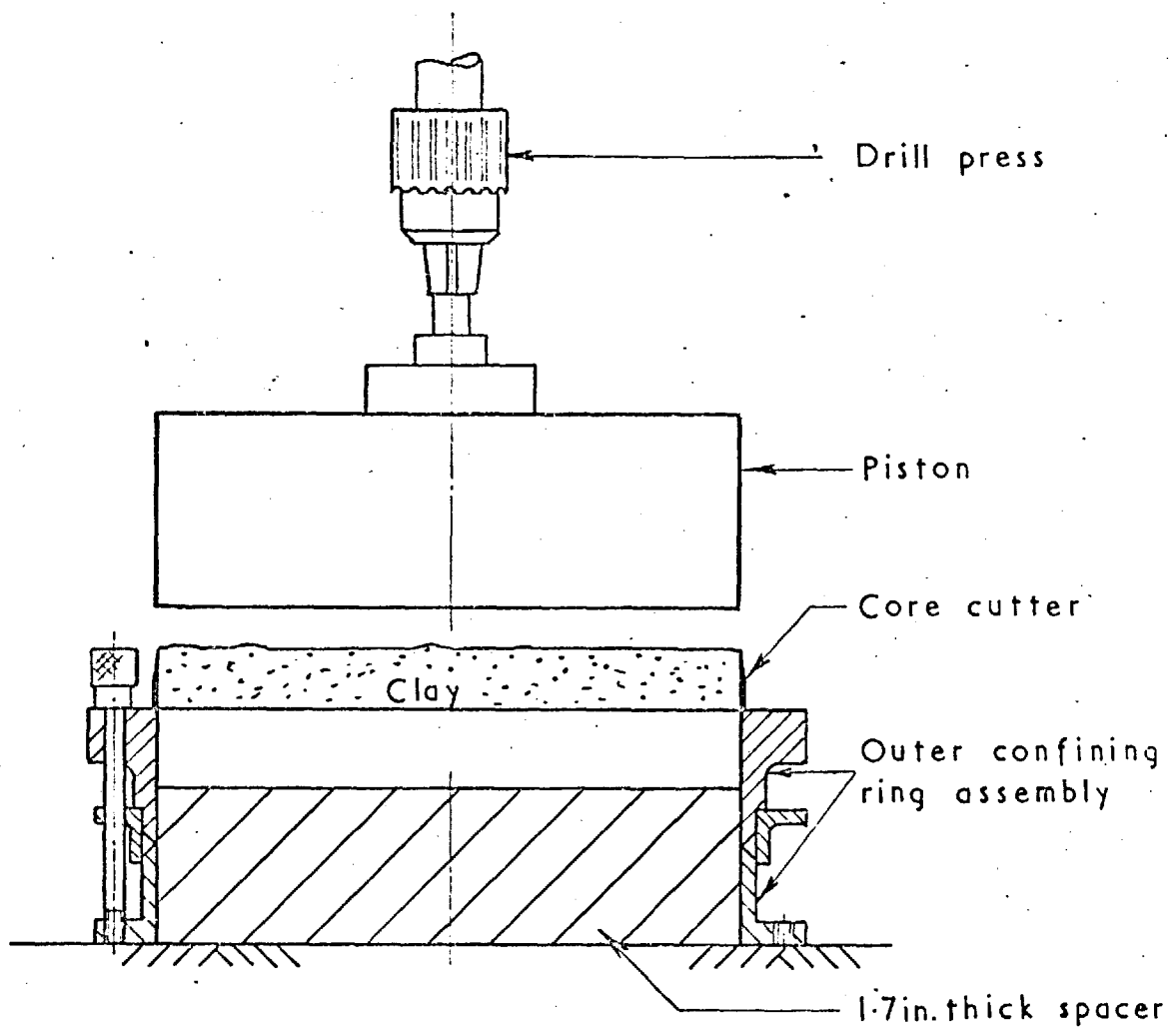
- 1 Stiffened core cutter pushed into clay block sample
- 2 Slice through block with wire saw

b



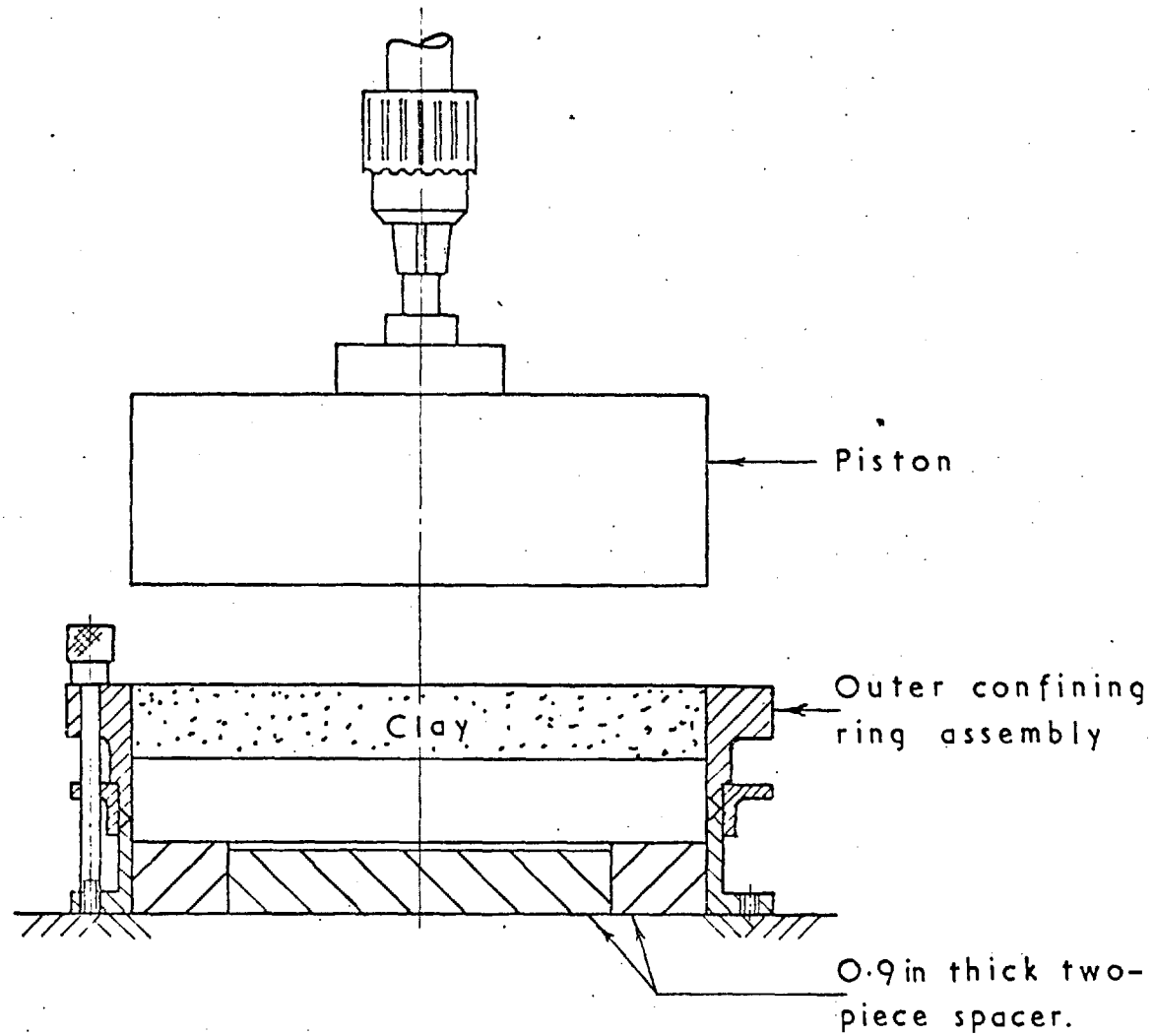
3. Trim off upper surface of clay until flush with core cutter

FIG. 3.1



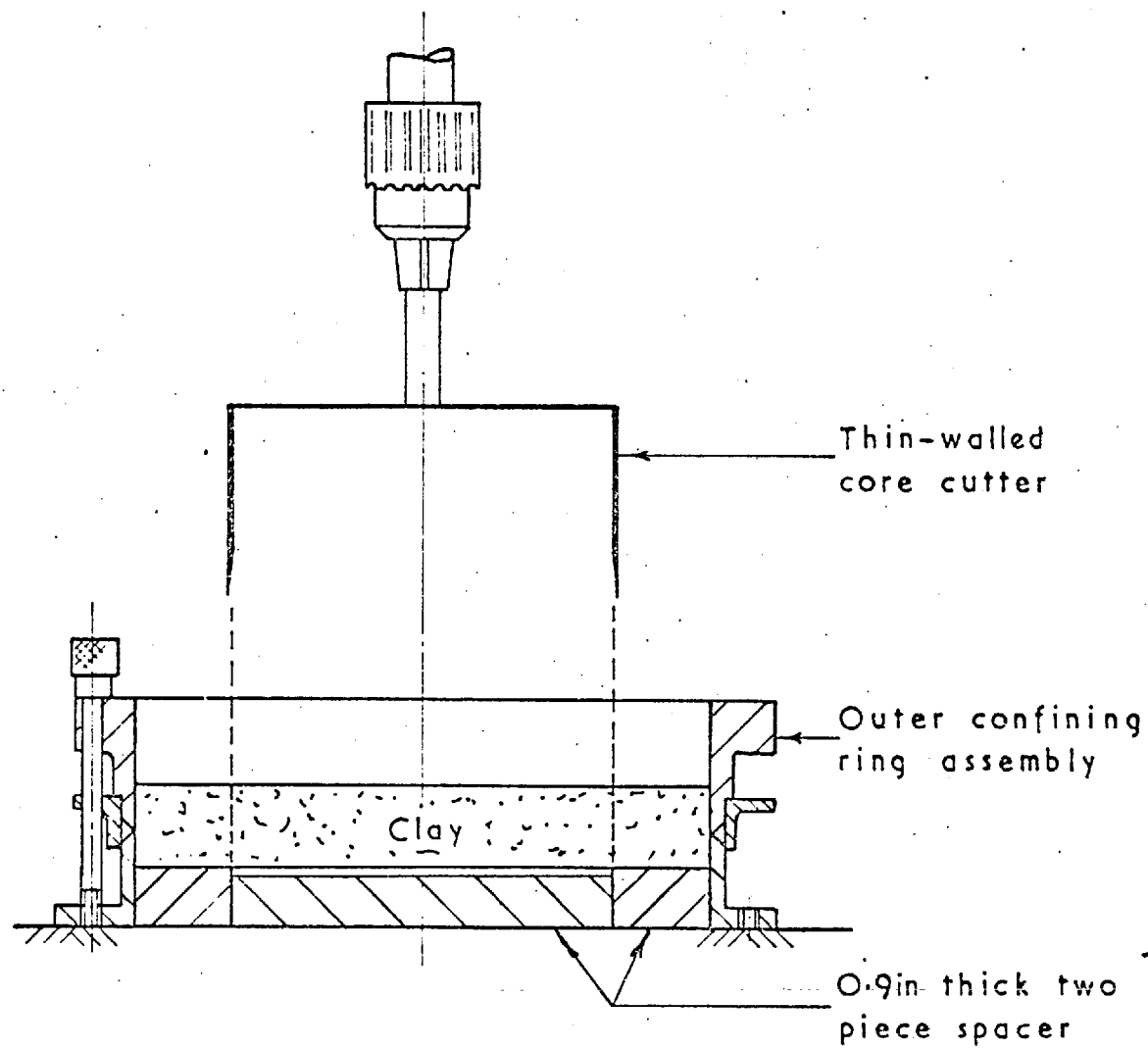
4. Assemble core cutter on outer confining ring assembly
5. Push clay into rings to rest against spacer (using drill press as guide)
6. Trim off upper surface of clay until flush with confining ring

FIG.3.1 (c)



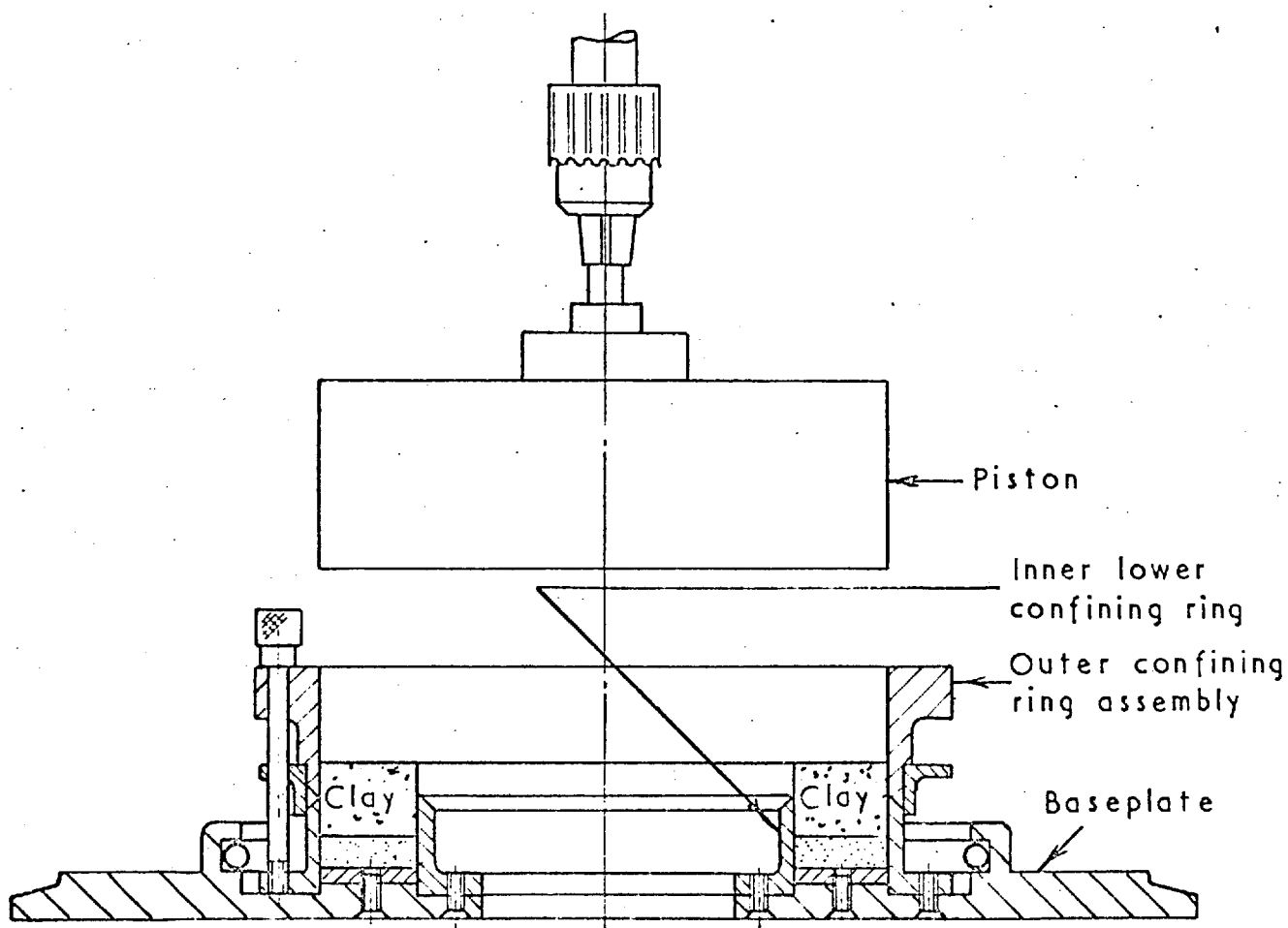
- 7 Remove core cutter and replace 1.7 in spacers by 0.9 in two-piece spacer
- 8 Push clay further into rings to rest against two-piece spacer

FIG.3.1(d)



9. Remove inner core with cutter,

FIG. 3.1 (e)



10. Preassemble lower annular loading platten with porous ceramic and inner lower confining ring on baseplate
11. Place outer confining ring assembly and sample on baseplate
12. Embed clay on to radial fins on lower porous ceramic
13. Insert upper inner confining ring (see Fig 2.3)

FIG. 3.1(f)

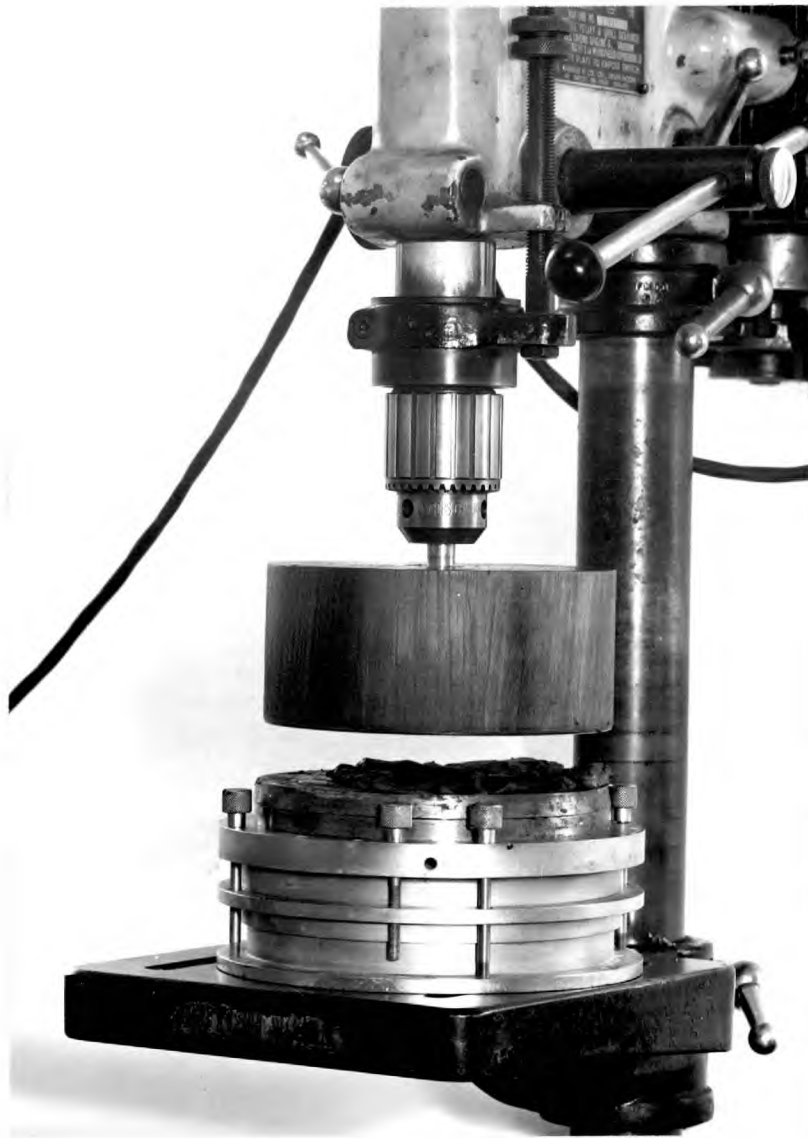


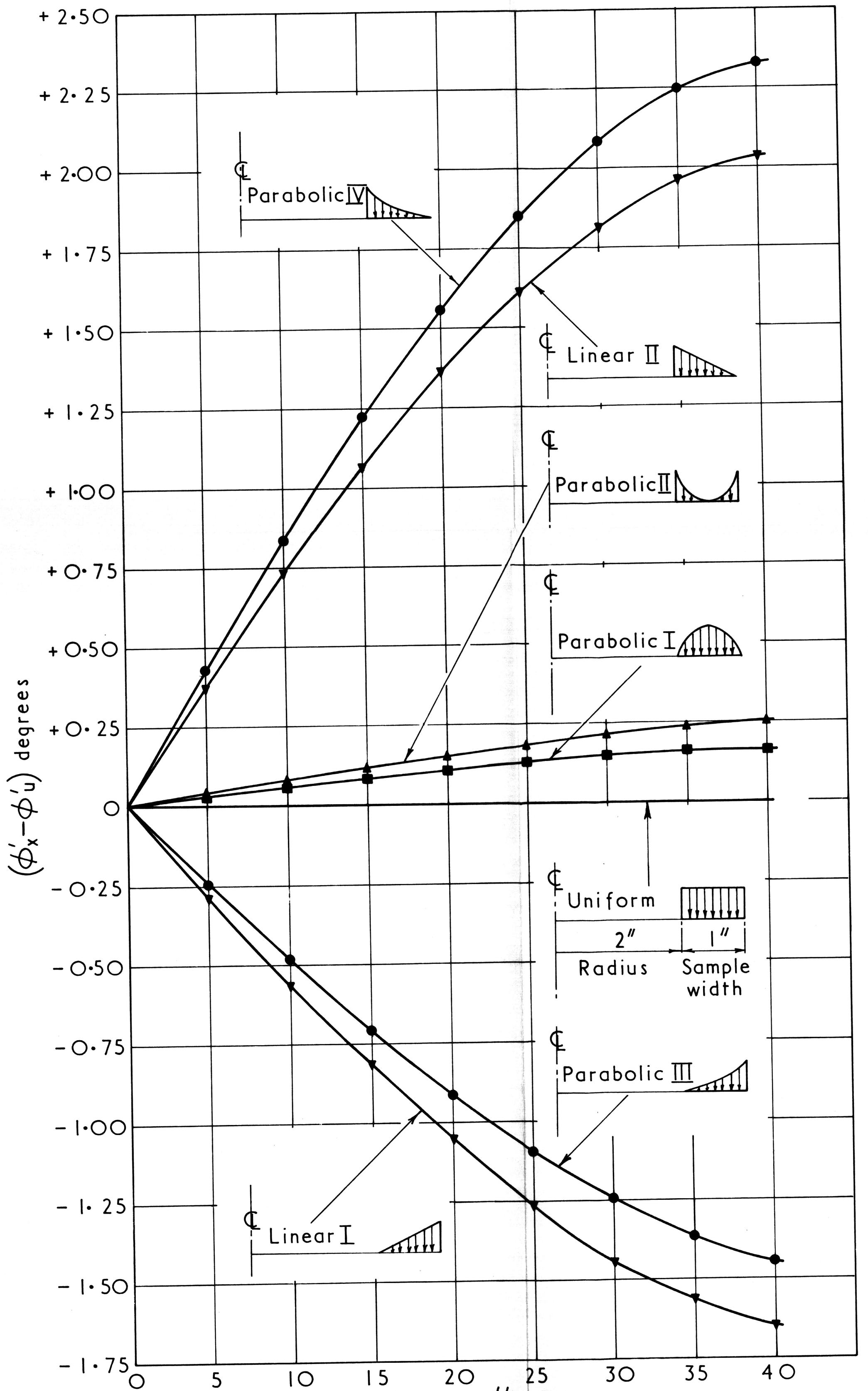
PLATE 3.1

Undisturbed clay in the cutting
ring before being pushed in the
sample assembly.



PLATE 3.2

Thin-walled core cutter removing
the core from undisturbed London
clay.



ϕ'_u and ϕ'_x : angles of shearing resistance under uniform and non-uniform stress distributions respectively.

CHAPTER 4

4.0 Ring shear tests on stiff clays

During the course of present study, the residual shear strengths of following clays were investigated in the ring shear apparatus:

- | | | |
|----|----------------------|-------------------------|
| 1) | Blue London clay | (Wraysbury) |
| 2) | Brown London clay | (Edmonton) |
| 3) | Weald clay | (Arlington) |
| 4) | Cucaracha clay-shale | (Panama Canal) |
| 5) | Dawson Formation | (Chatfield Dam, U.S.A.) |

The above mentioned clays can be conveniently studied under two categories; viz. plastic stiff clays from U.K. (clays 1 to 3), and the highly plastic clay shales from America. Hence for each of the two categories, the general geology of the areas, previous investigations and experiences, and test results and discussions would be mentioned separately.

4.1 Tests on stiff clays from U.K.

4.1.1 Brief review of previous work

The concept of analysing slope stability failures to investigate in-situ shear strength of clays just prior to failure is fairly recent. Terzaghi (1937) discussed the effect of discontinuities on shear strength of stiff clays and pointed out the dangers of progressive failure

if fissures and joint open out due to some form of stress release. Toms (1948) described a 60 ft. deep cut in Weald clay which failed 70 years after excavation, and showed that softening of clays was a time dependant process.

Skempton (1948) mentioned the softening effect due to presence of fissures in six slips in London clay. In all the cases, failure had taken place some time after the end of construction, and a tentative relationship of the decrease in shear strength of London clay with time was proposed. Cassell (1948) also described slips in overconsolidated fissured clays and attributed loss of strength to softening along fissures. He also made the important observation that laboratory tests on undisturbed samples would give a higher peak strength than that operating in the field due to the presence of fissures and the (now well recognised) effect of sample-size. He further demonstrated that moisture content on natural slip surfaces in stiff clays was invariably greater than in the adjacent unsheared material.

Henkel and Skempton (1955) investigated a shallow slip at Jackfield in the exposed Coalport Beds of the Upper Coal measures which, near the surface, are composed of stiff fissured clays. They found that the slip could be explained in terms of $\phi'_p = 21^\circ$ and $C' = 0$. Bishop and Henkel had experimentally demonstrated in 1953 with tests on overconsolidated, undisturbed Weald clay that the value of undrained cohesion dropped on cyclic loading; on softening due to the presence of opened fissures and

due to softening as shearing takes place. Henkel, (1957) described two long term slope failures at Wood Green and Northolt in Brown London clay, which he explained in terms of effective stress analysis using a $\phi'_p = 20^\circ$ and a reduced value of cohesion compatible with a factor of safety of one. A tentative curve for the decay of cohesion with time was also presented (assuming $\phi'_p = 20^\circ$), in which C' dropped from 250 lb/ft^2 to 70 lb/ft^2 in about 70 years. Skempton and DeLory (1957) suggested that slopes in brown London clay, greater than 10° would be unstable. This value would be consistent with ground water level at surface and $\phi' = 20^\circ$ ($C' = 0$) for shallow translatory slides. It would therefore appear that cohesion (as defined by Mohr-Coulomb shear strength envelope) is destroyed rapidly by shearing and also due to progressive weathering and softening of clay.* In this context it is interesting to note that Rankine, (1862) considered that cohesion "was destroyed by the action of air and moisture, and of changes in the weather" and that "friction is the only force which can be relied upon to produce permanent stability". The rate of weathering and softening of clay and hence the rate of decay of cohesion would depend on environmental factors and this is reflected in the varying times (after excavation or construction) at which slope failures occur for the first

*The same point was noted by Wilson and Johnson (1964) who found that there was negligible cohesion after any movement had taken place on horizontal bedding planes of the overconsolidated clays along the Seattle Freeway.

time in London clay. For example at Wembley Hill, the time to failure was 13 years, while at Upper Holloway, it took 31 years before a slip occurred (DeLory 1957).

Significant contribution was made by Skempton, (1964); Petley, (1966) and Skempton and Petley (1967) who reported a number of direct shear tests on natural slip surfaces, the results of which were compared with those from analysis of slip surfaces. The case of Walton's Wood slip involved multiple landsliding in weathered Upper Carboniferous Mudstone (Skempton and Hutchinson, 1969). The instability consists of two distinct parts - the lower, steeper part which has at least two rotational slides and an upper, less steep 9° slope containing multiple slides parallel to the slope. Since the profiles of the upper translatory slides have had to be approximated because of rotational slips in the lower part, the analysis of slides in the lower portion would appear to be more certain. The shear strength parameters as determined by multiple reversal direct shear tests and out plane triaxial tests were found to be:

$$\begin{array}{ll} \text{Peak:} & c' = 320 \text{ lb/ft}^2, \phi' = 21^{\circ} \\ \text{residual:} & c' = 0, \phi' = 13^{\circ}. \end{array}$$

The analysis of rotational slides in the lower region gave $\phi'_r = 15^{\circ} - 15.5^{\circ}$; $c'_r = 0$, while the upper slides (which were affected by the flow in the lower ones) required $\phi'_r = 13.5^{\circ} - 14^{\circ}$. Good agreement between field and laboratory values was claimed, although the more

reliable, lower portion yielded results which were about 20% higher than laboratory values. The statement "No particular significance is attached to this relatively small order of discrepancies" (Skempton and Hutchinson, 1969) is not justified in this case. Perhaps the results would imply that either the shear strength on the slip plane had not achieved residual state even though a highly pronounced slip plane had been formed or that the true residual shear strength envelope shows substantial curvature which was not evident from laboratory tests.

Skempton (1964) on an examination of shallow slips in natural as well as excavated slopes in brown London clay proposed a value of $\phi'_r = 15^\circ - 16^\circ$ ($C' = 0$). (The very shallow slips in natural slopes were reexamined by considering the groundwater table 2-3 feet below ground surface, which gave a lower value of ϕ' required for stability). An excellent example of a highly polished natural slip surface in brown London clay from Guildford was tested in the laboratory and gave average residual strength parameters of $C'_r = 60 \text{ lb/ft}^2$; $\phi'_r = 12^\circ$ (or $C' = 0$; $\phi'_r = 15^\circ$). (Skempton and Petley, 1967). It is worth noting that that shear strength envelope at residual showed marked curvature and that the residual values obtained from cut plane triaxial tests were invariably lower than those obtained from direct shear box tests. Curved shear strength envelopes were also obtained from tests on slip surfaces in the Atherfield and Weald clays, but no significance was attached to this. The results of residual

shear strength parameters on natural slip surfaces previously found have been summarised in Table 4-1.

Hutchinson (1967) on the basis of morphological evidence suggested that slopes greater than 8° in brown London clay would be unstable. This would be consistent with ϕ'_r lying between 11° and 15° depending on the depth of ground water conditions. Other work on residual shear strength of overconsolidated clays from U.K. has been described by Agarwal, (1967); Hutchinson, (1969) and James, (1970).

4.1.2 Brief geology of London clay at Wraysbury

At Wraysbury, near Staines, London clay overlies approximately 130 ft of Reading Beds mainly composed of purple and grey mottled clays of estuarine origin. The depth of London clay is also about 130 ft at this site. London clay is of marine origin except for the Basement Bed which indicates a transition from estuarine conditions and therefore indicates the beginning of submergence of land. After the sedimentation had continued to a depth of about 100 fathoms (Burnett 1968), there was an uplift and considerable erosion took place. The present clay lies about 45 feet about O.D. London clay at Wraysbury is overlain by 10 - 15 ft of flood plain gravel over which lies a 1 - 4 ft layer of alluvium consisting of organic soil and loam. The magnitude of preconsolidation pressure, P'_c , at Wraysbury has been disputed. At the Ashford Common site (close to Wraysbury), Bishop, Webb and

Lewin (1965) estimated, P'_c between 39 - 45 T/ft², which corresponds to a removal of about 1200 ft of overburden; though Fookes (1966) estimated P'_c to be approximately half that of the above experimental value from geological evidence. It is interesting to note that Som (1968) estimated P'_c to be about 40 T/ft² at this site on the basis of tests in constant rate of strain oedometer. Because of the high degree of overconsolidation, the clay shows insitu horizontal stresses much larger than the vertical stresses (Skempton, 1965; Bishop et. al. 1965).

The area has experienced tectonic activity in the past and the folding and faulting is probably a result of this activity, (Fookes 1967). At Wraysbury a uniformly horizontal shear zone, about 6 in. wide was discovered at an average depth of 6 - 7 feet from the top of clay stratum.

4.1.3 Tests on Blue London clay (Wraysbury)

Four tests on undisturbed clay and one on a remoulded sample were conducted in the ring shear apparatus. The samples were prepared for testing as described in chapter 3. The samples were taken from clay pits at a depth of about 10 ft from the top of the clay stratum.

Graphs of stress/displacement relationships have been plotted in figures (4-1 - 4-5). The strength is shown in terms of stress ratio τ/σ'_n and ϕ' degrees (taking $\phi' = \tan^{-1} (\tau/\sigma'_n)$). Test RS1 was remoulded at an

average moisture content of about 43%. The sample was subjected to a mean displacement of about 6.7 in., at an average normal stress of approximately 27 psi. It will be observed from fig. 4-1 that the shear strength decreases rapidly immediately after the peak shearing resistance has been overcome. The readings are normally taken with a small gap between the confining rings. However, it has been observed in all tests that the gap once opened, does not necessarily remain in this position as shearing proceeds. The development of the slip surface often results in uneven deformation of the sample and as a result the confining ring gap may not be entirely open. This effect would introduce a frictional component if metal to metal contact occurs at any point and the shear strength would show an apparent increase. This effect is demonstrated by closing the confining ring at a displacement of 5.4 in., which results in an apparent increase in shear strength. The torque drops to the true value when the confining ring gap is reopened as shown in fig. 4-1. Hence it is recommended that the confining ring gap should be enlarged periodically to ensure that side wall frictional effects are eliminated. It was also observed in the test that widely differing torque forces were registered on the side proving rings. This is partly due to the fact that the two proving rings are of unequal stiffness. This effect can be eliminated by placing a shim of suitable thickness between the torque arm and the wheel of the proving ring so that equal force is registered on both rings. This has been shown in the

figure at a displacement of 5.38 in. However, it would be observed that shear strength remains unaffected by the action of placing a shim. It is also worth noting that the remoulded clay shows a peak strength which is very significantly higher than the residual shear strength.

Test No. UDLC-RS.2, (fig. 4-2) was done on undisturbed clay which was subjected to a mean displacement of about 52.3 in. at an average normal stress of about 29 psi. The effect of closing and opening of the gap between confining rings and the consequent change in shear strength is again clearly evident from this figure. In order to investigate whether the London clay is capable of "healing", the test was stopped for two days. No gain in strength was noticed when shearing was commenced again. This clearly shows that London clay is not capable of regaining shear strength once shearing is terminated. This phenomena is reported to have occurred in some clays; Evorslev, (1960) for example, found that remoulded Vienna clay increased in strength by 20% of the residual strength when the ring shear test was stopped for 82 minutes.

The slow speed at which the test is normally conducted makes the test very time consuming if large displacements are required. However there is some evidence (Petley, 1966; Tchalenko, 1967; Kenney, 1967) that residual strength is relatively insensitive to rate of shearing and is mainly dependant on the magnitude of

displacement. Hence in this test, the confining ring gap was closed and the rate of shearing was increased, in order to decrease testing time. After the desired deformation had taken place, the gap was reopened and the speed was reduced to its original value. Usually 24 hours were allowed for any pore pressures to dissipate, before any readings were taken. It can be seen from fig. 2-2 that residual strength is dependant on the magnitude of displacement on the slip-surface. However, negligible drop in strength was noticed between average displacements of 16.6 to 52.2 in. and this constant value of shear strength gives the residual shear strength.

Tests UDLC-RS3; - RS4, - RS5 were conducted at normal stresses of about 11.6,^{5.9} and 41 psi. respectively. It should be obvious that if the confining gap is open on all sides, then any further enlarging of the gap should not affect the shear strength of the clay. This is indeed so, as shown in fig. 4-4 and 4-5.

The test results have been summarised in Table 4-2 and residual strength envelope drawn in fig. 4-6, together with other shear strength data on blue London clay from Wraysbury. A straight line can be drawn through the test results from which the residual shear strength parameters are found to be $C' = 0$; $\phi'_r = 5.2^\circ$. The index properties of the clay and moisture contents are also shown in Table 4-2.

Generally, the moisture content in the shear zone for the undisturbed samples is higher than the overall

final moisture content and this difference between the two tends to decrease with increasing normal stress. All undisturbed samples showed an increase in moisture content at the end of testing while the remoulded sample showed a decrease. The increase in moisture content in the shear zone has been noted by Cassell (1937). Agarwal (1967) observed that the moisture content of clay in the shear zone at Wraybury was 30 - 32.5% as compared with moisture content of about 27.5% for the unsheared material. Large variations in moisture content exist in undisturbed London clay (see also chapter 10). This initial variation is also reflected in the values of moisture contents at the end of the test and it should be noted that only average moisture contents are shown in Table 4.2.

The slip surfaces were photographed at the end of the tests. Plates 4-1(a) and (b) show typical striations and polished surfaces from tests RS1.

It is worth pointing out that lower the normal stress, the more difficult it was to separate the sample along the slip surface. This is due to high moisture content of the sample at the end of the test. In most cases, apart from the highly polished principal slip surface, a few other slip surfaces about 0.5 - 1.0 mm above or below were also found. The slip surfaces, even when highly polished, were not completely flat but wavy in character across the width of the sample. Some details of shear zones examined under optical microscope have been discussed in chapter 5.

4.1.3.1 Discussion of results

The value of $\phi'_r = 9.2^\circ$ for blue London clay obtained from tests in the ring shear apparatus is clearly much lower than the value obtained by Petley (1966) and Agarwal (1967) by conducting direct shear box tests (see Table 4.3). It implies that direct shear box is not a definitive test for measuring residual shear strength, since it does not permit the sample to undergo large unidirectional displacement. The reversing of the travel of the box does not permit the residual state to be achieved, since it changes the direction of the principal stresses and disturbs the orientation of the clay particles.

It is of interest to consider tests conducted by Agarwal (1967) in some detail since the samples were obtained from the same site and have almost identical physical properties to the samples used in ring shear apparatus. He conducted direct shear box tests on undisturbed horizontal and vertical samples under normal stresses lying between 7.5 psi. and 90 psi. for which average ϕ'_r of 14.5° and 16.8° respectively were obtained. It is most unlikely that such different residual strength values would be obtained for vertical and horizontal samples sheared to large displacements in the ring shear apparatus. Figs. 4.7 - 4.10 show a few typical multiple reversal tests on horizontal samples conducted by Agarwal. Fig. 4.7 and 4.8 show that the sample had not achieved residual state since the strength after a number of reversals had not achieved a constant minimum value.

The unsuitability of direct shear box test especially at high normal stresses is ably demonstrated by figs. 4.9 and 4.10, where on reversing, the shear strength continues to increase (instead of reducing) as displacement proceeds. This effect was also noticed by Petley, (1966) and is believed to be caused by tilting of the loading platten, and variations in the area of cross-section of the sample. The increase in apparent shear stress may also be caused by squeezing of soil through the gap. Agarwal tested a natural slip surface found at the site by the reversal shear box technique and found $\phi_r' = 8.3^\circ$ and 10.2° at normal effective stresses of 15 and 30 psi. respectively; the residual shear stress being taken as that at the end of the second travel. It is interesting to examine the test curves for these tests (fig. 4.11 and 4.12) and note that in both cases the strength obtained after the first reversal was slightly higher than that obtained on the first travel of the box. This clearly implies that the reversal action had, in fact, disturbed the particle orientation along the slip surface. If the residual strength is considered to operate at the end of first travel, then the average value of ϕ_r' is found to be 9.2° , as shown in fig. 4.6. This value is the same as that obtained from tests in the ring shear apparatus.

Bishop et. al. (1965) briefly reported polished cut plane triaxial tests on blue London clay from Ashford Common, near Wraysbury. The polished, inclined cut planes were artificially prepared in the laboratory and the

tests were conducted under effective normal stresses of 64 and 130 psi., for which the values of $\phi'_r = 11.6^\circ$ and 11.5° respectively were obtained. This technique orients the particles to much larger extent than the direct shear box test and this accounts for the lower value of residual shear strength. There is some evidence to suggest that even artificial polishing of slip surfaces by means of a flat spatula or plate glass does not permit maximum particle orientation. Unpublished work at the British Ceramic Research Association shows that maximum orientation of clay particles is likely to take place when the clay itself is sheared (i.e. if clay is 'polished' by another piece of clay). Hence a manual ring shear apparatus was designed in which a slip surface could be artificially prepared by shearing the clay through very large displacements very rapidly. The apparatus is only briefly described below.

The apparatus was very simple and consisted of an upper and lower cylinder, 11.70 in. I.D. and 12.0 in. O.D. and 4 and 5 in. high respectively. The lower cylinder can be fixed to a base plate, the upper surface of which is roughened with very coarse abrasive paper so that it can grip the soil sample. An upper loading platten, $1\frac{1}{4}$ in. thick, also roughened on the inside, rests on sample in the upper confining ring. Three L-shaped steel lugs are welded on the upper loading platten and rest against corresponding steel bracket stops welded on the outside of the upper confining ring, such that when the platten

is rotated, the upper confining ring and the platten move as a unit. The platten can be rotated manually. Only one test was conducted, since the apparatus was constructed at the end of the project. An undisturbed sample of blue London clay which had already been consolidated to 60 psi. in 12 in. diameter dissipation cell was sheared through 12 revolutions under an average normal stress of 55 psi. The load was applied by the crosshead of the 100 ton loading frame. The apparatus was then fixed to the working surface of a milling machine and three, thin walled, $1\frac{1}{2}$ in. diameter, brass tubes were pushed in at an inclination of 55° to the horizontal. The samples, containing the inclined shear plane were then sheared under drained conditions in a triaxial apparatus in the conventional manner, using one way drainage and carefully placed filter drains. Unfortunately, of the three tube samples taken, one was destroyed on extruding; while the second failed along two wedges which made the analysis extremely difficult. Hence, the result from only one test is available. The result of this test has been plotted in terms of stress ratio τ/σ'_n along the inclined plane and axial displacement in fig. 4.13. Assuming $C' = 0$; $\phi'_p = 14^\circ$ and $\phi'_r = 10.5^\circ$.

The peak strength in the above test is probably developed as a result of disturbance during sampling, extruding and setting-up operations. The value of ϕ'_r compares well with those obtained from ring shear tests. It is suggested that more such tests should be done to explore the use of this technique in lieu of ring shear tests.

Perhaps the most convincing evidence to show that multiple reversal direct shear tests do not furnish true values of residual shear strength is provided when stress ratio, τ/σ'_n is plotted against logarithm of displacement. ~~Residual shear strength, by definition, operates when there is no further drop in strength on shearing.~~ Residual shear strength, by definition, operates when there is no further drop in strength on shearing. However this definition is invariably not fulfilled by direct shear tests. To illustrate this point, the result of a multiple reversal direct shear test on brown London clay from Hendon is shown in the form of shear stress against displacement graph in fig. 4.14 (a). The stress-displacement curve is an enlargement of fig. 19 in Skempton's (1964) paper and shows that the curve appears to achieve a constant value. However, when the same information is plotted on log displacement scale as shown in fig. 4.14 (b), the curve shows considerable slope and it cannot be considered as having reached the residual state. Hence visual examination of τ (or τ/σ'_n) against displacement plot is unreliable and a plot of τ/σ'_n against log displacement should be drawn to confirm whether residual strength has been obtained. Figs. 4.15 - 4.17 show the curves from ring shear tests drawn on such a plot, and it will be noticed that in each case the slope of the curve finally becomes horizontal, thus confirming that residual shear strength has been achieved in these tests. In fig. 4.12 the points from multiple reversal direct shear tests on both undisturbed samples and slip surface are also plotted.

The direct shear test points for the undisturbed sample clearly show that a very erratic curve is obtained, which does not satisfy the definition of residual strength at all. The successive direct shear curves are seen to increase in shear strength as displacement proceeds and there can be no justification for using these curves to evaluate residual shear strength. The curve from test on the slip surface from Wraysbury, fig. 4.16, is almost horizontal and indicates that the value is close to the residual. It would also be noted that the effect of the first reversal on the slip surface test is to cause a disturbance in the orientation of particles and the second travel starts at a higher shear strength than that obtained at the end of the first travel. Fig. 4.17 again clearly demonstrates the points mentioned above. The points from direct shear tests are again erratic and cannot be used to define residual shear strength.

4.1.4 Tests on Weald clay (Arlington)

The undisturbed sample of Weald clay was obtained from a shear zone at Arlington near Eastbourne. Weald clay, which was deposited in shallow water is stiff to hard, fissured, and heavily overconsolidated. The upper portion - up to 30 ft is usually weathered. The Weald outcrop has been subjected to intensive periglacial activity during the pleistocene which has resulted in old landslips, valley bulging, solifluction sheets etc. Weald clay is overlain along the northern and southern

outcrop by Atherfield clay which is the lowest member of the Lower Greensand. The sample from the shear zone consisted of a mass of tiny lenses of clay, typically 5 mm x 5 mm x 1 mm thick with a dull polish on the surface. The texture was similar to damp soap flakes. The lenses lay flat and parallel to one another.

Stress-displacement curve

The stress displacement curve is shown in fig. 4.18 and the results are tabulated in Table 4-4. The first point to note is that the test was conducted by shearing the same sample under different normal stresses. After the residual strength under one normal stress had been determined, the motor was stopped and the new normal stress was applied. The sample was then allowed to consolidate (or swell) under the new load for at least 48 hours before shearing was recommenced. This technique, in effect, amounts to conducting a shear test on a preexisting slip surface.

It would be apparent from the tabulated results that the sample shows marked variation in residual strength as the effective normal stress is altered. The variation of residual strength is more clearly illustrated in fig. 4.19 where ϕ'_r changes from about 9.2° at normal stress of 120 psi. to about 13.9° at normal stress of 8.9 psi. A small peak strength is observed when the sample is sheared under a reduced normal stress. There is very little

change in height of the sample when the residual state is achieved. Shearing then takes place under almost constant volume conditions.

4.1.4.1 Discussion

The observation that residual shear strength of some clays varies with effective normal stress is not new and has been noted by others previously. Tiedemann (1937) found that ϕ'_r of plastic clay from Weser Elbe Canal changed from 12.3° at normal stress of about 15 psi. to 8.3° at about 60 psi (see Table 1-1). Skempton and Petley (1967) carried out some multiple reversal direct shear box tests on weathered Weald clay from Sevenoaks and found that ϕ'_r varied from 16.5° for effective normal pressures less than 30 psi. to about 13° for higher pressures and they averaged ϕ'_r to 15.5° in the stress range of 0 - 90 psi. Some direct shear box tests on shear zone from Arlington were conducted by Petley, (1969, unpublished) and typical stress/displacement curves are shown in figs. 4.20 - 4.22, which show that under higher normal pressures the shear strength increases as displacement proceeds. It is also of interest to note that in fig. 4.21 the shear strength on the fourth travel was higher than at the end of the second and third travels thus again showing that the action of reversing the box can result in serious disturbance of particle orientation. This aspect is highlighted if ϕ' is plotted against logarithm of displacement, as explained previously. Figs. 4.23 - 4.25 show that all the ring shear tests achieve residual condition

since the slope of τ/σ'_n against log displacement curve becomes zero. However this is not so for direct shear box tests and erratic curves are obtained at higher normal pressures. Since the curves for direct shear box tests show significant slope at large strain, they cannot be considered as giving a residual value. It should be noted again that, it is very difficult to judge visually from a conventional $(\tau/\sigma'_n)/\text{displacement}$ plot whether residual strength has really been achieved and it is necessary to draw τ/σ'_n against log displacement to confirm this. The shear strength envelope for Weald clay is shown in fig. 4.26, and the best fitting straight line through ring shear results gives $\phi'_r = 9^\circ$; $C'_r = 1$ psi in the normal stress range 10 - 120 psi and $\phi'_r = 15^\circ$, $C'_r = 0$ for σ'_n lying between 5 - 10 psi. Hence the results from ring shear tests are considerably below those from direct shear test at normal stresses greater than 10 psi. It is also interesting to note that the peak strength of the clay from shear zone is considerably larger than the residual strength despite the fact that the shear zone was heavily sheared and slickensided. It is most likely that the strain in the field was insufficient to reduce strength of the whole clay mass in the shear zone to residual although intense local straining might have brought some parts of it very close to residual value. This is also borne out by observing the degree of orientation in a section of the natural shear zone (see chapter 5) where wide range of particle orientation - from intense to slight (according to semi-qualitative scale proposed by Morgenstern and Tchalenko, 1967) - was observed.

The implications of a curved shear strength envelope are discussed more fully in the next section together with results from brown London clay which also displayed similar characteristics.

4.1.5 Tests on brown London clay (Edmonton)

Two test, one each on undisturbed and slurried brown London clay were conducted.

The sample was obtained from a site adjacent to the Banbury reservoir on the River Lea. The undisturbed sample was taken from a depth of 3.5 ft from the top of the clay. Numerous slickensides and shear planes have been found on this site. This is an evidence of instability in the past, probably caused as a result of valley erosion and undercutting by River Lea (Hutchinson 1969).

The slurried sample was prepared by first air-drying the clay which was then ground and sieved through B.S. sieve 100. The material passing this sieve size was homogenised in a rotary mixer and the water content was brought up to 160%. The slurry was poured in 9 in. diameter oedometer mould and consolidated to 250 psi. The sample was then unloaded and allowed to swell.

4.1.5.1 Test results and discussions

The stress displacement curves for the two tests are shown in figs. 4.27 and 4.28. The tests were again conducted by shearing the same sample under different normal stresses.

Fig. 4.27 shows that when the sample was first sheared under effective normal stress of 31 psi. a mean displacement of about 14 in. was required to bring it to residual state. When the sample is sheared under a reduced normal load, a small peak strength is first observed and the difference between the 'peak' and residual strength appears to increase as the effective normal stress decreases. There are two possible explanations for this behaviour. Firstly this effect could be a function of the test itself - when the motor is stopped at the end of the shearing stage, and the normal stress is decreased, there is little corresponding reduction in the shear stress because of the rigid torque measuring system. Hence soon after shearing is recommenced, the sample shows an apparent peak strength, which soon drops. It is probable that under low normal stress, the number of clay particle contacts on the slip surface decreases, and as a result peak strength is observed because of the irregular nature of the slip surface. Hence the smaller the load, the larger the displacement that would be required to overcome peak strength and achieve residual state. This effect is clearly illustrated in fig. 4.27. The stress displacement curve, fig. 4.28, for the remoulded sample shows that an average displacement of 6 in. was required to achieve residual shear strength at an effective normal stress of 13 psi.

The results have been tabulated in Tables 4-5 and 4-6 and the variation of ϕ_1' with effective normal stress, has been plotted in fig. 4.29. It is of interest to note

that this variation forms an almost unique curve and irrespective of the sequence of loading, the residual strength falls along the same curve. A similar unique curve was also obtained for tests on Weald clay (fig. 4.19). The residual shear strength for the first stage for undisturbed sample (shown as point no. 1 on fig. 4.29) lies 0.5° above the curve. No particular significance is attached to this discrepancy and it is most likely that after much larger displacement the strength would have fallen by this small amount. Fig. 4.29 also shows the results of test on slurried sample which lie about 0.5° above those of undisturbed sample although the slurried sample has a 10% higher initial moisture content. The small difference of 0.5° degree in the value of ϕ_r' could be caused by alteration of soil or pore water chemistry as a result of air drying. It is also possible that under certain wet moisture contents, some non-clay minerals (and perhaps the clay minerals as well) rotate rather than slide (Skinner 1969) thus giving a slightly higher value of ϕ_r' . This effect has also been noted by Petley (1964) in tests on brown London clay from Hendon. The value of $\phi_r' = 14^\circ$ for overconsolidated slurried sample was 1° higher than that for the normally consolidated slurried sample. The important conclusion to be drawn is that residual strength is independent of stress history since the points fall on a unique curve dependant only on the magnitude of effective normal stress.

It is not known why the brown London clay (and the Weald clay) shows a variation of ϕ_r' with normal stress whereas the blue London clay does not. It appears from the work of Kenney (1967) that soils with high montmorillonite content show such a tendency while soils composed of heavy non-clay minerals do not. X-ray diffraction of the brown and blue London clay shows that the former has a higher montmorillonite content and this may account for the variation in ϕ_r' . However without a detailed study of clay mineralogy of the sample, any suggestion must remain highly speculative.

The residual strength of brown London clay tested varies from 14° at effective stress of 1 psi. to 8° at 36 psi. It was earlier mentioned that Langer (1938) obtained a value of $\phi_r' = 8.8^\circ$ at normal effective stress of 90 psi. in tests conducted in his ring shear apparatus. Skempton and Petley (1967) tested slip surfaces from Guildford in direct shear box and reported a curved residual strength envelope, but no significance was attached to this. Petley (1966) conducted a number of multiple reversal tests on brown London clay from Hendon, Herne Bay, Brentwood and Guildford from which an average value of ϕ_r' lying between 13° and 15° was obtained. These results have been summarised in Table 4-7. The residual shear strength envelopes from direct shear box tests and ring shear test are plotted in fig. 4.30 (a) which shows that above a normal stress of 5 psi., the residual strength as given by the ring shear tests is at

least 4° below the value obtained from direct shear tests. The residual shear strength envelope from ring shear tests at low effective stress has been shown in greater detail in fig. 4.30 (b).

The variation of stress ratio, τ/σ'_n against log. displacement is shown in figs. 4.31 and 4.32, which again clearly show that the slope of curves for ring shear tests becomes zero, thus confirming the attainment of residual state. The test points from direct shear tests on brown London clay from Guildford and Edmonton (Petley 1966) have also been shown for comparison. The Guildford slip surface tests show that the shear strength increases with displacement during any travel, and this makes the analysis extremely difficult. This invariably seems to occur when stiff clays are sheared in direct shear box under a normal stress greater than 15 psi.

4.1.6 The significance of the variation of ϕ'_r with normal stress

Most of the previous work on slope stability of stiff clays has been done on the assumption that residual shear strength was independent of stress level. Hence, because the multiple reversal direct shear box tests gave a value of ϕ'_r consistent with some shallow slope failures, it has been suggested that this test is adequate for the determination of residual shear strength of stiff clays. However, it should be noted that the majority of shear box tests are conducted at an effective normal stress

which is invariably greater than the average normal stress acting on the slip surfaces in the field in the cases examined, and therefore it cannot be implied that this test gives accurate values of ϕ'_r . It has already been shown that at low stresses below 5 psi. there is very little difference in the value of ϕ'_r for brown London clay obtained from the ring shear tests and the shear box tests conducted at higher normal stresses (fig. 4.30 (a)). A significant difference on ϕ'_r arises at higher normal stress when the direct shear test overestimates the value of ϕ'_r as a result of insufficient displacement, and the development of erratic stress pattern which increases as displacement proceeds. The author is not aware of any published investigation of deep seated slides in brown London clay or Weald clay which show a correspondence with residual shear strength as obtained from direct shear box tests. Most slides in brown London clay which have been investigated have a very low normal effective stress. Table 4.8 prepared from DeLory's (1957) Ph.D. thesis shows some typical effective normal stresses on natural slopes in London clay whose stability has been investigated.* The normal stress on failure surface would depend on the assumptions about ground water table and the method of analysis (a circular failure would show a larger normal stress than plane failure). It is interesting to note from Table 4.8 that the oldest cases have the least normal stress on the failure surface. This in itself is an indication of the variation of residual shear strength on effective normal stress.

*Other similar data has been shown in Table 8 (a), compiled from James' Ph.D. thesis (1970).

It was mentioned earlier that Skempton and DeLory (1957) suggested a slope angle of 10° for natural slopes in London clay above which they would be unstable. Many natural slope failures are very shallow (under 10 ft deep) and the failure surface is long and runs parallel to the ground surface. For such type of translatory failure.

$$F.S. = \frac{(\gamma_t - m\gamma_w) \cos^2 \beta \tan \phi'_r}{\gamma_t \sin \beta \cos \beta} \quad (\text{assuming } C'=0) \quad (4.1)$$

where γ_t denotes unit weight of soil,

γ_w denotes unit weight of water,

m denotes ratio of height of water table to the

height of ground surface above the slip surface,

and β denotes inclination of slope.

ϕ'_r for limit equilibrium can be calculated from the above equation. For ground water table at surface, a slope angle of 10° would approximately correspond to $\phi'_r = 20^\circ$. Skempton (1964) reappraised natural slope failures on the basis of ground water table lying about 2 - 3 ft below ground surface. The slope angle of 10° is now consistent with $\phi'_r = 15^\circ - 16^\circ$. Hutchinson, (1967) suggested that natural slopes under 8° would be stable, for which, depending on the assumptions regarding ground water table, the residual angle of shearing resistance would lie between 11° and 15° . It would therefore appear that assumptions regarding ground water table have such a enormous effect on the value ϕ'_r (from 20° to 11°) that this method of determination of residual shear strength is of little value. Also, eq. 4-1 is sensitive to the

value of γ_t used. The factor of safety operating in the field would invariably be higher than that determined accurately from the ring shear apparatus and a conventional stability analysis because of three dimensional effects in the field (Kenney 1956), and non-uniform stress conditions along this slip surface (Bishop, 1967; Turnbull and Hvorslev, 1967, see chapter 7). Hence the author suggests that for very shallow slope failures in brown London clay (and most natural slope failures fall in this category) which fail during or soon after the rainy season, the actual ϕ'_r operating at the slip surface would be nearer 14° which is consistent with ring shear tests in very low stress range.

It is now apparent that brown London clay and Weald clay would show a higher ϕ'_r only under low normal stresses. If a failure in the field under high normal stresses is investigated, then it is reasonable to expect it to be compatible with the low values of residual angle of shearing resistance as obtained by tests in the ring shear apparatus.

It is of interest to note the re-analysis by computer of the Guildford slip in brown London clay, recently made by Dr Chandler, whose results are summarised overleaf.

Parameters for $F = 1$ (Morgenstern and Price Method)(Average $\sigma'_n = 10$ psi)

Condition	Piezometric surface	C'_R lb/ft ²	ϕ'_R
Before slip	Estimated from 1966 readings	60	12.4°
		0	14.6°
After slip	Measured Feb. 1966 readings	60	8.0°
		0	10.5°
	at ground surface	60	9.4°
		0	12.2°

It will be observed, that the residual value dropped from $\phi'_R = 14.6^\circ$ to $\phi'_R = 10.5^\circ$ after the slip while the multiple reversal direct shear tests on slip surfaces gave $\phi'_R = 15^\circ$, $C'_R = 0$ (Petley 1966). This indicates, firstly, that before the slip, the strength was not at residual but dropped to it after the slip and indeed, may perhaps still drop further by a small amount since signs of fresh movements have been noted in the past. Secondly, the multiple reversal direct shear tests overestimated the residual value mainly due to insufficient displacement. This effect may also be partially due to squeeze of clay through the gap. These limitations would have been removed in a ring shear test and a more satisfactory agreement between analytical and laboratory results would be found if such a test is conducted at the appropriate normal stress. It is interesting to note that the ring

shear test on brown London clay gives a value of $\phi_r' = 9.5^\circ$ at an effective normal stress of 10 psi.

Hutchinson (1969; private communication) investigated the stability of a site in brown London clay at Edmonton and found considerable variation of ϕ_r' with maximum depth (i.e. normal stress) of potential failure surface. For example, a long translatory slide surface running parallel to the slope at a depth of about 15 - 18 ft required $\phi_r' = 14.5^\circ$ while a circular failure arc at a maximum depth of about 40 ft required $\phi_r' = 9.3^\circ$ for a factor of safety of one. Other potential slip surfaces show similar trends and substantiate the results from ring shear tests. Unfortunately, it is not possible to define the slip surfaces very accurately because of insufficient number of trial pits.

Weeks (1969) has recently reported a number of extensive slip surfaces which have been found in slopes inclining between 3° and 7° in stiff clays. The main results of his observations on long translatory slides are listed in Table 4.9.

Table 4.2 (after Weeks 1969)

Clay	Location	Slope inclin- ation, degrees (β)	length of slip surface ft.	ϕ'_r as deter- mined by conven- tional tests	$\phi'_r = 2\beta$ assuming G.W.L. at surface
Weald clay	Sevenoaks By-pass	7°	1200	$C'_r = 0.2$ psi $\phi'_r = 15^\circ$	14°
Weald clay	Sevenoaks By-pass	4°	-	$C'_r = 0$ $\phi'_r = 15.6$	8°
Weald clay	Tonbridge By-pass	4°	1500	$C'_r = 0.3$ psi $\phi'_r = 16^\circ$	8°
Wad- hurst clay	Tongridge By-pass	7°	-	$C'_r = 0$ $\phi'_r = 12.4^\circ$	14°
Wad- hurst clay	Tonbridge By-pass	4°	1000	$C'_r = 0$ $\phi'_r = 12.4^\circ$	8°
Gault clay	Wrotham Heath	3°	-	$C'_r = 0$ $\phi'_r = 12.7^\circ$	6°
Brown London clay	Boughton	5°	-	$C'_r = 0$ $\phi'_r = 14^\circ$	10°
Gault clay	Tetsworth	3.5°	800	$C'_r = 0$ $\phi'_r = 14^\circ$	7°

Weeks has sought to reconcile the very low inclination of slip surfaces as found in the field and the values of residual shear strength as determined in the laboratory by means of multiple reversal direct shear tests and cut-plane triaxial tests, by suggesting that these failures took place when very high pore pressures were induced due to periglacial conditions. "For the 3° and 4° slopes investigated, the pore pressure required for instability to occur would be approximately equivalent to $0.54z$ and $0.31z$ respectively above ground level, i.e. 5 ft 6 in. and 3 ft for a 10 ft deep slip surface" (Weeks 1969) (where z is the depth of slip surface below ground level). However, the author suggests that it is not necessary to invoke the condition of abnormally high pore pressures to explain these results. It should be noted that in the equation the factor of safety is highly susceptible to the value of the total unit weight of material lying above the slip surface. Weeks (1969) has assumed a value twice that of unit weight of water. Infact, it is highly probable that the unit weight, γ_t , was much lower at the time of the slip than at present. The solifluction lobes flowed laterally downwards at the end of the glaciation periods when the water content of the material must have been very high. The total unit weight, γ_t , could have been of the order of 100 lb/cu ft especially as the fresh solifluction lobe carried the coarse detritus material from the solifluction lobe of the previous glaciation period. Bhandari (1970) on a study of mudflows in South East England has found γ_t to be even lower than 100 lb/cu. ft. for London clay at a water content of about 50 - 60%.

Assuming γ_t to be equal to 100 lb/ft^3 and ground water at surface, eq. (4.1) yields:

$$\tan \phi'_r = \tan \beta / 0.375. \quad (4.2)$$

Hence for $\beta = 3^\circ, 5^\circ$ and 7° , $\phi'_r = 8.3^\circ, 13^\circ$ and 16° respectively.

In all these cases the exact magnitude of effective normal stress on the slip surface at the time of failure is uncertain and it is not unreasonable to expect the above range of values of ϕ'_r depending on normal stress.

It was mentioned earlier that if residual shear strength for a clay is stress dependant, then the analysis of a slip surface under a high normal stress would not be compatible with the residual shear strength determined by direct shear tests. For such a case it is essential to use ring shear tests to give an accurate value for the residual angle of shearing resistance. An interesting case about the Folkstone Warren landslides has been described by Hutchinson (1969). The stability of a number of cross-sections has been considered, however, there is only one section (section W 4) in which the slip surface lies entirely in Gault clay, while for the other cases the slip surface passes through much stronger chalk at the rear of the slip. The result of analysis for 1915 landslides is shown in Table 4.10.

Table 4.10 (after Hutchinson, 1969: Table 5)

Cross-section	Pore pressure assumption on slip surface	Value of $\phi_r^!$ Gault required for $F=1.0$ Morgenstern and Price method	Remarks
W4 (average profile)	Maximum	9.7°	Slip surface entirely in Gault
	Minimum	7.7°	
W6	Maximum	16.3°	$\phi_r^!$ in chalk marl at rear of slip taken as 20°
	Minimum	13.8°	
W8	Maximum	16.6°	$\phi_r^!$ in lower chalk at rear of slip taken as 23°
	Minimum	13.9°	
Weighted average of sections	Maximum	16.6°	
	Minimum	13.9°	

It will be observed that the slip surface lying completely on Gault gave lowest $\phi_r^!$, while the slip surface passing partially through chalk gave, not surprising, higher values of $\phi_r^!$. Yet the results were 'averaged' for the three sections and reasonably good correspondence was claimed between the 'averaged' values of $\phi_r^!$ and the residual strength as determined by direct shear tests. Analysis were also carried out for 1937 and 1940 slips for which part of the slips passed through chalk strata. A length of 1937 slip passed through lower

chalk for which $\phi_r^!$ was taken to be 23° and average values of $14^\circ - 16^\circ$ were obtained for residual angle of shearing resistance for the Gault. The analysis of the 1940 slips, the rear sections of which also passed through lower chalk ($\phi_r^!$ assumed to be 20°) gave an 'averaged' estimation of $\phi_r^!$ for the Gault to lie between $14^\circ - 15^\circ$. The author can firstly find no justification for averaging the values of $\phi_r^!$ which tends to distort the data from the analysis. Secondly only the data from section W4 which lies entirely in the Gault should be considered reliable for determination of $\phi_r^!$ for the clay in absence of any residual tests on chalk. The effective normal stress on this section is about 90 psi. while no direct shear tests were conducted at this stress level. The multiple reversal direct shear test on the Gault test showed the usual erratic behaviour with shear strength increasing with displacement after each reversal and gave $\phi_r^! = 19^\circ$ while the cut plane direct shear test gave $\phi_r^! = 12^\circ$. It also appears that an unrealistically conservative value for $\phi_r^!$ for lower chalk ($20-23^\circ$) has been used in the analysis. No tests were done to determine this value, and infact the direct shear box tests on samples of Middle chalk which were precut at 45° to the bedding in order to simulate the situation at rear of the Warren landslips gave $\phi_r^! = 35^\circ$. The author suggests that a ring shear test on Gault would yield a curved residual strength envelope, (this clay is known to contain a considerable quantity of montmorillonite), which for high normal stresses would show values of $\phi_r^!$ which would be compatible

with the analysis of section W4, or with re-analysis of other sections with a more realistic value of ϕ'_r operating in the chalk.

4.1.7 Soil structure in shear zone

Although there is no doubt about the breakdown of particles in the shear zone, this feature cannot explain the development of lower values of ϕ'_r with increasing stress. In chapter 5, some qualitative evidence based on thin section studies and scanning electron micrographs is presented which show that certain amount of particle segregation or migration of fines, and breakdown of particles takes place when the clay is sheared. It is highly likely that the extent of particle breakdown would depend on the magnitude of effective normal stress. Now if lower values of ϕ'_r are obtained at high normal stresses due to particle breakdown then the value of ϕ'_r should not increase when the effective normal stress is reduced after the sample has achieved residual state under a higher normal stress. However this is not the case and the variation of ϕ'_r with normal stress appears to be unique relationship which is not dependant on the sequence of loading or unloading. There is also some evidence that particle breakdown also occurs in blue London clay which did not depict a stress dependant residual shear strength in the stress range examined. It would therefore appear that residual shear strength is not governed by segregation or breakdown of particles, but rather by contact

pressure at the interparticle contact points and the number of interparticle contacts per unit surface area on the slip surface. The slip surface, even after very large displacement remains wavy in character, with small grooves running in direction of shear, and it is possible that under low normal stresses, a relatively poor contact (on the sub-microscopic scale) between clay particles is made. The nature of interparticle contact would depend on the clay mineral type, the nature of adsorbed water layer and pore water, the consideration of which is beyond the scope of this thesis.

Determination of particle size of initial sample before shearing and the clay that squeezed out through the confining ring gaps shows an increase in clay fraction in the sheared material. This has also been noted on natural slip surfaces by Petley (1966) and Agarwal (1967), who found the following increases as shown in Table 4.11, overleaf.

Table 4.11

Soil-Location	% clay fraction (less than 0.002 mm)		Reference
	sheared	unsheared	
Oxford clay- Peterborough	47	42	
Carboniferous mud- stone-Waltons Wood	69	60	Petley, (1966)
Sandy-silty clay, Monmouth	46	38	
Blue London clay, Wraysbury	60	58	Agarwal, (1967)
Brown London clay, Guildford	59	57	Skempton and Petley, (1967)

Interesting results were obtained for undisturbed brown London clay tested in ring shear apparatus, where the clay fraction for the initial sample, general sample after shearing and the clay squeezed through the gap were found to be 53%, 55% and 55% * respectively. Corresponding mineralogical tests by X-ray analysis at the Royal School of Mines by A. Burnett showed the clay mineral

* It should be mentioned that particle size analysis on these samples done at the Norwegian Geotechnical Institute by using the accurate 'falling drop' technique showed that the squeezed material was in fact even more coarse than the parent material. Hence in future tests, sheared clay should be scraped off the slip surface and it is this material whose clay fraction should be determined.

fractions for the three samples to be 64%, 65% and 70% respectively with a slight increase in montmorillonite content in the sheared samples, (Table 4.5).

Similar increase in clay mineral fraction was also noticed for Weald clay for which percentage clay mineral fraction of 67% and 72% were observed for pre- and post-shear samples respectively (initial % clay size fraction 52%). This increase in clay mineral content is also consistent with increase in clay particle fraction since it is most likely that majority of the clay mineral fraction would be in clay particle size.

Tests done at the British Ceramic Research Association show that there is a migration of fine particles when a clay surface is 'smoothed' or polished. Nitrogen adsorption experiments, ethylene glycol retention experiments and flame photometric analysis have provided evidence for the migration of fines, (see B.C.R.A. Sp. Publication 29, 1961). Microscopic evidence of particle breakdown is presented in chapter 5. The author suggests that if a large increase in clay fraction occurs at residual state then it is necessary to determine clay fraction on the slip surface itself, rather than in the pre-shear sample if any useful correlation between ϕ'_r and clay fraction is to be established.

4.2 Tests on highly plastic clay-shales from North America

4.2.1 Introduction

Two tests on the Cucaracha clay-shale from Panama Canal and the Dawson shale from Chatfield Dam site were conducted in the ring shear.* Most of the highly plastic clay shales from North America are rich in montmorillonite. Underwood, (1967) and Kenney, (1967) have shown that it is the higher montmorillonite and illite content that impart such undesirable properties as low shearing strength and high swelling potential. It is of interest to note that the average chemical composition of clay shales throughout the world is very similar (Tourtelot 1952) and the main differences in behaviour can only be explained in terms of the type and quantity of clay minerals. Hence, certain similarities should emerge on a study of highly plastic clay shales which are considered in the following review.

4.2.2 Clays from Panama Canal

General description and geology of Panama Canal area

The geology of the Panama Canal region varies considerably because of successive marine transgressions and regressions and also because of the previous volcanic activity in this area. Sliding has been a special problem with three formations in particular: Cucaracha, Las Cascades and Culebra; and the inhomogeneity in the characteristics of these soils makes

* apparatus

the study of this problem more difficult.

The Cucaracha formation is mainly terrestrial in origin although signs of marine transgressions are also evident. About 60% of the total thickness of this formation is made of very weak, poorly bedded, bentonitic, slickensided, soapy textured clay shales. Silt stones and conglomerates are often interbedded with clay shales. Because of volcanic activity, fine volcanic ash could easily be altered into hydrous minerals of the montmorillonite group. This phenomena has been discussed by Hewett, (1917); Ross and Shannon, (1926) and more recently by Grim, (1953, 1962). The Cucaracha is the weakest formation encountered along the canal.

The Las Cascades formation is extremely variable in strength and stability. It consists mostly of scattered hard angular fragments of basalt and andesite embedded in a waxy, clayey, altered tuffaceous mix. Scattered lenses of metabentonitic clays, up to several feet in thickness are also present.

The Culebra formation, deposited in a marine environment, lies between Las Cascades formation at the bottom and the Cucaracha at the top. It consists of an upper calcareous member and lower argillaceous member. The upper harder member consists of soft to medium limestone, sandstone and tuffs; while the softer clayey member is composed of thinly laminated, variably silty

carbonaceous clays with interbedded sandstones and pebbles. Hirschfield et. al., (1965) have pointed out that the Culebra does not contain clay-shale - the bentonitic, blocky, waxy and highly slickensided material which is derived from alteration of volcanic ash. It contains 70% shale formed by subsequent lithification of clay after deposition in a marine environment. (The Cucaracha contains about 60% clay shale and less than 10% shale). *

4.2.2.1 Previous strength studies on Cucaracha clay-shale

Most investigations in Panama Canal region have been confined to study of Cucaracha clay-shale and very little information is available about other formations.

An intensive study was undertaken for the Third Locks investigations. An excavation slope curve was prepared based on laboratory studies, the analysis of a then existing slope and analysis of slides (see Binger, 1948; Binger and Thompson, 1949). Direct shear tests in laboratory on clay shale with polished cut plane to simulate slickensides gave an angle of friction of 10° .

* In literature on Panama Canal, the terms 'shale' and 'clay-shale' often appear and the difference between the two types, in context with the formations present at this site are as follows: (after Hirschfield et. al., 1965) -
 'shale': lithified mud, clay or silt; usually laminated because of its sedimentary origin.
 'clay-shale': altered, de-vitrified tuffs that are rich in clay minerals, but which are not true shales; from which they differ in mode of origin and absence of laminations.

The value of cohesion was found by carrying out a total stress stability analysis on a stable 200 ft cut on an average slope of 1 vertical to 2.7 horizontal. Slope curves for complete drawdown using $c = 16$ psi. $\phi = 10^\circ$; F.S. = 1.3 were then developed. It is of course incorrect to assume that the stable slope was at factor of safety of 1. In this connection, it is of interest to note Binger's (1948) comment that "once a slope in Cucaracha fails as a result of having been cut oversteep, it will then assume an angle of repose much flatter than would have been required originally for a safe design." This comment is not surprising because later investigations indicated residual strength parameters to be much lower than those considered by Binger. A number of consolidated undrained tests were also done at Harvard in 1948, but the loading rates were very fast. The results have been tabulated by Hirschfield et. al., (loc. cit.). In-situ field shear tests were carried out as early as 1941 in a drift on Pedro Miguel test pit No. 3. Six tests on one foot square sections were conducted under normal stresses of 4 ton/ft² and 7.5 ton/ft² (Hirschfield et. al.). It appears that these tests were not run sufficiently slow to achieve drained conditions. The large variation in peak strength is a clear indication of the non-homogeneity and varying degrees of slickensides in the Cucaracha formation. On the basis of equal weightings for each test result, the following parameters were established.

$$\phi'_{\text{peak}} = 21.6^{\circ}; c'_{\text{peak}} = 700 \text{ ton/ft}^2.$$

$$\phi'_{\text{residual}} = 8.8^{\circ}; c'_r = 1300 \text{ ton/ft}^2.$$

A program of systematic study has recently been undertaken at the M.I.T. Laboratories. Block samples of Cucaracha clay shale were taken from a test pit on the Escobar slope along the Panama Canal. The index properties were found to be L.L.: 75-80%, P.L.: 32-35%. The test samples were prepared from the block sample by grinding and sieving through size 40 sieve, and then reconsolidating at a pressure of 5 kg/cm² from an initial moisture content above liquid limit. The details of both, multiple reversal direct shear tests and tests conducted in an annular ring shear (sample dimensions; 4.5 in. O.D., 2.5 in. I.D., and 0.85 in. thick) have been tabulated in Table 4.12, and plotted in fig. 4.36.

More recently, annular ring shear tests have been conducted at Harvard University (La Gatta, 1969)*. The result of such a test on a sample of remoulded Cucaracha clay shale (L.L. 156%; P.L. 42%) has also been shown on fig. 4.36 along with other data on this clay-shale.

4.2.3 Bearpaw shale

Peterson (1954), Peterson et. al., (1960) and Ringheim (1964) describe the geotechnical investigations for slope stability in Bearpaw clay shale under South Saskatchewan Dam, Western Canada. Geologically, these clay

* Private communication

shales are sedimentary deposits of marine origin of the Upper Cretaceous age. It is estimated that about 2000 - 2500 ft of sediment has subsequently been eroded; i.e., the maximum consolidation pressure was of the order of 1500 - 2250 psi. It contains high percentage of montmorillonite with some pure bentonitic layers. The clay can be divided into three zones; soft, medium and hard on the basis of physical properties and it is of interest to compare these.

	Soft		Hard	
Thickness	0 - 20 ft		100 - 200 ft	
Slickensides and joints	closely spaced (up to 1/8 in.) surfaces glossy and wet.		Generally spaced at 12 in. or more, frequency decreasing with depth.	
Water content	29 - 36%		19 - 27%	
L.L.	80 - 150%		same	
P.L.	18 - 27%		same	
Clay fraction	30 - 65%		same	
<u>Undisturbed peak strength:</u>	c' psi. ϕ'°		c' psi. ϕ'°	
Direct shear test	6	28 $^{\circ}$	10	35 $^{\circ}$
Triaxial test	-		26-52	30.5 $^{\circ}$
<u>Remoulded peak strength:</u>				
Direct shear test	1	22 $^{\circ}$	1	15.5 $^{\circ}$
Triaxial test	2	18 $^{\circ}$	1	15.5 $^{\circ}$

A number of slides took place in the soft shale for which attempts were made to analyse failure on the basis of peak strength, and not surprisingly, no correspondence was reached. These slides, some of which had moved 10 - 15 ft suggested a ϕ' of 7° to 10.5° for a factor of safety of one, with ground water table at the surface. Because the residual strengths were not considered, it was suggested that effective stress slope stability analysis was not valid for highly plastic clays and the term "instability factor" was proposed to account for the apparant contradiction, (Terzaghi, 1960; Peterson et. al. 1960). The low residual shear strength could easily be attained due to the presence of numerous slickensides in the softer clay, which were probably formed when the large horizontal stresses were released on unloading. The presence of high lateral stresses has been demonstrated by conducting an earth pressure test using calibrated vertical and horizontal steel struts in a gallery (Peterson, 1954), where the results indicated a vertical pressure of 100% overburden and a horizontal stress of 150% of overburden pressure. The 'cut' which released the horizontal pressure thus causing slickensides was probably the erosion of the valley itself.

Middlebrooks (1942) describes similar experiences in the Bearpaw formation in United States where the Fort Peck slide was caused in weathered bentonite. On an analysis of this slide, he obtained the required strength parameters for F.S. unity as $\phi = 10.5^{\circ}$, $c = 2.8$ psi.

However, his concept of total and effective stresses appears to be unsound.

Hardy, Brooker and Curtis, (1962) attempted to analyse two slides in overconsolidated clays in Alberta, Canada, by modifying basic shear strength equation to:

$$\tau = c' + (\sigma_n - u - p_s) \tan \phi'$$

where p_s = swelling pressure of the soil, on the basis that overconsolidated soils would tend to swell on failure. This expression is, of course, invalid and shows little understanding of the principle of effective stress. The first slide considered was in the Dunvegan slope for which the general Atterberg limits were L.L. 50%; P.L. 24%. $w/c = 22\%$, although the limits in the vicinity of slip plane were L.L. 30%. P.L. 29%, and water content of 30%. The peak strength parameters were found to be $c' = 0.2 \text{ ton/ft}^2$ and $\phi' = 21.5^\circ$, whereas the analysis of the slide without considering p_s , suggested ϕ' of 9° ($c' = 0$) for F.S. = 1. In the second slide on Little Smoky Hill (L.L. = 42; P.L. = 10, $w/c = 24\%$), the peak strength parameters were $\phi' = 19.5^\circ$, $c = 0.38 \text{ ton/ft}^2$, and the field analysis without considering p_s , gave $\phi' = 12^\circ$ for stability. In both cases, aerial photographs had clearly indicated previous landslides; hence the shear strength at failure would certainly have been reduced from the peak value.

4.2.4 Pierre shale:

Knight (1963) discusses a slide in Oahe Dam near the eastern edge of Missouri Plateau. The only bedrock formation exposed at the dam site is the Pierre shale, formed in the Late Cretaceous geological period. It is derived from clay and silts which has been lithified into dark bedded shales and marls due to heavy overburden pressure. Slides occurred on 1:3 cut slopes passing through weathered bentonite, which could be explained on the basis of $\phi' = 8.5^\circ$ and $c' = 2$ psi. The shale contained 40 - 60% montmorillonite; 30% quartz and cristoballite and some illite. Bishop (1966) quoted results of triaxial tests on Pierre shale conducted by Webb who obtained $\phi' = 7^\circ$ for strength along a fissure in the stress range of 220 to 330 psi. However it is most likely that this strength would be much larger at low effective stresses, especially as the peak strength displayed a marked curvature with effective stress.

4.2.5 Pepper shale

The Pepper clay shale is fissile, noncalcareous fat clay sandwiched between the older and strong calcareous, fossiliferous clay shale and the younger Eagle Ford formations. Beene (1967) describes a slump slide in the Pepper shale at Waco Dam before any water was impounded. This shale is about 60 ft thick beneath the portion of the embankment where slide took place. An interesting feature is the presence of two faults which

straddle the axis of the dam. Since the faults are not parallel either in dip or in strike, they could easily have had a shearing effect on the Pepper shale. At the crest of the dam, between the two faults, the Pepper shale is exposed.

The movements, which were not rapid, took place entirely in the Pepper. It is of particular interest to note that the cracking and bulging 770 ft downstream from the axis occurred before any significant bulging and distortion took place near the downstream toe of the dam about 300 ft from the axis. The movement continued on a horizontally downstream direction. This suggests that extensive movement had taken place along the slip plane.

Open standpipe piezometers showed widespread distribution of pore pressures. The pore pressure at mid depth of Pepper formation was about 70% of the embankment load under the axis of the dam. Away from the axis, the pressure was generally at ground water level. Shear strength tests were performed in direct shear apparatus and the following parameters were obtained.

Peak: $\phi' = 14^{\circ}$; $c' = 0.4 \text{ ton/ft}^2$.

Residual: For envelope drawn through tests done at

$$\sigma'_n = 12 \text{ ton/ft}^2 \text{ and } 24 \text{ ton/ft}^2,$$

$$\phi'_r = 4^{\circ} - 6^{\circ}; c'_r = 1.6 \text{ ton/ft}^2,$$

and for envelope drawn through test at

$$\sigma'_n = 24 \text{ ton/ft}^2 \text{ only,}$$

$$\phi' = 7^\circ - 9^\circ \text{ and } c' = 0.$$

preformed cut plane tests gave $c' = 0$; $\phi'_r = 8^\circ - 10^\circ$, (Van Auker, 1963; Beene, 1967). The average Atterberg limits of Pepper shale in this site are L.L. 80; P.L. 22; natural water content: 17%. The above residual strength data clearly indicates a curved shear strength envelope.

The analysis of the slide was done on two assumptions of pore pressure distribution. The first assumed a value of 0.7 at any point on midplane of the Pepper ~~for~~ formation and gave $\phi' = 8^\circ$ ($c' = 0$) for F.S. of unity. The second method assumed that pore pressure at the base of Pepper formation, directly under the axis of the dam was equal to the height of overlying embankment. This pore pressure then dissipated linearly to surface ground water table, as given by a piezometer outside the region of instability. This method gave a limiting ϕ'_r of 15° . However, an analysis of post-failure embankment and porepressures gave an average ϕ' of 6° .

More recently (La Gatta 1969), tests done at Harvard in rotary ring shear apparatus on remoulded Pepper shale gave a residual angle of 7.2° . This test was conducted at an effective normal stress of 4 kg/cm^2

and at a nominal shearing strain rate of 0.0022 in/min.

The case record described above is an excellent example where large displacements under high normal stress reduced shear strength to very low values. The correspondence between test results from direct shear and ring shear tests and analysis of the slide is remarkable.

4.2.6 Test results

4.2.6.1 Test on remoulded Cucaracha clay shale

The sample was found to be dried and decomposed due to imperfect sealing and long storage. The remoulded sample ^{was} at a water content of 40.2%. The stress displacement curve is presented in fig 4.34. It will be noticed that ϕ' dropped from 24° at peak to 7° after a displacement of 3 in., and a subsequent drop of 0.25° occurred after a further displacement of 7 in., when residual strength was reached.

The data has also been drawn on τ/σ'_n against log. displacement plot in fig 4.34 shows that the slope of the curve became zero thus showing that residual strength has been achieved.

The value of $\phi'_r = 7.9$ is slightly lower than that obtained by Hirschfield et. al., (1965) and La Gatta, (1969) as shown in fig 4.36. Kenney (1967) obtained a

value of $\phi'_r = 6.3^\circ$ for Cucaracha clay shale in a 'smear test'. The material used for ring shear test had undergone considerable alteration due to drying and exposure over a number of years and perhaps a direct comparison is not relevant.

Residual strength: $\phi'_r = 7.9^\circ, c'_r = 0$

Index properties were: L.L. = 62.6
F.L. = 42.6

Initial clay fraction (less than 0.002 mm) = 48%.

A clay mineralogical examination by X-ray diffraction revealed the following:

Quartz	18%
Calcite	4%
Gypsum	7%

Clay mineral fraction 71%; of which 60% was montmorillonite.

4.2.6.2 Tests on Dawson clay-shale (Chatfield Dam)

The undisturbed sample was found to be dry, hard and brittle and it was not possible to prepare an undisturbed sample for testing in the ring shear apparatus. Hence a remoulded sample was prepared as described in chapter 3.

The test was conducted on the same sample under two different normal stresses. The stress displacement

curve is shown in fig. 4.37 from which it is seen that the clay shale shows a value of ϕ'_r which is stress dependant. The values of ϕ'_r at normal stresses of 14.7 and 85 psi were found to be 12.5° and 8.2° respectively.

These value may be compared with a residual value of about 8° obtained in cut plane direct shear tests. This test (performed by U.S. Army Corps of Engineers) was conducted at an effective normal stress of 83 psi. The material properties were:

L.L. = 50%

P.L. = 22%

clay fraction less than 0.002 mm = 20%

X-ray diffraction (by U.S. Army Corps, of Engineers) showed that the sample contained 35% quartz and 65% of clay mineral fraction which was composed as follows:

Kaolinite:	11%
Illite:	5%
Montmorillonite:	72%
Mixed-layer: clay minerals	12%
Dominant ion:	Calcium

Other data on residual strength of highly plastic clay shales is given by Kenney, (1967). The main point to note is that these clay shales contain high clay mineral content (as distinct from clay size fraction), a high proportion of which is montmorillonite. Some clay shales e.g. the Cucaracha and the Pepper have

seams of almost pure bentonite. It is conceivable that slides may be triggered off as a result of swelling action of bentonite, thus reducing the strength to residual. Some typical values for residual strength of bentonite are shown below:

Material	Salinity gms/litre	σ'_n tsf.	ϕ'_p ϕ'_r	Reference
Sod. Montmor- illonite	0	0.915	- 4°	Kenney, (1969)
"	30	0.915	- 8.5° - 10.2°	
Ca "	0	0.915	- 9.7° - 10.2°	
Sod."	0	1.1	8° 5°	Yudhbir, (1969)

Similar results on the Edmonton formation which consists of interbedded bentonitic shales and sandstone with coal seams and bentonite have been obtained by Sinclair and Brooker (1967). The effective residual angle of shearing resistance of the Edmonton clay shales and bentonites was found to be approximately 8.5° , and the peak angle of shearing resistance was also 8.5° at normal stresses above 150 psi.

Clay mineral and particle size data on highly plastic clay shales highlights the inadequacy of plotting ϕ'_r against clay size fraction as proposed by Skempton (1964) and later adopted by Hutchinson, (1965) Petley, (1966) and Chandler, (1967). Kenney (1967)

found no such correlation when a large number of different soil types were considered. Two examples will suffice to illustrate this point. A sample of pure bentonitic seam from the Cucaracha contained almost 100% clay mineral fraction (90% of which was sodium montmorillonite) and only 36% clay fraction according to particle size determination. The value of ϕ_r' for this material at $\sigma_n' = 15$ psi. would be $4^\circ - 5^\circ$ on basis of Kenney's results, while ϕ_r' from Skempton's plot would be typically 20° . On the other extreme, Kenney (1967) tested finely crushed quartz with clay size fraction of 100% and obtained $\phi_r' = 35\%$.

4.3 Conclusions

From a study of the tests in the ring shear apparatus, the following conclusions may be drawn.

1. The ring shear apparatus is most suitable for measuring residual shear strength of clays since it permits uninterrupted displacements whereby the clay particles on the slip surface can achieve maximum particle orientation. The multiple reversal direct shear box test gives apparent values of ϕ_r' which are higher than the values obtained from ring shear test. This is due to disturbance of particles orientation on the slip surface on reversals, and the ambiguity of interpretation of direct shear box stress-strain curves. The direct shear box test

curves for stiff clays at normal stress greater than 15 psi. invariably tend to show an increase of shear strength with displacement on the second and subsequent travels. The observation that polished cut plane triaxial tests always give values of ϕ'_x which are smaller than those from direct shear tests confirms that the latter do not measure residual shear strength accurately.

2. The determination of residual state by visual examination of direct shear stress-displacement curve can be extremely misleading. Such a case is exemplified in fig 4.14 where case (a) shows the clay at its apparent residual shear strength while case (b) shows the erroneous interpretation on stress - log displacement curve very markedly. Hence it is suggested that the attainment of residual state should be confirmed by plotting τ/σ'_n versus log displacement curve when the slope of this graph should become zero. The slope of all ring shear tests drawn on the new scale becomes zero thus confirming that residual strength is being measured.
3. Very large displacements are required to drop shear strength from peak to residual value. Also, samples sheared under large effective normal stresses appear to require less displacements than those sheared under lower normal stresses. Typical displacements for undisturbed blue London clay ranged from about

20 in. at normal stress of 6 psi. to approximately 5 in. at normal stress of 40 psi.

4. The most significant conclusion is that the residual shear strength is not necessarily independent of the magnitude of effective normal stress. The residual strength of Weald clay, brown London clay and the clay shales from North America showed marked dependence on stress level while that of blue London clay did not. The precise reasons for such a behaviour are not known. The residual shear strength of brown London clay and Weald clay dependent on normal stress level, was, however, completely independent of previous stress history and the residual strength followed a unique relationship irrespective of the sequence of loading or unloading.
5. For long term analysis of stability of slopes in clays which show a pronounced effect of normal stress on residual shear strength, it is essential to conduct the tests in the ring shear apparatus at the effective normal stress that acts in the field. Direct shear box tests would greatly overestimate residual strength under high normal stresses.
6. It is probable that there is an increase in clay particle-size fraction in the slip surface after a clay has been sheared through large displacements. This increase can be explained in terms of particle

breakdown, and/or migration of fines, and also the increase in clay mineral fraction.

7. Highly plastic clay shales which have high clay mineral (especially montmorillonite) content would generally show low residual shear strength. This value would be dependant on the magnitude of effective normal stress in the cases examined.
8. It is suggested that in the absence of a ring shear apparatus, residual tests on stiff clays should, if possible, be conducted by using polished cut plane triaxial testing technique. Such tests would give values of residual shear strength which would approach the ring shear determination of ϕ'_r more than the values from direct shear box tests.

Table 4-1

Shear strength on natural slip surfaces (after Skempton and Petley, 1967)

<u>Site</u>	<u>Clay</u>	L.L.	P.L.	$\angle 2\theta$ C.F.	Peak		Residual	
		%	%	%	C'	ϕ'	$C'_r=0$	ϕ'_r
					psi.	degrees		degrees
Walton's Wood	weathered Carboniferous Mudstone	57	26	69	10	13°		13°
Guildford	Brown London Clay	83	32	57	70	12°		15°
Guildford, Dedham, Sudbury Hill.	Brown London clay	87	31	56	60	12°		14°
Sevenoaks	Atherfield Clay	75	29	59	80	10.5°		12°
		69	29	51	100	15°		15°
	reworked Atherfield Clay	71	31	58	20	16°		16°
	weathered Weald clay	69	27	71	60	15°		15.5°
		62	26	50	70	15.5°		16.5°

Table 4-2

Results of ring shear tests on blue London clay (Wraysbury)

Test No.	Normal stress psi.	Shear stress psi. Peak Residual	ϕ_r degree ($C'_r=0$)	Average initial w/c %	Average final w/c %	Av. final w/c in shear zone%	Av. displacem. to residual (ins)	Brittleness Factor %
Remoulded: RS 1	26.92	10.03 4.51	9.5°	43.85	37.34	-	6	55.0
Undisturbed: UDLC-RS2	28.95	13.6 4.78	9.4°	28.74	33.66	33.0	15	64.8
UDLC-RS3	11.92	3.6 1.97	9.3°	29.02	33.40	36.20	20	45.3
UDLC-RS4	5.86	6.2 0.96	9.3°	29.20	35.33	40.2	20	84.5
UDLC-RS5	40.63	15.5 6.615	9.2°	28.85	31.20	32.0	5	57.3

Index properties: L.L. 72%; P.L. 29%; clay fraction = 57% (less than 2μ)

Results of X-ray diffraction at Royal School of Mines:

Quartz	30%
Chlorite	10%
Illite	35%
Kaolinite	15%
Montmorillonite	10%

Table 4-3

Results of other residual strength data on blue London clay. (Direct shear tests, unless otherwise mentioned).

Location and type of sample	Normal stress psi.	Shear strength psi.		ϕ_r' degrees $C_r' = 0$	Reference
		Peak	Residual		
Ashford Common cut plane triaxial test; undisturbed. L.L. 67, P.L. 27, C.F. 54%	64	-	-	11.6°	Bishop, Webb and Lewin, (1965)
	130	-	-	11.5°	
Herne Bay; Undisturbed L.L. 81; P.L. 33. C.F. 61%	17.7	14.6	4.0	12.7°	Petley, (1966)
	31.2	15.6	6.6	11.9°	
	44.7	31.2	11.0	13.8°	
Herne Bay; Slurry L.L. 85; P.L. 35, C.F. 59 %	13.2	5.8	2.8	12.0°	
	26.7	10.0	5.7	12.2°	
	35.7	13.0	6.8	10.8°	
Leith: Undisturbed, L.L. 67; P.L. 26; C.F. 51%	17.7	10.1	5.0	15.8°	
	31.2	16.1	7.5	13.5°	
	44.7	22.1	11.0	13.8°	

Leigh:	10.2	4.5	3.4	18.5°	Agarwal, (1967)
Slurry	22.2	9.5	5.6	14.2°	
L.L. 70; P.L. 26;	31.2	11.5	8.2	14.8°	
C.F. 49 %					
Wraysbury	7.5	5.7	2.3	17.1°	
Undisturbed	15.0	11.4	3.8	14.2°	
L.L. 70; P.L. 29;	30.0	13.4	9.0	14.7°	
C.F. 58 %	45.0	23.1	10.0	12.5°	
	60.0	25.2	15.3	14.3°	
	75.0	32.8	17.2	12.9°	
	90.0	34.4	17.3	10.9	
Wraysbury:					
Undisturbed	15.0	2.8	2.2	8.3°	
slip surface.	30.0	6.0	5.35	10.1°	
L.L. 74.2; P.L. 30:					
C.F. 60%					

Table 4-4

Ring shear tests on Weald clay (Arlington).

Test stage	Normal stress σ'_n psi.	Residual shear stress τ_r psi.	ϕ'_r ($c'_r = 0$)
1	8.9	2.20	13.9°
2	17.53	3.72	11.95°
3	36.80	6.92	10.7°
4	9.70	2.31	13.4°
5	6.22	1.72	15.4°
6	64.72	10.55	9.3°
7	119.44	19.34	9.2°

Results of X-ray diffraction tests at Royal School of Mines:

	Parent material before shearing in apparatus	Material squeezed through gap on shearing
Quartz	20%	13%
Feldspar	5%	4%
Carbonates	2.5%	2%
Clay minerals (Kaolinite, illite and montmorillonite)	67%	72%
Initial w/c = 25.2%		
L.L. = 65		
P.L. = 32		
B.S. Clay fraction (less than 0.002 mm)	= 52%	
Clay fraction from falling drop method (N.G.I.)	= 60%	

Table 4-5

Ring shear tests on undisturbed brown London clay (Edmonton).

Test stage	Normal stress σ'_n psi.	Residual shear stress τ_r psi.	ϕ'_r ($C'_r=0$)
1	31.0	4.774	8.75°
2	15.97	2.506	8.9°
3	8.63	1.462	9.6°
4	5.0	0.912	10.4°
5	23.73	3.512	8.4°
6	36.30	5.095	10.0°
7	4.48	0.838	10.6°
8	2.31	0.500	12.2°
9	1.0	0.246	13.8°

Av. initial w/c = 31.3%

L.L. = 66; P.L. = 24.

	B.S. method	H.G.I. Falling drop method
1) % clay fraction before shearing	= 53%	(60%)
2) % clay fraction of sample after shearing	= 55%	(53%)
3) % clay fraction of soil squeezed through gap during shearing	= 55%	(50%)

Results of X-ray diffraction tests at Royal School of Mines:

Percentage	1) Undisturbed sample	2) Sample after shearing	3) Sample squeezed through gap
Quartz	22	22	20
Feldspar	3	3	2
Plagioclase	4	4	1
Calcite	2	2	1
Dolomite	5	4	5
Claymineral fraction	64	65	70

Subdivision of clay mineral fraction:

Kaolinite	23	23	26
Illite	49	48	42
Montmorillonite	27	29	31

Table 4-6

Ring shear tests on slurried brown London clay.

Test stage	Normal stress σ'_n psi.	Residual shear stress τ_r psi.	ϕ'_r degrees ($C'_r = 0$)
1	13.1	2.20	9.75°
2	9.52	1.68	10.0°
3	13.53	2.30	9.75°

Average initial w/c = 40.85%

Average final w/c = 38.5%

L.L. = 68%

P.L. = 25%

% clay fraction (B.S.C.) = 64%

% clay fraction (falling drop method; N.G.I.) = 63%

Results of X-ray diffraction on initial sample:

Quartz	21%
Feldspar	5%
Plagioclase	3%
Calcite	2%
Clay mineral fraction	69% - of which
Kaolinite	23%
Illite	49%
Montmorillonite	28%

Table 4-7

ϕ'_r for brown London clay tested in direct shear box.

Location	Sample	L.L.	P.L.	C.F. 2.11	ϕ'_r ($C'_r = 0$)	Pressure range psi.	Reference
Hendon	undisturbed	82	33	60	14°	0-50	Petley, 1966
					17°	0-10	
	slurry N.C.	80	35	64	14°	0-30	
	slurry O.C.	81	33	58	13°	0-30	
Herne Bay	undisturbed, slurry	79	30	67	12.5°	0-50	
Brentwood	undisturbed	72	28	52	14°	0-40	
	slurry	77	29	53	13°		
	cut-plane	72	28	53	12.5°		
Guildford Dedham Sudbury hill	slip surfaces	87	31	56	14°		Skempton and Petley, 1967
Edmonton	undisturbed, cut-plane	43	21	40	13°		Petley, 1969 (unpublished)

Table 4-3

Some typical effective normal stresses acting on failure surfaces in natural slopes in brown London clay (after DeLory, 1957).

Location	Age at failure years	σ'_n on failure surface psi.
Wembley Hill	13	11.4 - 13.0
Park Village East	19	12.5 - 13.6
Mill Lane	28	11.0 - 12.5
Northolt	35	5.3 - 5.75
Sudbury Hill	49	4.4 - 5.3
Kensal Green	53	5.3 - 6.5
Wood Green	55	7.3 - 8.3
Upper Holloway	81	2.6 - 3.7

Table 4-8 (a)

Some typical effective normal stresses acting on failure surfaces in London clay and Weald clay, (after James, 1970).

<u>London clay sites:</u>	Average normal stress psi.	Age at failure years
Northolt	5.05	35
Wembley Hill	20.5	13
Upper Holloway	5.78	82
West Acton	2.26 - 2.64	46
Tulse Hill	5.0 - 5.48	101
Grove Park	4.35	98
Dedham	3.93 - 4.5	112
Kingsbury	2.51	15
St. Helier	5.35	22
Crews Hill	3.68	47
Althorne	4.25	70
Cuffley	4.6	40
Hadley Wood	3.06 - 5.96	90
Whitstable	0.826 - 1.44	99
Grange Hill	6.75	(50)
Isle of Sheppey	2.7 - 3.36	6 - 8
Bradwell	4.32 - 9.36	0
Wood Green	10.6	55
Sudbury Hill	5.15	49
<u>Weald clay sites:</u>		
Hilbenborough	4.27	68
Earlswood	10.5	68

Table 4-12

Summary of M.I.T. results on shear testing of remoulded Cucaracha clay shale.

No.	Test type	No. of shears	Av. rate of deformation in/day	Total displacement ins.	Total time days	ϕ_r^0
1.	Ring shear		0.08	0.06	1	
2.	Same as (1)		0.08	7.4	93	9.1°
3.	Direct shear	1	0.05	0.10	2	
4.	Same as (3)	13	0.07		7	9.1°
5.	Ring shear		0.08	0.02	1/4	
6.	Same as (5)		0.08	3.0	38	10.2°
7.	Direct shear	1	0.01	0.02	2	
8.	Same as (7)	15	0.02-0.07		27	10.2°
9.	Direct shear	1	0.07	0.06	1	
10.	Same as (9)	31	0.07	1.12	16	9.6°

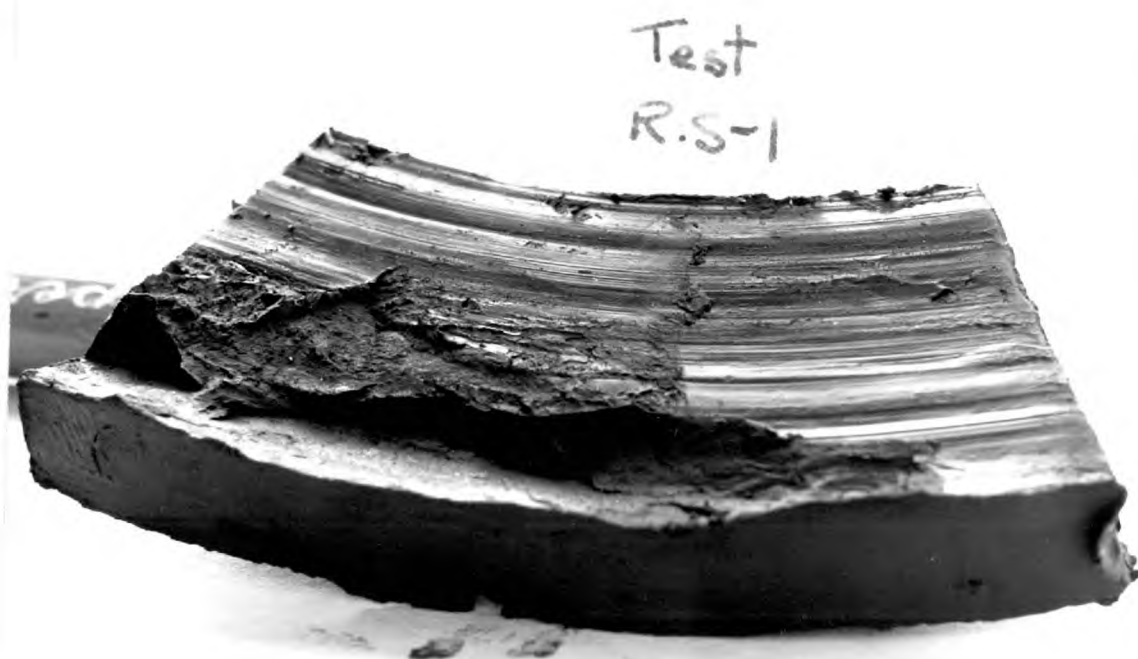


PLATE 4.1

Slip surface at the end of ring shear test

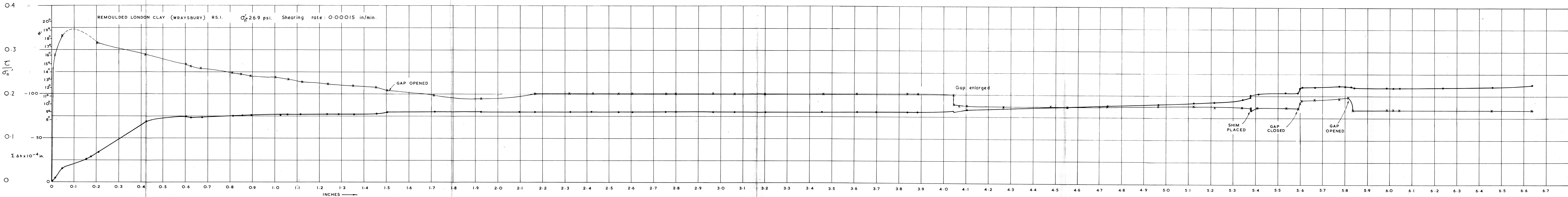


FIG. 4.1

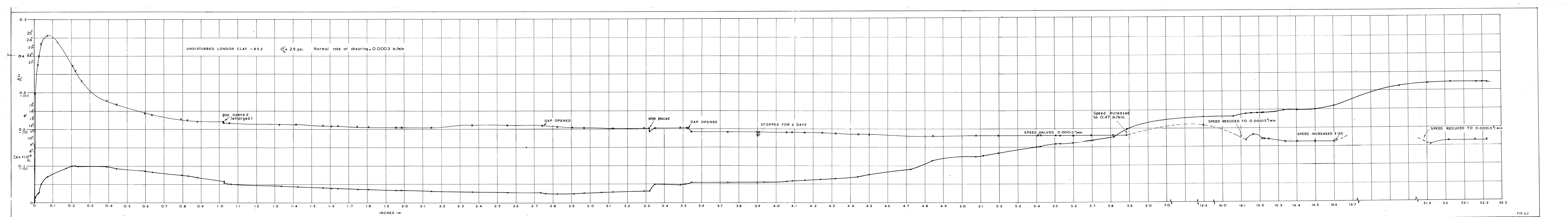


FIG. 4.2

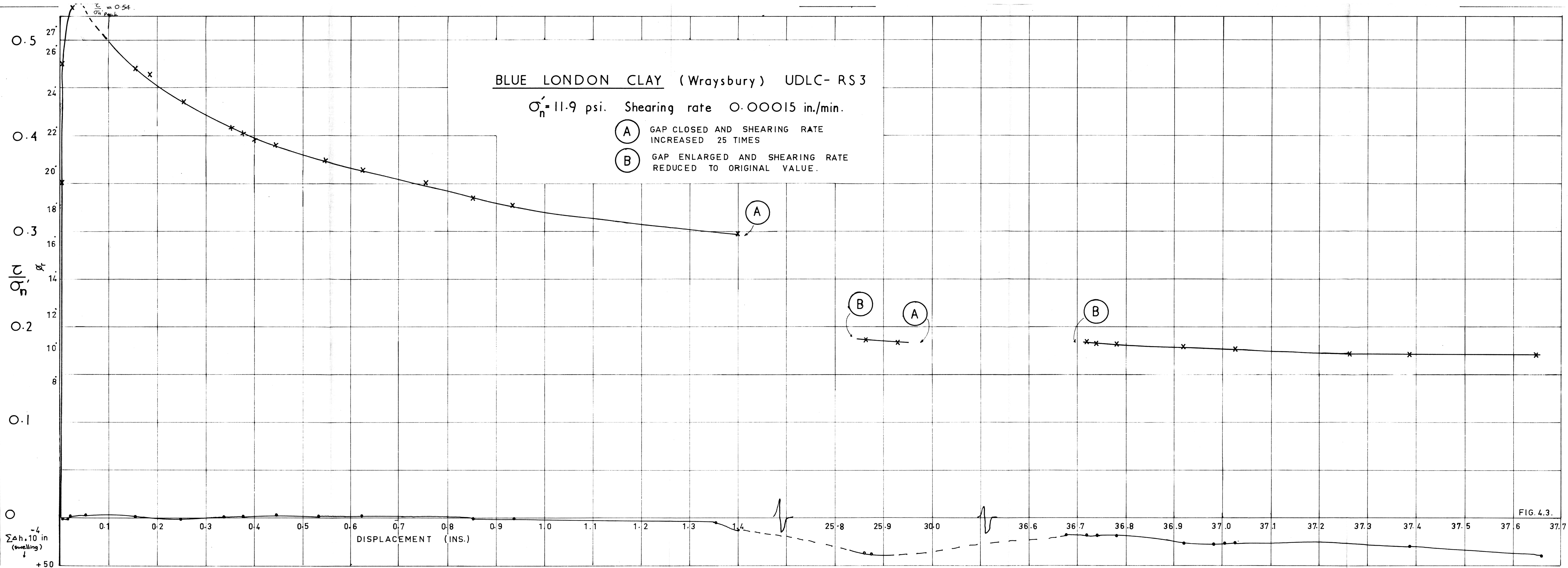


FIG. 4.3.

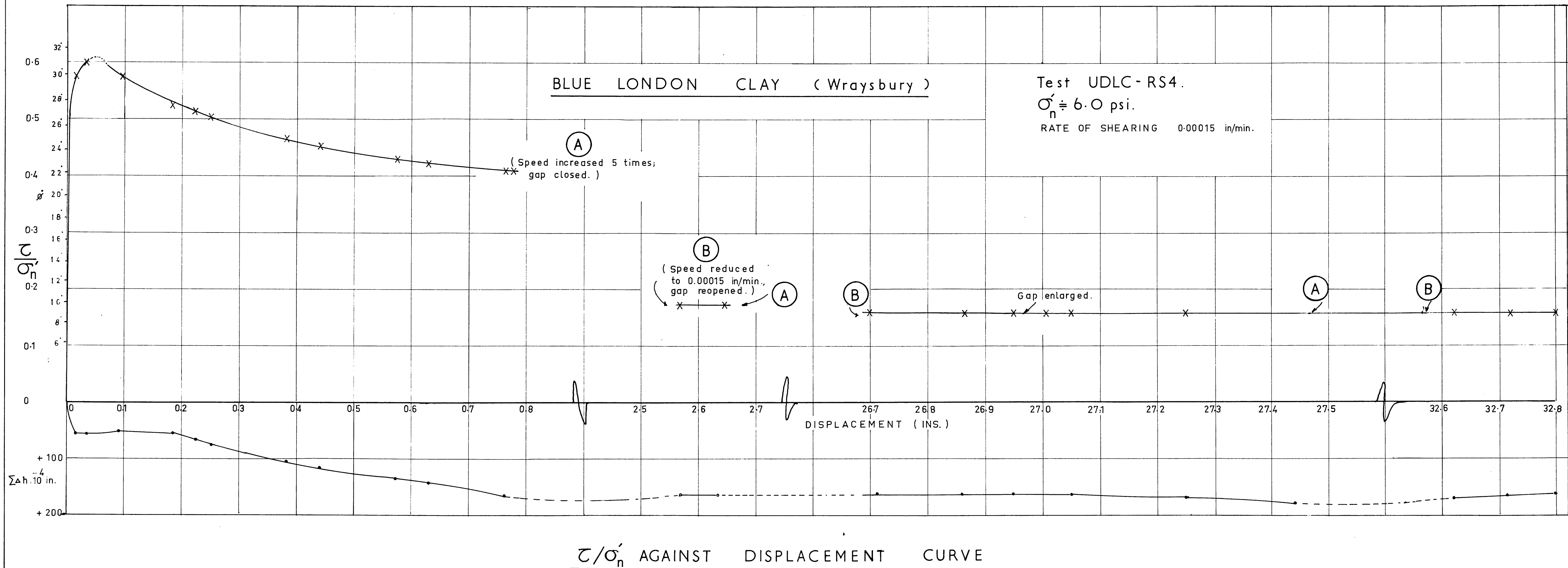
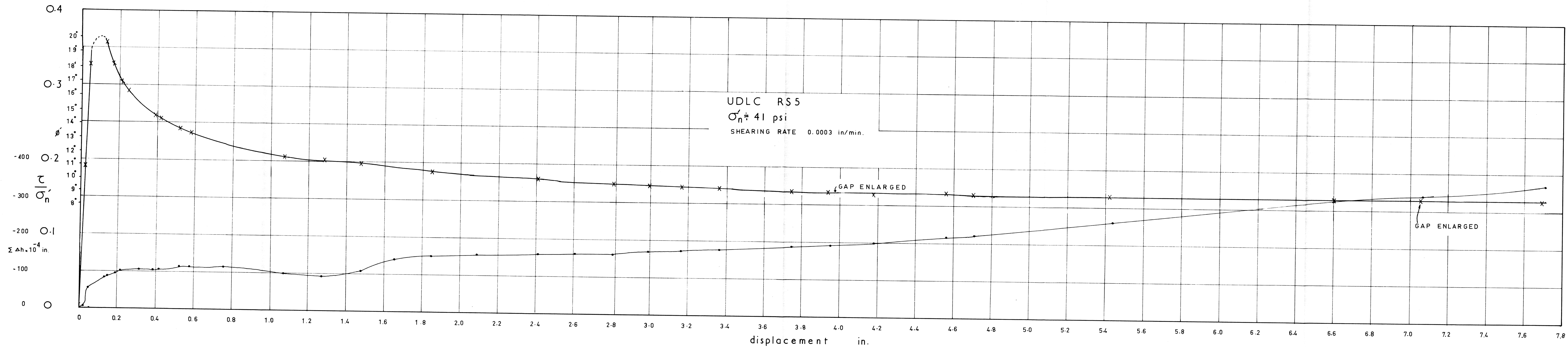


FIG. 4.4



Blue London Clay (Wraysbury)

FIG. 4.5.

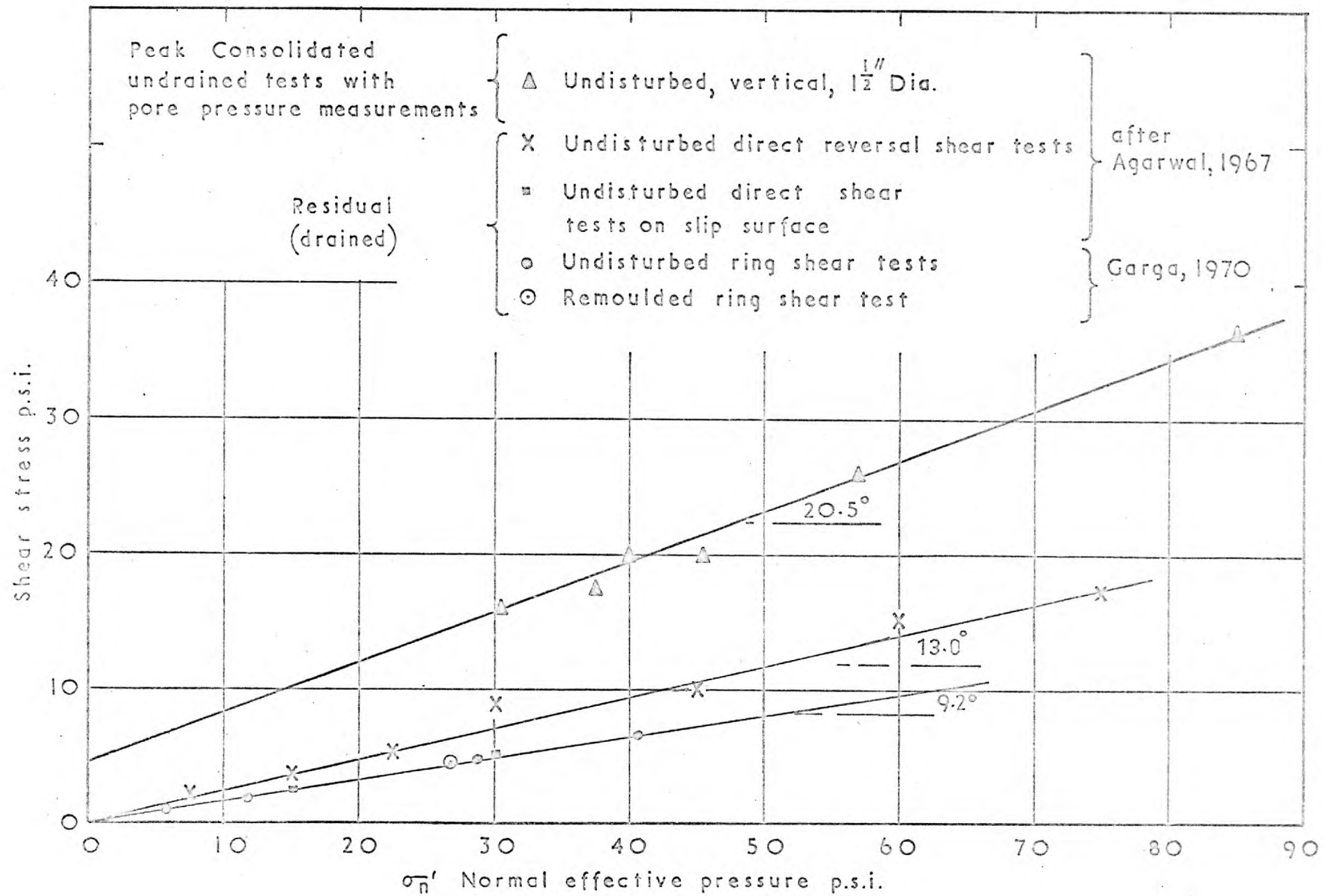
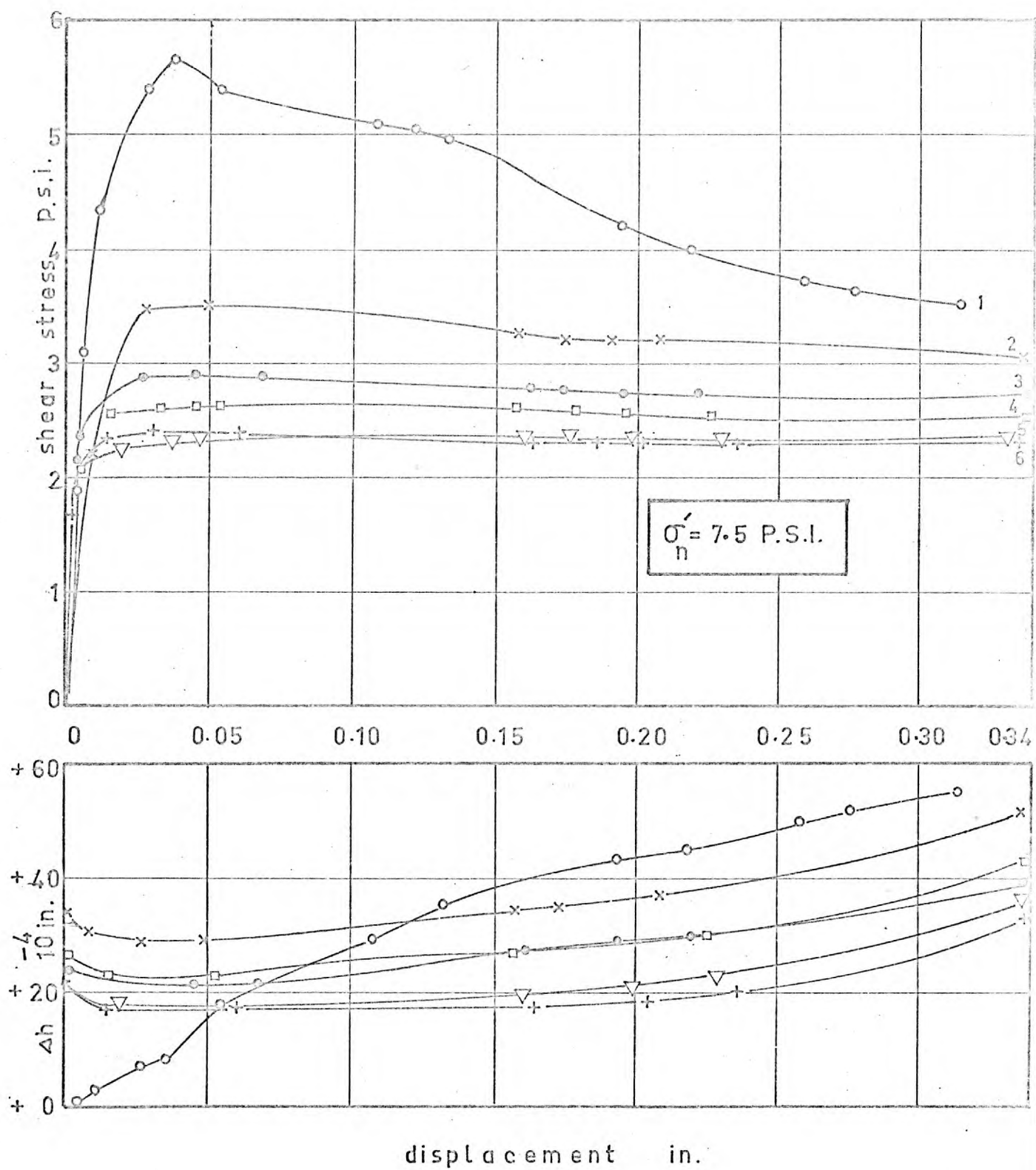


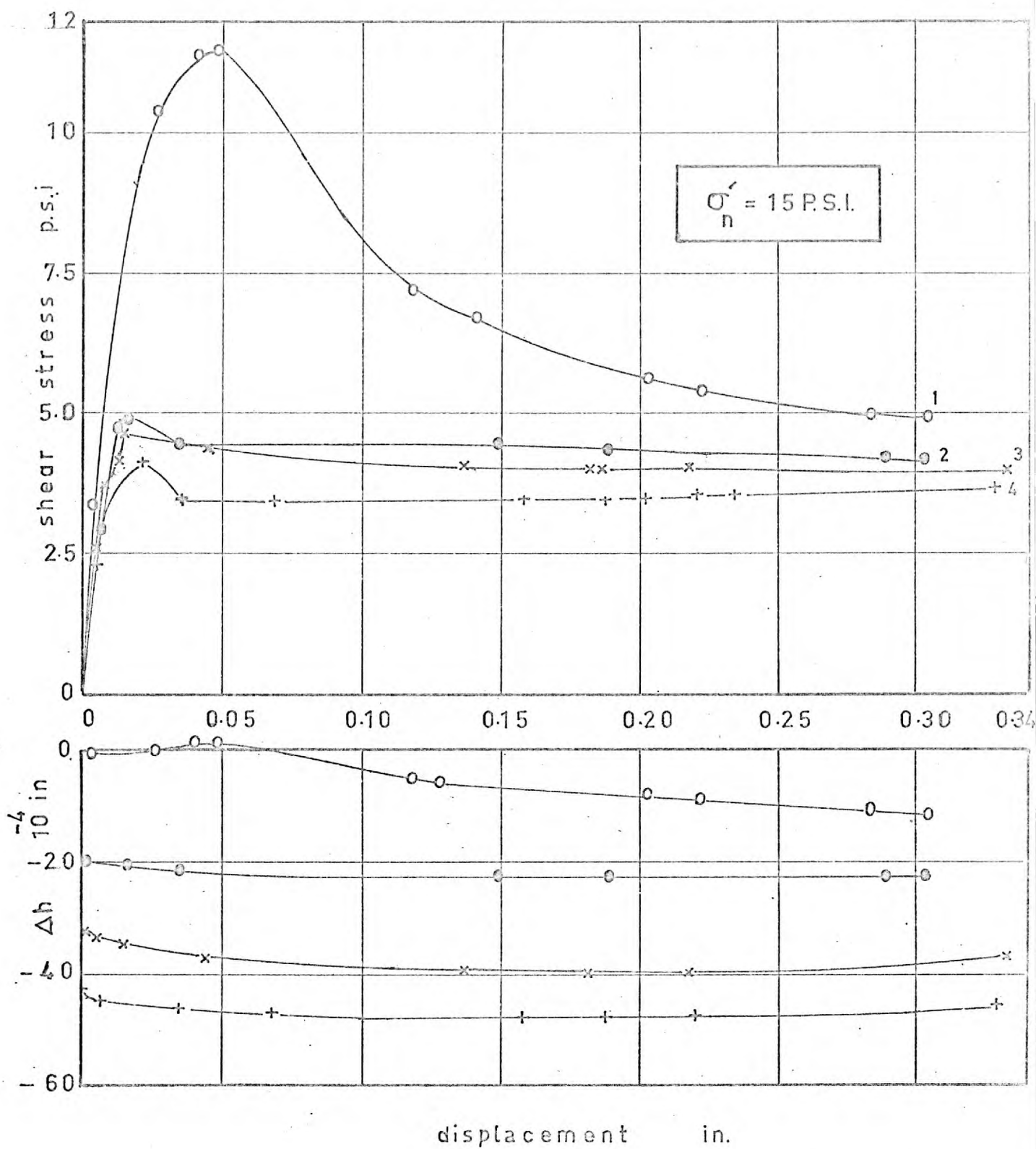
FIG.4.6 EFFECTIVE STRESS TESTS ON BLUE LONDON CLAY (WRAYSBURY)



HORIZONTAL D.S.B. TEST

FIG. 4.7

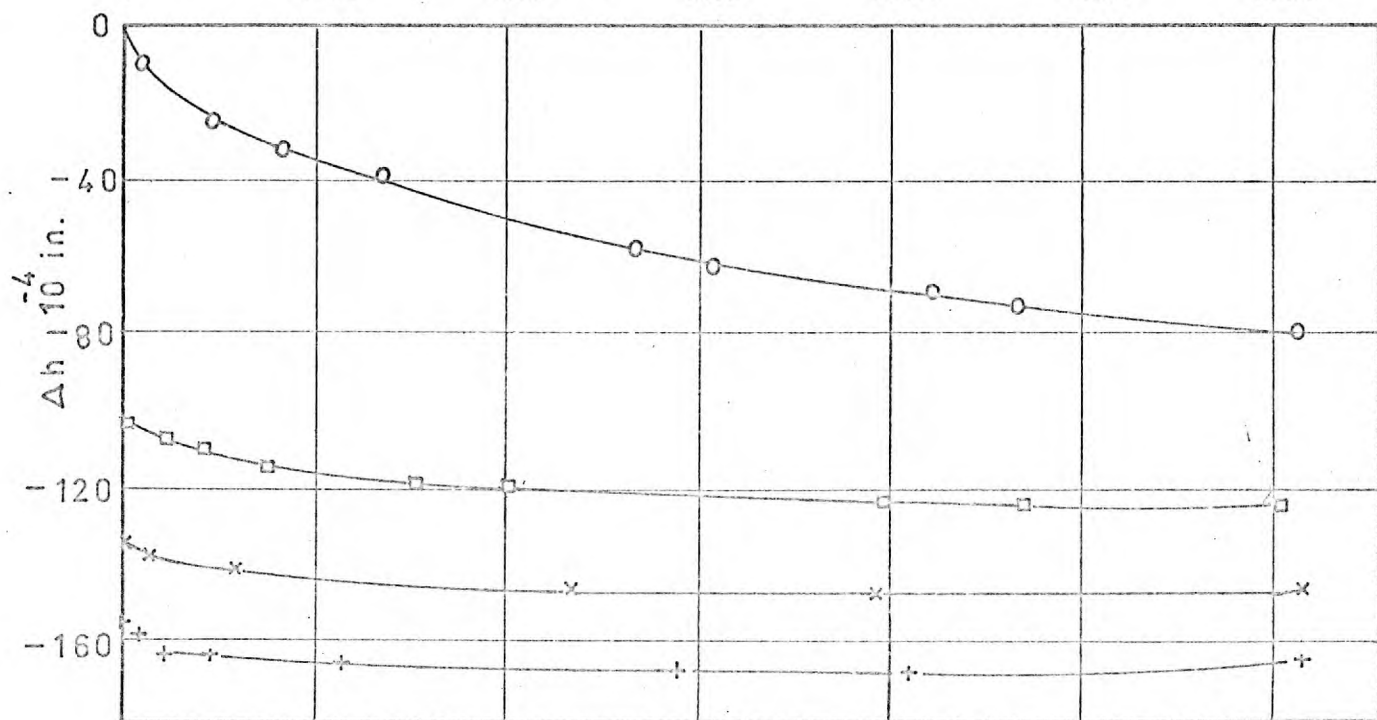
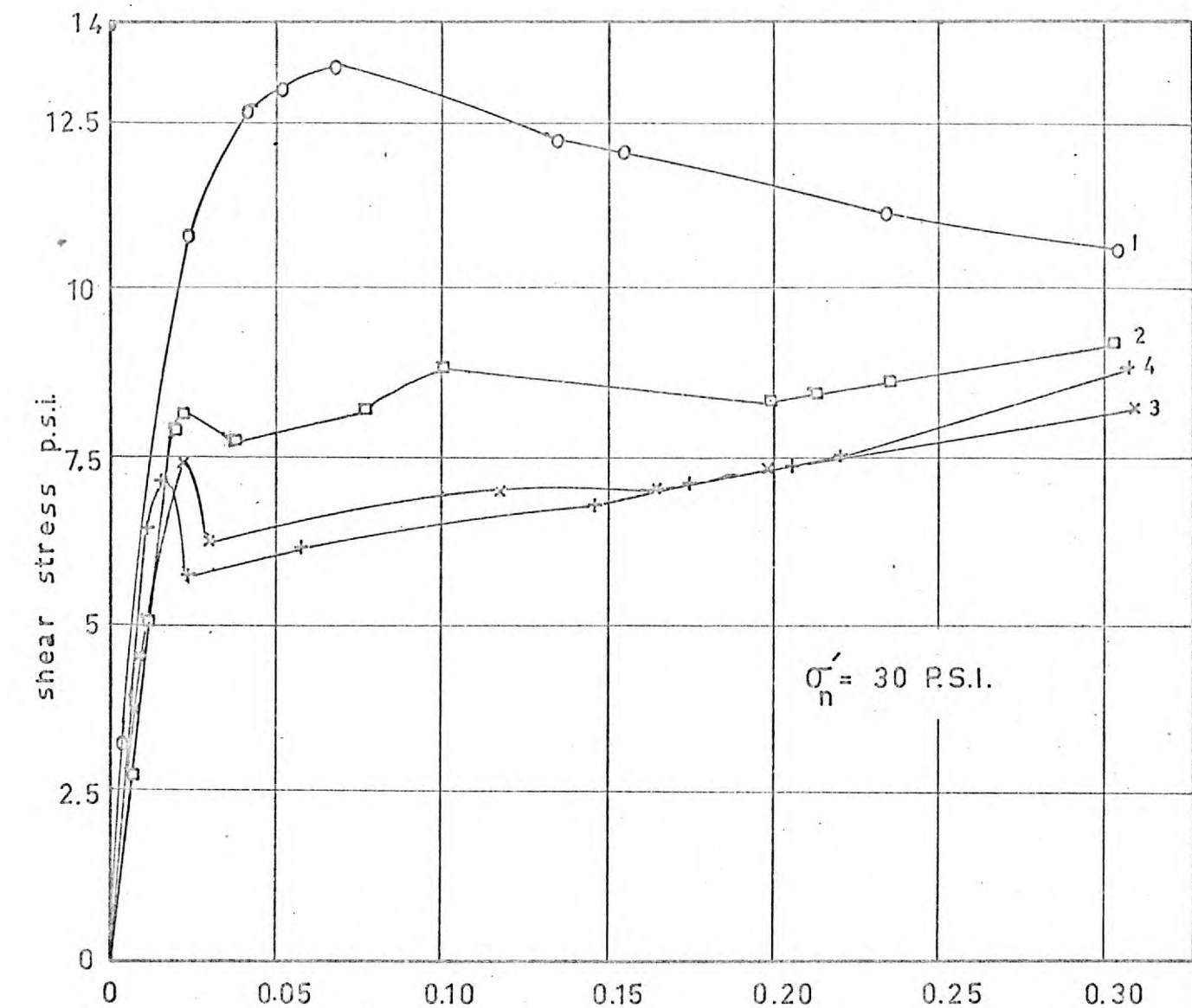
(Agarwal, 1967)



HORIZONTAL D.S.B. TEST

FIG. 4.8

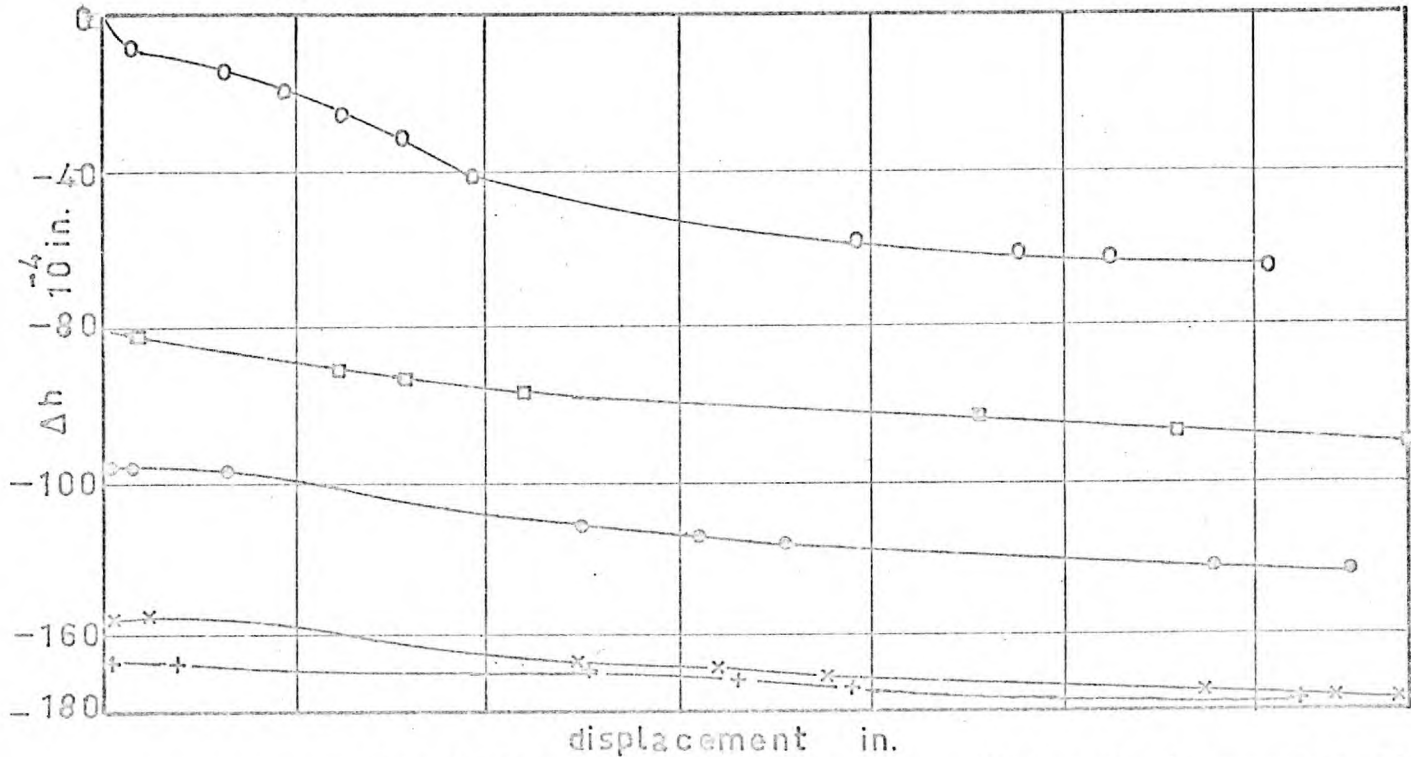
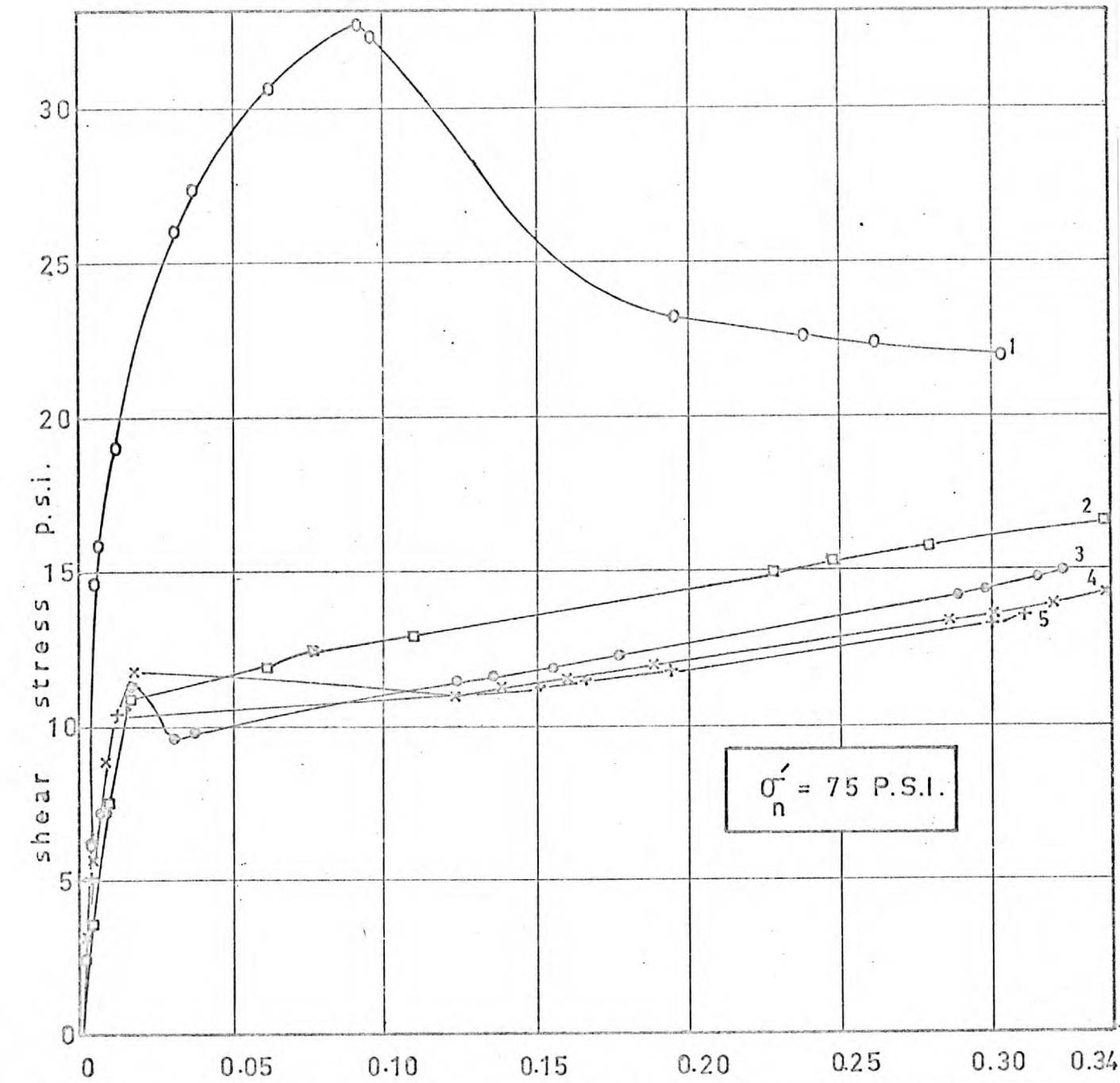
(after Agarwal, 1967)



displacement in.
HORIZONTAL D.S.B. TEST

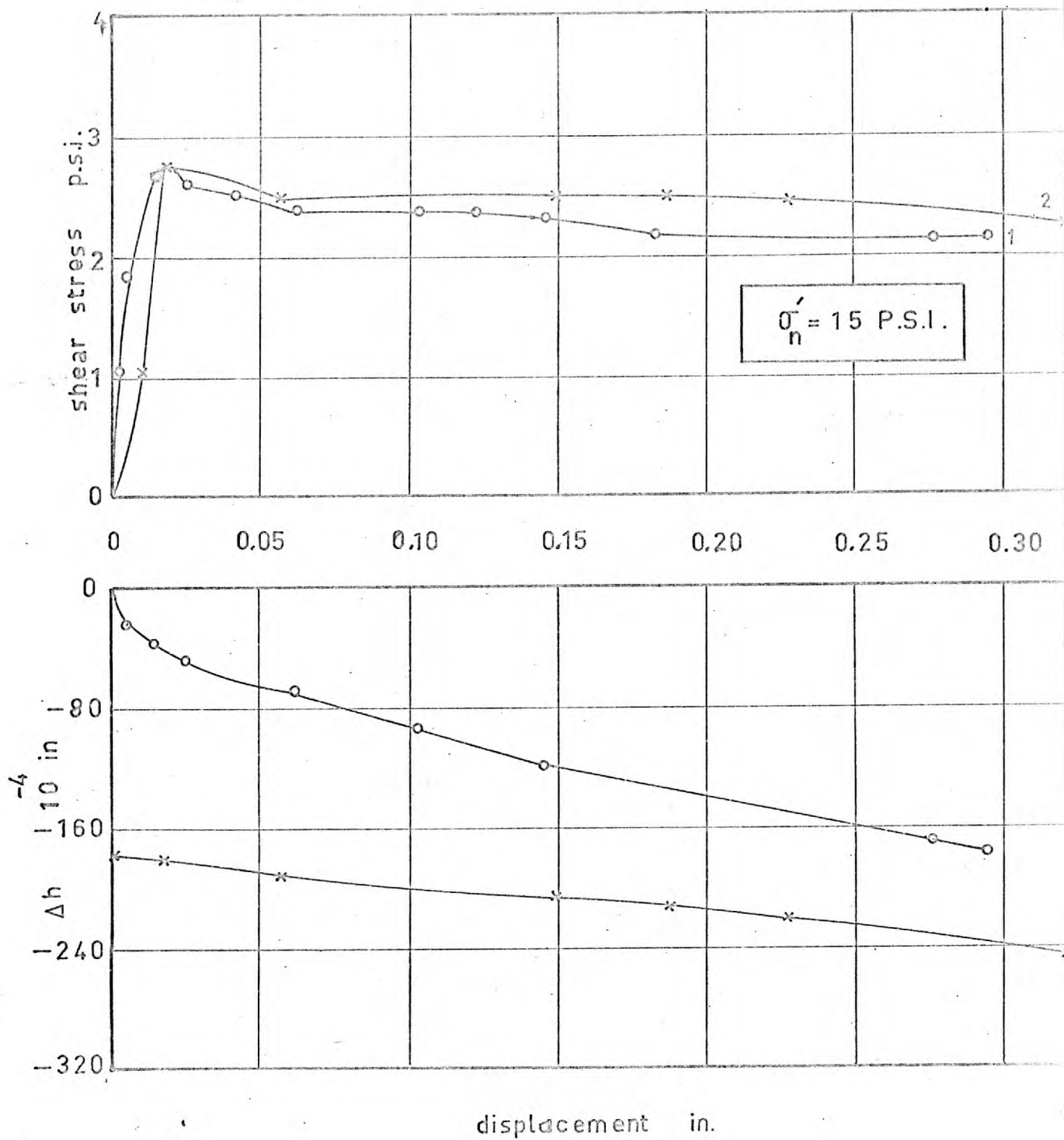
FIG. 4.9

(Agarwal 1967)



HORIZONTAL D.S.B. TEST

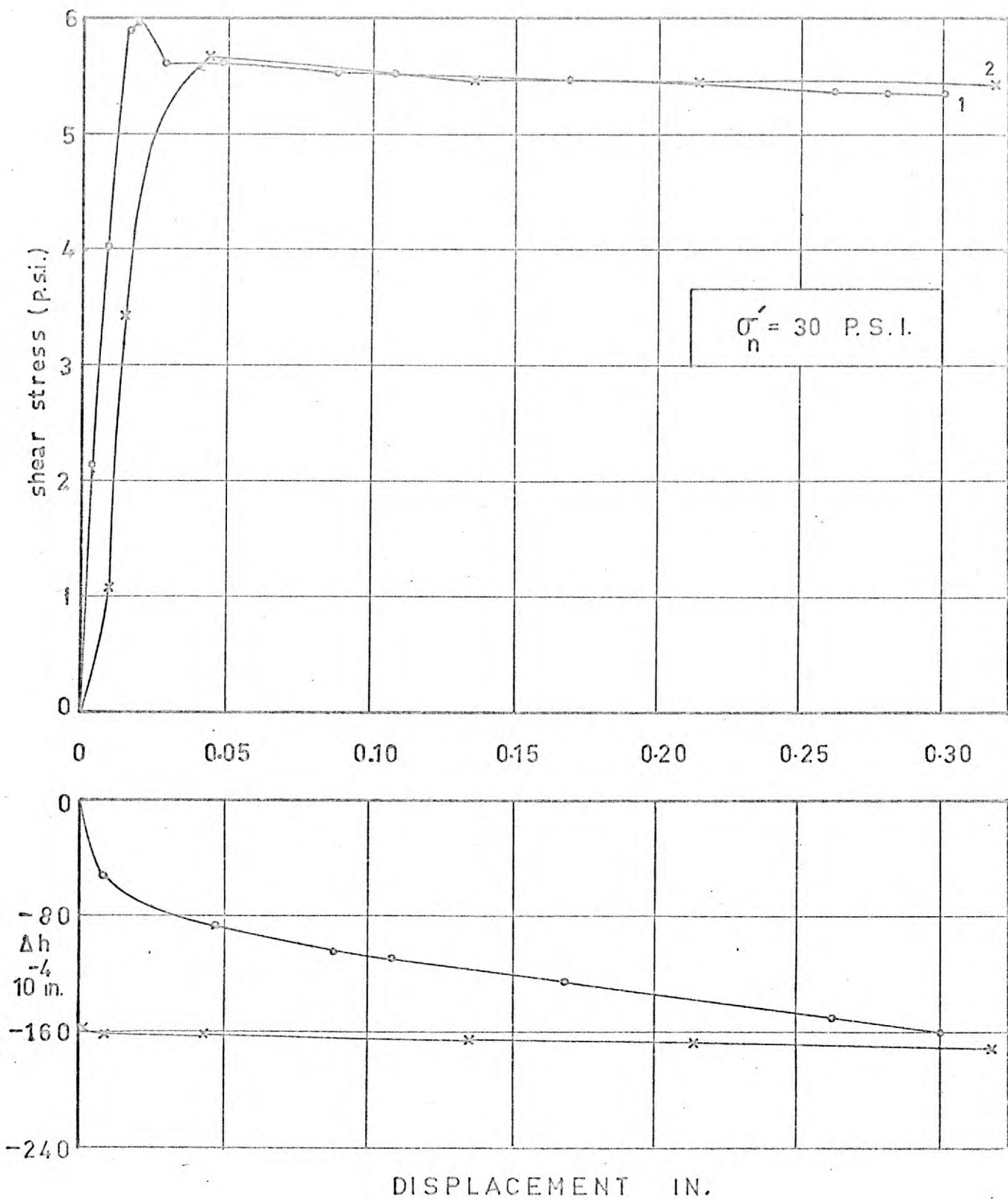
FIG. 4.10 (Agarwal, 1967)



D.S.B. TEST ON SLIP SURFACE

FIG. 4.11

(Agarwal 1967)



D.S.B. TEST ON SLIP SURFACE

FIG. 4.12

(after Agarwal 1967)

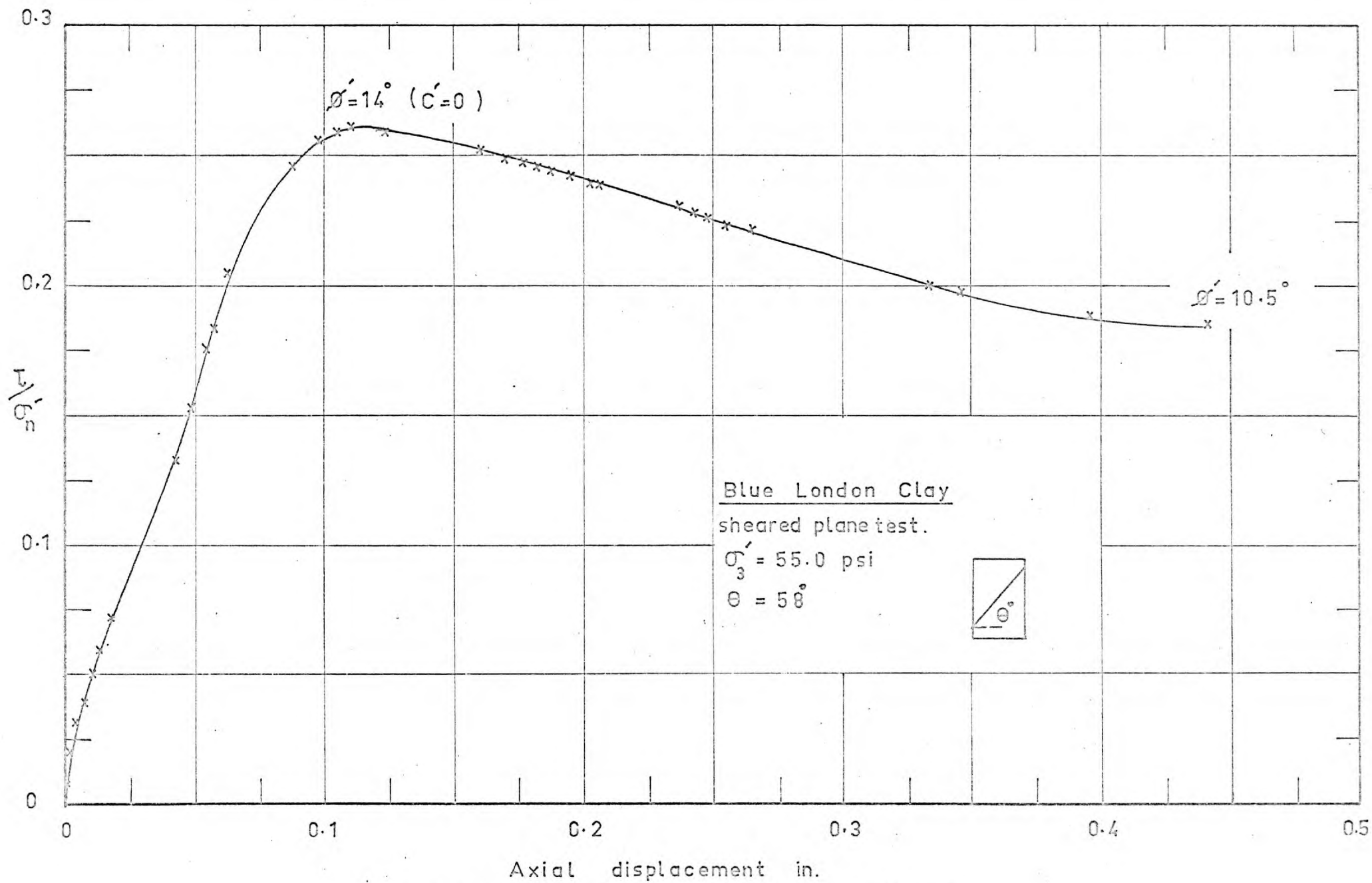
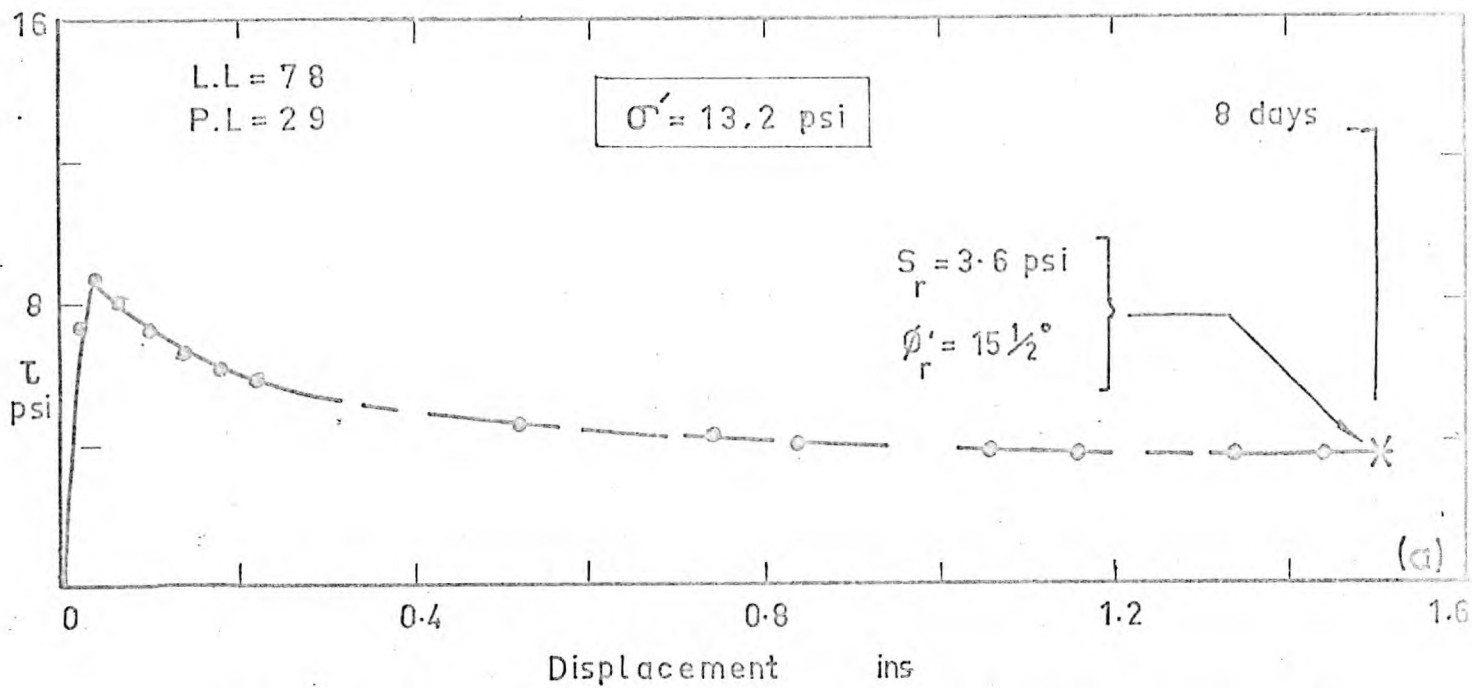


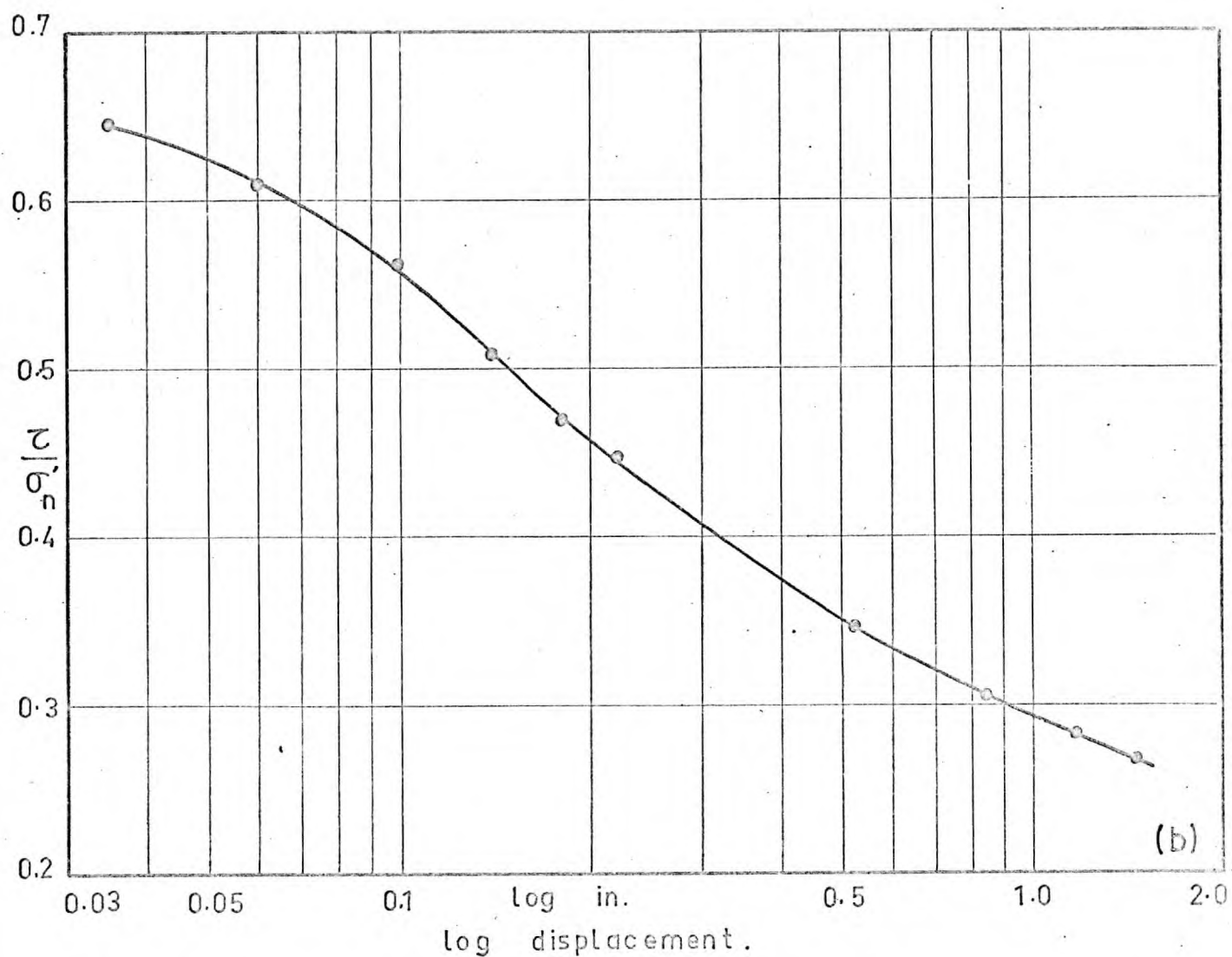
FIG.4.13 τ/σ'_n AGAINST DISPLACEMENT CURVE



HENDON

Large strain test

(fig.19: Skempton, 1964)



Development of residual strength with displacement.

FIG.4.14

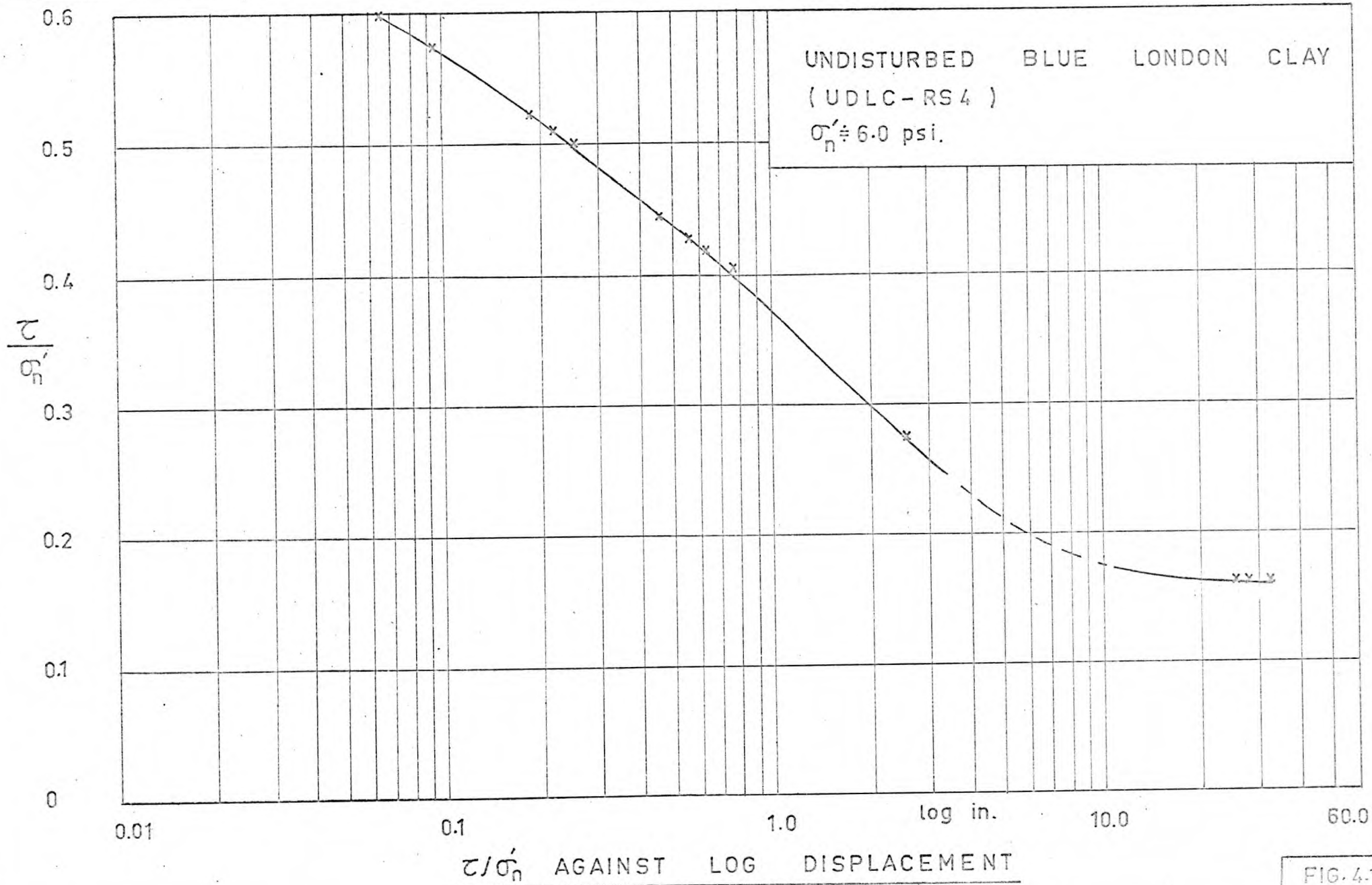


FIG. 4.15

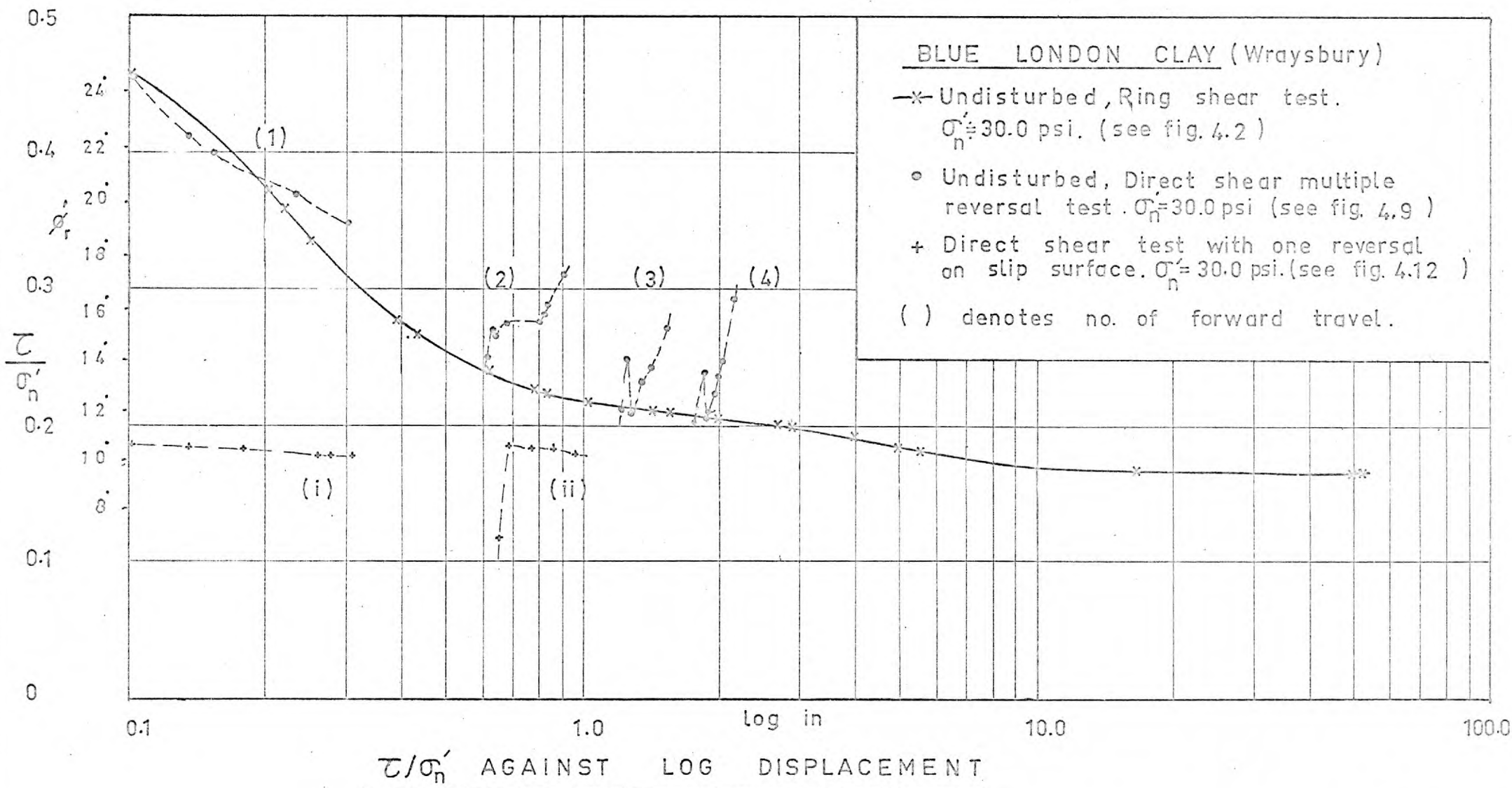


FIG. 4.16

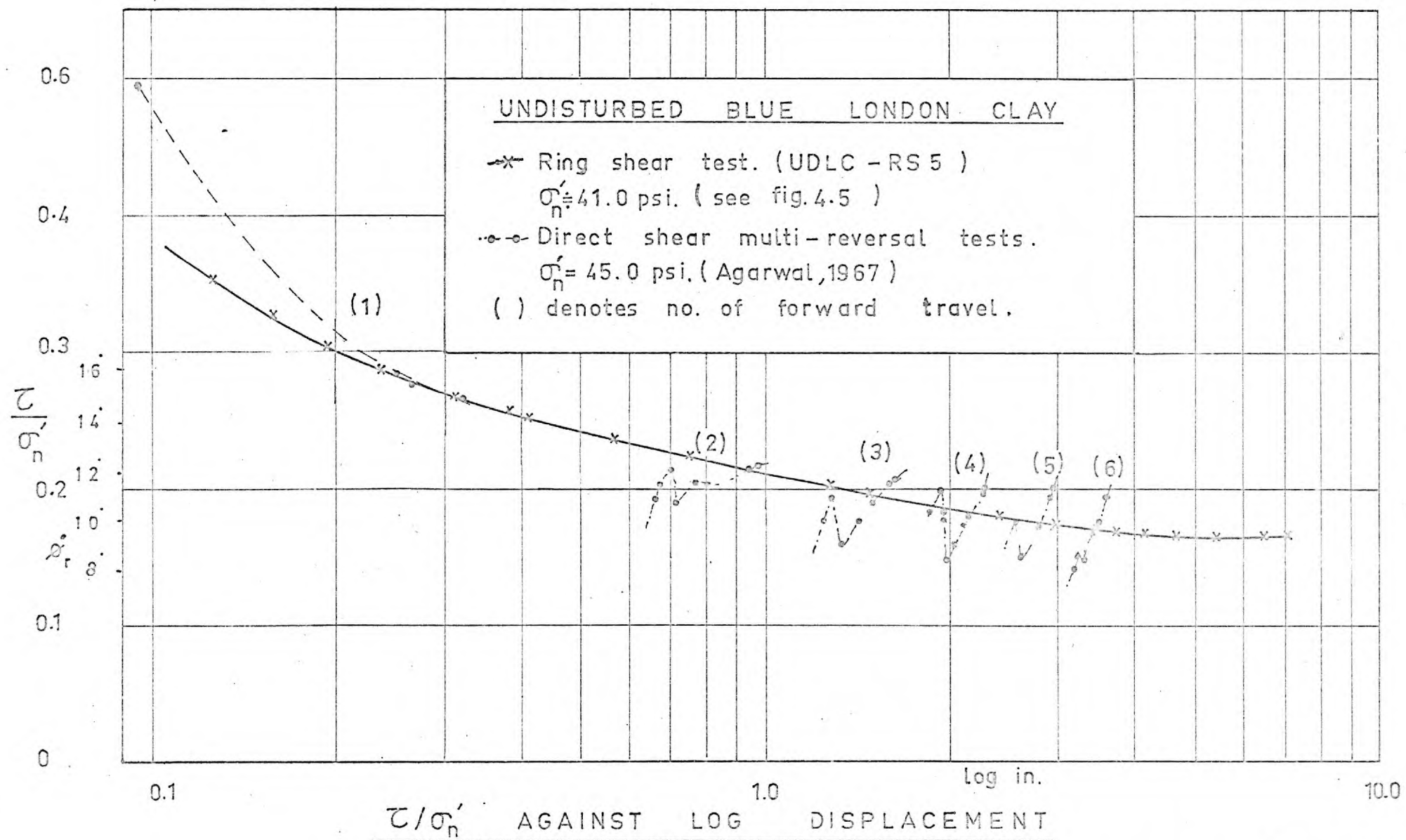
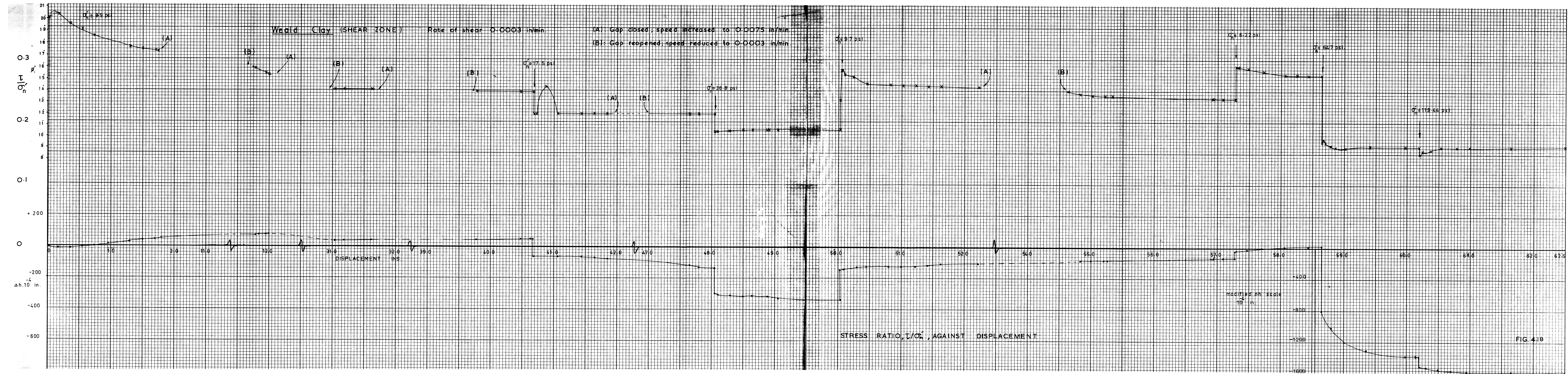
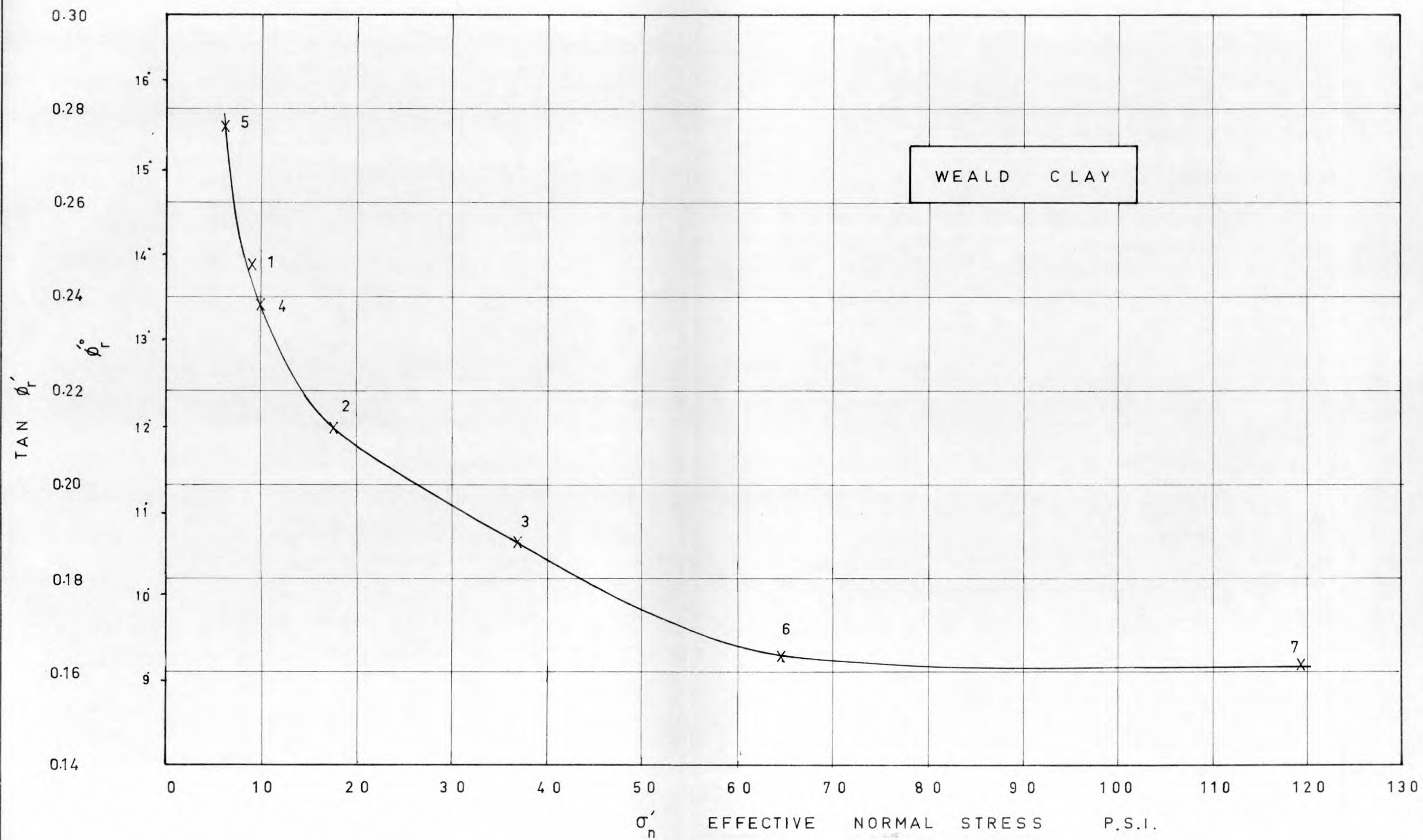
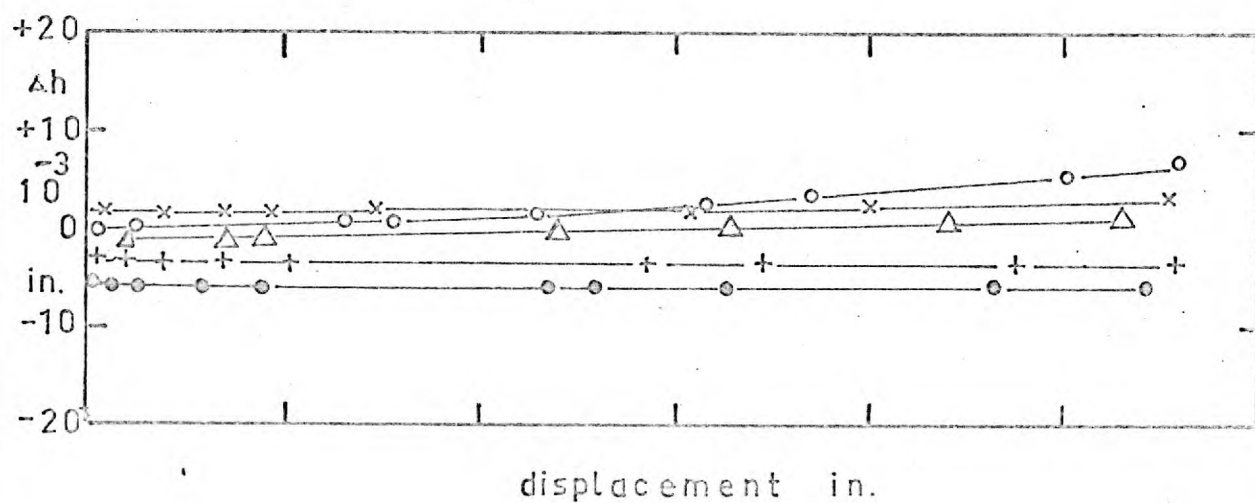
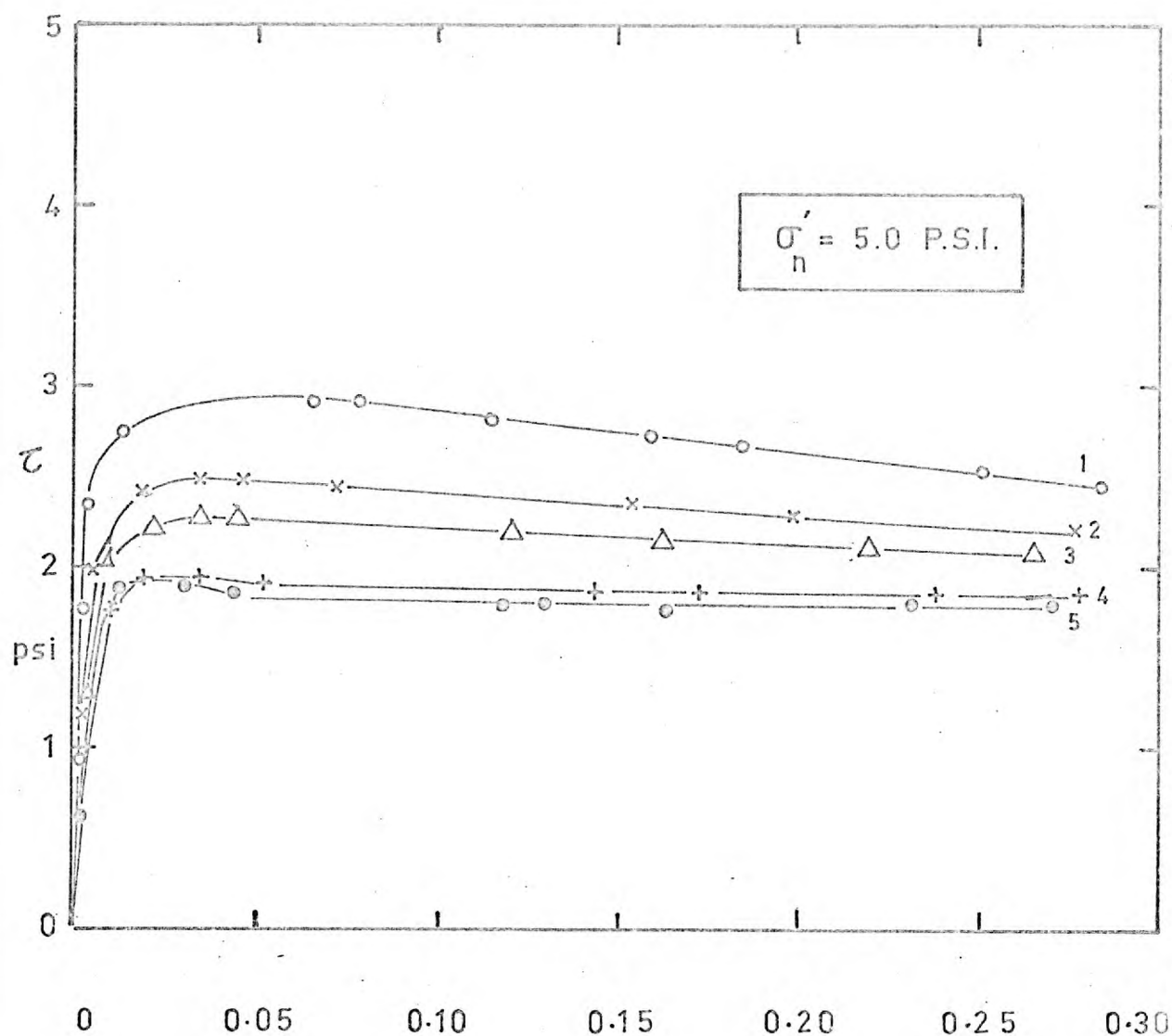


FIG. 4.17





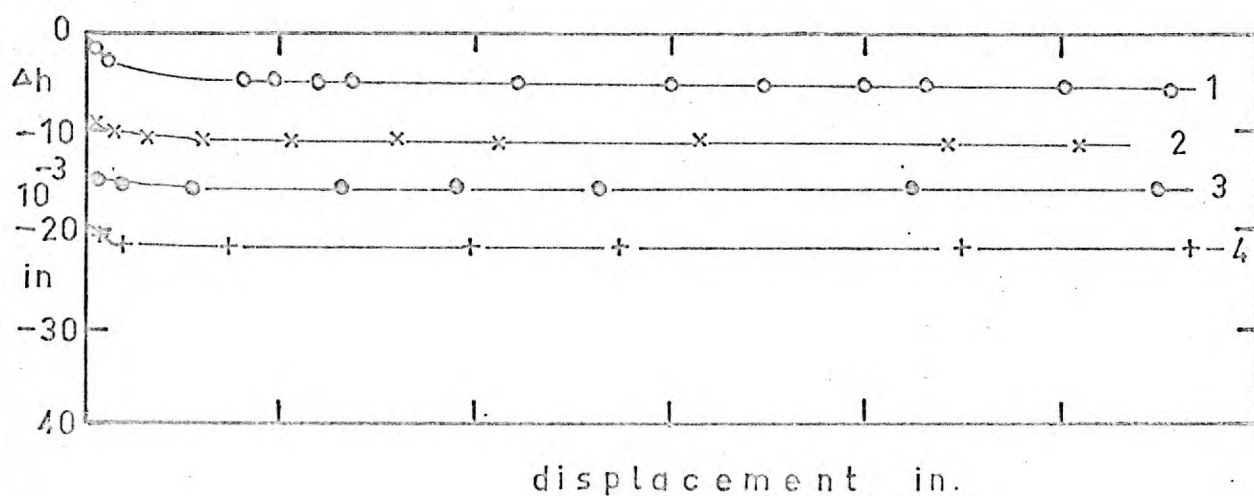
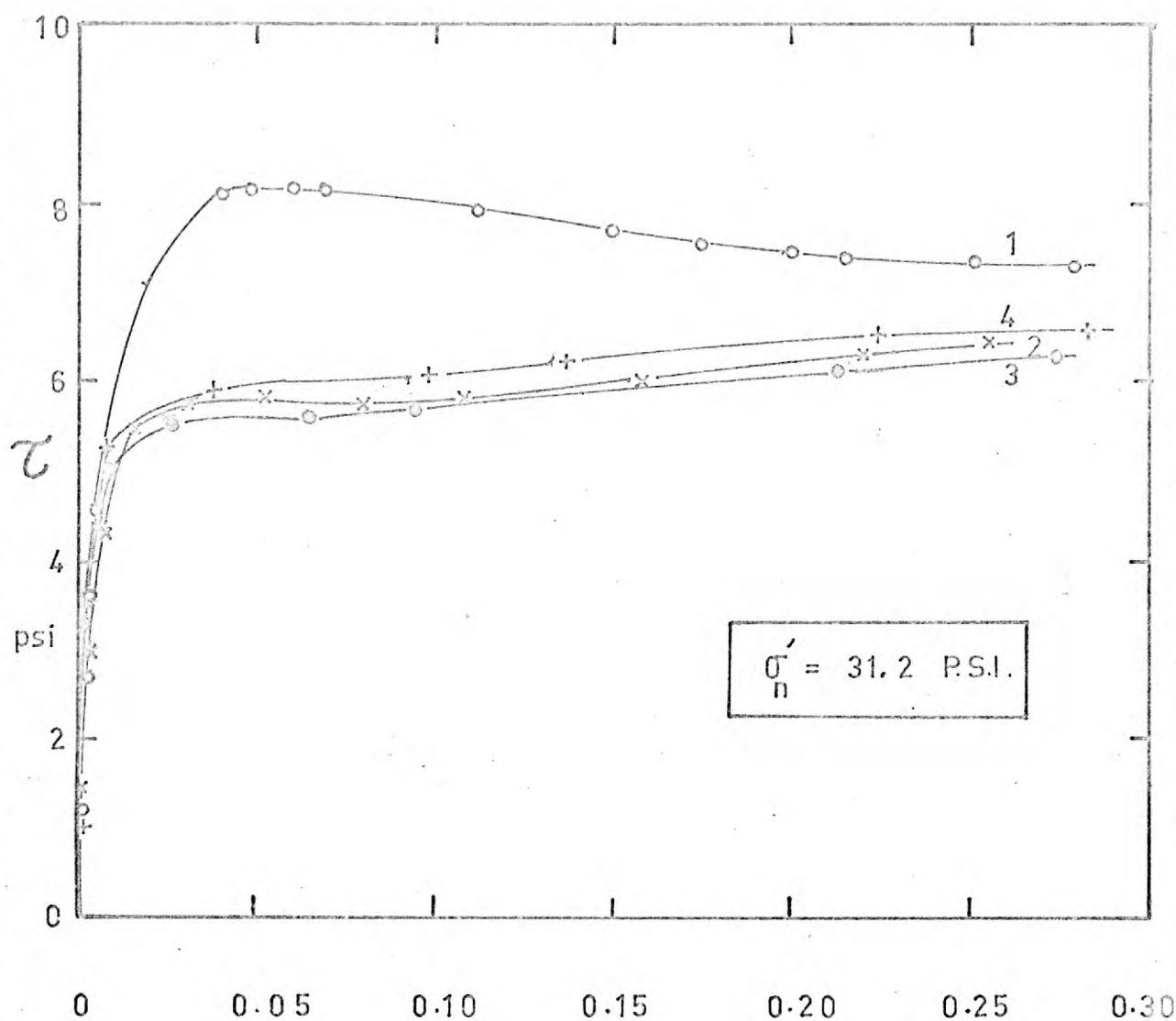
VARIATION OF TAN. ϕ_r' WITH σ_n' FIG. 4.19.



WEALD CLAY

FIG. 4.20

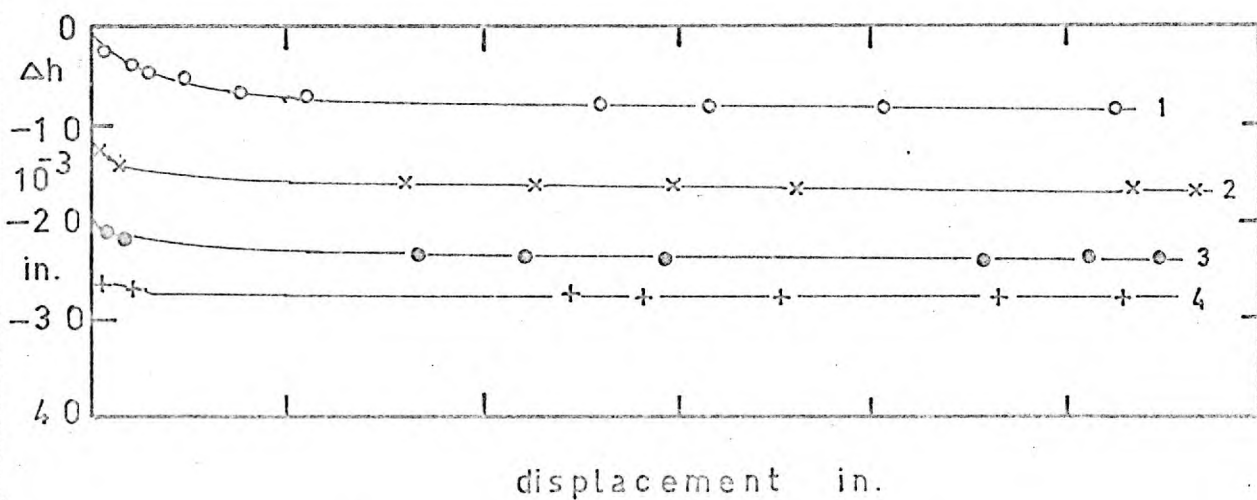
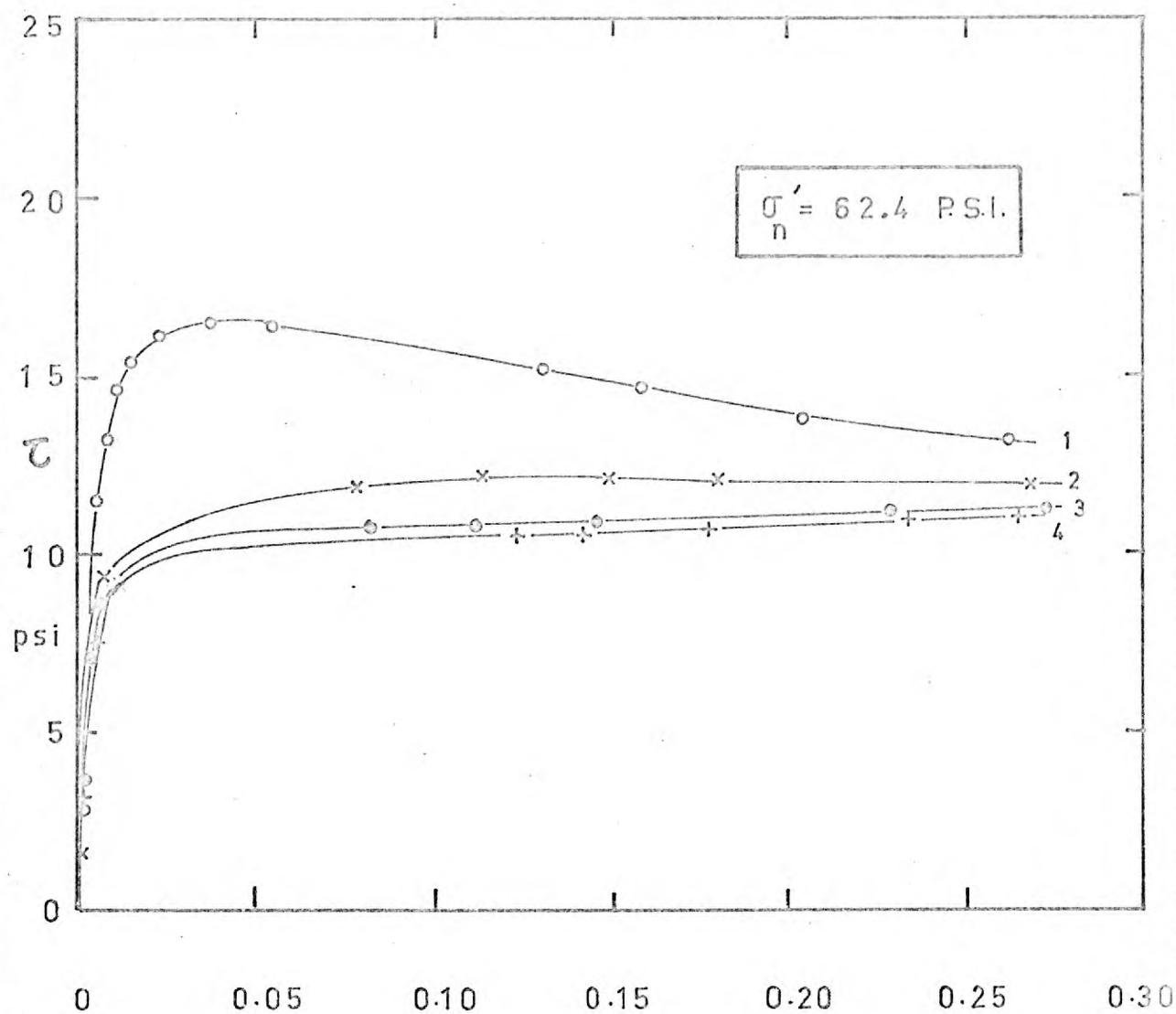
(After Petley, 1969)



WEALD CLAY

FIG. 4.21

(After Petley, 1969)



WEALD CLAY

FIG. 4.22

(After Petley, 1969)

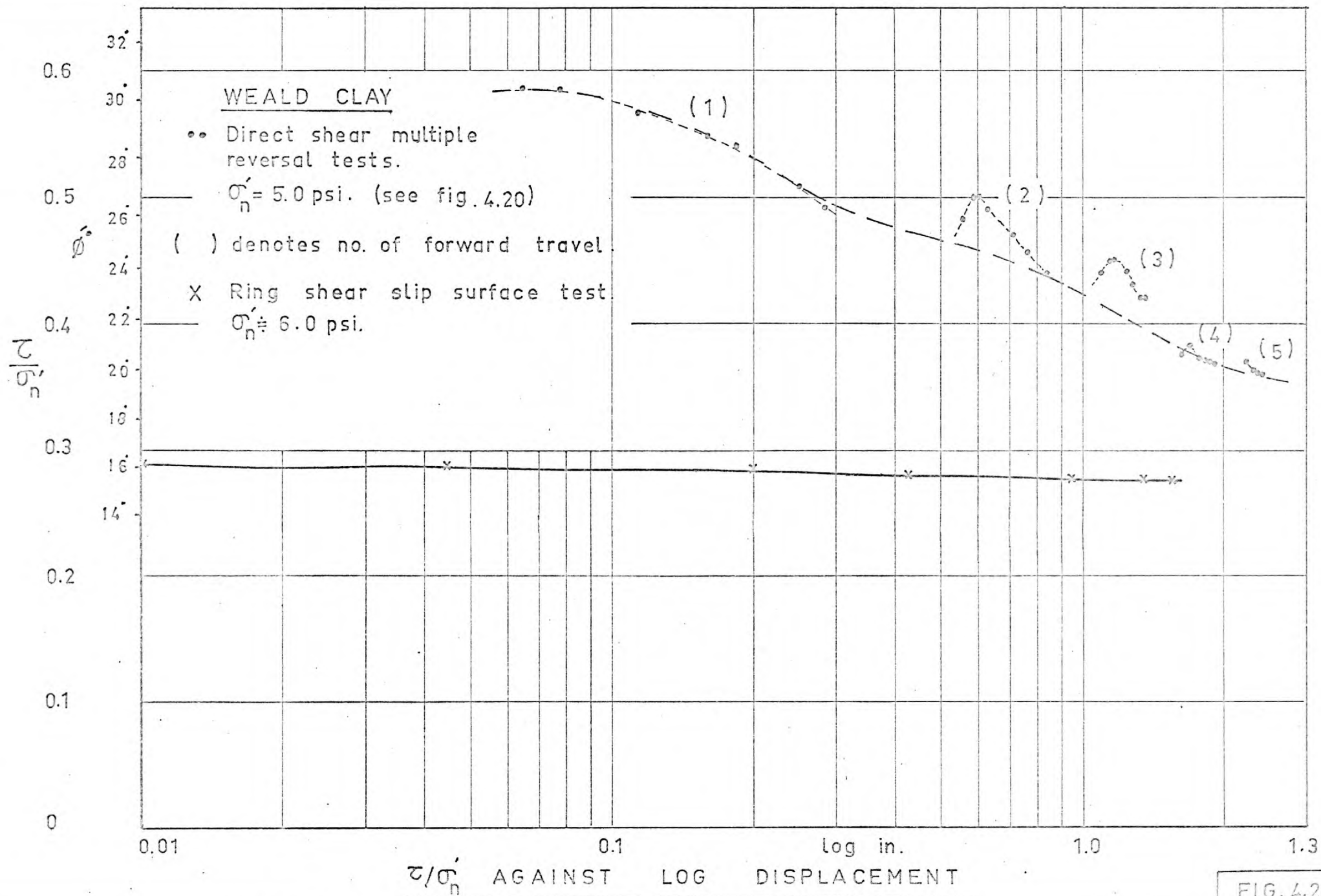


FIG. 4.23

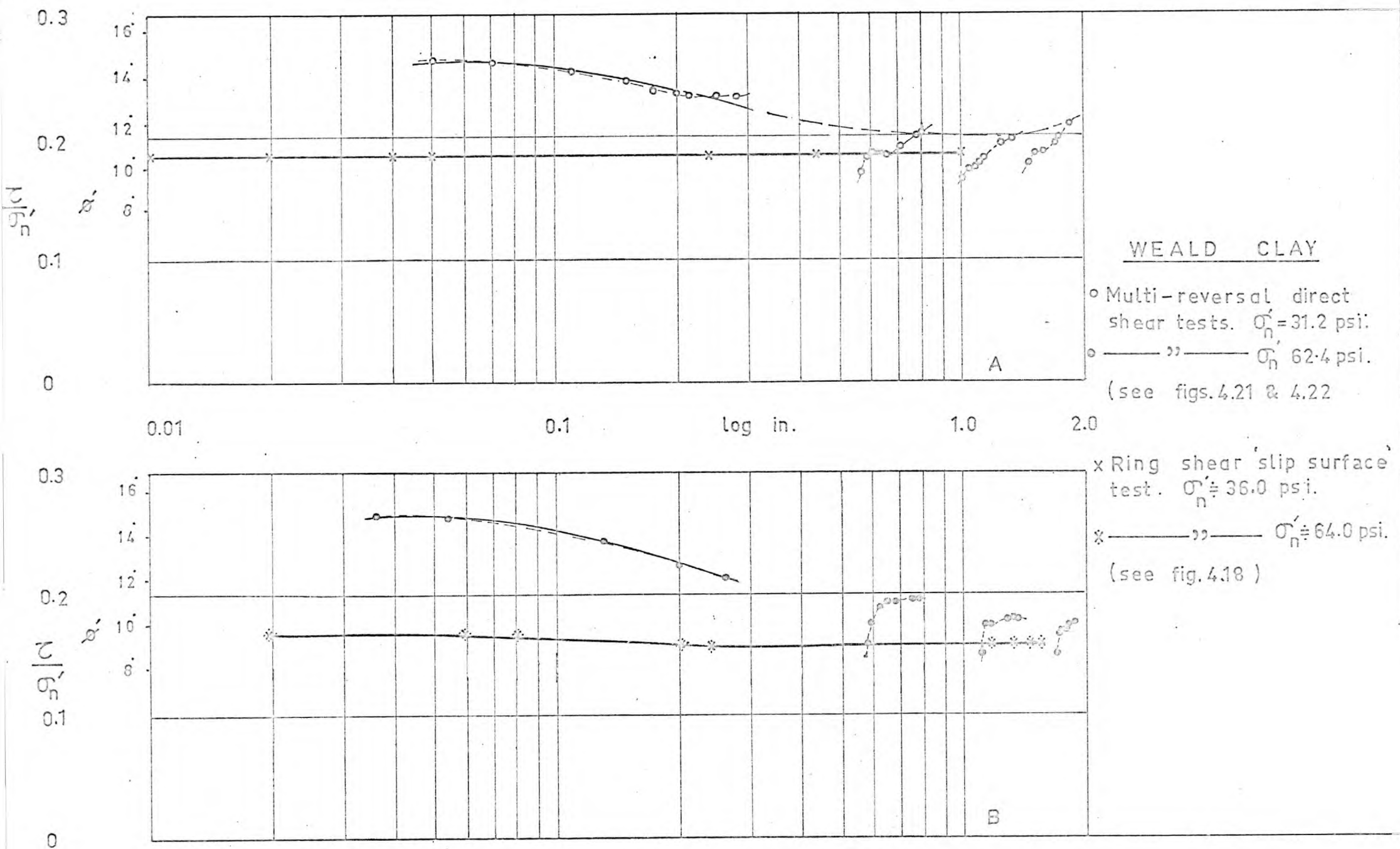


FIG. 4.24: τ/σ'_n AGAINST LOG DISPLACEMENT

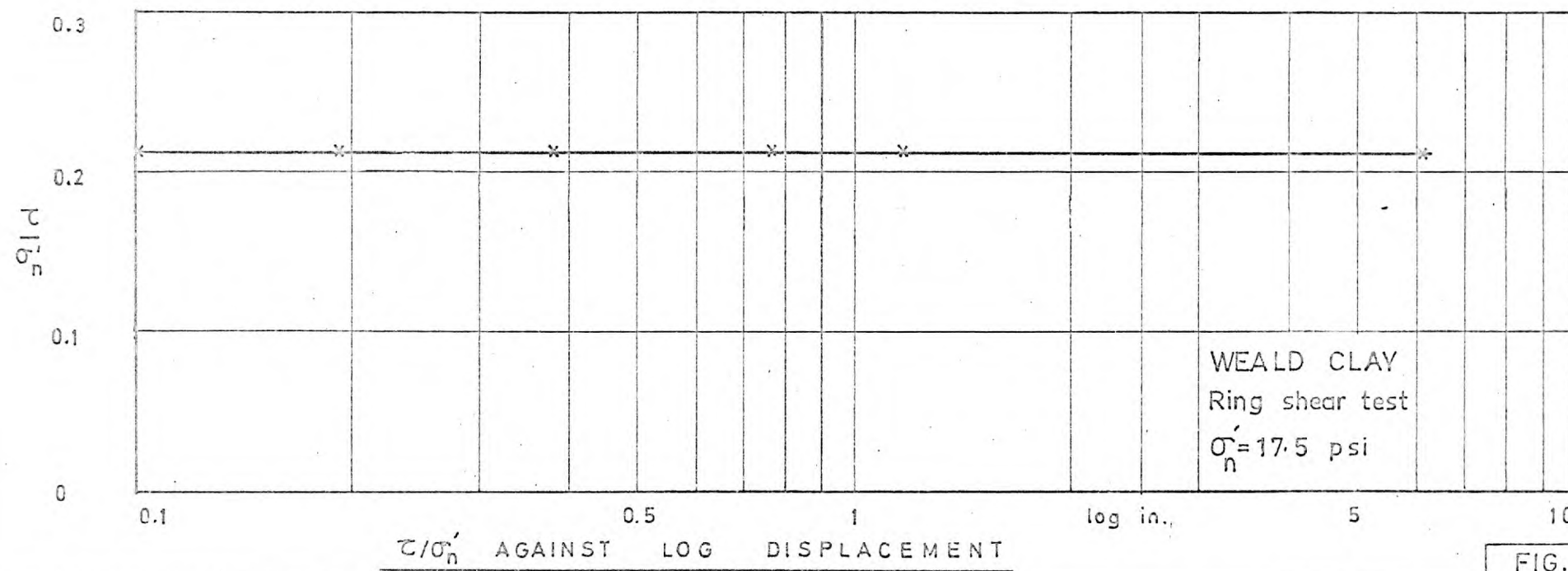
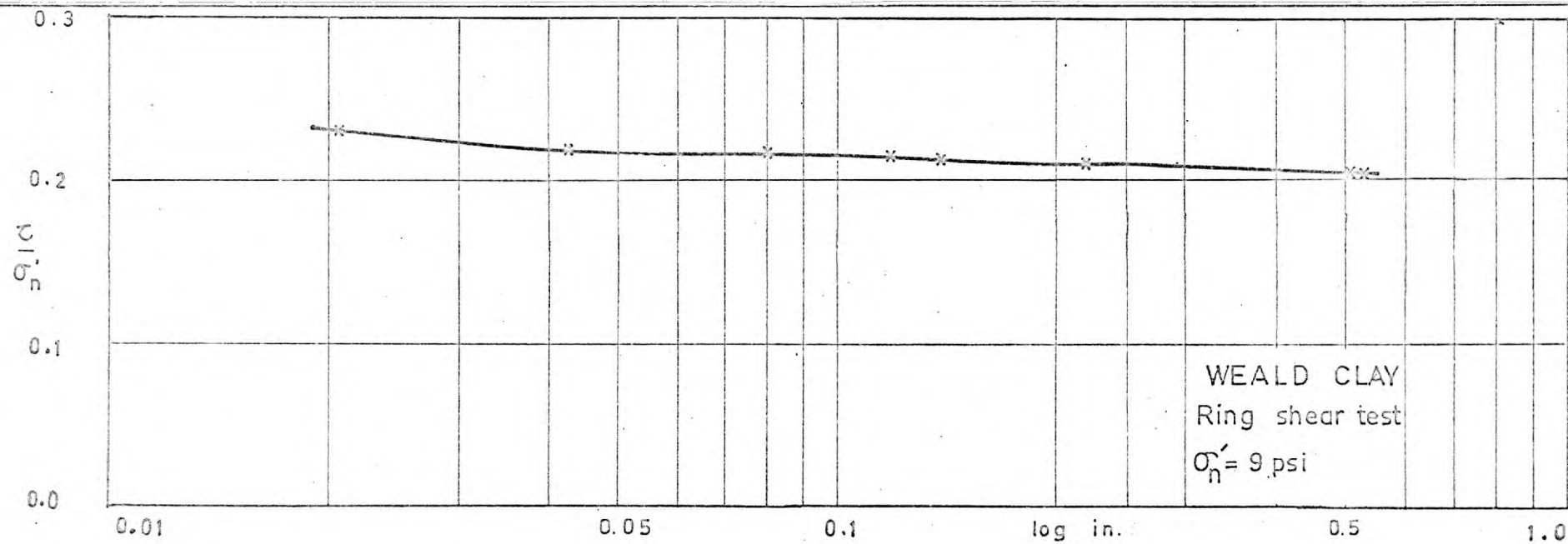
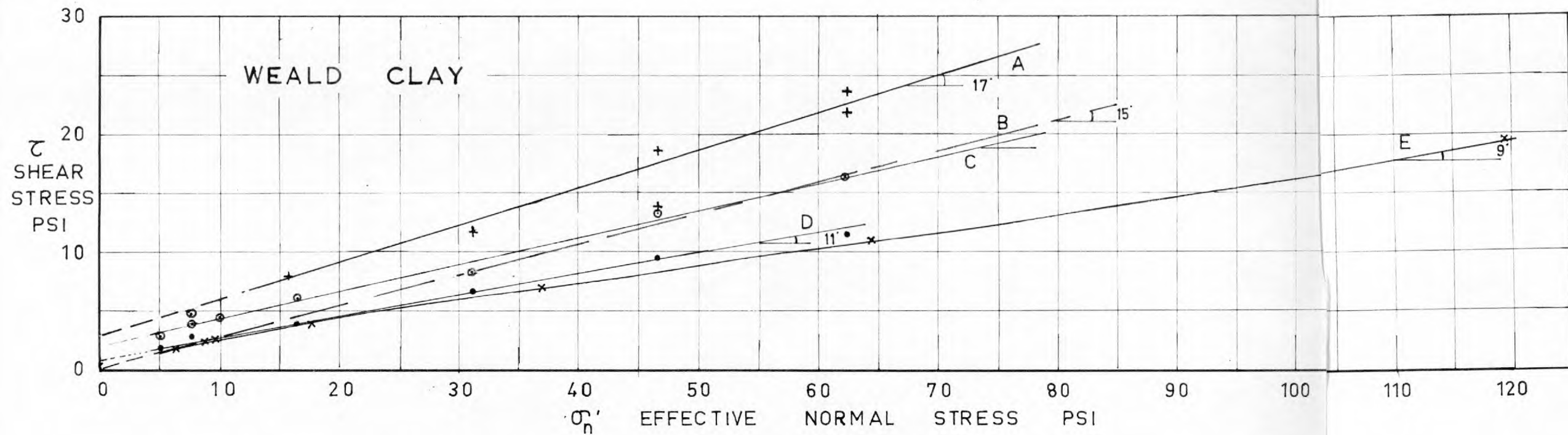


FIG.4.25



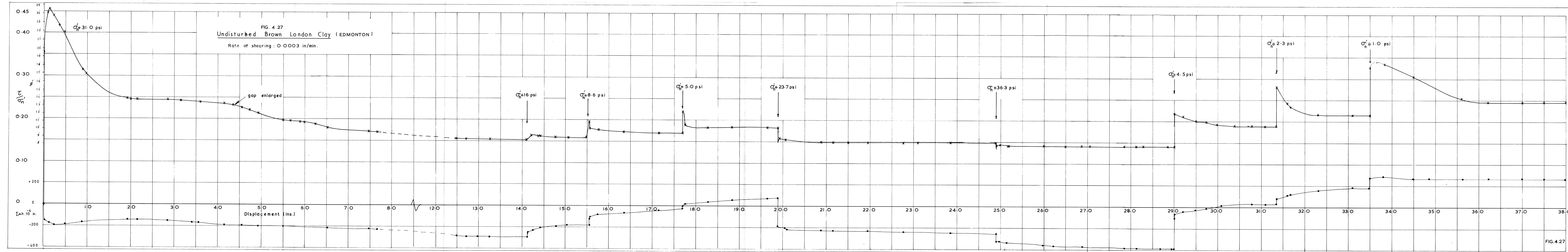
Direct shear box tests

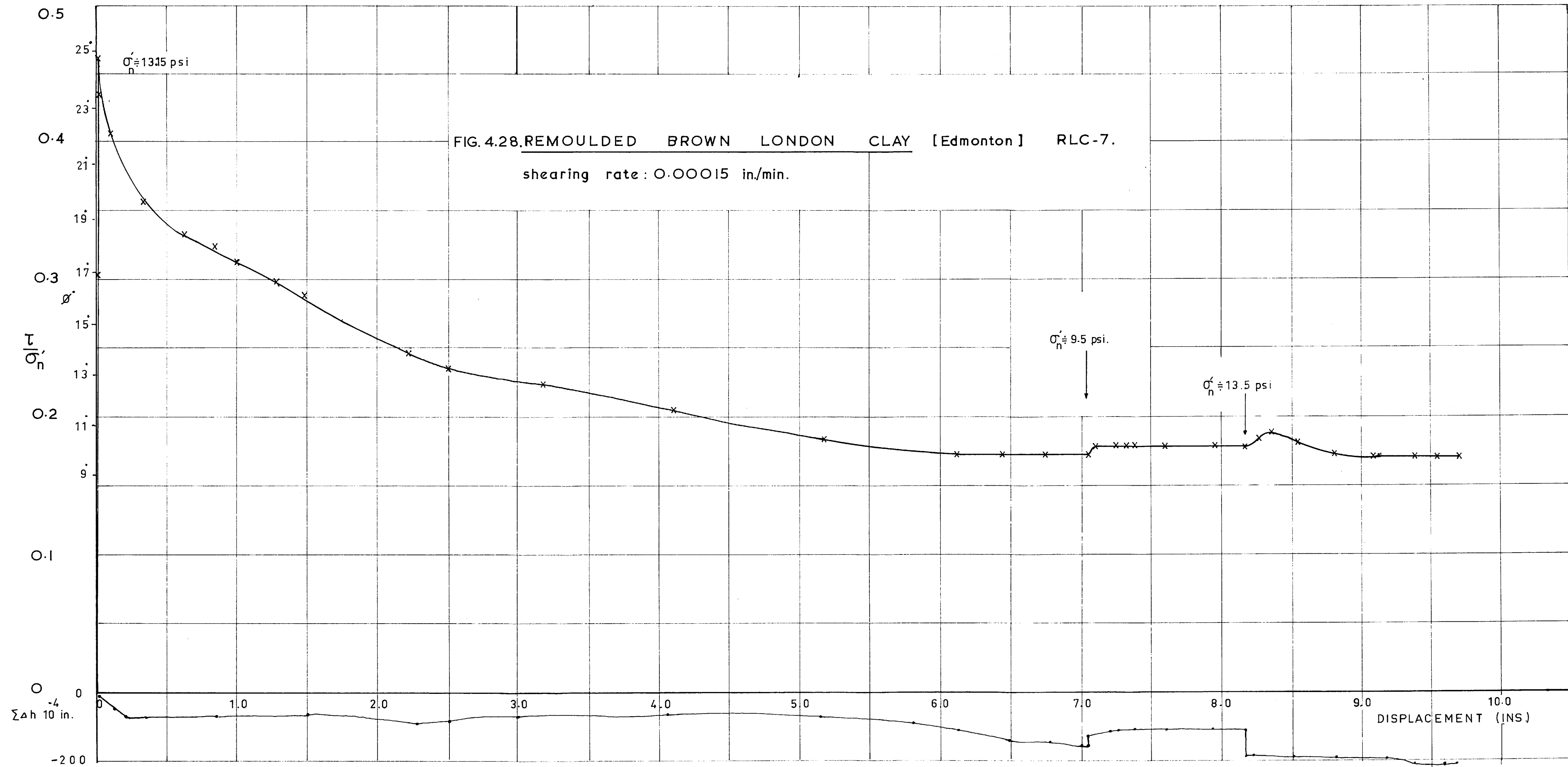
- | | | | | |
|-----|---|--|---|---------------------------------|
| + | A | Peak - intact clay from Arlington. | } | Petley,
(1969) |
| o | C | Peak - shear zone " " | | |
| • | D | Residual - " " " " | | |
| --- | B | Residual - weathered clay from Sevenoaks | } | Skempton
& Petley,
(1967) |

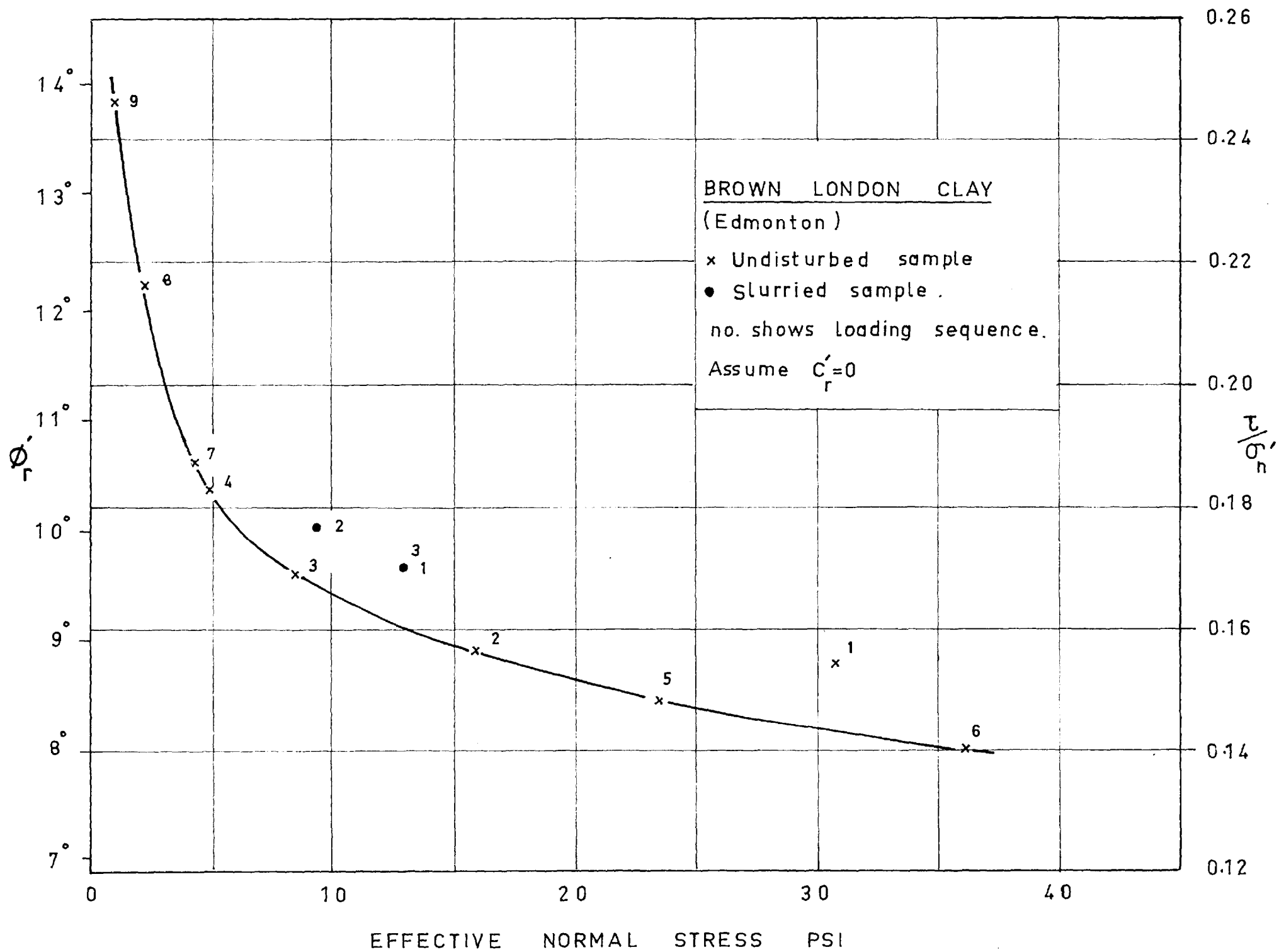
Ring shear tests

- | | | |
|---|---|---|
| x | E | Residual - shear zone from Arlington |
| | | $\phi'_r = 9^\circ$; $c'_r \approx 1$ psi for $\sigma'_n = 10 - 120$ psi |
| | | $\phi'_r = 15^\circ$; $c'_r = 0$ " $\sigma'_n = 5 - 10$ psi |

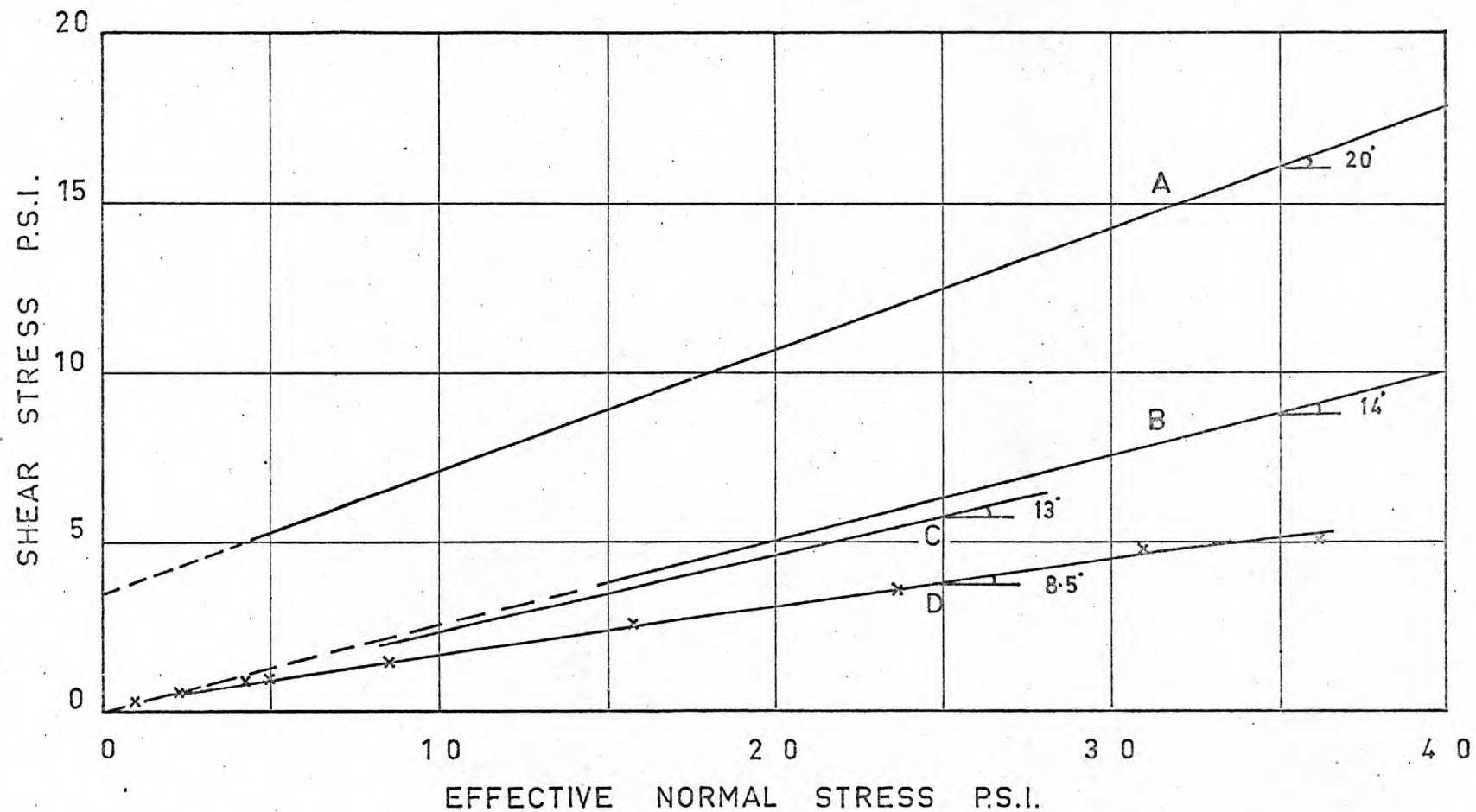
EFFECTIVE STRESS TESTS ON WEALD CLAY







Variation of ϕ_r' with effective normal stress

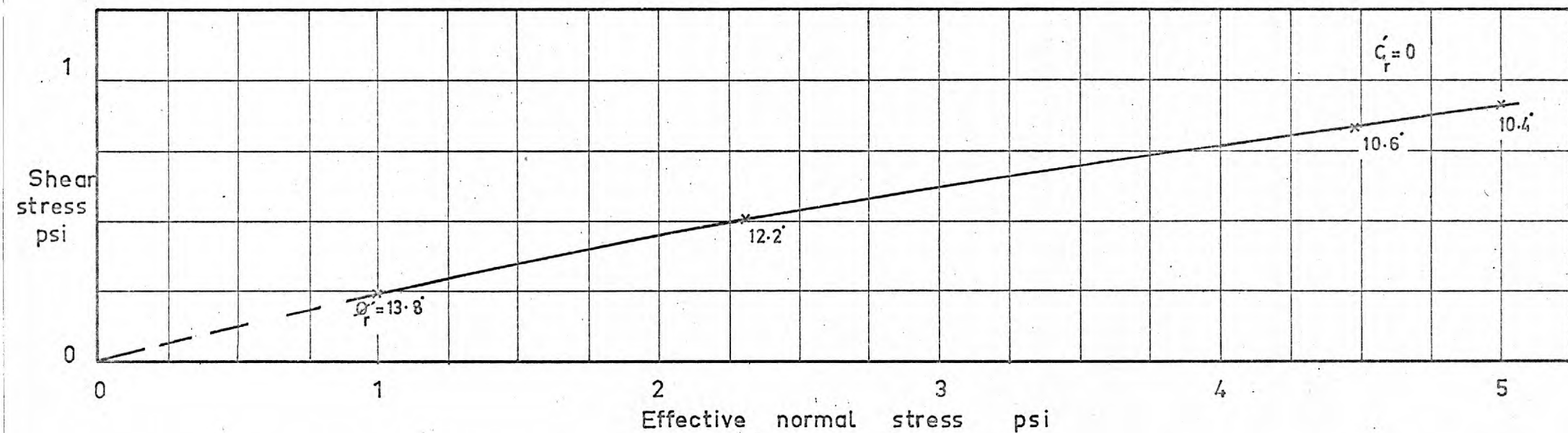


A and B : Average of 17 drained peak and residual strength tests respectively, in direct shear box on brown London Clay from Hendon, Herne Bay and Brentwood.

C : Direct shear residual tests on brown London Clay, Edmonton. (Petley, 1969).

x, D : Same clay as C but tested in ring shear apparatus.

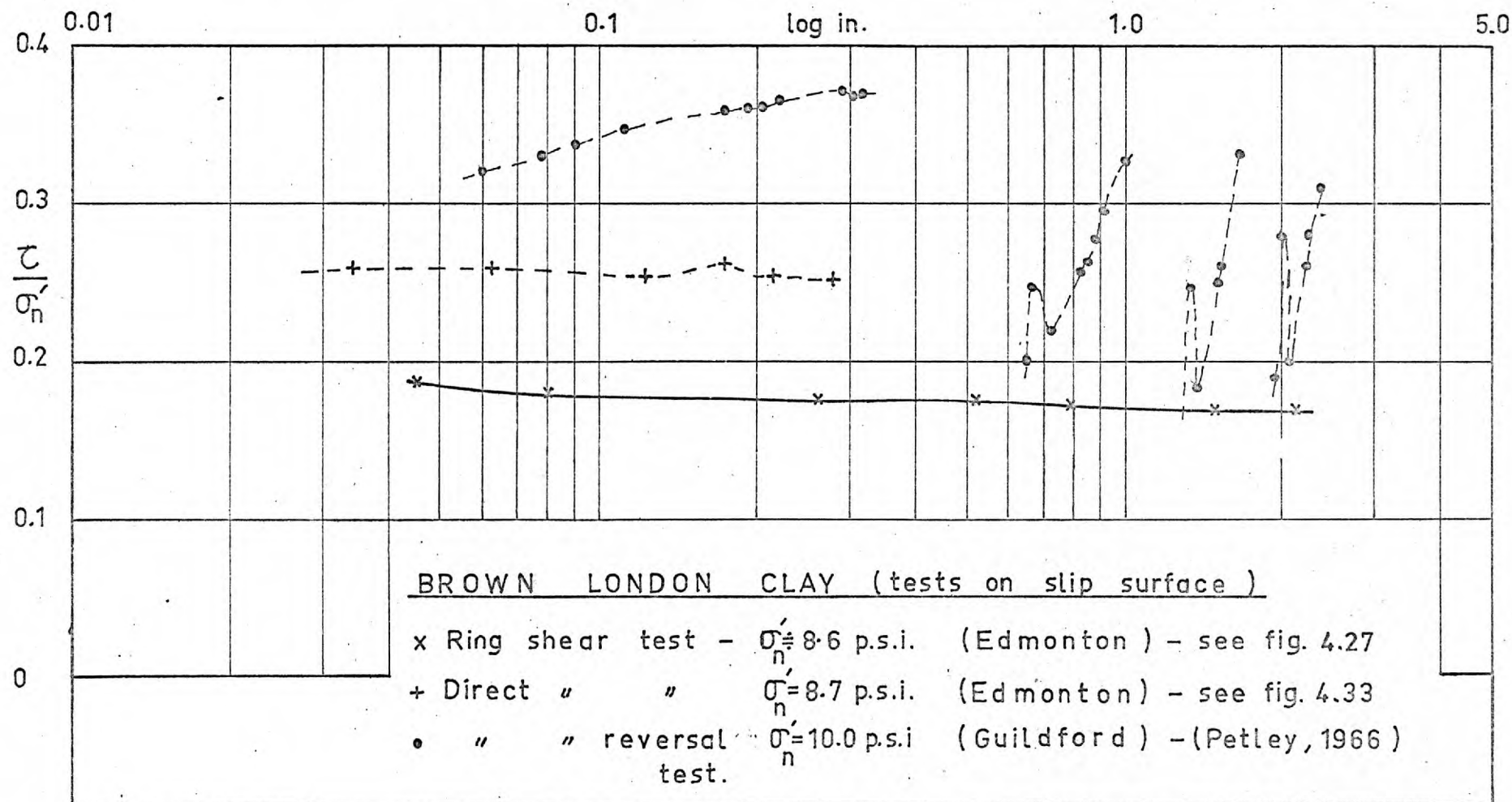
FIG.430a: Effective strength parameters for Brown London Clay



BROWN LONDON CLAY (Edmonton)

Residual shear strength envelope at low effective stress range

FIG.4.30 b



τ/σ'_n AGAINST LOG DISPLACEMENT

FIG.4.31

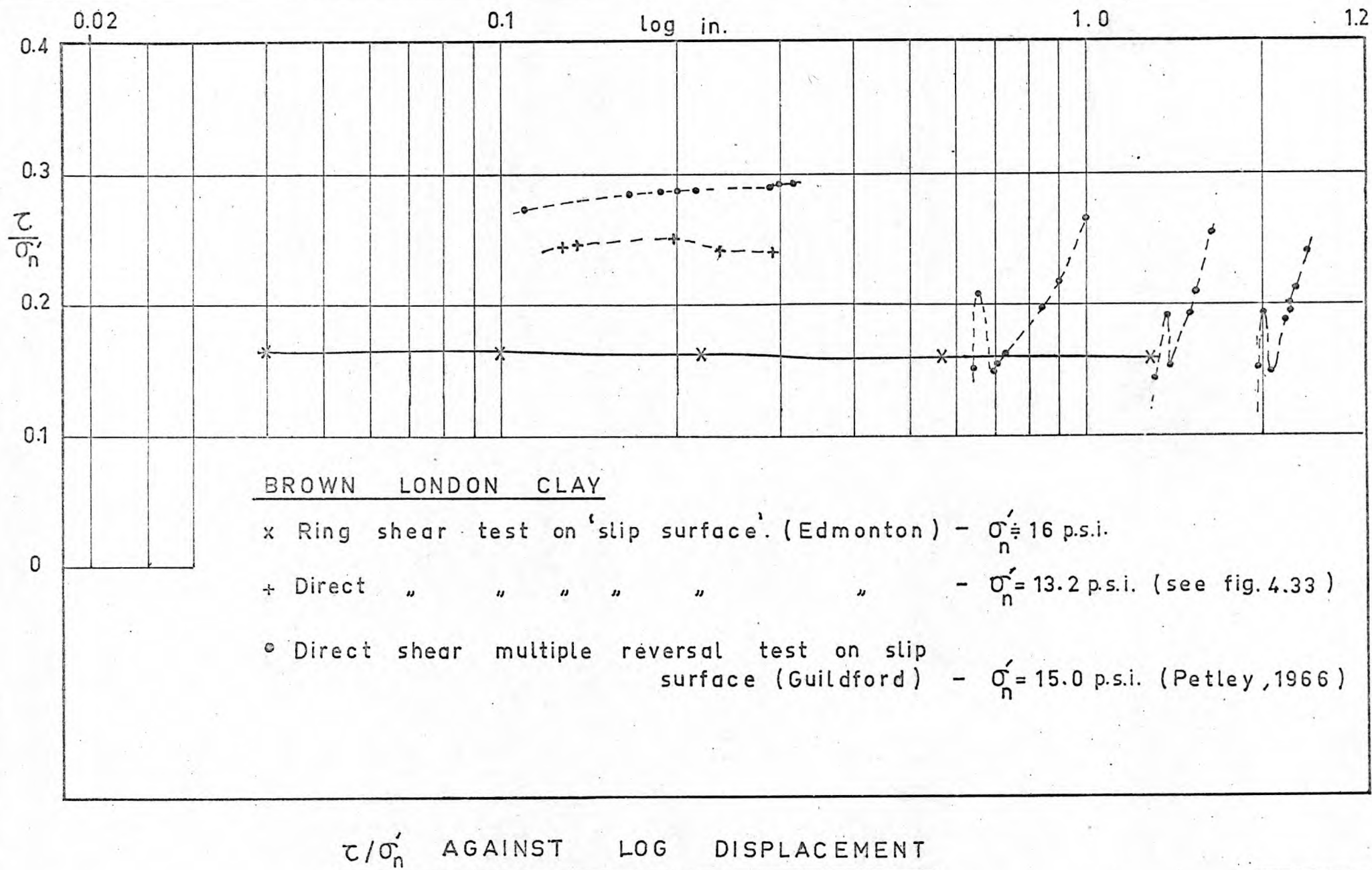
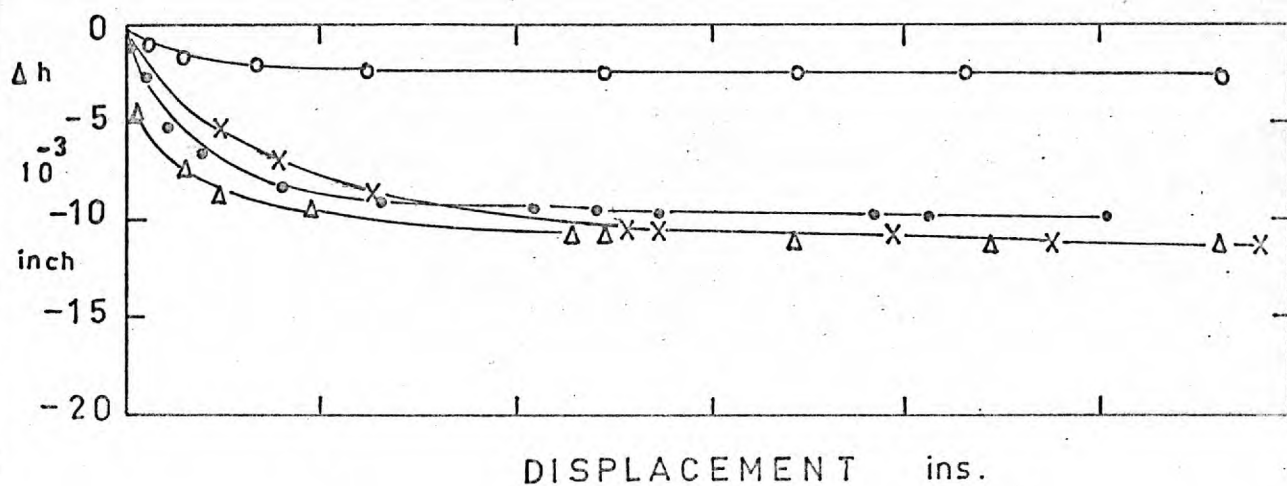
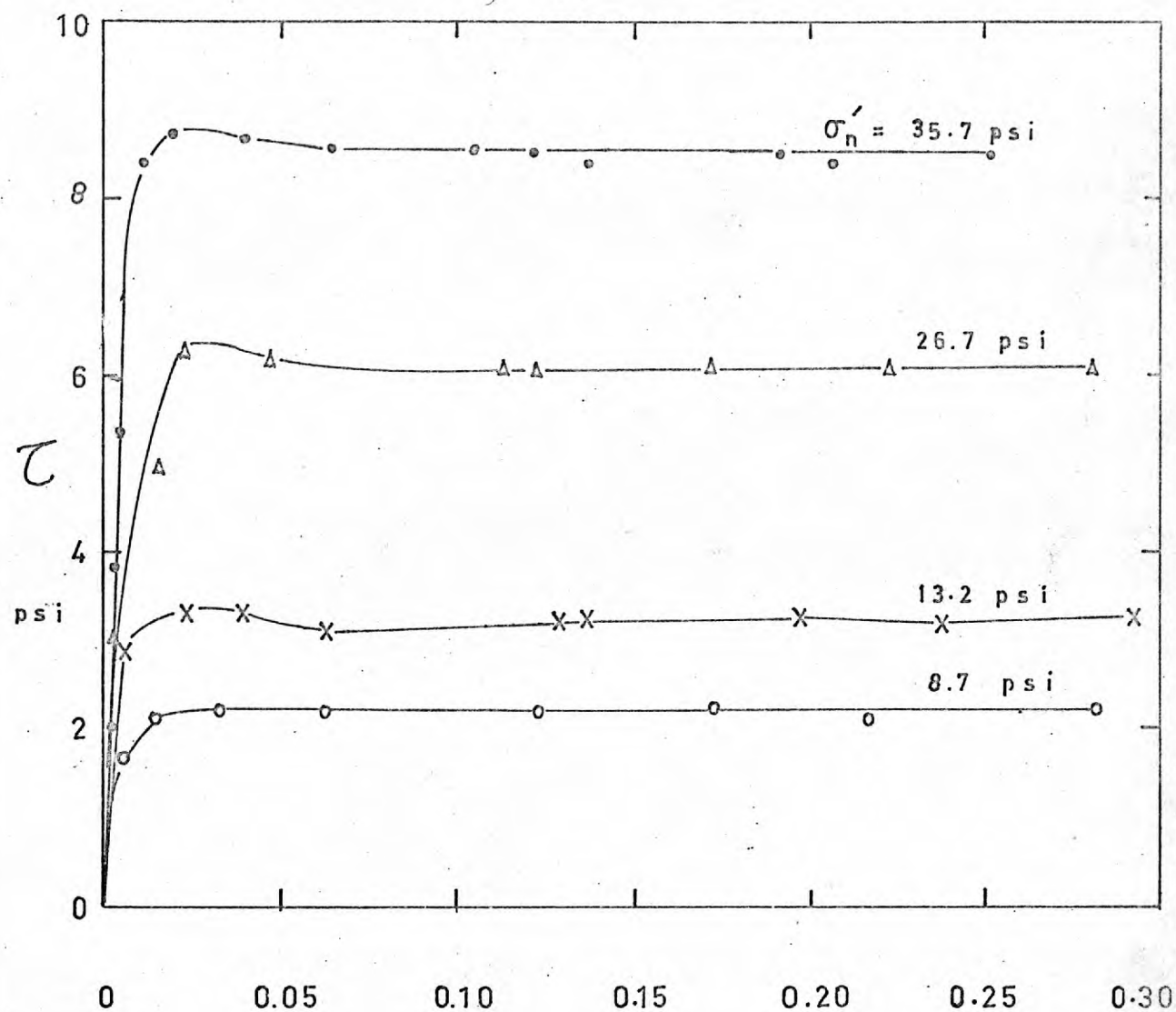


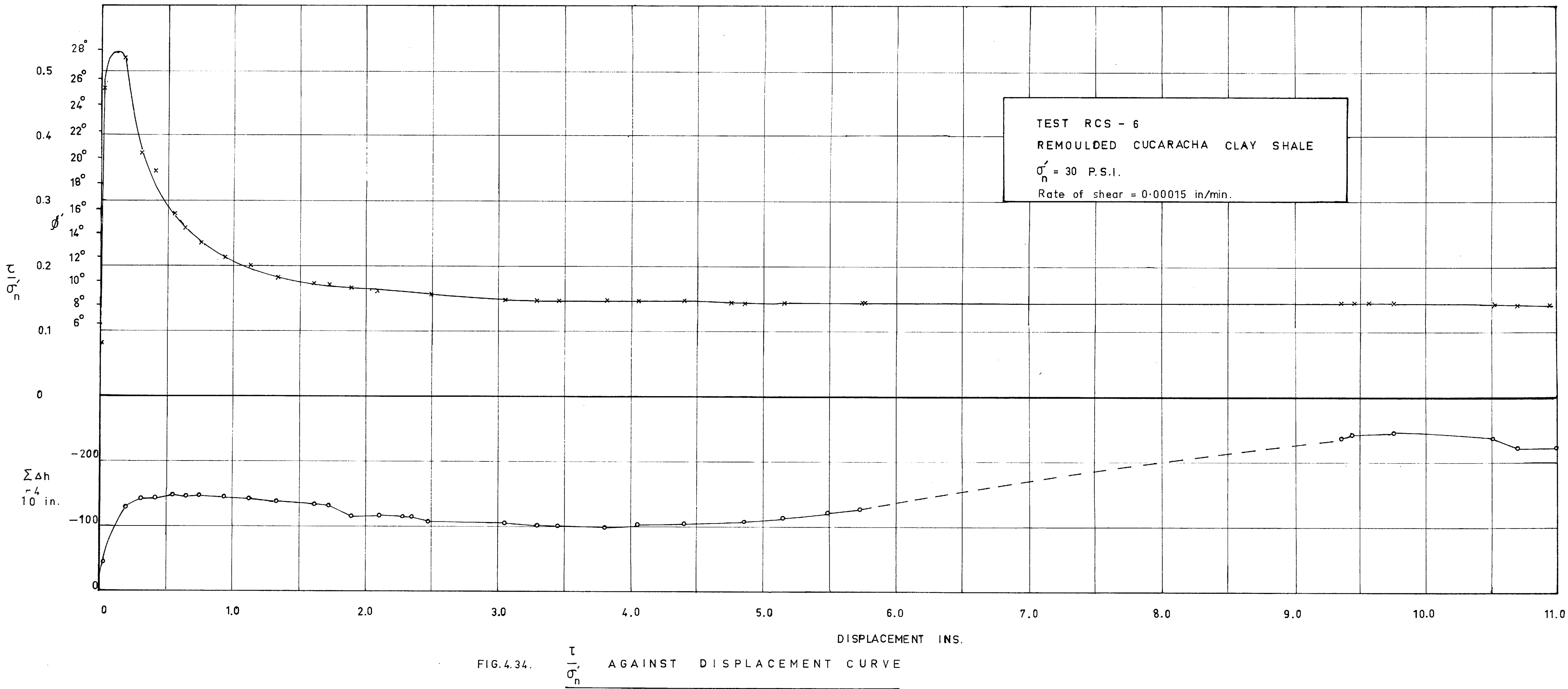
FIG. 4.32



BROWN LONDON CLAY

(after Petley, 1969)

FIG. 4.33



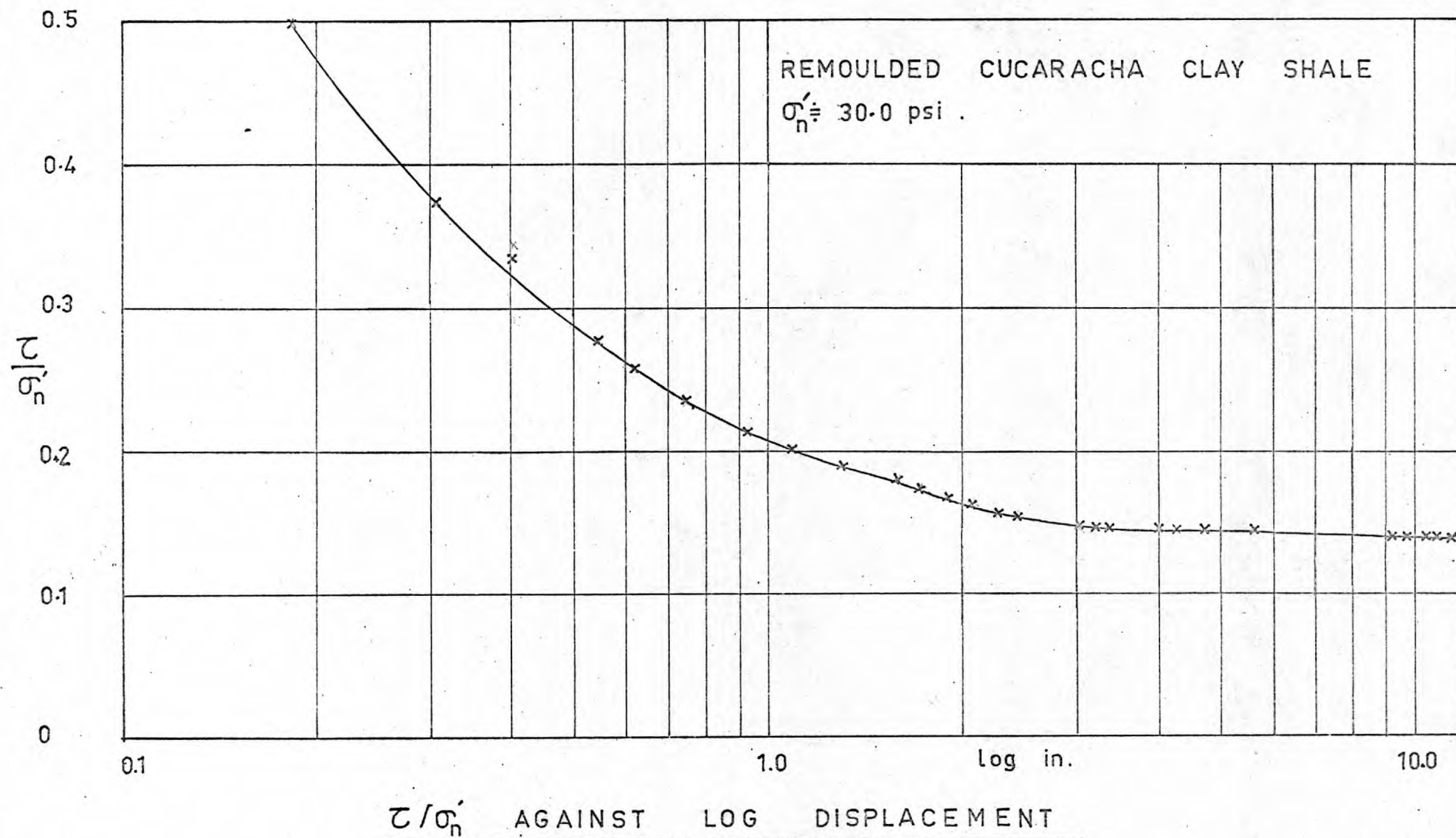
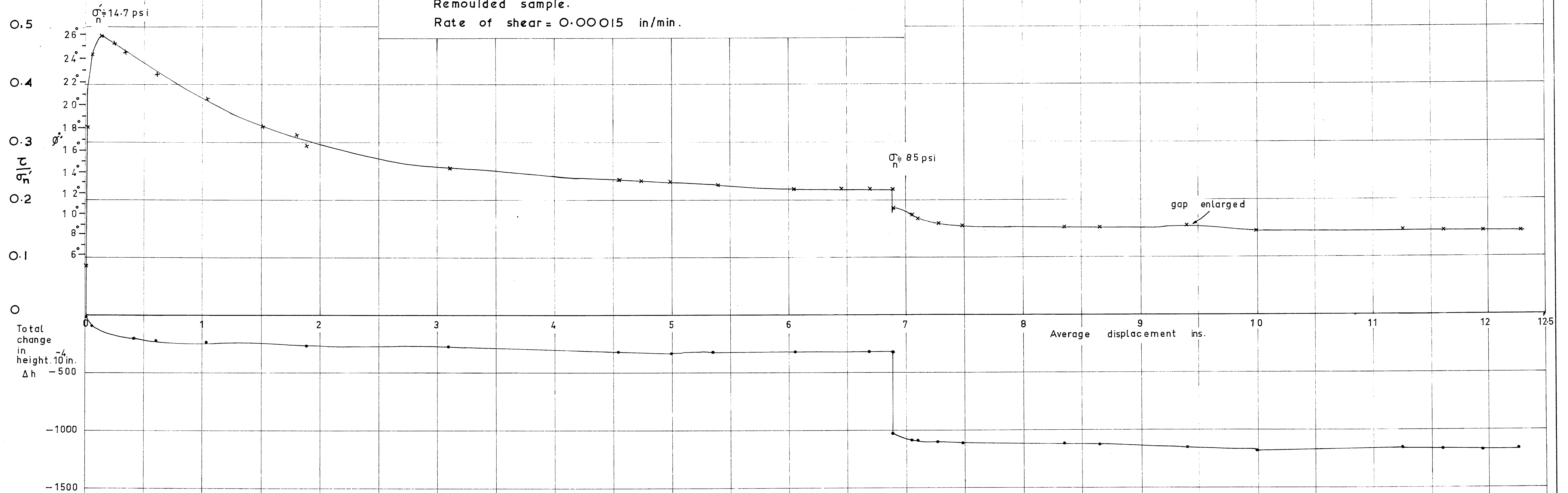


FIG. 4.35

FIG.4.37 DAWSON FORMATION CLAY-SHALE (Chatfield Dam)

Remoulded sample.

Rate of shear = 0.00015 in/min.



CHAPTER 5

Microstructure of sheared clays

5.1 Introduction

The laboratory study of the structure of clays can be made on three scales: the macroscopic, the microscopic and the submicroscopic. The macroscopic structures are those that can be viewed by a naked eye and the effects of such structures as joints, fissures, bedding planes, intrusions etc, on the engineering properties of soils are now well appreciated (see, for example, Bishop etc. al., 1965, Duncan and Seed, 1966; Bishop and Little, 1967). The microstructure is one which requires the resolving power of an optical microscope and includes such features as post-peak failure shear structures, shear planes etc. However, to observe the soil structure at the submicroscopic level, for example particle to particle contact, it is necessary to examine it under the electron microscope. Increasing use is now being made to study the microstructure of soils to explain their fundamental mechanical properties.

Clay particles are generally colloids, shaped as thin plates or rods and are surrounded by adsorbed water layer (Lambe, 1958; Bolt, 1956; Rosenquist, 1962). Soil structure is built up as a result of negative and positive charges operating in the clay - water - electrolyte domain. There is evidence to suggest that negative

charges are present on clay particle surfaces while positive charges operate on the particle edges (Lambe, 1958; Rosenquist, 1958; Van Olphen, 1963). The exact nature of the structure and properties of adsorbed water layer is not understood, though it is well established that its structure differs from that of ordinary water (Mitchell, 1956). Low (1961) has presented evidence that the adsorbed water is more ordered, less dense and more viscous than ordinary water while Martin (1962) reached just the opposite conclusions.

Orientation of clay particles can take place in two systems - dispersed and flocculated. A dispersed system consists of clay particles in parallel arrangement while in a flocculated system, the clay particles are randomly orientated as a result of attraction of the negatively charged particle surfaces to positively charged ends of other particles. The depositional environment of the sediment is a controlling factor in making up the fabric of the particle orientation. Hence, if the sediment is deposited in marine conditions (where the electrolytic content of the medium is strong due to salt concentration), then a flocculated structure would predominate while a dispersed structure is a characteristic of clays deposited in fresh water environment or samples prepared in the laboratory using distilled water. It would be readily apparent that a clay with parallel particle orientation in a horizontal direction would show greater horizontal permeability, smaller compressibility in the vertical direction and smaller

horizontal shear strength than a clay with flocculated structure. Mitchell and Houston (1969) have recently considered the effect^{of} eight chemical and physio-chemical processes which can affect the sensitivity of clays.

Morgenstern and Tchalenko, (1967) and Tchalenko, (1967) studied microstructures associated with shearing using a polarising microscope. Their main conclusions (after Morgenstern et. al., 1969) are:

- 1) the structure at residual strength is not simply one of particles oriented in the direction of motion but is also associated with a compression texture in which the basal planes of the particles are approximately normal to the major principal stress, i.e., it is inclined to the direction of movement. Generally, the compression texture is finer, with less homogeneity than other structures in the shear zone. The compression texture, which has two bounding principal displacement shears, has been found in both natural slip surfaces where it occurs up to several mm in thickness (Morgenstern and Tchalenko, 1967) and samples sheared in the laboratory.
2. Most natural stiff clays have a moderate to high degree of orientation as a result of large overburden pressures. (This is not necessarily true for montmorillonitic clays).
3. Shearing of clays is discontinuous at the level of observation of an optical microscope, although the

visual examination of a highly polished slip surface may suggest it to be a smooth plane.

The polarising microscope works on the principle that most clay minerals are birefringent, i.e., when rotated, they alternate through extinction and illumination when viewed through cross nicols of the polarising microscope. "When a random aggregate of clay particles in thin section is rotated, no variation of transmitted light intensity is found. But, as the degree of orientation is increased, the birefringence of the aggregate converges on to that of the single particle. Therefore, the ratio of the light transmitted in the extinction position to that transmitted in the illumination position is a measure of the degree of orientation of the aggregate of clay particles", (Morgenstern et. al., 1969). The ratio of the minimum to maximum light intensity through the cross polars as the thin section is rotated is termed the birefringence ratio (β) and is measured by a photoelectric cell.

$$\text{i.e. } \beta = I_{\min}/I_{\max}. \quad 0 \leq \beta \leq 1$$

$\beta = 0$ for perfect particle orientation,
and $\beta = 1$ for random particle orientation.

In the present study, use was made of both the polarising microscope and the scanning electron microscope. The latter has the advantage that the clay particles can be viewed in three dimensions under very high magnifications, if so desired (see also Gillott, 1969).

However, it should be mentioned that as yet there is no method of quantifying the degree of particle orientation in the electron microscope. Roscoe (1967) has used this technique to observe clay particle contacts in shear zones and consolidated samples of Kaolin. Gillott (1969) has described the observation of a number of fine grained rocks especially with regard to particle contacts and the nature of cementing material along grain boundaries. The author observed that clay particles in the shear zone generally appeared to be smaller and broken down than particles in the unsheared material (plates 5.2 and 5.3). However, it is readily admitted that this is only semi-qualitative evidence and a statistical count of particle size on many micrographs would be required to confirm this observation.

5.2.1 Preparation of thin sections for optical study

One basic requirement of any acceptable technique of thin sample preparation is that it should not alter the optical properties or the fabric of the material. Many impregnation techniques have been utilised with varying degrees of success by several authors. The impregnating agent usually fills the pore space previously occupied by pore water and then hardens to bind the soil together. Weatherhead (1940) and later Weymouth and Williamson (1953) prepared thin section by impregnating previously dried soft rock and kaolinite surface by pyroxlin. Preliminary drying of soil surface induces a large shrinkage in volume, thus resulting in the

alteration of clay fabric.

Rosenquist (1955) impregnated some soft Norwegian clays with sulphonated alcohol at 60°C , and the sample was then cut with a microtome at a temperature of -10°C . This method results in an extremely small thin section and any coarse intrusion of silt or sand in clay makes the use of a microtome rather difficult.

Carbowax 6000, a high molecular weight polyethylene glycol compound, has been successfully used by Mitchell (1956), Quigley and Thompson (1966), Martin (1966) and Tchalenko (1967). Carbowax impregnation does not completely eliminate shrinkage effects. Mitchell (1956) considered the shrinkage effects to be negligible. Quigley and Thompson (1966) reported a linear shrinkage of 7.2% in the horizontal direction and 8.8% in the vertical direction in impregnated Leda clay. Tchalenko (1967) has measured external horizontal shrinkage of 6.5% and average external vertical shrinkage of up to 9.8%, on impregnation in laboratory consolidated samples of kaolin. However, there is strong evidence that shrinkage does not affect microstructure. The thin section prepared after impregnation does not show any signs of disturbance to the most delicate shear features when viewed under the microscope. Further, Martin (1966) has shown by means of direct measurements that shrinkage has negligible effect on the degree of particle orientation.

The advantages of the use of Carbowax 6000 may be summarised as follows:

- 1) It minimises volume shrinkage effects.
- 2) Its optical properties are such that they do not interfere with the optical properties of clay.
Tchalenko, (1967) observed no overall birefringence effect on areas containing several spherulites of crystallised Carbowax 6000.
- 3) Normal grinding of impregnated samples does not alter clay fabric, (Mitchell, 1956; Martin, 1966).
- 4) Since Carbowax 6000 stabilizes a large volume of soil, a larger sample can be impregnated so that any cutting or disturbance effects on the sides of the sample may be eliminated by subsequent grinding.
- 5) This technique does not require any special working conditions or apparatus.

Sampling

Some segments from the ring shear sample were cut carefully by a cheese-cutter. Tchalenko (1967) recommends that samples with high moisture content or those which tend to disintegrate on impregnation should be impregnated in plastic frames or moulds. The direction of shear and identification number were clearly marked by scratching the surface of the clay.

Impregnation

Carbowax 6000 was melted and maintained in an oven at a temperature of 60° - 65°C . After the method recommended by Martin, (1962) the top edge of the sample was kept free of Carbowax and the evaporating dish was covered to preserve Carbowax vapour, for the first 24 hours. After this period, Carbowax was renewed to submerge the sample in solution. The sample was maintained at this temperature from 7 - 10 days. After the impregnation was thought to be complete, the sample was removed from molten Carbowax and allowed to cool and harden at room temperature.

Grinding

Grinding was done by hand on different grades of coarseness of emery paper, (from coarser to the finer). Paraffin was used as a cooling and grinding fluid. Care should be taken while grinding to ensure that no undue pressure is exerted on the sample. The sample was finally ground using the finest grade of optical grinding powder (Aloxite 700). The sample was then gently wiped with polishing cloth and left for 24 hours for drying.

The sample is now ready for mounting. Tchalenko found that a mixture of epoxy resin adhesive Araldite A.Y. 103 and Hardner E.Y. 951 in proportion of 100 parts (by weight) of Araldite to 10 parts of hardner was most suitable for affixing the clay surface on to a glass

slide. The mixture is mixed and kept in a vacuum cell for 20 minutes to ensure that any air bubbles that might have been introduced during the mixing stage are removed. It is then evenly spread on to a glass slide which has previously been cleaned with methylated spirit. The sample is introduced at a slight angle and gently pressed so as to remove any air bubbles. After the adhesive has hardened (usually after 24 hours), the orientation signs can be etched on to the glass slide.

A similar grinding process is then repeated on the other side when the thickness is so small as to make the clay transparent, the cover slide is affixed using the same process as described above.

5.2.2 Preparation of sample for scanning-electron microscope.

Two samples of blue London clay and Weald clay after shearing in the ring shear apparatus were observed in the scanning electron-microscope. After the sample had air dried, it was fractured along the required plane by making a cleavage. Since the stub on which the sample is affixed in the electron-microscope is 1 cm in diameter, the sample was usually trimmed into approximately 5 mm cuboidal shape, taking great care not to touch the area which is to be examined. The sample was next kept in vacuum to deair for 48 hours. The sample was then affixed to the stub by means of 'Durafix'. After the adhesive had hardened, the sides of the cuboid

not to be viewed and the top of the stub were coated with 'silver dag' suspension. The atomiser was next used to blow^{off} any stray soil particles and the stub was mounted in the vacuum coating system. 100 Å of gold preceded by 150 Å of carbon were sprayed at an angle of 45° on the face of the cuboid which is to be viewed. The spraying was done under a vacuum of 10^{-5} torr. The specimen was now ready for viewing in the scanning electron-microscope.

5.3 Observations on thin sections

5.3.1 Undisturbed sample of blue London clay

This sample was sheared in the ring shear apparatus at a nominal vertical effective stress of 30 psi. The thin section was prepared after the sample had been subjected to a shear displacement of about 52 in. A sketch of the complete slide is shown in fig. 5.1.

The plane at the middle of the sample along which shearing had commenced, at least in early stages, has become separated and a crack is formed. Since this crack is filled with Carbowax, it must have preceded impregnation. This is very likely due to disturbance sustained by the sample during removal from the ring shear apparatus. The crack is typically 30 - 80 μ wide.

Fig 5.1 shows the shear zone, easily distinguished in the thin section from surrounding material which is bounded by two principal displacement shears, AA and BB.

The shear zone is about 27 mm long, starting from one end of the section to the point where the two shear planes AA and BB converge. The width of the shear zone varies from about 1 mm at the end of the section to about 1.7 mm in the middle and decreasing to a typical value of approximately 0.6 mm in the region where the upper discontinuity merges with the lower. The upper principal displacement shear is typically 5μ wide.

A marked feature in the shear zone is the presence of a large number of secondary structures. The 'R-type' sub-structures (shown on fig 5.1) are typically oriented at angles of $20^{\circ} - 30^{\circ}$ * with respect to the lower shear plane B-B. The 'R-type' structures become more steep, about 33° , in the region where the upper shear plane (or shear discontinuity) converges with the lower. A few 'P-type' structures are also present. The 'R-type' structures could be Riedel shears which might have been formed at or around the peak state but have survived the large displacements and are now passive in response to the large strains at the two boundaries. The second possibility is that this is a new type of structure, not hitherto observed, which forms at very large strains and is perhaps in a process of continuous formation. Two arguments can be advanced in support of the second possibility. The angle of inclination of these structures is much larger than $\phi_p/2$ for this clay and hence they

*Measurements of angles are positive in the counter-clockwise sense. This is also the sense of the shearing couple in all plates and figures described here.

could not be Riedel shears. Further, the 'R-type' structures do not extend beyond the shear zone as would be expected if they were Riedel shears.

The interesting point to note is that the material in the shear zone is not completely remoulded and structureless. The fact that the sub-structure has been maintained in the shear zone even after very large displacements, suggests the the material in the shear zone moves 'en-masse' on either or both of the slip surfaces AA and BB. An examination of the region around a large silt pocket just above the upper shear surface suggests that straining has taken place along this surface. The silt pocket is not truncated by the slip surface and the silt particles tend to be dragged along the displacement discontinuity in a fine streak. This feature is shown on detail in plate 5.1. The larger white spots in this photomicrograph are quartz crystals.

The birefringence ratio, β , of the material away from the shear zone lies between 0.7 to 0.85; the higher ratios being obtained in areas with higher quartz content and hence these are probably not the true birefringence ratio for the clay. The average birefringence value in the shear zone is approximately 0.55, indicating a medium to strong particle orientation according to the semi-quantitative scale proposed by Morgenstern and Tchalenko, (1967). However, it is of interest to note that there are a few patches in the shear zone where the birefringence

ratio is as high as in the matrix away from the shear zone. This indicates that not all the material in the shear zone is highly oriented and is consistent with observations made on other slip surfaces. The β value in the silt pockets is about 0.7 - 0.8, i.e., the orientation is the same as that of the clay away from the shear zone.

There is some evidence to suggest that some particle breakdown or migration of fines takes place on shearing under large displacements. Under the optical microscope, the general texture of the material in the shear zone was very homogeneous as compared to material on either side. There was a noticeable presence of more coarse grained aggregates (eg. quartz) in areas outside the shear zone. Plates 5.2 and 5.3 show scanning electron micrographs of this sample both inside and away from the shear zone. The two micrographs were taken at almost the same magnification, and show that the average size of clay aggregates is much smaller inside the shear zone than away from it. The general appearance of the two plates also suggests that there is more particle orientation inside the shear zone.

5.3.2 Natural shear zone in Weald clay

A thin section from the shear zone in Weald clay at Arlington is shown in fig 5.2. The sample contained numerous shear planes and discontinuities, generally horizontal, as a result of shearing in the field. The

birefringence values vary from 0.92 to 0.23, i.e., the particle orientation ranges from random to strong. This variation shows that strong particle orientation was only exceptionally achieved in limited areas, probably as a result of excessive local straining. This may imply that only these local areas had achieved the residual state. The general direction of particle orientation in the sample was horizontal as shown in fig.5.2.

Two distinct types of surface texture could easily be observed under the polarised light. The area enclosed by the dotted pattern had a distinct 'mosaic-type' texture while the rest of the section appeared to be fairly homogeneous. The mosaic type texture appeared to consist of coarser particle aggregates and it is clear from the birefringence measurements shown in fig.5.2 that it had lesser particle orientation than the homogeneous type texture. However, the birefringence ratio of individual mosaic components, taken under high magnification is low, about 0.35. It is possible that either particle breakdown or at least breakdown of coarse aggregates occurred in the homogeneous structure or that there has been a migration of fines from the structure with the mosaic-type texture, subsequent to which the coarser particles formed yet larger aggregates. However the origin of the two types of textures cannot be conclusively determined until information on depositional conditions and the origin of shear zone are considered. The area marked as dotted rectangle in fig.5.2 is shown in greater detail in

plate 5.4, where the homogeneous and the mosaic-type textures in the upper and lower half of the photograph are easily distinguished.

5.3.3 Weald clay from natural shear zone further tested in the ring shear apparatus.

The complete thin section is shown in fig. 5.3. This sample had undergone a mean average displacement of 67.8 in. under various normal loads in the ring shear apparatus. Since the original sample was from a natural shear zone, the thin section had a number of natural shear planes and discontinuities as shown by the dotted lines in fig 5.3. For the same reason, the overall birefringence values are generally low indicating medium to strong particle orientation.

The shear zone, 1 - 3 mm wide, is distinctly visible in the thin section and is bounded by a number of shear planes. It is possible that shearing could have taken place along some of these planes depending on the magnitude of the effective normal stress and the position of the gap between the confining ring and the sample. The final height of the section was smaller than the initial height as a result of consolidation under normal stresses up to 120 psi. and squeeze through the gap. The scanning electron micrograph in plate 5.5 shows the horizontal shear zone in the middle bounded by relatively intact material on either side. The structure in the shear zone appears to show higher particle orientation in the direction

of movement than the structure in the more intact clay. The shear zone is shown in greater detail in fig 5.4. The most pronounced shear planes along which most of the movement would have taken place have been marked as X and Y in fig 5.4. A photomicrograph of the principal shear plane X is shown in plate 5.6 where its width was measured to be about 24μ . However, the principal shear planes are not so clearly visible at all places. At some locations, they become extremely small in width and their presence can be detected by observing allied shear features as shown in plate 5.7. In this case, the band of organic matter shows clear signs of shearing and a careful examination reveals extremely fine principal displacement shears running above and below the organic matter.

A very pronounced feature of the shear zone is the presence of a compression texture, in which the clay particles are strongly oriented. This is shown in plates 5.8 and 5.9 where the organic matter inside the compression texture gives the direction of particle orientation. There are a series of bands spaced at about 0.03 mm, inside the compression texture. The compression texture is considered to be indicative of very large displacements in which the basal planes of the particles are approximately normal to the direction of the major principal effective stress (Tchalenko, 1968; Morgenstern et. al., 1969). Below the compression texture lies a kink-band which is shown in plate 5.10. Similar combination of compression

texture and kink-band has been found in the Walton's Wood shear zone by Tchalenko, (1968) who also showed that the compression texture is brought about by the interaction of the microscopic shear discontinuities and kink-bands which form in simple shear deformations taken to large strains. An example of the extreme particle parallelism that can be achieved in a compression texture is shown in the scanning electron micrograph, plate 5.11.

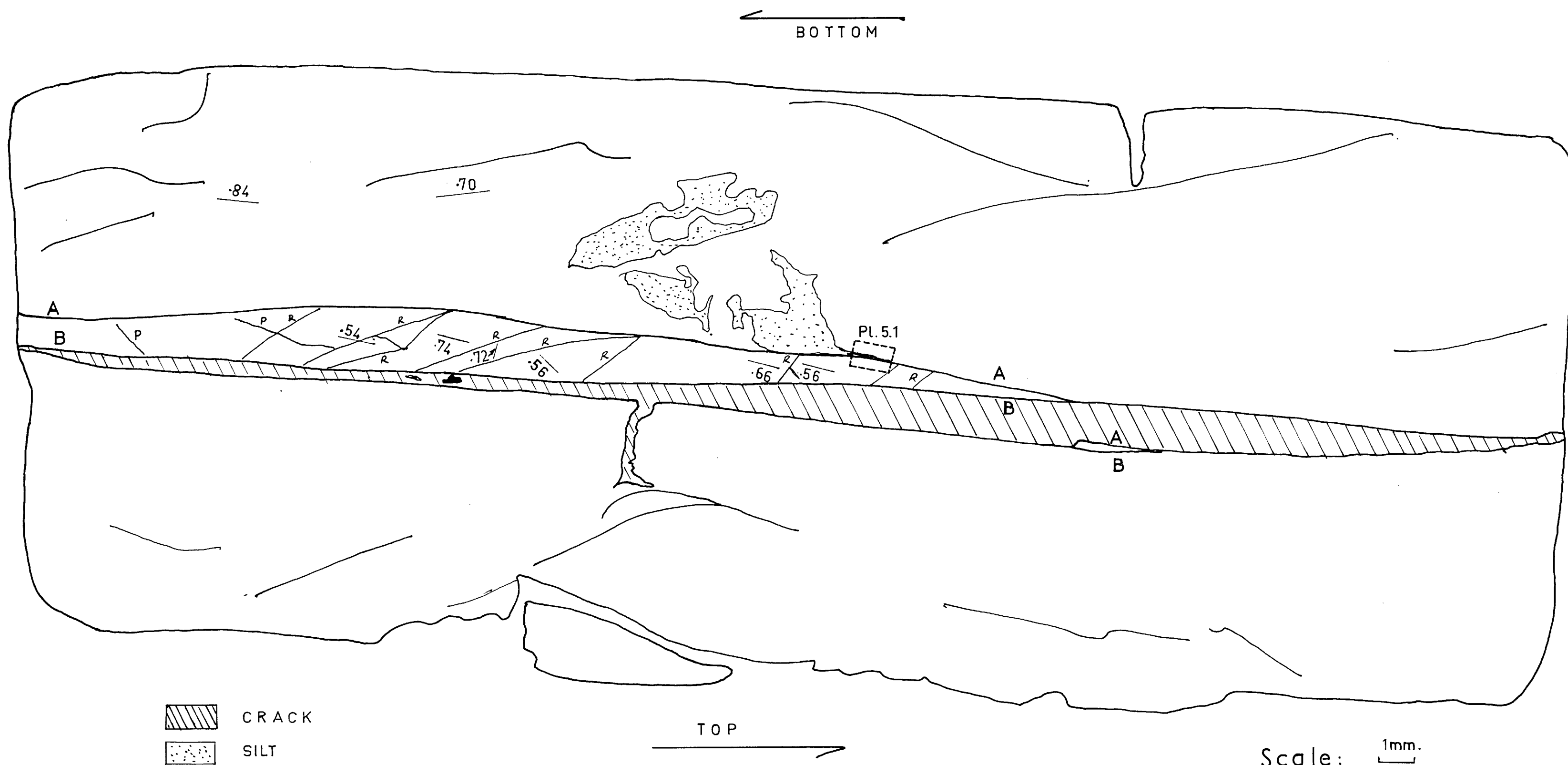
Above the principal displacement shear X the structure has mosaic-type features while below X , the structure is more homogeneous, (fig 5.4). Some patches of mosaic-type texture are also found inside the shear zone. These features are similar to those found in the natural shear zone. Plates 5.12 and 5.13 show the homogeneous and the mosaic-type texture, As mentioned earlier, this difference in texture is considered to be caused either due to migration of fines or due to breakdown of aggregates or a combination of both. This feature is otherwise hard to explain and is found in both the London clay and the Weald clay when subjected to large displacements.

5.4 Summary of observations

In all thin sections of samples tested in the ring shear apparatus, an easily distinguished shear zone is noticed, which may be up to 3 mm wide. In every case, at least two principal displacement shears have been

observed; their number may increase if the sample is tested under a wide range of normal stresses. The principal displacement shears may vary from about 5 to 50 microns in width. Principal shear planes much larger in width than those produced in the ring shear apparatus have been noticed in the field. Morgenstern and Tchalenko (1967) observed a dominant shear plane up to 300μ wide in the shear zone in brown London clay from Guildford and displacement shears $8 - 80\mu$ thick in the Atherfield clay at Sevenoaks. It may be mentioned again that even after very large displacements the principal displacement shears are not perfect "planes" and that sub-structure still exists in the shear zone. A marked difference in texture of the sample on either side of the shear zone has been observed which may be due to particle breakdown or migration of fines. The observations in the scanning electron microscope suggest that there is a breakdown of particles in the shear zone and a detailed study on this aspect is needed. Most of the structure observed in the ring shear samples (e.g. compression textures, kink bands) is similar to that of samples subjected to small strains in the direct shear box apparatus. Hence the difference between ϕ'_r at small strains and ϕ'_r in the ring shear apparatus cannot be explained by 'structure' alone, although migration of fine clay minerals, particle breakdown and higher degree of particle orientation may account for the difference. This is in contrast

with the difference between ϕ_p' and ϕ_r' which is strongly reflected in the respective structures. A systematic study on the optical and electron microscopes coupled with chemical and X-ray analysis is therefore required to study the structure at large strains.



Undisturbed 'Blue' London Clay in Ring Shear Apparatus.
 PARTICLE ORIENTATION AND BIREFRINGENCE VALUES SHOWN

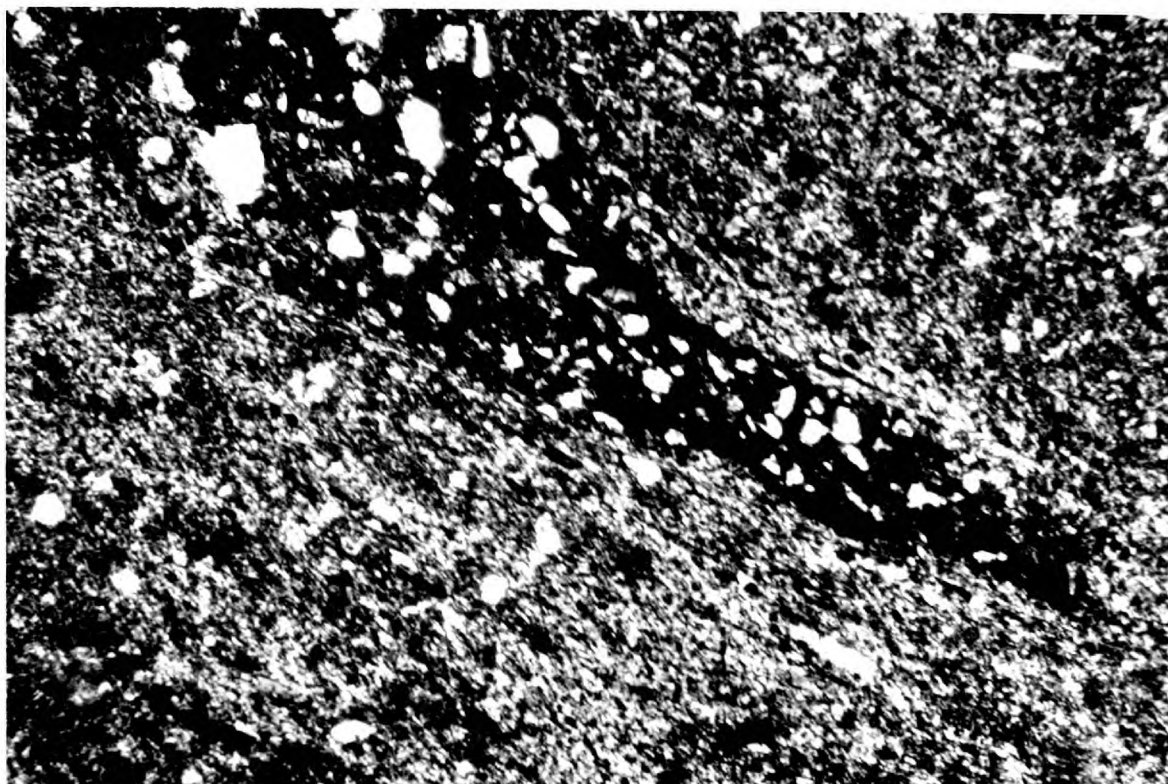


PLATE 5.1.

Blue London Clay: silt particles being
dragged above a principal shear plane
(crossed nicols).

scale: 100μ
[]



PLATE 5.2

Scanning electron micrograph showing clay particles inside the shear zone in Blue London Clay.

Magnification: x21000

Scale:

1μ

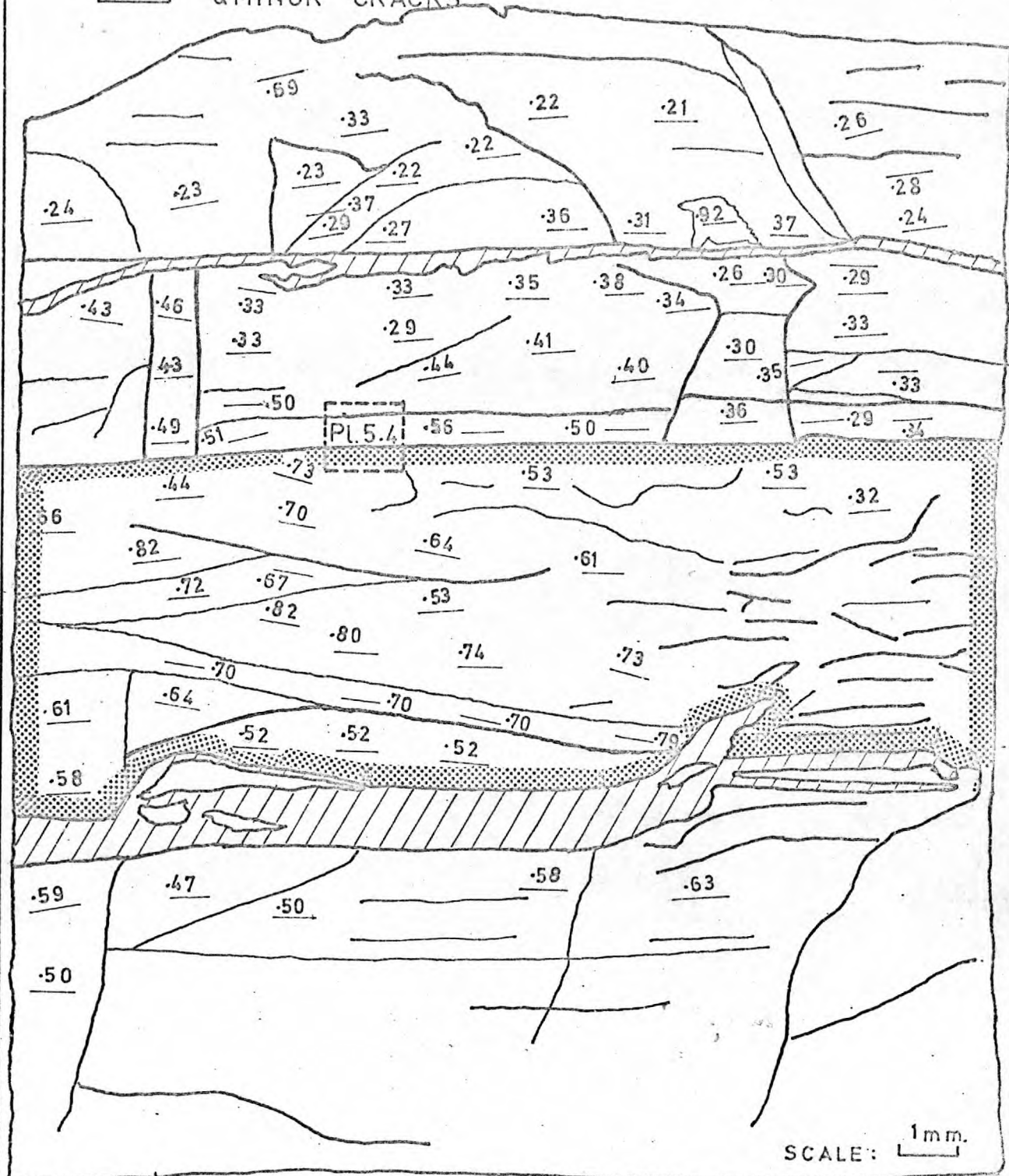


PLATE 5.3

Scanning electron micrograph showing clay particles outside the shear zone in Blue London Clay.

Magnification: x 22500 Scale: 1 μ

 CRACK ;  AREA ENCLOSED THUS HAS
 SHEAR PLANES 'MOSAIC' TYPE STRUCTURE
 & MINOR CRACKS



WEALD CLAY

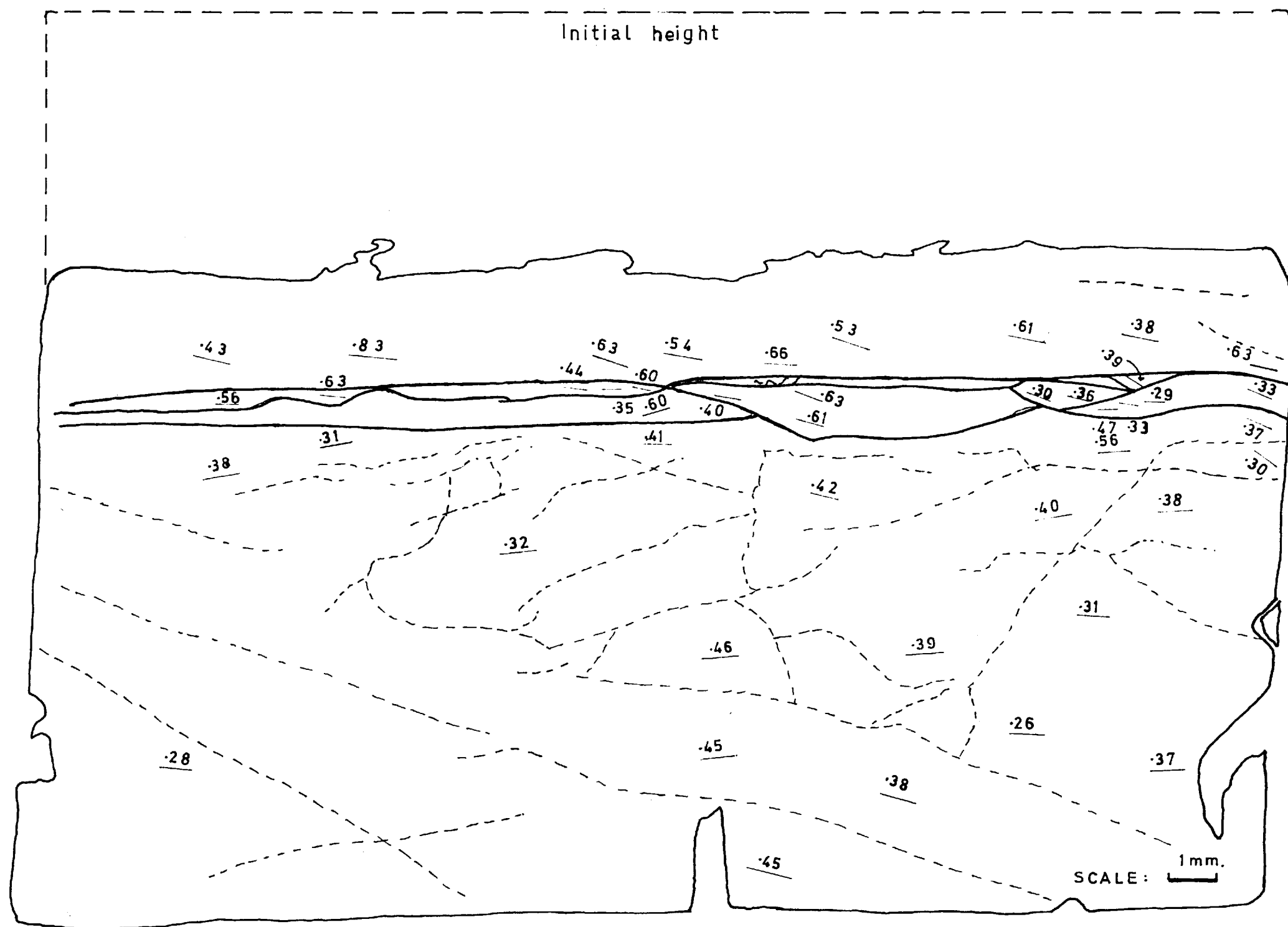
SECTION THROUGH NATURAL SHEAR ZONE
 SHOWING PARTICLE ORIENTATION AND
 BIREFRINGENCE RATIOS.



PLATE 5.4.

Section through natural shear zone in
Weald Clay showing homogeneous and
'mosaic type' texture (crossed nicols)

Scale: 100μ
└────────┘



FINAL AV. DISPLACEMENT: 67.8 ins.

--- Natural shear planes and
discontinuities

BOTTOM

WEALD CLAY

LONGITUDENAL SECTION OF NATURAL SHEAR ZONE FURTHER SHEARED
IN RING SHEAR APPARATUS. PARTICLE ORIENTATION AND
BIREFRINGENCE VALUES SHOWN.

FIG. 5.3

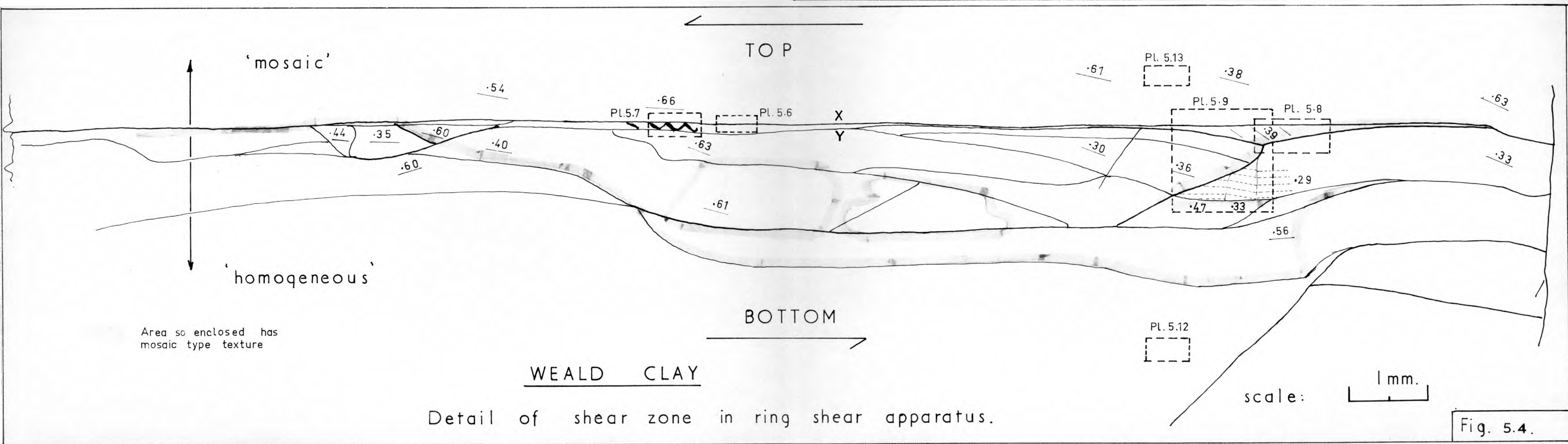




PLATE 5.5

Scanning electron micrograph showing shear zone in Weald Clay bounded by relatively intact material.

Magnification: $\times 100$

Scale: $\overbrace{\hspace{1cm}}^{100\mu}$



PLATE 5.6

Principal shear plane in
Weald Clay. (crossed nicols)

Scale: 100μ



PLATE 5.7.

Organic matter in Weald Clay showing
signs of shearing movement, (crossed nicols)

Scale: 100 μ



PLATE 5.8

Compression texture in Weald Clay

Scale: 100 μ

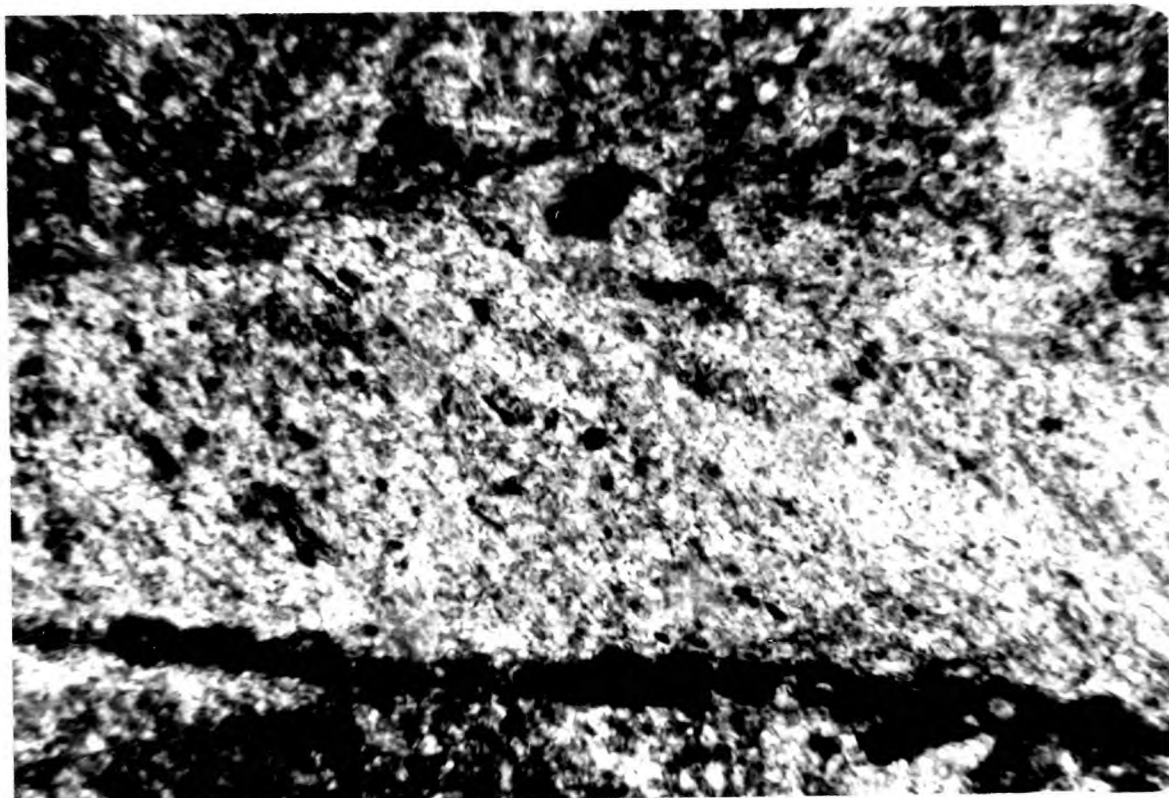


PLATE 5.2

Detail of compression texture
(crossed nicols)

Scale: $\overbrace{\hspace{1.5cm}}^{100\mu}$



PLATE 5.10

Section showing compression texture
and kink band in Weald Clay (crossed nicols)

Scale: $\boxed{100\mu}$

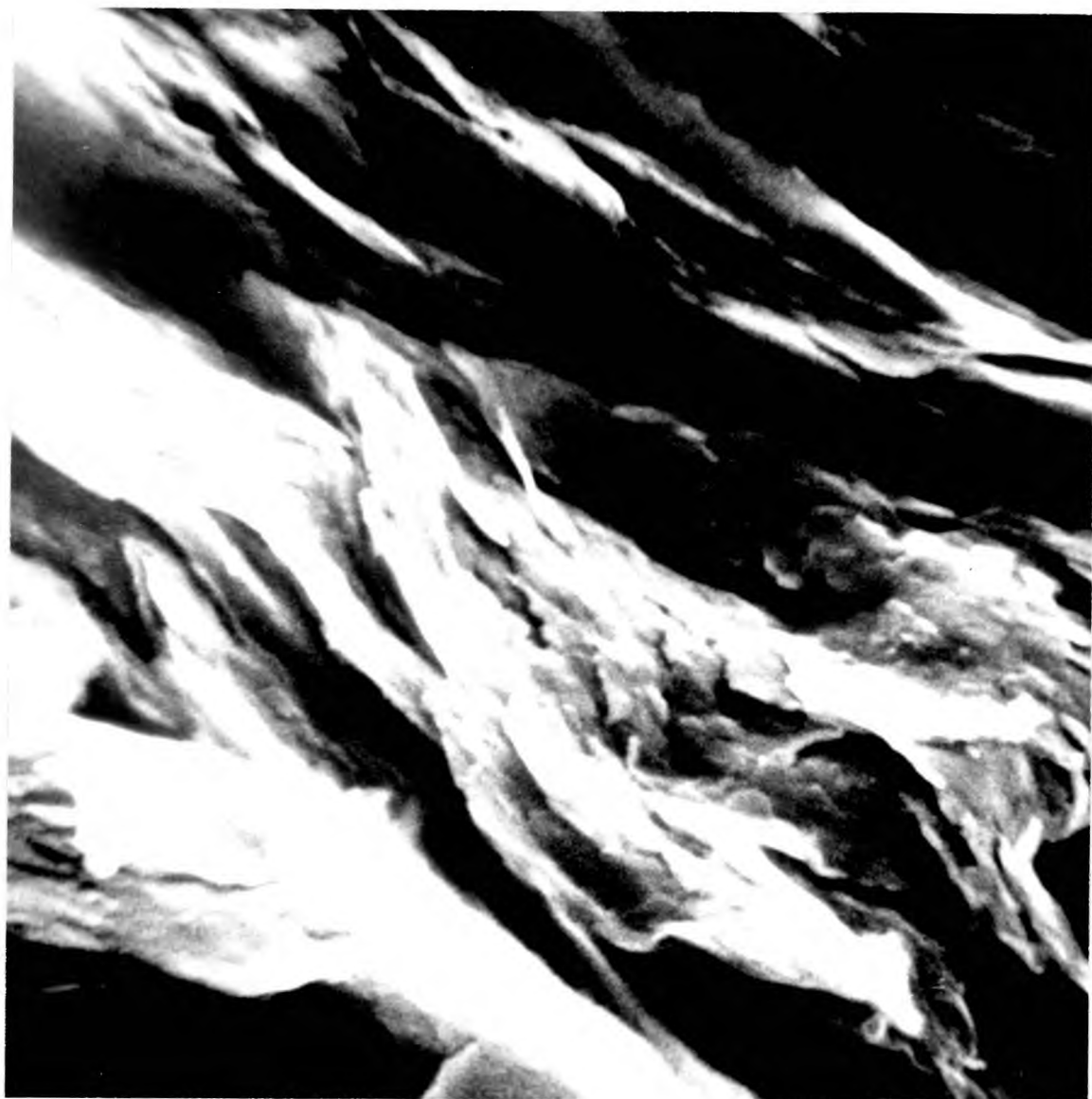


PLATE 5.11

Strong clay particle orientation in
the shear zone in Weald Clay as observed
in the scanning electron microscope.

Magnification: x 20000

Scale: $\underline{1\mu}$

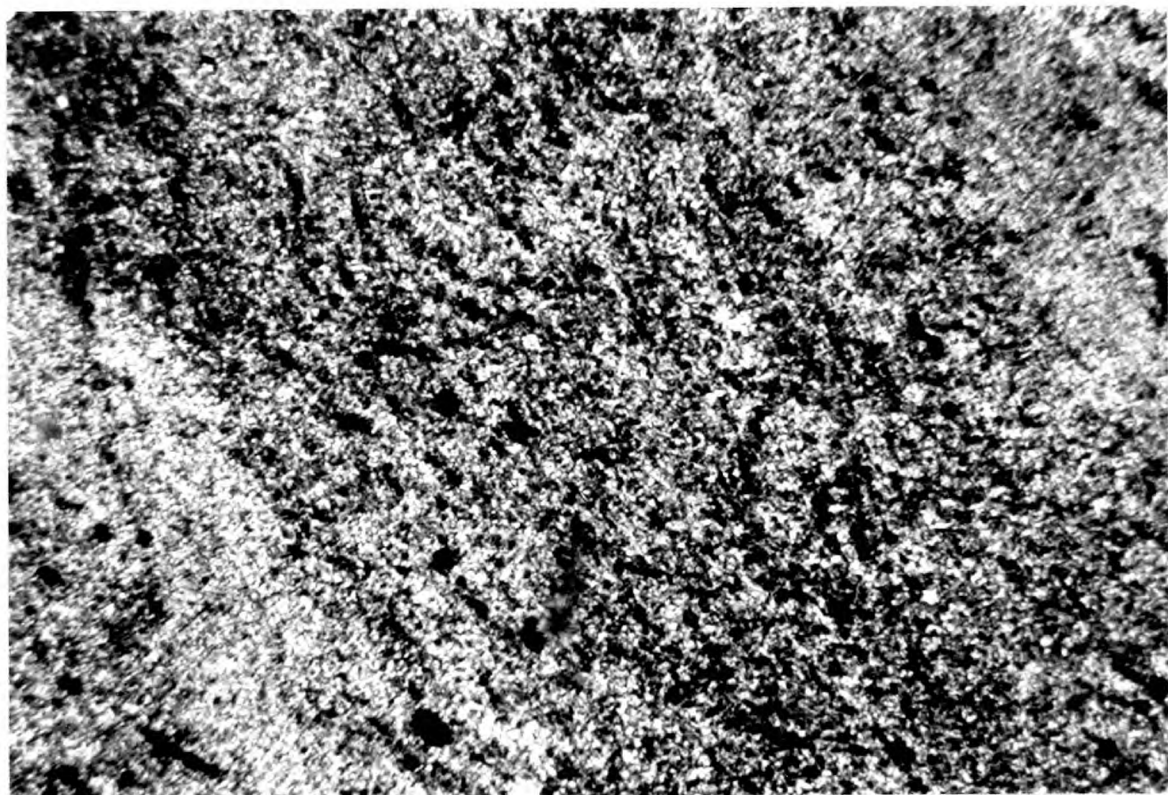


PLATE 5.12

Section showing homogeneous type
texture in Weald Clay (crossed nicols)

Scale: 100 μ

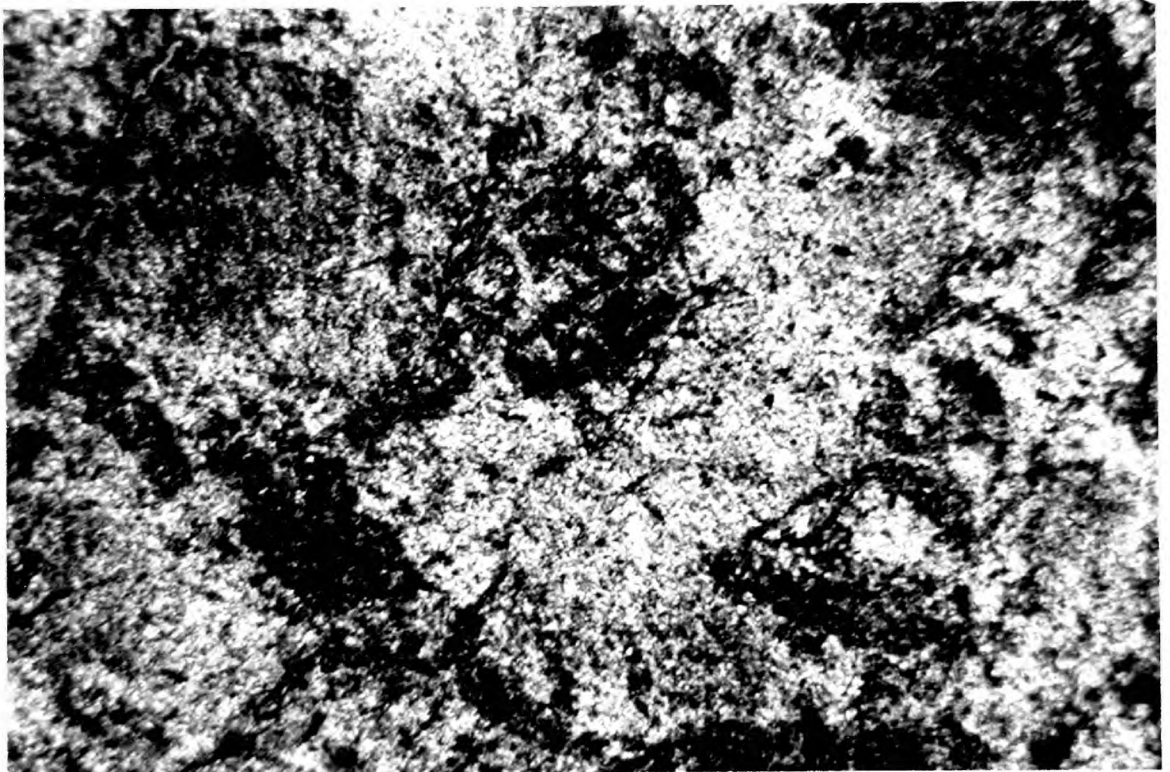


PLATE 5.13

Section showing 'mosaic' type texture
(crossed nicols)

Scale: 100 μ

CHAPTER 6

Tensile strength of clays

6.1 Introduction

The practice of neglecting the tensile strength of soils in most stability calculations has lead to an almost complete absence of test data on the behaviour of soils in the negative effective stress range. It appears that drained tension tests may provide an insight in 'cementation bonds' (Crawford, 1963; Bjerrum, 1967; Conlon, 1966) and also an explanation for the differences in behaviour of undisturbed and remoulded soils.

The present study of tensile strength of intact London clay was inspired by Bishop's (1967) contribution to discussion on progressive failure. He noted that a slip surface does not by-pass intact lumps and progress along fissures only. The intact clay is observed to be sheared and the progressive failure mechanism could be similar to that suggested by Griffith, (1921) where the crack propagates as a result of small local stress concentration. For such a mode of progressive failure it is essential that the material should be able to withstand some tensile stress. Since there was no information about drained tensile strength of stiff clays available in the literature, it was decided to investigate it in the laboratory.

Griffith's theory, (1921) first developed for failure in tension of glass, is an important theoretical advance and provides a convenient method for the study of tensile strength of materials.

6.1.1 Basic concepts of Griffith's theory

If σ_1 and σ_3 are major and minor principal stresses respectively;

and $-\sigma_t$ denotes tensile stress,

and θ the angle of failure plane with respect to the major principal stress, then, Griffith's theory states that:

$$(1) \sigma_1 + 3\sigma_3 = 0, \text{ when failure occurs at } \sigma_3 = -\sigma_t \text{ and } \theta = 0., \text{ and,} \quad (6.1)$$

$$(2) \text{ if } \sigma_1 + 3\sigma_3 > 0, \text{ then failure occurs when}$$

$$(\sigma_1 - \sigma_3)^2 - 3\sigma_t (\sigma_1 + \sigma_3) = 0 \quad (6.2)$$

$$\text{at } \cos 2\theta = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 + \sigma_3)} \quad (6.3)$$

Clausing, (1959) and Hoek (1969) have shown that both Mohr-Coulomb theory of failure in shear and Griffith theory of failure in tension yield the same failure plane θ . If τ and σ_n are the normal and shear stress on a failure plane, then Griffith criteria is represented on a Mohr circle by the relationship:

$$\tau^2 - 4\sigma_t^2 - 4\sigma_t \sigma_n = 0. \quad (6.4).$$

The Mohr failure envelope at $\tau = 0$ will have a radius of curvature of $2\sigma_t$. Hence all Mohr stress circles which fall within the above circle of radius $2\sigma_t$ can touch the Griffith rupture envelope only when $\tau = 0$, and $\sigma_3 = \sigma_t$. Hence, a pure tensile failure can only take place if $\sigma_1 \leq -3\sigma_t$.

Brace (1960) extended Griffith's theory to take into account friction operating in the cracks, which are supposed to close under a state of compressive stress. He proposed:

$$\tau = -2\sigma_t + \mu\sigma_n, \quad (6.5),$$

where μ = coefficient of sliding friction in crack surface (or the tangent of the angle of friction, say, along a microfissure).

Hoek (1965) derived the theoretical relationship between σ_t and μ in terms of σ_c , the unconfined compression strength as:

$$\sigma_c = -\frac{4\sigma_t}{\sqrt{(1+\mu^2)} - \mu} \quad (6.6)$$

6.2 Strength of rocks in tension

Very little work has been done to investigate tensile strength of soils, although tests to evaluate tensile strength of rocks have been developed in the field of rock mechanics. One of the most common method is to subject the

unjacketed sample of rock to a uniaxial tensile pull. Brace (1964), Murrell (1965), and Hoek (1965) have used necked "dog-bone" shaped specimens in triaxial extension with a view to reduce stress concentrations at the ends. Another common method is the 'Brazilian test' in which a disc is subjected to uniaxial compression and the tensile forces so generated are calculated using an analytical solution based on theory of elasticity (see for example, Jaeger, 1962). Hobbs (1964) has modified the 'Brazilian test' by diametrically compressing a thin disc with a small concentric hole. Approximate solutions are then obtained by already known solution for a solid disc diametrically loaded between knife-edges and Kirsch's (1898) solutions for stress distribution along a circular hole. Jaeger (1962) has given solutions for determining tensile strength from hollow cylinder tests in which the sample may be subjected to external and internal all round pressures as well as an axial stress. Table 6.1 shows some typical values of compressive and tensile strength of some rocks. The agreement between measured tensile strength and that calculated from modified Griffith criteria (eq. 6.6) is generally good.

6.3 Strength of soils in tension

Attempts to measure tensile strength of soils have been mainly confined to disturbed and recompacted soils. Haefeli (as reported by Tschebotarioff et. al. 1953, and Haefeli, 1951.) conducted early tension tests by

subjecting a reconsolidated soil specimen to centrifugal tensile forces. However the determination of stress/strain relationships does not appear possible by this method. Later he used a triaxial apparatus in which fixing of ends to platens was achieved by freezing.

Tschebotarioff, Ward and De-Phillippo (1953) carried out tests to determine the tensile strength of re-compacted montmorillonite, illite and kaolin and other artificially prepared soil mixtures.

Corresponding tests were also done to determine strength in shear but it is not made clear whether these tests were drained or undrained. It appears, however, that these are more likely to be undrained tests.

The apparatus is shown schematically in fig. 6.1. The sample is in the shape of a large briquette and the large size permits the soil to be compacted by Proctor compaction. One end of the specimen remains fixed and stationary during the test, while the rest of the section can move on frictionless ball bearings. The side sections along the central 18 inches length of sample are removed during test and strain measurements are taken along this gauge length. They concluded that highest tensile strengths and the largest failure strains of pure clays were obtained for montmorillonite and the lowest for kaolinite and that "the angles of internal friction of some natural clay minerals and of sand clay mixtures when determined from Mohr circles of uniaxial tension and compression tests are considerably higher than the angles

usually considered possible", - clearly implying the existence of a curved Mohr failure envelope.

Hasegawa and Ikeuti (1964) also tested compacted clays in tension under undrained conditions. The samples were compacted in 'dog-bone' shaped moulds, 19 cm long and 2 cm across the middle portion. Tensile pull was applied through hooks buried in the soil.

Suklje and Druovsek (1965) conducted hollow cylinder tensile tests on the lines of solutions presented by Jaeger (1962) as mentioned earlier, but no results have been published.

Frydman (1967) conducted indirect tension tests on stabilised soil in the manner of tests done on concrete and showed that the tensile strength in terms of total stress was in reasonable agreement to that predicted by Griffith theory.

Only one drained tension test on clay seems to have been published (Conlon, 1966). This was done for a clayey silt of low plasticity (L.L. 20-25; F.I. 3.7-4.6; clay fraction: 26 - 34%) when a slide in Toulmoustouc River, Quebec, took place. The sample for these tests was necked in the middle and it was held firm on to the end platens by using clamps and fine emery paper to provide the grip. The 3 inches long sample failed in 10 hours at an elongation of 0.01 inch and showed a tensile strength of 2.4 psi.

Thus, an almost complete lack of data on drained tension tests lead to work being carried out at Imperial College. A new method of holding the sample firmly to the platens, without the use of end clamps was evolved. The theory and the tests were briefly described by Bishop and Garga, (1969) and will now be mentioned.

6.4 Drained tension tests

6.4.1 Theory

Referring to fig. 6.2 (a), a sample with a reduced centre section enclosed by a rubber membrane sealed to the end caps will only become detached from the end under the action of an axial tensile force T when the average effective stress at the ends of the sample (section E_1 and E_2) drops to zero. At this stage the axial effective stress throughout the central section will be negative and the magnitude of this tensile stress will depend on the ratio of end-section and mid-section areas.

Notation:

σ'_E average axial effective stress on the sections E_1E_1 and E_2E_2 ,

σ'_c axial effective stress on the centre section,

σ'_a ambient cell pressure,

A_E area of end section

A_c area of centre section,

T tensile force,

u back pressure applied to the drainage connections.

$$\text{Thus, } \sigma_E' = \sigma_a - u - T/A_E$$

$$= \sigma_1' - T/A_E \quad (6.7)$$

$$\text{since in an extension test } \sigma_1' = \sigma_a - u.$$

$$\text{Similarly, } \sigma_c' = \sigma_a - u - T/A_c$$

$$= \sigma_1' - T/A_c \quad (6.8)$$

The limiting value of T is given by $\sigma_E' = 0$ i.e. $T = \sigma_1' A_E$.

Hence the limiting value of σ_c' from equation 6.8 is:

$$\begin{aligned} \sigma_c' &= \sigma_1' - \sigma_1' (A_E/A_c) \\ &= -\sigma_1' (A_E/A_c - 1) \end{aligned} \quad (6.9).$$

A controlled rate of strain test with constant cell pressure will thus be a test with $\sigma_1' = \sigma_2' = \text{constant}$ and σ_3' decreasing until the peak stress difference $(\sigma_1' - \sigma_3')$ is reached, unless, of course, the tensile stress $(-\sigma_3')$ exceeds the limit given by equation (6.9). For the dimensions used in the current tests (end diameter of 1 inch and central diameter of 0.75 inch), the limiting $\sigma_c' = -0.78 \sigma_1'$.

6.4.2 Description of the apparatus

The present series of tests was carried out in a controlled rate of strain testing machine using a standard triaxial cell (Bishop and Henkel, 1962) as shown in fig. 6.3, and plate 6.1.

The cell base is fixed to the moving platten of testing machine with three 'G-clamps'. The clay sample rests on a coarse porous stone on a 1 inch diameter base with bottom drainage channel. On top of the sample rests another 1 inch diameter, 1/8 th. inch thick coarse porous stone. This in turn carries a perspex sample cap with a slot as shown in the diagram. The tensile load is applied to the stainless steel ram through a 'Mollart' Hooke's joint. Inside the elongated cell, the load is transmitted through a sensitive proving ring fitted with an inductance transducer. Another Hooke's joint is placed between the brass connector and the proving ring to ensure a uniform distribution of tensile stress. The connector is lowered into the slot of the sample cap after the cell has been assembled and is turned through 90° to be in a position to engage the cap when an axial displacement is applied, the clearance permitting free movement during initial consolidation or swelling.

The tensile load on the proving ring is read with a Boulton Paul C - 61 meter giving 600 units/lb; an automatic plotter being used when the apparatus is not attended. The volume changes are small and are measured by the

displacement of an air bubble in a 1/8 inch diameter saran tubing. Axial extension is measured in the conventional manner by attaching a dial gauge (0.0001 inch) to the ram outside the cell.

6.4.2.1 The convoluted rubber membrane

The sample was enclosed in a convoluted rubber membrane which was specially prepared for the test in the laboratory. The need for this type of membrane arose because it was found that a conventional membrane withstood considerable tensile load as straining along the central section of the sample took place. The load carried by the membrane was a function of its extension over a very small length.

The membrane was made by successively applying coats of diluted prevulcanised latex 'Revultex' on a finely polished perspex former. The former was rotated in a lathe and the latex was applied evenly with a brush. After each application, the former was kept in an oven at 55°C for at least 6 hours. When the membrane was at least 0.005 inch thick, it was carefully peeled off the former using talc powder to prevent the sides from sticking; washed in cold water and pressure tested for any leaks.

6.4.2.2 Constant pressure system

It is essential to keep constant cell pressure in view of very low effective cell pressures used (3 to 10 psi.). This is achieved by using the technique (Bishop and Henkel

1962, Som, 1968) of connecting cell pressure and back pressure through the mercury pot system such that any change of pressure in one line is equally reflected in the other, thus maintaining a constant effective cell pressure. This is illustrated in fig. 6.2 (b).

6.5 Results and Discussion

The results of a series on 'intact' (visually free from fissures) samples of the blue London clay from Wraysbury with their axis horizontal are presented in table 6-2 and fig. 6.4. Two typical stress/strain curves are also presented in figs. 6.5 and 6.6.

Fig. 6.4 also shows drained compression strength envelope for 'intact' samples, 0.75 in. in diameter, cut with their axis horizontal. Only the results in the lower stress range are given in this figure. References may be made to Bishop et. al., (1965) and Bishop, (1967) for the strength - effective stress relationship over a wider range of stress. From fig. 6.4, it is evident that London clay can withstand a substantial tensile stress in drained conditions, and that this stress is almost independent of the value of σ'_1 in the range examined (3-10 psi.). The tensile stresses observed (3.8 to 4.8 psi.) lie very close to the values predicted from compression strength by the modified Griffith theory. The theoretical relationship is given (Hoek, 1965) by eq. 6.6,

$$\sigma_c = \frac{4 \sigma_t}{\sqrt{1+\mu^2} - \mu}$$

where σ_c is the unconfined, drained compression strength and σ_t denotes drained tensile strength. The coefficient μ denotes the tangent of the angle of friction along a microfissure. This may be taken to have a value of at least $\tan 18.5^\circ$ (Skempton et. al., 1969) and may rise or even exceed $\tan 30^\circ$ at low stresses. Substituting in eq. 6.6 gives value of σ_t/σ_c of 0.178 and 0.144 respectively. For a value of compression strength of 28 psi., the predicted values of tensile strength are 5.0 and 4.0 psi. respectively. Hence a very close agreement between predicted and experimental values is found.

In all the cases, the mode of failure was typical of brittle fracture as shown by the photomicrograph plate 6.2(a) which shows the interesting case where the fracture was still on a typically horizontal plane despite the presence of a vertical microfissure. (The 'vertical' microfissure in the test was of course horizontal in the ground). Appreciable strains developed before brittle fracture took place - the highest strains were achieved for samples at very low effective cell pressures thus indicating a high coefficient of volume expansion even when the clay is in 'intact' form.

In fig. 6.4. are also shown the results of compressive strength from undrained tests with pore water pressure measurement for remoulded clay from the same area (plasticity index = 42). The cohesion intercept is less than 0.2 psi. and the theoretical tensile strength is almost negligible. Thus, remoulding seems to have

destroyed any bonds that could withstand tensile stresses. In the case of the slide in the Toulmoustou River mentioned earlier, it was suggested that cementation bonds were destroyed by tension resulting from (a) diagonal tension due to shear; (b) flexural tension due to bending and (c) direct tension due to local dilatancy, (Conlon 1966).

Further research may be fruitfully extended in carrying a series of tension tests on this clay after it has been reconsolidated at high pressures from slurry to ascertain whether reconsolidation (or compaction) helps to develop bonds capable of withstanding appreciable tensile stresses.

Cracking of dams and embankment under tensile stresses is an important problem for which almost no analytical solutions are yet available. The author's colleague, J. Laing, is currently engaged on this by using finite element techniques. Once such solutions are available which predict the magnitude of tensile stresses in a clay embankment, the soil can then be tested by using the technique described in this chapter to investigate whether it would be able to withstand these stresses. Special attention could then be given to problem zones in the embankment which are likely to be subjected to tensile stresses.

6.6 Conclusions

An accurate triaxial apparatus and testing technique has been developed which enables the tensile strength of clays to be determined in the laboratory. The intact samples tested in a horizontal direction show a tensile strength of 3.8 to 4.8 psi. in the major principal stress range of 3-10 psi. The experimental values agree closely with the theoretical values predicted by the modified Griffith theory. This evidence provides a convenient method of assessing the tensile strength of undisturbed and compacted clays provided the shear strength behaviour under compression in low normal stress range is known. The fact that intact London clay can withstand tensile stresses suggests that progressive failure can take place in this and other stiff clays. It is also shown that remoulded London clay would show insignificant tensile strength due, by inference, to the destruction of bonds.

Table 6.1 (after Jaeger and Cook, 1969)

Some tensile and compressive strength values of rocks.

Rock type	measured		σ_c / σ_t	μ	$\sigma_t = \frac{1}{4} \sigma_c [(\mu^2 + 1)^{\frac{1}{2}} - \mu]$
	σ_t psi.	σ_c 10^3 psi.			
Quartzite, Cheshire	4060	66.8	16.5	0.9	7420
Granite, Westerly	3040	33.2	10.9	1.4	2655
Diorite, Frederick	5800	70.5	12.1	1.7	4700
Sandstone, Gosford	520	7.25	13.5	0.5	1320
Marble, Carrara	1000	13.0	13.0	0.7	1730
Trachyte, Bowral	1990	21.75	10.9	1.0	2290
Quartzite, Witwatersrand	3000	28.0	9.3	1.0	2950

Table 6.2

Tension tests

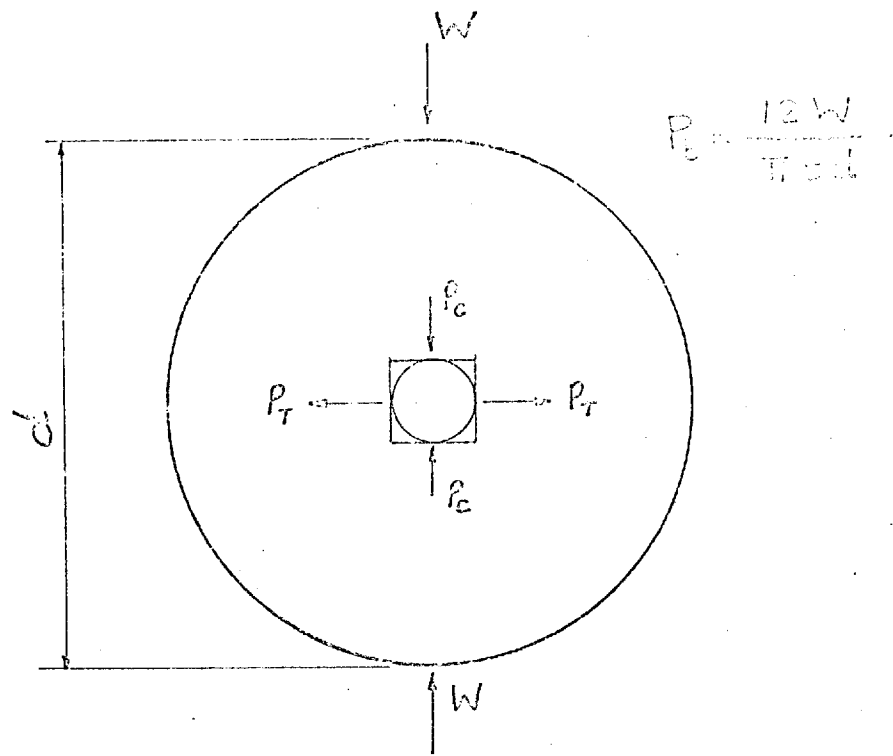
Cell Pr (psi.)	$(\sigma_1 - \sigma_3)_f$	σ_{3f} psi.	$\frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$ 3	Initial w/c %	Final w/c at failure zone	ϵ_f % on mean gauge length of 1.0 in.	Time to failure (hours)
3.0	7.83	-4.83	0.39	29.9	31.73	16.7	55.2
5.0	9.02	-4.02	2.0	29.9	34.4	9.4	43.3
6.0	10.64	-4.64	2.45	28.0	29.0	2.66	6.7
8.0	11.82	-3.82	4.06	30.6	34.2	7.55	10.4
10.0	13.84	-3.84	5.38	29.9	29.9	2.19	8.9

Compression tests

Cell Pr	$(\sigma_1 - \sigma_3)_f$	ϵ_f %
5.0	45.0	2.18
0.0	28.0	1.35

Index properties

W_{L.L.}% 75; W_{P.L.}% 29; W_{P.I.} 46; % Clay fraction 58.



(after Hobbs, 1964)

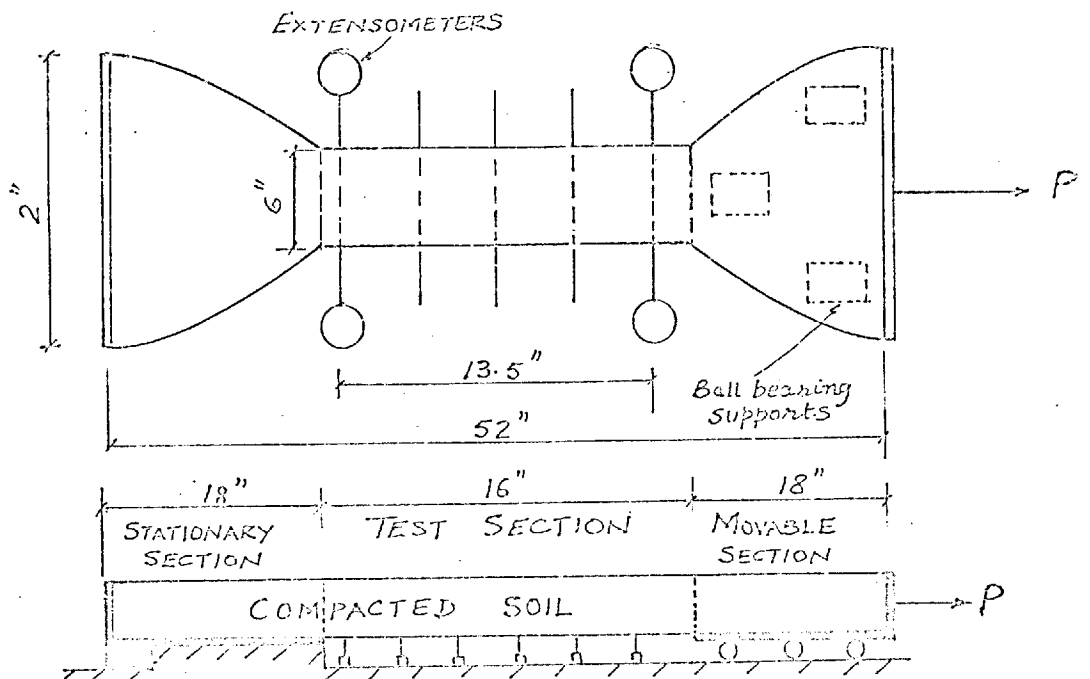
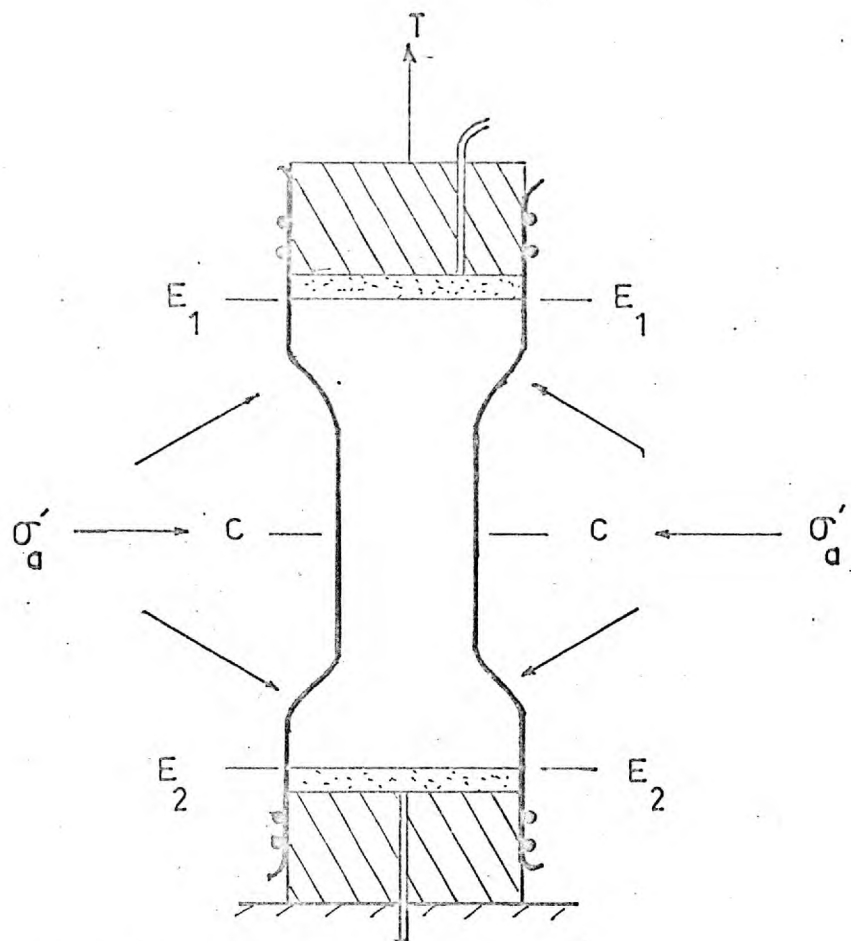
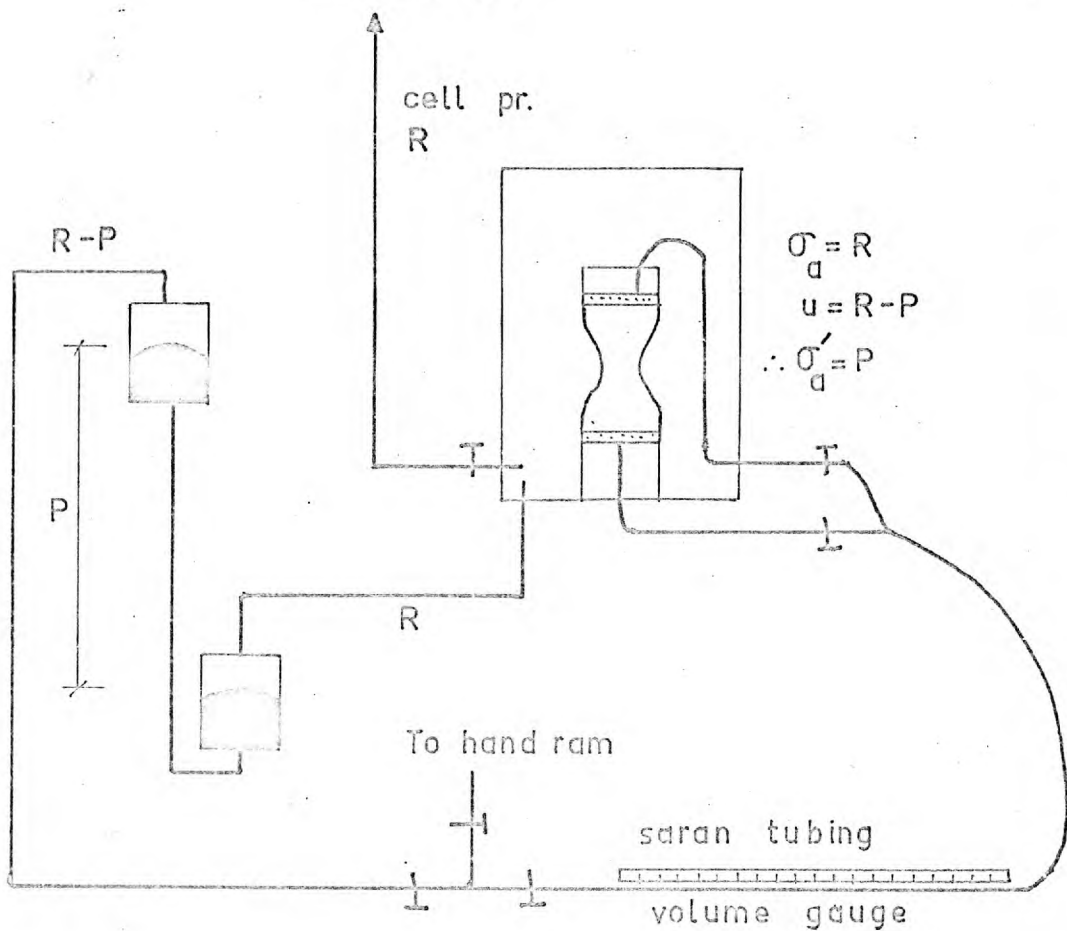


FIG. 6.1: (after Tschiboroff, et al., 1953)



(a) PRINCIPLES OF TENSION TEST

mercury pot



(b) CONSTANT LOW CELL PRESSURE SYSTEM

(not to scale)

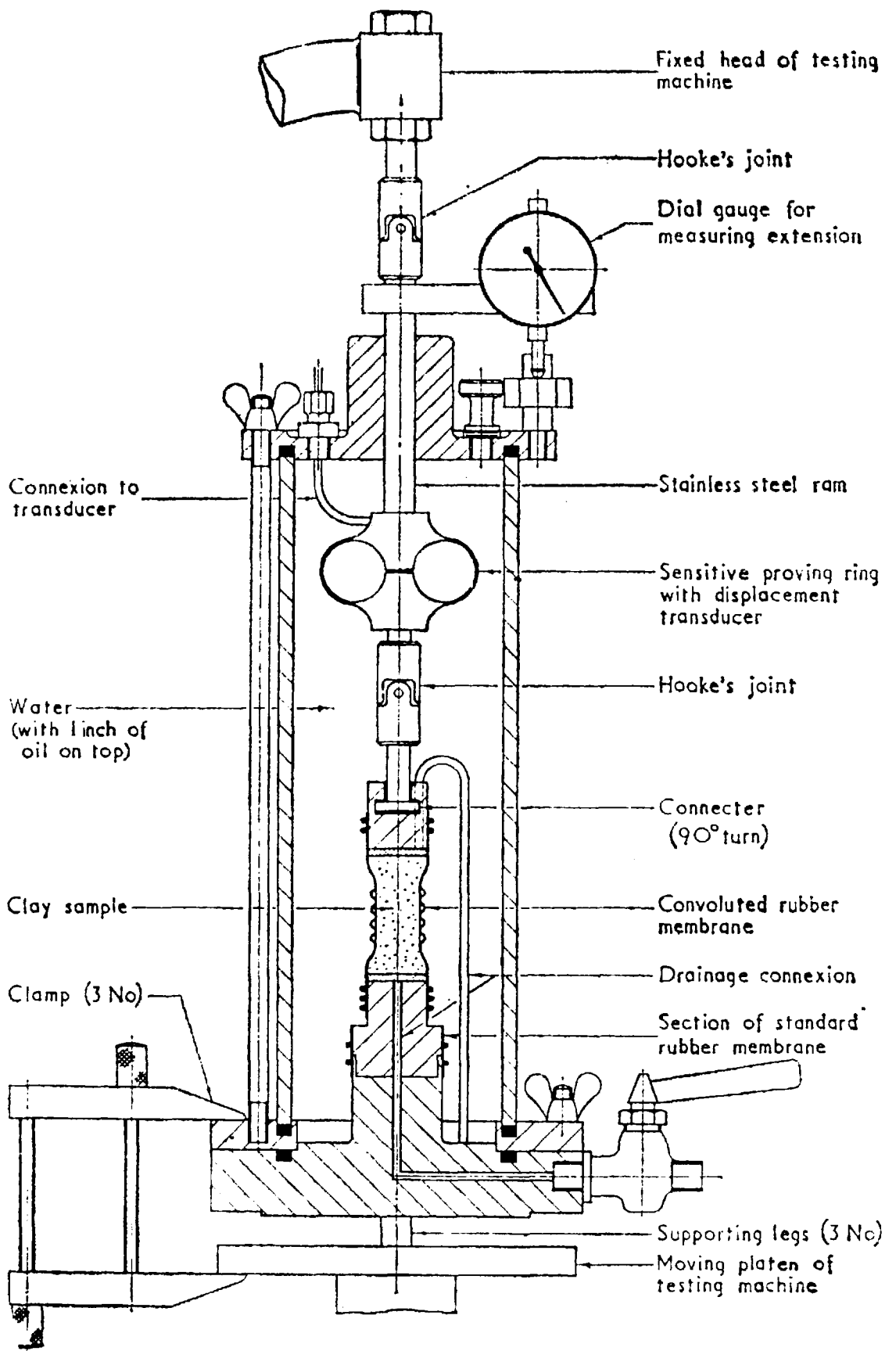


FIG.6.3.

Triaxial cell modified for tension tests.



PLATE 6.1

Tension test apparatus.

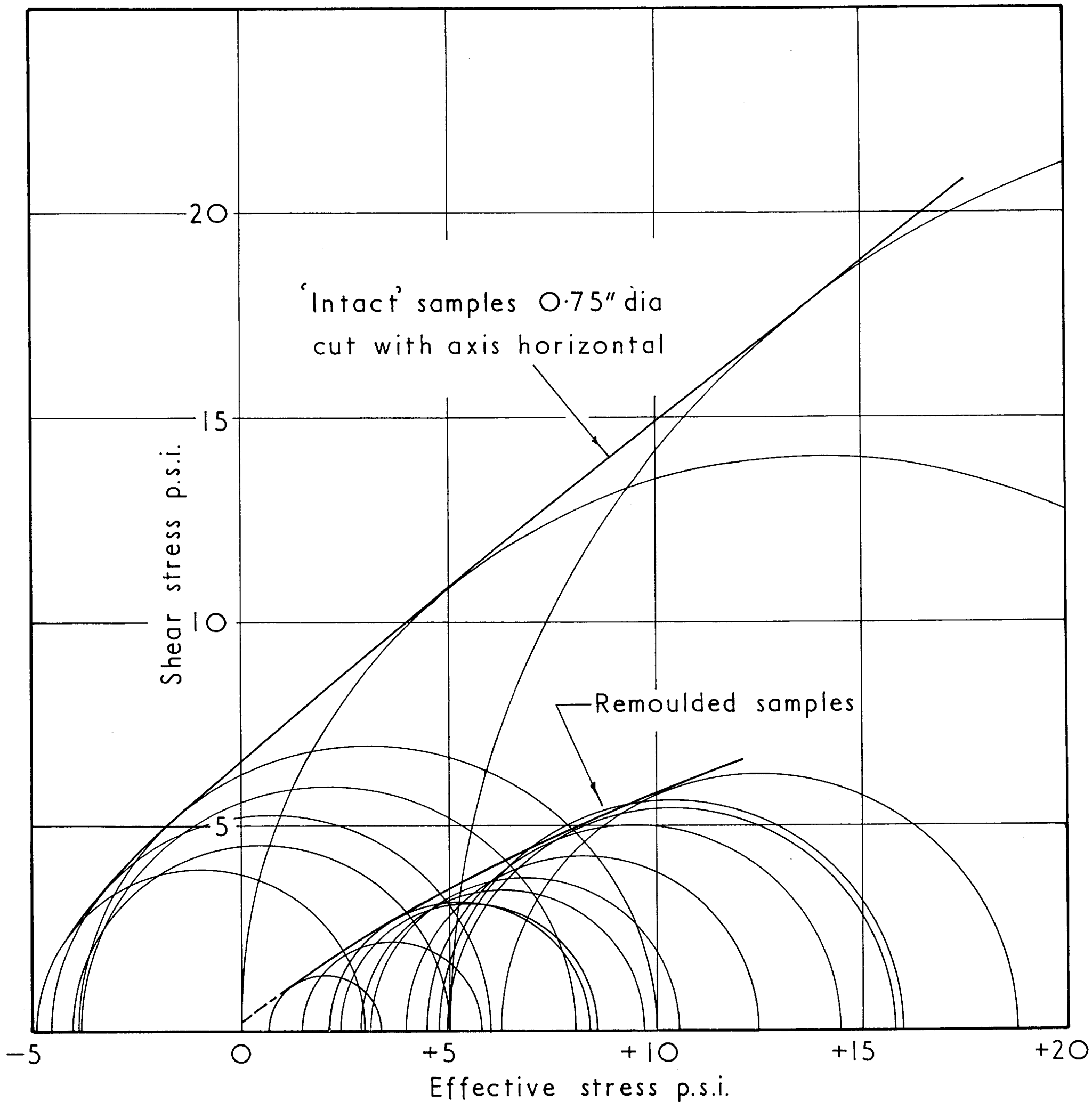
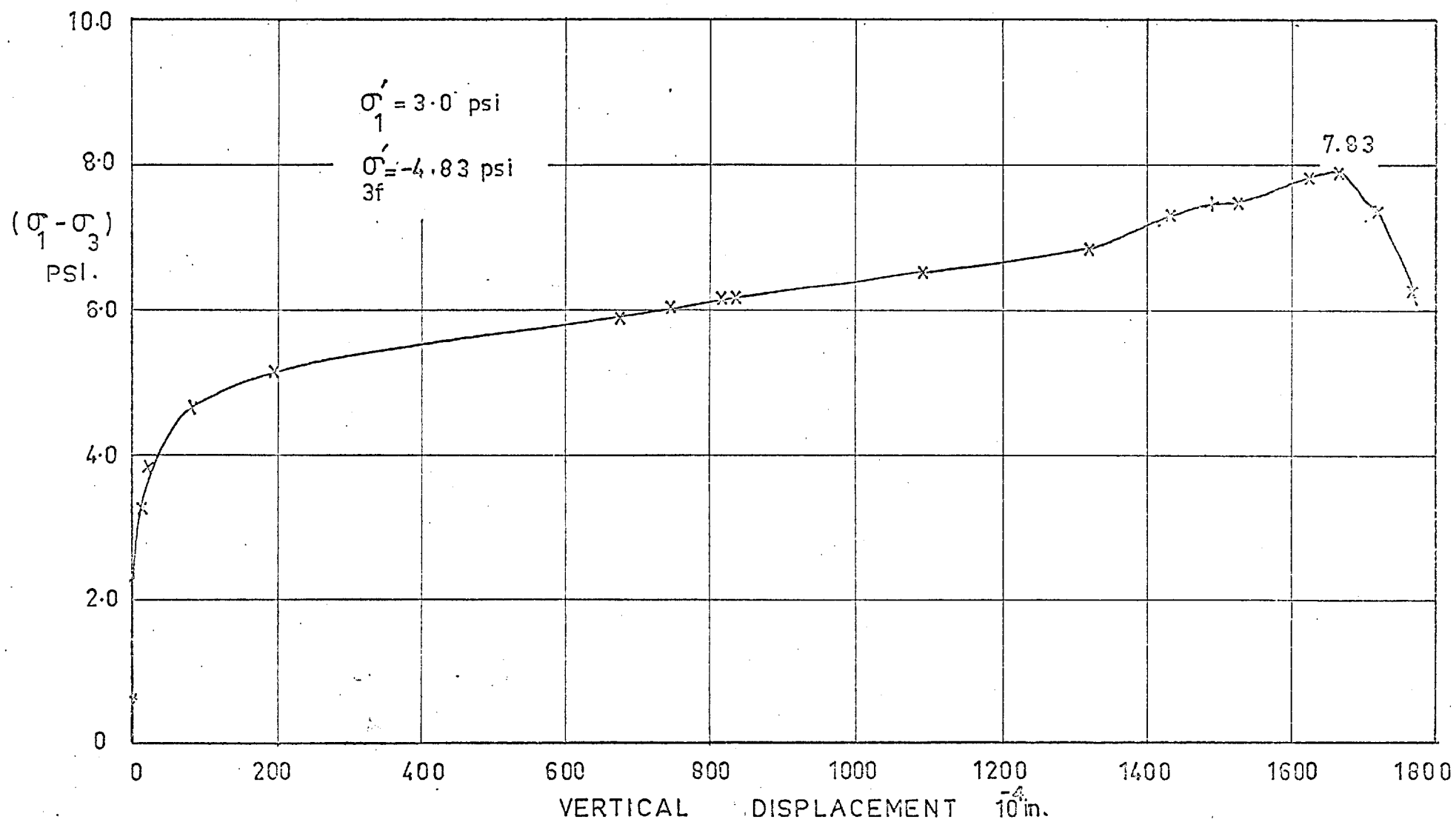
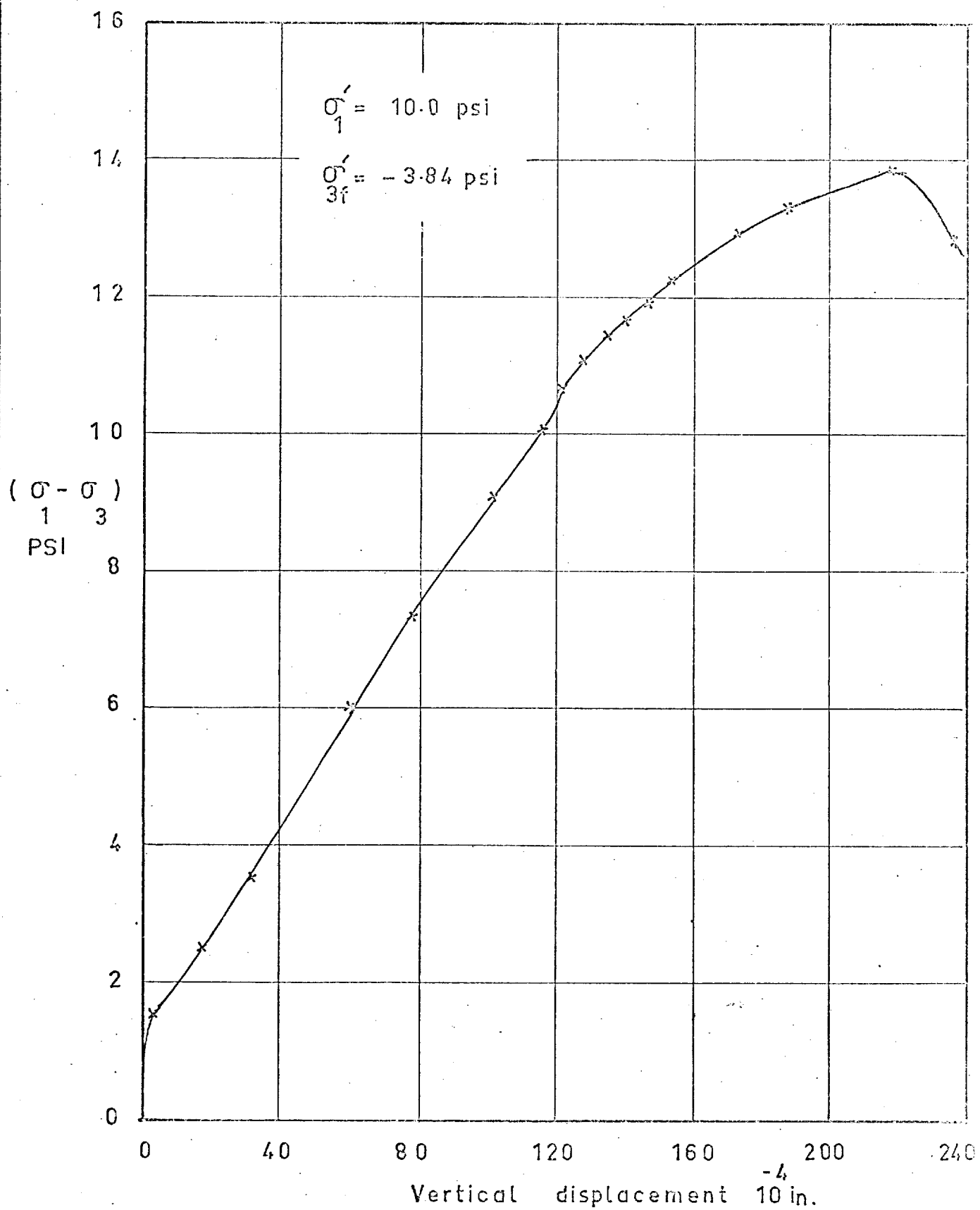


FIG.6.4 : SHEAR STRENGTH IN LOW STRESS RANGE



STRESS DISPLACEMENT CURVE

FIG. 6.5



STRESS - DISPLACEMENT CURVE



(a)



(b)

PLATE 6.2

Brittle tensile failure in blue London clay.

CHAPTER 7

Concluding remarks

Published case records suggest, at first sight, that a slope in stiff clay slope may show a slightly different residual strength than that obtained from the ring shear apparatus. Some of the reasons for the discrepancy between field and laboratory behaviour are discussed below.

There is evidence to suggest that the rate of shearing has very little effect on residual strength and the effect is relatively more marked on peak strengths, (Petley, 1966; Kenney, 1967; Skempton and Hutchinson, 1969). Petley (1966) investigated the influence of rate of strain on residual shear strength by using direct shear box tests. Taking the residual strength at a rate of displacement of 0.06 in/day as 100%, his results show that the strength increased by 3.1% at a rate of displacement of 6 in/day and dropped by 1.2% when the shearing rate was decreased to 0.003 in/day, in tests on brown London clay.

The author carried out tests in the ring shear apparatus to investigate this effect. These tests were carried out on the sample of slurried brown London clay, and the results are presented in fig 7.1. The residual strength at rate of shear of about 0.2 in/day (0.00015 in/min) has been taken as 100%. Again the

rate of strain appears to have negligible effect on residual strength and decreases by about 1% per log cycle of shearing rate. It would appear that once movement starts taking place on a thin, well defined smooth plane, the excess pore water pressures generated as a result of excessive rates of shear are able to diffuse in the adjacent material thus showing very small effect on the strength along the slip surface. It may perhaps be possible that the shear plane with strong clay particle orientation is able to act as a 'drainage channel' through which the excess pore water pressures generated on the slip surface can dissipate. Similar conclusion was also reached by Tchalenko (1967) on tests on cut plane kaolin sample tested in direct shear box, who found virtually no increase in residual strength at rates faster than 0.06 in/day. Kenney (1967) also investigated the influence of rate of strain on residual strength of a number of clays, including montmorillonites and found it to be negligible. The above information suggests that considerably larger rates of strain than usually employed can be used to measure residual strength after peak strength has been overcome and a well defined shear plane has been formed.

Skempton and Hutchinson (1969) have tabulated a number of cases of both pre-failure and post-failure movements in slopes. The average velocities of movements lie in a wide range from almost imperceptible (Gould, 1960) to about 50 cm/day. The prefailure

movements in stiff clays generally accelerate with time and achieve large rate of movement just prior to slipping. However, it is highly unlikely that field residual strength would vary significantly as rate of movement increases.

Some work has been done to investigate the effect of rate strain on peak strength. Bishop and Henkel (1962) describe drained triaxial tests on remoulded Weald clay and found that the peak strength dropped by about 3.5% per log cycle of time. The effect of time to failure on peak undrained strength of clays are now well appreciated and some test results have been reported by Casagrande and Wilson, (1951); Bjerrum, Simons and Torblaa, (1958) and Skempton and LaRochelle, (1965). Casagrande and Wilson showed that the undrained strength of Bearpaw shale measured in a test of one week and 30 days duration was about 20% and 22% less respectively than the strength from conventional rapid tests. The average loss of strength of Cucaracha clay shale was less by as much as 60% and 65% after a test duration of one week and one month respectively.

In the conventional limit equilibrium method of long term stability analysis, it is assumed that at the moment of failure, residual strength operates all along the length of failure surface. However in practice, different strengths operate along the failure surface due to the development of non uniform stress and strain conditions (Bishop, 1952; Turnbull and Hvorslev, 1967;

Bishop, 1967). For example, an analysis by finite differences and relaxation method of a uniform, elastic, cohesive embankment by Bishop (1952) shows that some local overstress develops when the factor of safety of an undrained circular slip passing through it is as high as 1.8. However, for a long term analysis, the overstress would first develop at either ends of the slip surface and proceed inwards, (Bishop, 1967). Peck (1967) has quoted an interesting analysis by Conlon. He commented on the observation that the horizontal displacement required to achieve peak failure in a direct shear box test was a function of effective normal stress, i.e. for high normal stress greater displacement would be required than for smaller normal stress. Since in a circular failure, the average normal stress can vary considerably along the failure surface, it would imply that for a given deformation, the ends and the middle of the slip would not necessarily show the same shear strength. It is quite possible for some portion of the failure surface to show peak shearing resistance while other portions may be near the residual state. The situation is further complicated when the soil shows a residual shear strength which is dependant on the magnitude of effective normal stress, as some tests on ring shear apparatus have shown. For a deep seated slide, there could be considerable variation in the magnitude of residual strength operating along the failure surface because of the variation of normal stress along it. Also, the magnitude of displacement required to drop strength to residual is a

function of the degree of particle orientation in the direction of movement. In most stiff clays the particles are generally oriented in the direction of the horizontal bedding planes. In a situation where a slip surface traverses across horizontal bedding planes and subsequently proceeds along them, different shear strength will be mobilized in directions parallel and perpendicular to bedding planes for the same amount of movement. Agarwal (1967) found from multiple reversal direct shear tests on blue London clay that the angle of 'residual' strength after five reversals for shearing parallel and perpendicular to bedding planes was 14.5° and 16.5° respectively. Similarly, James (1970) found that the cross-bedding strength of heavily overconsolidated Oxford clay did not fall to the same value even after seven reversals in direct shear box as the strength along bedding planes after one reversal only. Hence the average value of shearing resistance on a slip surface can lie at some value between the peak and residual. A major advance in the method of analysis would be made with the development of techniques which would enable the shear stress at any point to be evaluated from the appropriate stress-strain relationship of the soil.

As assumption which is generally made in analysing slope stability problems is that it is two dimensional, i.e., the shear forces acting on the two sides of the section being considered are zero. Kenney (1956) has shown that where the slip is a section of a sphere

(i.e. it is 'saucer-shaped'), in which case the ratio of length to width of the slip is 1, then the factor of safety, assuming it to be a two dimensional case, would be underestimated by about 6%. For slip surfaces where the running length is at least twice the width (in direction of slip movement), the field factor of safety would always be greater than the calculated value.

The significance of fissures in stiff clays which assist in softening and hence the reduction in strength of the clay is now well recognised. However, there may be other factors due to which the clay may show a decrease in shear strength. Perhaps the most important factor is the presence of internal structural discontinuities which may be formed either in the depositional or post depositional period.

Examples of depositional features are bedding planes, laminations, syneresis cracks and contorted layers, (Skempton and Northey, 1952; Moore and Scrutton, 1957). The clay may achieve preferred particle orientation due to above features. Submarine slumping is a powerful mechanism which can produce a slip surface in the unconsolidated sediment. Morgenstern (1967) has carried out a review of submarine slump features in sediments. He suggested that the most common mechanism that can induce slumping is oversteepening of slope which may occur due to crustal tilting as a result of tectonic activity (see also Shepard, 1963). Severe earthquakes can apply large accelerations to submarine

sediments and trigger off slides. An example of such a case in the Greek Archipelago has been presented by Ambraseys, (1960). Henkel (1969) has, both analytically and experimentally, demonstrated that slides in under-consolidated sediments can take place as a result of pressure of waves. Due to periodic nature of waves, the stress exerted by them reverses in direction each half cycle and the underlying soil is subjected to cyclic stress which can cause remoulding and reduction in shear strength of the sediment. Subsequently, due to rise in land mass or lowering of sea level the sediment is exposed together with slip surfaces and other internal structures formed during the depositional period.

Folding and fracturing of bedding planes can be caused by tectonic activity. Skempton (1966) has described the example of shear zone at Mangla, where the strength of clay had dropped significantly from the peak shear strength value as a result of inter-bedding movement caused by folding of the beds.

When stiff clays are eroded and the overburden pressure is removed, they undergo unloading under K_0 conditions. As a result, large locked in horizontal stresses are released which may cause weaker beds to move laterally, thereby causing a movement along the horizontal bedding planes. This may be especially true for clays with high swelling potential, e.g. bentonite seams which would expand on becoming moist. It is likely that fissures are caused as a result of horizontal

stress release, though it should also be noted that there is no evidence to suggest that any shear movement has taken place along the surface of a fissure and the orientation of clay particles adjacent to a fissure is the same as that of the clay mass when observed in an optical microscope (D'Elia, 1968). The release of horizontal stresses could also have been caused by undercutting of valley slopes; Bearpaw shale and Pierre shale are good examples of such a case. Yudhbir (1969) has observed on tests on different stiff clays that strength under drained K_0 unloading is much smaller than the drained compressive strength.

Creeping of clays can also bring about a decrease in peak strength, Lovenbury (1969) on the basis of long term creep tests on brown London clay where σ'_n was decreasing suggested that the effect may account for about 15% drop in strength from the peak value.

In many instances, for example, in a slope or behind a retaining wall, the clay is often subjected to alternating stresses, which may be caused by seasonal variations in groundwater table and climatic variations. DeLory (1957) attempted to study the decrease in strength in brown London clay by altering pore water pressure under drained conditions, and maintaining total normal load constant under dead loading. The sample was subjected to 36 alternating stress cycles and it was noted that failure was caused at stresses smaller than would be

necessary in a drained test.

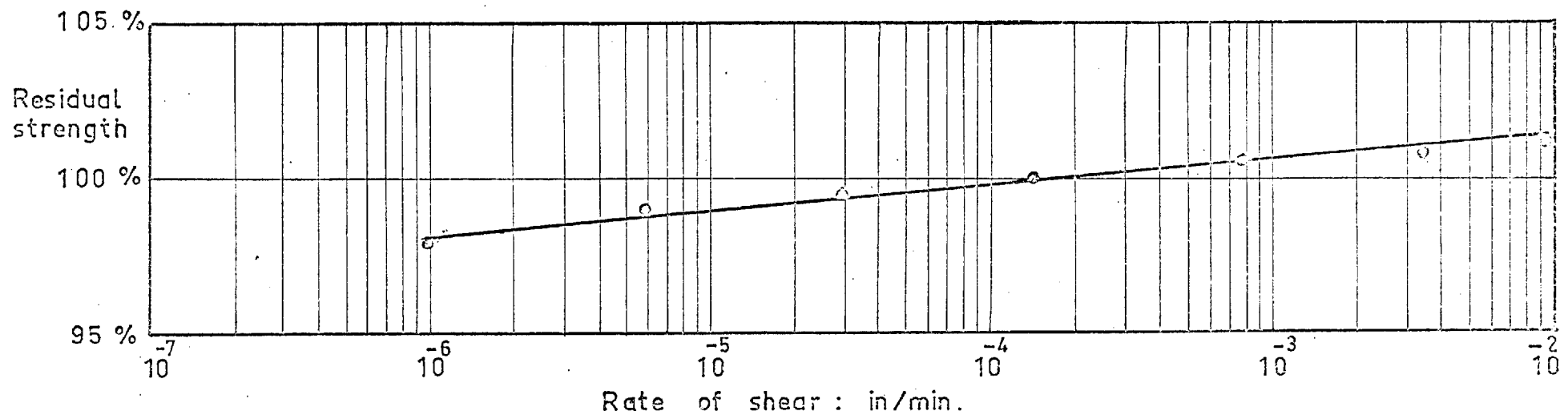
Thus in the above discussion, some reasons for loss of strength and the development of slickensides and slip features in overconsolidated stiff clays have been discussed. For such clays it is necessary to determine the residual shear strength to assess long term stability of slopes. A new ring shear apparatus has been described which enables residual shear strength to be determined accurately. Unlike other residual tests, this apparatus permits very large uni-directional deformations to take place, thus allowing clay particles on slip surface to achieve maximum particle orientation. It is again emphasised that visual inspection of reversal direct shear residual test curves can be extremely misleading since a closer scrutiny reveals that this test does not enable the residual shear strength to be achieved. It has also been shown that a number of clays exhibit marked variation in residual strength with the magnitude of effective normal stress. For such clays, it is essential that residual strength should be determined in the ring shear apparatus at the appropriate normal stress which is acting on a potential sliding surface. More detailed discussion and conclusions were presented in chapter 4. Thin section studies and electron micrographs, shown in chapter 5, show some features associated with large displacements and also provide some qualitative evidence that particle breakdown or segregation of clay particles takes place on shearing.

Results of some drained tension tests on intact London clay have been discussed in chapter 6, and the results show close agreement with modified Griffith theory. The fact that London clay can withstand some tension suggests that it is possible for slip surface to propagate progressively as a result of local stress concentration.

Some suggestions for further research

The practical significance of a curved shear strength envelope is immense and further ring shear tests on pure clays, clay mixtures and natural soils should be conducted to investigate the factors which cause this behaviour. There is some evidence from Kenney's, (1967) work that montmorillonitic soils show such a behaviour and this aspect should be further explored.

The use of the manual ring shear apparatus and the subsequent testing of shear surface in the triaxial apparatus should be further explored, and it is believed that this technique would be found to be more accurate than the multiple reversal (or cut-plane) direct shear tests.



Slurried Brown London Clay

EFFECT OF RATE OF SHEAR ON RESIDUAL STRENGTH

FIG.7.1

PART BCHAPTER - 8

Introduction; previous studies on consolidation of London clay

8.1 Introduction

It has been observed that overconsolidated, stiff soils like the London clay generally give a lower magnitude of settlement and faster rate of settlement in the field than that predicted from laboratory tests on conventional 3 in. diameter oedometer samples, (MacDonald and Skempton, 1955; Cooling and Gibson, 1955). The study of the phenomena of soil deformation is often made more complicated by marked inhomogeneity of the insitu clay especially with regard to the degree of fissuring, three-dimensional drainage and anisotropic stress conditions in the soil.*

The prediction of the rate of settlement in stiff soils is still very crude. This is so mainly because most calculations are based on the assumptions of Terzaghi theory of one dimensional consolidation, whereas the flow of water

* Conflicting evidence exists as to whether the volume change of soils is governed primarily by the major principal effective stress or whether it is controlled by the magnitude of average principal effective stress. Henkel (1958), for example, found from tests on remoulded clay that there was a unique relationship between the average principal effective stress and the water content at failure, though this was shown to be not so for remoulded Weald clay under K_0 consolidation, (Henkel and Sowa, 1963). Similar conclusions were also reached by Wade (1963), Sowa, (1963) and Akai and Adachi (1965). However, Rutledge (1947), Bjerrum (1954), and Raymond (1965) found that the volume deformation is principally controlled by the major principal effective stress. It would therefore appear that no general hypothesis can yet be applied to all soils.

in the field is often three dimensional. This is especially true for London clay which is fissured and jointed and exists in deposits of great depths. Actually, even if the flow is one dimensional, departures from Terzaghi theory are still observed because of the non-linear behaviour of the soil. Non-linear consolidation characteristics have not been specifically considered in this project, and as such only a brief mention of the work done in this field will be made.

8.2 Brief outline of work on non-linear consolidation

Schiffman (1958) considered the case of one dimensional consolidation with variation in permeability and time dependant loading, while Gibson (1958) has considered the case of a clay layer whose depth varies with time. Schiffman and Gibson (1964) in an important paper considered one dimensional consolidation of thick clay layers in which the coefficient of compressibility and permeability vary with the depth of the clay stratum. These assumptions are especially valid for London clay loaded at a shallow depth. They were able to show that in such a case, nonhomogeneity in k and m_v alone could substantially account for faster rate of consolidation observed in the field. Davis and Raymond (1965) extended Terzaghi theory by considering a non-linear void ratio - effective stress relationship and the effect of pressure increment ratio. Barden and Berry (1965) formulated a new non-linear theory for normally consolidated clays in which both k and m_v are allowed to vary as the consolidation proceeds. Recently, Gibson, Hussey and England (1967) have derived

a theory of one dimensional consolidation for saturated clays using Darcy's Law in a form in which the relative velocity of the soil skeleton and the pore fluid is related to the excess pore water pressure gradient. More recently, Poskitt (1969) developed another non-linear theory which embraces the cases and concepts developed by Davis and Raymond (1965), Barden and Berry (1965) and Gibson et. al. (1967). Moreover, a special feature is that results have been presented in the form of convenient graphs. It should be noted that all the references mentioned above deal with one dimensional consolidation only. General three dimensional solutions for wide range of soil properties are not yet available, although solutions for a few cases, especially with regard to radial flow in a sand drain, have been developed. (For example, Barron, 1948; Richart, 1957; Gibson and McNamee, 1957, 1960, 1963; Hansbo, 1963; Davis and Foulos, 1968; Tan, 1968; etc.).

8.3 Previous study on London clay

Much work has now been done to investigate the effect of fissures on shear strength of London clay. Simons (1967), Agarwal (1967), for example found that the undrained strength of 12 in. diameter samples of undisturbed, fissured London clay is about 65% that of 1.5 in. diameter samples. Other work on shear strength has been described by Bishop et. al. (1965); Ward et. al. (1965) and Bishop and Little (1967). However, surprisingly, no record is found in the literature of any work undertaken to study the consolidation characteristics of London clay by either

conducting insitu tests or testing different sized samples in the laboratory. The work done in the past on the consolidation of London clay seems to have been confined to testing samples in the conventional oedometer. Cooling and Skempton (1941, 1942) and Skempton (1944), carried out such tests with drainage in vertical direction only. Skempton and Henkel (1957) described oedometer tests on London clay from deep borings at Paddington, Victoria and the South Bank. They presented profiles of variation of compressibility (m_v) with depth which showed that m_v at shallow depths of about 10 ft. was about 4 - 15 times that at greater depth (about 100 ft.).

Ward, Samuels and Butler (1959), carried out oedometer tests on both horizontal and vertical samples and found that the ratio of horizontal to vertical coefficient of consolidation was at least 2 at shallow depths. This ratio decreased with increase in effective stress; the decrease in C_v in horizontal direction being larger than that in the vertical direction. At high pressure of about 50 T/ft², the ratio was approximately one.

Recently Agarwal (1967), also tested samples of blue London clay from Wraysbury in both horizontal and vertical directions in the oedometer. He found that on the average for pressures up to 32 T/ft², m_v for vertical samples was about 16% higher than that for horizontal samples. The value of the ratio, C_v (horizontal) / C_v (vertical) appeared to decrease from about 2.4 to 1.5 in the pressure range 0.5 - 1.0 T/ft² to 16 - 32 T/ft². However, the decrease in permeability, as calculated from

Terzaghi theory, with effective stress in both horizontal and vertical directions was observed to be the same and the ratio of k (horizontal) to k (vertical) was approximately constant at a value of 1.5.

More recently, Som (1968) has carried out tests on blue London clay from Ongar, Oxford Circus and Wraysbury in conventional oedometer, hydroulic oedometer, constant rate of strain oedometer and triaxial apparatus in which both isotropic and anisotropic tests were conducted. It was found that generally values of m_v obtained from special oedometer tests and triaxial tests lay between the upper and lower limits provided by tests in conventional oedometer under first and second loadings.

It will be seen from the above brief review that data on consolidation characteristics of fissured London clay with respect to sample size is singularly lacking. The little data that exists has been obtained mainly from the conventional oedometer tests. The limitations and uncertainties involved in the conventional oedometer have been discussed in detail by Tan (1967) and Som, (1968). The more serious of these are that there is absolutely no control over the drainage of sample. The drainage begins as soon as the load is applied, and before the sample has had sufficient time to reach peak pore pressures. Also, the mode of load application subjects the sample to shock loading, and, further there is no facility to measure pore pressure or drainage from the soil.

Hence, a short programme was commenced to investigate consolidation characteristics of blue London clay from

Wraysbury from 1.5 in., 4 in. and 12 in. diameter dissipation tests in which pore pressure, volume change and (in some cases) the change in height could be directly measured. Four Casagrande type piezometers were also installed at a site at Wraysbury, from which in-situ permeabilities were measured by conducting constant head permeability tests. Vertical and horizontal oedometer tests on samples from the same depth at which the piezometers were installed were also carried out. It may be mentioned here that secondary consolidation characteristics have not been investigated nor has any analytical work on non-linear consolidation been attempted.

Later in the project, a good opportunity arose to conduct a series of consolidation and constant head permeability tests in the laboratory on 4 in. diameter samples of boulder clay from the M. 6. Motorway trial embankment. The results obtained from laboratory tests have been compared with those obtained from in-situ tests and have been briefly described in chapter 11.

CHAPTER 9

Constant head tests (C.H.T.) in the field

9.1 General Considerations

The field determination of permeability very often gives the best indication of the field consolidation characteristics of the soil since the test is done on a relatively undisturbed mass of clay. Also, the insitu determination is more accurate since it takes radial flow of water into account. This effect can be the dominant factor in determining rates of consolidation in many anisotropic soils. For stiff clays, only the constant head type of test (C.H.T.) is generally employed and this would be considered in the subsequent discussion. The limitations of the falling head permeability test have been discussed by Hvorslev, (1951) Bishop et. al., (1964) and Tan, (1968).

Generally, the C.H.T. calculations are based on Gibson's solution (1963), for C.H.T. on spherical piezometers. He considered the case of a spherical pot radius a , at some depth d in the ground ($a \ll d$). The pore pressure u at a distance radius r from the centre of the sphere is governed by:

$$c_v \left(\frac{\partial^2 u}{\partial r^2} + \frac{2}{r} \frac{\partial u}{\partial r} \right) = \frac{\partial u}{\partial t} \quad (9.1)$$

Boundary conditions are:

at $t = 0$, $u = u_0$ for $r > a$.

and as $r \rightarrow \infty$, $u \rightarrow u_0$.

Also, if a constant differential head $\Delta u/\gamma_w$ is maintained then,

$$u(a, t) = u_0 + \Delta u; \quad t > 0.$$

$$\text{then, } Q_t = 4\pi a \frac{k}{\gamma_w} \Delta u \left[1 + \frac{1}{\sqrt{\pi T}} \right] \quad (9.2)$$

when k denotes permeability of soil,

$$\text{and } T = C_v t/a^2.$$

At infinite time, equation (9-2) reduces to

$$Q_\infty = 4\pi a \frac{k}{\gamma_w} \Delta u = F \frac{k}{\gamma_w} \Delta u \quad (9.3)$$

where F denotes the intake factor.

It is interesting to note here that the time required to achieve, say, 95% steady state value of flow, in the piezometer increases as the intake factor increases. This is in contrast with the response time of a piezometer which is inversely proportional to the intake factor. (Høvorslev, 1951, Bishop and Al-Dhahir, 1969). Gibson in a later paper (1966) extended the above analysis to take into account the permeability of the piezometer tip with respect to the permeability of surrounding soil.

9.1.1 Intake factors for cylindrical tips

Spherical tips are seldom used in practice and a certain amount of uncertainty lies in the determination of equivalent intake factors for cylindrical tips.

Al-Dhahir (1967) made a comprehensive review of intake factors of cylindrical tips and also developed solutions using numerical analysis.

Hvorslev (1951) listed a number of intake factors for various cases (see fig. 9.1). Dachler's formula (case no 8 in fig. 9.1):

$$F = \frac{2 \pi L}{\ln \left[\frac{L}{D} + \sqrt{\left(\frac{L}{D} \right)^2 + 1} \right]} \quad (9.4)$$

has been widely used for cylindrical tips (see for example, Bishop, Kennard and Penman, 1960), but it is only accurate in the region of $L/D = 1$, (Al-Dhahir 1967).

Frevert and Kirkham (1948) derived expressions for case (1A) (fig. 9.2) using electrical tank analogue. Their tests were repeated by Smiles and Young, (1965) who found that the 1948 results gave values of intake factors consistently higher by 11%. Kadir (1951) considered case (2A) in the analogue and his results are also shown in fig. 9.2.

Al-Dhahir (1967) derived numerical solutions by solving Laplace equation:

$$\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial z^2} = 0 \quad (9.5)$$

for different boundary conditions. The results agree well with those of Smiles and Young, (1965) and confirm that the results of Frevert and Kirkham, (1948) and Kadir (1951) were on the low side by about 10% and 15% respectively.

It was also found that the effect of the presence or absence of the impermeable borehole on intake factor was negligible. As an example, for a L/D ratio of 2, the intake factor for case (1E) fig. 9.2 is only 0.1% less than that of case (1C) while for a ratio of 1, the corresponding difference is only 0.4%.

Al-Dhahir (1967) also showed that for a CHT, for $L/D \leq 4$, Gibson's solutions for spherical tip may be used accurately for cylindrical tips, provided the equivalent radius r of the cylindrical tip is obtained by either of the following two methods:

- 1) by equating the intake factors of the cylindrical tip and the equivalent sphere:

$$\text{i.e. } r = \frac{\text{intake factor of cylindrical tip}}{4\pi} \quad (9.6)$$

- 2) by equating the surface areas of the two tips,

$$\text{i.e. } r = \frac{(\text{surface area of cylindrical tip})^{1/2}}{\sqrt{4\pi}} \quad (9.7)$$

It is worth emphasising that eq. (9-7) is an extremely convenient method of determining intake factor of cylindrical tips provided $L/D \leq 4$, when the intake factor of the cylindrical tip is approximately equal to

$$(4\pi \times \text{surface area of cylindrical tip})^{1/2}$$

For $L/D \geq 8$, the cylindrical tip behaves, not unexpectedly, as an infinitely long sand drain and the use of following two equations may then be made for evaluating the values of k and C_v from constant head tests (Al-Dhahir, loc. cit.).

$$k = \frac{Q_{\infty} \gamma_w}{5.336 r (L/D) \Delta u.} \quad (9-8)$$

$$c = \frac{1}{(0.424)^2} \frac{Q_{\infty}^2 r^2}{s^2} \quad (9-9).$$

Where s is given by the slope of against $1/\sqrt{T}$ curve, and where r denotes the radius of piezometer and s denotes the slope of q against $1/\sqrt{T}$ plot.

Wilkinson (1968) modified Gibson's solution for CMT on a spherical tip by introducing variable compressibility and permeability relationships in the theory of consolidation (see also Barden and Berry, 1965; Davis and Raymond, 1965). He derived the equation for a flow through a cylindrical piezometer by solving the finite differences form of Terzaghi's 3-D consolidation equation in radially symmetrical form. For a length L and diameter D , Q_t is given by:

$$Q_t = k a \Delta u / \gamma_w \left[2 \pi (L/D - 0.5) + \frac{4\sqrt{\pi} (L/D - 0.5) + 12.6 + 7.0}{\sqrt{T}} \right] \quad (9-10)$$

for $T < 0.44$ (i.e. $1/\sqrt{T} > 1.5$).

However because of the limitation on value of $\sqrt{T}$, the above solution is of very limited use since it does not completely define the steady state flow for $1/\sqrt{T} < 1.5$.

Hence Dachler's solution (eq. 9-4) may be modified to consider an equivalent spheroid with the same volume as the cylinder. (Schmidt 1966, Wilkinson 1968). This yields an intake factor:

$$F = \frac{3 \pi L}{\log (1.5 L/D + \sqrt{1 + (1.5 L/D)^2})} \quad (9-11)$$

9.1.2 Effect of anisotropy and three dimensional flow

The flow of water in a piezometer would seldom be one dimensional; hence the value of permeability derived from an insitu constant head test gives an estimate of the average permeability of soil. For most piezometer installations in stiff clays (as well as most layered and lacustrine clays), the major part of the flow will take place in the horizontal direction. Yet, for comparison of insitu values of permeability with those obtained from one-dimensional tests in the laboratory, one is likely to be interested in the vertical value of permeability in the field. Hvorslev, (1951) suggests the use of the expression:

$$k_{\text{average}} = \sqrt{k_h \times k_v} \quad (9-12)$$

provided equal weighting is given to flow in the two directions.

The expressions (eq. 9-10 and 9-11) derived by Wilkinson (1968) may be in error if the soil exhibits marked anisotropy. Assuming eq. (9-12) to be valid, and by using the transform for the new vertical direction V_m given by:

$$V_m = V \sqrt{k_h/k_v} \quad (9-13)$$

(equations 9-10 and 9-11) yield:

$$Q_t = k_h a \Delta u / \gamma_w \left[\frac{2 \pi (N - 0.5 \sqrt{k_v/k_h}) + 12.6 \sqrt{k_v/k_h}}{\sqrt{T}} + \frac{4 \sqrt{\pi} (N - 0.5 \sqrt{k_v/k_h} + 7 \sqrt{k_v/k_h})}{\sqrt{T}} \right] \quad (9-14)$$

and

$$\text{intake factor} = \frac{k_h \Delta u / \gamma_w \cdot 2 \pi L}{\log \left(1.5 N \sqrt{k_v/k_h} + \sqrt{1 + (1.5 N k_v/k_h)^2} \right)} \quad (9-15).$$

where $N = L/B$.

A rough estimate of the relative amount of flow in vertical and horizontal direction from or to a piezometer can easily be made using relationships shown in fig. 9.1, after Hvorslev, (1951).

9.1.3 Sealing of piezometers

To obtain any meaningful results from a constant head permeability test in the field, it is essential that good sealing of piezometer be achieved. Vaughan (1965, 1969) derived expressions for the magnitude of errors due to imperfect sealing of piezometers in both a permeable layer and homogeneous material. In general, the piezometer error is reduced if the length of piezometer is increased (hence larger intake factor) and the ratio of grout to soil permeability is decreased. However the permeability of the bore hole seal can be substantially greater than that of the soil before a significant error in the permeability measurement occurs. Vaughan (1969) has shown, for example, that for a grout permeability to be 200 times that of the soil, the average insitu permeability would be overestimated by a factor of about 1.4.

Modern chemical grouts, immiscible with water, can achieve grout permeability of 10^{-10} cm/sec., and thus may be especially suitable for highly impervious soils. However, these do not 'wet' the soil and may allow voids around the wall of the borehole to be filled with water. Chemical grouts are generally more expensive than cement-bentonite

grouts. Cement-bentonite grouts have been used successfully at Balderhead, Peterborough and Selset. (Bishop et. al. 1960, Vaughan, 1965). A 5:4 mixture by weight of air dry bentonite and cement with a water-solids ratio of 2:1 and a permeability of 5×10^{-8} cm/sec. was used at Balderhead. The writer used a 1:1 mixture of air dry bentonite and cement with water solids ratio of 2:1 for the piezometers installed in blue London clay at Wraysbury.

9.2 In-situ constant head permeability tests in London clay

It was decided to conduct some insitu permeability tests to compare values with those obtained from laboratory tests. It was not possible to conduct these tests in the same clay pit from which the laboratory samples were taken because of construction operations. Instead, a relatively level surfaced site adjacent to the reservoir was selected.

9.2.1 Installation of piezometers

It has already been mentioned earlier that blue London clay at Wraysbury is overlain by about 8 - 14 ft. of gravel cover with ground water about 1 - 2 ft. below the surface. It was therefore decided to bore the hole dry and seal a casing through the gravel into the clay layer. The gravel was removed using standard shell and mandrel technique and a steel casing, 18 in. diameter was sunk in. Special care was taken to ensure that the casing was well sealed into the clay. An 8 in. diameter bore hole was then formed using a clay cutter with sharp edges to produce a 'clean' hole.

When the hole had reached 1'-6" above the desired depth, an 18 in. long U - 4 sample was taken. The adjoining clay over this length was then removed using a sharp, clean clay cutter falling with longer drops. A second 18 in. long U - 4 sample was next taken and the bore hole cleaned again as mentioned above. All depths were noted using a length of capped FVC tubing.

About 8 kg of water was carefully dropped at the base of the hole, and coarse, uniformly graded sand was then fed through a tremie pipe dipped in water to provide a base course of 6 in.. Sand was constantly stirred to remove air bubbles. The preassembled Casagrande piezometer tip, 2 in. dia., 10 in. long sealed to 0.5 in. dia. PVC tubing was placed in the centre of the borehole on the sand base. More sand was placed until the sand plug was about 2' - 6" long. The top was then tamped and leveled with the capped stirring rod. Four large balls of dry bentonite in paper bags were weighed with stones and dropped down the borehole. This method was first used by Casagrande, (1949) and provides an efficient seal just above the piezometer since the bentonite tends to swell and pack hard in the borehole on contact with water and also because dry bentonite is more dense than bentonite - cement grout mixture and therefore has a lower permeability.

Air dry bentonite - cement grout mix in 1:1 ratio by weight and with water to solids ratio of 2:1 was prepared in a rotary mixer. The hole was grouted by pumping this mix down a pipe which was always kept below the level

of the grout being placed. The borehole was grouted till the top of clay layer was reached, when the casing was withdrawn and the hole backfilled with gravel. A profile of the four boreholes is shown in fig. 9.3.

9.2.2 Constant head test apparatus (see fig. 9.4)

The apparatus consists of a simple flow measuring unit and a constant head water reservoir. The flow of water to the soil is measured through a 100 cc. glass burette, calibrated to 0.1 cc. and clamped vertically to a wooden board which in turn is screwed to a wooden post near the standpipe. The lower end of the burette is sealed to a simplifix fitting to which a klinger valve A is attached. A short length of $1/4$ in. diameter nylon tubing connects valve A to 3-way brass connector to which klinger valves B and C are attached as shown in the figure. A length of $3/8$ in. dia. nylon tubing runs to the top of standpipe from valve B while another length of tubing connects valve C to the constant head water tank. A length of tubing runs from a small coloured paraffin reservoir to the burette passing through an undersized hole in the rubber bung which fits tightly in the upper end of the burette.

The constant pressure is supplied by water in a 50 gallon drum resting at some height above the burette. (A conveniently placed small 'spoil' embankment proved ideal for this purpose.) The level of the paraffin reservoir is so adjusted as to give the same head as that provided by the constant head water tank. In this series of tests, it

was decided to use a separate measuring system for each bore hole, but the water and paraffin heads were maintained by the same reservoirs. It is worth emphasising that the apparatus for permeability tests in relatively impervious soils (like the London clay) should be very carefully checked for leaks under pressure in the laboratory, prior to installation in the field.

9.2.3 Test procedure

The test is proceeded with only * when water level, in the standpipes has reached a steady level, which is accurately determined with a dipper.

Next, the burette and the water leads are completely filled with water and all air bubbles are flushed out. Some paraffin is introduced in the burette by closing valve C and opening valves A and B to atmosphere. The measuring system is now ready for use. The standpipe is now completely filled with water and allowed to stand for a minute to allow any air bubbles to come out. A simplifix fitting is then quickly fitted to the top of the standpipe and the nylon tubing from valve B freely running with water is fixed to this fitting. (During this stage, valves B and C are kept open and valve A closed).

The test is commenced by immediately noting time t_0 ; closing valve C and opening valve A so that flow measurements may now be taken on the paraffin/water interface.

* Ground Water level in the deeper bore hole (4) had not reached a steady level when the test was commenced, and it was assumed that it would rise to a similar level as in other three boreholes.

When paraffin has reached near the end of the burette, nylon tubing running from constant head water tank to valve C is disconnected at C and instead a water filled hand ram is connected. Valve B is now closed and the paraffin level is pumped up the burette by using the hand ram keeping valves A and C open. After the paraffin level has been raised to a convenient height, valves A and B are opened: valve C closed: the hand ram disconnected and the tubing from the tank reconnected at C. The whole operation need not take more than two minutes. After every raising of paraffin level, it is advisable to wait for a short time before measuring flow rate as the temporary closure of valve B during pumping operation would disturb the flow conditions.

If the flow rates are large compared with the capacity of burette, but insignificant in relation to water in constant head reservoir, the apparatus may be left unattended by keeping valves B and C open and A closed.

A graph of rate of flow (cc/min.) against $1/\sqrt{t}$ is plotted as readings are taken.

Vaughan (1965) plotted flow rate against $\frac{1}{\sqrt{t_1 + \Delta t/2}}$

where t_1 is the time at the start of the increment and Δt is the time increment over which flow is measured. However, Al-Dhahir (1968) has shown that a more correct plot would be flow rate against $\frac{2}{\sqrt{t_1} + \sqrt{t_2}}$ where t_1 and t_2 are times before and after the increment.

9.2.4 Results and Discussion

The main results from the four boreholes have been summarised in the table below.

B/H. No.	1	2	3	4
Depth below ground surface	24'-2" to 26'-8"	31'-5" to 34'-3"	16'-6" to 19'-0"	40'-3" to 42'-11"
Depth below gravel	8'-2" to 10'-8"	20'-0" to 22'-10"	7'-6" to 10'-0"	28'-9" to 31'-5"
Dimensions of sand plug	2'-6"x8"Dia.	2'-10"x8"	2'-6"x8"	2'-8"x8"
Effective vertical stress, p_o psi.	10.6	14.2	8.1	18.0
Stress increment, Δu psi.	5.10	5.11	5.47	5.25
$k \times 10^{-9}$ cm/sec.	34.0	10.0	20.2	2.0

The intake factors were derived on the basis of using the equivalent intake factor of cylindrical tip as outlined by eq. (9-7). The less accurate, but often used Dachler's formula would yield an intake factor lower approximately lower by 7% (about 7.5 in. less).

The above values of permeability are of course average values which take the horizontal permeability of the clay into account as well. As mentioned earlier, conventional oedometer tests were done on both vertical and horizontal samples from the four boreholes at the same

depth at which the piezometers were installed. This was done in order to obtain some indication of the anisotropy in permeability and the results showed that the ratio of horizontal to vertical permeability, k_h/k_v , determined from conventional oedometers and Terzaghi theory, was 1 for boreholes (1) and (2), and 2 for the deeper boreholes (2) and (4), for the stress range considered. Hence by applying the relationship:

$$k_{av} = k_h \cdot k_v \quad (\text{Hvorslev 1951}),$$

the following very approximate values for vertical permeability are obtained.

B/H No.	1	2	3	4
$k_v \times 10^{-9}$				
cm/sec.	34.0	7.1	20.2	1.4

To be strictly correct, the insitu permeabilities should be corrected for temperature effect in the clay since the tests were conducted at a temperature lower than the standard 20.2°C . The correction factor may be determined on the basis of relationship between temperature and coefficient of viscosity for water (the coefficient of viscosity for water at 20.2°C (63.36°F) is one centipoise).

$$\text{Then } k_o = k_t \mu_t / \mu_o,$$

where k_o and k_t denote coefficient of permeability at temperatures of 20.2°C and $t^\circ \text{C}$ respectively,

and μ_o and μ_t denote coefficient of viscosity of water at temperatures of 20.2°C and $t^\circ \text{C}$ respectively.

Fig. 9.5 shows a plot of correction factor μ_t/μ_o against temperature in degrees centigrade. Clearly this temperature effect can be large if the test is conducted in a soil at a temperature substantially lower or above the standard 20.2°C .

Assuming that the soil was at an insitu temperature of 10°C , the correction factor is found to be 1.31. Hence all results should be increased by this factor.

The results, plotted in fig. 9.6 clearly show the effect of ~~fissures~~^{depth} on permeability of blue London clay. The permeability decreases remarkably as the more 'intact', deeper stratum is approached. It is interesting to compare this work with that described by Skempton, Schuster and Petley. (1969) on fissure studies at Wraysbury, which shows that both the size and intensity of fissuring decreases rapidly as the depth increases.

In this context, it is worth noting that although the dimensions of the sand plugs (at least 2'-6" long and 3 in. in diameter) are sufficiently large to take the flow through most fissures into account, it may not be large enough to assess the influence of joints and fissures at large spacing. It is quite possible that fissure spacing may exceed the length of sand plug specially at great depths, in which case the permeability values would be underestimated. It would, therefore, be of interest to conduct similar tests at the same site using much longer sand plugs.

The permeability for borehole (1) appears to be slightly higher in relation with depth when compared with the other results. This is not surprising in view of the intense fissuring observed at this borehole when the first U-4 sample was taken.

The graphs of flow rate against $1/\sqrt{t}$ showed considerable departure from the theoretical Gibson plot, especially at earlier stages of the test. This may be due to a number of factors which are now discussed.

One of the main limitations of the insitu test is the disturbance of soil around the borehole. The process of making the borehole, especially with the shell and mandrel system can seriously alter the soil structure around the periphery of the borehole. This disturbed zone may extend to a maximum of 3 times the diameter of the borehole (Gibson 1966) and it is in this region that the permeability characteristics are investigated. This is specially serious in soils which show pronounced horizontal laminations (Rowe, 1964, 1969) and for such soils a screw auger method is generally to be preferred, (see, for example, Landau, 1967; Rowe, 1968). For soils with a wide zone of smear, the permeability of the region adjacent to the piezometer would generally be reduced. Hence the permeability may be greatly underestimated if flow measurements are not taken over long period of time. This effect can be extremely serious for structured soils. However, the effect is likely to be small at large time for London clay, especially for the deeper boreholes (nos. 2 and 4) where the soil does not appear to show much

anisotropy in permeability. On the basis of permeability of remoulded London clay to be 1×10^{-9} cm/sec. * and using Gibson's solution, the insitu permeability would perhaps be overestimated by 1-3%. There would of course be more appreciable departures at small times.

The effect of grout on constant head test results (Vaughan 1969) has been mentioned previously. Due to strict control and supervision that was exercised in the preparation and pumping of grout mix, it is unlikely that any serious errors would result because of this factor. Assuming for the most pessimistic case that the permeability of grout was 100 times that of clay, the insitu permeability would be overestimated by about 10% maximum.

The effect of k and c_v varying with effective stress has been investigated by Al-Dhahir (1967) who derived modified constant head test equations based on nonlinear theory by Davis and Raymond (1965) and that proposed by Barden and Berry, (1965). The results show that if Davis and Raymond theory is held to be valid then

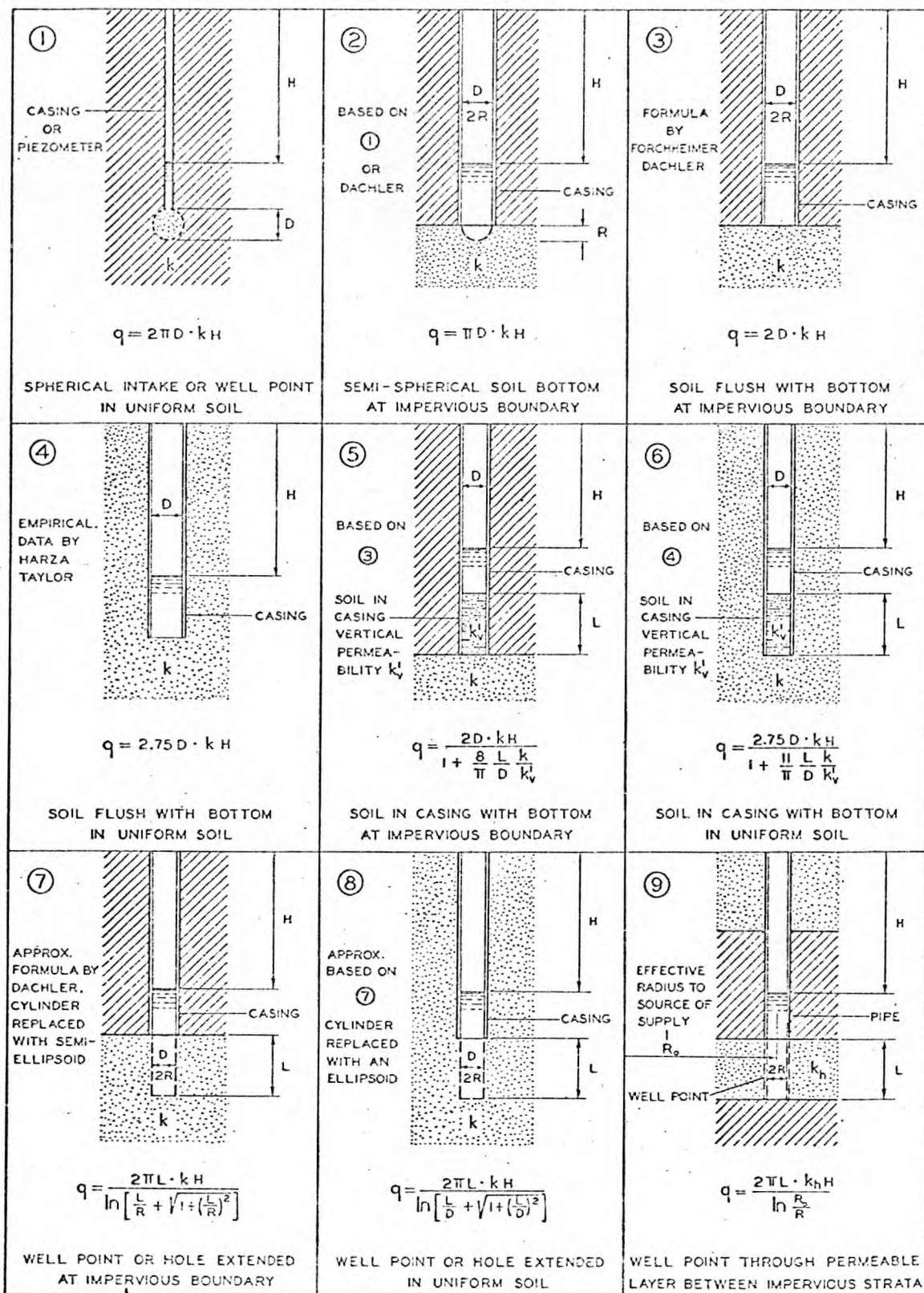
$$k_{D-R} = k_{\text{Gibson}} \times \frac{1}{(1 - \sigma_1^i / \Delta u) \ln (1 - \Delta u / \sigma_1^i)}$$

where σ_1^i denotes the initial effective stress in soil, and Δu denotes the effective pressure increment. Al-Dhahir's plot shows that in a consolidation test, as Δu increases,

* The average value of 1×10^{-8} cm/sec. for permeability of remoulded London clay mentioned by Tan (1968) was obtained on normally consolidate remoulded samples at twice the void ratio of the undisturbed samples used in the present test.

lower flow rate than that predicted by Gibson (on standard Terzaghi theory) would be obtained: while in a swelling test, the flow rate would be overestimated. The non-linear behaviour can also cause considerable curvature in the initial stages of flow rate against $1/\sqrt{t}$ plot.

The results clearly appear to establish that insitu permeability of fissured London clay is dependant on intensity of fissuring and generally decreases with depth. A comparison of insitu permeability with laboratory results on undisturbed samples will be made in chapter 10.



q = RATE OF FLOW IN CM³/SEC, H = HEAD IN CM, k = COEF. OF PERMEABILITY IN CM/SEC, $\ln = \log_e$, DIMENSIONS IN CM.

CASES 1 TO 8: UNIFORM PERMEABILITY AND INFINITE DEPTH OF PERVIOUS STRATUM ASSUMED

FORMULAS FOR ANISOTROPIC PERMEABILITY GIVEN IN TEXT

Fig. 9.1, Inflow and shape factors

(HVORSLEV, 1951)

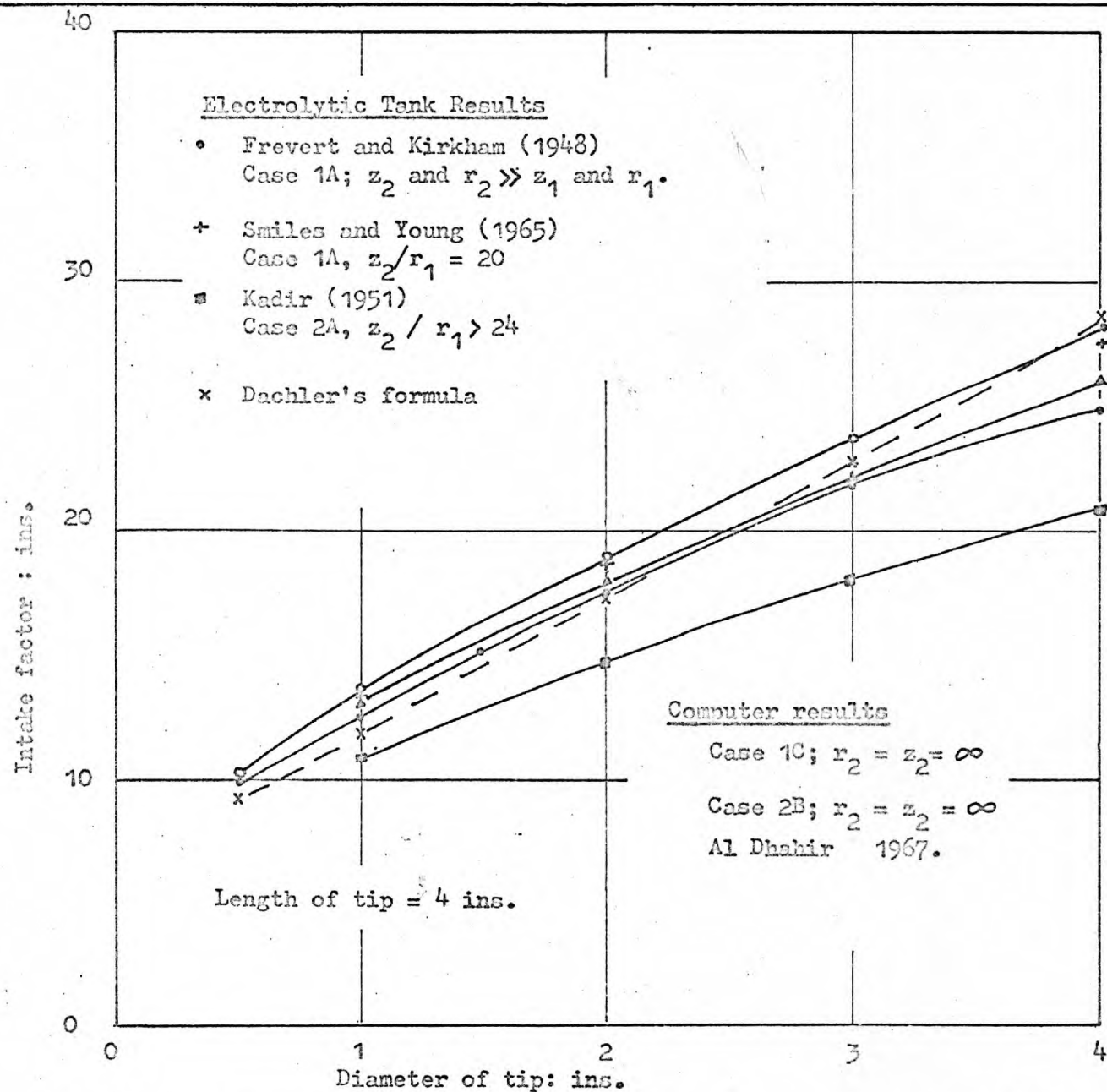
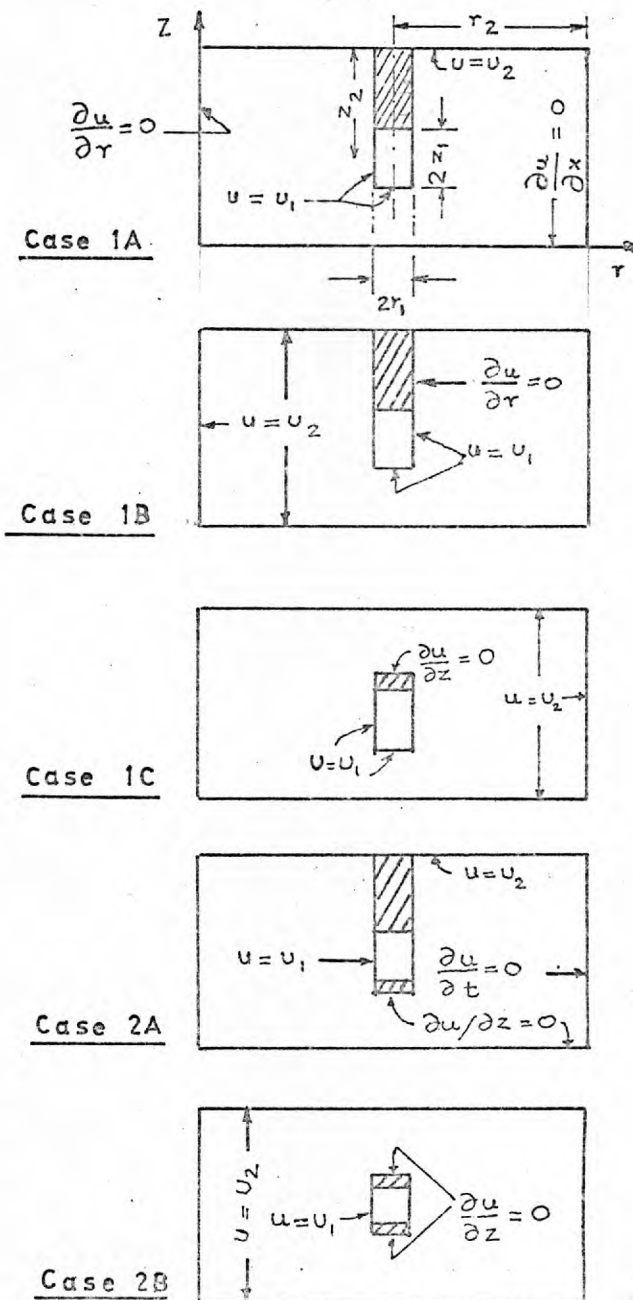


FIG. 9.2 INTAKE FACTORS OF CYLINDRICAL TIPS (after Al Dhashir 1967)



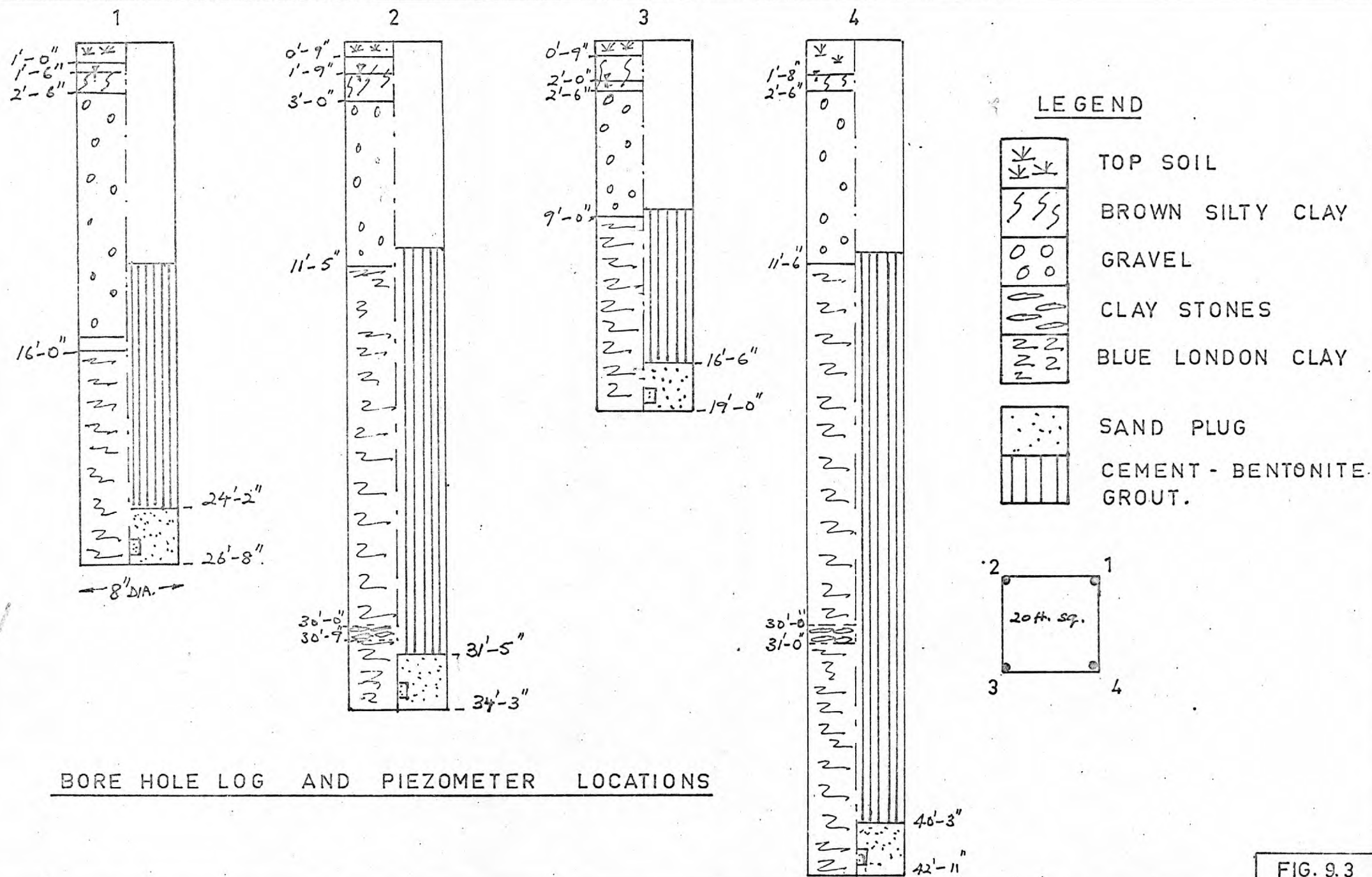
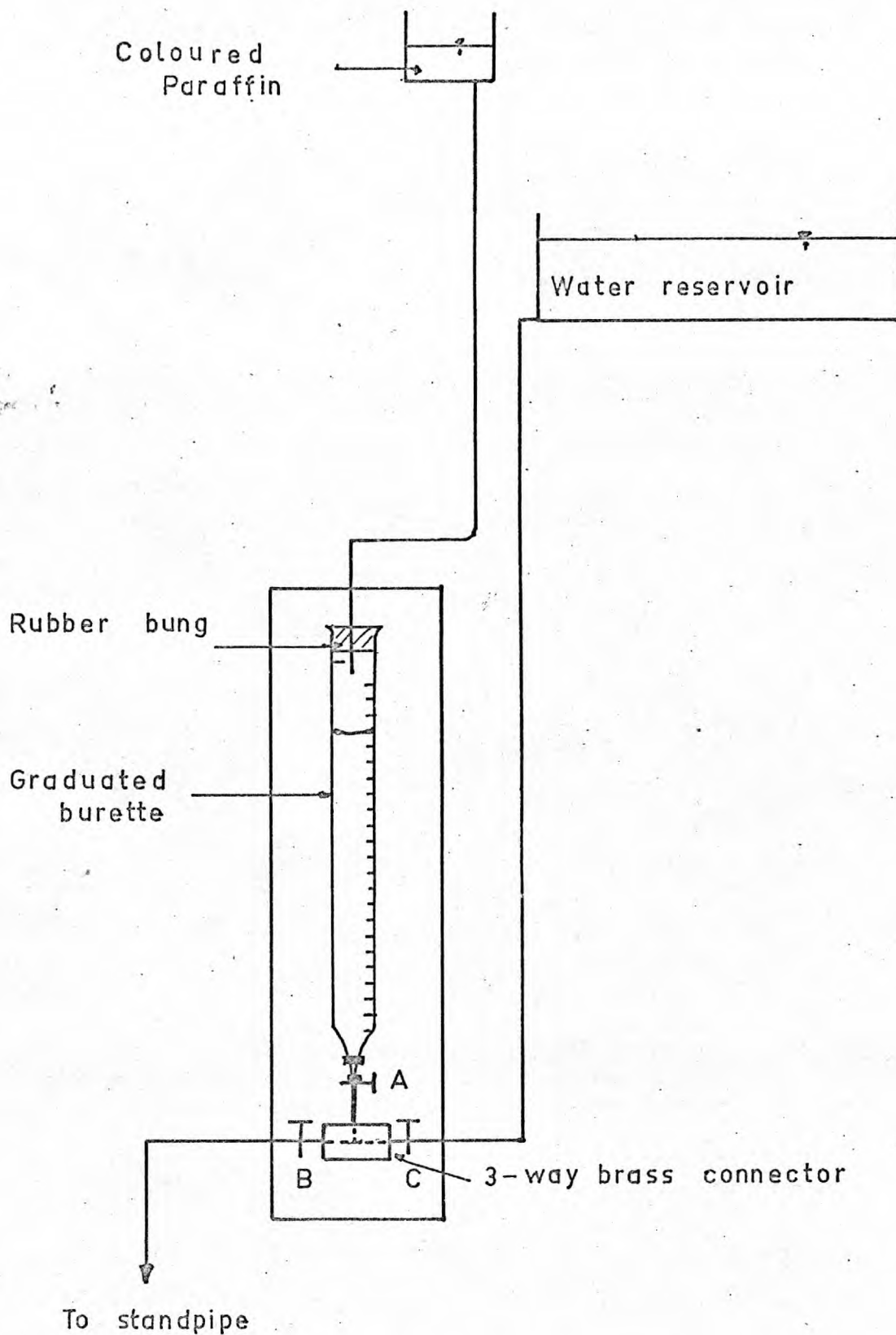


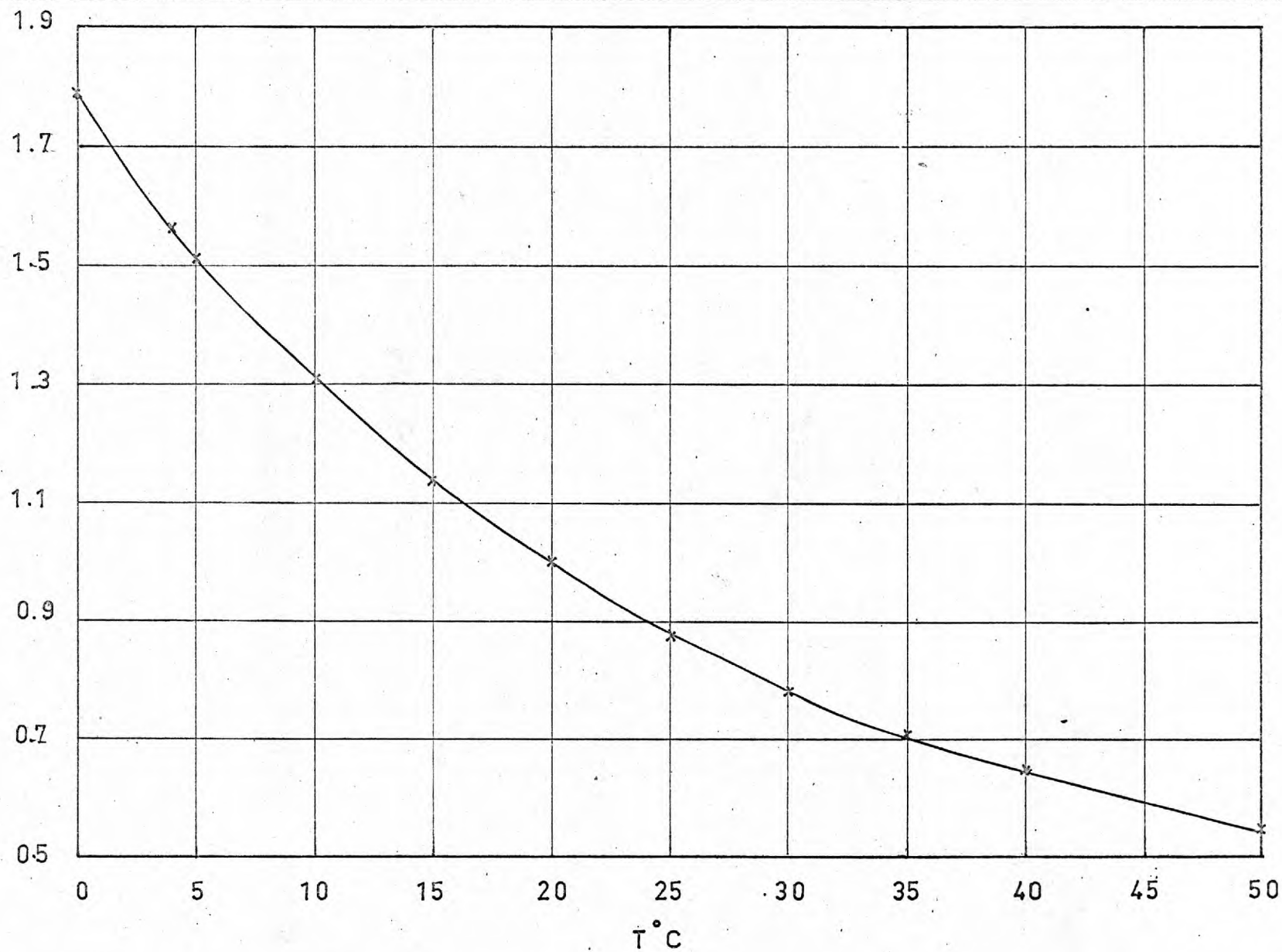
FIG. 9.3



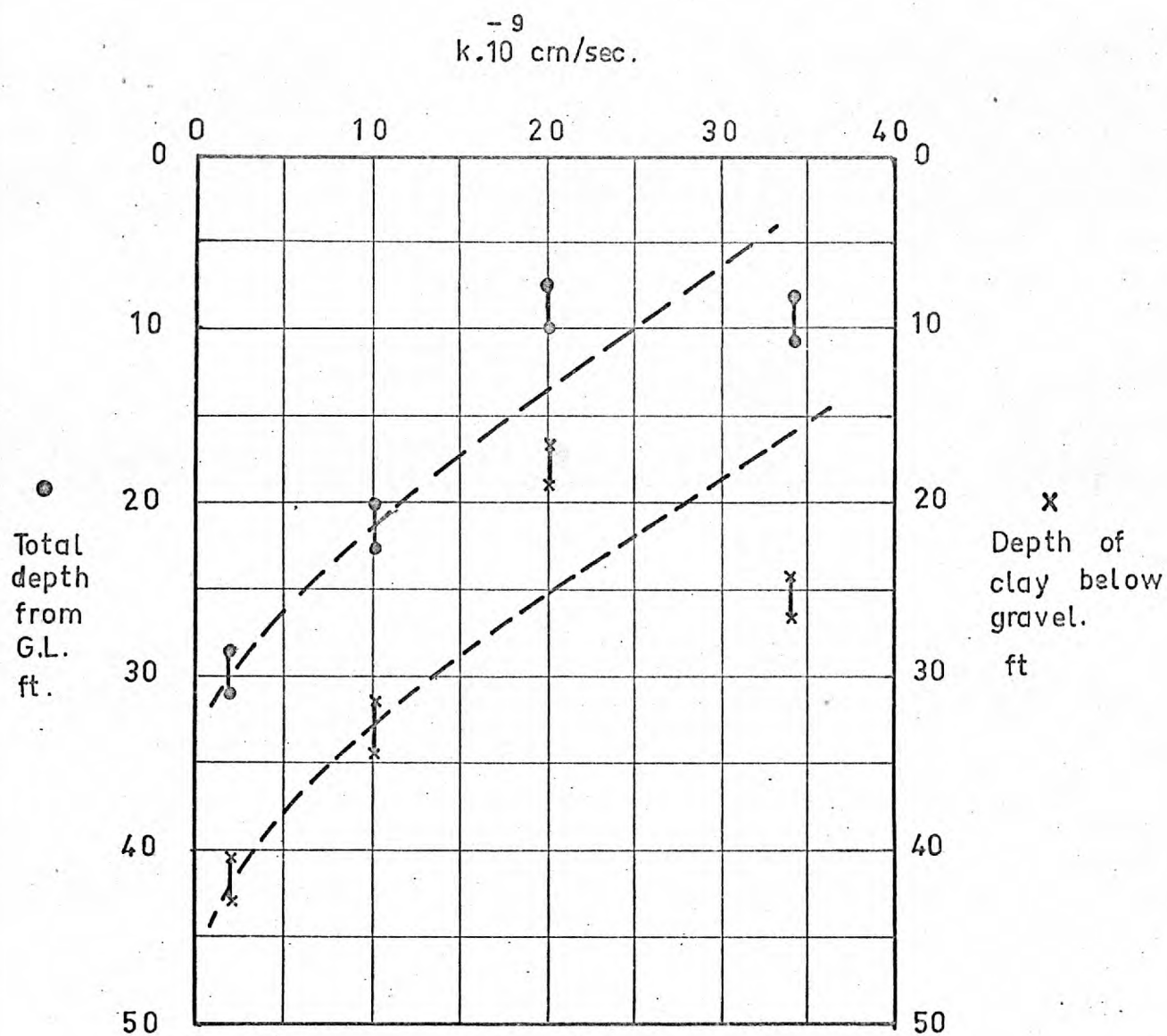
LAYOUT OF CONSTANT HEAD APPARATUS

TEMP
CORRECTION
FACTOR,

$$\frac{U_T}{U_0}$$



GRAPH FOR TEMPERATURE CORRECTION FACTOR.



Variation of insitu permeability
with depth.

FIG. 9.6

CHAPTER 10

Laboratory tests on London clay

10.1 Details of experimental work

Samples of blue London clay from Wraysbury were tested in the laboratory in 1.5 in., 4 in. and 12 in. diameter dissipation cells. The clay samples were obtained from the clay pits at a depth of about 10 ft. below the gravel. 3 in. diameter oedometer tests on horizontal and vertical samples at low pressures were also carried out.

10.1.1 1½ in. and 4 in. diameter tests

The dissipation test apparatus for 1.5 in. and 4 in. diameter samples has been described by Bishop and Henkel, (1962). The cylindrical sample with height to diameter ratio of 1 rests over a saturated, coarse porous stone on the pedestal of a triaxial cell. A perspex top cap carrying a drainage lead rests on another saturated coarse porous stone placed over the top of the sample. Moistened fine filter paper discs are placed between stones and soil at either end of the sample to avoid clogging of filter stones by the more impermeable clay. This effect, if allowed to develop, can seriously impair drainage from the sample, (Newland and Alley, 1960; Bishop and Gibson, 1963). The sample is enclosed in a rubber membrane sealed with rubber O-ring in the usual manner.

The pore pressure at the base of the sample is measured by mercury null indicator, while the outflow of

water from the clay is measured by means of paraffin/water volume gauge, through which the back pressure is also applied. In most of these tests, the axial deformation of the sample was also monitored by means of an ordinary dial gauge mounted on a yoke fixed to the base of the cell. The spindle of the dial gauge rests on top of the perspex top cap. Lovenbury (1969) found that the ordinary dial gauges worked satisfactorily under liquid pressure provided that the gauge mechanism is open to the cell pressure and that it is placed in a thin oil (Shell Diala 'E' oil was found suitable) and not water. Hence the cell was only partially filled with water up to the top cap level and the rest was filled with oil. In tests where the pressure increment was below 15 psi., the pore pressures were measured more accurately by connecting the null indicator in series with a mercury manometer, the other end of which is connected to a pressure manifold.

10.1.2 Test procedure

The pore pressure was first allowed to build up under a constant cell pressure. After enough time (usually three days) had elapsed to achieve this, isotropic consolidation was commenced by applying a constant back pressure through the upper drainage lead. Readings of pore pressure, volume change and change in height of the sample were recorded at periodic intervals.

After complete dissipation of pore pressure and volume change had been achieved, the next stage could be started by simply reducing the back pressure to another

constant value, while maintaining the same cell pressure.

10.1.3 Description of 12 in. diameter dissipation cell

This cell made of electro-plated mild steel consists essentially of five parts, namely:

- I) The bottom plate,
- II) the lower pedestal,
- III) the hollow cylinder,
- IV) the top plate,
- V) the top cap on the sample.

The bottom plate, (fig. 10.1) is 21 in. in diameter and 4 in. high. Twelve holes to carry tie-rods are spaced at intervals of 30° and drilled and tapped to 1.25 in. B.S.F. size on a P.C.D. of 18.5 in. A central hole, 1.5 in. diameter is also drilled through which a transducer can be screwed in the base of the lower pedestal for measuring pore pressures. Six service ports from the side of the plate to the inside are also provided. Standard 1/4 in. Simplifix fittings are fitted to the inner ends of these service ports while klinger valves may be fitted to the outside. An O-ring on the side of the plate prevents any leakage when the hollow cylinder forming the side walls of the cell is positioned on the bottom plate. The top surface of this plate is very smoothly machined.

The lower pedestal (fig. 10.2), is a thick disc, 12 in. diameter and 2 in. high. The lower surface is also very smoothly machined so that it may rest flush on the upper surface of the bottom plate. A two inches high insert,

carrying the transducer, is screwed on the underside of the pedestal and it fits tightly in the central hole of the bottom plate; leakage being prevented by an O-ring seal. A groove, $1/8$ in. wide and $1/16$ in. deep, runs across the diameter to two service ports to which Simplifix fittings are attached. Nylon tubings extend from these fittings to the corresponding fittings on the bottom plate. In this way, deaired water can be made to flow along the groove to flush out any air bubbles that may be present. Three holes are provided at the top of the pedestal to which a 12 in. filter stone may be screwed. However, in the present series of tests, no filter stone was used on the pedestal. Instead, the groove was filled with washed sand; the sand being prevented from falling down the service ports and the transducer by glass wool plugs. After all air had been flushed out, two 12 in. diameter moist filter papers were placed on the pedestal and the top surface was continuously kept moist until the sample was placed on it.

The top cap, is similar to the bottom pedestal, except that it is made as light as possible by removing the metal in the annular section as shown in fig. 10.3. A 12 in. diameter coarse filter stone can be screwed to the underside of this platten. A groove, $1/8$ in. wide and $1/16$ in. deep is provided to flush and deair the top filter stone. Two water ports on either side of the groove act as drainage channels.

The hollow cylinder, 24 in. long and $1\frac{1}{4}$ in. thick, forms the sidewalls of the cell and fits tightly over the bottom plate. O-ring seals on upper and bottom plates prevent any leakage from the cell.

The top plate (fig. 10.4) is 21 in. diameter and 4 in. high. Twelve holes, 1.25 in. diameter are drilled on a P.C.D. of 18.5 in. The tie rods are passed through the top plate and screwed into the bottom plate. The top plate rests on the hollow cylinder and the tie rods are firmly fastened by nuts on the top plate. A $1\frac{1}{4}$ in. diameter vent allows air to escape as the cell is filled with water. This can later be sealed off by screwing a bolt on a Dowty seal.

The pore pressure at the base is measured by a Bell and Howell transducer (type 4-326-L100); the readings being monitored on Budd P 359 strain gauge meter. The transducer showed extremely good linearity when calibrated against a Budenberg dead weight calibrator. By selecting a suitable gauge factor on the Budd strain gauge meter, the pore pressure reading could be read directly in psi. units. The flow of pore water under consolidation is measured on a 100 cc. paraffin/water reversible volume gauge (Bishop and Henkel, 1962).

Installation of sample in the cell

The undisturbed blue London clay for these large diameter tests was sampled in the field using a 12 in. diameter cylindrical clay cutter which had earlier been used successfully by Agarwal, (1967).

The sample was carefully removed from the rubber membrane in which it was stored and any cavities on the side were filled with remoulded clay. The sample was left standing upright; a 12 in. x 12 in. plyboard mould, which could be separated in two parts was fitted tightly around it; and the ends were trimmed with a straight edge. The mould (with the sample) was hoisted with the lifting tackle and placed on the lower pedestal of the cell which had carefully been prepared as described earlier. (plate 10-1)

A 1/16 in. rubber membrane was then carefully slid over the sample and the upper saturated filter stone and top cap were placed on the sample. The membrane was sealed by 'Hi-Torque' screw clips running over PTFE tape wound over the membrane around the pedestal and the top platten. (plate 10-2)

The cell is next quickly assembled, filled with water and maintained under a constant pressure. At this stage, any air in the top filter stone and the pedestal can be flushed out by circulating water through the service ports, under a back pressure equal to the value given by the transducer.

The pore pressure under undrained conditions is allowed to build up and when uniform pore pressure conditions have been achieved throughout the sample, (as shown by the bottom transducer and an upper transducer in the drainage line outside the cell), consolidation is commenced under the desired constant back pressure.

10.1.4 Oedometer tests

The samples for the oedometer tests were prepared from the U - 4 samples obtained from the same depths at which the piezometers had been installed at Wraysbury. Both, vertical and horizontal samples were tested in the usual manner under low effective stresses (up to 2.0 T/ft^2), corresponding to field overburden pressures.

10.2 Results and Discussion

10.2.1 Dissipation tests

The results from the dissipation tests have been summarised in Table 10.1, where typical index properties of the clay are also given. In each case, the values of C_v in terms of maximum pore pressure dissipation ($C_{v_{mx}}$) and average volume change, ($C_{v_{av}}$) have been calculated at their respective times for 50 and 90% dissipation. In most cases, except the 12 in. diameter dissipation cell, the change in height was also measured by an internally mounted dial gauge. Graphs for the degree of consolidation (with respect to both volume change and pore pressure dissipation) against log time, and the average volume change against log time have also been presented at the end of this chapter. (Note that in some cases it has not been possible to show both the average and maximum degree of consolidation on the same graph, in which case they have been plotted separately.)

It might be useful to mention some test details about these tests. Test UDLC-12-~~A~~ was accidentally wetted prior to consolidation, when the top porous stone was flushed before the cell pressure had been applied. The sample was reinstalled the following day - but during this operation it is likely that sample had undergone swelling. Also, the test was terminated because of a leak in the membrane when the average degree of dissipation was about 94%.

It is also worth pointing out that the 4 in. and $1\frac{1}{2}$ in. diameter samples tended to be composed of rather intact clay with few fissures. It was extremely difficult to trim highly fissured samples for mounting in the smaller dissipation cells and many samples were lost during this process. Samples UDLC- $1\frac{1}{2}$ -D and E were however highly fissured samples while samples UDLC- $1\frac{1}{2}$ -I were relatively intact clay.

In some of the $1\frac{1}{2}$ in. dissipation tests, very minute leakage, either in the volume gauge itself or the klinger valves, resulted in erroneous volume change readings. Such small volume changes should have been measured on a calibrated saran tubing. (Normand, 1964; Som 1968).

Flexibility of the pore water measuring system has important effects on the development of pore pressure under undrained conditions. Because no pore pressure measuring system is absolutely rigid, there is bound to be some 'drainage' from the sample into the measuring system due to difference in pressures in the two at the moment the load is applied. If the measuring system has high flexibility, then the time lag may be large because of delay in achieving equilibrium between the two systems. These effects have been discussed by Whitman, Richardson and Healey (1961) Bishop and Henkel (1962), Gibson (1963), Tan (1968) and others. In the present series of tests, the pore pressure in $1\frac{1}{2}$ in. and 4 in. diameter samples was measured by Bishop type mercury null indicator fixed directly to the base of the triaxial cell. Its flexibility (1×10^{-6} in.³/psi.) would introduce

negligible error provided sufficient time is allowed to achieve equilibrium before consolidation stage is commenced (Gibson 1963). A time of at least 4 days was allowed in the present tests. The pore pressure in 12 in. diameter samples was measured by a Bell and Howell transducer screwed in the base of the cell. Its flexibility was found to be 0.4×10^{-6} in.³/psi. and at least 14 days were allowed for pore pressure to achieve equilibrium between the ends of the sample and the measuring system under undrained conditions.

10.2.2 Oedometer tests

The results of the oedometer tests have been summarised in Table 10.2. The samples for these tests were obtained from the same depths at which piezometers had been installed at the Wraysbury site. The tests were conducted in the low pressure range only. The purpose of these tests was to obtain values of C_v , m_v and k as would be obtained in a commercial laboratory. Also, these tests provided some indication of the anisotropy of clay in vertical and horizontal directions.

The zero readings for calculation of average degree of consolidation were obtained by applying Casagrande construction, (Taylor 1948), by plotting the change in height readings on log time plot. Agarwal (1967) found that Taylor $\sqrt{t}$ plot could not be very satisfactorily applied to fissured London clay below effective stress of $4T/\text{ft}^2$. Of course, the correction from either of the methods would not give the true zero as the oedometer

assembly itself shows elastic and inelastic deformation on the application of a load on the hanger. The magnitude of deflection of the oedometer assembly had earlier been investigated by Som (1968) by using a dummy steel ring in place of the sample. For pressures used in the present work (up to 2.0 T/ft^2), the deformation of the oedometer frame is of the order of 15×10^{-4} in. only. This has been neglected in the present calculations. This correction has to be taken into account if higher pressures are used.

10.2.3 Coefficient of consolidation, C_v

The experimental curves for the degree of dissipation against log time have been compared with the theoretical Terzaghi curve fitted at $U=50\%$. It will be seen from the shape of the two curves that the Terzaghi theory predicts the average rate of consolidation reasonably accurately up to about 70% average dissipation but there is little correspondence between the experimental dissipation curve and the theoretical Terzaghi curve. In every case the experimental rate of pore pressure dissipation is faster than that predicted by Terzaghi one dimensional consolidation theory. Similar conclusions were obtained by Som (1968). It is perhaps not surprising that in some cases the average rate of settlement becomes slow after $U \approx 70\%$ because of secondary settlement and viscous effects. The discrepancy between the rate of pore pressure dissipation and the Terzaghi theory, is evident for samples of all sizes and reflects highly non-linear consolidation behaviour. It is known that Terzaghi theory is based on the assumptions that during consolidation, the coefficient of consolidation, C_v , coefficient of compressibility, m_v ,

and permeability k remain constant. However none of these assumptions is valid for London clay. Recently a number of attempts have been made to analyse the process of consolidation by considering some of the variations in C_v , m_v and k . (Schiffman, 1958; Davis and Raymond, 1965; Barden and Berry, 1965; Gibson, Hussey and England, 1967 and Poskitt, 1969). Davis and Raymond (1965) assumed variation in compressibility m_v according $e - \log p$ relationship and that k and m_v decrease in such a way that their ratio, C_v , remains constant. They found that the rate of pore pressure dissipation was a function of the pressure increment ratio $\Delta p/p'_0$, (the results fit better with Terzaghi theory when $\Delta p/p'_0 = 1$); while the average rate of volume change remains unaffected. The Davis and Raymond theory was applied to results from tests UDLC-12A and UDLC-12-B-1 where the pressure increment ratio was greater than one. It was found that this theory made virtually no difference to the shape of the pore pressure dissipation curves. The value of C_v increased from 10.4 ft²/yr to 14.6 ft²/yr for test UDLC-12-A and from 5.83 ft²/yr to 6.5 ft²/yr for test UDLC-12-B. This shows that the assumptions in Davis and Raymond theory are not true for London clay. Very recently Poskitt (1969) has formulated a theory of non-linear consolidation which incorporates the results of Barden and Berry, (1965); Davis and Raymond, (1965) and Gibson, England and Hussey, (1968). An attempt was made to analyse experimental results using Poskitt's theory, by using the solutions in graphical form given in his paper. It was found that the behaviour was so non-linear that it was not dealt by this theory.

The values of C_v based on dissipation of pore pressure have been plotted in fig. 10.31, for all the dissipation tests. It will be observed that C_v from 12 in. diameter samples are approximately $1\frac{1}{2}$ times the values of C_v from 4 in. diameter samples. No distinct variation can be observed from $1\frac{1}{2}$ in. diameter samples, when compared to 4 in. diameter samples. However it should be noted that the effect of fissures would perhaps be over emphasised in the $1\frac{1}{2}$ in. diameter samples and would tend to give a higher value of C_v . The values of C_v shown in fig. 10.31; from the oedometer samples are, of course, average C_v values based on change in height of the sample. It will also be seen that at an average effective stress of 25 psi. the values of C_v from oedometer samples from bore holes (1) and (3) which correspond more to the depth from which dissipation samples were taken would underestimate C_v by at least 4 times*. Figure 10.31 also shows that C_v decreases with effective stress for all samples, though much larger scatter has been found by other workers (Ward et. al. 1959, Agarwal, 1969). Both the variation of C_v with sample size and the decrease of C_v with effective stress could be a function of fissure density and fissure openings. A sample with larger diameter would have more fissures which would accelerate the rate of dissipation of pore pressure than a sample with a smaller diameter, especially if the fissures are randomly orientated as appears to be the case with London clay at Wraysbury (Skempton et. al., 1969). That the fissures are open, especially at low effective

* These tests also show that C_v for horizontal samples is about 2.5-4 times that for vertical samples.

stress, is evident on examination of clay obtained from freshly opened pit, when often free moisture can be observed along fissures. This strongly suggests that fissures act as channels along which pore water can travel.

It has now been established by several workers that the C_v obtained from max. pore pressure dissipation may be quite different from that obtained from change in volume of the sample, and in fact, the two values appear to be a function of the pressure increment ratio, $\Delta p/p'_0$. As this ratio decreases, the ratio of rate of dissipation of pore pressure to the rate of volume change increases. Studies by Nowland and Alley (1960) and Wahls (1962) showed that secondary settlements increased (i.e. C_v based on volume change decreases) remarkably as $\Delta p/p'_0$ decreased. However, no pore pressures were measured in their work. Leonard and Girault (1961) studied the effect of pressure increment ratio as well as the nature of pore fluid and the effect of side friction on undisturbed sensitive Mexico City clay in specially designed oedometers in which the pore pressure could also be directly measured. Their results clearly showed that the ratio of $C_{v_{mx}}/C_{v_{av}}$ increased as $\Delta p/p'_0$ decreased. The results of tests done on remoulded clay in hydraulic oedometer at Imperial College (Tan, 1968) also confirm this observation.

The variation of $C_{v_{mx}}/C_{v_{av}}$ on $1/(\Delta p/p'_0)$ for all the dissipation tests done in the present work has been shown in fig. 10.32. The trend is quite remarkable despite large differences in sample size and size of fissures. It appears from results shown in Table 10.1 and also from

tests on blue London clay by Skinner, (1967) shown in Table 10.3 that $C_{v_{av}}$ changes more rapidly than $C_{v_{mx}}$ as $\Delta p/p'_0$ is altered.

Table 10.3 (after Bishop and Al-Dhahir, 1969)

Soil	p'_0 psi	$\frac{\Delta p}{-p'_0}$	$C_v/\text{ft}^2/\text{yr.}$		Ratio $\frac{(2)}{(1)}$	Reference
			(1)	(2)		
			from	from		
			Δu	Δv		
Undisturbed	2.8	2.0	44.0	43.0	0.97	Leonard and Girault (1961)
Mexico City	15.6	3.0	3.2	5.0	1.57	
clay	74.4	0.15	3.1	0.44	0.014	
Blue London	27.8	0.8	10.7	7.23	0.68	Skinner (1967)
clay with	50.0	0.6	10.6	5.62	0.53	
silt intrus- ions	80.0	0.6	10.0	4.52	0.45	
M6	6.0	$\rightarrow \infty$	14.6	19.1	1.31	Author's tests.
Boulder clay	6.0	0.67	29.6	33.7	1.14	
fill remould.	2.0	0.33	42.3	31.8	0.89	

This effect has important implications on the behaviour of London clay in the field. It has been observed in a number of cases that field rates of settlement are many times faster than that predicted from tests on conventional oedometers. (MacDonald and Skempton, 1955, , Skempton, Peck and MacDonald, 1955; Som, 1968). The laboratory tests are usually

carried out under a pressure increment ratio of one while the pressure increment ratio in the field would very often be less than one and progressively decrease with depth. Also the conventional consolidation test in the laboratory is conducted in small oedometers, which do not often adequately represent the fissured mass in the field. The laboratory results can thus underestimate the rate of consolidation in the field. Three dimensional drainage in the field is another important factor and will be discussed later.

The values of C_v of London clay obtained from oedometer tests have been found to be susceptible to sample disturbance and initial swelling, especially at low pressures, (Cooling and Skempton, 1942; Som, 1968). In the present tests, the average value of C_v so obtained decreases from about $8.2 \text{ ft}^2/\text{yr}$ at effective stress of 5 psi. to about $3.2 \text{ ft}^2/\text{yr}$ at average effective stress of 25 psi. These values may be compared with $3.1 \text{ ft}^2/\text{yr}$ (for stress range 1 - 9 T/ft²) obtained by Cooling and Skempton, (1942). Skempton and Henkel (1957) mention values of C_v obtained from oedometer tests on London clay which have been summarised in Table 10.4.

Table 10.4 (after Skempton and Henkel, 1957)

Site	Range of depth below gravel	Range of C_v ft^2/yr
Paddington	4'-6" - 138'-6"	2.7 - 5.8
Victoria	0'-6" - 32'-0"	2.6 - 16.0 (Max. at depth of 32'-0")
South Bank	3'-0" - 102'-0"	2.6 - 17.8 (Max. at depth of 39'-0")

The fact that the max. value of C_v for Victoria and South Bank sites was obtained at depths of 32'-0" and 39'-0" could suggest the presence of a silt band or other coarse material.

In conclusion, fig. 10.31, clearly indicates that higher values of C_v are obtained as the size of the sample increases. Hence the larger diameter dissipation tests are to be preferred to assessing the value of C_v in the laboratory. It should be pointed out that in dissipation tests some shear forces develop at the top and bottom of the sample, which may affect true consolidation behaviour, but their effect cannot yet be evaluated.

10.2.4 Water Contents

Before discussing the effect of sample size on volume compressibility, m_v , and permeability, k , it would be useful to discuss the variation of water content

in London clay. If the clay consolidates as a uniform mass, then there should be good correlation between the actual water content after consolidation and the water content calculated from measured changes in volume. Table 10.5 gives the initial, calculated final (calculated on volume change and $G_s = 2.67$) and measured final water contents of the dissipation tests. It will be observed that there is considerable variation in the initial water contents, and generally the measured final water contents are higher than the calculated values. This reflects on the inhomogeneous nature of London clay and the natural variation of moisture in the undisturbed mass. The process of consolidation merely decreases the void ratio and the final moisture contents only reflect the initial variation in the clay. However it should be noted that the graphs of average volume change against time are remarkably smooth for all the tests. Further, the volume changes give values of m_v which appear to be consistent. That the average final water contents are higher than calculated values could be due small undetectable leak or diffusion through the rubber membrane.

It may be mentioned here that no sense could be made from the water content results for the first 12 in. diameter test (UDLC-12-A). Two factors explain this observation. Firstly, on installation of the sample in the large cell, the upper filter stone was erroneously flushed before all round cell pressure had been applied. The sample was therefore wetted on the sides and had to be reinstalled the following day. Hence, there is a distinct

possibility that the initial water content of the sample prior to consolidation would have increased. This is also suggested by the small initial effective stress of 9.6 psi. that was measured, under undrained conditions; otherwise, the 'initial' water content of 28.8% should have shown considerably higher effective stress. Also, a leak developed in the membrane when the sample had undergone about 94% average dissipation. There would probably have been an increase in moisture content because of this leak. The final, probably increased, moisture contents are given below. These were obtained along the axis of the sample, on either side of a diagonal, after the soil in the immediate vicinity of the membrane had been removed.

		<u>Water content %</u>	
Depth from top of sample (in)		1st length	2nd length
(Top)	0.5	35.73	31.8
	3.0	28.50	29.10
	6.0	28.70	29.0
	9.0	28.5	29.5
(Bottom)	11.5	29.0	29.5

It will be seen that one length gives consistently higher results than the other.

The fact that fissured London clay shows considerable variation in moisture content is not a new observation, and has been noted by other workers. Cooling and Skempton (1942) reported a variation of 4% with in a

distance of 2 in. in a borehole sample from Waterloo Bridge site. They also determined index properties at intervals of two inches down the length of two undisturbed samples, and obtained very interesting results given in Table 10.6. The determination of limits over the full depth of a borehole at the same site showed that the variation over a depth of about 40 ft. did not exceed that found over a few inches.

Ward, Samuels and Butler (1959) reported a variation of 2% with in a distance of 1/2 in., 4% variation with in 6 ft. and no greater variation with in a distance of 50 ft. It is worth noting that this variation was not confined to clay near the surface but was noticable in samples as deep as 80 ft. Agarwal (1967) on tests on blue London clay from Wraysbury found that the mean value of water content for the whole mass was 28%, while the intact lumps gave an average moisture content of 27%, with the variation between 26.8 and 27.3%. Som (1968) also noticed that the correspondence of the initial void ratio with that obtained from the volume change was not satisfactory which he attributed to lack of agreement between the water content of the sample measured at the end of the test and that calculated from initial water content and change in volume. Recently in an extreme case, Bhandari (1970) has noted the difference in water content between the intact lumps and the matrix material in a mudflow originating from London clay to be 11% in the wet season and 3% in the dry season.

There is, thus, ample evidence that London clay displays wide variation in moisture content. This may be

partly due to pattern of sedimentation; sand and silt seams and partings; and also possibly due to varying conditions at the time of deposition of the sediments (e.g., sea-bed currents or source of origin of particles), and as a result of swelling and weathering of fissured clay.

It is therefore extremely important to have a larger sample (and hence more representative) for consolidation tests as tests on individual small samples may be extremely misleading. Mirata (1965) for example, mentions two consolidation tests in oedometers conducted by Soil Mechanics Ltd. in 1952 on undisturbed samples of London clay taken from the Manningfield Reservoir site. The figures are quoted as mean for five determinations on the same sample. The first sample taken from a depth of 12 ft. and described as "brown mottled clay containing partings of sand", gave the mean value of C_v as $37.6 \text{ ft}^2/\text{yr}$. For the second sample taken from the depth of 22 ft. where the clay was described as "firm to stiff, slightly sandy, with occasional pockets and laminations of fine sand". the mean value of C_v was found to be $35.3 \text{ ft}^2/\text{yr}$. These values are exceptionally high for C_v obtained from oedometer test on London clay.

10.2.5 Coefficient of compressibility, m_v .

m_v is defined as $\Delta e / \Delta p (1 + e_0) = \Delta V / \Delta p (1 + V_0)$ (for saturated clay) where Δp denotes effective pressure increment, and e_0 and V_0 denote void ratio and volume at beginning of the increment.

The values for the limited number of tests done have been plotted in fig. 10.33. It will be noticed that the 12" diameter samples give higher values of m_v than the 4 in. samples, above average effective stress of 60 psi. The values of m_v decrease with increase in effective stress and perhaps, at high enough pressures, when all the fissures have closed, there is not much difference between m_v obtained from different samples. No distinct trend can be observed on 1½ in. samples. The values obtained from tests on oedometer samples are high, especially for samples obtained from the more shallow boreholes (nos. 1 and 3) which have a higher degree of fissuring. This observation has also been made by Som (1968) who found that values of m_v in oedometer samples were higher because of almost unavoidable swelling of the sample and the disturbance to the soil when preparing the sample for testing. He found that the graph of m_v versus effective stress for the second loading was consistently lower than that obtained from first loading (see also Burmister, 1952). He also conducted tests in the constant rate of strain oedometer and hydraulic oedometer. The results from hydraulic oedometer lay between the curves for the first and second loading of the conventional oedometer, while the constant rate of strain oedometer gave values of m_v which were lower than those for the second loading at low pressures. Cooling and Skempton (1942) remarked that the compressibility of remoulded London clay may be approximately doubled on remoulding at natural moisture content while

even greater values may be found if the sample is allowed to swell. Hence, it would appear that conventional oedometer test cannot be relied upon to furnish reliable values of m_v , especially at low stresses. It is preferable to carry out oedometer tests in the hydraulic oedometer where the flexible membrane ensures uniform pressure over the top of the sample. Further, there is complete control over drainage and the consolidation is not commenced until the build up of pore pressure in the sample is complete. This also enables the use of back pressure to achieve complete saturation of the sample. By measuring the change in height and pore water pressure over a small central area of the sample, the effects of wall friction can be largely eliminated. This apparatus has been described fully by Tan, (1968).

The ratios of axial to volumetric strain, ϵ_a/ϵ_v , have also been tabulated in Table 10.1, and vary from 0.62 to 0.8 for the 4 in. diameter samples. This probably reflects the tendency of the sample to deform axially because of intensity of fissuring at shallow depths (about 10 ft. below gravel). This ratio also shows that the clay did not behave as ideally elastic, isotropic medium, for which ϵ_a/ϵ_v would be 1/3 at small strains. Som (1968) obtained $\epsilon_a/\epsilon_v = 0.45 - 0.55$ for clay from Ongar; but this was from a much greater depth (about 30 ft.). From Table 10.1, it also appears that m_v tends to decrease with decrease in pressure increment ratio. There is, unfortunately, no direct assessment of m_v in the field available for comparison.

10.2.6 Coefficient of permeability, k

The results of permeability against average effective stress have been plotted in fig. 10.34, and show the remarkable effect of sample size and the depth of stratum on the permeability of fissured London clay. The conventional oedometer test results based on Terzaghi theory greatly underestimate permeability and would predict a much slower rate of consolidation than that obtained in the field. London clay, especially at relative shallow depths, is highly fissured and jointed. These discontinuities would greatly increase the rate of consolidation. Further, the clay is distinctly laminated with horizontal particle orientation, as has been observed by Tchalenko, (1967) in thin section studies of the clay. These laminations and fissures imply that the flow of water can take place in both horizontal and vertical directions. Hence, any one dimensional vertical consolidation test of whatever size would tend to underestimate the permeability to certain extent.

In Table 10.2, the results of oedometer tests done on horizontal samples have also been tabulated. The values of permeability have been calculated from C_v and m_v and only give a trend of the relative permeabilities in vertical and horizontal directions, and show that k (horizontal) / k (vertical) can clearly be greater than 1, especially for samples at shallow depths. It must be emphasised that the values of k from oedometer tests should be treated with caution. The difference in the

degree of fissuring, water contents, and other limits can vary so much even for adjoining samples that the true relative permeabilities in the horizontal and vertical directions cannot be accurately assessed in the laboratory by using two different samples. For such a determination to be carried out in the laboratory, it is essential that the tests be carried out on the same sample. Cuboidal samples tested in triaxial cells have been used in the past to determine horizontal and vertical permeabilities of other clays (Normand 1964). It has not been possible to conduct these tests in the present project. The author suggests that if such a determination is carried out, then by using the results of vertical permeability in the 12 in. dissipation tests, a realistic behaviour of two dimensional flow in the field can be ascertained. The insitu test on borehole (4) gives a much lower value of permeability, (2×10^{-9} cm/sec.) compared to 30×10^{-9} cm/sec. for the more shallow borehole (1). This could be a reflection on the more intact clay mass at greater depths, and the possibility that fissure openings and micro-voids between successive laminations are closed at large depths thus resulting in almost same permeability in the vertical and horizontal directions. D'Elia (1968) has recently made some fissure studies on blue London clay from Wraysbury and the limited work clearly indicates that intensity of fissuring decreases with depth. For example, he examined two blocks; one 13x12x9 in. in dimensions located about 5 ft. from the top surface of the clay and the second, 18x19x12 in. located 10 ft.

below the top surface of clay. The results showed that if the number of fissures per unit volume of clay is assumed as degree of fissuring, then the degree of fissuring of block I (482 fissures per cubic ft.) was 3.9 times greater than the degree of fissuring of block II (124 fissures per cubic ft.). It is also possible that the size of the sand plug used for C.H. tests, especially for the deeper clay, was not large enough to intersect flow through the more largely spaced joints and fissures to give a truly representative value of permeability.

10.2.7 Rate of consolidation

There is surprisingly, no data available in the literature about insitu permeability tests in London clay, with which to directly compare the results obtained in the present tests. However, an indication of the validity of the insitu permeability results may be obtained by considering the field rate of consolidation of two earth dams situated on London clay, described by Mirata, (1965). For the first case, settlements of the outlet conduit of the embankment of the William Girling Reservoir (of the Metropolitan Water Board), situated at Chingford Essex, were observed. The Chingford embankment has a height of 41.7 ft. at the section considered and comprised of gravelly fill with central puddled clay core keyed into London clay which lies about 17 ft. below the ground surface. The London clay is overlain by Thames ballast and has a thickness of 40 ft. An average

of 36 determinations using conventional 3 in. diameter oedometer tests gave $C_v = 4.5 \text{ ft}^2/\text{yr}$. However, no correspondence was found between the observed and the calculated rate of settlement (on the basis of $C_v = 4.5 \text{ ft}^2/\text{yr}$). Mirata found that the timesettlement curve when replotted assuming $C_v = 45 \text{ ft}^2/\text{yr}$ gave very close values to the actual behaviour.

The second example is that of the 50 ft. high embankment (at the section considered) at Manningfield. The thickness of the clay is about 450 ft. below the outlet conduit on which settlement measurements were taken. In this case the calculated rate of settlement based on $C_v = 61.4 \text{ ft}^2/\text{yr}$ was only slightly greater than the observed behaviour.

If the coefficient of consolidation is calculated from the insitu permeability values obtained from the author's tests and the approximate values of m_v obtained by extrapolating the graph of the 12 in. diameter dissipation tests (fig. 10.33), then the following values of C_v are obtained.

B/H	insitu $k \times 10^{-9}$ cm/sec.	assumed m_v (ft^2/ton)	calculated C_v (ft^2/yr)
1	34	0.0251	49.3
2	20	0.0224	29.0
3	10	0.0255	14.5
4	2	0.023	2.9

This indicates that most of the settlement would take place in upper, shallow clay and field rate of consolidation would be much faster than that predicted from laboratory tests because of three dimensional flow. Attention has been focussed on the disparity between the field and laboratory rates of consolidation by Rowe (1968) and the subsequent discussion on his paper. A number of case records have been presented which show the influence of geological features like laminations, fissures, silt and sand intrusions which can accelerate field dissipation of pore pressure and consequently, in a number of such cases sand drains designed on the basis of one dimensional laboratory tests would be relatively uneconomical. Rowe, for instance, describes the consolidation of light gray, weathered, highly fissured mudstone clay (L.L. 59%, P.L. 25%, clay fraction 46%) at Staunton Herold Reservoir, where the clay, similar to London clay, was "observed as having hard and soft patches". Laboratory oedometer samples (3 in. diameter and 1 in. thick) gave mean values of C_v as 30 and 60 ft^2/yr for vertical and horizontal drainage. The values of C_v based on insitu permeability and laboratory compressibility ($m_v = 0.0085 \text{ sq.ft/ton}$) were found to be 455, 515 and 1580 ft^2/yr , while the piezometer dissipation records gave C_v ranging from 280 to 520 ft^2/yr . Bishop and Vaughan (1962) describe tests on remoulded boulder clay from Selset Dam where C_v obtained from 4 in. diameter dissipation tests was three times less than the field value. Bishop, Kennard and Vaughan (1964) reported

that the average value from four insitu constant head tests conducted in the foundation strata at Selset gave permeability values which were about 2.5 times that obtained from constant head tests in the laboratory on 4 in. diameter sample remoulded at the same void ratio even though the difference between horizontal and vertical permeability was small. Bromwell and Lambe, (1968) compared the value of C_v (swelling) from oedometer tests with those obtained from piezometer observations for Boston Blue clay (L.L. 50%, P.L. 20%), and found that the field value of $350 \pm 50 \times 10^{-4} \text{ cm}^2/\text{sec.}$ was about six times the laboratory value. Other such cases have been described by Lewis, (1963); Rivard and Kohusha, (1965); Henkel, (1965); Rowe, (1964, 1968) Bishop and Al-Dhahir, (1969) and others.

Apart from the effect of lateral drainage in the field, the variation of m_v with depth can also account for substantial increase in the field rate of settlement as has been illustrated by Schiffman and Gibson, (1964) who showed that for London clay the rate of consolidation would approximately double if decrease of m_v and k with depth as shown by oedometer tests, is considered. Another factor about which relatively little is known is the process of three dimensional consolidation in the field. Gibson, Schiffman and Pu, Gibson and McNamee, (1957, 1960, 1963) and Davis and Poulos, (1968) have shown that apart from the ratio of lateral to vertical permeability, 3-D consolidation is also greatly affected

by the ratio of the vertical drainage to radius of loaded area. Solutions for 3-D consolidation are very complex mathematically and cover limited cases. Som, (1968) compared the rate of consolidation of a 45 ft. layer of clay having C_v of 11 ft²/yr and found that the three dimensional consolidation would take 15 years to reach 80% of total settlement as compared to 50 years for one dimensional consolidation. Hence, all one dimensional laboratory tests would underestimate the overall permeability of London clay, especially at shallow depths, since three dimensional flow will not be taken into account.

10.3 Conclusions

From the limited number of tests done, it would be wrong to offer general conclusions about the consolidation behaviour of London clay on sample size. However the following trends are noted:

- 1) C_v from 12 in. diameter samples is about $1\frac{1}{2}$ times that of 4 in. diameter samples and approximately 3 times that obtained from oedometer samples.
- 2) The values of m_v obtained from 12 in. diameter samples is about twice that obtained from smaller samples and decreases rapidly with effective stress.
- 3) The permeability of London clay is greatly affected and influenced by sample size. The value of permeability from 12 in. diameter samples is about

twice that of 4 in. samples. The insitu permeabilities as determined by constant head tests are considerably higher because of vertical and lateral drainage in the clay. At shallower depths, the insitu permeability is much higher than that of the deeper stratum since at shallow depths, the intensity of fissuring, and k_h/k_v are likely to increase and fissure spacing would decrease.

- 4) It has been observed that Terzaghi one dimensional consolidation theory, Davis and Raymond (1965) theory, and Poskitt (1969) theory do not adequately represent the one dimensional consolidation behaviour of London clay.
- 5) Finally it is suggested that laboratory tests would greatly underestimate the rate of consolidation in the field. It is essential to conduct insitu permeability tests at different depths to accurately estimate field rates of consolidation in fissured London clay.

10.4 Some suggestions for further research

Since fissured London clay shows marked variation of in-situ permeability with effective stress, it would be of considerable practical interest to investigate this aspect more fully by conducting other field constant head tests at other site in London clay.

The size of sand plug used in this testing programme was typically 2'-6" high x 8" diameter. It is suggested that constant head tests using sand plugs of larger dimensions should be used to determine in-situ permeability of relatively intact clay at greater depths at the site at Wraysbury.

Consolidation test data from 4 in. and 12 in. diameter dissipation tests at low effective stress is lacking. It would be useful to conduct these tests on clay from Wraysbury at low effective stresses for comparison with in-situ tests, and field records.

The permeability of London clay in the laboratory has been determined indirectly from dissipation and oedometer tests and by the use of Terzaghi consolidation theory. Because of the uncertainty of Terzaghi theory, a direct comparison of these results with in-situ permeability test results is not valid. It is therefore suggested that laboratory value of permeability in both vertical and horizontal directions should be determined from constant head tests on cuboidal samples.

Table 10.1

Results of dissipation tests

Test No.	p'_o psi.	Δp psi.	$\frac{\Delta p}{p'_o}$	av. σ'_n at 50% pp. dis- sipation	ft ² /yr		
					(1) c_v 50	(2) c_v 50 pp. av.	$\left\{ \frac{(1)}{(2)} \right\}$
UDLC-4-A-1	25.5	29.5	1.157	45.86	3.35	3.27	1.02
UDLC-4-A-2	55.2	5.3	0.096	58.86	3.83	2.39	1.6
UDLC-4-A-3	60.0	20.0	0.333	73.8	2.78	2.27	1.22
- 4-B-1	27.0	25.0	0.926	44.25	4.23	3.55	1.19
- 4-B-2	51.5	15.0	0.29	61.85	3.75	2.49	1.52
- C-1	22.65	6.2	0.2737	26.93	8.54	4.97	1.72
- C-2	28.73	5.27	0.183	32.37	6.0	3.10	1.94
- C-3	33.88	5.12	0.151	37.42	6.2	3.05	2.03
UDLC-12-A	9.6	30.4	3.16	29.97	10.4	7.93	1.31
UDLC-12-B1	21.5	27.0	1.258	40.1	5.83	5.04	1.16
UDLC-12-B2	48.5	5.5	0.113	52.18	6.82	2.71	2.51
UDLC-12-B3	54	6	0.111	58.02	5.52	3.93	1.40
UDLC-1 $\frac{1}{2}$ -D-1	10	20	2.0	19.52	7.88	9.07	0.81
UDLC-1 $\frac{1}{2}$ -D-2	30	20	0.67	43.8	3.89	6.02	0.65
UDLC-1 $\frac{1}{2}$ -I-1	47.5	21	0.44	62.0	1.805	1.24*	1.45
UDLC-1 $\frac{1}{2}$ -I-2	68.5	15.5	0.226	79.2	2.99	1.54*	1.94
UDLC-I-1 $\frac{1}{2}$ E	45.8	30.2	0.66	66.6	3.33	3.52*	0.95

* calculated on basis of average change in height

ft ² /ton m _v	x10 ⁹ cm/sec.		ft ² /yr		x10 ⁹ cm/sec.		ε _a %	ε _v %	$\frac{\epsilon_a}{\epsilon_v}$
	k 50(1)	k 50(2)	C _v 90mx. (3)	C _v 90av. (4)	k 90(3)	k 90(4)			
0.01197	1.03	1.05	2.77	3.489	0.89	1.12	1.66	2.265	0.73
0.0076	0.77	0.48	1.386	1.69	0.27	0.34	0.175	0.259	0.675
0.0081	0.61	0.51	2.33	2.326	0.52	0.51	0.666	1.032	0.655
0.0095	1.08	0.91	4.26	3.506	1.08	0.89	1.23	1.56	0.79
0.0099	1.0	0.66	2.42	1.994	0.64	0.53		0.985	
0.0127	2.87	1.7	3.66	3.75	1.25	1.27	0.394	0.505	0.78
0.0158	2.54	1.31	3.42	2.87	1.46	1.22	0.33	0.535	0.62
0.00976	1.73	0.85	3.136	3.36	0.82	0.88		0.32	
0.02217	6.2	4.72	5.06	4.0	3.05	2.37		4.34	
0.020	3.03	2.67	2.21	3.50	1.35	1.69		3.465	
0.0135	2.47	0.98	7.08	2.94	2.57	1.06		0.497	
0.0096	2.24	1.01	5.90	5.52	1.52	1.41		0.37	
0.0226	4.79	5.51	5.36	9.3	3.26	5.66		2.92	
0.019	1.99	3.08	3.26	5.23	1.66	2.67		2.49	
0.006*	0.26	0.29	1.38	1.078	0.21	0.17	0.8		
0.0046*	0.37	0.19	1.692	1.378	0.20	0.17	0.456		
0.0102*	0.91	0.96	3.81	3.17	1.042	0.86	0.61	1.98	0.31

Table 10.2

Results of oedometer tests on blue London clay
(Wraysbury)

B/H	Depth below gravel	Direction	0.2 - 0.4 T/ft ²		
			c_v ft ² /yr	m_v ft ² /ton	$k \times 10^9$ cm/sec.
1	9'-3"	V	6.62	0.0115	2.04
		H	16.11	0.00226	0.98
		$\left(\frac{H}{V}\right)$	2.44	0.197	0.48
2	21'-3"	V	10.07	0.0097	2.62
		H	25.16	0.004	2.93
		$\left(\frac{H}{V}\right)$	2.35	0.41	1.12
3	9'-0"	V	7.81	0.016	3.36
		H	40.10	0.008	15.56
		$\left(\frac{H}{V}\right)$	5.14	0.50	4.62
4	30'-6"	V	7.0	0.005	0.94
		H	25.16	0.002	1.35
		$\left(\frac{H}{V}\right)$	3.6	0.40	1.435

Note: Results shown for vertical and horizontal directions
were obtained from tests conducted on different samples.

0.4 - 0.8 T/ft ²			0.8 - 1.2 T/ft ²			1.2 - 2.0 T/ft ²		
C _v	m _v	kx10 ⁹	C _v	m _v	k	C _v	m _v	k

3.19	0.022	1.9	2.50	0.0228	1.52	2.72	0.0133	1.26
15.6	0.0055	2.32	9.96	0.008	2.12	9.39	0.0101	1.09
4.80	0.25	1.22	3.98	0.35	1.39	3.45	0.85	0.86

6.25	0.014	2.35	4.48	0.0152	1.82	4.04	0.013	1.79
11.95	0.007	3.64	10.47	0.008	2.26	10.05	0.0072	1.96
1.91	0.5	1.55	2.34	0.536	1.24	2.49	0.55	1.10

4.59	0.0258	3.17	4.35	0.023	2.69	2.67	0.0211	1.51
18.46	0.0097	4.81	13.08	0.0118	4.15	12.21	0.0127	4.12
4.0	0.38	1.52	3.0	0.51	1.54	4.57	6.03	2.73

4.38	0.009	1.14	4.16	0.0107	1.197	3.43	0.0114	1.05
20.08	0.0038	1.87	12.2	0.0044	1.92	5.16	0.0058	0.81
4.56	0.39	1.64	2.93	0.41	1.6	1.5	0.51	0.77

Table 10.5

Final calculated and measured water contents of
dissipation test samples.

Water contents %

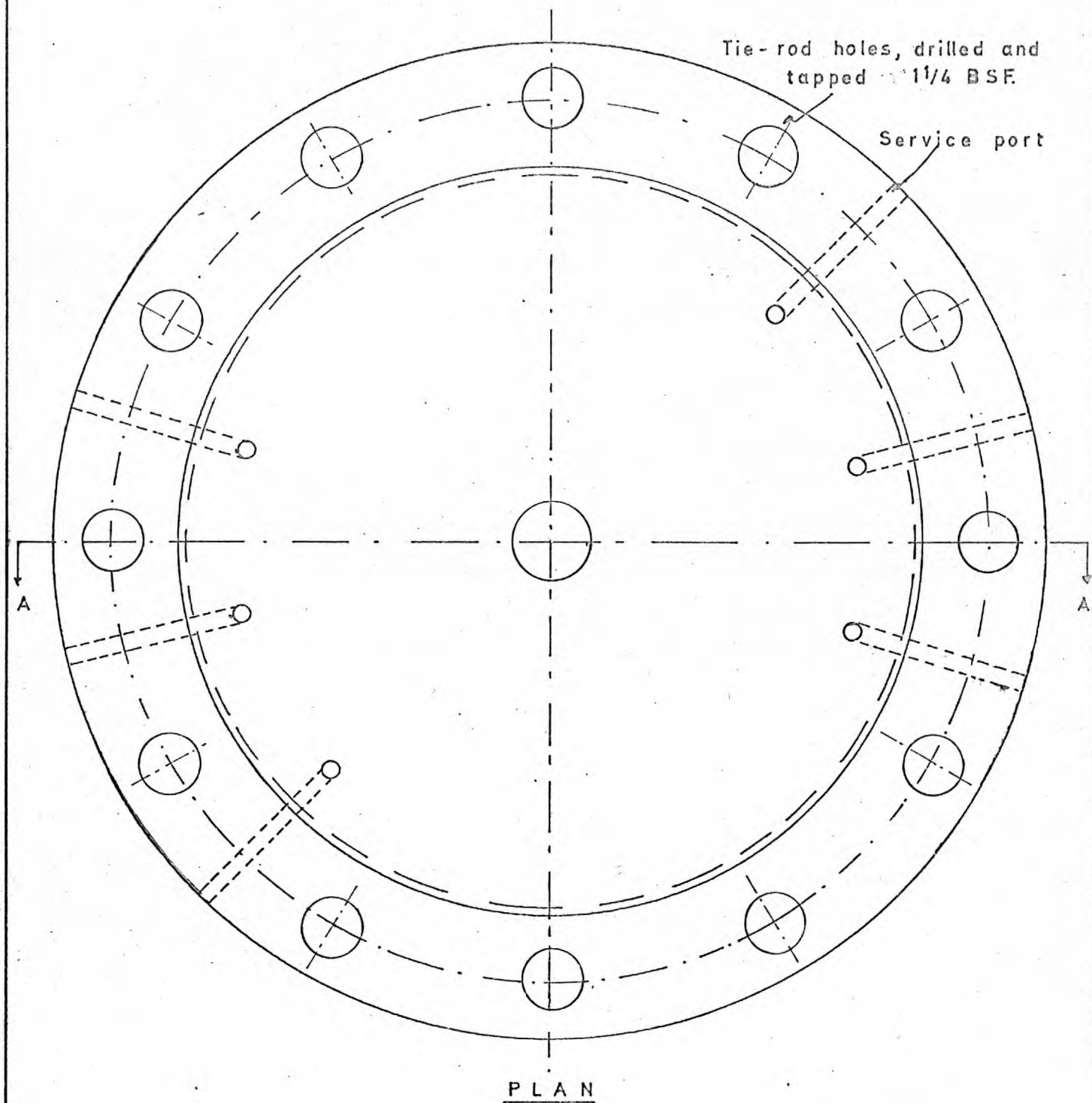
Test No.	Initial	Final from vol. change	Measured final
UDLC-12-B	29.5	26.6	29.14
	30.24 (30.31)	27.33 (27.36)	27.65 (28.39)
	31.20	28.14	29.10
			30.05
			27.90
			27.69
			27.20
			27.64
UDLC-4-A	28.62	26.27	27.16
	29.0 (28.81)	26.63 (26.45)	26.33 (26.64)
			26.52
UDLC-4-B	28.0	26.36	26.50
	29.1 (28.55)	27.35 (26.85)	27.6 (27.13)
			27.3
UDLC-4-C	30.9	29.8	29.9
	30.6 (30.97)	29.66 (30.04)	30.2 (30.05)
	31.4	30.65	
UDLC-1½-D	29.04	26.74	26.78 (26.78)
	29.18 (29.11)	26.63 (26.69)	
UDLC-1½-E	29.5	28.17	27.95 (27.95)
	29.0 (29.25)	27.73 (27.95)	

() figures shown thus are average water contents,

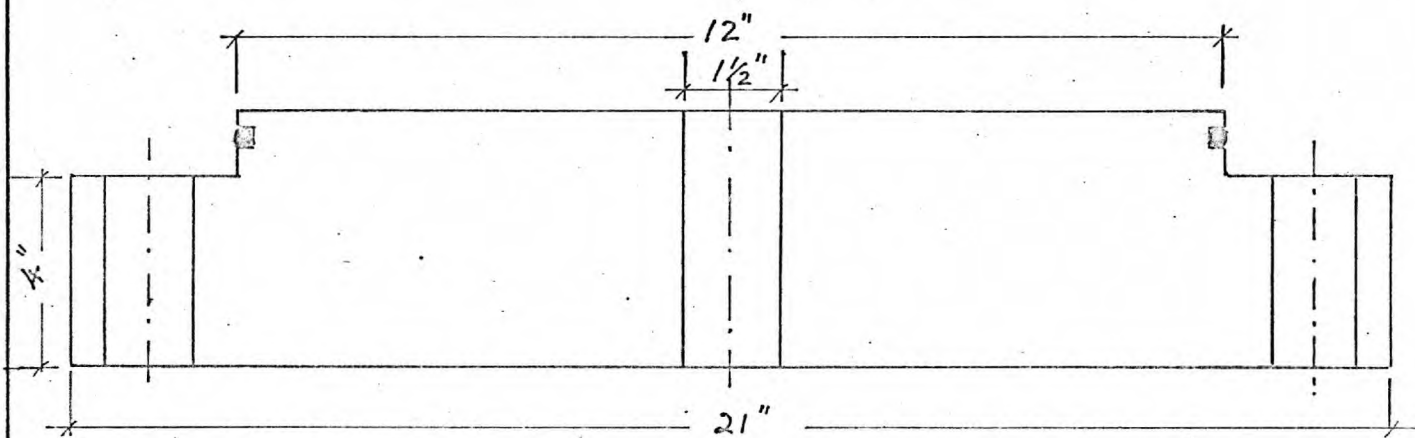
Table 10.6 (after Cooling and Skempton, 1942)

Variation of water content and Atterberg limits in a sample of London clay.

Distance from top of sample (in.)	Sample 72/2/a			Sample 72/s/f		
	w/c	L.L.	P.L.	w/c	L.L.	P.L.
	%	%	%	%	%	%
2	24	71	24	27	78	28
4	24	70	24	28	73	24
6	22	60	22	28	80	27
8	24	61	22	24	75	27
10	25	67	24	25	83	26
12	25	72	25	24	81	27



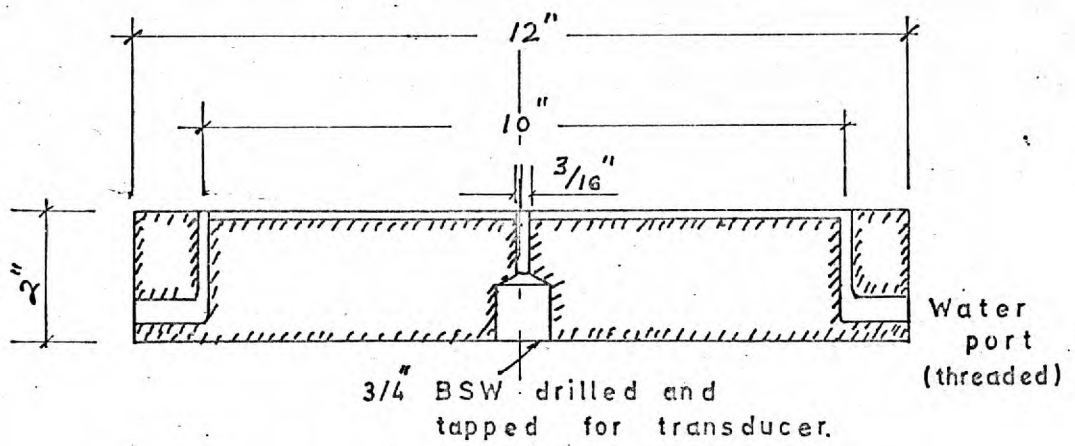
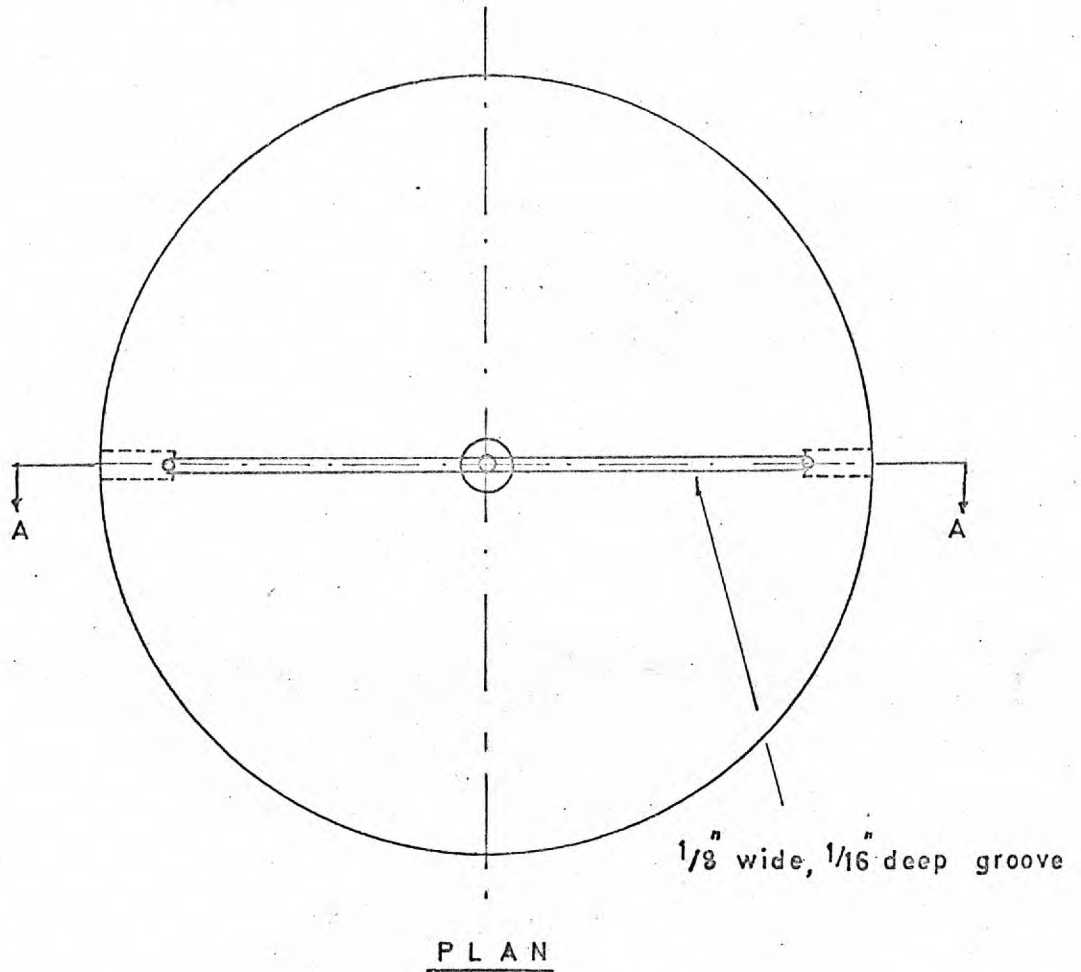
PLAN



SECTION A-A

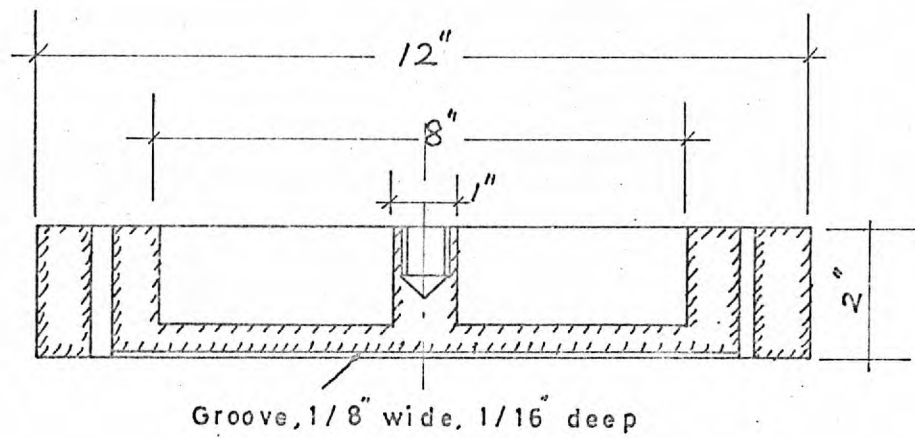
12 in. DISSIPATION CELL
Bottom Plate

FIG. 10.1

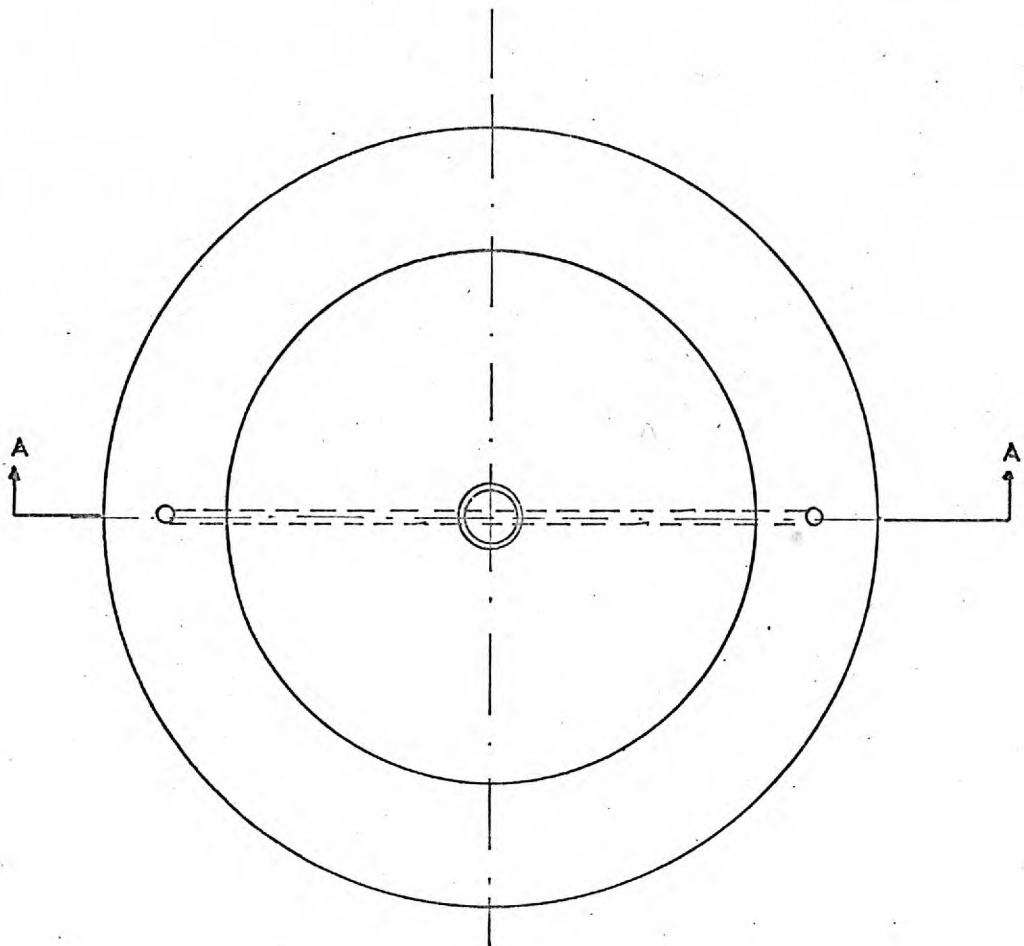


Sample Pedestal

12 in. DISSIPATION CELL



SECTION A-A



PLAN

Sample Top Cap.

12 in. DISSIPATION CELL

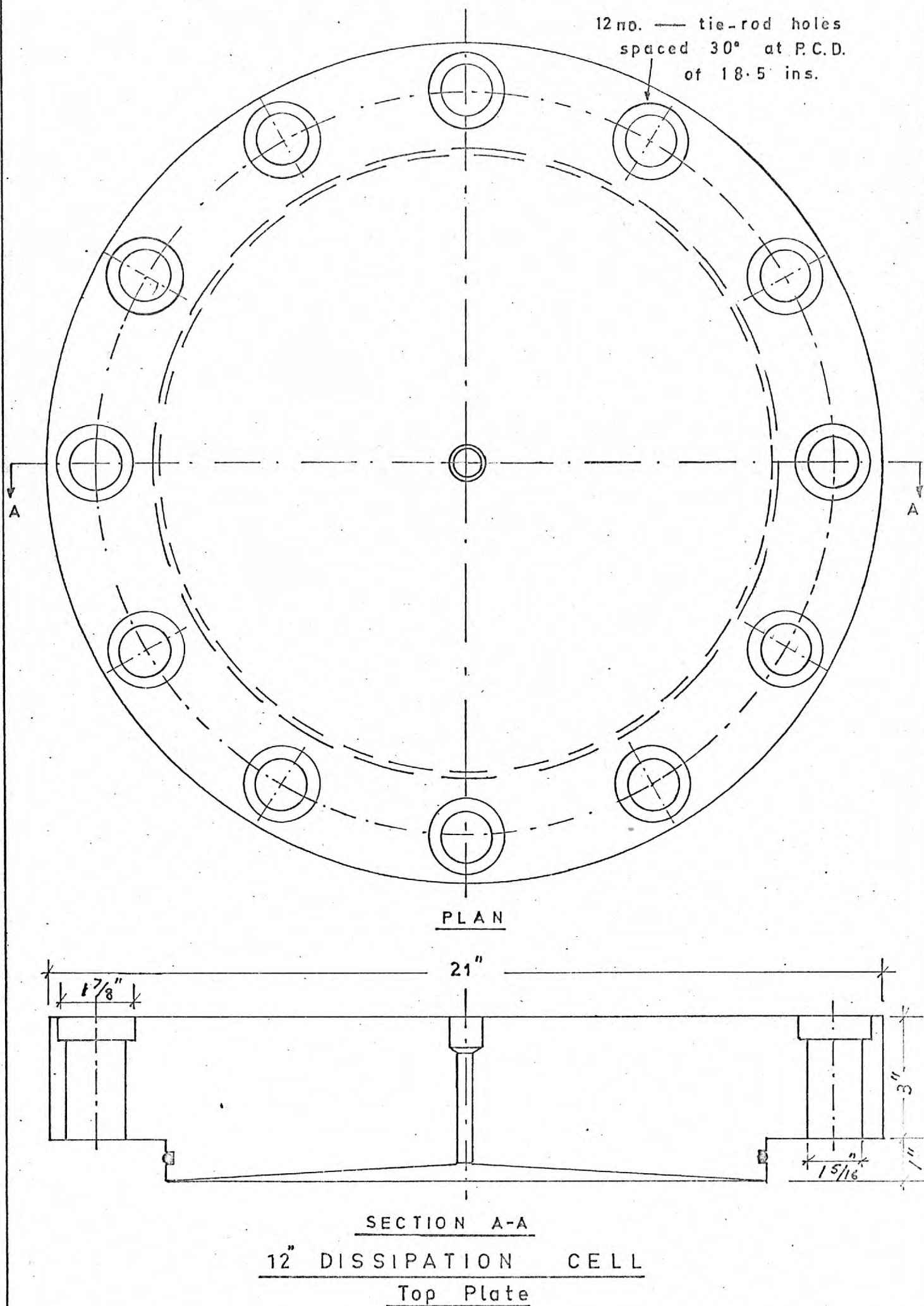


FIG. 10. 4

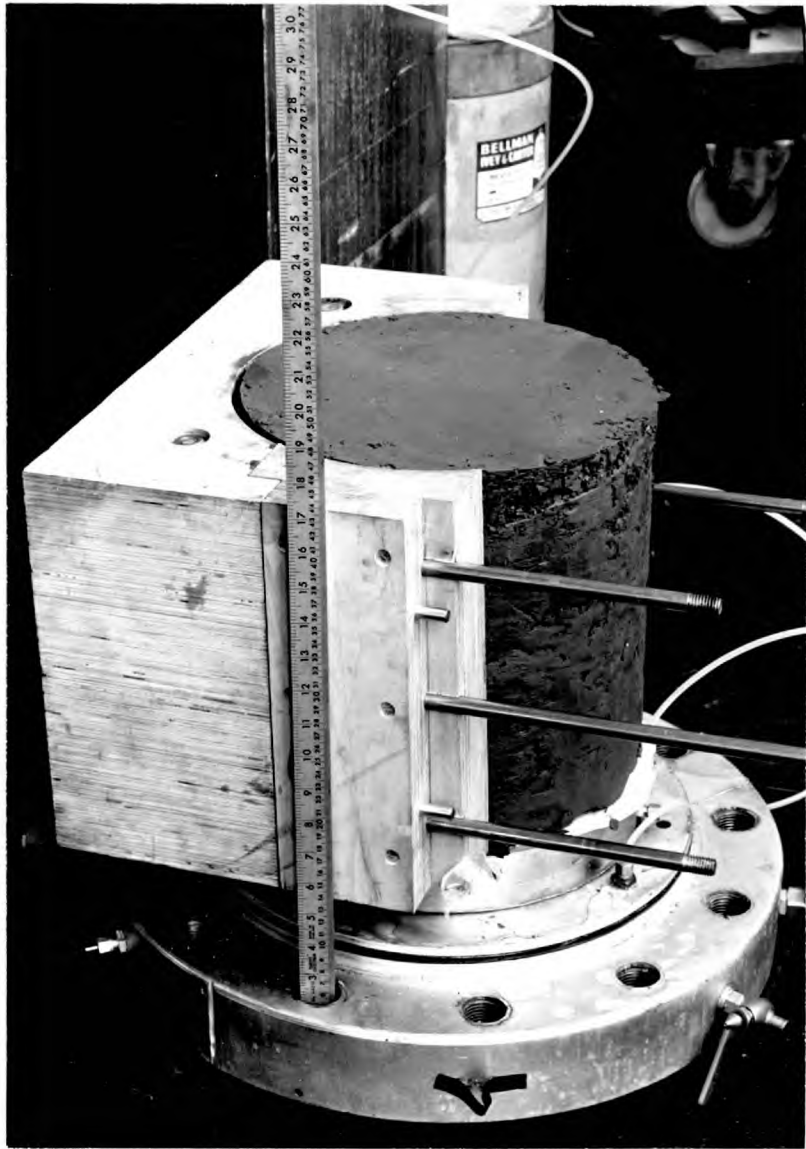


PLATE 10.1

12 in. diameter sample of blue London
clay being removed from the mould for
assembly in the dissipation cell.



PLATE 10.2

12 in. diameter sample mounted on
the pedestal of the dissipation cell.

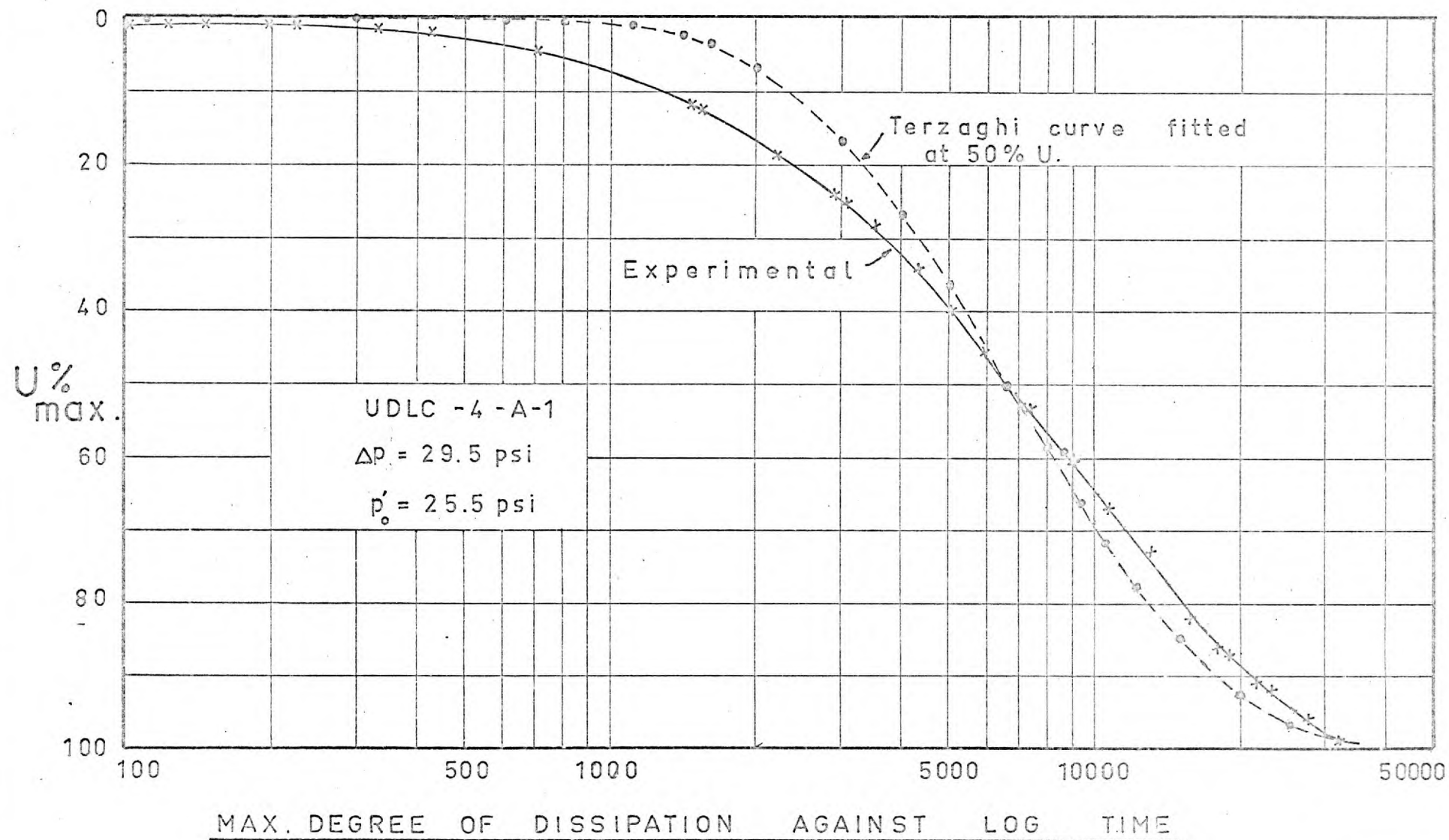


FIG. 10.5

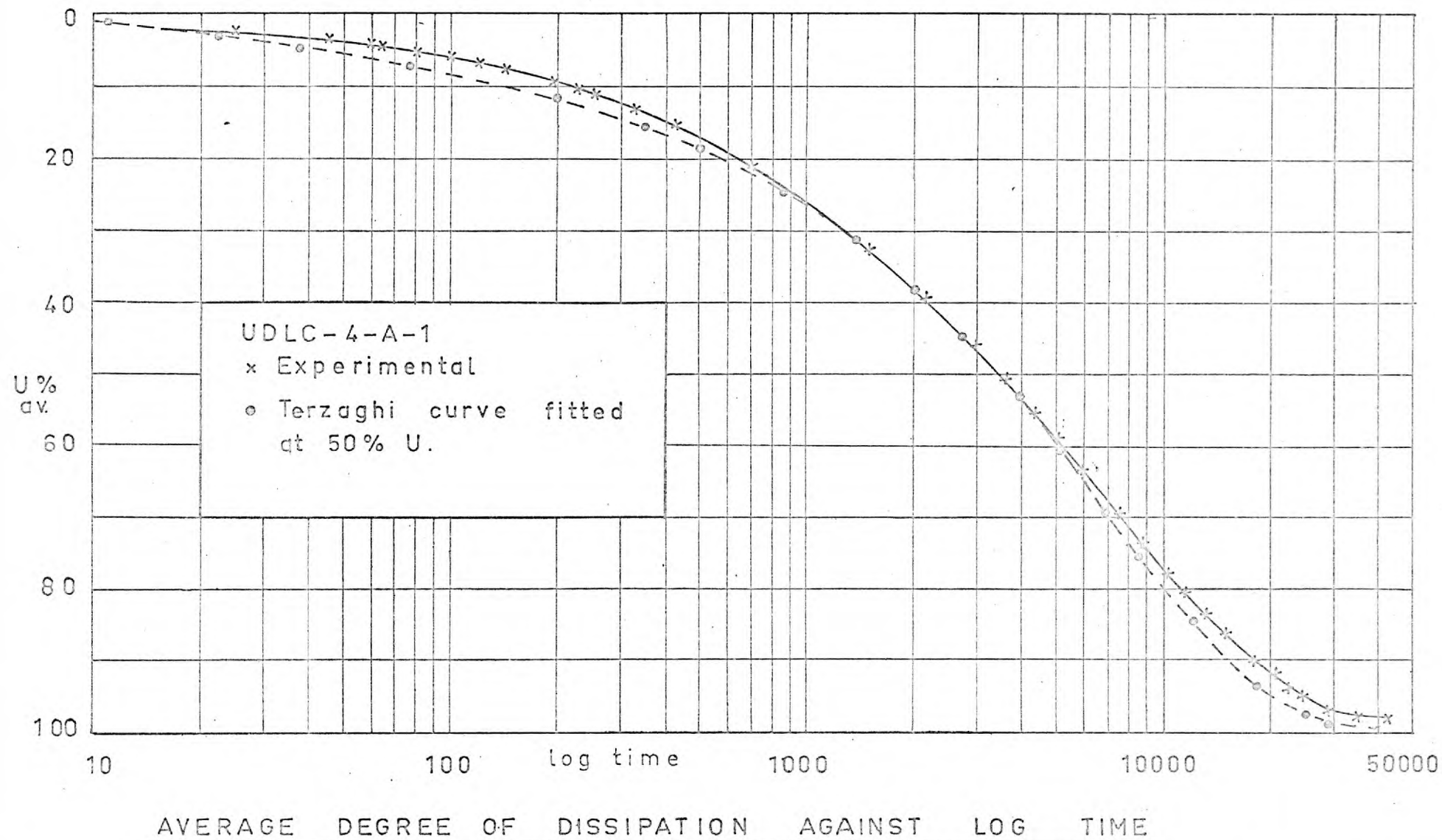
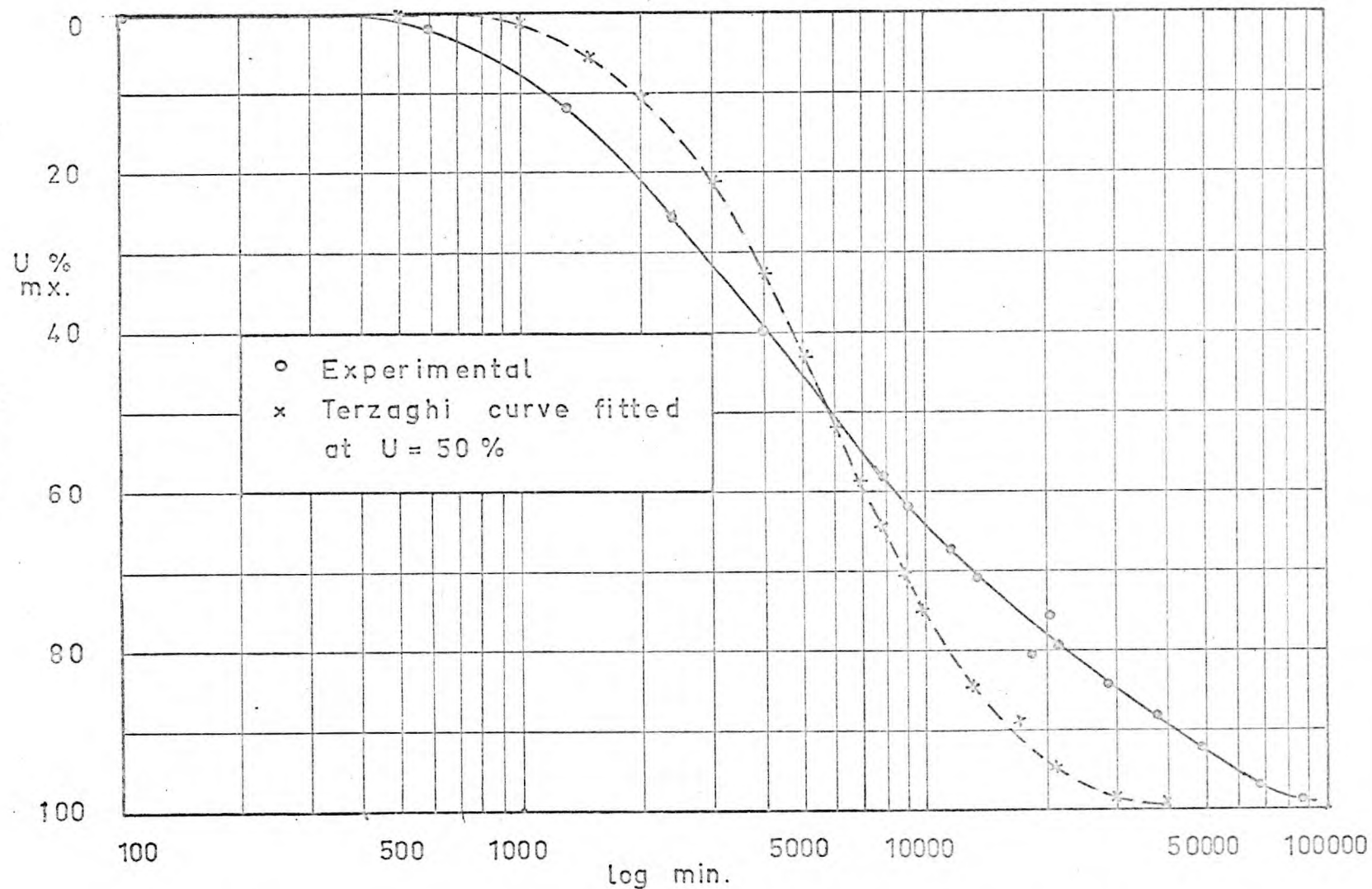


FIG.10.6

UDLC-4-A-2

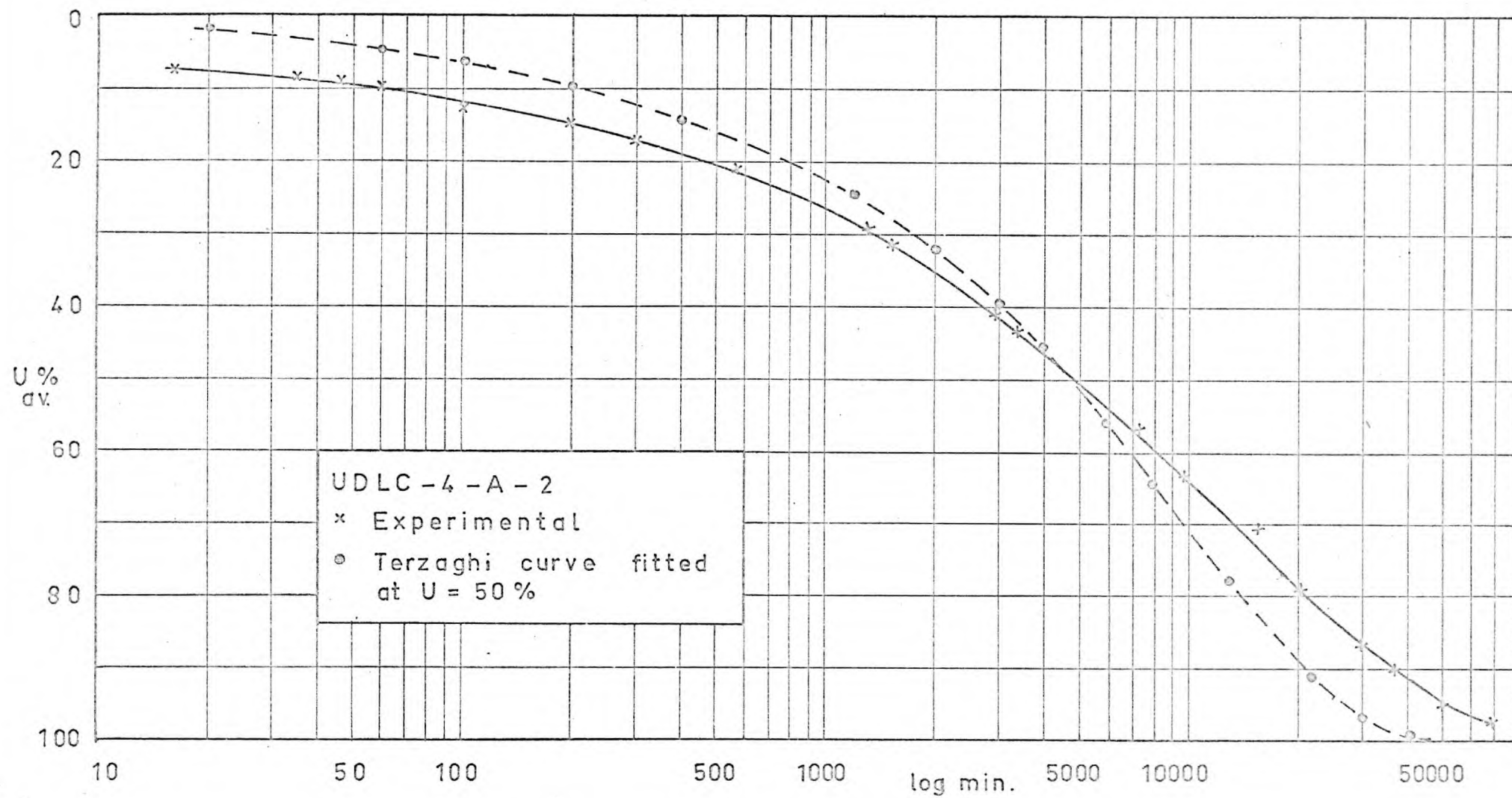
$p'_0 = 55.2$ psi

$\Delta p = 5.3$ psi



MAX. DEGREE OF DISSIPATION AGAINST LOG TIME

FIG. 10.7



AVERAGE DEGREE OF DISSIPATION AGAINST LOG TIME

FIG. 10.8

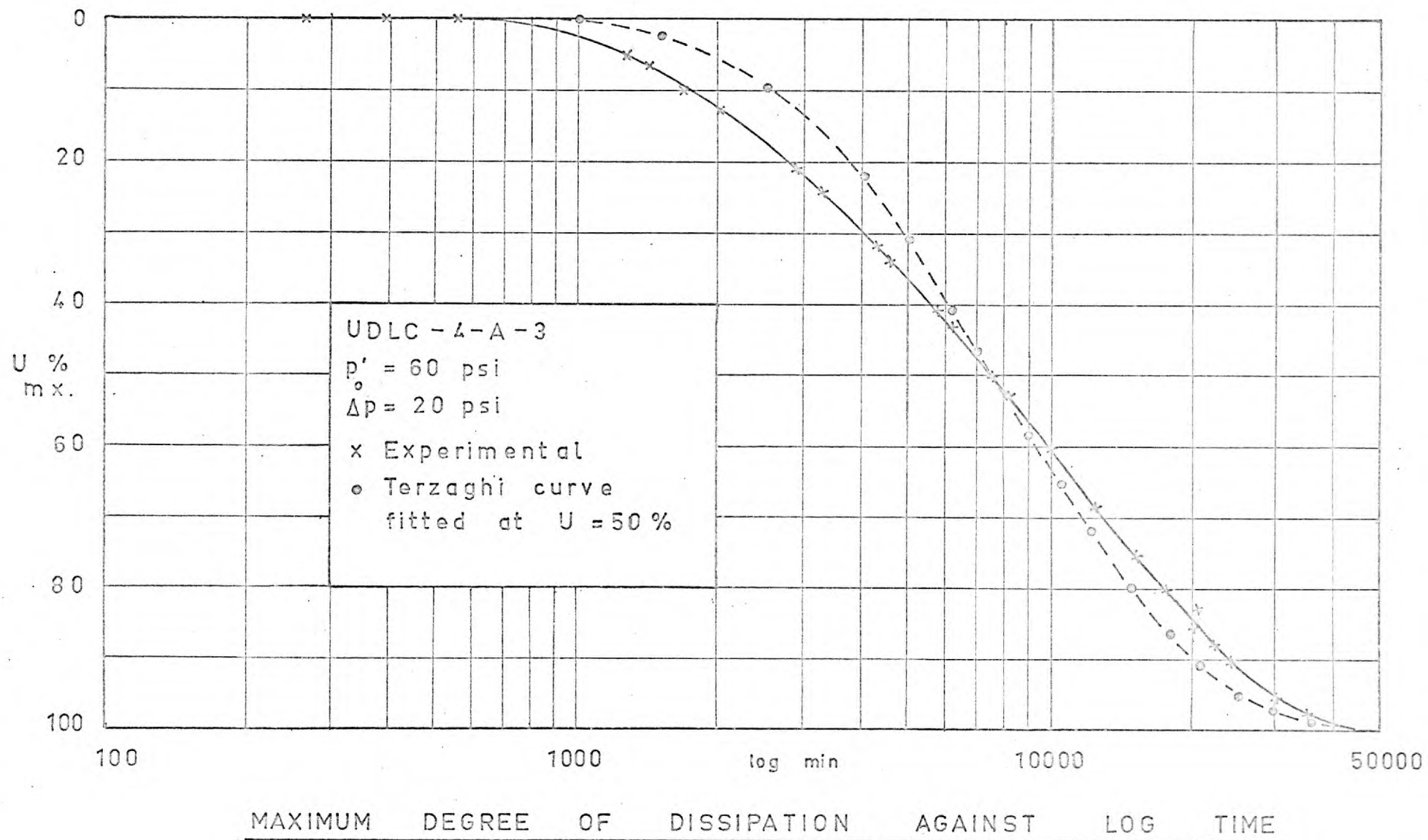
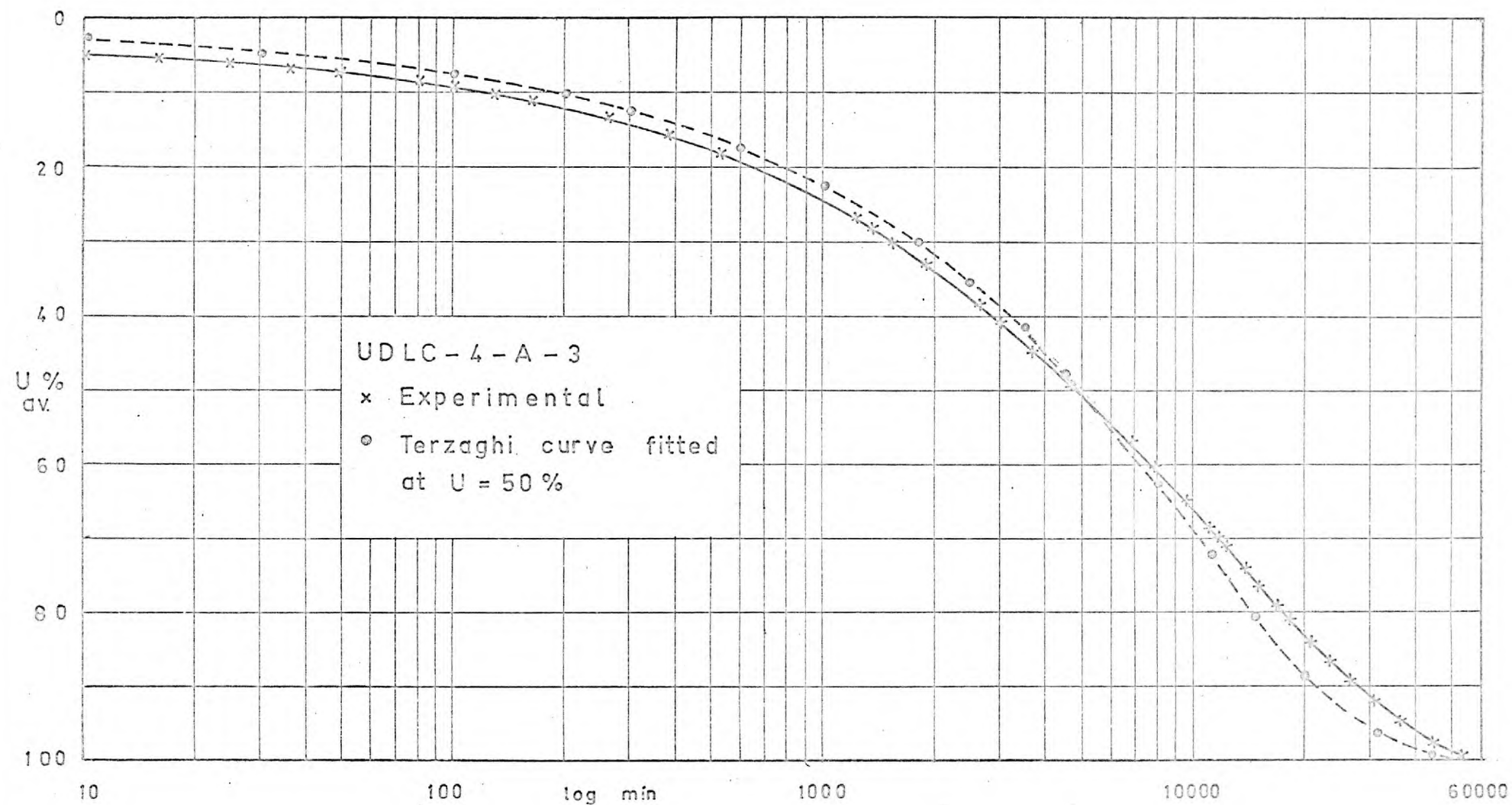
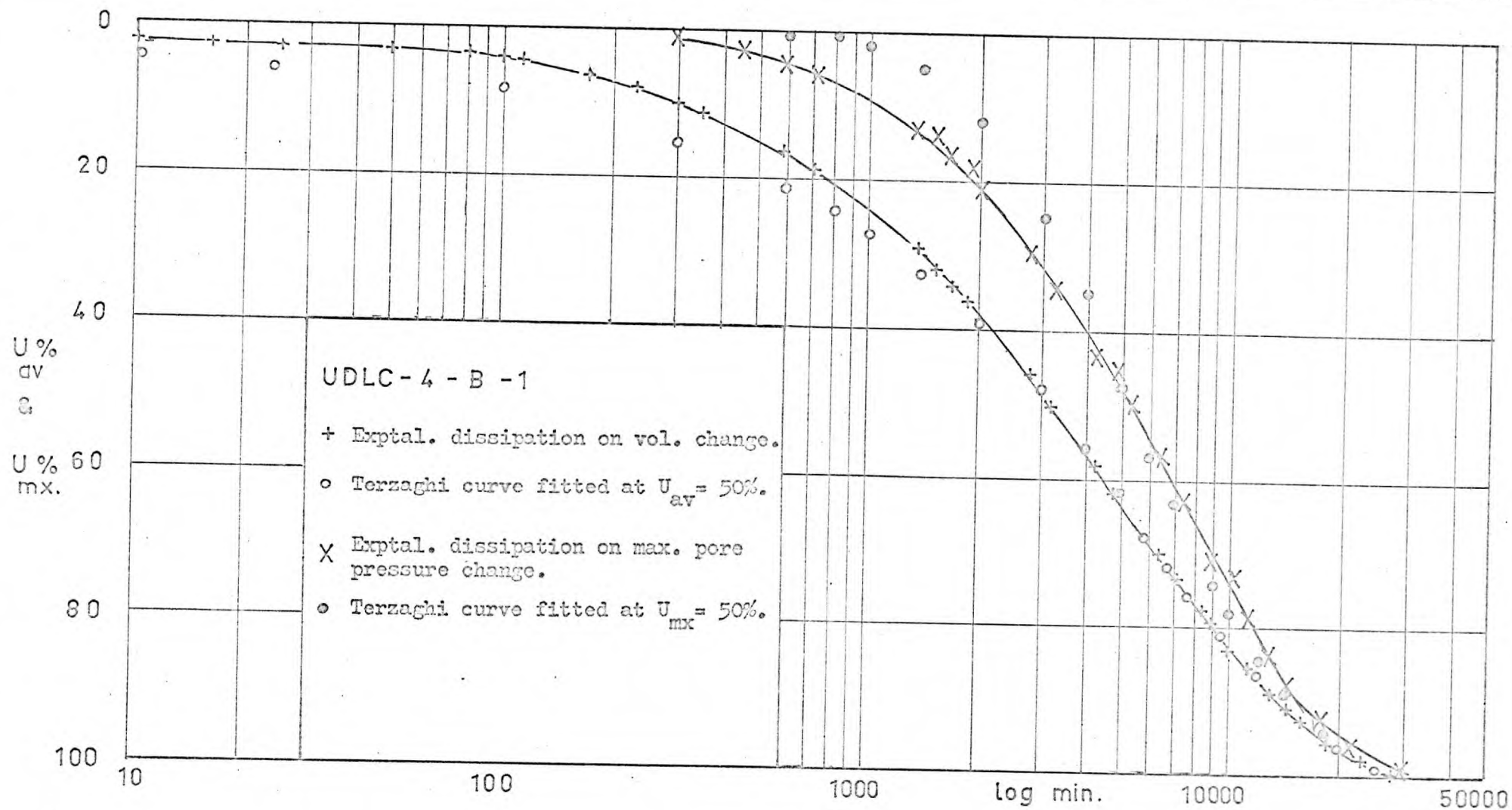


FIG. 10.9

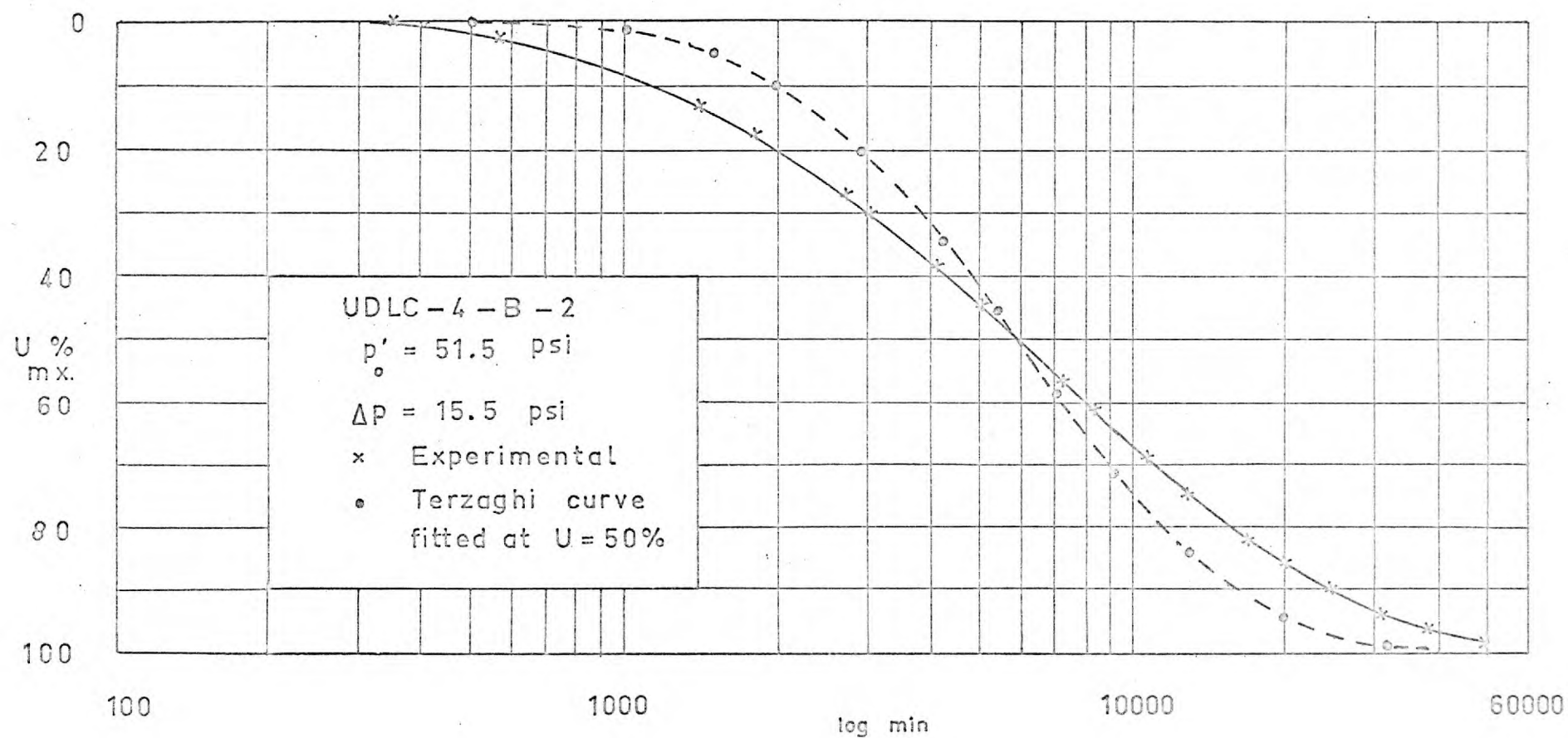


AVERAGE DEGREE OF DISSIPATION AGAINST LOG TIME

FIG.10.10

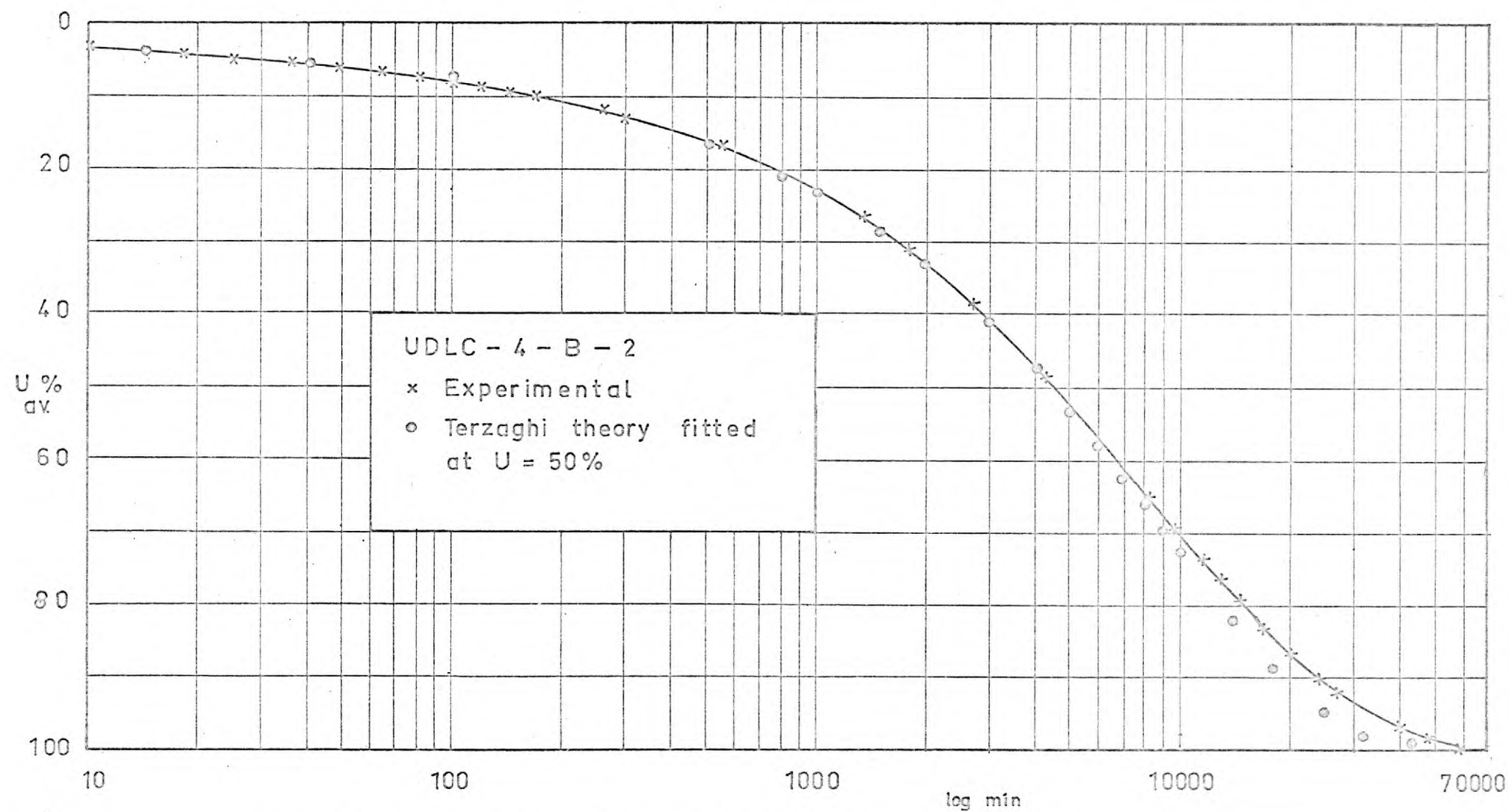


DEGREE OF DISSIPATION BASED ON AVERAGE VOLUME CHANGE AND MAXIMUM PORE PRESSURE CHANGE AGAINST LOG TIME.



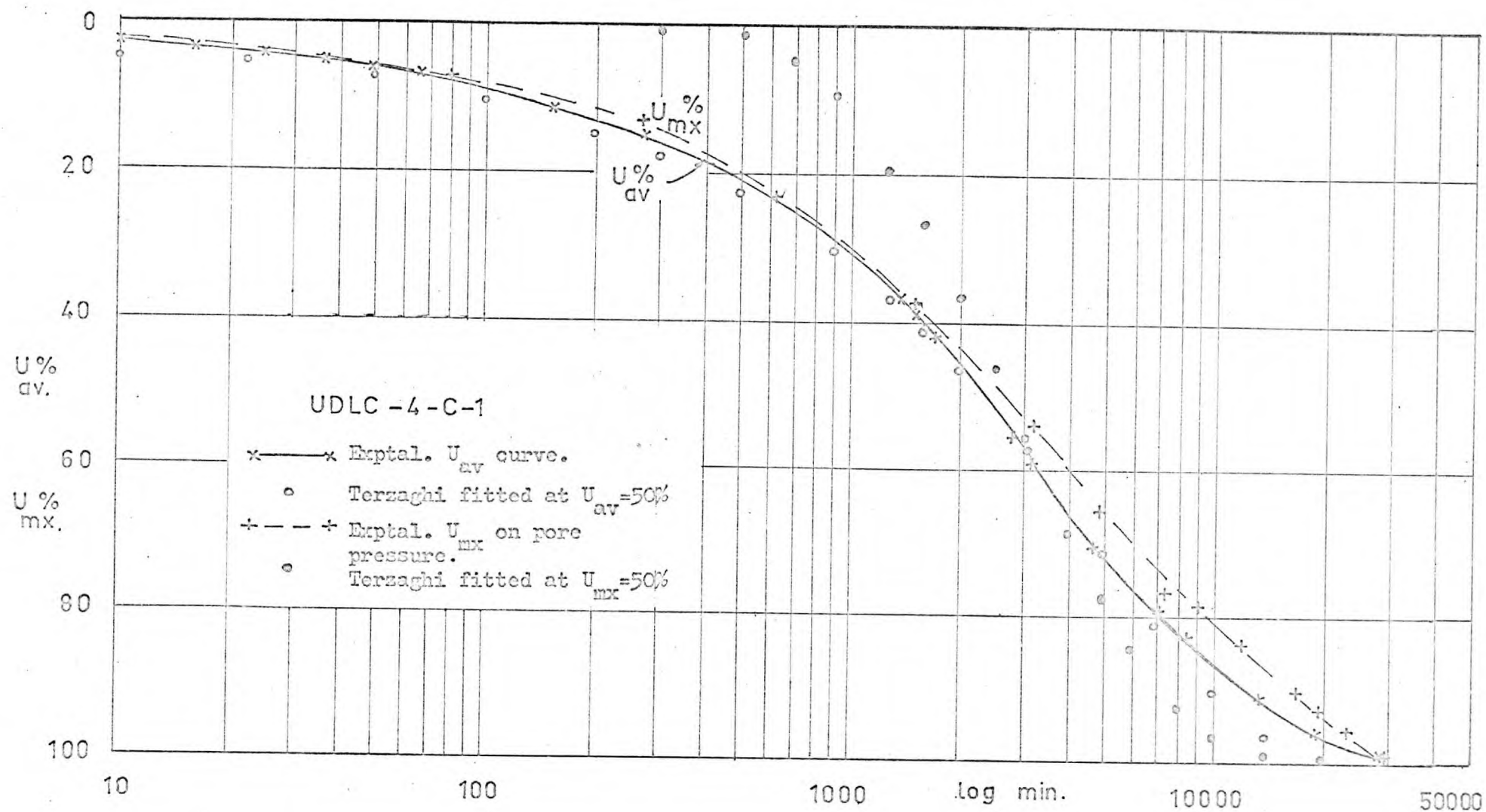
MAXIMUM DEGREE OF DISSIPATION AGAINST LOG TIME

FIG. 10.12

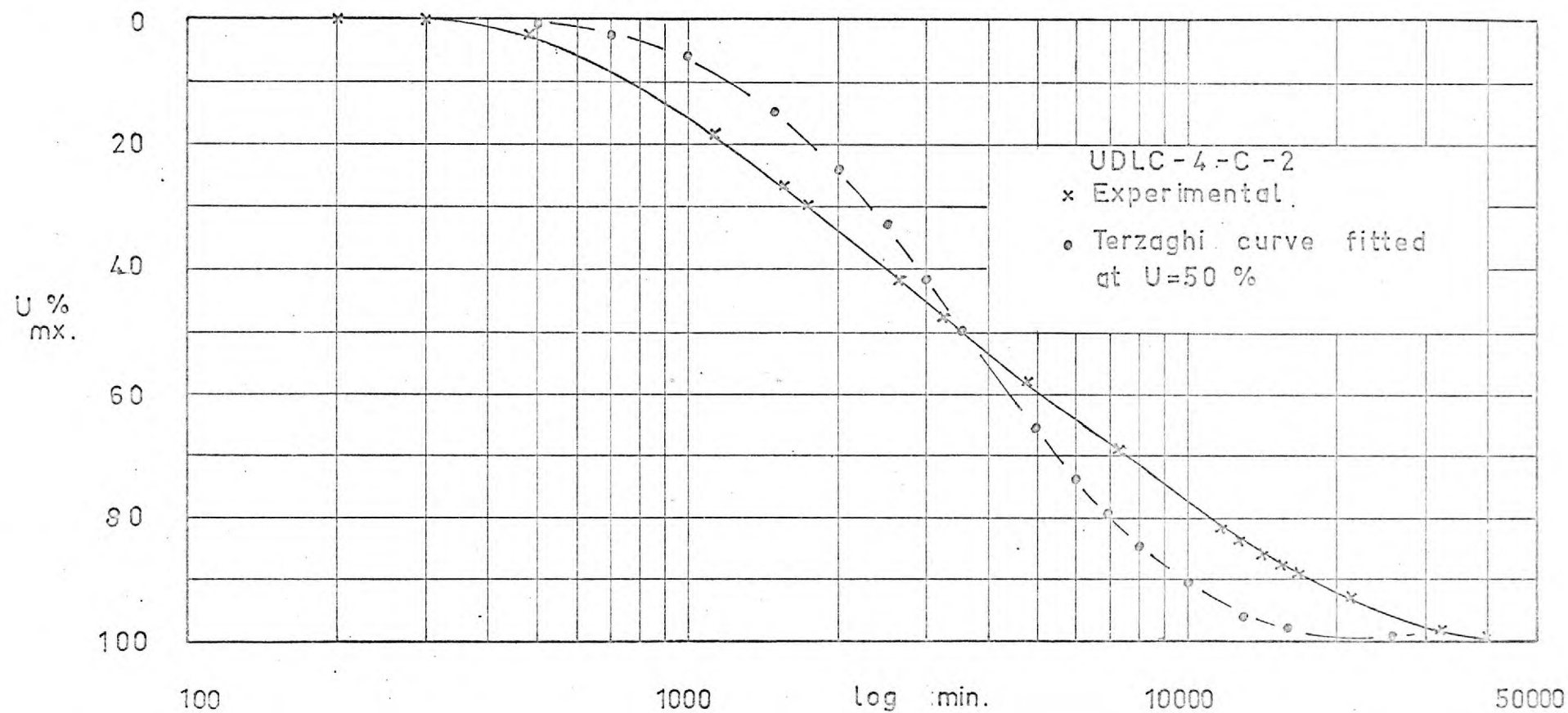


AVERAGE DEGREE OF DISSIPATION AGAINST LOG TIME

FIG. 10.13

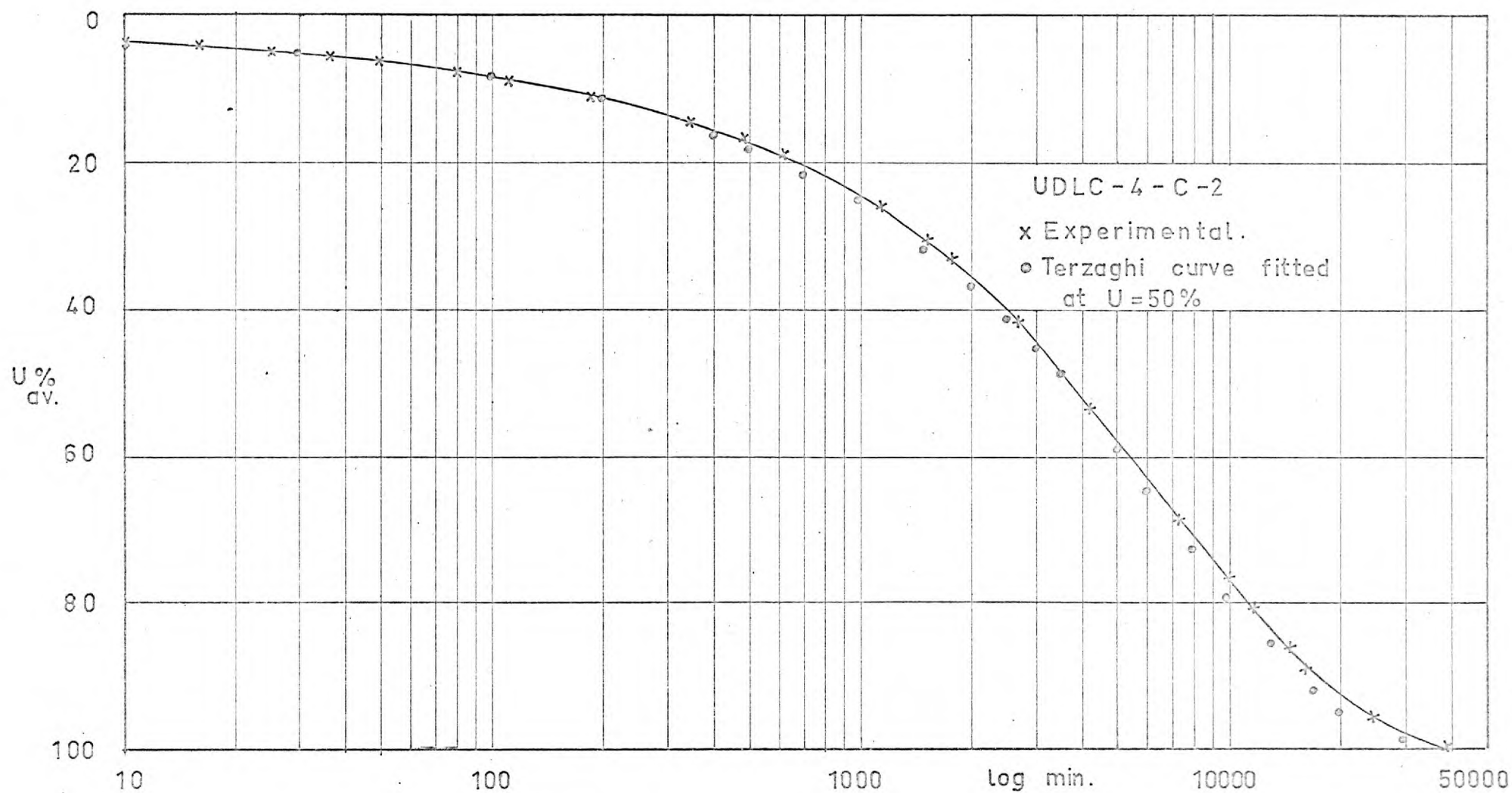


DEGREE OF DISSIPATION BASED ON AVERAGE VOLUME CHANGE AND
MAXIMUM PORE PRESSURE CHANGE AGAINST LOG TIME.



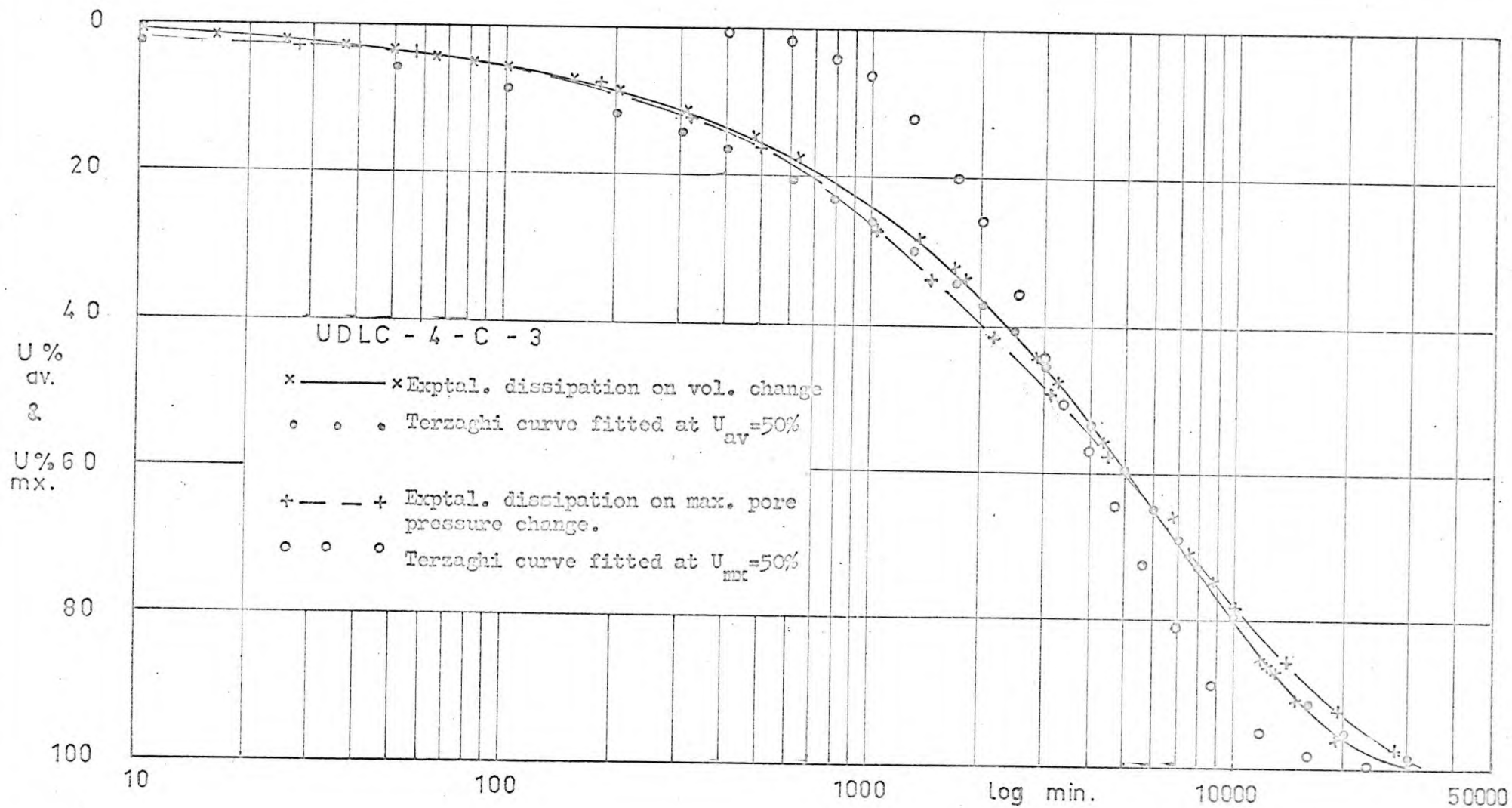
Maximum degree of dissipation against log time

FIG. 10.15

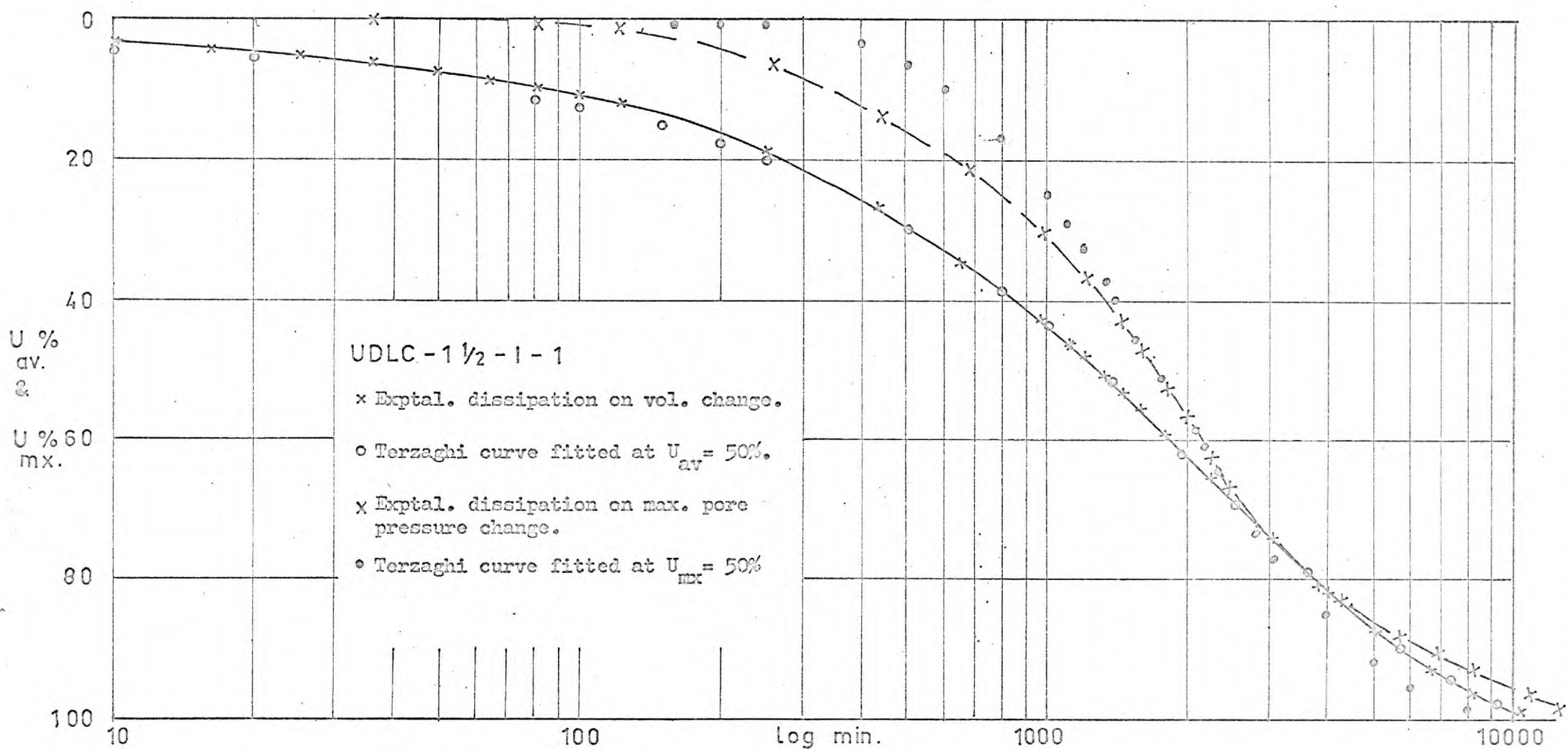


AVERAGE DEGREE OF DISSIPATION AGAINST LOG TIME.

FIG. 10.16



DEGREE OF DISSIPATION BASED ON AVERAGE VOLUME CHANGE AND
MAXIMUM PORE PRESSURE CHANGE AGAINST LOG TIME.



DEGREE OF DISSIPATION BASED ON AVERAGE VOLUME CHANGE AND MAXIMUM PORE PRESSURE CHANGE AGAINST LOG TIME.

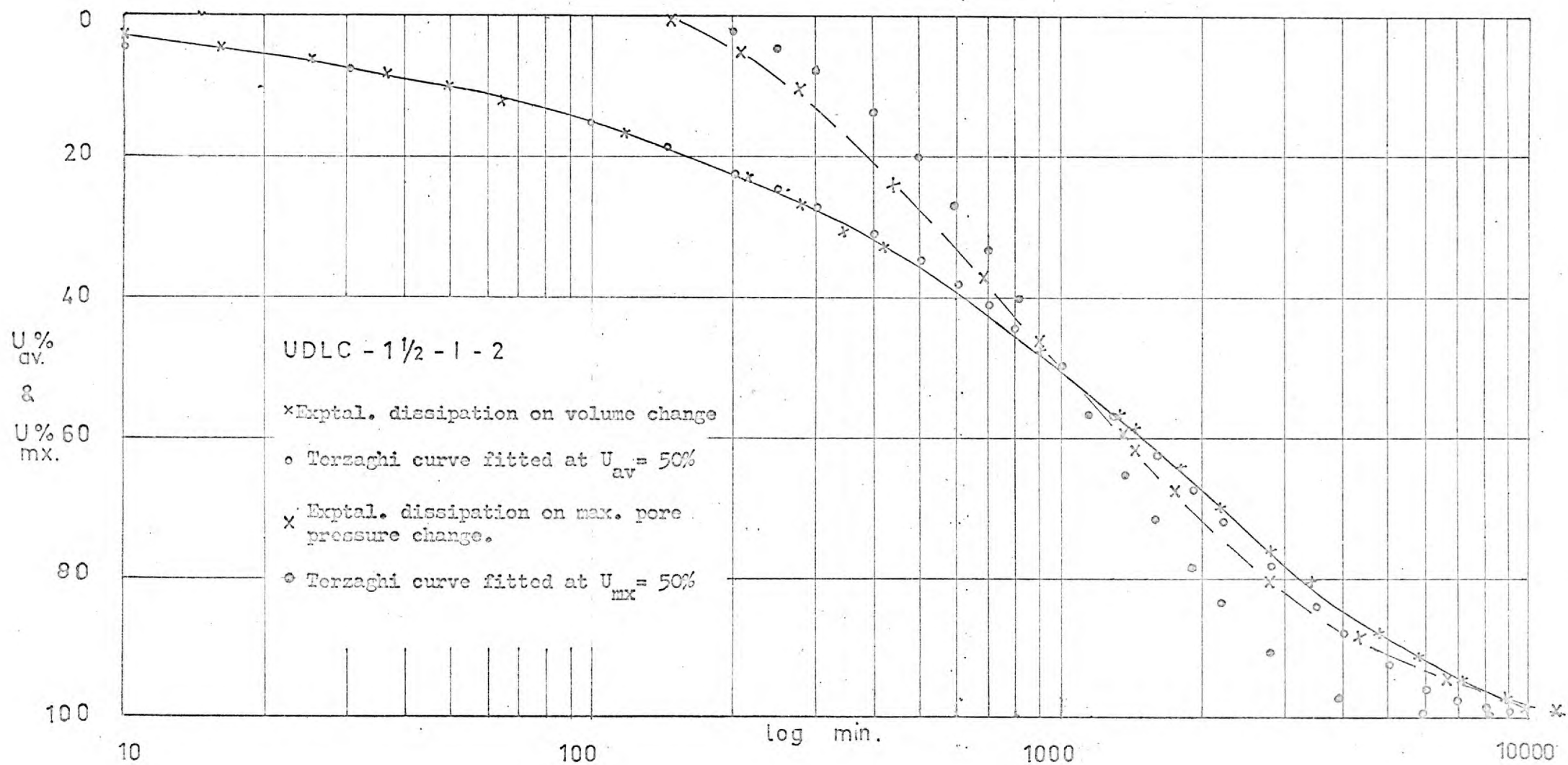
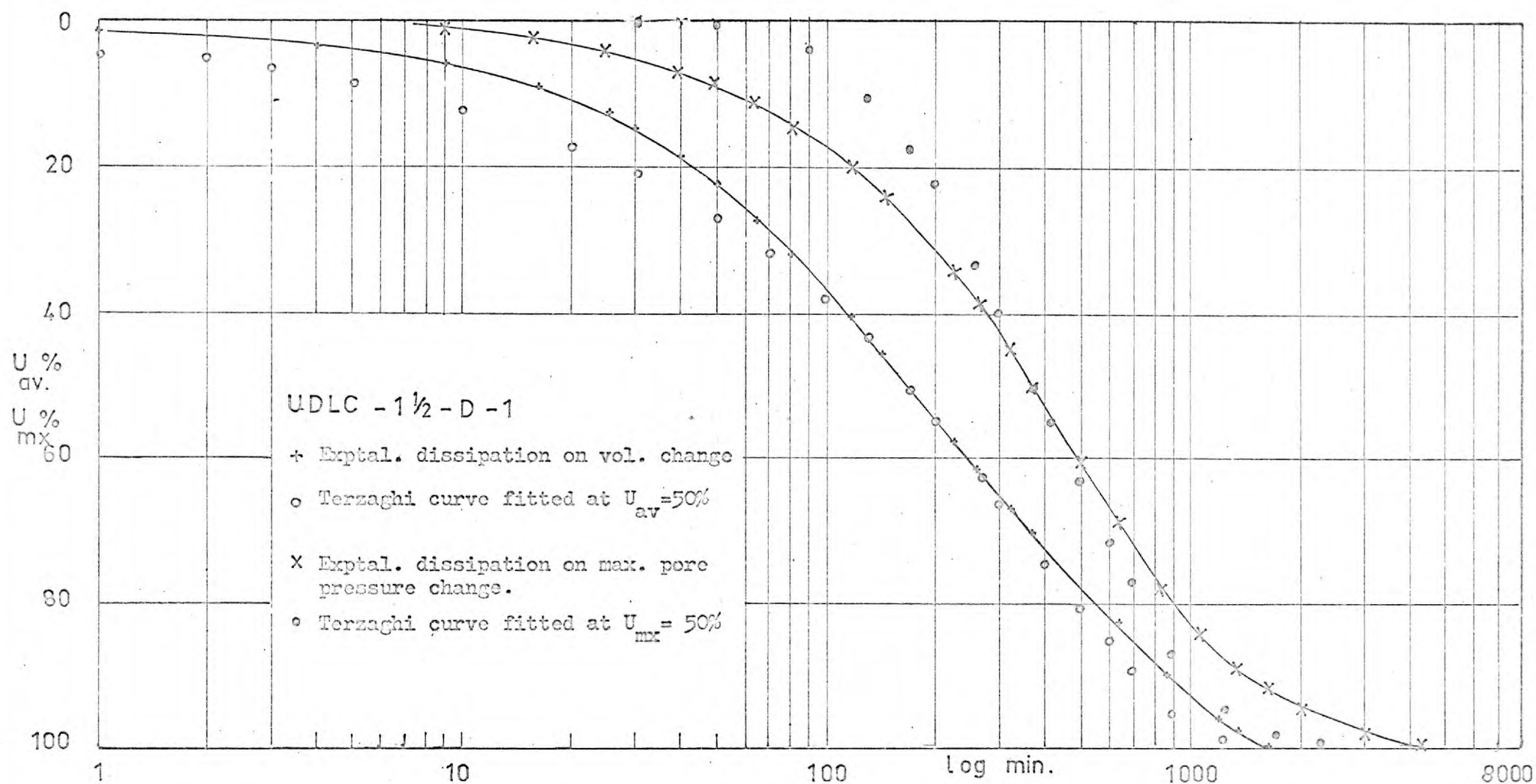
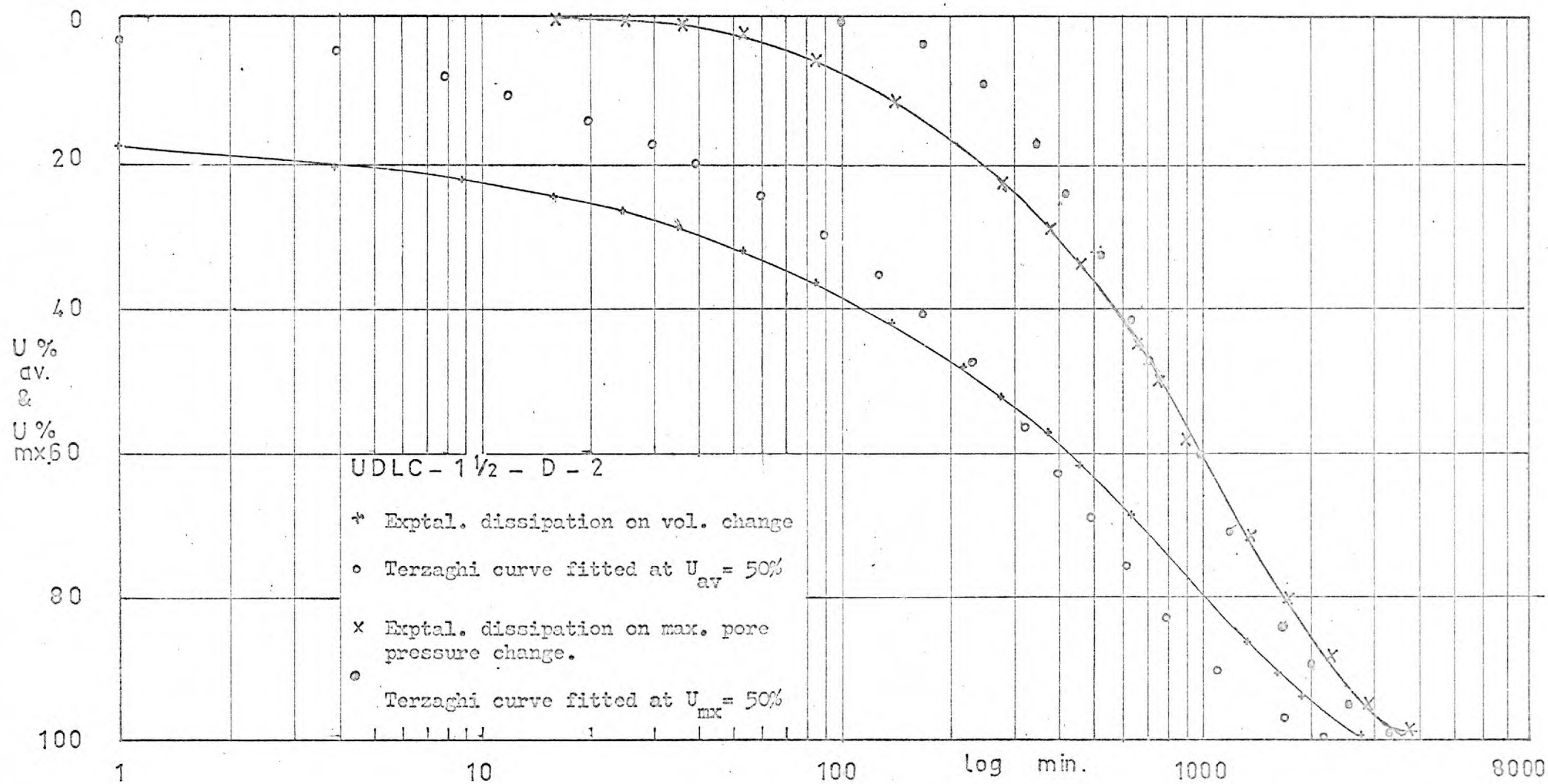


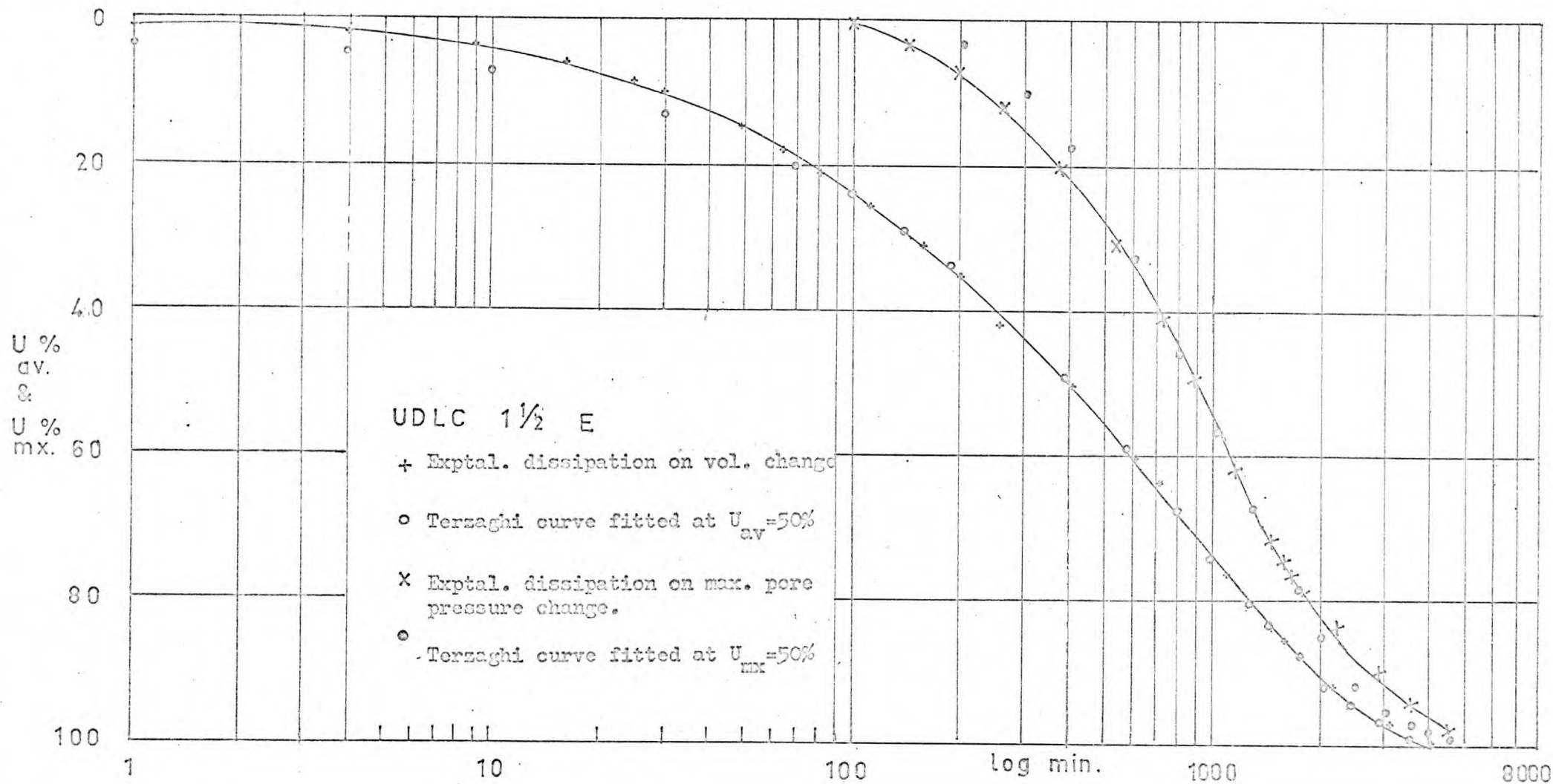
FIG. 10-19 : DEGREE OF DISSIPATION BASED ON AVERAGE VOLUME CHANGE AND MAXIMUM PORE PRESSURE CHANGE AGAINST LOG TIME.



DEGREE OF DISSIPATION BASED ON AVERAGE VOLUME CHANGE AND MAXIMUM PORE PRESSURE CHANGE AGAINST LOG TIME.



DEGREE OF DISSIPATION BASED ON AVERAGE VOLUME CHANGE AND MAXIMUM PORE PRESSURE CHANGE AGAINST LOG TIME.



DEGREE OF DISSIPATION BASED ON AVERAGE VOLUME CHANGE AND MAXIMUM PORE PRESSURE CHANGE AGAINST LOG TIME.

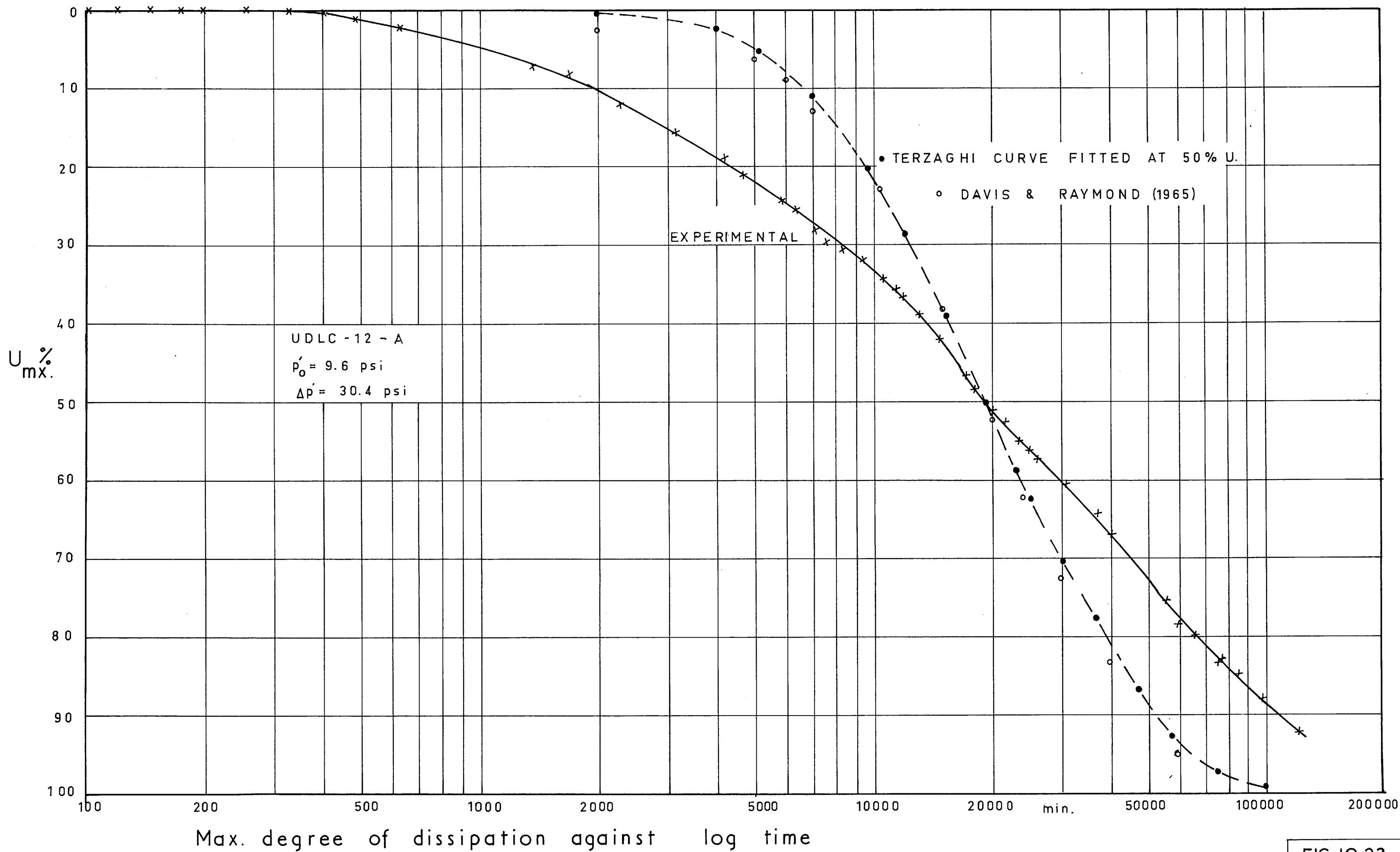


FIG. 10.23

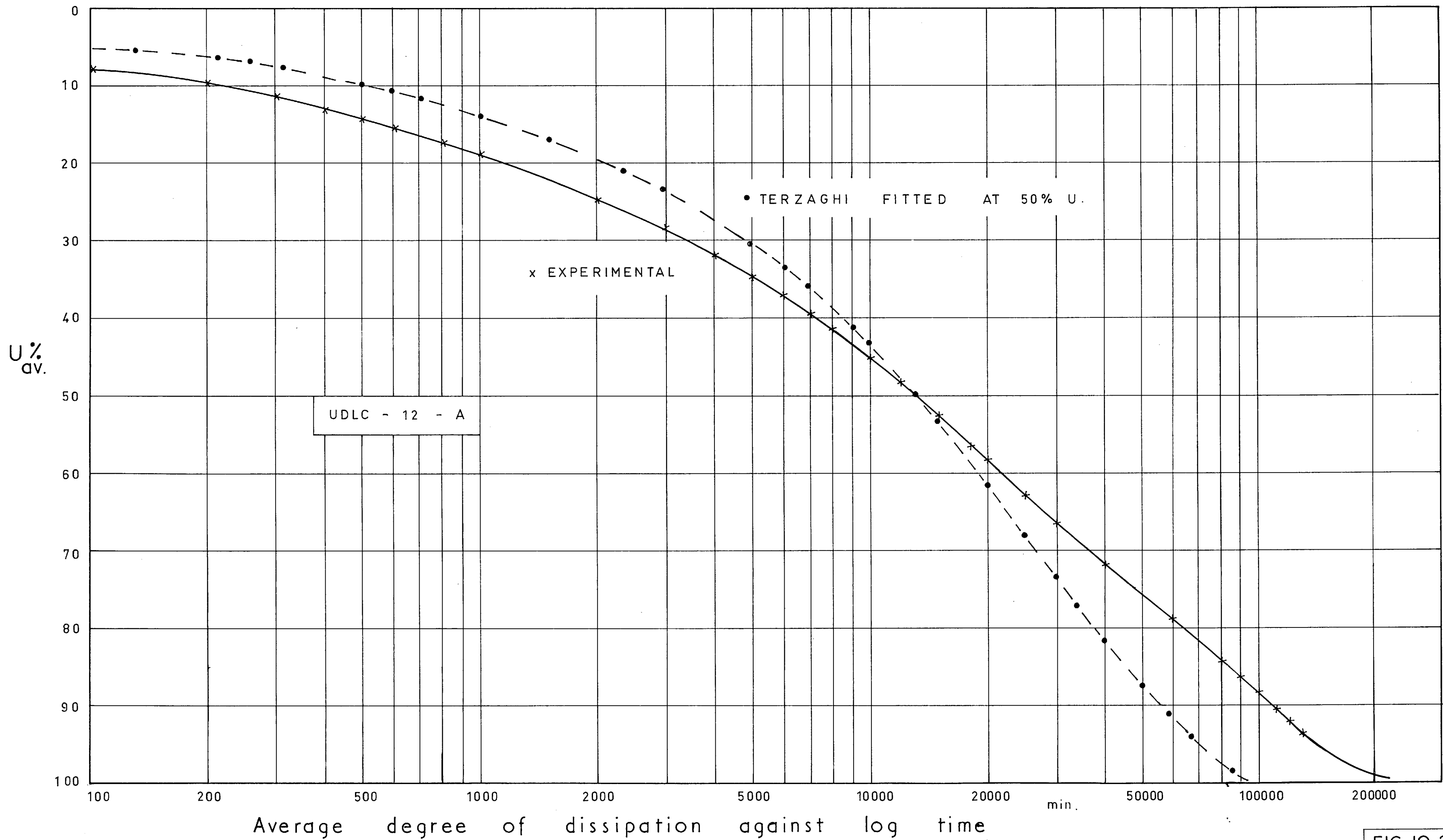


FIG. 10.24

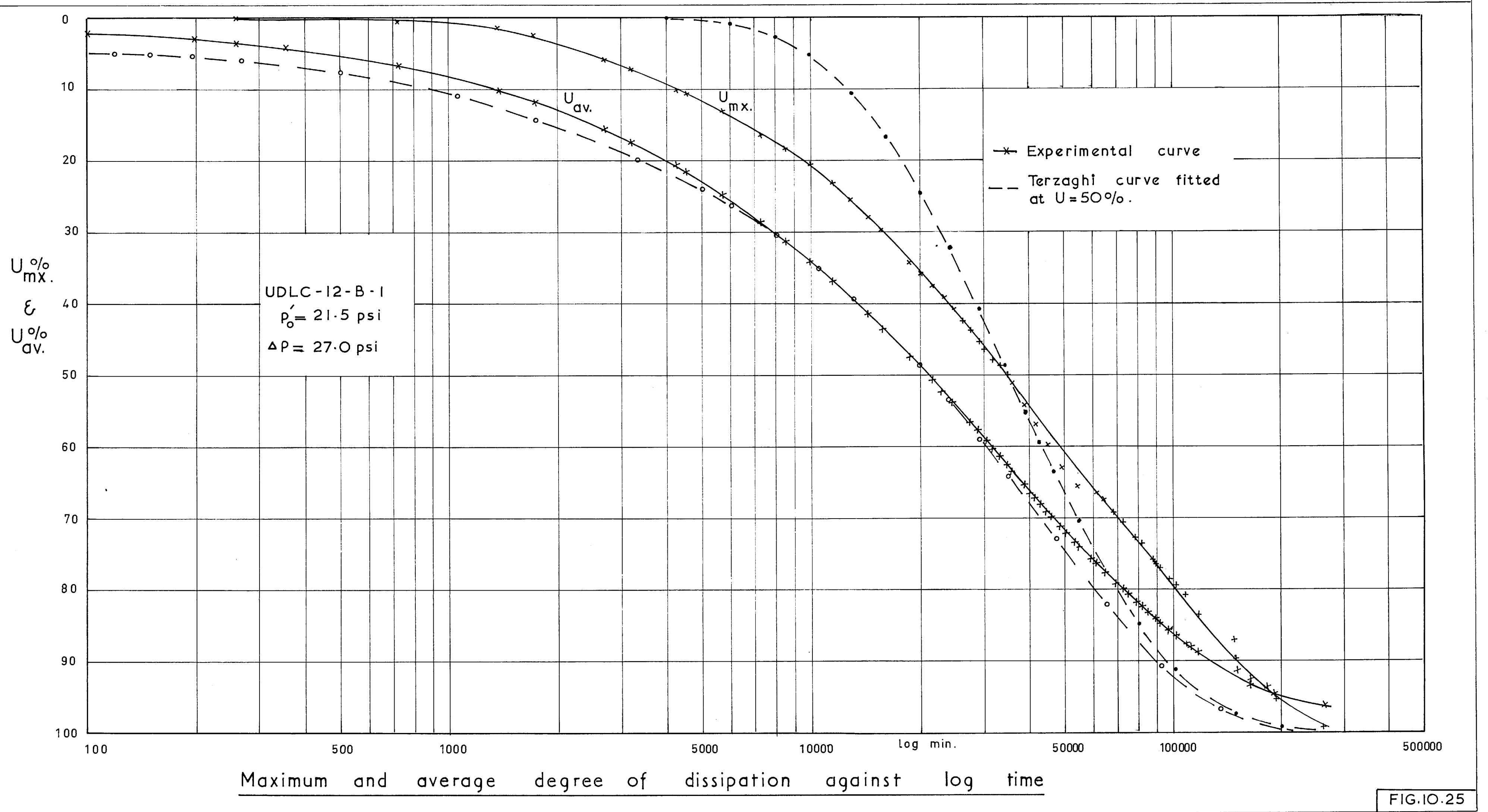
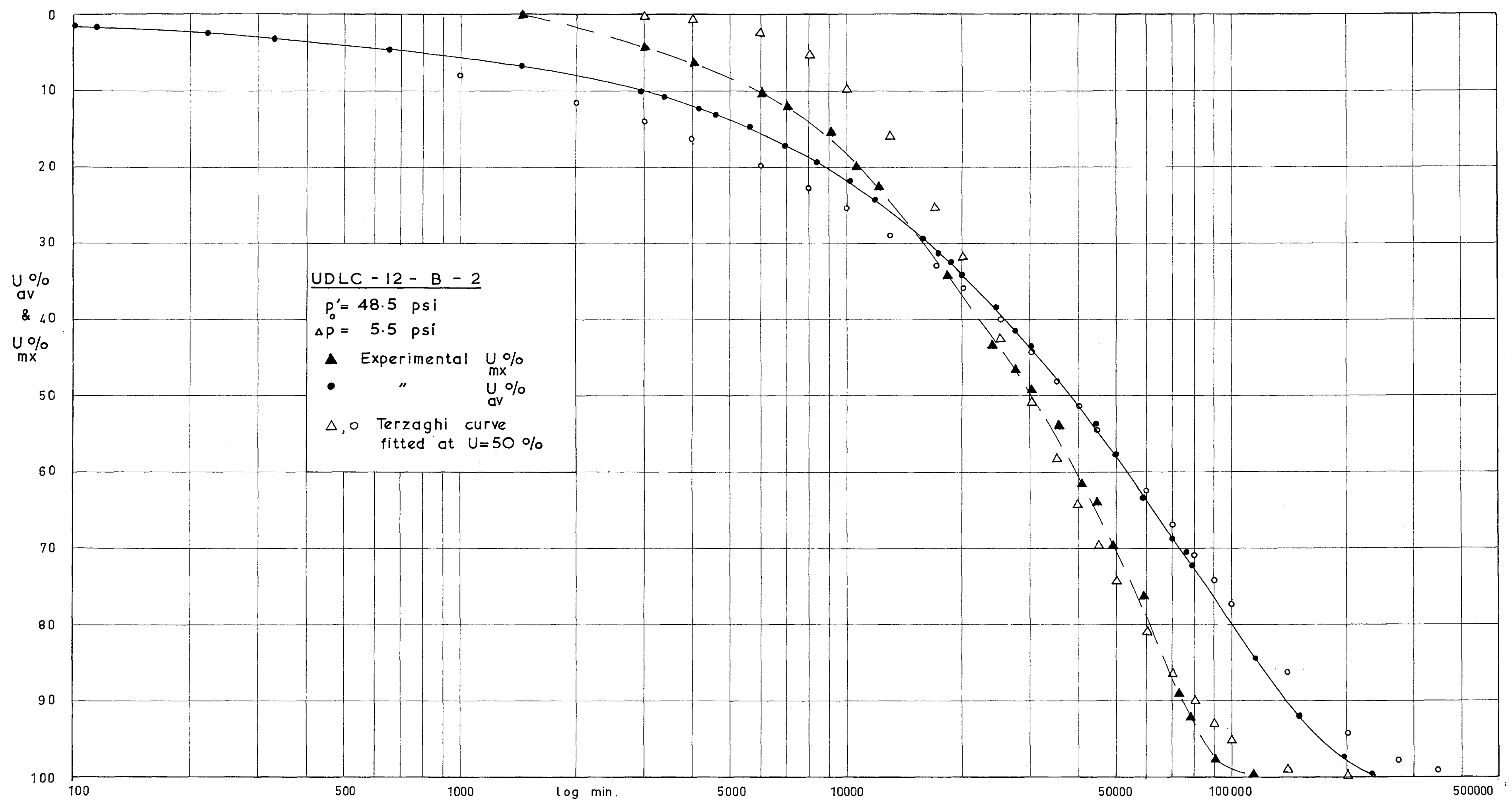
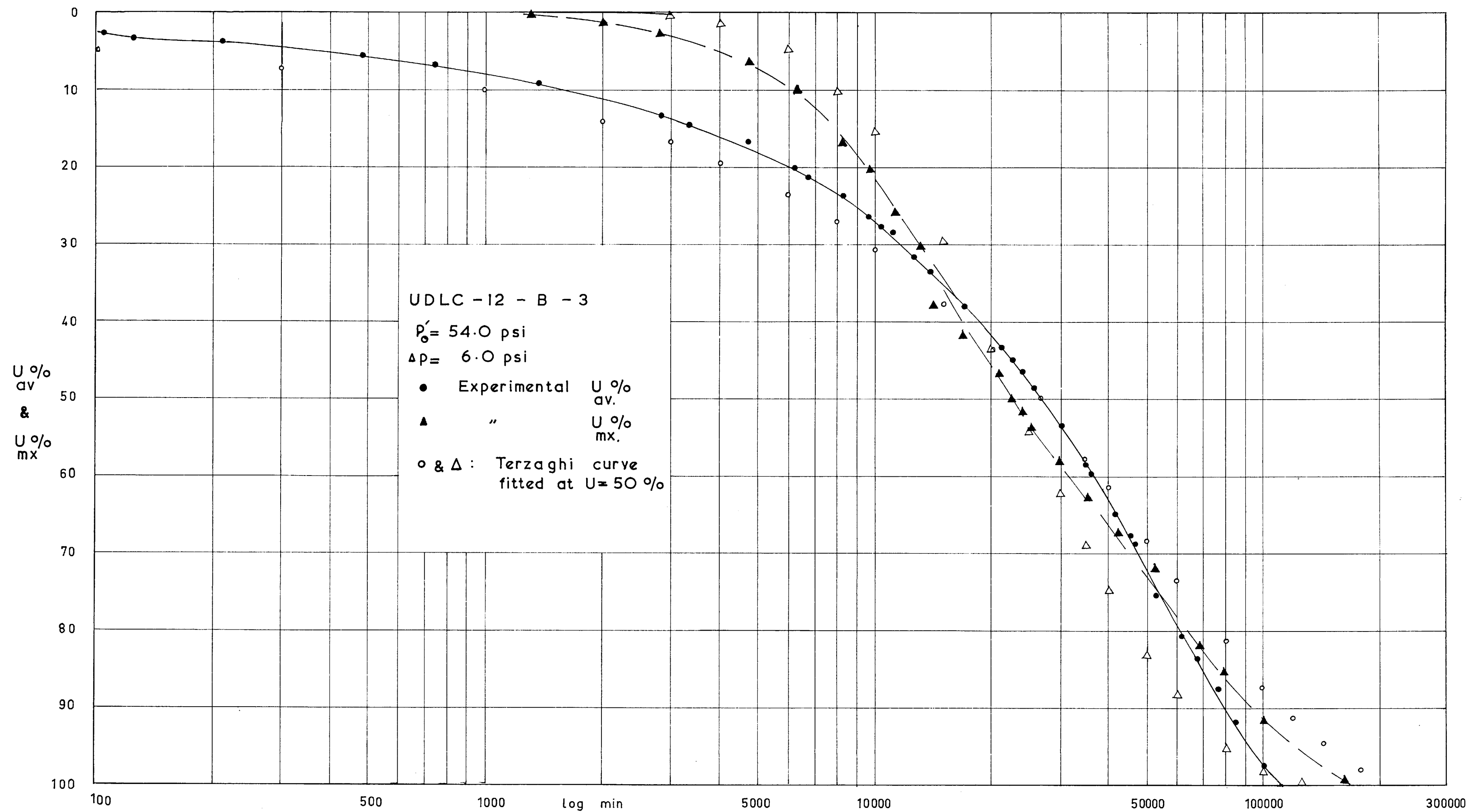


FIG.10.25

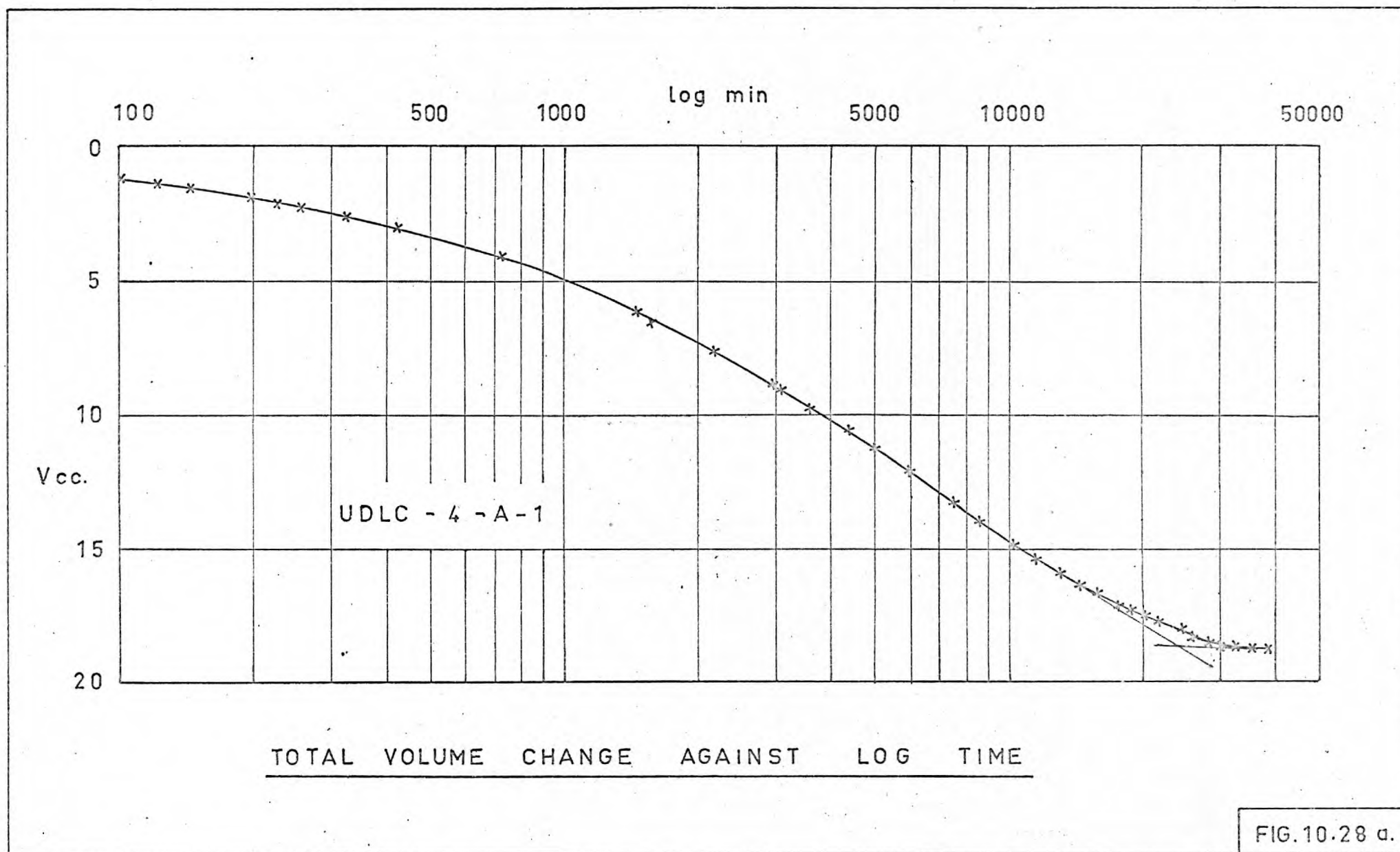


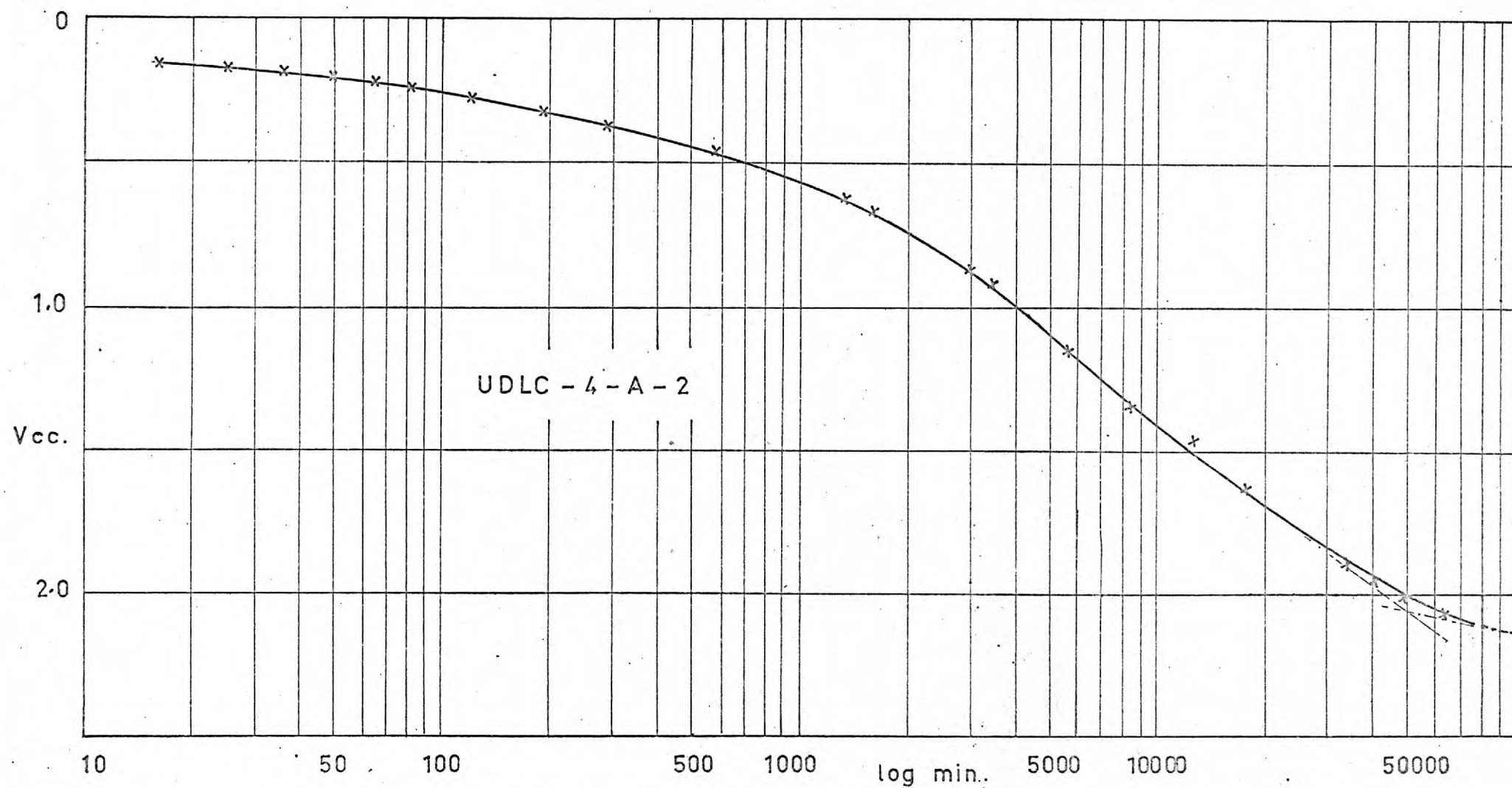
AVERAGE AND MAXIMUM DEGREE OF CONSOLIDATION AGAINST LOG TIME



AVERAGE AND MAXIMUM DEGREE OF CONSOLIDATION AGAINST LOG TIME

FIG. 10.27





UDLC - 4 - A - 2

TOTAL VOLUME CHANGE AGAINST LOG TIME

FIG.10.28 b.

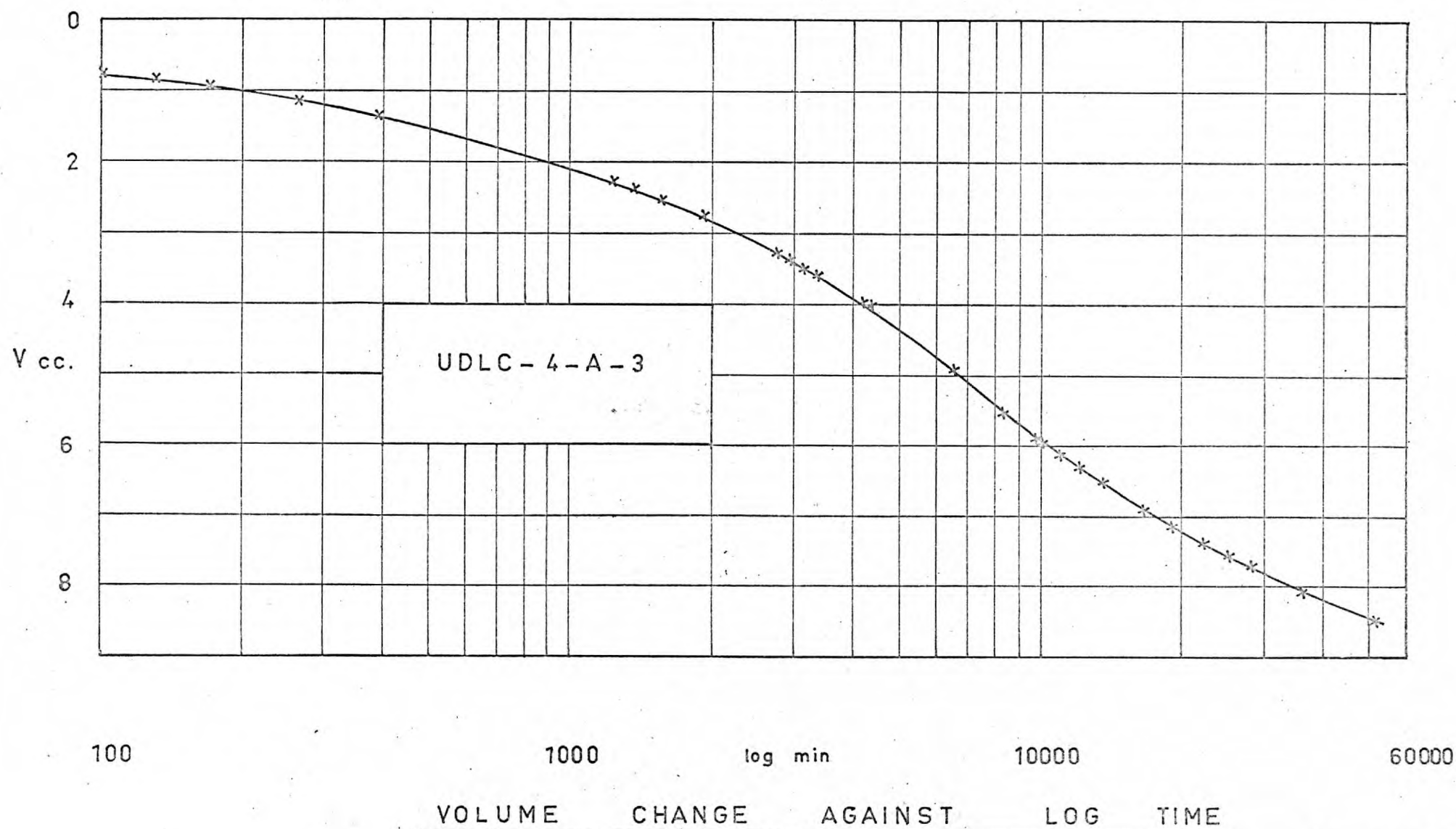
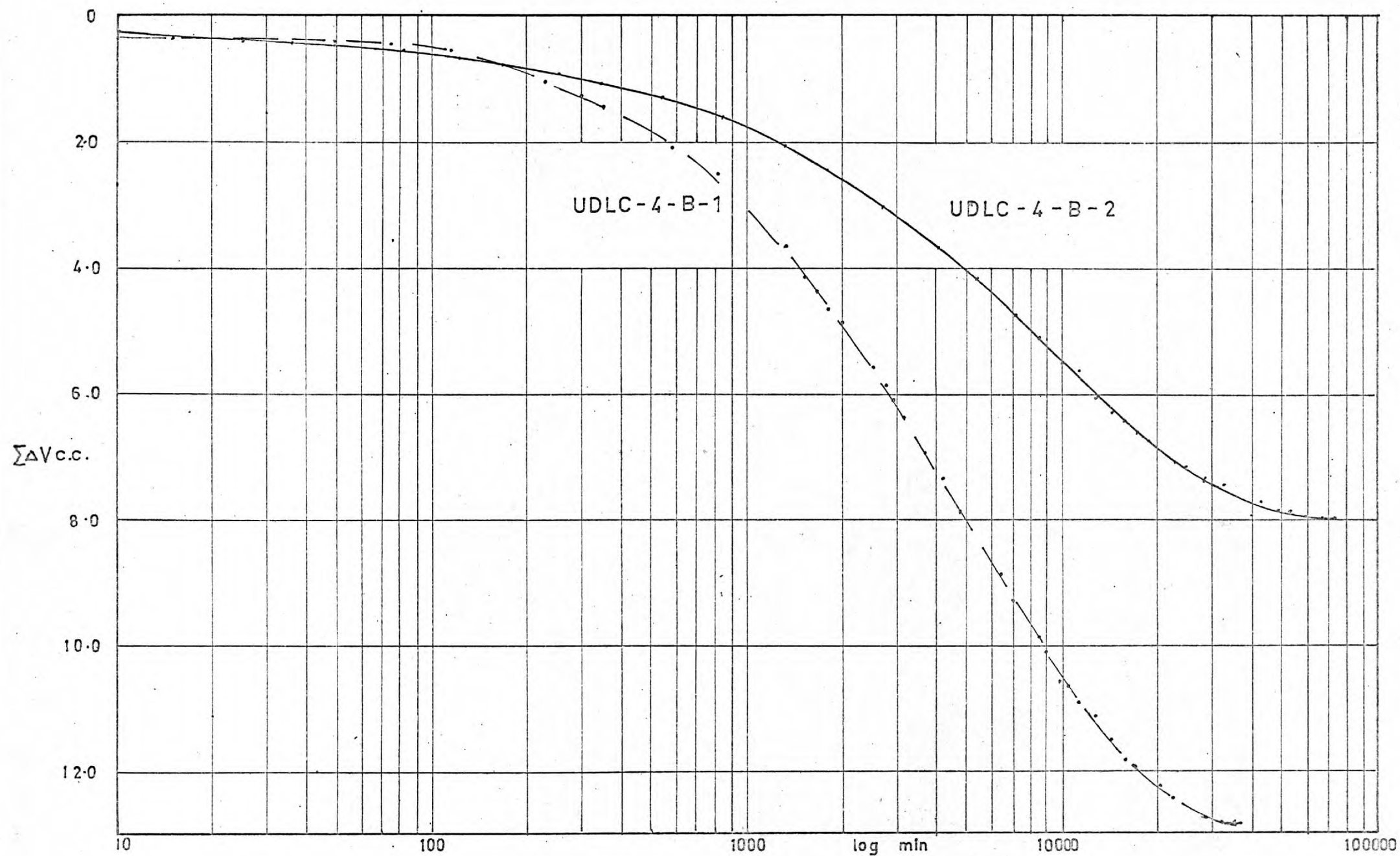
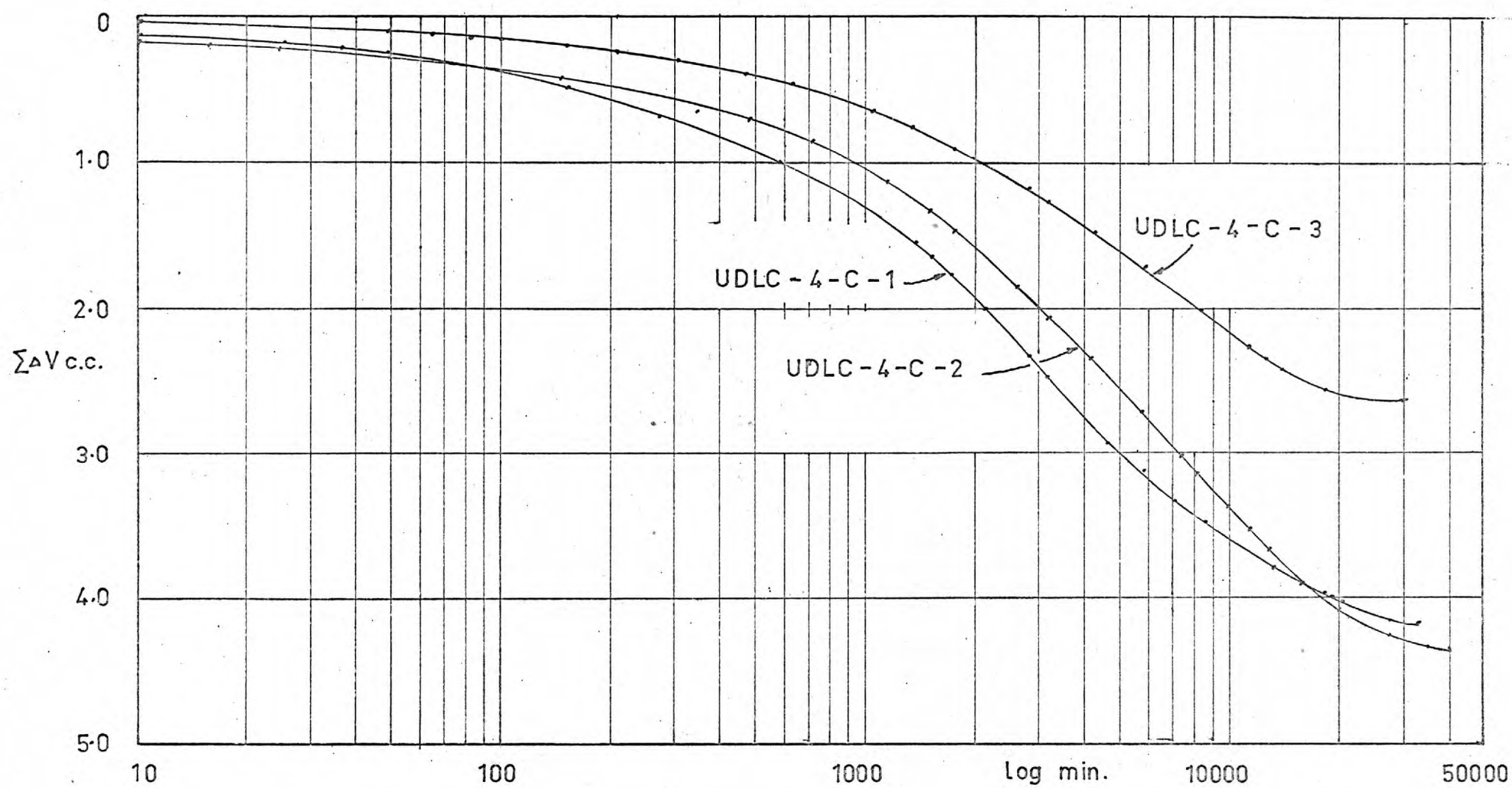


FIG.10.28 c.



TOTAL VOLUME CHANGE AGAINST LOG TIME.

FIG.10.29 d.



TOTAL VOLUME CHANGE AGAINST LOG TIME.

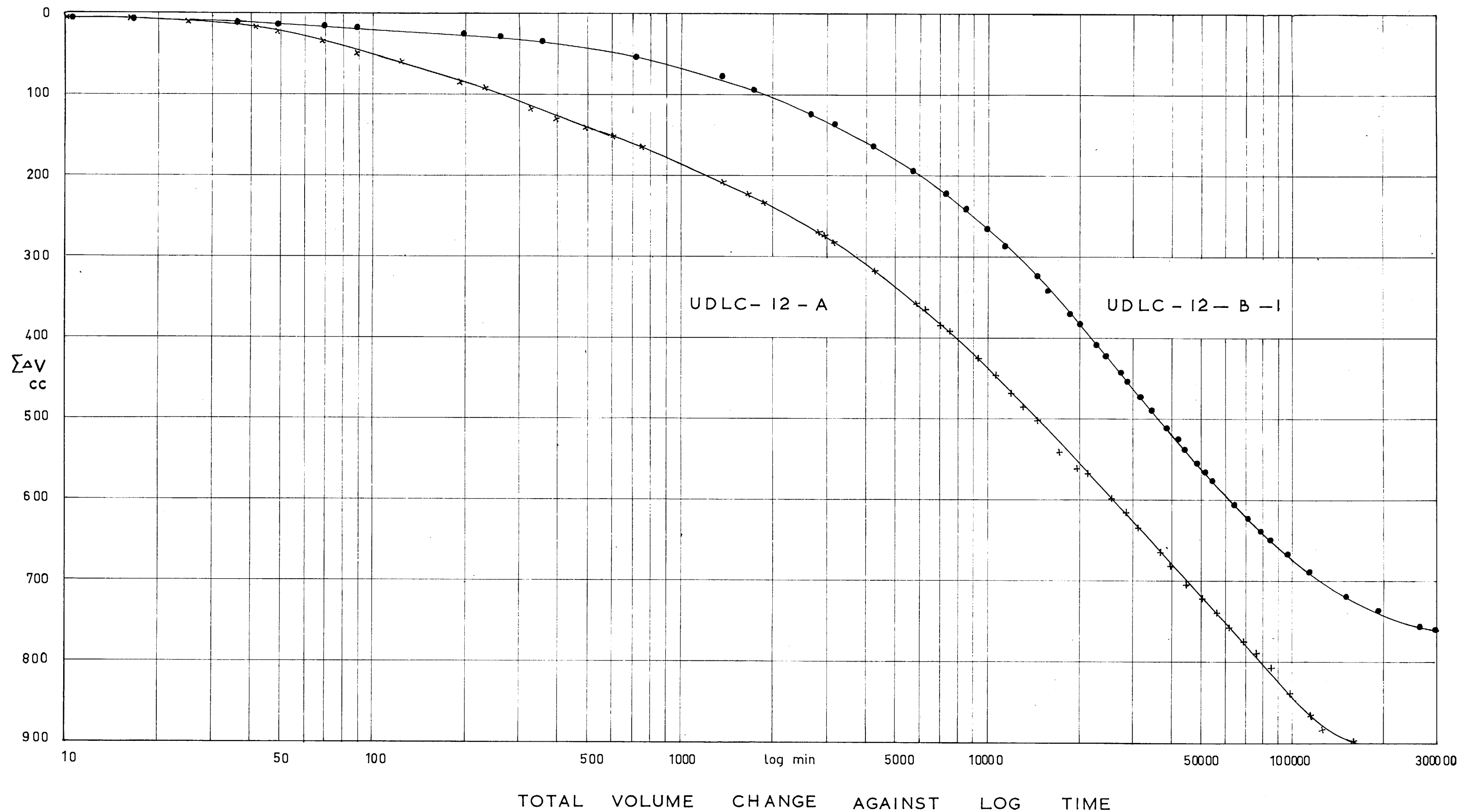


FIG.10.30a

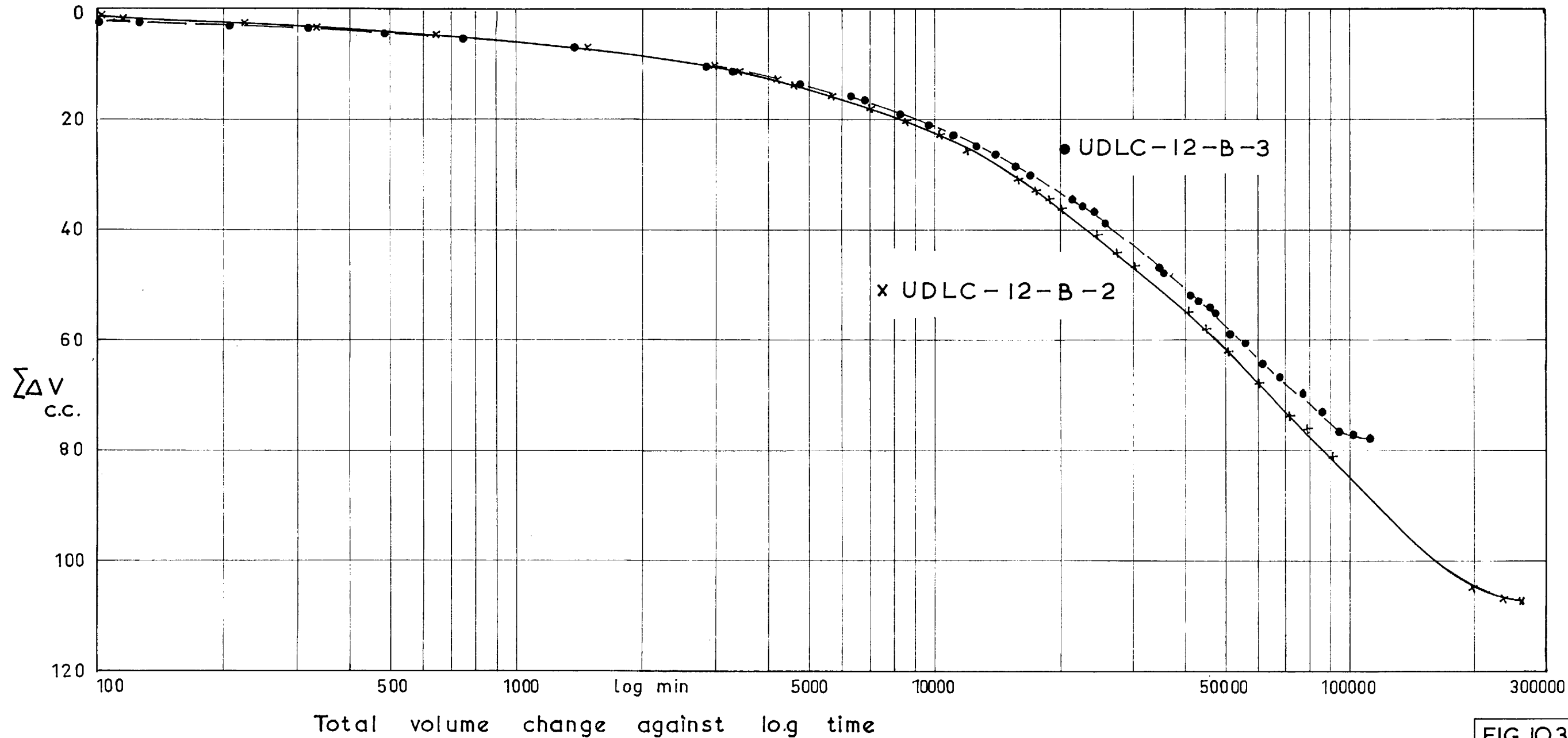
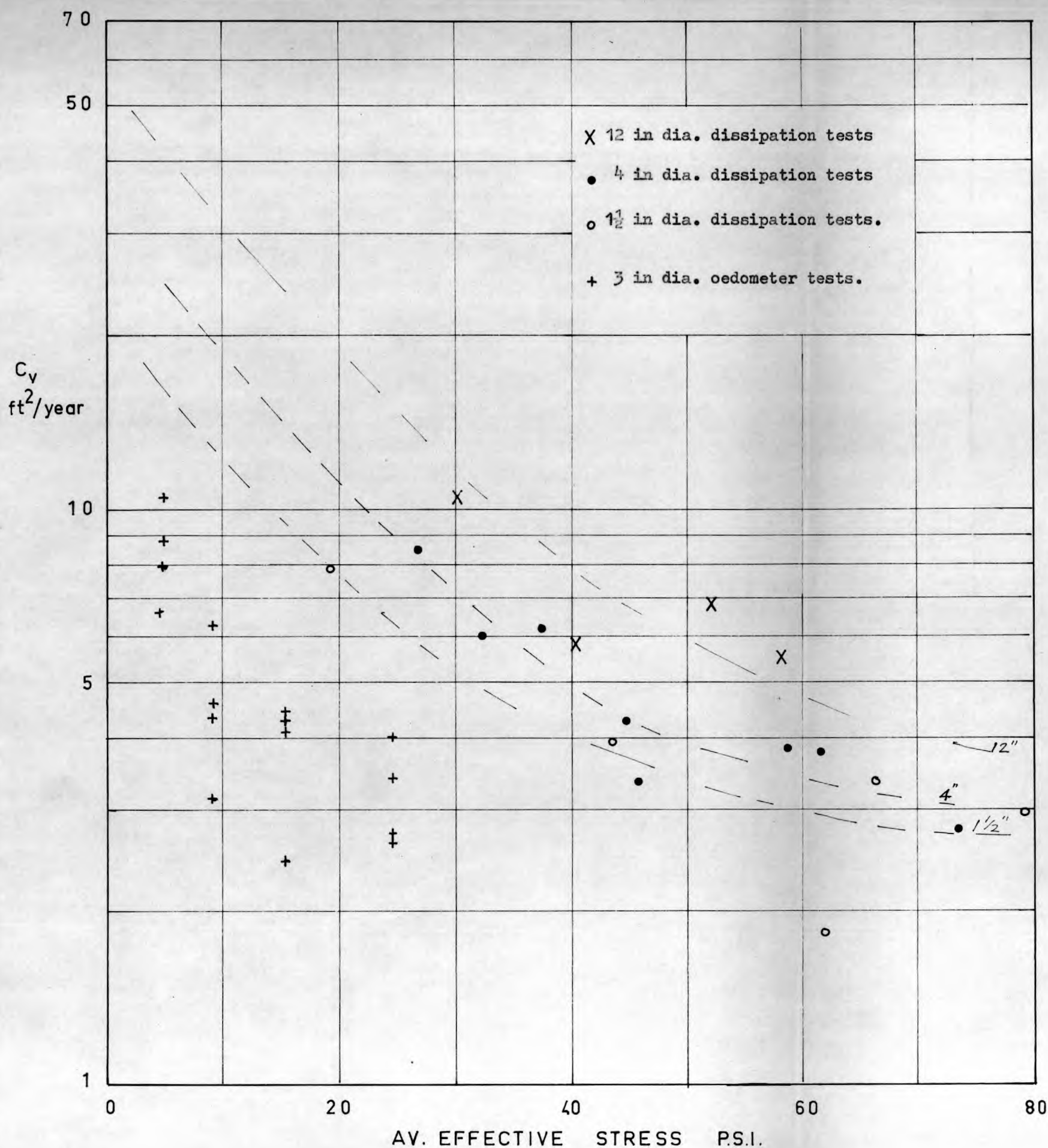
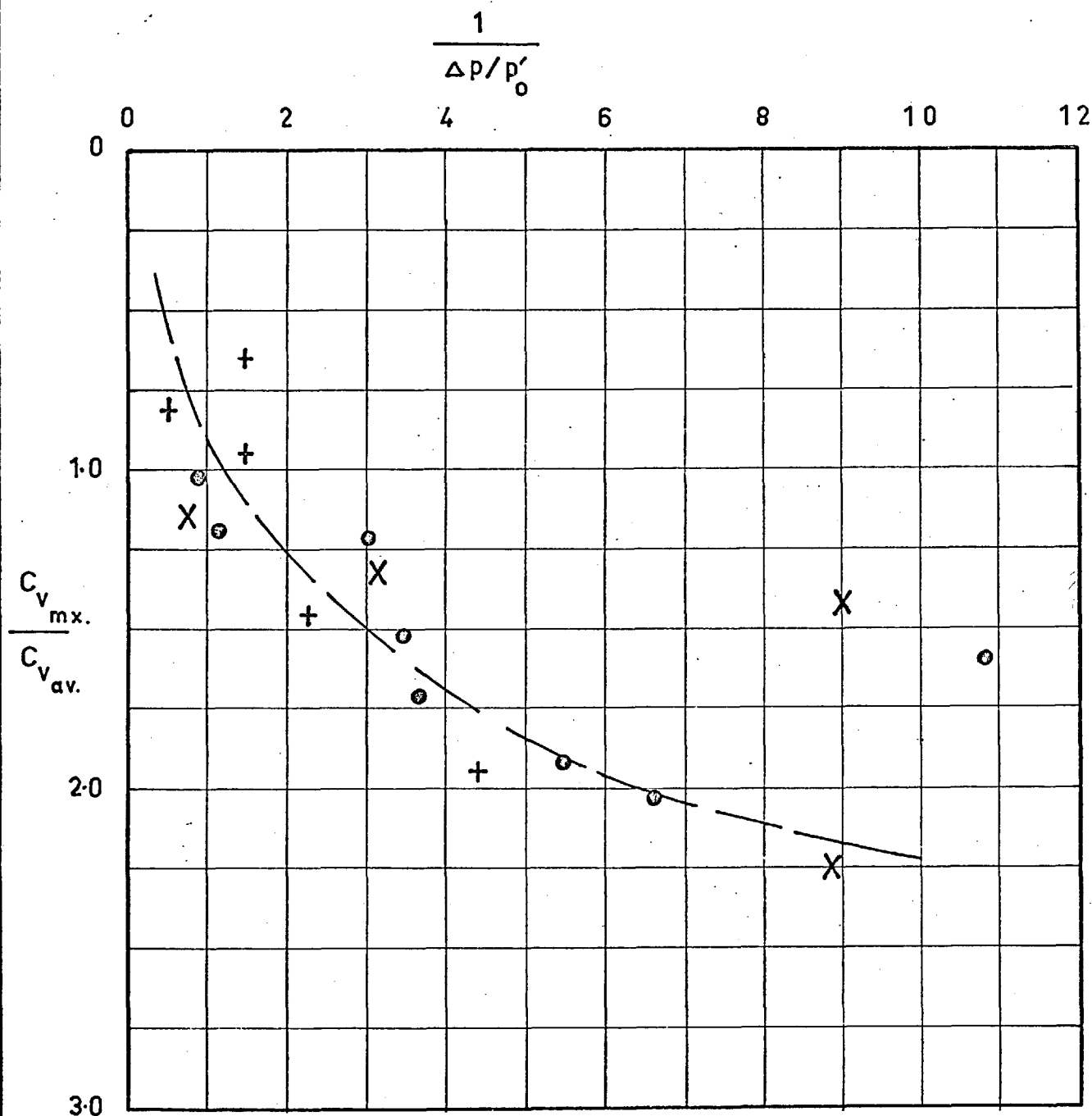


FIG.1030b

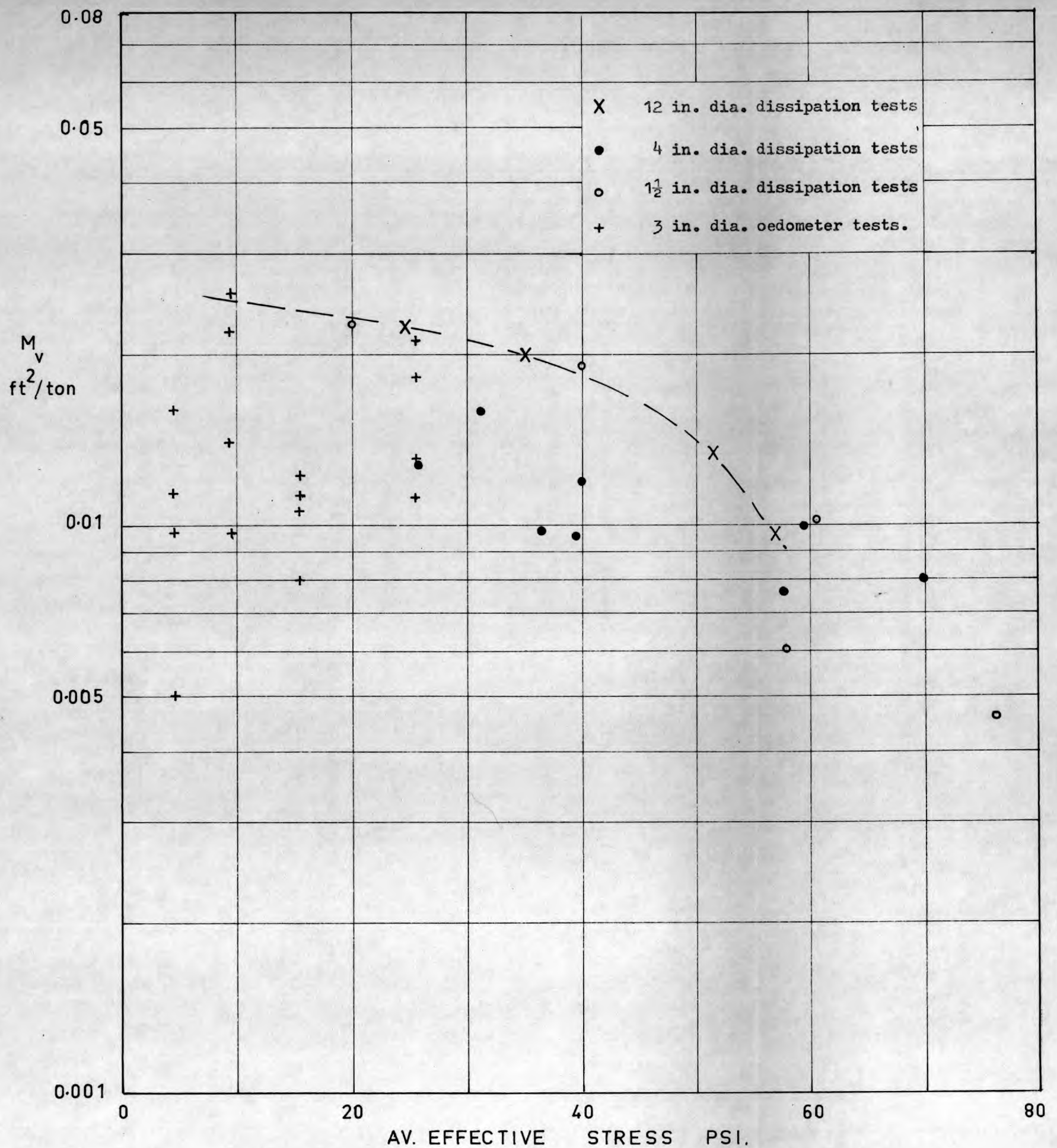


GRAPH OF COEFFICIENT OF CONSOLIDATION AGAINST AVERAGE EFFECTIVE STRESS.

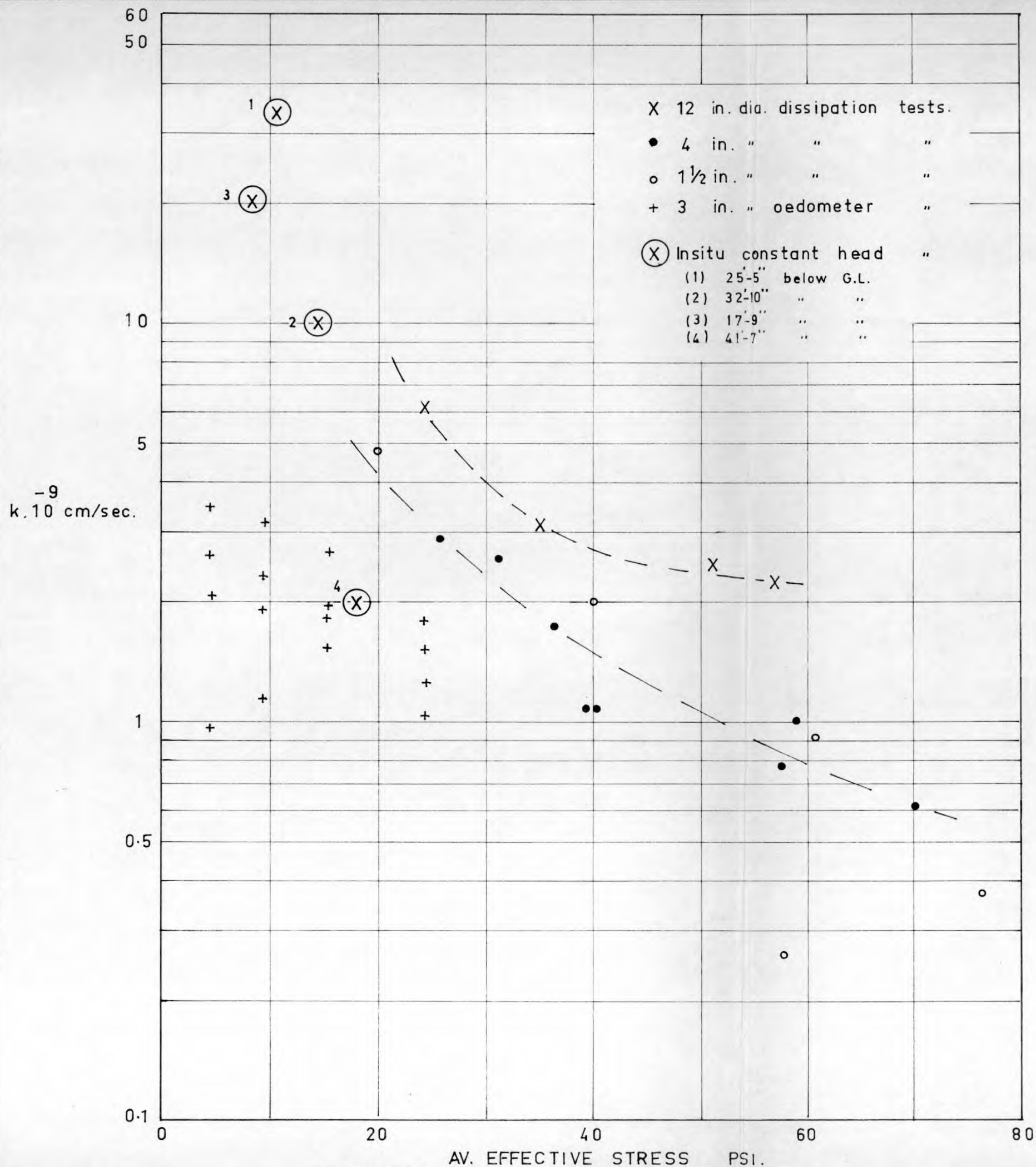


X 12 in. dia. dissipation tests.
 o 4 " " " "
 + 1 1/2 " " " "

EFFECT OF PRESSURE INCREMENT RATIO ON $\frac{C_{v_{mx.}}}{C_{v_{av.}}}$



COEFFICIENT OF VOLUME COMPRESSIBILITY AGAINST AVERAGE EFFECTIVE STRESS.



PERMEABILITY AGAINST AVERAGE EFFECTIVE STRESS.

CHAPTER 11

Tests on boulder clay

11.1 Introduction

As part of the investigations for the M6 Motorway, a 6 meters high trial embankment was constructed and instrumented to evaluate the effectiveness of horizontal drainage blankets in boulder clay. The boulder clay was a light brown sandy clay and the grading curve of material passing BS 3-in. is shown in fig. 11.1. (The clay contained boulders up to 1 meter in size.) The index tests on clay passing BS 36 sieve showed a liquid limit of 26%, plastic limit of 19% and a very small clay fraction of 7%. An interesting feature was the high moisture content at which boulder clay fill was placed. According to the Ministry of Transport specification (1963), the placement moisture content for this boulder clay should be about 9 - 11%; however, under the construction conditions, the actual bulk water content could rise to about 15%. A sample of excavated boulder clay was brought to the laboratory and pore pressure dissipation and constant head permeability tests were conducted on 4 in. x 4 in. diameter sample in a triaxial cell.

Laboratory tests

The laboratory tests were conducted on material passing BS 3/8 in. sieve size. The grading curve of the

test material is also shown in fig 11.1.

The dissipation tests were conducted in a triaxial cell in the same manner as described earlier for 4 in. diameter tests on London clay in chapter 10. However, at the completion of each dissipation stage, a constant head test was performed before commencing the next dissipation stage.

The permeability may be directly measured by maintaining a constant differential head across the sample and by applying Darcy's Law when steady state flow is achieved. Tan (1968) obtained solutions for C.H.T. in laboratory and showed that the rate of flow varies linearly with $1/\sqrt{t}$ until at approximately $1/\sqrt{t} = \sqrt{\pi}$, it turns towards the steady state value, q_{∞} . From such a plot permeability can be determined by the following expression:

$$k = \frac{q_{\infty} \gamma_w H}{A \Delta u} \quad (11.1)$$

where A denotes area of cross-section of the sample,

H denotes length of drainage path,

Δu denotes constant differential head across the sample,

γ_w denotes unit weight of water.

As in the case of in-situ constant head permeability tests (chapter 9), it is convenient to plot rate of flow against $2/(\sqrt{t_1} + \sqrt{t_2})$ where t_1 and t_2 are times before and after the increment during which the flow is measured.

11.2 Layout of apparatus

The layout of the apparatus is shown in fig 11.2. The sample enclosed by rubber membrane and top and bottom coarse porous discs rests on the pedestal of a tri-axial cell base. The pore pressure at the base of the sample is measured by a null indicator. The drainage lead from the top of the sample is connected to a 5 cc. capacity volume gauge which measures the change in volume of the sample during dissipation stage. One limb of the null indicator which is connected to the base of the sample is also connected to a 5 cc. capacity volume gauge while the other limb is connected to the hand ram. In fig 11.2, the pressure line to the cell is also connected to a volume gauge. This arrangement is used only when the test is conducted in stages at different cell pressures. However, in the present series of tests only one constant cell pressure was used for all the stages. Since very small pressures were used in the tests, the bourdon pressure gauge could not be relied upon to measure those accurately. Hence all pressures were read on a mercury manometer as shown in the figure.

11.3 Test procedure

To simulate the field conditions as closely as possible, the water content at which remoulded boulder clay would have an undrained strength of 2 psi. was determined by carrying out unconfined compression tests on $1\frac{1}{2}$ in. diameter, 3 in. high samples. The moisture

content was found to be 18.5%, and the soil was remoulded at this moisture content. The test sample was then tamped in a 4 in. diameter split mould and mounted in the cell. A constant cell pressure of 20 psi. was maintained throughout the test.

After the pore pressure in the sample has built up to a steady value under undrained conditions, a constant back pressure, p_1 is applied through the upper drainage lead and dissipation stage is commenced. The volume change in the sample is measured on volume gauge V_2 , and the pore pressure are recorded. Note that during this stage, the volume gauge V_1 , connected to the null indicator is disconnected from the circuit. The permeability test is conducted after completion of the dissipation stage (when p_1 is the pore pressure throughout the sample). An effective stress increment of 4 psi. was used for all consolidation (or swelling stages) while the permeability tests were conducted under a constant pressure differential of 2 psi.

The null indicator is now disconnected from the circuit and the volume gauge V_1 is brought in the circuit. A constant pressure differential of 2 psi. across the sample is maintained by keeping the same back pressure (p_1) at the bottom of the sample, while the upper end is subjected to a backpressure of $(p_1 - 2)$ psi. Hence the flow of water takes place from bottom to top of the sample and the inflow and outflow measurements are

taken on volume gauges V_1 and V_2 respectively. After the flow has approximated to steady state flow, the back pressure on top of the sample is altered to the same value as the bottom back pressure, (p_1-2) psi., in order to establish uniform pore pressure in the sample. The next dissipation stage can now be commenced by disconnecting V_1 ; bringing the null indicator in the circuit and applying a back pressure of (p_1-6) psi. through the top drainage lead.

In the present work, series of pore pressure dissipation tests and constant head permeability tests were conducted on the consolidation, swelling and reconsolidation cycles to compare the laboratory consolidation behaviour with insitu tests.

11.4 Results and discussion

The results have been tabulated in Table 11.1. The values of coefficient of consolidation have been calculated from standard Terzaghi theory fitted at 50% consolidation time, while m_v is calculated from volume change during pore pressure dissipation stage.

Fig 11.3 shows the variation of C_v , m_v and k with effective stress on initial consolidation. The coefficient of consolidation based on pore pressure change increases almost linearly with effective stress on a log - log plot, as m_v (and k) decrease with effective stress. It would appear from the figure, that the rate of increase in C_v

would become small after 20 psi. as the rate of decrease in m_v becomes progressively smaller. Figures 11.4 and 11.5 show the variation in consolidation parameters with effective stress on swelling and reconsolidation. There is remarkably little change in permeability with effective stress on swelling and reconsolidation cycle. The laboratory values of permeability have been compared with those from in-situ constant head tests in fig 11.6 and show that very close agreement is obtained. This indicates that in cases where soil is fairly uniform and apparently structureless, it is possible to assess insitu permeability reasonably accurately from laboratory constant head permeability tests. The values of insitu C_v as deduced from piezometer readings lie 2-3 times above the laboratory C_v values on first loading (see fig 11.7). This may be accounted for by the presence of high stone content (about 40% more) in the fill in the embankment (Bishop and Al-Dhahir, 1969).

Bishop and Al-Dhahir (1969) have compared some cases where in situ and laboratory tests have been conducted to determine k and C_v . For the boulder clay foundation of Selset reservoir embankment, the field C_v was about 3 times the laboratory value as determined by 4 in. diameter dissipation tests. The value of k by direct measurement in laboratory was 0.43×10^{-8} cm/sec. or, omitting one high value, 0.84×10^{-8} cm/sec. Very good correlation was also obtained for the fill material between C_v from dissipation test and field performance

results (see also Bishop and Vaughan, 1962). Also in-situ CH tests in the fill gave $k = 1.2 \times 10^{-8}$ cm/sec., while $k = 1.1 \times 10^{-8}$ cm/sec in same stress range was directly measured in the laboratory. For the puddle clay core at Selsot embankment, $k = 0.74 \times 10^{-8}$ cm/sec and 0.84×10^{-8} cm/sec were obtained for consolidating and swelling stages respectively from in situ CH tests while a value of $k = 0.77 \times 10^{-8}$ cm/sec was directly measured in the laboratory. Similar data on boulder clay from Balderhead Dam and Diddington Dam is shown in Table 11.2 (after Bishop and Al-Dhahir, 1969) from which the general conclusion may be drawn that provided the material is structureless and uniform, very good correlation can be obtained from laboratory and insitu determinations of k and C_v . The agreement in C_v values will improve if the boulder clay contains high clay content and less stone content. This is not surprising in view of the fact that large sized stone fraction has to be excluded in most laboratory tests. For 4 in. dia. laboratory tests, $\frac{3}{8}$ in. is a maximum particle limit. The results of permeability of boulder clay may be contrasted with those obtained for London clay (chapter 10) where in situ permeability values, especially at shallow depths, were much higher than those deduced from laboratory dissipation tests, because London clay shows high fissure intensity at shallow depths.

The coefficient of consolidation C_v is not only a function of the effective stress but also structure

(expressed in terms of void ratio) as shown by figures 11.3 - 11.5. It would appear that m_v is especially sensitive to stress history; for example, at effective stress of 10 psi. $m_v = 0.03$, 0.0054 and $0.0102 \text{ ft}^2/\text{ton}$ are obtained for initial consolidation, swelling and reconsolidation curves respectively. Since k from direct laboratory measurements is relatively insensitive to effective stress or stress history, a much higher value of C_v would be expected on reconsolidation than on virgin consolidation at the same effective stress. This can be clearly seen from figures 11.3 and 11.5. In these figures, the value of C_v from direct measurement of permeability and m_v from volume change ($C_v = k/m_v \gamma_w$) has also been plotted. This value of C_v is lower than that calculated from standard Terzaghi theory fitted at $t = 50\%$ for swelling and reconsolidation tests while for the initial consolidation tests, it is lower only after normal stress of 10 psi. This would imply that C_v as determined from pore pressure dissipation and Terzaghi theory would overestimate the rate of consolidation of boulder clay by a small amount (about 2 times in the stress range considered). This effect may be especially important for swelling tests because coefficient of swelling is much greater than the coefficient of consolidation.

Table 11.1

Results on remoulded boulder clay (116)

Test	Pressure	C_v ft ² /yr		m_v	$k \times 10^{-8}$
	range, psi.	mx.	pp.	Av. vol. change	cm/sec. measured
Initial consoli- dation	0-4	14.6	19.1	0.097	
	4-6				3.54
	6-10	29.6	33.7	0.033	
	10-12				2.65
	12-16	41.9	39.0	0.027	
	16-18				1.81
Swelling	18-14	317	490.8	0.0058	
	14-12				2.72
	12-8	436	332.7	0.0054	
	8-6				2.80
	6-2	239	100.5	0.011	
	2-0				4.12
Re-consol- idation	0-4	22.2	399	0.037	
	4-6				3.61
	6-10	171	140	0.0160	
	10-12				3.19
	12-16	542	125	0.0045	
	16-18				2.99

Table 11.2

Comparison of laboratory tests, insitu tests and field

Site	Zone	Index values			Laboratory test values *1			
		LL	PL	CF	C_v ft ² /yr		k cm/sec. x10 ⁻⁸	
Selset	Foundation	32	13	24	25	dh (40)	0.67 ^{*3}	(40)
							0.43 ^{*3}	(30)
							0.38 [*] h	(77)
	Fill	29	15	20	24	d (40)	0.67 ^{*3}	(40)
							1.1 ^{*3}	(12)
	Puddle core	29	15	20	16	d (40)	0.67	(40)
							0.77	(30)
Balderhead	Fill	28	14	18	25-170	d (46)	0.56 ^{*4}	(53)
							0.64-0.43	(5)
Diddington	Foundation (Boulder clay and Oxford clay)	58	20	38	12.0-4.3	d (40)	2.2-0.17 [*]	(40)
					17	o (70)		
					9-5	t(c.20)		
	Fill	58	20	38	6	d1½(30)	0.4 [*]	(30)
M6 (Kendal)	Fill	26	19	7	15	d (3)	2.1	(5)
					30	d (9)	1.6	(11)

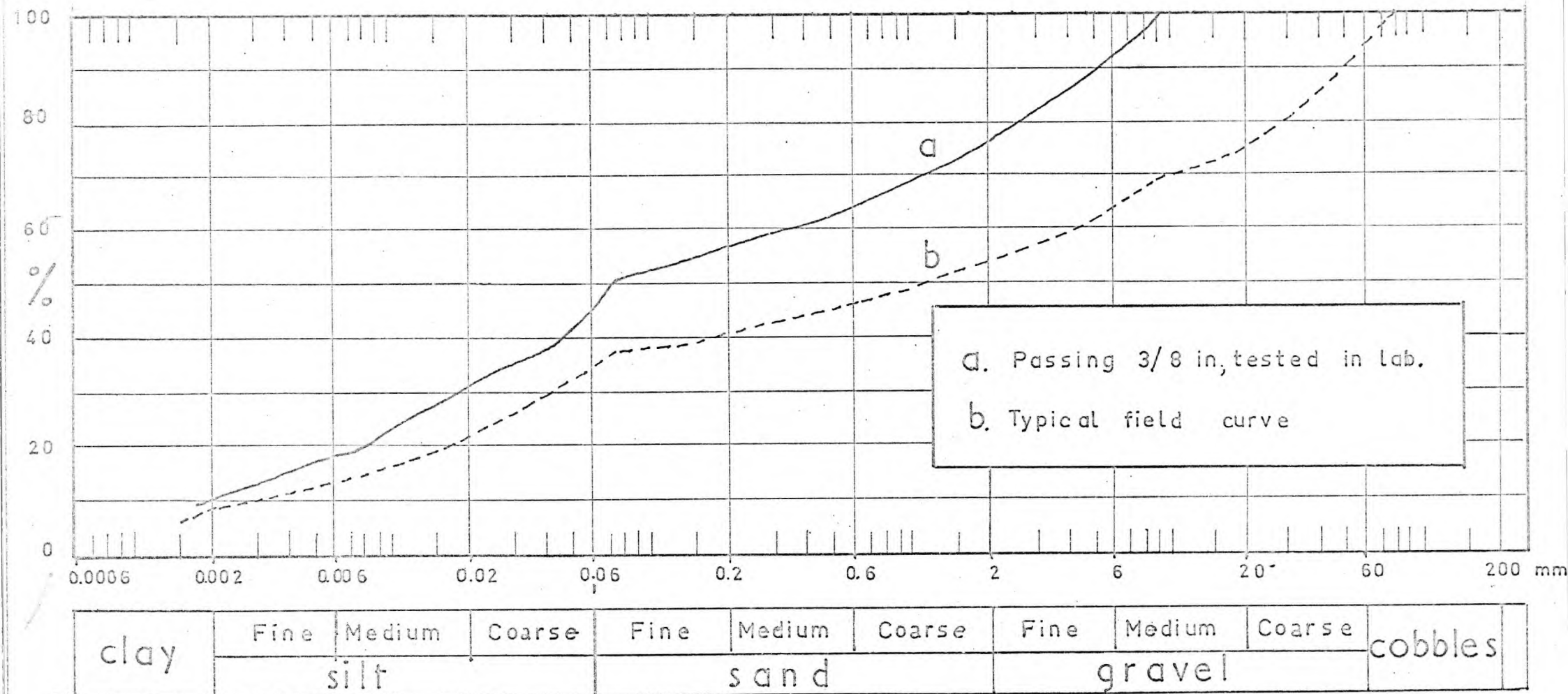
performance for three embankments on boulder clay.
(after Bishop and Al-Dhahir, 1969)

Insitu test values				Field performance	
C_v ft ² /yr	k cm/sec.x10 ⁻⁸			C_v ft ² /yr	
				64	(30)
460 C_s (80)	0.84	(30)			
				30	(40)
14 C_s (12) ^{*2}	1.2	(12) ^{*2}	44		(c.8) ^{*2}
84 C_v (30)	0.79	(30)			
75 C_v (53)	0.7	(53)	128		(47)
			c100		(45) ^{*5}
7.5 C_s (9.4)	0.33	(9.4)	11		(15)
1.4 C_s (16)	0.87	(16)	5	C_s	(34)
1.4 C_v (21)	0.18	(21)			
50 C_v (3)	4.9	(5)	20		(3)
	2.8	(11)	50		(9)

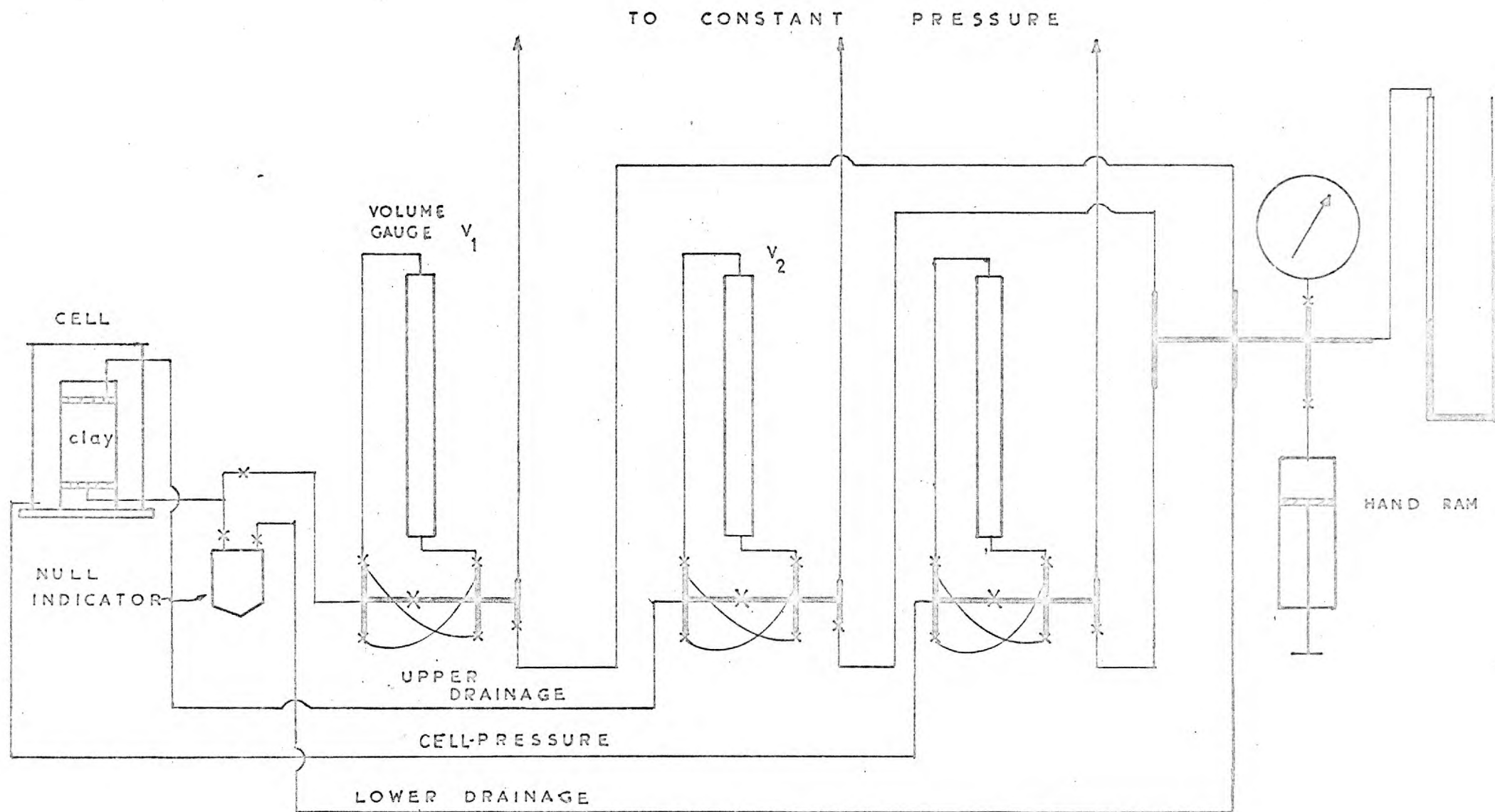
(see following page)

Notes for Table 11.2

- * denotes k value calculated from C_v and m_v .
- *1 denotes vertical samples except stated otherwise by symbol h.
- *2 upper part of fill was slightly more sandy.
- *3 Test on remoulded material.
- *4 From test on remoulded material of similar index properties from Selset.
- *5 Allowing for the decrease in total stress during consolidation as measured on a total stress cell.
- () denotes effective stress value to which test result applies, in psi.
- d denotes dissipation test on 4 in. diameter sample.
- t: From volume change in consolidation stage of $1\frac{1}{2}$ in. diameter triaxial samples with side drains.

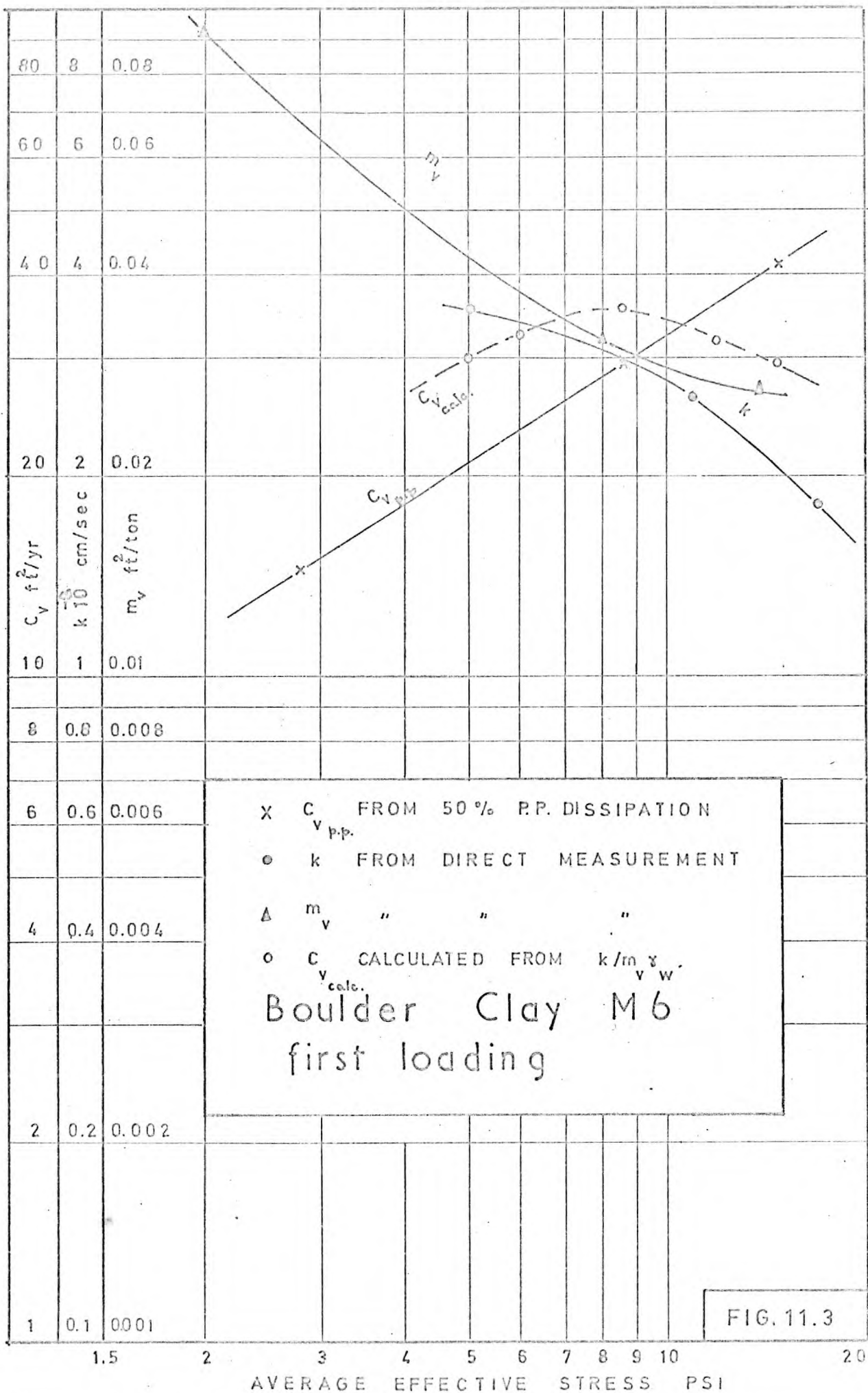


Particle Size Distribution
Boulder Clay M6



OUTLINE OF DISSIPATION AND PERMEABILITY APPARATUS

FIG.11.2



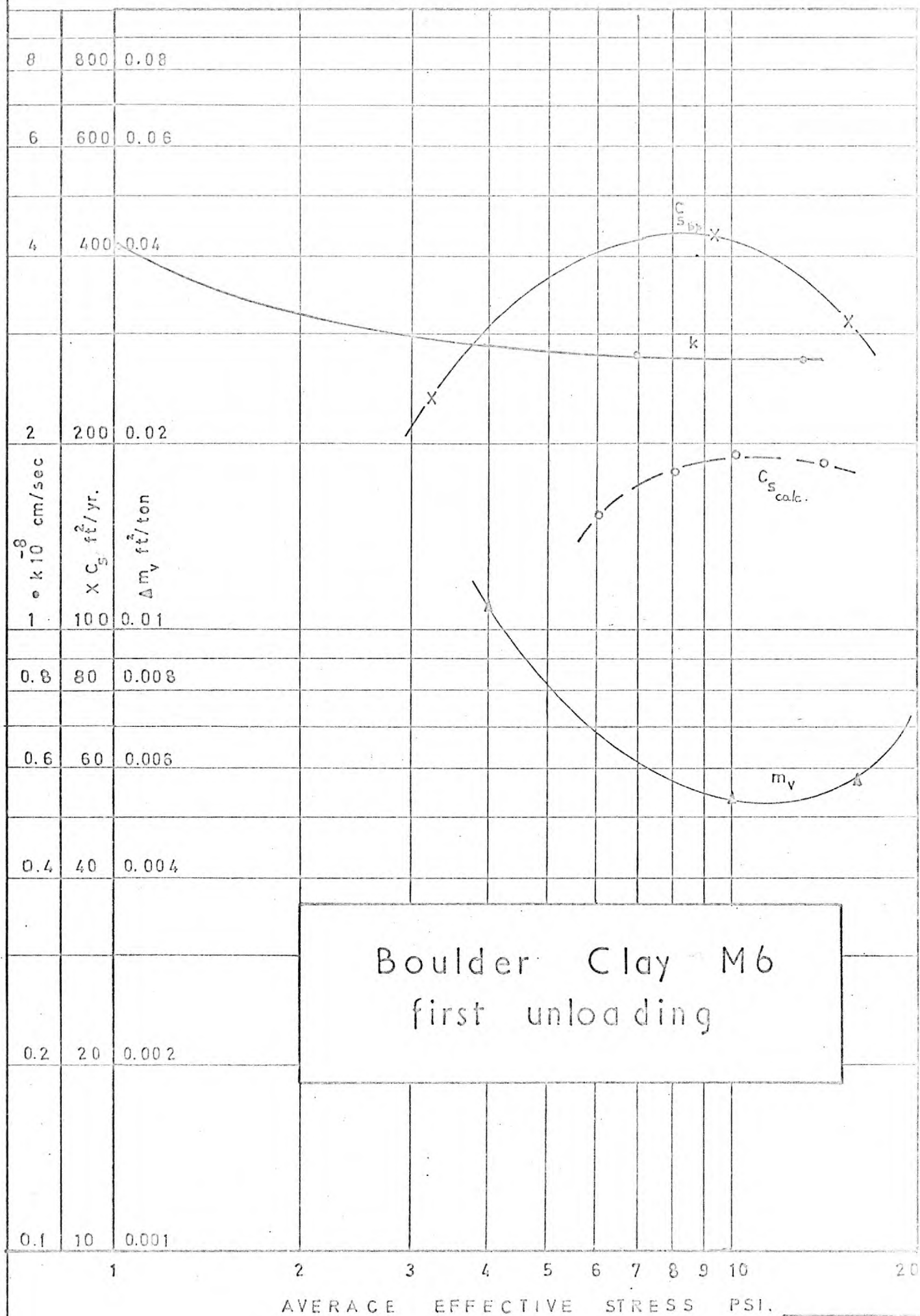


FIG. 11.4

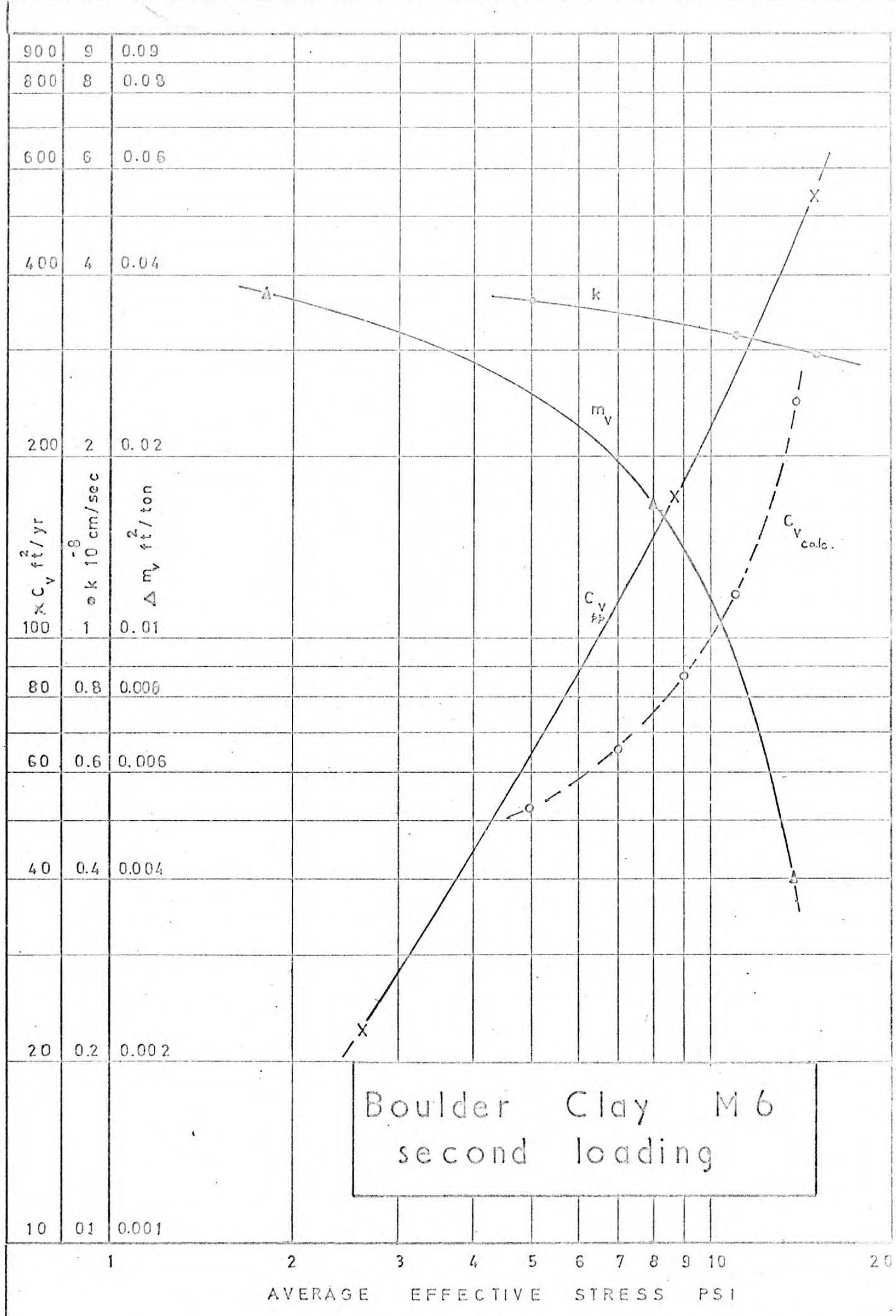
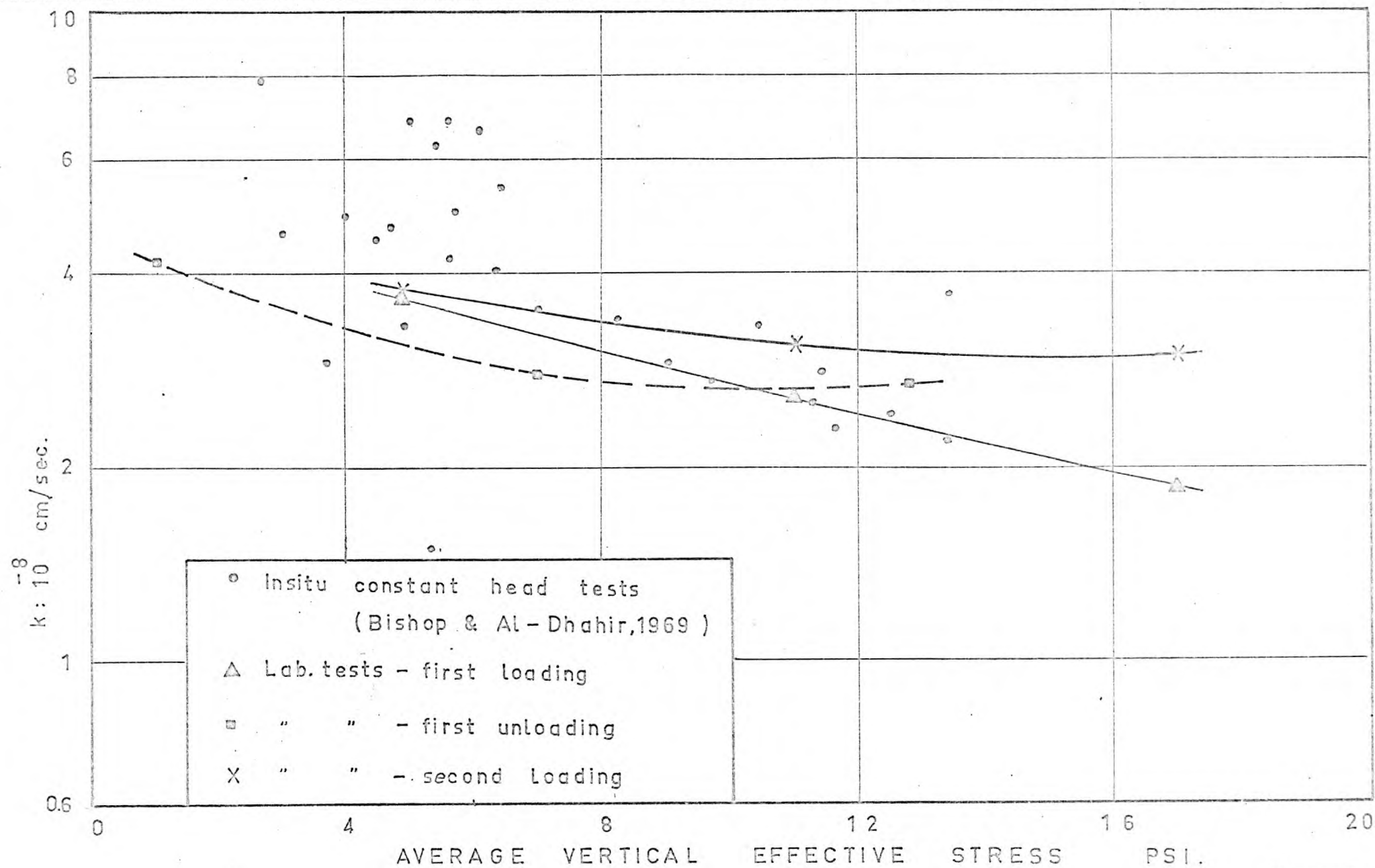
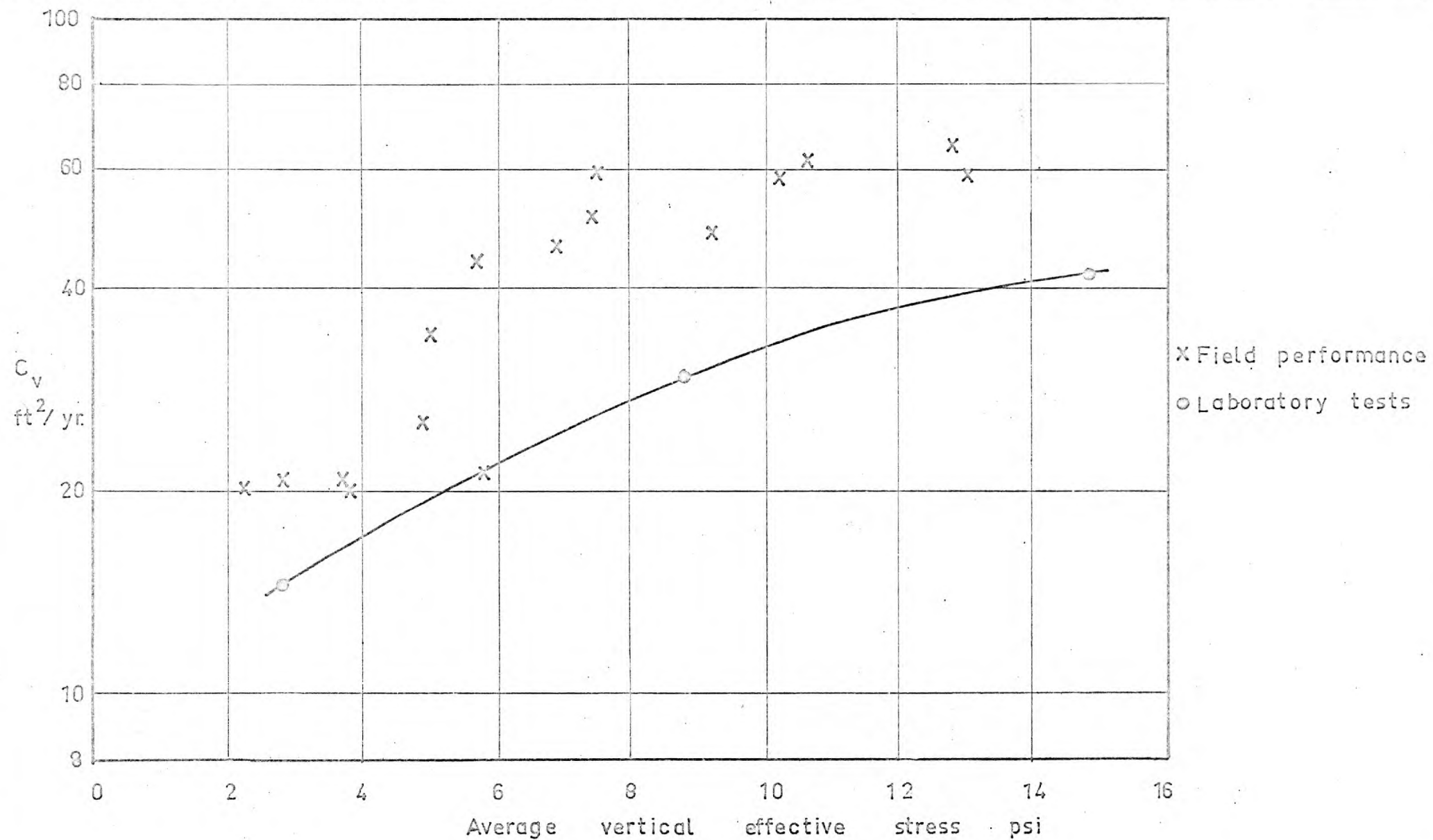


FIG.11.5



INSITU AND LAB. k VALUES

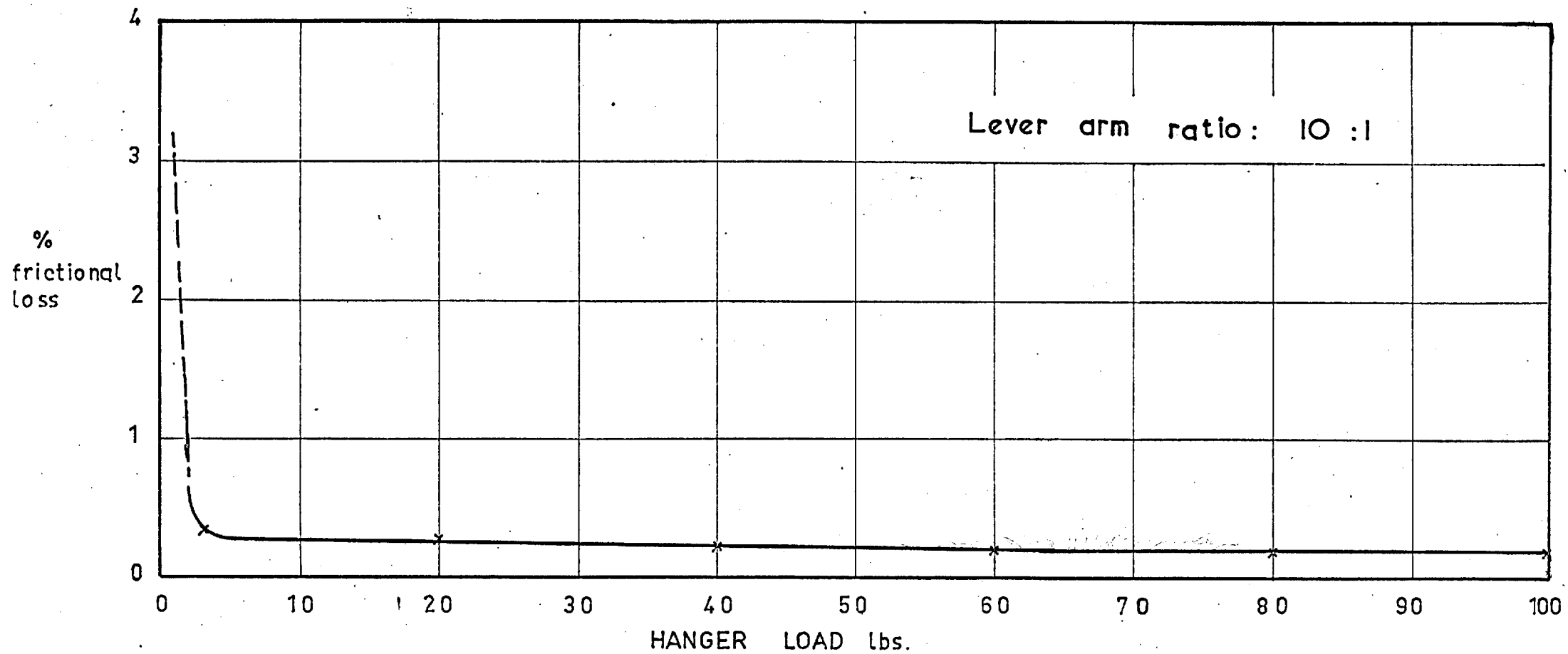
FIG. 11.6



Comparison of C_v from field and laboratory tests

APPENDIX A.

Calibration curves for the ring shear apparatus.



Calibration for frictional loss in the linear bearings.

FIG.A.1

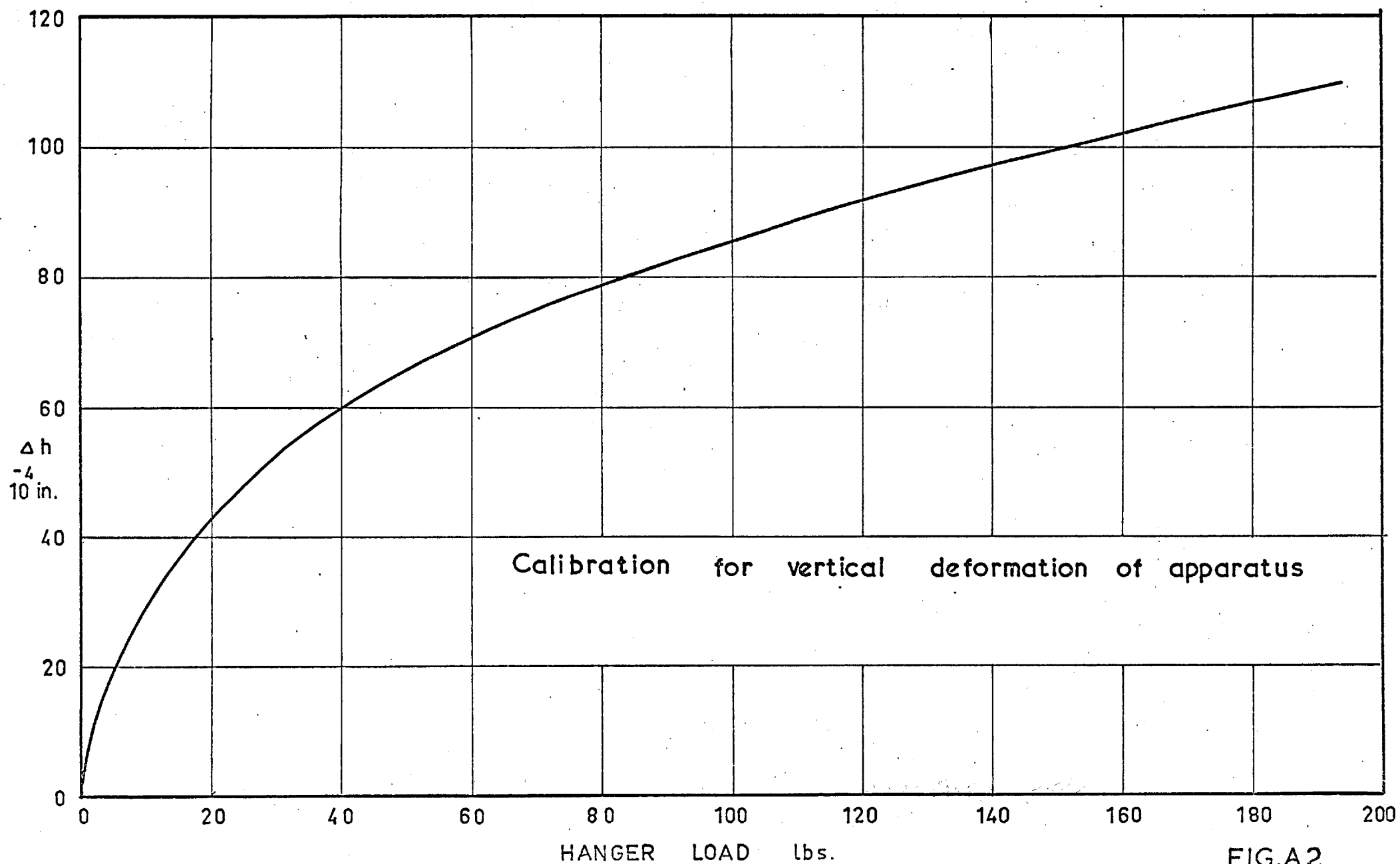


FIG.A2

Effect of non-uniform stress distribution on the failure plane.

The calculations are outlined in the following steps:

1. The equation for the non uniform stress distribution σ_n' at any radius r is defined in terms of maximum stress p , and the external and internal radii, r_2 and r_1 , respectively.

$$\text{i.e. } \sigma_n' = f(p, r). \quad (i)$$

2. Then, total normal load,

$$P = \int_{r_1}^{r_2} \sigma_n' \cdot 2\pi r \cdot dr, \quad (ii)$$

from which an expression for p in terms of P , r_1 and r_2 is obtained. This is next substituted in eq. (i) so that σ_n' may be expressed in terms of P and r .

3. Assuming $\tau = \sigma_n' \tan \phi'$ (iii)

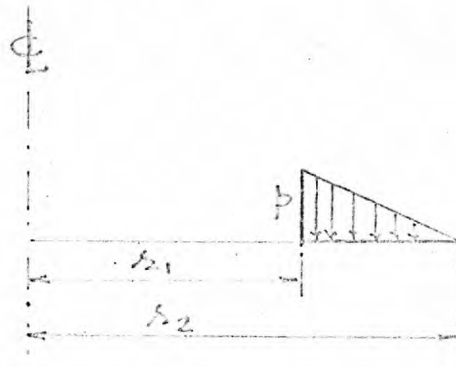
4. Then, torque $M = \tau \times \text{area} \times \text{lever arm}$.

$$= \int_{r_1}^{r_2} \tau (r_2 - r_1) \cdot 2\pi r \cdot dr. \quad (iv)$$

Substituting eq. (iii) in eq. (iv) gives an expression for $\tan \phi'$ in terms of M, P, r_1 and r_2 .

A specimen derivation for one of the cases is given below.

Case Linear II



$$1 \quad \sigma_n' = \frac{(r_2 - r)}{(r_2 - r_1)} p \quad \text{for } r_1 \leq r \leq r_2$$

$$2 \quad \text{Total normal load, } P = \int_{r_1}^{r_2} \sigma_n' \cdot 2\pi r \cdot dr$$

$$= 2\pi p \int_{r_1}^{r_2} \frac{(r_2 - r)}{(r_2 - r_1)} \cdot r \cdot dr$$

$$= \frac{\pi p}{3} (r_2 + 2r_1) (r_2 - r_1)$$

$$\therefore \sigma_n' = \frac{(r_2 - r) \cdot 3P}{(r_2 - r_1)^2 (r_2 + 2r_1) \pi}$$

$$3. \quad \tau = \sigma_n' \tan \phi'$$

$$= A (r_2 - r)$$

where $A = \frac{3P}{(r_2 - r_1)^2 (r_2 + 2r_1) \pi}$

$$4. \quad \text{Torque } M = \sum_{r_1}^{r_2} A (r_2 - r) \cdot 2\pi r \cdot dr \cdot r$$

$$= \frac{P(r_2^3 + 2r_1 r_2 + 3r_1^2) \tan \phi'}{2(r_2 + 2r_1)}$$

$$\therefore \tan \phi' = \frac{2M(\lambda_2 + 2\lambda_1)}{P(\lambda_2^2 + 2\lambda_1\lambda_2 + 3\lambda_1^2)}$$

Note that for a uniformly distributed load, (normal case), the solution can be easily obtained :

$$\therefore \tau = \text{constant}$$

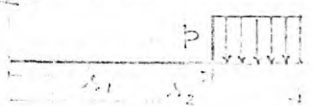
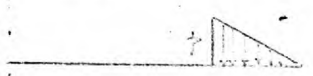
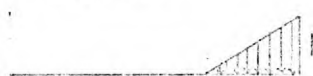
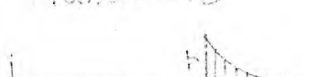
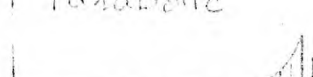
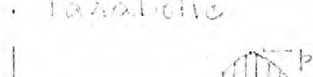
$$M = 2\pi \int_{\lambda_1}^{\lambda_2} \tau \lambda^2 d\lambda$$

$$\text{or } \tau = \frac{3M}{2\pi(\lambda_2^3 - \lambda_1^3)}$$

$$\text{and } \sigma_n' = \frac{P}{\pi(\lambda_2^2 - \lambda_1^2)}$$

$$\therefore \tan \phi' = \frac{3}{2} \frac{M}{P} \frac{(\lambda_2 + \lambda_1)}{(\lambda_2^2 + \lambda_1\lambda_2 + \lambda_1^2)}$$

The Table, overleaf, summarises the expressions for σ_n' and $\tan \phi'$ for different stress distributions.

STRESS PROFILE	σ_m'	$\tan \phi'$	$\tan \phi'$ for $\lambda_2 = 3''$ & $\lambda_1 = 2''$	$\frac{\tan \phi'_{\text{asymptotic}}}{\tan \phi'_{\text{uniform}}}$
Uniform 	p	$\frac{3}{2} \frac{M}{P} \frac{(\lambda_2 + \lambda_1)}{(\lambda_2^2 + \lambda_1 \lambda_2 + \lambda_1^2)}$	$0.3947 \frac{M}{P}$	—
Linear 	$\frac{(\lambda_2 - \lambda_1) p}{(\lambda_2 - \lambda_1)}$	$\frac{2M}{P} \frac{(\lambda_2 + 2\lambda_1)}{(\lambda_2^2 + 2\lambda_1 \lambda_2 + 3\lambda_1^2)}$	$0.4242 \frac{M}{P}$	1.0747
Linear 	$\frac{(\lambda_2 - \lambda_1) p}{(\lambda_2 - \lambda_1)}$	$\frac{2M}{P} \frac{(2\lambda_2 + \lambda_1)}{(3\lambda_2^2 + 2\lambda_1 \lambda_2 + \lambda_1^2)}$	$0.3721 \frac{M}{P}$	0.9427
Parabolic 	$\frac{(\lambda_2 - \lambda_1) \lambda_1 p}{\lambda_2 (\lambda_2 - \lambda_1)}$	$\frac{M}{P} \frac{3}{(\lambda_2 + 2\lambda_1)}$	$0.4286 \frac{M}{P}$	1.0859
Parabolic 	$\frac{(\lambda_2 - \lambda_1) \lambda_2 p}{\lambda_2 (\lambda_2 - \lambda_1)}$	$\frac{M}{P} \frac{3}{(\lambda_1 + 2\lambda_2)}$	$0.3750 \frac{M}{P}$	0.95
Parabolic 	$p \left[\frac{1 - \{2\lambda - (\lambda_1 + \lambda_2)\}^2}{(\lambda_2 - \lambda_1)^2} \right]$	$\frac{5M}{3P} \frac{(\lambda_2^2 - \lambda_1^2)(\lambda_2^2 + \lambda_1^2 - 2\lambda_1 \lambda_2)}{\left[\frac{5}{3}(\lambda_2 \lambda_1^4 - \lambda_1 \lambda_2^4) + (\lambda_2^5 - \lambda_1^5) \right]}$	$0.3970 \frac{M}{P}$	1.006
Parabolic 	$p \left[\frac{2\lambda - (\lambda_1 + \lambda_2)}{(\lambda_2 - \lambda_1)^2} \right]^2$	$\frac{15M}{P} \frac{(\lambda_2^2 - \lambda_1^2) \left[(\lambda_2^2 + \lambda_1^2) + \frac{(\lambda_2 + \lambda_1)^2}{2} - \frac{4}{3}(\lambda_2^2 + \lambda_1^2 + \lambda_1 \lambda_2) \right]}{\left[12(\lambda_2^5 - \lambda_1^5) + 5(\lambda_1 + \lambda_2)^2 (\lambda_2^3 - \lambda_1^3) - 15(\lambda_1 \lambda_2)^2 (\lambda_2^4 - \lambda_1^4) \right]}$	$0.3984 \frac{M}{P}$	1.009

Acknowledgements

The work described in this thesis was carried out in the Dept. of Civil Engineering under the supervision of Professor A.W. Bishop. The author wishes to express his gratitude to Professor Bishop for his guidance and interest throughout this project.

The author is grateful to Mr. A.E. Skinner, Dr. J.T. Tchalenko, Dr. G.E. Green and Dr. P.R. Vaughan for helpful discussions and assistance at various stages of the project.

The author is very appreciative of the willing help provided to him by his many colleagues. He is especially grateful to Dr. D.J. Petley for the permission to use his test results. The work on the scanning electron microscope and the X-ray diffraction analysis was made possible by the generous assistance of Mr. D. Wield and Mr. A.D. Burnett, to whom the author is extremely grateful.

The author is also very grateful to Mr. D.T. Evans, Mr. L.F. Spall, Mr. F.D. Evans and Mr. E. Harris for their excellent technical assistance throughout this project. Thanks are also expressed to John Laing Ltd., and especially to their site-agent at Wraysbury, Mr. Hornsby, for their help on numerous visits to the site.

The author is very grateful to his parents for providing financial assistance in the initial stages of the project. Subsequent financial support was provided by the Construction Industry Research and Information Association (CIRIA) to whom thanks are also expressed.

Finally the author is grateful to his wife, Renate, for typing this thesis, and yet having maintained a pleasant temperament.

References

- Agarwal, K.B. (1967). The influence of size and orientation of sample on the undrained strength of London clay.
Ph.D. Thesis, Univ. of London.
- Akai, K. and Adachi, T. (1965). Study of one dimensional consolidation and the shear strength characteristics of fully saturated clay in terms of effective stress.
Proc. 6th ICSMFE. Vol. 1. p. 146.
- Al-Dhahir, Z.A.R. (1967). Correlation between field and laboratory measurements on earth dams.
Ph.D. Thesis, Univ. of London.
- , and Tan, S.B. (1968). A note on one-dimensional constant head permeability tests.
Geotech., 18: 4.
- Ambraseys, N.N. (1960). The seismic sea wave of July 9, 1956, in the Greek Archipelago.
Int. Geophysical Res. 65: 1257 - 1265.
- American Society of Civil Engineers (1917). Progress report of the special committee to codify present practice on the bearing value of soils for foundations.
ASCE Papers and Discussions, p. 1174.
- Barden, L. and Berry, F.L. (1965). Consolidation of normally consolidated clays.
Proc. ASCE. Vol. 91: 15 - 35, SM 5.
- Barron, R.A. (1948). Consolidation of fine grained soils by drain wells.
Trans. ASCE. Vol. 113, 1948.
- Beene, R.R.W. (1967). Waco Dam slide.
Int. SMFE, ASCE. Vol. 93, No. SM4. 35 - 44.
- Bhandari, R.K. (1970). Ph.D. Thesis, Univ. of London.
- Binger, W.V. (1948). Analytical studies of Panama Canal slides.
Proc. IIInd ICSMFE. Vol. II, 54 - 60.
- and Thompson, F.F. (1949). Panama Canal the sea level project.
A symposium: Excavation slopes.
Trans. ASCE. Vol. 114, 735 - 754.

- Bishop, A.W. (1952). The stability of earth dams.
Ph.D. Thesis, Univ. of London.
- _____ and Henkel, D.J. (1953). Pore pressure changes during shear in two undisturbed clays.
Proc. 3rd ICSMFE Vol. 1, 94 - 99.
- _____ (1958). Tests requirements for measuring the coefficient of earth pressure at rest.
Proc. Brussels Conference on Earth Pressure Problems.
- _____, Kennard, M.F. and Penman, A.D.M. (1960). Pore pressure observations at Selset Dam.
Proc. Conf. Pore Pressure and Suction in Soils. Butterworth, London, 91 - 102.
- _____ and Henkel, D.J. (1962). The measurement of soil properties in the triaxial test.
Edward Arnold, (Publishers) Ltd. London.
- _____, Kennard, M.F. and Vaughan, P.R. (1964) Developments in the measurement and interpretation of pore pressures in earth dams.
Trans. 8th Int. Cong. Large Dams, Edinburgh, Vol. 2, 47 - 52.
- _____ and Gibson, R.E. (1964). The influence of the provisions for boundary drainage on strength and consolidation characteristics of soils measured in the triaxial apparatus.
ASTM Sp. Publ. 361.
- _____, Webb, D.L. and Lewin, P.I. (1965). Undisturbed samples of London clay from Ashford Common shaft: strength - effective stress relationships.
Geotech., Vol. 15, 1 - 31.
- _____ (1966). Soils and soft rocks as engineering materials.
Inaugural Lecture, Imperial College, 289 - 313.
- _____ (1966). Strength of soils as engineering materials.
Geotech. 16, 2: 89.
- _____ (1967). Progressive failure, with special reference to the mechanism causing it.
Proc. Geotech. Conf. Oslo, 2: 3 - 10.
- _____ and Little, A.L. (1967). The influence of size and orientation of the sample on the apparent strength of the London clay at Maldon, Essex.
Proc. Geotech. Conf. Oslo, Vol. 1: 89 - 96.

- Bishop, A.W. and Al-Dhahir, Z.A.R. (1969). Some comparisons between laboratory tests, in situ tests and full scale performance, with special reference to permeability and coefficient of consolidation. Conf. on Insitu Investigations in Soils and Rocks. London, Paper 19, 161 - 174.
- and Garga, V.K. (1969). Drained tension tests on London clay.
Geotech. 19, 2: 309 - 313.
- Bjerrum, L. (1954). Theoretical and experimental investigations on the shear strength of soils.
N.G.I. Publ. No. 5.
- , Simons, N.E. and Torblaa, I. (1958). The effect of time on the shear strength of a soft marine clay. Proc. Conf. Earth Pressure Problems, Brussels, 1: 148 - 158.
- (1967). Progressive failure in slopes of over-consolidated plastic clay and clay shales.
Jnl. SMFE Div. ASCE. 93.
No. SM 5,1 - 49.
- Bolt, G.H. (1956). "Physio-chemical analysis of the compressibility of pure clays."
Geotech. 6: 86 - 93.
- Borowicka, H. (1963). Vienna method of shear testing.
ASTM STP. 361: 306 - 314.
- Brace, W.F. (1960). An extension of the Griffith theory of fracture to rocks.
Jnl. Geophysical Research, Vol. 65, No. 10.
- Burmister, D.M. (1953). The place of the direct shear test in soil mechanics.
ASTM, STP 1953.
- Burnett, A.D. (1968). A geotechnical study of weathered London clay from Staines and Herne Bay.
M.Sc. Dissertation, Univ. of London.
- Carr, J.F. and Walker, D.M. (1967). An annular shear cell for granular materials.
Powder Technology, 1, 369 - 373.
- Casagrande, A. and Wilson, S.D. (1951). Effect of rate of loading on the strength of clays and clay shales at constant water content.
Geotech., 2: 251 - 263.
- Cassell, F.L. (1948). Slips in fissured clay.
Proc. 2nd, ICSMFE 2: 46 - 50.

- Chandler, R.J. (1966). The measurement of residual shear strength in triaxial compression.
Geotech. 16: 181 - 186.
- (1967). Shear strength of Keuper Marl.
Ph.D. Thesis, Univ. of Birmingham.
- Clausing, D.P. (1959). Comparison of Griffith's theory with Mohr's failure criteria.
Q. Col. Sch. Mines, 54: 3: 287.
- Conlon, R.J. (1966). Landslide on the Toulmoustou River, Quebec.
Can. Geotech. Jnl. 3: 3: 113 - 114.
- Cooling, L.F. and Smith, D.B. (1936). The shearing resistance of soils.
Proc. 1st ICSMFE. Vol. 1: p. 37.
- and Skempton, A.W. (1941). Some experiments on the consolidation of clay.
Jnl. L.C.E. 16: 381 - 398.
- and ————— (1942). A laboratory study of London clay.
Jnl. I.C.E., 17: p. 251.
- and Gibson, R.E. (1955). Settlement studies on structures in England.
Proc. Conf. on correlation between calculated and observed stresses and displacements in structures.
I.C.E. London, 295 - 317.
- Crawford, C.B. (1963). Cohesion in an undisturbed sensitive clay.
Geotech., 13: 2: 132 - 146.
- Davis, E.H. and Raymond, G.P. (1965). A non-linear theory of consolidation.
Geotech. 15: 2: 161.
- and Poulos, H.G. (1968). The use of elastic theory for settlement prediction under three-dimensional conditions.
Geotech., 18: 1: 67.
- D'Elia, B. (1968). A study of structures in the London clay at Wraysbury, Middlesex.
M.Sc. Dissertation, Univ. of London.
- DeLory, F.A. (1957). Long term stability of slopes in overconsolidated clays.
Ph.D. Thesis, Univ. of London.

- Duncan, J.M. and Seed, H.B. (1966). Strength variations along failure surfaces in clay.
Jnl. SMF Div. ASCE. 92, SM6, 81 - 104.
- Franzius, Streck and Hinderke, (1929). The Experiment Institute for Hydraulics and Foundations at the Technical University at Hanover.
Hydraulic Laboratory Practice, 599 - 601, ASCE.
- Frevert, R.E. and Kirkham, D. (1948). A field method of measuring permeability of soil below a water table.
HRB. Proc. 28: 433 - 442.
- Frydman, S. (1967). Triaxial and tensile strength in stabilised soil.
Proc. 3rd. Asian Conf. SMFE. Haija.
- Gibson, R.E. and McNamee, J. (1957). The consolidation settlement of a load uniformly distributed over a rectangular area.
Proc. 4th ICSMFE, Vol. 1. p. 297.
- (1958). The progress of consolidation in a clay layer increasing in thickness with time.
Geotech. Vol. 8. No. 4.
- and McNamee, J. (1960). Plane strain and axisymmetric problems of the consolidation of a semi-infinite clay stratum.
Qtly. I. of Mech. and app. Maths. Vol. 13, p. 210.
- and McNamee, J. (1963). A three-dimensional problem of the consolidation of a semi-infinite clay stratum.
Qtly. J. of Mech. app. Maths. Vol. 16, p. 115.
- (1963). An analysis of system flexibility and its effect on time-lag in pore water pressure measurements.
Geotech., Vol. 13, 1 - 11.
- (1966). A note on the constant head test to measure soil permeability in situ.
Geotech., Vol. 16, 256 - 257.
- , Hussey, G.L. and England, M.J.L. (1967). The theory of one dimensional consolidation of saturated clays.
Geotech., Vol. 17, No. 3.
- (1969). Closing remarks.
In situ Conf. London.

- Gillott, (1969). Study of the fabric of fine grained sediments with the scanning electron microscope. *Jnl. of Sed. Petrology*, 39: 1: 90 - 105.
- Gould, J.F. (1960). A study of shear failure in certain Tertiary marine sediments. *Proc. Research Conf. on shear strength of cohesive soils*, Boulder, Colorado, 615 - 641.
- Griffith, A.A. (1921). The phenomena of rupture and flow in solids. *Phil. Trans. Roy. Soc. London*, A221, 163 - 198.
- Grim, R.E. (1953). *Clay Mineralogy*. McGraw Hill Book Co. Inc. New York.
- (1962). *Applied Clay Mineralogy*. McGraw Hill Book Co. Inc. New York.
- Haefeli, R. (1938). Mechanische Eigenschaften von Lockergesteinen. *Schweizer Bauzeitung*. Vol. III., 299 - 303; 321 - 325.
- (1950). Investigation and measurements of the shear strengths of saturated cohesive soils. *Geotech.*, 2: 3: 186 - 208.
- Hansbo, S. (1960). Consolidation of clay with special reference to influence of vertical sand drains. *Proc. No. 18*, Swedish Geotech. Inst.
- Hansen, B. (1961). Shear box tests on sand. *Proc. 5th ICSMFE Vol. 1*, Paris.
- Hardy, R.M. Brooker, E.W., and Curtis, W.E. (1962). Landslides in overconsolidated clays. *The Engineering Journal (Montreal)*, 45: 6: 81 - 89.
- Hasegawa, H., and Ikeuti, M. (1964). On the tensile test of disturbed soils. *Int. Symp. on Rheology and Soil Mechanics*, (Grenoble).
- Helenlund, K.V. (1965). Torsional field shear tests. *Proc. 6th ICSMFE 1*: 240 - 243.
- Henkel, D.J. and Skempton, A.W. (1954). A landslide at Jackfield, Shropshire, in overconsolidated clay. *Geotech.*, 5: 2: 131 - 137.
- (1957). Investigations of two long-term failures in London clay slopes at Wood Green and Northolt. *Proc. 4th ICSMFE 2*: 315 - 320.

- Henkel, D.J. (1958). The correlation between deformation, pore water pressure and strength characteristics of saturated clay.
Ph.D. Thesis, Univ. of London.
- (1969). The role of waves in causing submarine landslides. (Unpublished).
- Howett, D.F. (1917). The Origin of Bentonite.
Jnl. Washington Academy of Science.
Vol. 7: 196 - 198.
- Hirschfield, R.C., Whitman, R.V. and Wolfskill, L.A.
(1965). Engineering properties of nuclear craters.
Research report R 65 - 33. Dept. of Civil Engineering, M.I.T., Mass.
- Hobbs, D.W. (1964). The tensile strength of rocks.
Int. Jnl. Rock Mech. and Mining Sciences. Vol. 1: 385 - 396.
- Hook, E. (1965). Rock fracture under static stress conditions.
CSIR Report MEG 383, Pretoria, South Africa.
- Horn, H.M. and Deere, D.V. (1962). Frictional characteristics of minerals.
Geotech., 12: 4: 319 - 335.
- Hutchinson, J.N. (1965). The stability of cliffs composed of soft rocks, with particular reference to the coasts of south-east England.
Ph.D. Dissertation, Cambridge University.
- (1967). The free degradation of London clay cliffs.
Proc. Geotech. Conf. Oslo, 2: 183 - 184.
- (1969). A reconsideration of the coastal landslides at Folkestone Warren, Kent.
Geotech. 19: 6 - 38.
- (1969). Proposed development at Sinnott Road/Cogan Av.
Report on slope stability of the site (unpublished).
- Hvorslev, M.J. (1939). Torsion shear tests and their place in the determination of the shearing resistance of soils.
Symp. on shear testing of soils, Proc. ASTM, 39: 999 - 1020.
- (1951). Time-lag and soil permeability in groundwater observations.
Bull, No. 36, U.S. Waterways Experiment Station, Vicksburg, Mississippi.

- Hvorslev, M.J. and Kaufman, R.I. (1952). Torsion shear apparatus and testing procedures.
Bull, 38, Waterways Experimental Station Corps.
of Engineers. U.S. Army, Vicksburgh, Miss.
- (1960). Physical components of the shear strength of saturated clays.
Proc. Research Conf. on shear strength of Cohesive Soils, Boulder, 169 - 273.
- Jaeger, J.C. (1966). The brittle fracture of rocks.
Symp. Rock Mech. 8th. Minnesota, (published by AIME).
- and Cook, N.G.W. (1969). Fundamentals fo Rocks Mechanics.
Methuen and Co. Ltd., London.
- James, P.M. (1970). Time effects and progressive failure in clay slopes.
Ph.D. Thesis, Univ. of London.
- Kadir, N. (1951). Measurement of permeability of saturated soils below the water table.
Ph.D. Thesis, Utah State Agric. College, Logan, Utah, U.S.A.
- Kenney, T.C. (1956). An examination of the methods of calculating the stability of slopes.
M.Sc. Thesis, Univ. of London.
- (1967). The influence of mineral composition on the residual strength of natural soils.
Proc. Geotech. Conf. Oslo, 1. 123 - 129.
- Kirsch, G. (1898). Die Theorie der Elastität and die Bedürfnisse der Festigkeitslehre.
Z. Ver. dtsh. Ing. 32: 797 - 867.
- Kjellman, W. (1951). Testing of shear strength of clay in Sweden.
Geotech., 2: 225 - 235.
- Knight, D.K. (1963). Oahe Dam: Geology, Embankment and cut slopes.
Jnl. SMFE. ASCE. Vol. 89, No. SM2 paper 3466.
- Krsmanovic, D. (1967). Initial and residual shear strength of hard rock.
Geotech., 17: 2: 145 - 160.

- La Gatta, (1969). Private communication.
- La Rochelle, P. (1967). Membrane, drain and area correction in triaxial test on soil samples failing along a single shear plane.
Proc. 3rd Pan-American Conf. on SMFB. Vol., 1.
- Lambe, T.W. (1958). The structure of compacted clay.
Proc. ASCE. 91, SM4, 85 - 106.
- Langer, C. (1938). Propriétés mécaniques et physiques des terrains cohérents. Laboratoires du Batiment et des Travaux Publics, Paris. Complet Rendu de Recherches Effectuées durant l'Année. 19 - 27.
- Leonards, G.A. and Altschaeffl, A.G. (1964). The Compressibility of Clay.
Jnl. SM and F. Div. ASCE. 90, SM5.
- Lovenburg, H.T. (1969). Creep characteristics of London clay.
Ph.D. Thesis, Univ. of London.
- Low, P.F., (1968). Physical chemistry of clay - water interaction.
Advances in Agronomy, Vol. 13.
- MacDonald, D.H. and Skempton, A.W. (1955). A survey of comparisons between calculated and observed settlements of structures on clay.
Proc. Conf. on correlation between calculated and observed stresses and displacements in structures, I.C.E. London, 318 - 337.
- Marsal, R.J., Ramirez, L., and Campos, J. (1965). Field tests on fissured clay shales.
Sp. publ., Comision Federal de Electricidad, Mexico City.
- Martin, R.T. (1962). Adsorbed water on clay a review.
Proc. 9th Nat. Conf. Clays and clay Minerals 28 - 70. Pergamon Press, New York.
- Meade, R.H. (1964). Removal of water and rearrangement of particles during the compaction of clayey sediments - review.
U.S. Geol. Survey, Prof. Paper 497 B.
- Middlebrooks, T.A. (1942). Port Peck slide.
Trans. ASCE., Vol. 107. Paper 2144. p. 723.
- Mirata, T. (1965). Deformations in compressible foundations of embankments.
M.Sc. Thesis, Univ. of London.

- Mitchell, J.K. (1956). The fabric of natural clays and its relation to engineering properties.
Proc. H.R.B. 35: 693 - 713.
- and Houston, W.H. (1969). The causes of clay sensitivity.
Proc. ASCE, 95, SM3. 845 - 871.
- Moore, D.G. and Scruton, P.C. (1957). Minor internal structures of some recent unconsolidated sediments.
Bull, Am. Assoc. Petrol. Geologists. Proc. 41 $\frac{1}{2}$
- Morgenstern, N.R. and Tchalenko, J.S. (1967). The optical determination of preferred orientation in clays and its application to the study of microstructure in consolidated kaolin, I.
Proc. Royal Soc., A. 300: 218 - 234.
- (1967). The optical determination of preferred orientation in clays and its application to the study of microstructure in consolidated kaolin, II.
Proc. Royal Soc. A, 300: 235 - 250.
- (1967). Micro-scopie structures in kaolin, subjected to direct shear.
Geotech. 17: 309 - 328.
- (1967). Microstructural observations on shear zones from slips in natural clays.
Proc. Geotech. Conf. Oslo, 1: 147 - 152.
- (1967). Submarine slumping and the initiation of turbidity currents.
Marine Geotech., Univ. of Illinois, 189 - 220.
- (1968). Shear strength of stiff clay.
Proc. Geotech. Conf. Oslo, 2: 59 - 71.
- , Mitchell, J.K., and Quigley, R.M. (1969). Structural and physio-chemical effects on the properties of clays.
Summary report on speciality session No. 7.
Proc. 7th ICSMFE, Mexico.
- Newland, P.L. and Alley, B.H. (1960). A study of consolidation characteristics of a clay.
Geotech. 10: No. 2.
- Payne, F.C.J. and Fountaine, E.R. (1952). A field method of measuring the shear strength of soils.
Jnl. Soil Science.
- Peck, R.B. (1967). Stability of natural slopes.
Jnl. S.M.F. Div. ASCE. 93. SM4. 403 - 417.

- Peterson, R. (1954). Studies of Bearpaw shale at a dam site in Saskatchewan.
Proc. ASCE, Vol. 80: separate No. 476.
- (1958). Rebound in the Bearpaw shale, Western Canada.
Bull. Geological Society of America.
Vol. 69, No. 9: p. 1113.
- , Jaspar, J.L., Rivard, P.J. and Iverson, N.L. (1960). Limitations of laboratory shear strength in evaluating stability of highly plastic clays. Research Conf. on shear strength of cohesive soils. Colorado.
- Petley, D.J. (1966). The shear strength of soils at large strains.
Ph.D. Thesis, Univ. of London.
- Poskitt, T.J. (1969). The consolidation of saturated clay with variable permeability and compressibility.
Geotech. 19: 2: 234 - 252.
- Raymond, G.P. (1965). Contribution to discussion.
Can. Geotech. Jnl. Vol. 2: p. 101.
- Richart, F.E. (1957). A review of theories for sand drains ASCE. SMFE.
- Ringheim, A.S. (1964). Experiences with the Bearpaw shale at the South Saskatchewan River Dam.
Trans. 8th Int. Conf. on Large Dams. Edinburgh, Vol. 1: p. 529.
- Roscoe, K.H. (1953). An apparatus for the application of simple shear to soil samples.
Proc. 3rd. ICSMFE, Vol. 1 Zurich.
- (1967). Discussion, Proc. Geotech. Conf. Oslo, 2, 167 - 171.
- Rosenquist, I. Th., (1962). The influence of physio-chemical factors upon the mechanical properties of clays.
Proc. 9th. Nat. Conf. Clays and Clay Minerals, 12 - 27. Pergamon Press, New York.
- Ross, C.S. and Shannon, E.V. (1926). Minerals of bentonite and related clays and their physical properties.
Jnl. Am. Ceramic Soc. Vol. 9: 77 - 96.

- Rowe, P.W. (1964). The calculation of the consolidation rates of laminated, varved or layered clays, with particular reference to sand drains.
Geotech., 14: 4: 321 - 340.
- (1968). The influence of geological features of clay deposits on the design and performance of sand drains.
Proc. I.C.E. London, paper 7058 S.
- Rutledge, P.L. (1947). Relation of undisturbed sampling to laboratory testing.
Trans. ASCE. Vol. 109. p. 1175.
- Scarlett, B. and Todd, A.C. (1968). A split ring annular shear cell for the determination of the shear strength of a powder.
J. Sci. Instruments. Series 2. 1. p. 655.
- Schiffman, R.L. (1958). Consolidation of soil under time dependant loading and varying permeability.
Proc. HRB. Vol. 37. p. 584.
- and Gibson, R.E. (1964). The consolidation of non-homogeneous clay layers.
J. ASCE, SMFE, Vol. 90: 1 - 30.
- Schultze, E. (1957). Large scale shear tests.
Proc. 4th ICSMFE. Vol. 1 London.
- Sembenelli, P. and Ramirez, A.L. (1969). Measurement of residual strength of clay with a rotating shear machine.
Proc. 7th ICSMFE, Mexico City, Vol. 3.
- Shepard, F.P. (1963). Submarine Geology.
(2nd. ed.). Harper and Row, New York.
- Simons, N.E. (1967). Discussion, Proc.
Geotech. Conf. Oslo, 2: 159 - 160.
- Sinclair, S.R. and Brooker, E.W. (1967). The shear strength of Edmonton shale.
Proc. Geotech. Conf. Oslo, 1: 295 - 299.
- Som, N.N. (1968). The effect of stress path on the deformation and consolidation of London clay.
Ph.D. Thesis, Univ. of London.
- Skempton, A.W. (1943). The rate of softening of stiff fissured clays, with special reference to London clay.
Proc. 2nd ICSMFE 2: 50 - 53.
- and Northey, R.D. (1952). The sensitivity of clay.
Geotech., 3: 30 - 53.
- , Peck R.B. and MacDonald, D.H. (1955). Settlement of six structures in Chicago and London,
Proc. I.C.E. London Vol. 4.

- Skempton, A.W. and DeLory, F.A. (1957). Stability of natural slopes in London clay.
Proc. 4th ICSMFE London, 2: 378 - 381.
- and Henkel, D.J. (1957). Tests on London clay from deep borings at Paddington Victoria and the South Bank.
Proc. 4th ICSMFE 1: 100 - 106.
- (1961). Horizontal stresses in an overconsolidated Eocene clay.
Proc. 5th ICSMFE 1: 351 - 358.
- (1964). Long-term stability of clay slopes.
Geotech., 14: 77 - 101.
- and La Rochelle, P. (1965). The Bradwell slip, a short term failure in London clay.
Geotech., 15: 221 - 242.
- (1966). Some observations on tectonic shear zones.
Proc. 1st. Int. Conf. Rock. Mech. Lisbon, 1: 329 - 335.
- and Petley, D.J. (1967). The shear strength along structural discontinuities in stiff clays.
Proc. Geotech. Conf. Oslo, 2: 29 - 46.
- , Schuster, R.L., and Petley, D.J. (1969). Joints and fissures in the London clay at Wraysbury and Edgeware.
Geotech., 19: 205 - 217.
- and Hutchinson, J.N. (1969). Natural slopes and embankment foundations.
7th ICSMFE Mexico, State of the Art Volume, 291 - 340.
- Skinner, A.E. (1969). A note on the influence of inter-particle friction on the shearing strength of a random assembly of spherical particles.
Geotech., 19: 1: 150 - 157.
- Smiles, D.E. and Youngs, E.G. (1965). Hydraulic conductivity determination by several field methods in a sand tank.
Soil. Sc. Soc. of Am. Proc. 10: 58 - 68.
- Sowa, V. (1963). A comparison of isotropic and anisotropic consolidation in the shear behaviour of a clay.
Ph.D. Thesis, Univ. of London.
- Sowers, G.F. (1963). Strength testing of soils.
Laboratory shear testing of soils.
ASTM. STP. No. 361: 3 - 21.
- Suklje, L. and Drnovsek, J. (1965). Investigation of the tensile deformability of soils using hollow cylinders.
Proc. 6th ICSMFE Vol. 1: p. 368.

- Tan, S.B. (1968). Consolidation of soft clays with special reference to sand drains.
Ph.D. Thesis, Univ. of London.
- Tchalenko, J.S. (1967). The influence of shear and consolidation on the microscopic structure of some clays.
Ph.D. Thesis, Univ. of London.
- (1968). The evolution of Kink-bands and the development of compression textures in sheared clays.
Tectonophysics, 6: 159 - 174.
- (1968). The microstructure of London clay.
Q. J. Eng. Geol. 1: 155 - 168.
- Terzaghi, K. (1936). Stability of slopes in natural clay.
Proc. 1st ICSMFE Vol. 1: 174 - 179.
- (1960). Closing remarks.
Research Conf. on shear strength of cohesive soils, Boulder, Colorado. 1117 - 1119.
- Thorfinson, S.T. (1954). A large scale field shear test on a bentonite seam.
Proc. ASCE. Vol. 80: p. 549.
- Tiedemann, B. (1937). Ueber die Schubfestigkeit bindiger Boeden.
Die Bautechnik, Vol. 15: No. 30 and 33.
400 - 403, 433 - 435.
- Toms, A.H. (1948). The present scope and possible future development of soil mechanics in British railway civil engineering construction and maintenance.
Proc. 2nd ICSMFE 4: 226 - 237.
- Tourtelot, H.A. (1962). Preliminary investigation of the geological setting and chemical composition of the Pierre shale. Great Plains Region.
Professional Paper 390, Geological Survey, U.S. Dept. of Interior.
- Tschebotarioff, G.P., Ward, D.R. and De-Phillippe, A.A. (1951). The tensile strength of disturbed and re-compacted soils.
Proc. 3rd ICSMFE Vol. 1: p. 207.
- Turnbull, W.J., and Hvorslev, M.J. (1967). Special problems in slope stability.
Jnl. S.M.F. Div. ASCE. 93. SM4 499 - 528.
- U.S. Army Corps of Engineers (1968). Design Memorandum No. EC-24: Chatfield Dam and Reservoir.
U.S. Army Engng. District, Omaha.
- Van Auken, F.M. (1963). Shear strength of clay shales found in the Southwestern U.S.A.
Proc. 2nd Panam. Conf. on SMFE. Brazil, 1: 255 - 287.

- Vaughan, P.R. (1965). Field measurements in earth dams.
Ph.D. Thesis, Univ. of London.
- Vaughan, P.R. (1969). A note on sealing piezometers in boreholes.
Geotechnique, 19: 3: 405 - 413.
- Wade, N.H. (1963). Plane strain failure characteristics of saturated clay.
Ph.D. Thesis, Univ. of London.
- Ward, Samuels, S.G. and Butler, M.E. (1959).
Further studies of the properties of London clay.
Geotech. Vol. 9: No. 2.
- , Marsland, A. and Samuels, S.G. (1965).
Studies of the properties of undisturbed London clay at the Ashford Common shaft.
Geotech. Vol. 15: No.3.
- Weatherhead, A.V. (1940). Minerals Magazine,
Vol. 25: p. 529.
- Weeks, A.G. (1969). The stability of natural slopes in south east England as affected by periglacial activity.
Q. Jnl. Engng. Geology. 2: 1: 49 - 61.
- Weymouth, J.H., and Williamson, W.O. (1953). The effect of extrusion and some other processes on the microstructure of clay.
Am. Jnl. of Science Vol. 251.
- Whitman, R.V., Richardson, A.M. and Healey, K.A. (1961).
Time lags in pore pressure measurements.
Proc. 5th ICSMFE 1: 407 - 411.
- Wilkinson, W.B. (1968). Constant head in situ permeability tests in clay strata.
Geotech., 18: 2: 172 - 194.
- Wilson, S.D. and Johnson, K.A. (1964). Slides in over-consolidated clays along the Seattle Freeway.
2nd Annual Engng. Geology Soil Engng.
Symp. Proc. 29 - 42.
- Yudhbir (1969). Engineering behaviour of heavily over-consolidated clays and clay shales with speciale reference to long term stability.
Ph.D. Thesis, Cornell Univ.

References (addendum)

Bromwell, L.G. and Lambe, T.W. (1968). A comparison of laboratory and field values of C_v for Boston blue clay.
M.I.T. soils publ. No. 205.

Ghani, M.A. (1966). Determination of shear stress of a fine cloddy soil with a "guarded" shear head.
Proc. 2nd Int. Conf. of the Int. Soc. for Terrain Vehicle Systems, Univ. of Toronto Press, 279 - 310.

Henkel, D.J. and Sowa, V.A. (1963). The influence of stress history on stress path in undrained triaxial tests on clay.
Lab. shear testing of soils, ASTM, STP 361
280 - 291.

Landau, R.E. (1966). Method of installation as a factor in sand drain stabilization.
Highway Research Record, No. 133, H.R.B. Washington.

Normand, J. (1964). One dimensional consolidation of saturated clays with large strains.
M.Sc. Thesis, Univ. of London.