

A SANDY CLAY TILL - SOME PROPERTIES MEASURED DURING
CONSOLIDATION AND SHEAR

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by

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To the ladies in my life,

Laila, Reem, Yasmine and Nermin

with love

Abstract

An experimental study is carried out in the laboratory to investigate the behaviour of a low plasticity sandy clay. This work forms part of a programme to investigate off-shore gravity structure in the North Sea and to relate laboratory behaviour to model testing and field behaviour. Lower Cromer Till from Norfolk, England is used instead of the low plasticity clays of the North Sea bed. Its geological and geotechnical properties are studied and found to be similar to some North Sea soils. Undisturbed, remoulded and resedimented samples from slurry are tested. A new experimental technique is developed for the slurry samples to achieve full control of stresses and strains throughout consolidation and shearing, without involving a sampling cycle. Test results are presented so that the behaviour can be observed simultaneously in different stress domains. Normalised relationships are used to study the influence of soil structure, shearing rate, anisotropic consolidation and stress history. Water content-undrained strength relationships are also used in investigating some of the aspects of behaviour. Sampling is found to be difficult and some field tests are recommended. Minor investigations are carried out on the effects of fissures, consolidation age, undrained creep, stress path, loading increment ratio and the relationship between drained and undrained strengths.

A routine is given to evaluate design parameters of the Lower Till in certain field conditions. This routine

is based on the results of the laboratory investigation. Suggestions are put forward for the choice of a range of design ratios of moments, vertical and horizontal forces for off-shore gravity structures and model testing. These suggestions are based on known bearing capacity theories and other calculations but not on the laboratory data.

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NOTATIONS

A_f	pore pressure parameter A at Failure
AC	alternative current
(A)	anisotropic consolidation (when used in a title of a test)
BP	back pressure
c'	cohesion intercept
C_c	compression index
CD	consolidated drained triaxial test
C_s	swelling index
CT	conventional triaxial apparatus (strain-controlled)
CU	consolidated undrained triaxial test
C_u	undrained shear strength
C_v	coefficient of consolidation
D	drained shearing
DC	direct current
$E_{i,u,sec}$	initial, undrained and secant deformation moduli
ESP	effective stress path
HT	hydraulic triaxial apparatus (stress-path apparatus)
I	isotropic consolidation (when used in a title of a test)
K	ratio of σ'_3/σ'_1 during anisotropic consolidation
K_o	coefficient of earth pressure at rest = σ'_3/σ'_1 when $\Delta\sigma'_3=0$
$K_{nc,oc}$	ratios of σ'_3/σ'_1 during normal consolidation and unloading respectively.
k	coefficient of permeability
m_v	coefficient of volume change
N	normal consolidation
OD	rear-loading Bishop's type oedometer
O(x)	overconsolidation, x indicates overconsolidation ratio
OCR	overconsolidation ratio
P, P'	total and effective mean stress, $\frac{1}{2}(\sigma_1 + \sigma_3)$ & $\frac{1}{2}(\sigma'_1 + \sigma'_3)$
P'_o	mean effective stress at the start of shearing
q	a value of the shear stress $\frac{1}{2}(\sigma_1 - \sigma_3)$
R	remoulded sample
Ra	shearing rate (mm/min), 0.0001 : 0.0005
Rb	shearing rate (mm/min), 0.0050 : 0.00762
Rc	shearing rate (mm/min), 0.0500 : 0.0762

Rd	shearing rate (mm/min), 0.5000 : 0.7620
Re	shearing rate (mm/min), 5.0000 : 7.6200
RS	ring shear apparatus
S	resedimented sample from slurry
SP(1&2)	stress path (σ_1 increasing $\Delta\sigma_3=0$ & $\Delta\sigma_1=0$ and σ_3 decreasing)
TSP	total stress path
U	undisturbed sample
UU	unconsolidated undrained triaxial test
u	pore pressure
v	volumetric strain
W	water content
γ_{bulk}	bulk density of soil
γ_w	weight density of water
$\epsilon_1 = \epsilon_{v1}$	axial (vertical) strain
$\epsilon_3 = \epsilon_h$	radial (horizontal) strain
$\dot{\epsilon}$	rate of strain
ϕ'	angle of shearing resistance
ν'	Poisson's ratio
ρ	settlement
σ_a	axial stress
σ_r	radial stress
$\sigma_{vo}, \sigma'_{vo}$	total and effective vertical stress at the start of shearing
$\sigma_{vm}, \sigma'_{vm}$	total and effective maximum vertical stress
σ_1, σ'_1	total and effective major principal stress
σ_3, σ'_3	total and effective minor principal stress

Other symbols are defined where introduced in the text.

Chapter 1

Introduction

1.1 General

The recent construction of off-shore gravity structures in the North Sea prompted research into the foundation soils which are likely to be critical to design. The combined forces of loads and waves imposed on these structures are orders of magnitude higher than what is usually encountered for on-shore structure. More understanding of the foundation soils is therefore an important safety and economical factor.

Undrained loading of clayey foundation soils is the critical design condition for the above structures (Bjerrum 1973 and Vaughan et al. 1976). The clayey layers in the North Sea bed vary in plasticity. In this research some of the fundamental soil properties of a low plasticity sandy clay have been studied.

Testing soil samples from the North Sea bed is impractical and on-shore soil, similar to the offshore sediments, has been used instead. The experimental programme has been aimed at trying to broadly investigate the properties of this soil and set up a frame work for further research.

Contemporary research work on the same soil (e.g. Gens 1978 on static conditions, Hight, 1978 and Takahashi 1978 on cyclic loading and Martins 1978 on model testing) has

been carried out. Some of the tests in this work have been directed so that an interrelationship between the results of all these investigations can be achieved, leading to a deeper understanding of the behaviour of this soil.

1.2 Layout of the Thesis

In Chapter 2 the geology of both the North Sea and that of the Norfolk coast, England, from where the on-shore soil (the Lower Till) was obtained, is discussed. The Lower Till is a product of the Pleistocene and the soils of the North Sea bed which are of engineering interest as foundation soils are the product of the Pleistocene and the Holocene. Both soils also show closely similar geotechnical properties. This justifies the use of the Lower Till in this research to represent a low plasticity foundation soil in the North Sea.

Chapter 3 describes the testing programme and the types of samples and pieces of equipment used in this work. It also explains the different methods of consolidation and shearing followed during testing. In the last section of this chapter a description is given of each of the groups of tests carried out in the research.

In Chapter 4 the sampling of the Lower Till is described and details are given of the apparatus used in laboratory testing and the ways they are calibrated, operated and controlled. The steps for preparing various types of samples, consolidating and shearing are described in detail. In this chapter the new (and advantageous) experimental tech-

nique of testing resedimented slurry samples, which is used in this research, is explained.

Chapter 5 presents the results of oedometer and ring shear tests and explains the way the results of the tri-axial results are presented in Appendix 3.

In Chapter 6 the data obtained from all test results are compiled and interpreted in studying several aspects of soil behaviour. The causes of scatter of the test results are discussed then the influence of soil structure, rate of shearing, anisotropic consolidation and stress history are studied. Predicting ultimate strength when the soil has undergone only small straining is also investigated. The experimental data allowed minor investigations to be carried out into the effects of fissures, consolidation age, undrained creep, stress path and the relationship between drained and undrained strengths.

Chapter 7 consists of two main parts. The first part serves as a summary for chapter 6 and in the same time is presented as a methodical way of obtaining soil parameters for design should the Lower Till or a similar soil (which is deposited in a similar way as described in this section) be used as a foundation soil. The second part of this chapter presents possible ranges of design ratios of moments, vertical and horizontal forces for off-shore gravity structures in the North Sea or for model testing. These ratios are based on manipulating available data of existing structures in the North Sea into bearing capacity equations and into simple relationship of settlements and contact pressures.

CHAPTER 2

THE NORTH SEA BED AND NORFOLK SOILS

2.1 THE SOILS OF THE NORTH SEA BED

2.1.1 Geology of the North Sea

The North Sea bed comprises Quaternary sediments which in the central part of the North Sea reach a depth of 500 - 700 m. (Løken 1976), i.e. much thicker than the depths considered to be affected even by the largest of structures on or in the North Sea bed. The Quaternary sediments consist of glacial, interglacial and late Holocene deposits.

The Pleistocene glacial and interglacial sediments are either a product of land or water ice since most southern parts of the North Sea were dry land during peak periods of glaciation (Bjerrum 1973, Eide 1974, Løken 1976).

During these peak periods the world balance of sea water levels was greatly affected because of the amounts of water absorbed within the freezing bergs which caused the withdrawal of water from many areas, the southern parts of the North Sea being one.

The present water depths of the North Sea range from 70 m. in the south to 170 m. in the north. See table (2-1), and reach depths of 200 - 300 m. in a strip that

runs North-South not far to the west of the Norwegian coast. During the periods of peak glaciations the North Sea water levels were 100 - 300 m. shallower than today (Bjerrum 1973, Eide 1974). Løken (1976) considered that there were areas of dry land where the present water depths are 120 - 140 m. and presents maps showing the advance of water during the last 17000 years.

In the southern part of the North Sea the sediments were exposed during the glaciations and so were subjected to wind and river erosion then to beach erosion as the sea advanced from the north then to wave and current erosion as a sea bed.

The underlying glacial and interglacial deposits have undergone several cycles of sedimenting, loading and unloading producing a complicated stress history which has left most of the clayey soils overconsolidated. Another minor result of the Pleistocene glaciation is that after the retreat of the ice around 10000 years before present, the ground levels in parts of Northern Europe are heaving up while other parts are subjected to downwarping.

The North Sea bed is believed to be settling; this has been measured to be 5 mm./year in the north of the North Sea (Løken 1976). This settlement can also be deduced from extrapolation of known maps of on-shore contours of land level variations.

A map for the North Sea bed based on results from

shallow samples was given by Bjerrum (1973), fig.(2-1), while another more detailed map for the northern part of the North Sea was given by Løken (1976), fig.(2-2). On both maps the position of the important oil and gas fields are shown.

2.1.2 Soils of the North Sea Bed

It is shown in fig.(2-1) and (2-2) that the largest part of the North Sea bed including the sites of the oil and gas fields is covered by sand. This sand layer is actually thin and may disappear in places and is underlain by layers of glacial tills and bands of interglacial sands and silts.

Submarine banks of sand and gravel and depressions filled with soft normally consolidated clays are present in different locations.

The bedrock is found along the coasts of Norway, the Orkney and the Shetland Islands; these areas were subjected to intensive glacial erosion.

A more detailed description of the bed soils follows:

- 1) The submarine banks are sands, gravels and rocks. They are outcrops of moraine with outwash fan complexes. These coarse materials are usually the result of temporary stationary periods of ice sheets or of the

ploughing up of older tills. Bjerrum (1973) attributes these coarse materials to the fact that during exposure to wind and river erosion while dry land the fines were washed away leaving only the coarse materials behind.

- 2) The depressions in the North Sea bed are valleys between end moraines. They are generally filled with normally consolidated clays and silts of recent age. These depressions are found along the Norwegian Trench beneath depths of water of 200 - 300 m. They also occur in several other locations, refer to the clay areas shown in figs. (2-1) and (2-2). Eide (1974) measured depths of 10 - 20 m. of these soft clays in the middle British side of the North Sea; he also reported a value of C_u of about 10 kPa for this soil. Løken (1976) reported information from several deep borings and found out that the relationship between C_u and depth is close in these clays to the theoretical values for normally consolidated clays of similar plasticity.
- 3) The surface fine sand layer can reach about 8 m. in thickness and only a fraction of a

metre elsewhere. There are locations where this layer disappears and large boulders are found sitting on the tills with little or no embedding, Løken (1976).

In locations where this layer is thick (4 m. or more), it is believed that the top sand is resedimented from the reworking of glacial deposits elsewhere by current and tidal waves which occurred in late or post glacial times whereas the lower layer is thought to be a glacial outwash. This has generally been proved by crude measurements of C^{14} reported by Løken (1976).

The fine sands from this layer obtained from different borings are similar in uniformity and composition and their gradings lie within the range of gradings of sands susceptible to liquefaction, Bjerrum (1973) and fig. (2-3). The relative density of this layer of fine sand is 100% as measured in the laboratory and in-situ by specially designed Fugro and N.G.I. cone tests (Bjerrum 1973 and Eide 1974). The cause of this very high density is the wave and current compaction and probably, to some extent, preloading.

- 4) The clayey layers underlying the fine sand are sandy and silty clays, usually over-consolidated, presumably by ice sheets, with frequent bands and layers of coarser-grained soils. They are non-laminated when deposited during periods of peak glaciation and more laminated when deposited in warmer interglacial climates. In the North Sea bed these tills are very stiff to hard in consistency. The upper layers are also non-laminated and associated with the last ice advance during the latest glaciation. The lower layers are more laminated. These tills are generally not fissured. Their C_u is equal to or greater than 500 kPa (Bjerrum 1973). Eide (1974) measured C_u of 100 - 800 kPa in-situ for these tills. It is obvious that there is a large scatter in the values of C_u for a location (Eide 1974). They have wide range of plasticity; their P.I. range from 12 - 40% and their natural water contents are near their plastic limit 12 - 25% and they may be less than their plastic limit in places (Bjerrum 1973, Eide 1974 and Løken 1976). There is no evidence of cementation in these tills and the low natural water contents are due, mainly, to overconsolidation. The overconsolidation

ratios are higher at sites of lesser present water depths and at these sites the tills are more brownish-grey due to possible oxidisation while these sites were dry lands. The varying mineralogy and the clay fractions of these tills are given by Løken (1976) and Hight (1978).

In studying the foundation soils for gravity structures in the North Sea the bed was assumed to be sand underlain by tills. Furthermore the behaviour of the upper sandy layer was excluded as critical and the undrained behaviour of the underlying tills was taken to be the most critical for design. In this research only low plasticity tills will be studied (see Vaughan et al 1976).

Obtaining representative samples for this research from the North Sea bed would have been difficult and expensive and a more convenient on-shore site was needed for this purpose. Both the off and on-shore soils should be as close as possible in their geological histories and geotechnical properties in order to allow for this substitution.

2.2 THE NORFOLK SITE

2.2.1 Geology of the Site

The on-shore site chosen was the coast line cliffs along the Northern Norfolk coast near Happisburgh. Samples of a glacial till of low plasticity from the foot of the cliffs just above the beach level were taken.

These cliffs represent the finest development of glacial deposits in the North-Eastern part of Norfolk. They are exposed in a series of sections from Weybourne in the west to Happisburgh in the east.

The geological sequence in the exposed cliffs above the beach line is Pleistocene and Holocene deposits the details of which have been given by Boswell (1923), as follows;

Epoch	Description
Holocene	. Alluvium, brown sand, shingle ...etc.
Pleistocene	. Post-glacial river terraces, gravels, sands and loams.
	. Glacifluvial gravels
	. Glacial sands, gravels and Brickearths
	. North Sea Drift (sediments of the first glacier to cover this part).

The above layers are underlain by the latest of the Pliocene namely; Arctic Freshwater bed, Shingle and Leda Myalis bed. Very rarely do these layers rise above the beach level. Pliocene stratam invariably underlay the Pleistocene.

It is worth mentioning that the thickness of the exposed layers varies along the coast and some layers disappear almost completely or become too small to be positively identified at some locations.

It was the task of the research team to locate a site from which samples of a low plasticity till similar in several aspects to the low plasticity tills of the North Sea bed could be obtained and the coastal cliffs near Happisburgh were chosen.

These exposed cliffs above the beach line consist almost entirely of the North Sea Drift overlain by thin recent layers. In this particular site the North Sea Drift consists of Lower and Upper Tills separated by an interglacial layer. These three layers together are commonly known as the Cromer Till. The two tills are unsorted and unlaminated contrasting with the laminated interglacial strata. Blocks of the Lower Till were sampled for this research.

2.2.2 The North Sea Drift

2.2.2.1 The Ice Age in Britain

During the Pleistocene which started two million years and ended 10000 years BP with the beginning of the Holocene, several glaciations covered Britain among other places in the Northern and Southernmost parts of the globe.

The glaciers advanced, halted, retreated and sometimes readvanced. The Pleistocene was not an ice age throughout since inbetween episodes of glaciations warm interglacial periods prevailed, some of which resembled today's climate and probably even warmer.

2.2.2.2 Glaciation in East Anglia.

The number of glaciations and the differentiation between mere ice advances and major glaciation in East Anglia and indeed elsewhere in Britain and the correlation with North-Western Europe has been, and still is, a subject for debate.

Solomon (1937) identified four glaciations and so did Baden-Powell (1948) who also clearly identified the deposits during those four peak glaciations. Slater (1927) suggested only two major glaciations. West and Donner (1956) also claimed that there were only two major glaciations and that the other two mentioned above were mere advances within the same glaciations. They also related each of the two big glaciations to their contemporaries on the European Continent.

Kazi and Knill (1969) suggested that the various glacial sediments in the north Norfolk coast are products of a single glacial period. Hutchinson (1976) mentioned three major glaciations in Britain and confirmed the aforementioned argument that only the earliest glaciation covered that part of Norfolk.

Spark and West (1972) gave a review of all the events related to the ice age. They also identified three major glaciations and mentioned that only the earliest covered that part of Norfolk.

Despite all the arguments not one scholar disputes the fact that our Lower Till is the product of the North Sea Drift of the Earliest glaciation.

2.2.2.3 Formation and Movement of the North Sea Glaciation.

The North Sea Glacier was most probably formed over the present Norway and parts of west Scandinavia and North-West Germany and it advanced over the North Sea and invaded the North Norfolk coast. The present Cromer Ridge is its terminal moraine (Boswell 1923). In Fig.(2-4), the extent of the North Sea Drift in Norfolk is shown.

Boswell (1914), Solomon (1932) and Dhonau and Dhonau (1963) agree that the North Sea ice invaded Norfolk from the East or North East. Baden-Powell (1948) proved this direction from the orientation of the contained pebbles in the Till.

However, West and Donner (1956) stated that the direction of movement of ice sheets was from North-West to South-East. fig. (2-5), and Kazi and Knill (1969), stated that the ice sheets came from the east by the evidence of field axis orientation and also stated that they were moving

and thinning towards the South-East.

These facts urge the author to suggest possible routes for the ice sheets as shown in Fig. (2-6); these indicate ice sheets forming at the North East then moving and spreading across the North Sea and North Western Europe. The evidence suggests that the sheets moved straight to the south west (routes 1) in the early stages when most of the Lower Till was deposited. After the ice sheets spread more to the west they were deviated by the Cheviot mountains and the Pennine hills and lastly by the Cromer ridge (route 2). After moving along the Cromer ridge the ice sheets halted after covering the whole of the present Norfolk coast and the sea to its west.

The absence of any Neocomian or Jurassic matter and the almost non-existing chalk at the Lower Till rules out the possibility of an early invasion from the west.

The link with Scandinavia in the Lower Cromer Till of the North Sea Drift has been established by all researchers from the erratics found in the till: Solomon (1932) attributed the similarity in the phenomena between the Norfolk coast and those at Möen in Denmark to the agency of ice. Dhonau and Dhonau (1963) compared similar structures to the Lower Cromer Till in Lønstrup in Denmark.

The rocks and shells ploughed up from the North Sea bed and found in the Lower Till establishes the route of the ice sheets across the North Sea. Boswell (1916) and Solomon

(1932) also identified Tertiary deposits from the North Sea bed in the magnetic fraction of the minerals of the Lower Till.

Baden-Powell and West (1960) reported Professor Steer's work on the evolution of the North Norfolk coast by experimenting on the movement of artificial radioactive pebbles and his suggestion that some shingle in the Lower Till is brought up to the beach from deeper parts of the North Sea bottom.

As for the erratics from Scotland, they have been reported by Boswell (1923), Solomon (1932), Baden-Powell (1948) and Kazi and Knill (1969). They are subordinate in quantity to the Scandinavian erratics and sometimes not mentioned at all by researchers.

The North Sea Glaciation has later undergone more complicated succession of movements and deposition but it is in the author's opinion that as far as the Lower Cromer Till is concerned within the limits of this research, the above general explanation is adequate.

2.2.2.4 Rocks, Shells and Mineralogy of the Lower Cromer Till.

A compilation of the rocks and shells found in the Lower Cromer Till is given in Appendix (1). A recent morphological description of a collection of pebbles collected from block samples of the Lower Till is also given in Appendix (1).

2.3 COMPARISON BETWEEN THE NORTH SEA TILLS OF LOW
PLASTICITY AND THE LOWER CROMER TILL FROM
(NEAR) HAPPISBURGH.

2.3.1 Geological Histories

Both tills belong to the Pleistocene. The Happisburgh Lower Till was deposited during the Cromer Advance which is the earliest advance in the British North Sea Glaciation occurring early in the North-Western European Elster Glaciation (West and Donner 1960, Sparks and West 1972, Hutchinson 1976). This advance took place about one million years before present.

The upper 150m. of the North Sea bed which are of engineering interest were deposited during the last glaciation known in Britain as the "Devensian" and in the Continent as the "Weichselian" glaciation, and also during the Holocene.

The "Weichselian" glaciation started about 70000 years before present and ended 10000 years ago. (Bjerrum 1973 and Løken 1976).

Both soils have the same general composition and they are both unstratified and unlaminated due to similar modes of deposition during peak periods of ice advance. Both soils are overconsolidated, the Happisburgh Lower Till by loading and some unloading from inundation, erosion and instability of the cliffs (see Hutchinson 1976), while that

of the North Sea bed from melting out of ice sheets of enormous load and also from the wave and current compaction.

The present in-situ overconsolidation ratio of the Lower Till at Happisburgh was roughly calculated from estimating a height of 61 m. of ice sheets to have covered that site at peak periods of the North Sea glaciation (Kazi and Knill 1969). The height of the cliffs above the level of sampling ranged from 10m: 15m at the time of sampling. The wet density was measured by the author to be 22 KN/m^3 and by Hutchinson (1976) to be 20.9 KN/m^3 . Hence the overconsolidation ratio ranged between 1.84: 4.22 depending on the assumed density of the ice sheets.

2.3.2 Geotechnical Properties

i) Gradation: Gradation curves of different samples of the Happisburgh soils are shown in Fig.(2-7), while the band of gradation within which the low plasticity tills of the North Sea bed lie is shown in Fig. (2-8). On the same Fig.(2-8) the gradation curves of the two bulk samples of Happisburgh used in this research are also shown. The similarity in gradation is apparent.

ii) Atterberg Limits and Clay Fraction: In table (2) the Atterberg Limits and the Clay Fraction of both soils are shown. Kazi and Knill (1969) and Hutchinson (1976) reported higher clay fraction of 20: 25% for the Lower Till at Happisburgh.

iii) Stones and Mineralogy of Clay: In Appendix (1) a compilation of the analyses of stones, shells and clay fraction of the Lower Till at Happisburgh is given. In Appendix (2) recent analysis of low plasticity tills from Happisburgh and the bed of the North Sea is given. Again the similarity is apparent.

iv) $C_v - \sigma'_v$ Relationship: This is shown in Fig.(2-9) For the Happisburgh tills. This figure shows the results of 75 mm diameter and 225 mm diameter oedometer tests.

v) Specific Gravity of the Soil Particles: The mean value of G_s calculated by the author is 2,6185. Hutchinson (1976) reported a value of 2,6500 for a similar strata from a different location.

Other tills of the North Sea Drift in Norfolk which relate more closely agewise to the low plasticity tills of the North Sea bed differ in composition especially by the existence of a larger percentage of chalk or in being higher in plasticity.

2.4 SUGGESTION FOR FURTHER RESEARCH

For further and probably more representative information about the Lower Cromer Till and consequently the similar beds in the North Sea floor, in-situ shear strength measurement tests are recommended.

The site from which the samples used in this research

were taken was a cliff face (refer Chapter 4) and is not suitable for such in situ testing. An inland site with an undisturbed stratum of this Lower till should be found, (since sampling and testing undisturbed samples of this till in the laboratory, though obviously important, proved rather difficult).

A site in an area that lies not more than 15 miles inland and runs parallel to the Norfolk Coast from Weybourne to Happisburgh is recommended.

It is not recommended, however, to locate a site in a deeper inland area since although the Cromer Ridge was the terminal moraine of the North Sea glaciations it is not known whether the first ice advance which sedimented the Lower Till actually reached the Ridge that far to the west.

For the site work, the Lower Till should be readily classified from the information given in this chapter. Care should be practiced not to mix it with the Upper Till which resembles the Lower Till in many aspects. The most important difference between the two is the almost complete absence of chalk in the Lower Till.

FIELD	DEPTH m.
EKOFISK.	70
FRIGG	100
BRENT	150

TABLE(1.1) SEA WATER DEPTHS AT THREE SITES IN THE NORTH SEA

SOIL	NOTES	L.L. av. %	P.L. av. %	CLAY FRAC. %	ACTIVITY %
BLOCK SAMPLES FROM HAPPISBURGH LOWER TILL.	BLOCK V 1	25,30	13,48	12,00	0,99
	BLOCK V 2	26,10	14,10	15,00	0,80
	BLOCK V 3	26,39	13,39	15,00	0,87
	BLOCK V 4	25,00	12,75	10,00	1,23
	BLOCK V 5	25,00	13,00	13,50	0,89
	BLOCK V 6	25,30	12,50	14,00	0,91
	BLOCK V 7	25,50	12,05	11,00	1,10
	BLOCK V 8	25,40	12,80	10,00	1,26
	BLOCK V 9	25,10	12,55	14,00	0,90
	BLOCK V 10	24,95	15,00		
	BLOCK V 11	23,60	12,75		
	BLOCK V 12	24,85	13,61		
	MIXID SOIL (V 10+11+12). MODEL TEST	25,50	12,50		
LOW P.I. TILLS. NORTH SEA BED (after HIGHT 7).	STATISTICAL CALCULATIONS :				
	FULL RANGE	20 : 33	10 : 18	21 : 25	
	MEAN RANGE	26 : 30	12 : 14		

TABLE(2.1) ATTERBERG LIMITS AND CLAY FRACTION OF HAPPISBURGH LOWER TILL AND LOW PLASTICITY NORTH SEA TILL .

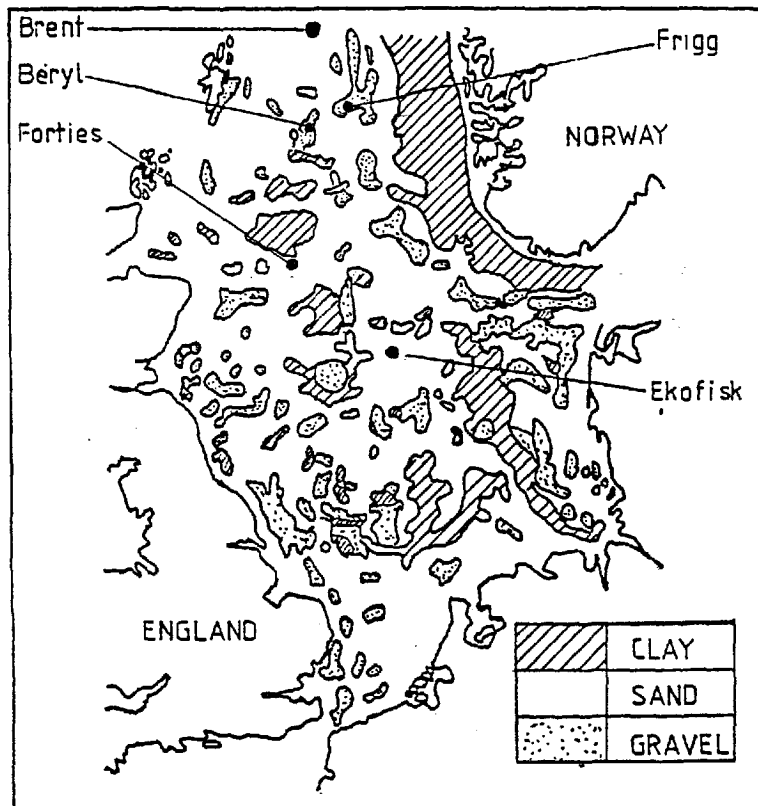


FIG:2-1 MAP OF THE NORTH SEA BED (Bjerrum 73)

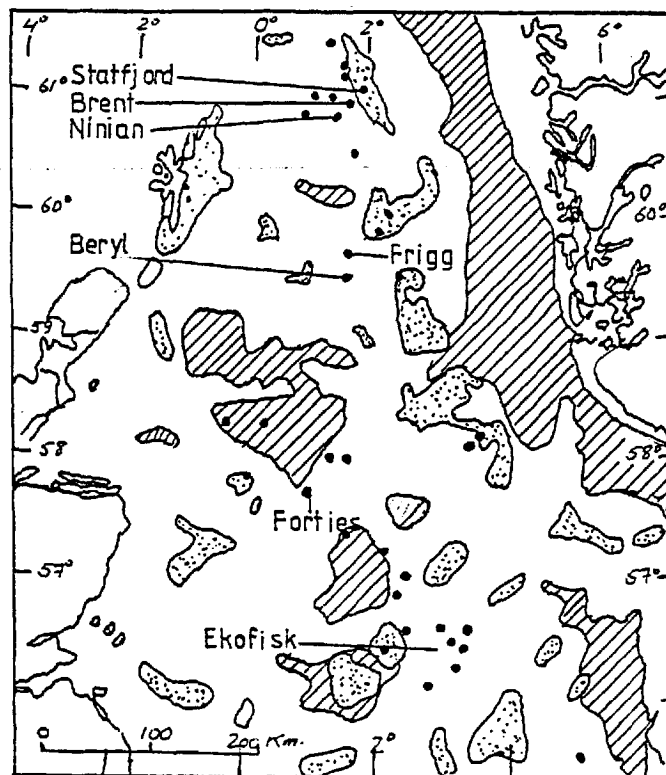


FIG:2-2 MAP OF THE NORTHERN NORTH SEA BED (by Eden 75 after Løken 76) .

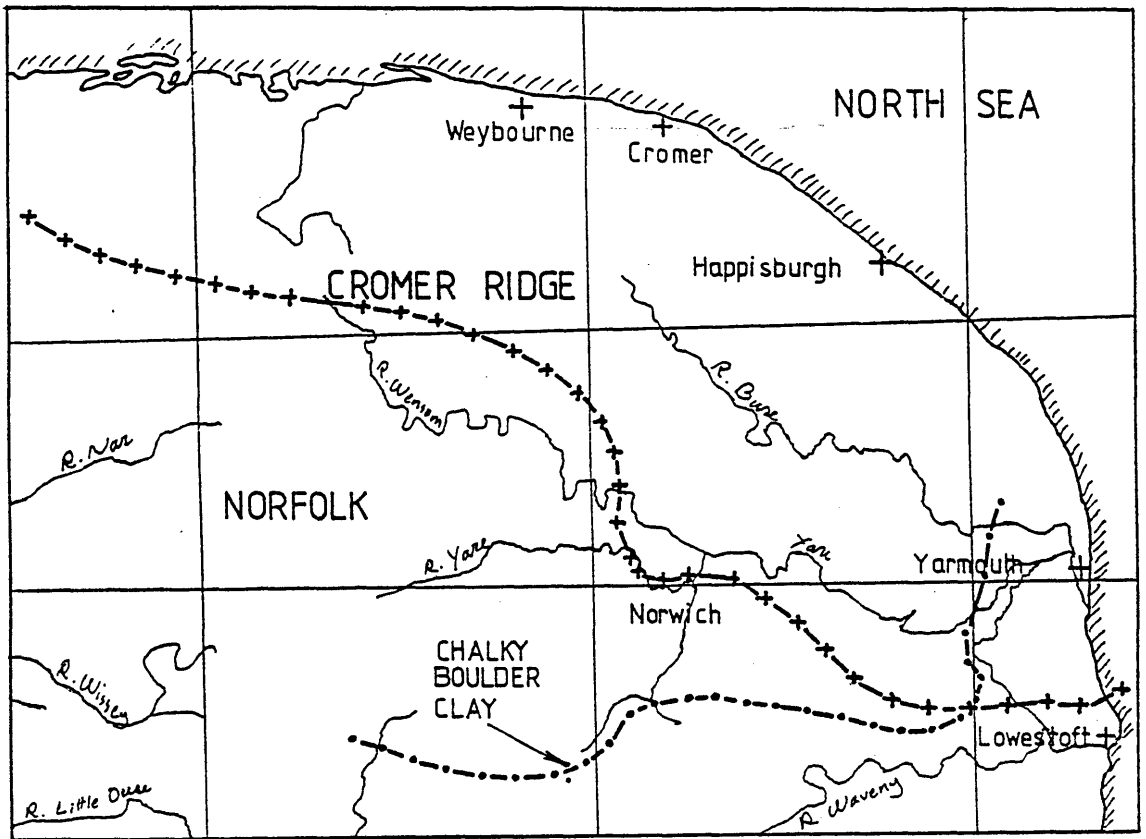


FIG:2-4 THE NORTH SEA DRIFT IN NORFOLK (Boswell 14).

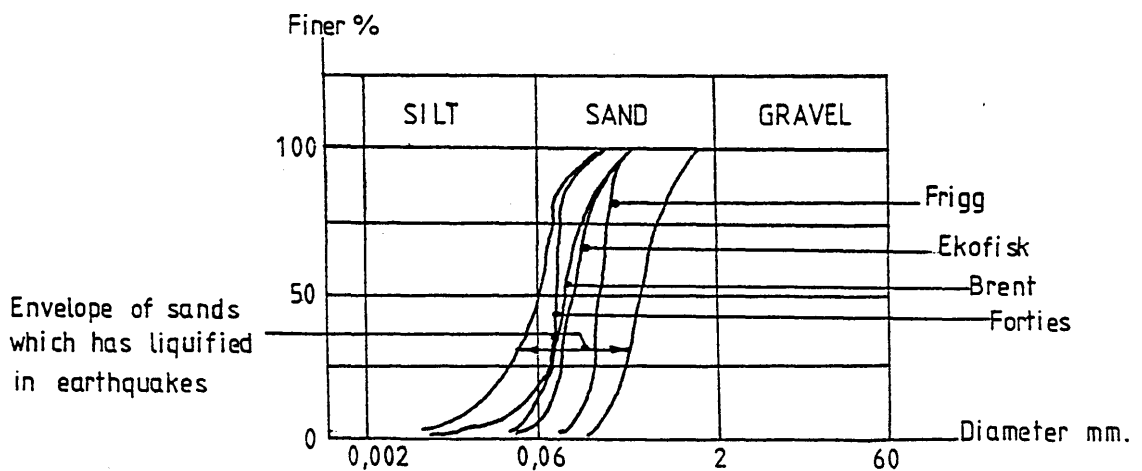


FIG:2-3 GRAIN SIZE DISTRIBUTIONS OF SAND SAMPLES OBTAINED AT SHALLOW DEPTHS FROM DIFFERENT SITES OF THE NORTH SEA (Bjerrum 73).

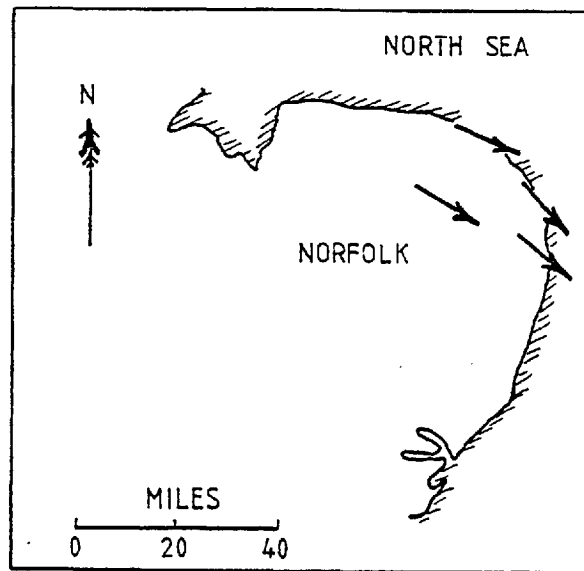


FIG:2-5 DIRECTION OF PREFERRED ORIENTATION IN TILLS OF THE CROMER ADVANCE (West & Donner 56).

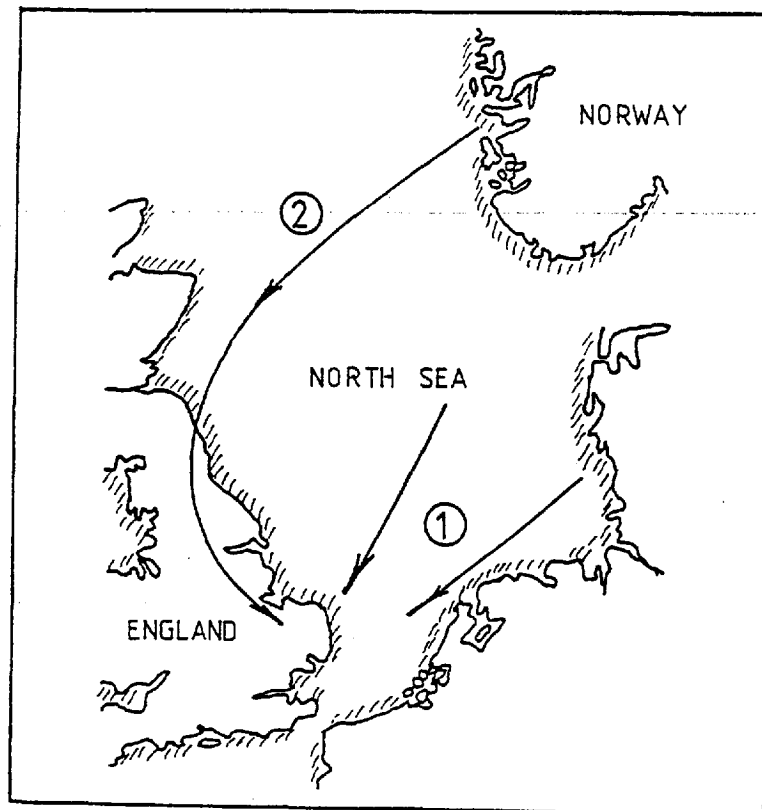


FIG:2-6 DIRECTION OF ADVANCE OF THE NORTH SEA GLACIATION

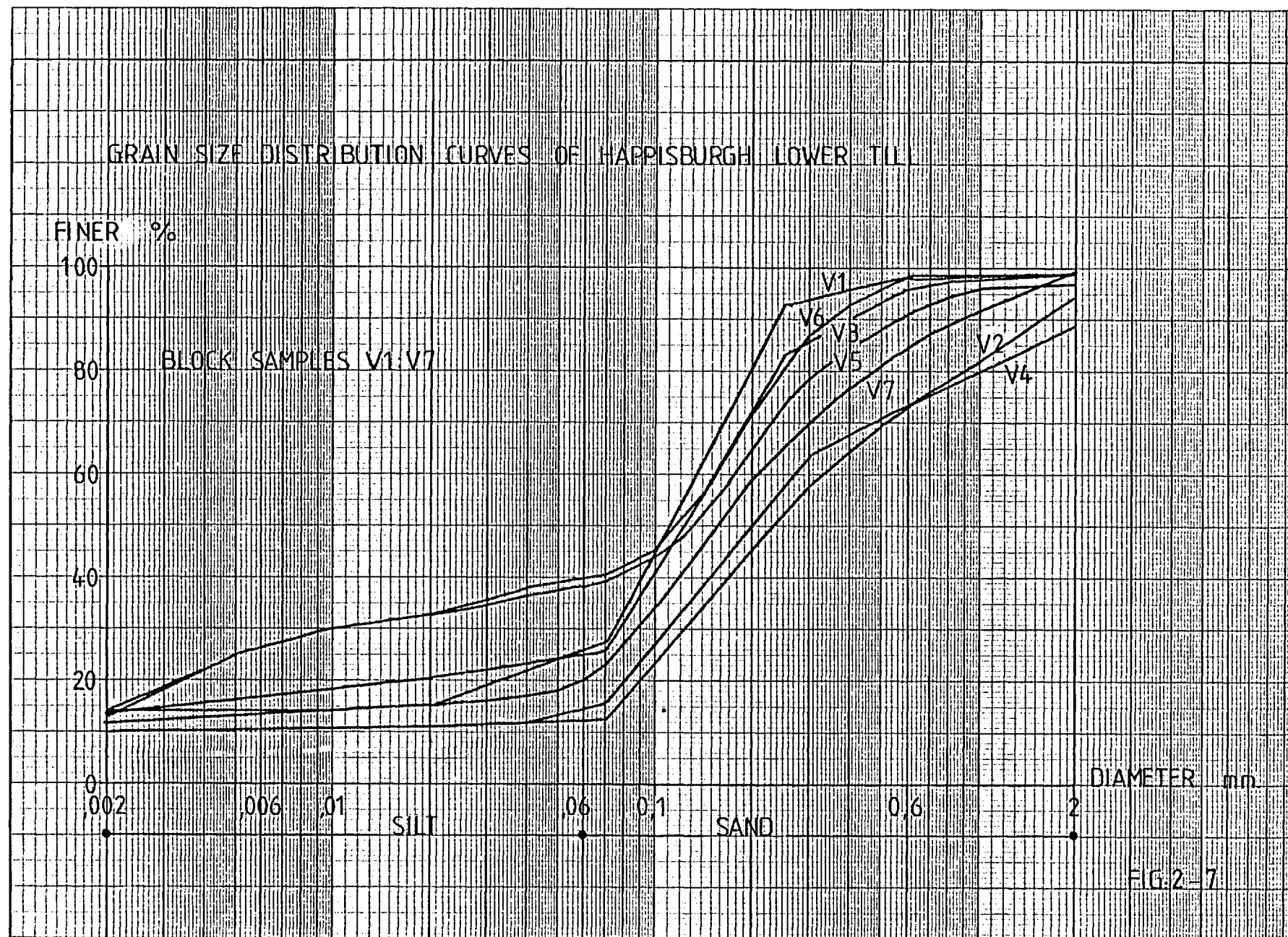


FIG. 2-7

GRAIN SIZE DISTRIBUTION CURVES OF LOW PLASTICITY TILLS FROM THE NORTH SEA BED (after HIGHT 77)

FINER %

100

80

60

40

20

0

.002

.006

.01

SILT

.06

.1

SAND

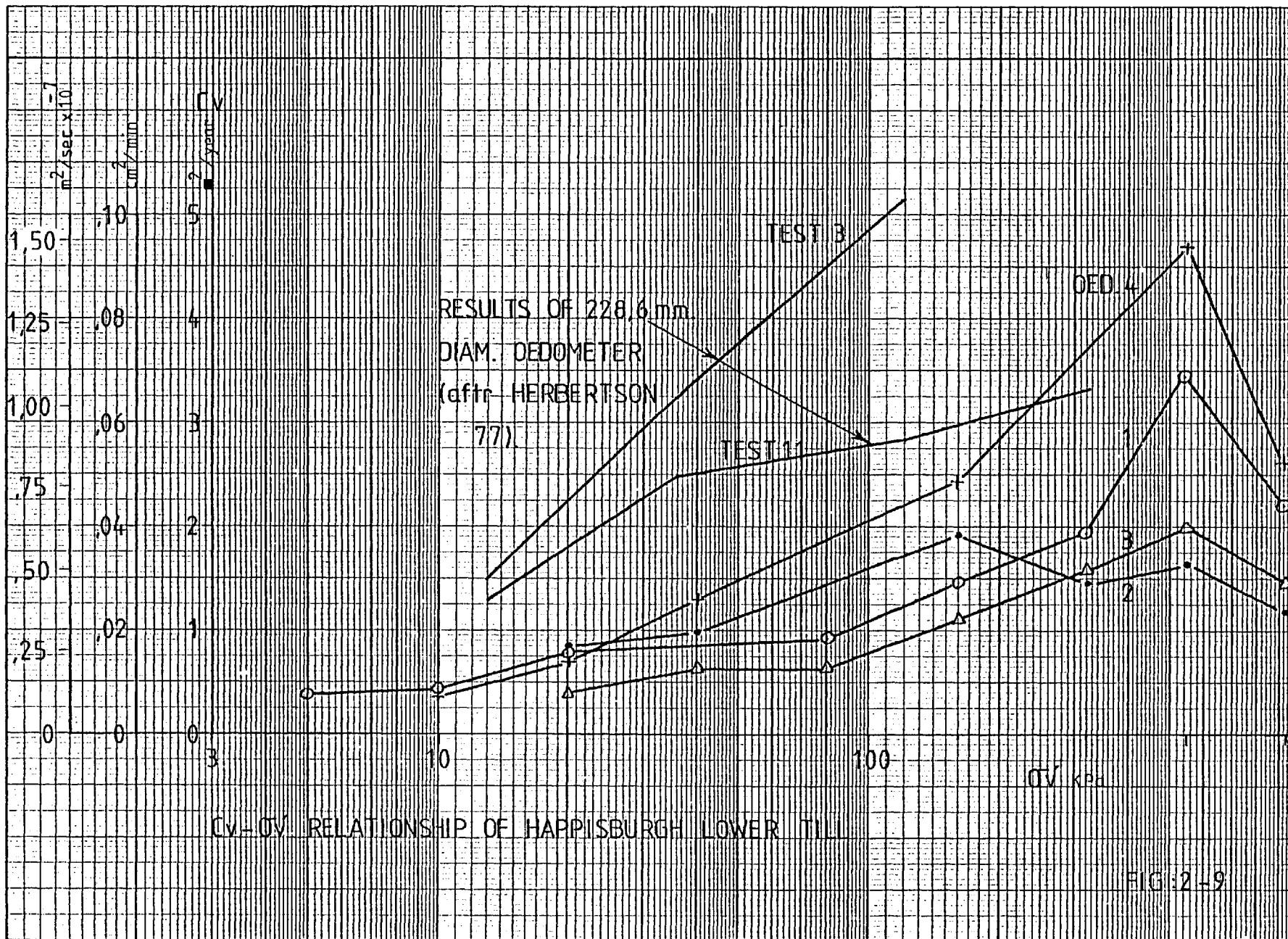
.6

2

DIAMETER mm

HARRISBURGH TILLS
USED

FIG 2-8



CHAPTER 3

TESTING PROGRAMME

3.1 INTRODUCTION

As mentioned in Chapter 1 the work described in this thesis formed part of a programme of research into the behaviour of North Sea structures and their foundations. The programme of laboratory testing described in this chapter, which constituted the main bulk of the author's research, was directed towards providing data for understanding the in-situ behaviour of a typical North Sea soil type and for predicting and analysing the performance of the model footing.

In Chapter 2 the selection of a material to represent one of the North Sea soils was discussed. The stress history of this material, (the Lower Till from Happisburgh), its mode of deposition and its in-situ stress state are uncertain; this is also the case for North Sea soils. Taking this into account together with the difficulty of obtaining undisturbed samples, referred to in Chapter 4, it was decided to test soils which were either remoulded or consolidated from a slurry, in addition to the undisturbed material.

The use of samples starting from a slurry eliminated as many variables as possible and permitted good control of tests; parameters which needed studying could thus be singled

out. As described in Chapter 4 the samples from slurry were consolidated and sheared inside the triaxial cell throughout and were not at any time, subjected to stress or strain changes which were not controlled.

For similar reasons, viz obtaining uniformity and consistency, it was decided to consolidate soil beds for the model work from slurry.

In addition to studying the compressibility and shear behaviour tests were selected to enable the effects of the following to be studied in detail:-

- i - rate of shearing
- ii - anisotropic consolidation
- iii - sampling disturbance
- iv - stress history
- v - hyperbolic transformation and the evaluation of deformation modulus.

In the course of trying to fullfil the above, other aspects were also examined which included:-

- i - effect of Fissures
- ii - effect of consolidation age
- iii - effect of undrained creep
- iv - effect of loading increment ratio
- v - effects of stress path
- vi - drained and undrained shear strengths

Owing to the obvious broad nature of this research

and the neccessity to establish, at least, a trend for the effect of behaviour of each of the above factors large numbers of tests had to be performed. Tests of similar type or with a similar purpose have been grouped together for convenience as described below.

3.2 NOTATIONS

It is important at this stage to put down the notations used to identify the tests and the groups to which they belong, as well as the type of test and apparatus.

3.2.1 Types of Soil Samples

<u>Type</u>	<u>Symbol</u>
- Undisturbed	U
- Remoulded	R
- Resedimented (Slurry)	S

All the soil tested were "Lower Till" from Happisburgh and all triaxial samples were 38 mm in diameter and 76 mm long unless otherwise indicated.

3.2.2 Types of Apparatus Used

- Conventional strain-controlled triaxial	CT
- Hydraulic stress-path triaxial	HT
- Rear-loading Bishop-type oedometer	OD
- Ring shear	RS

3.2.3 Types of Consolidation and Shearing Process

Type

- | | |
|---|------|
| - Isotropic Consolidation | I |
| - Anisotropic Consolidation | A |
| - Normal Consolidation | N |
| - Over consolidation, the number next to the symbol "O" indicates overconsolidation ratio in terms of effective vertical stress | O(x) |
| - Drained Shearing | D |

The above symbols were mostly used for tests on residimented material. Two points here have to be clarified:-

1) all shearing in triaxial tests was carried out in undrained conditions unless the symbol D was added at the end of the title to indicate drained shearing.

2) the symbol P in a test title indicated the mean effective pressure $\frac{1}{2}(\sigma_1' + \sigma_3')$ at which shearing started (P'_0). This symbol "P" is succeeded by the value of this mean effective pressure in kilo-Pascals "kPa".

3.2.4 Types of Triaxial Tests

Symbol

Type

- | | |
|----------------------------|------|
| - Unconsolidated Undrained | - UU |
| - Consolidated Undrained | - CU |
| - Consolidated Drained | - CD |

3.2.5 Types of Imposed Total Stress-Paths (TSP)
During Shearing.

<u>Type</u>	<u>Symbol</u>
- Vertical stress increasing and horizontal stress is kept constant.	1
- Vertical stress is kept constant and horizontal stress decreasing.	2

3.2.6 Different Rates of Shearing.

<u>Rate mm/min.</u>	<u>Symbol</u>
- 10^{-4} : 10^{-3} (mainly 0.0001 and 0.0005)	Re
- 10^{-3} : 10^{-2} (mainly 0.0050 and 0.00762)	Ra
- 10^{-2} : 10^{-1} (mainly 0.0500 and 0.07620)	Rb
- 10^{-1} : 10^0 (mainly 0.5000 and 0.76200)	Rc
- 10^0 : 10 (mainly 5.0000 and 7.62000)	Rd

The details of every individual test are shown in Chapter 5.

3.2.7 Examples

1) A test titled A04P160D indicates anisotropic overconsolidation giving a ratio of 4 between maximum and present vertical effective stresses ($\frac{\sigma_{vm}'}{\sigma_{vo}'}$). The present effective mean pressure P'_o is 160kPa and the soil was sheared drained to failure .

2) A test titled IP80 indicates isotropic consolidation to a mean effective pressure of 80 kPa followed by undrained shearing.

3.3 DESCRIPTION OF GROUPS OF TESTS

1) Group "G" included the first set of tests carried out in this research, table (3-1). Undisturbed and remoulded samples were tested; some undisturbed samples were tested and later remoulded with water added and retested after consolidation, under similar conditions. They were tested with a wide range of shearing rates. Artificially fissured samples were only tested in this group. All tests of this group were carried out in Hydraulic Stress-Path Apparatus with no reason but to acquaint oneself with the new testing technique. Actually conventional strain-controlled triaxial apparatus would have sufficed. Results of this group served to furnish preliminary ideas of the characteristics of the soil. These results were later used in studying several specific properties. The tests of this group numbered 19 in all and are arranged in table (3-1) according to soil type and rate of shearing.

- 2) Group "NS" consisted of only three tests, one on an undisturbed sample and two on remoulded ones from the same material from the North Sea bed. This early set of tests helped to establish the geotechnical similarity between the "Lower Till" from Happisburgh and the low plasticity tills from the "North Sea" bed. They were performed as in group "G" in the new Hydraulic Stress-Path triaxial apparatus with two rates of shearing. The tests are tabulated in Table (3-2).
- 3) Group "WCR" was designed mainly to study the various effects a change in the rate of shearing might have on the behaviour of the soil under undrained conditions. It also established the relationship between the water content and undrained strength as well as giving a broad spectrum of effective stress paths. All tests, 35 in all, were performed on remoulded soil in conventional triaxial apparatuses. Four rates of shearing were used, each differing by an order of magnitude from the other. Some later tests were added to this group the aim of which was to give a more definite answer to the problem of the rate effect. Tests of this group are tabulated in table (3-3).

- 4) Group "V" (table 3-4) was performed to answer the question about how variable the soil was in-situ and how would this variability affect the consistency of the laboratory test results? Unusual stress paths were chosen mainly as a time saver, however they served other purposes later. Nine tests were performed on undisturbed and remoulded samples from four different blocks of the soil. An undisturbed sample from each block was tested then remoulded and tested similarly at least once. Hydraulic Stress-Path triaxial apparatus were used and the samples were sheared in a stress-controlled way under drained conditions and with slow rates. (see item 4.3.2).
- 5) Tests of group "IN" were the first of many tests performed on reconstituted samples from slurry. In group "IN" samples were isotropically consolidated to various consolidation pressures then sheared under undrained conditions. Two shearing rates were used and the tests were 15 in all. Hydraulic Stress-Path triaxial apparatus were used throughout. The tests are tabulated in table (3-5).
- 6) Tests of group "IO" were on reconstituted samples which were isotropically loaded and unloaded to

various stress levels then sheared under undrained conditions. Two shearing rates were used. 17 tests were run in Hydraulic Stress-Path triaxial apparatuses and are tabulated in table (3-6).

- 7) Tests of group "AN" were on reconstituted samples which were anisotropically consolidated to different consolidation pressures and then sheared either under drained or undrained conditions. Only one shearing rate was used for either drainage condition. Eight tests were run in Hydraulic Stress-Path triaxial apparatus and are tabulated in table (3-7).
- 8) Tests of group "AO" were on reconstituted samples which were anisotropically loaded and unloaded to various stress levels then sheared under either drained or undrained conditions. Only one shearing rate was used throughout. 9 tests were run in Hydraulic Stress-Path triaxial apparatus and are tabulated in table (3-8).
- 9) Tests of group "SMP" were on reconstituted samples in Hydraulic Stress-Path triaxial apparatus. They were designed to study the effect of sampling on the soil. One shearing rate was used and shearing was done under undrained conditions.

4 tests were run and are tabulated in table (3-9).

- 10) Tests of group "ST" were special tests each served a particular purpose, mainly to substantiate a certain argument or try to briefly investigate another. Reconstituted and remoulded samples were used in conventional as well as Hydraulic Stress-Path triaxial apparatus. Different rates of undrained shearing were used. 7 tests were run and are tabulated in table (3-10).
- 11) Tests of group "C" were to study the compressibility of the soil and the effect of the size of loading increments loading on the compressibility. The tests were run using oedometers, conventional and Hydraulic Stress-Path triaxial apparatuses. Anisotropic as well as isotropic cycles of loading were performed on reconstituted and remoulded samples. 23 tests were done and are tabulated in table (3-11).
- 12) The two tests of group "RS" were run in the ring shear apparatus under drained conditions on remoulded samples. The purpose was to study the residual strength and post-peak behaviour of the soil and also to investigate how the soil

memorised previous states of stress in terms of peak and residual strengths. (table 3-12).

Tests of one or more of the above groups were used to study each of the soil properties mentioned before. (see section 3-1). The testing programme started after Easter 1975 and finished at the end of February 1978.

TABLE 3-1

TESTS OF GROUP "G"

Group	Test	Soil	Type	Aprts.	Rate	TSP
G	T10	U	UU		Re	
	T12	U	UU		Re	
	T1	U	UU		Ra	
	T2	U	UU		Ra	
	T14	U	UU		Rb	
	T24	R	CU		Re	
	T31*	R	UU		Re & Ra	
	T27	R	CU		Re	
	T4	R	CU		Ra	
				HT		1
	T21	R	CU		Ra	
	T16 [#]	R	CU		Ra	
	T18 [#]	R	CU		Ra	
	T20	R	CU		Ra	
	T30	R	CU		Ra	
	T302	R	CU		Ra	
	T32	R	CU		Rb	
	T13	R	CU		Rb	
	T19 [#]	R	CU		Rb	
	T26	R	CU		Rc	

TABLE 3-2

TESTS OF GROUP "NS"

Group	Test	Soil	Type	Aprts.	Rate	TSP
NS	T17	U	UU		Ra	
	T23	R	CU	HT	Ra	1
	T25	R	CU		Rc	

* sample was 71.1 mm in diameter and 101.16 mm high, tested with lubricated ends.

[#] artificially Fissured.

TABLE 3-3

TESTS OF GROUP "WCR"

Group	Test	Soil	Type	Aprts.	Rate	TSP
WCR	Ra19	R	CU	CT	Ra	1
	Ra18		CU		Ra	
	Ra14		CU		Ra	
	Ra12		CU		Ra	
	Ra11		CU		Ra	
	RaUU19		UU		Ra	
	RaUU14		UU		Ra	
	RaUU13		UU		Ra	
	Rb19		CU		Rb	
	Rb15		CU		Rb	
	Rb16		CU		Rb	
	Rb14		CU		Rb	
	Rb12		CU		Rb	
	Rb11		CU		Rb	
	Rb 9		CU		Rb	
	Rc20		CU		Rc	
	Rc18		CU		Rc	
	Rc16		CU		Rc	
	Rc15		CU		Rc	
	Rc15 ₁		CU		Rc	
	Rc14		CU		Rc	
	Rc13		CU		Rc	
	Rc11		CU		Rc	
	RcUU19		UU		Rc	
	RcUU15		UU		Rc	
	RcUU15 ₁		UU		Rc	
	RcUU15 ₂		UU		Rc	
	RcUU12		UU		Rc	
	Rd21		CU		Rd	
	Rd19		CU		Rd	
	Rd15		CU		Rd	
	Rd16		CU		Rd	
	Rd14		CU		Rd	
	Rd12		CU		Rd	
	Rd 9		CU		Rd	

TABLE 3-4

TESTS OF GROUP "V"

Group	Test	Soil	Type	Aprts.	$\frac{\partial \sigma'_3}{\partial t}$ kPa/hr	TSP
V	V1U	U	D			2
	V1R	R	CD			2
	V1R'	R	CD			1
	V2U	U	D			2
	V2R	R	CD	HT ~ 1.1		2
	V3U	U	D			2
	V3R	R	CD			2
	V4U	U	D			2
	V4R	R	CD			2

TABLE 3-5

TESTS OF GROUP "IN"

Group	Test	Soil	Type	Aprts.	Rate	TSP
IN	INP20				Ra	
	INP40				Ra	
	INP60				Ra	
	INP80				Ra	
	INP160				Ra	
	INP200				Ra	
	INP300				Ra	
	INP630	S	CU	HT	Ra	1
	INP800				Ra	
	INP40				Rc	
	INP50				Rc	
	INP80				Rc	
	INP200				Rb	
	INP200				Rc	
	INP800				Rc	

TABLE 3-6

TESTS OF GROUP "10"

Group	Test	Soil	Type	Aprts.	Rate	TSP
IO	IO2P40	S	CU	HT	Ra	1
	IO4P40				Ra	
	IO10P40				Ra	
	IO20P40				Ra	
	IO7P50				Ra	
	IO2P80				Ra	
	IO4P80				Ra	
	IO10P80				Ra	
	IO2P200				Ra	
	IO4P200				Ra	
	IO2P40				Rc	
	IO4P40				Rc	
	IO2P50				Rc	
	IO4P50				Rc	
	IO7P50				Rc	
	IO2P200				Rc	
	IO4F200				Rc	

TABLE 3-7

TESTS OF GROUP "AN"

Group	Test	Soil	Type	Aprts.	Rate	TSP
AN	ANP40	S	CU	HT	Ra	1
	ANP80		CU			
	ANP160		CU			
	ANP320		CU			
	ANP40D		CD			
	ANP80D		CD			
	ANP160D		CD			
	ANP320D		CD			

TABLE 3-8

TESTS OF GROUP "AO"

Group	Test	Soil	Type	Aprts.	Rate	TSP
AO	AO2P40	S	CU	HT	Ra	1
	AO4P40		CU			
	AO10P40		CU			
	AO2P80		CU			
	AO7P80		CU			
	AO2P146		CU			
	AO2P40D		CD			
	AO4P40D		CD			
	AO10P40D		CD			

TABLE 3-9

TESTS OF GROUP "SMP"

Group	Test	Soil	Type	Aprts.	Rate	TSP
	SMP1					
	SMP2					
SMP	SMP3	S	CU	HT	Ra	1
	SMP4					

TABLE 3-10

TESTS OF GROUP "ST"

Group	Test	Soil	Type	Aprts.	Rate	TSP	Consolidation type
	ST1	S	CU	HT	Ra		I
	ST2	S	CU	HT	Ra		I
	ST3	S	CU	HT	Ra		A
	ST4	S	CU	HT	Rd		I
ST	ST5*	R	UU	CT	different variable rates	1	I
	ST6*	R	UU	CT			I
	ST7	S	CU	HT	Ra		I & A

* sample was 71.1 mm in diameter and 101.16mm high, tested with lubricated ends.

TABLE 3-11

TESTS OF GROUP "C"

Group	Test	Soil	Aprts.	Group	Test	Soil	Aprts.
C	OD1	R	OD	C	OD13	R	OD
	OD2	R			OD14	R	OD
	OD3	R			OD15	R	OD
	OD4	S			OD16	R	OD
	OD5	S			OD17	R	OD
	OD6	S			AR1	R	HT
	OD7	R			AR2	R	HT
	OD8	R			IS3X	S	CT
	OD9	R			IR1	R	CT
	OD10	R			IR2	R	CT
	OD11	R			IR3	R	CT
	OD12	R					

TABLE 3-12

TESTS OF GROUP "RS"

Group	Test	Soil	Aprts.	Rate(s)
RS	RS1	R	RS	0.0008
	RS2			0.00008 & 0.0008

CHAPTER 4

SAMPLING, APPARATUS AND TESTING TECHNIQUE

4.1 SAMPLING

The sampling operation consisted of obtaining blocks of "undisturbed" samples for laboratory testing. The place was the coastal cliffs near Happisburgh in Norfolk, England. (see Chapter 2). From the foot of the cliffs eight blocks from different locations were obtained, these locations were within a few yards of each other.

By hand tools, namely shovels and pick axes trenches of about 250mm width were dug around each block until it separated. Some blocks had faces which were exposed, the others were taken from less exposed places from inside the foot of the cliffs. The blocks were about 300 x 300 x 300mm.

The blocks were immediately waxed, wrapped with aluminium foil and placed in wooden boxes with the vacant spaces inside the boxes filled with sand. They were transferred to the Imperial College laboratory hours after sampling. After 48 hours the best four of the blocks were cut to smaller blocks, by band saw, to dimensions of 50 x 50 x 100mm., rewaxed, wrapped and stored in airtight tins in the laboratory. The laboratory had a controlled temperature of $20^{\circ} \pm 1^{\circ}\text{C}$ (except for a period of 15 days during which the system failed).

Samples of this soil dried up even during the fast process between cutting and storing and the discontinuities which were present made the trimming to the required test size a rather tedious job. It is difficult to get representative samples of such a low plasticity soil.

4.2 APPARATUS

4.2.1 Testing Apparatus

4.2.1.1 Conventional Triaxial Apparatus (CT) (strain-controlled)

Two different machines were used, one with a maximum capacity of 5 Tonnes and the speed of the pedestal was controlled by the manually chosen combination of gears. The other had a capacity of 10 Tonnes and was based on thyristor controlled motors which were programmed by two sets of digital range switches. These switches indicated a direct feed rate in mm/min. Both the above machines were supplied by Wykeham Farrance Engineering Ltd., England.

These two machines were used with 38mm and 100mm cells as described in Bishop and Henkel (1962), except that the deviator stresses were measured by internal load cells and the displacements were continuously monitored by displacement transducers (see item 3-1).

The 38mm cells, which were used in most tests, had flushing facilities and were used with porous stones resting on the pedestal above which circular discs of filter paper

were placed. The porous stones were 2.5mm thick and had a permeability of 1×10^{-3} cm/sec. The filter papers Whatman's No.54 were 0.0165 cm thick and had a permeability of 2×10^{-5} cm/sec. (Bishop and Henkel 1962).

The 100 mm cells were used to test three 71.1 mm diam. X 101.1 mm long samples with lubricated ends. The lubricated ends consisted of two rubber layers alternating with grease and placed on a mirror polished surface. At the centre of the bottom surface a guiding pin was there to help in centring the sample (see Sandroni 1977). A useful review on the effect of end-restraint on the shear stress distribution inside the sample was given by Maguire (1975).

The cell and back pressures were supplied throughout this research by the self-compensating mercury control system, described fully in Bishop and Henkel (1962).

4.2.1.2 Hydraulic Triaxial Apparatus (HT) (Stress-Path Apparatus)

1) Description: This triaxial apparatus was developed by Wesley (1975) and reported by Bishop and Wesley (1975). It was used for extension tests by Maguire (1975) and generally by Buri (1978) and Damamola (1978). The apparatus and advantages of details of its design were described adequately by the aforementioned authors. Only a brief description will therefore be given here.

This apparatus has been developed to enable

symmetric tests to be performed following any stress path in compression or extension (see Fig.4-1) which was difficult if not sometimes impossible to achieve using the conventional triaxial apparatus. Both axial and radial stresses should therefore be able to be varied simultaneously and independently. This was achieved by separating the cell pressure from the axial pressure by two sets of Bellofram rolling seals. A loading ram was forced vertically, between a lower pressure chamber and an upper cell, by hydraulic pressure in the lower chamber. This pressure was sealed within the lower chamber by the lower set of Bellofram seals. In the space between the chamber and the cell, the loading ram was housed in a Rotolin linear bearing. Similar arrangements have been employed in creep cells (Lovenbury 1969 and Davis 1975) and in a plane strain apparatus (Atkinson 1973). The pressure in the upper cell was retained inside it by the upper set of Bellofram seals.

The axial and radial stresses could therefore be changed to any combination by simply changing the pressures of the lower chamber and the upper cell.

2) Operation and Control

To operate the apparatus to follow a required stress path one had to adjust the rates of axial and radial stress changes as well as their direction. The one equation that governs these relationships was actually the equation of vertical equilibrium of the components (see Fig.4.2):

$$P \cdot \frac{a}{A} + \sigma_r(1 - \frac{a}{A}) = \sigma_a - \frac{W}{A} \quad (4-1)$$

where : P = pressure in lower chamber
 a = effective area of Bellofram seal (29.4 cm²)
 A = area of sample (11.4 cm²)
 σ_r = radial stress
 σ_a = axial stress
 W = total weight of moving parts. (~ 40 N)

writing equation (4.1) in terms of stress changes:-

$$\Delta\sigma_a = \Delta p \frac{a}{A} + \Delta\sigma_r(1 - \frac{a}{A})$$

$$\Delta\sigma_a = 2.58\Delta p - 1.58\Delta\sigma_r \quad (4-2)$$

To control these rates and directions, two control systems were used for raising and lowering the two lines of mercury pots connected to the lower chamber (Δp) and the upper cell ($\Delta\sigma_r$):-

- i) the first system was the one used and described by Wesley (1975). It is shown diagrammatically in Fig.(4-3a). Tables of average calibrations were given in tables (4-1) and (4-2).
- ii) the second system was similar in principle to the first but had different components Fig.(4-3b).

The speeds and directions of the two D.C. servo motors were independently controlled by a double channel D.C. Linear

Servo Amplifier System. The motors were further connected to gear boxes the output shafts of which were connected to the winches of the mercury pots. Similar control systems were enlarged upon by Buri (1978). The outputs of the motors were plotted against the control setting of the Linear Servo Amplifier in Fig. (4-4). If all items of the control system functioned satisfactorily, the resulting changes in pressures Δp and $\sigma \Delta_r$ were supposed to remain at constant rates during one single setting of the controls. However changes took place which were attributed to:

- i) irregular threading of wires over spindles which acted as winches for the mercury pots.
- ii) small leaks in the tubings.
- iii) blockages in the tubing due to dirt or the common algae which was offered a suitable environment especially during long term tests.
- iv) incorrect resilience in the springs of the mercury pots to minimise their effects, periodic checks were made.

The causes of the above were periodically checked to rid the tests from their effects. The test results (Chapter 5) showed at least partial success in this direction.

3) The Frictional Resistance

When the pieces of apparatus were taken over by the author, estimates of the frictional resistance of the Rotolin linear bearing were made using both methods described

by Wesley (1975). The results agreed with Wesley's. Rapid assessments of frictional resistance were made periodically using Wesley's first method. In two incidents during the testing period the author had to dismantle the apparatus, clean, grease it and put it back to overcome unacceptably high values of frictional resistance.

4.2.1.3 The Oedometer

Ordinary Bishop-type rear loading oedometers were used. These were described in many previous texts (e.g. Akroyd 1964 and Lambe 1951).

4.2.1.4 The Ring Shear

The apparatus used was the one reported by Garga (1970) and Bishop et.al. (1972). Different descriptions and results from it were obtained by Wesley (1975), Maguire (1975), Walbancke (1976) and Lupini (1978). Proving rings were used to measure torque and dial gauges to measure gap and displacements. No further description will be given here.

4.2.1.5 Classification and Specific Gravity.

These were performed using ordinary sieves, hydrometers (with hexametaphosphate as a dispersing agent) and specific gravity bottles. All tests were carried out according to BS 1377: 1975.

4.2.2 Measurement Instruments

4.2.2.1 Load Cell

i) Introduction: The axial loads were measured by Imperial College load cells designed by Dr. A. Skinner and manufactured by the technical team of the Soil Mechanics laboratory of Imperial College under Mr. D. Evans.

These load cells have been used by previous researchers (e.g. El-Ruwayih (1975), Maguire (1975), Wesley (1975), Apted (1977), Sardroni (1977) and others).

ii) Structure and Principle of Operation: The load cell consisted of a cylindrical loading element and a load transducer . The load transducer consisted of three triangular cantilevers radiating from a common boss and bearing on the edge of a groove in the loading element.

Electrical resistive strain gauges were connected on each triangular cantilever and were wired to form a bridge circuit and to give complete insensitivity to eccentric loading or to horizontal components of inclined loads.

When an axial load was applied to the load cell, bending moments at each triangular cantilever of the load transducer were transformed to almost uniform surface strains which allowed the output of the bridge circuit to be calibrated directly against axial loads (regardless of the positions of the strain gauges). Calibration curves of the load cells used are shown in Figs. (4.5 & 4.6).

The load cell was filled with oil and sealed with a flexible rubber membrane, thus a change in the surrounding all round pressure (σ_r) would have no significant effect on zero readings or calibration.

The sensitivities of the load cells used are shown in table (4-3). An average area of $1.14 \times 10^{-3} \text{m}^2$ was used to calculate these sensitivities in terms of stresses to be comparable with those of the pressure transducers (in the case of the 8896 N load cell used in 100 mm cells for bigger samples an area of $3.97 \times 10^{-3} \text{m}^2$ was used). The arithmetic mean of the above sensitivities was $7.20 \mu\epsilon/\text{KPa}$. Wesley (1975) stated that use of maraging steel for the load transducer increased the sensitivities of the load cells.

iii) Deflection: The components of the load cell deflected when subjected to axial loads. These deflections had to be calculated and subtracted from the measured deformations in order to study the proper deformation characteristics of the tested soil. The method to calculate the load cell deflections was to measure outputs of a load cell when a metallic sample was tested. The deformations were only those of the load cell. Fig. (4-7) and (4.8) showed the deflection curves of the load cells used. These deflections were accounted for in every test.

Wesley (1975) stated that a small "play" between the load transducer and the cylindrical element brought about improved characteristics of load cells in terms of stability

of outputs and hysteresis but in the same time increased deflection. An individual compromise would thus have to be reached according to the nature of research. This gap (play) was measured in the load cells used in this research. The allowance ranged between 0.08: 0.15 mm.

4.2.2.2 Pressure Transducers

Pore pressures, cell pressures and lower chamber pressures in the hydraulic triaxial tests were measured by commercial Bell and Howell pressure transducers of a capacity of 1034.25 kPa in compression and about 103.42 kPa in tension (about 10% of compression value).

A pressure transducer of this type operated when resistive strain gauges which were wired as a bridge circuit registered the deformation of a diaphragm which formed part of the transducer pressure cavity. Calibration curves of transducers which were used are shown in Fig.(4-9) and (4-10).

The transducers were connected to special blocks and the systems were carefully deaired. The compliance time was 4 seconds on the average and the response time was about 40 seconds for the tested material. The sensitivities of the used transducers were shown in table (4-4) their arithmetic mean was $8.25\mu\epsilon/\text{kPa}$ which compared well with the average from the load cells ($7.20\mu\epsilon/\text{kPa}$)

4.2.2.3 Displacement Transducers

In all triaxial tests linear displacements were

measured by both mechanical dial gauges and resistive type displacement transducers. A transducer consisted of a tapered wedge travelling between and deflecting metal strip spring on which strain gauges were mounted. The output of the bridge-wired strain gauges was calibrated against displacement. Quite a linear relationship existed between displacements and output Fig. (4-11) and the sensitivities of the transducers ranged between 500: 800 $\mu\epsilon/\text{mm}$.

4.2.2.4 Pore Pressure Probe

This method of measuring pore pressures was used three times during testing large samples (71.1mm x 101.1mm). The system and technique of this measurement method was developed at Imperial College and described abundantly by various researchers (e.g. Sodha 1974, Maguire 1975 and Sandroni 1977).

The probe used was not different from the one used by Sandroni (1977). Its porous element was Aerox "Celloton" Grade V1 ceramic and had a coefficient of permeability of $1.4 \times 10^{-6} \text{cm/sec}$. The fluid pressure in the probe was registered by a pressure transducer. The system was carefully deaired and the compliance time was 4 seconds. It was installed by the method described by Sandroni (1977).

4.2.2.5 Activation and Data Print-out of Transducers.

This was carried out by the Peekel (Holland) unit which consisted of an AC/DC power supply and a signal timing

and print-out unit which allowed each transducer to be read manually or automatically. The automatic recordings were possible every 5 minutes for each transducer which easily allowed for unmanned overnight recordings.

The Peekel unit had a capacity of 50 channels and both it and the transducers were insignificantly affected by changes in temperature even during the time when the airconditioning system broke down (see Sandroni 1977).

Differences were noticed between readings of a particular transducer when monitored manually and when remotely read. The causes of these differences (about $8\mu\epsilon$) were incomprehensible to the author. Daramola (1978) suggested that the manual readings expressed inevitable natural scatter of any instrument where as a single remote reading might be the equivalent of a probable mean to this scatter.

4.2.2.6 Conventional Instruments

- 1) Dial Gauges: used to monitor displacements in oedometers and triaxial apparatuses and to monitor both displacements and gap opening in the ring shear apparatus. They were all commercial gauges with sensitivities ranging between 0.002: 0.025 mm/division. Regular checks before each test that the internal springs functioned were regularly made since jamming of the plungers of the dial gauges had to be avoided.

- 2) Pressure Gauges: used to monitor the pressures whenever there was a shortage in pressure transducer. The pressure gauges were commercial Budenberg gauges. (see Bishop and Henkel 1962). They were adjusted and calibrated before the program started and checked about every six months.
- 3) Proving Rings: Two high quality proving rings were used to measure torque in the ring shear apparatus. See Bishop et.al. (1972) and Walbaneke (1976).
- 4) Volume Gauges: They were the only measure of volume change in the triaxial tests, and were as described in Bishop and Henkel (1962). Their sensitivity was 0.01 cm^3 . They were cleaned periodically to prevent dirt causing unreliable results.

4.2.3 Calibration

1) Calibration of Load Cells, Pressure Transducers and Pressure Gauges: A Modified Budenberg calibrator was used to induce precise pressures to calibrate against. This was done following an established procedure (see Harris 1966, Green 1970 and Reades 1972). The accuracy of the system was not measured by the author but Maguire (1975) reported an accuracy of 0.03%.

2) Calibration of Displacement Transducers and

Dial Gauges: Dial gauges were calibrated by a vernier caliper of an accuracy of 0.01mm/division and the displacement transducers were calibrated against the dial gauges.

3) Proving Rings: Calibration curves were given to the author by Dr. Walbancke. They were further checked against static dead loads by Lupini (1978) and found that the calibration remained fairly constant.

4.3 TESTING TECHNIQUE

4.3.1 Triaxial Apparatus

4.3.1.1 Sample Preparation

1) Undisturbed Samples: The samples were trimmed from the small blocks (50mm x 50mm x 100mm) by a sharp knife to 38mm diameter x 76mm long samples. They were put in a special guide frame during trimming in order to adjust diameter and length. The samples dried up quickly and the operation was long and hard and the percentage of bad samples was high.

Eight samples from Happisburgh and one from the North Sea bed were tested successfully. However the inevitable drying up during the long trimming time as well as the presence of shell fragments and pebbles inside most of the samples cast some doubt on the reliability of these tests.

2) Remoulded Samples: The soil was dried, ground and sieved through a 2.00mm sieve. The finer than 2.0mm material was then brought to the required water content by two methods: i) by adding a measured amount of water to the required water content (as in V and WCR groups) or ii) by adding large amounts of water to bring the mix to near its liquid limit then partial drying was allowed. (as in most of G group).

The difference between these two methods might explain some of the differences in behaviour between seemingly identical samples.

A check was made during testing group WCR to make sure that grinding did not affect the grain size distribution of the soil due to crushing of the particles. The distribution curves before and after grinding were identical.

Some undisturbed samples were remoulded after adding water and consolidated then retested. (all remoulded V samples, three in group G and two in NS group).

3) Reconstituted Samples: The soil was dried, ground and sieved through the 2.00mm sieve. The material passing this sieve was kept in layers of polythene until two days before testing. The following procedure and system were developed and improved by the author to produce the best possible results:-

- i) 200 gramme of soil was weighed and put in a dish in an oven at 105° for 24 hours to ensure complete hydration.
- ii) An amount of distilled water of 60 gramme was added and the dish was immediately sealed in two layers of polythene bags for another 24 hours. During this time the water would mix evenly with the soil and the mix was fairly uniform. (this water content of 30% was later reduced to 28% to produce better samples. The W/C-P' relationship was slightly altered by this change, see Chapter 5).
- iii) The two drainage leads in the Hydraulic triaxial apparatus were carefully deaired and the pedestal was covered by a coarse porous stone ($K=1 \times 10^{-3}$ cm/sec) above which a filter paper ($K= 2 \times 10^{-5}$ cm/sec) was placed. The porous stone was deaired before placing and the filter paper was saturated. A rubber membrane of an average thickness of 0.254mm was thoroughly checked for punctures and uniformity and placed around the pedestal of the apparatus and kept in place by a pair of rubber rings. The length of the membrane was adjusted before placing so as to ensure proper stretch and avoid as much disturbance to the slurry as possible.

- iv) A split former was then fixed around the pedestal embracing the rubber membrane from the outside. The split former (3 x 120° segments) was as the one described in Bishop and Henkel (1962) for the 38mm diameter saturated cohesionless samples (p 90) except that this one had a mid-height connection to a vacuum source. The former was lightly greased from inside and outside and was kept tightly in place by a circlip. The rubber membrane was rolled over the former from the top after making sure that it was properly stretched. A vacuum was then applied to the mould by a vacuum pump of a capacity of 82.74kPa. Inside the former the hole to the vacuum connection was loosely covered by a piece of tissue paper to prevent the membrane from being sucked in and to permit the vacuum to penetrate around it to keep the rubber membrane in place.
- v) The prepared mix of soil was then poured into the mould by a small tea spoon. The proper consistency of the slurry was of utmost importance to the success of this type of test. Compaction was carried out using a small rod of a diameter of 3mm. It was pushed gently in and out of the mix to help get rid of air bubbles and to distribute the soil evenly inside the mould. This compaction took place after every two or three spoonfuls of soil were poured.

- vi) After filling the mould with slurry it was trimmed and made flush with its upper edge and a perspex top cap was placed over. The placing of the top cap was done slowly so as not to trap air between it and the soil and to keep the upper surface of the soil as horizontal as possible. A long top cap of about 40mm was preferred to avoid severe consequences of tilting and to give a bit of extra vertical weight needed to stabilize the soil in this stage. External weights were most of the time placed above the top cap to help the soil settle down during the first part of the test. A weight of 0.44N was found suitable.
- vii) The sample was afterwards exposed to a suction through one of the drainage leads of the pedestal, the other lead was closed. The suction line included an air-trap close to the apparatus to trap the air sucked away from the sample before it escapes to the rest of the system. The suction line then passed through a volume gauge which was connected from the other end to the suction system which was either simply a burette at a low level or a self compensating mercury system reversed so as to produce suction instead of pressure. The system is schematically shown in Fig.(4-12).

- viii) The shear stresses applied during this initial consolidation were determined by the magnitude of suction pressure and top weights. The sample was usually subjected to a net negative pressure of about 8 kPa. see Fig.(4-13). After consolidation was complete, the sample was usually left for 24 hours, the mould was removed and the upper cell was placed and filled with water. The suction line was closed and the volume gauge connected to a back pressure line at 200kPa. The valve between the air trap and the sample was closed and the air trap bled to get rid of air bubbles inside it. A cell pressure of 210kPa was first applied then the line to the back pressure reopened. The sample was left at this effective isotropic pressure of 10kPa for 24 hours (point b in Fig.4-13). In later tests an anisotropic pressure equal the one at point a was applied by combination of the cell pressure, lower chamber pressure and the back pressure. It was also applied for 24 hours.
- ix) Due to pore pressure gradients across the samples, perfect cylindrical shape was at first difficult to achieve and some necking was noticeable at the mid-height of samples. This necking was, however, improved after placing the light weights over the top cap during the suction stage; thus, uniformity and effective pressure distribution along the sample axes were improved.

4.3.1.2 Testing Procedure

4.3.1.2.1 Consolidation Stage.

I Isotropic Consolidation (conventional and Hydraulic triaxial apparatus).

1) Remoulded Samples:

The sample was trimmed to the required size as described before and the rubber membrane which was to be used was checked for leaks. In tests in which mid-height pore pressure probes were used the method for preparing the membrane (and installing the probe afterwards) was the one adopted by Sandroni (1977).

The base pore pressure measuring system (which was used in all tests but three) was flushed then a coarse porous stone and a filter paper (see item 4.2.1) were placed on the base pedestal; both were deaired and saturated. At this stage a quick check on zero readings of transducers and load cell was made. The sample was then placed on the pedestal and the membrane put around it by a membrane stretcher (see Bishop and Henkel 1962).

The cell pressure required for the first stage of consolidation was applied before the back pressure. The volume changes were sometimes minute because the effective pressure was chosen to be approximately equal to the pore water tension expected at the moulding water content.

Each consolidation stage lasted 24 hours and the effective consolidation pressures were doubled at each stage

In the case of unloading each swelling stage lasted 24 hours and and the effective consolidation pressure was halved.

2) Resedimented Samples:

After preparing the sample as described in item 4.3.1.1 the effective consolidation pressure was increased by increasing σ_r . The back pressure was kept constant throughout. The consolidation and swelling durations and increments were as in remoulded samples.

II Anisotropic Consolidation (Hydraulic Triaxial Apparatus).

1) Remoulded Samples:

There were only two tests of this type in which applying the cell pressure and measuring the suction, the consolidation process was carried out along a constant effective stress ratio line (K - line). See tests ARI and ARII. The process of consolidation was continuous and the choice of rate will be discussed below.

2) Resedimented Samples:

The samples were consolidated from either point a or b Fig. (4-13) along a K - line where the ratios of $\Delta\sigma_r/\Delta P$ were calculated from equation (4-2) in which $\Delta\sigma_r = K\Delta\sigma_a$.

In both anisotropic loading and unloading conditions, K was chosen to achieve approximately K_0 value.

K was chosen for anisotropic loading from reviewing the failure envelope of the results of the remoulded soils of groups G and WCR. An angle $\psi = \tan^{-1} 0.5$ was measured for the inclination of that envelope. To follow a " K_0 " line, a K value of 0.5 should have been used ($K_0 = 1 - \sin \phi' = 1 - \tan 4^\circ$, Jaky 1948 & Bishop 1958). For practical reasons; however, a value of $K = 0.55$ was used instead.

As for anisotropic unloading K was calculated using the method suggested by Wroth (1975) for lightly over-consolidated soils. The method involved the assumption that anisotropic unloading was linear up to $K = 1$ ($OCR = 4: 5$) see Ladd (1965). To calculate K during unloading (K_{oc}) the following parameters had to be used:-

- i - ϕ' obtained as explained before.
- ii - K_{nc} ($= 1 - \sin \phi'$).
- iii - P.I. (plasticity index) measured about 12%.
- iv - v' (Poisson's ratio) was obtained from the value of P.I. as shown in Wroth's Fig. (2) about 0.275.
- v - an estimated OCR (taken = 2 as estimated from pilot tests).

K_0 for unloading was calculated using the following equation:

$$K_0 = OCR, K_{nc} = \frac{v'}{1-v'} (OCR-1) \quad (4-3)$$

Bishop (1978) pointed out that the above method relies on the assumption of a constant v' in anisotropic loading and unloading whereas this might not be the case. (See Bishop and Hight 1977).

In order to determine the rate of stress change to ensure that the applied stresses are effective stresses, a useful review on consolidation at constant rate of strain was put forward by Egeli (1978). (e.g. Lowe et.al. 1969, Smiths and Wahls 1969 and Wissa 1971). Wissa's approach was the most appropriate for this research. He derived relationships for consolidation behaviour when:-

1) the transient state of pore pressure generation has finished and the steady state has been achieved;

2) the compression characteristics are linear, i.e.

$$m_v = \frac{\Delta \epsilon}{\Delta \sigma_v} = \text{constant.}$$

3) the rate of change of axial strain $\frac{\delta \epsilon_1}{\delta t}$ at any point is constant, consequently the rate of change of effective vertical stress $\frac{\delta \sigma'_v}{\delta t}$ and the pore pressure distribution are constant.

After rearranging, the relationship comes out as follows

$$\frac{\partial P'}{\partial t} = \frac{2 P' C_v (1-U\%) }{H^2} \quad (4-4)$$

where P' = effective consolidation pressure
 C_v = average coefficient of consolidation
 $U\%$ = required degree of consolidation
 H = drainage length inside the sample.

As for the first assumption it was recommended that Wissa's approach be best applied if the excess pore pressure did not exceed 10% of the effective vertical pressure. This check was not practicable since both pore pressure measurement and back pressure lines were one in this particular set up.

The assumption of a linear stress-strain consolidation relationship was reasonable to be applied to this case (see Figs. 5.8 & 5.15).

The third assumption that $\frac{\partial \epsilon_1}{\partial t}$ at any point is constant is applied to strain controlled oedometers (Som 1968, Skinner 1973 and Egeli 1978), whereas in the hydraulic triaxial apparatus it is rather the stresses which are controlled. Several trials with pilot tests had to be made to achieve a stress control which gave fairly constant rate of strain.

The C_v for this soil as measured from oedometer tests is shown in Fig. (2-9). Wissa (1971) showed C_v - P' relationship Fig.(4-14) typical of many in the literature. The C_v decreases sharply until a certain pressure is reached then remains fairly constant with a slight tendency towards increasing with increasing pressure which coincides with the left hand part of Fig.(2-9) for this soil. For the pressure range used in this research C_v was assumed to be $2m^2/\text{year}$ ($5.48 \times 10^{-3} m^2/\text{day}$).

The degree of consolidation was required to be 95% and since the drainage was only from the base, full height of the sample was taken as H (usually 0.076m). An average effective consolidation pressure of 100kPa was substituted in equation 4; this pressure was thought to be a reasonable average. This gave $\frac{\partial P'}{\partial t} \sim 17.00 \text{ kPa/day}$

For the K - line chosen, ($K' = 0.55$ and $\alpha = 16.17^\circ$ Fig. 4-13) the rates of changing the axial and radial stresses were:

$$\frac{\partial \sigma_1}{\partial t} = 21.93 \text{ kPa/day}$$

and

$$\frac{\partial \sigma_3}{\partial t} = 12.07 \text{ kPa/day}$$

From equation (4-2) the rate of change of the lower chamber pressure could be readily calculated:

$$\frac{\partial P}{\partial t} = 1.317 \quad \frac{\partial \sigma_3}{\partial t} = 15.90 \text{ kPa/day}$$

The spindles of the ratchet winches of the mercury pots were calibrated and the average value of the pressure they exert per turn of the spindle was 17.31 kPa. From the calibration of the control systems (see item 4.2.1) reasonably good control was obtained. The average value of $\frac{\partial \epsilon_1}{\partial t}$ in these tests was 1.53% /day.

Before anisotropic consolidation starts the chamber pressure should be raised to overcome the weight of the components as well as the frictional resistance of the apparatus. Otherwise calculated controls will not give the required path.

4.3.2.1.2 Shearing Stage

I) Strain Controlled Shearing: In the conventional triaxial apparatus the sample was made to touch the load cell after the consolidation stage then the pedestal was forced upwards by the motor. The speed of the motors was controlled in two different ways (see item 4.2.1).

In the hydraulic triaxial apparatus the lower chamber pressure was opened to the water pump system. (see Fig. 4-12), after consolidation and before the actual shearing started. This was done to ensure that both systems were subjected to the same pressure to prevent any sudden movement of the loading ram if this difference existed. In the case of isotropic consolidation the pedestal was raised to bring the sample in contact with the load cell. The shearing was done by turning the water pump on (clockwise in case of compression) which in turn caused the loading ram to move upwards and the sample was thus sheared against the fixed load cell. (the reverse should take place in the case of extension tests).

The water pump was turned by the first control system (see item 4.2.1) and the outputs agreed well with the

calculated rates. One turn of the ram to the water pump caused an average of 0.334 mm displacement of the loading ram if the system was properly deaired.

The rates of strain in drained and undrained shearing were conventionally calculated from Bishop and Henkel (1962 2nd edition). The rate most commonly used in undrained tests was 0.005 mm/min. Rates of an order of magnitude faster (0.05mm/min) were also used and gave the same pore pressure response. In the drained shearing, rates ranging from 0.0001: 0.005mm/min were used. The slower rates were used at first as a trial and when the failure strains became more predictable the faster strains were used.

Figs.(4.15) and (4.16) show contours of water contents in the vertical and horizontal directions after shearing in two tests chosen at random. The distribution of water contents indicated that the rates of testing were proper and that even distribution of water contents and consequently pore pressures prevailed inside the samples. This method for confirming correct rates of shearing is, of course, not suitable if the soil has a tendency to fail along shear zones.

II) Stress Controlled Shearing: Choosing unconventional stress paths for shearing was done in 8 drained tests in group V. In these tests, σ_1 was kept constant and σ_3 was reduced until failure occurred (see group V, Appendix 3). The rates of stress change in those cases were so chosen as

to give strain rates between 0.0001mm/min and 0.0005mm/min.

The weight of the moving parts and sample and the frictional force of the Rotolin linear bearing had to be calculated for and overcome in order to be able to control the stresses as required. This was achieved by imposing a difference between the cell pressure and the lower chamber pressure. This process was also done, but in reverse, in the case of starting a process of anisotropic consolidation (see item 4.3.1).

Stress-controlled shearing paths followed the designed stress paths until very near failure, then the stresses were difficult to control and in the case of the group V tests, keeping a constant σ_1 was not possible since failure was brittle and relatively large strains took place in a short time. The effect of such a situation on the apparent effective stress parameters will be discussed later.

4.3.1.3 Notes on Calculations

1) Lengths of resedimented samples were initially measured from the dimensions of the mould, porous stone and top cap and changes of length of all samples were measured by the difference in in displacement measurements when the load cell touched the sample before and after any stage. Initial water contents were measured from trimmings and from slurry and care was taken not to affect the readings of final water contents by sucking water from the stone or pore measure measuring system.

The final water contents were measured by slicing the sample into three horizontal layers and again slicing each horizontal layer into three equal segments. The water content of each segment was calculated and their average compared with the water content obtained from the total weight of the sample before and after testing. In most tests of group G and WCR water content was calculated as described, but by slicing the tested samples into 3 or 5 horizontal layers only.

2) Deformation characteristics were corrected for load cell deflection and gap. See item 4.2.2.

3) Area of sample corrections were carried out by an averaging method described by Bishop and Henkel (1962):

at a point x where $\epsilon_1 = \epsilon_{1x} = \frac{L_0 - L_x}{L_0} = \frac{\Delta L}{L_0}$, the sample area was A_x .

$$A_x = A_0 \frac{1 + \frac{\Delta v}{v_0}}{1 + \frac{\Delta L}{L_0}} \quad (4-5)$$

where

ϵ_1 = axial strain

L_0 = initial length of sample

A_0 = initial area of sample

Δv = change of volume at point x.

This correction was made at each step of the calculations in different stages of the tests.

4) The correction to the measured deviatoric stress for the effect of the strength of the rubber membrane was studied on the basis of the findings of Henkel and Gilbert (1952). The correction to be subtracted from the measured deviatoric stress was:

$$\frac{\pi \cdot D_o \cdot M \cdot \epsilon_1}{A} \quad (4.6)$$

where D_o denotes the initial diameter of the sample.

M denotes the compression modulus of the membrane per unit width. It was measured for membranes used in this research to be 0.25N/m.

ϵ_1 denotes the axial strain at which the correction is made.

A denotes the corrected area of the sample at ϵ_1 .

Bishop and Henkel (1962) calculated this correction to be 4.137 kPa at $\epsilon_1 = 15\%$.

The membrane stiffness was small relative to the stiffness of the samples in most tests. The tests done on large samples had, for the same water content, similar shear strengths as in smaller ones which indicated the insignificance of the effect of the membrane stiffness on the measured shear strength.

In the case of measuring pore pressures in undrained tests the grain size of the soil was small enough not to be

affected by the penetration of the membrane inside the sample (Kiekbusch and Schuppener, 1977).

Furthermore when it was decided that the samples had failed (see item 6- 8) it was for almost all tests by bulging (4 tests in group V failed along a slip plane). Thus the complicated correction for membrane stiffness when failure took place along slip planes was not needed (Chandler 1966 and Sandroni 1977). It was decided that the effort involved in introducing the membrane correction was unnecessary and it was ignored in this research.

5) Calculations were carried out manually by following a flow chart designed to ease the process by inserting data inside the program in the proper sequence. The output included all the data required (see Chapter 5). This manual process was preferred because it enabled one to monitor each test point by point, in different domains of stresses and strains, as it runs and not waiting until it finishes to discover any mishappenings (some testes lasted up to three weeks).

However, groups G and NS were processed by computer after being manually calculated. This was done because a small deviation in calibration of load cells was discovered after the tests were run. It was an early chance to check on the manual flow chart which proved to be correct and was used throughout and the manual effort spent in calculations was in the author's opinion worthwhile.

4.3.2 The Oedometer

Sample Preparation: Remoulded samples were prepared as explained for triaxial tests (first method) and resedimented samples were prepared also as explained for triaxial tests (see item 4-3).

Testing Procedure: Steps to run this test are well explained in text books (Akroyd 1964 and Lambe 1951) and specifications and will not be repeated. However, a few points are worthy of mention:-

- 1) the porous stones used at top and bottom of the samples were boiled and deaired and kept permanently under water until testing.
- 2) a filter paper (Whatman's No.54, $K = 2 \times 10^{-5}$ cm/sec) was saturated and placed between the sample and the bottom porous stone.
- 3) although care was exercised to trim samples flush with the edges of the ring, some central doming was sometimes noticed.
- 4) the measured displacements represented an average displacement due to side friction in oedometers and although the resiliences of the dial gauge springs were regularly checked, displacements during unloading were difficult to detect.
- 5) exact pressures in kPa were brought about by use of lead shot to balance the lower weights to the required load.

- 6) the loads were normally doubled or halved in loading and unloading respectively.

4.3.3 The Ring Shear

Sample Preparation: Remoulded soil was prepared as for the triaxial apparatus (first method) and brought to a water content near to its plastic limit; it was then placed in the ring by hand and spatula and trimmed flush with the upper surface of the ring.

Testing technique: The technique for testing in the ring shear was described in detail (e.g. Garga 1970 and Bishop et.al. 1971) and will not be dealt with here.

Thin petrological sections were made at the shear plane to study the orientation of particles. The technique of preparing the thin section was described in detail in Tchalenko (1967) and the basic principles were adequately described in Barden and Sides (1970) and Apted (1977). A photograph of one of the thin sections is shown in Fig.(4-17).

TABLE 4-1

Calibration of
Control System no.1

KA Setting	A _{rpm}	KB Setting	B _{rpm}	$\frac{B}{A}$	$\frac{A}{B}$
min.6	275	min.2	80	0.29	3.44
		10	94	0.34	2.93
		20	124	0.45	2.22
		30	170	0.62	1.62
		40	230	0.84	1.20
		50	310	1.13	0.89
20	370	min.2	110	0.30	3.36
		10	124	0.34	2.98
		20	168	0.45	2.20
		30	230	0.62	1.61
		40	308	0.83	1.20
		50	405	1.09	0.91
30	500	min.2	150	0.30	3.33
		10	168	0.34	2.98
		20	228	0.46	2.19
		30	315	0.63	1.59
		40	422	0.84	1.18
		50	550	1.10	0.91
40	680	min.2	204	0.30	3.33
		10	230	0.34	2.96
		20	312	0.46	2.18
		30	426	0.63	1.60
		40	560	0.82	1.21
		50	720	1.06	0.94
50	880	min.2	270	0.31	3.26
		10	305	0.35	2.89
		20	415	0.47	2.12
		30	560	0.64	1.57
		40	740	0.84	1.19
		50	980	1.11	0.90
60	1080	min.2	348	0.32	3.16
		10	390	0.36	2.77
		20	520	0.48	2.08
		30	650	0.60	1.66
		40	880	0.81	1.23
		50	1150	1.06	0.94

* For KB Setting 60, B/A is 0.73, 0.74, 0.75, 0.76, 0.73 and 0.94 respectively.

TABLE 4-2

Calibration of
Control System no.2

KA Setting	A _{rpm}	KB Setting	B _{rpm}	$\frac{B}{A}$	$\frac{A}{B}$
min.5	240	min.7	60	0.28	3.62
		20	98	0.41	2.45
		30	128	0.53	1.88
		40	173	0.72	1.39
		50	210	0.88	1.14
		60	295	1.23	0.81
20	325	min.7	90	0.28	3.61
		20	138	0.42	2.36
		30	184	0.57	1.77
		40	247	0.76	1.32
		50	310	0.95	1.05
		60	413	1.27	0.79
30	450	min.7	130	0.28	3.61
		20	184	0.41	2.45
		30	245	0.54	1.84
		40	318	0.71	1.42
		50	390	0.87	1.15
		60	500	1.11	0.90
40	550	min.7	180	0.33	3.06
		20	266	0.48	2.07
		30	347	0.63	1.59
		40	465	0.85	1.18
		50	580	1.05	0.95
		60	750	1.36	0.73
50	800	min.7	235	0.29	3.40
		20	352	0.44	2.27
		30	438	0.55	1.83
		40	600	0.75	1.33
		50	840	1.05	0.95
		60	950	1.19	0.84
60	1050	min.7	310	0.30	3.39
		20	460	0.44	2.28
		30	590	0.56	1.78
		40	770	0.73	1.36
		50	950	0.90	1.11
		60	1220	1.16	0.86

Max. allowable force (N)	Sensitivity	
	$\mu\epsilon/N$	$\mu\epsilon/kPa^*$
889.6	1.0	1.10
4448.0	3.23 : 6.25	3.55 : 6.88
8896.0	4.35	17.27

TABLE 4-3 Sensitivities of Load Cells

Max. allowable Pressure kPa		Sensitivity $\mu\epsilon/kPa$
Compression	Extension	
1034.25	103.42	7.63
1034.25	103.42	8.33
1034.25	103.42	6.9
1034.25	103.42	11.76
1034.25	103.42	7.87
1034.25	103.42	8
1034.25	103.42	7.8
1034.25	103.42	7.69

TABLE 4-4 Sensitivities of Pressure Transducers

* An area of sample of $3.97 \times 10^{-3} M^2$ was assumed.

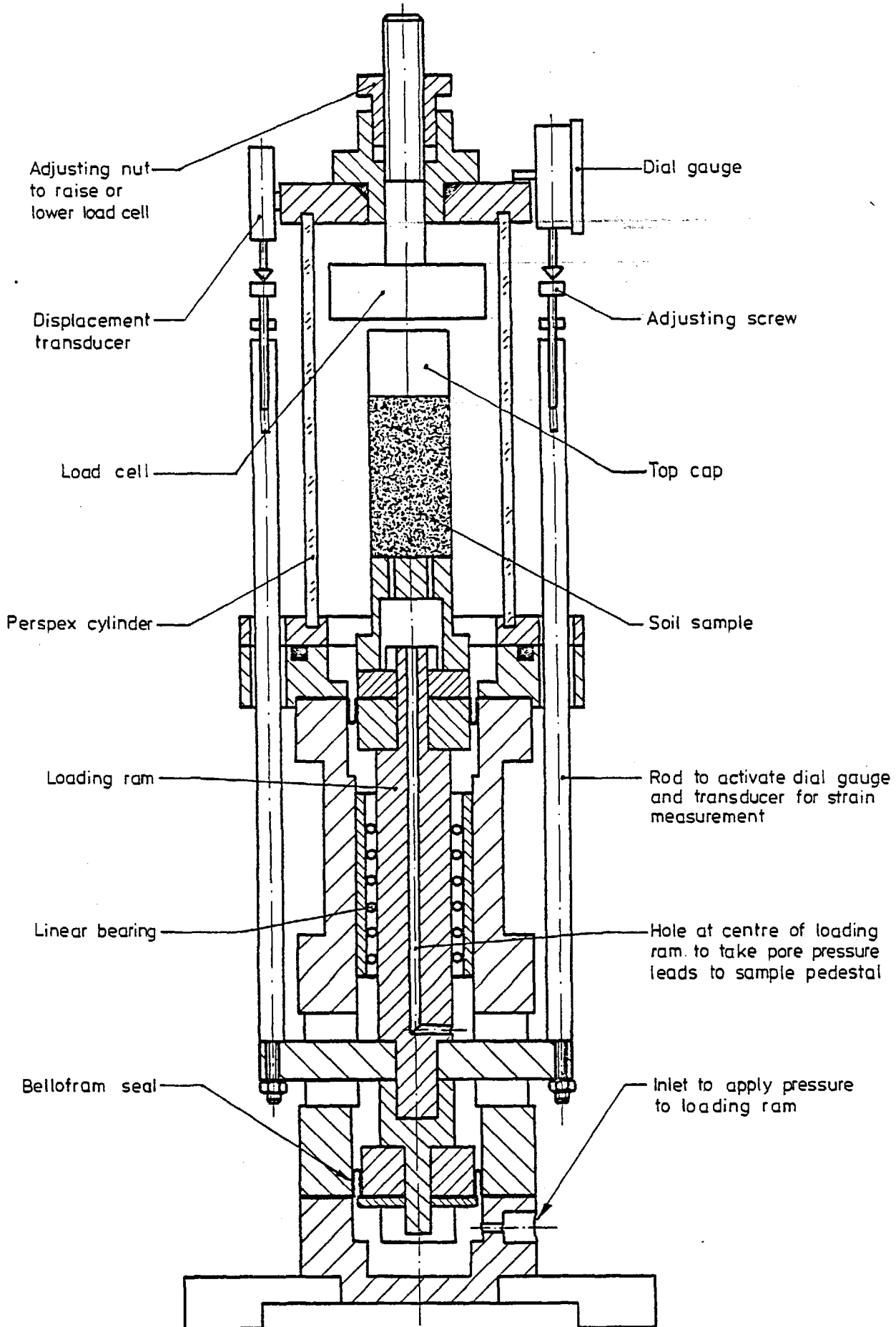
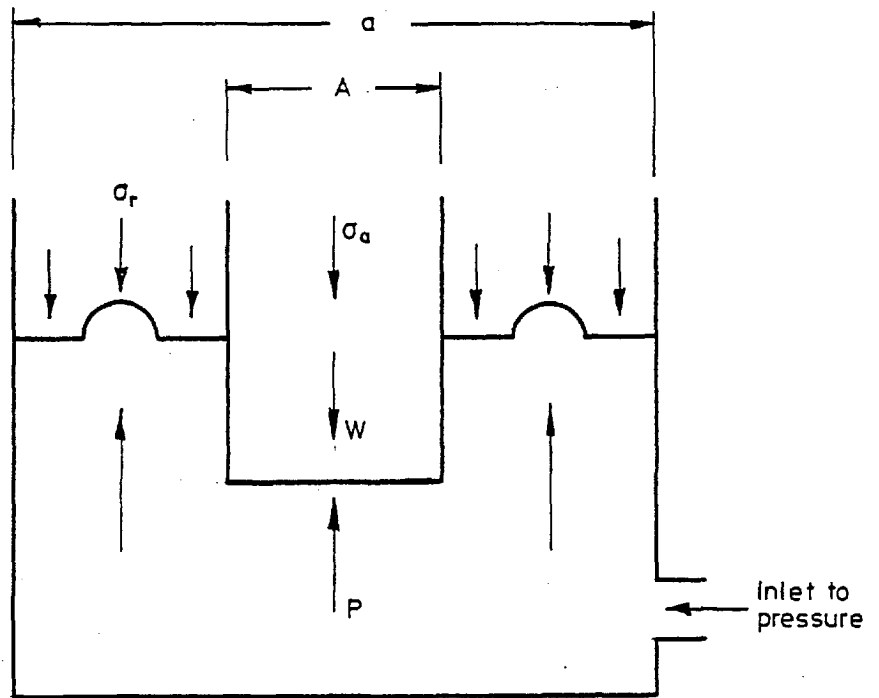


Fig. 4-1 CROSS SECTION OF HYDRAULIC STRESS
CONTROLLED TRIAXIAL APPARATUS
(after Wesley 1975)

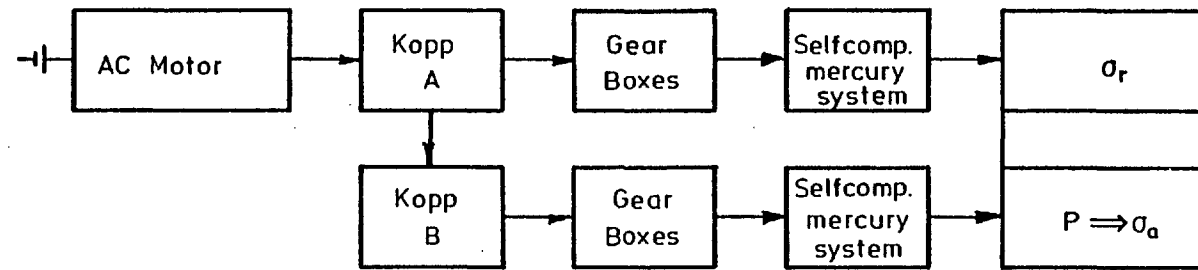


$$\sigma_a A + W + \sigma_r (a - A) = P a \quad \text{Vertical equilibrium}$$

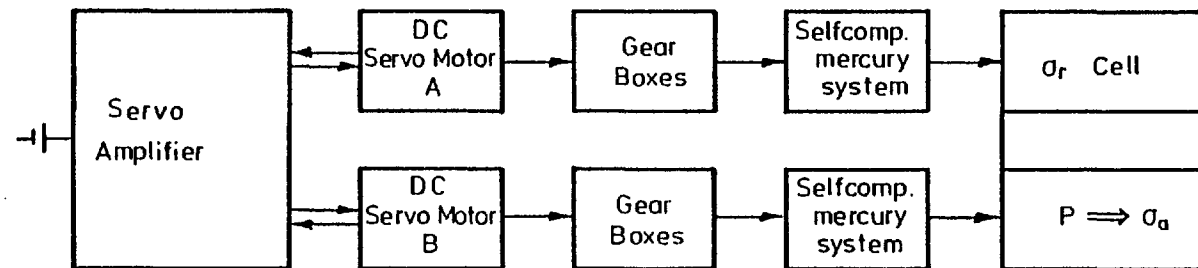
rearranging

$$P \frac{a}{A} + \sigma_r \left(1 - \frac{a}{A}\right) = \sigma_a + \frac{W}{a}$$

Fig. 4-2 Equation for operating the hydraulic triaxial apparatus.



a)



b)

Fig. 4-3 Control systems of hydraulic triaxial cylinder .

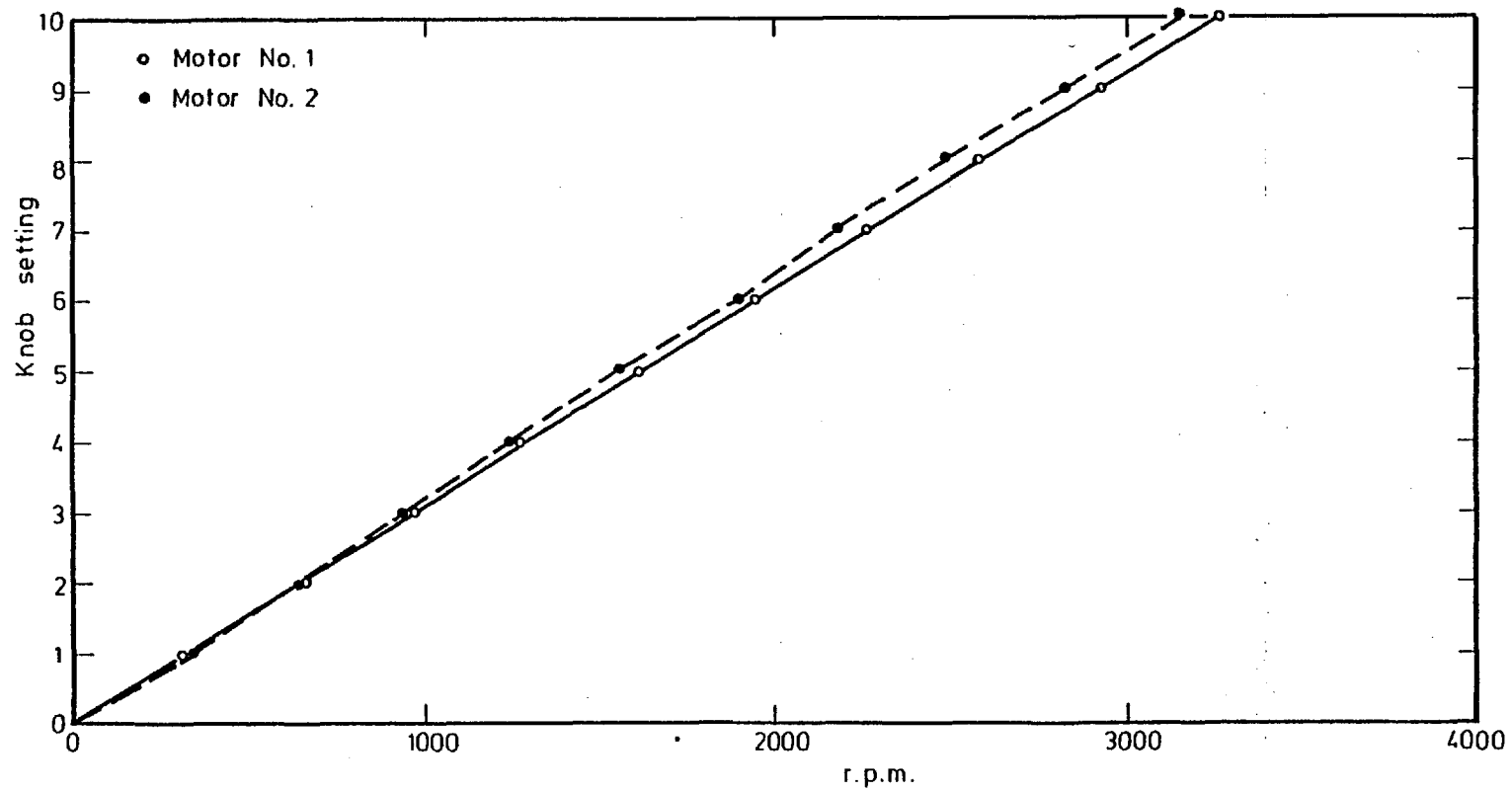


Fig. 4-4 Calibration Curves of D.C. Servo Motors.

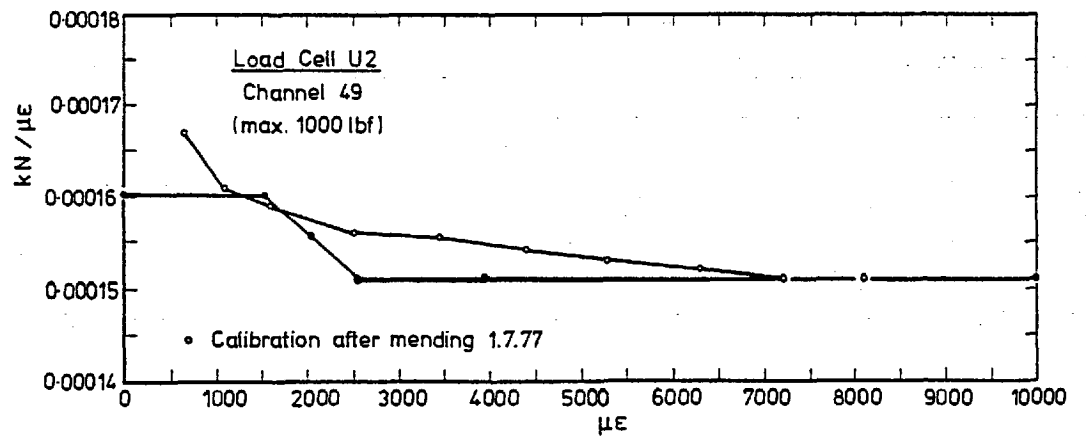
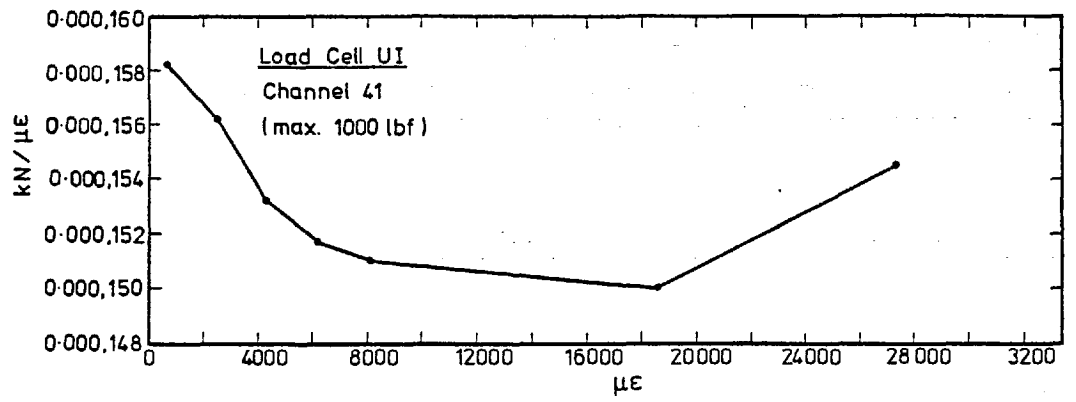


Fig.4-5 Calibration of load cells

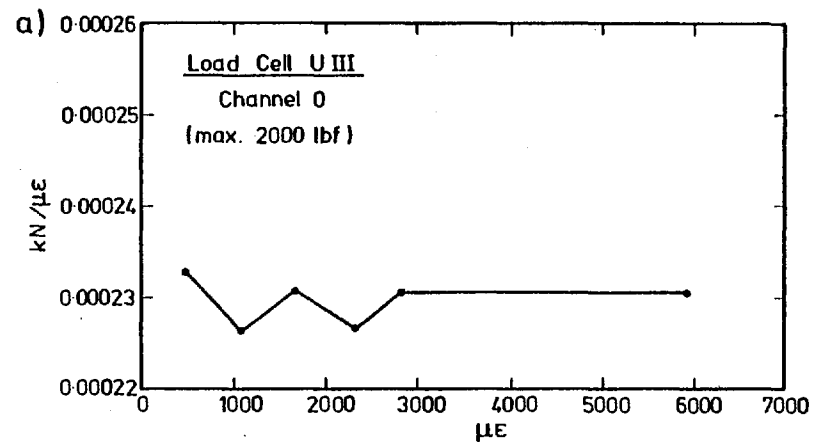
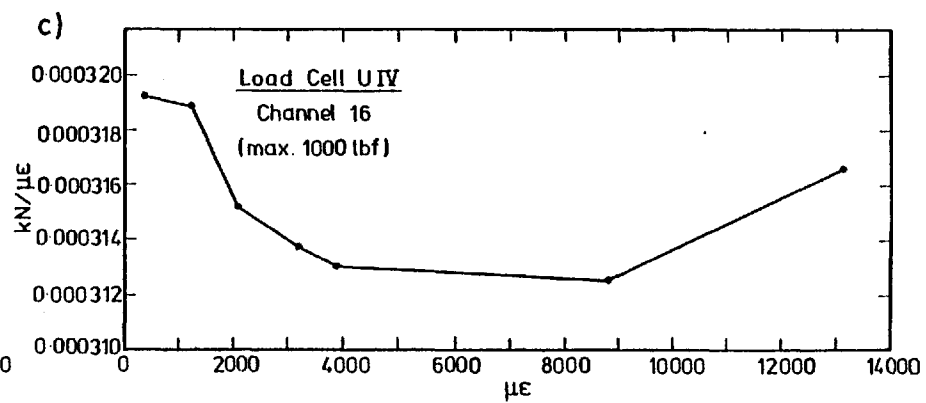
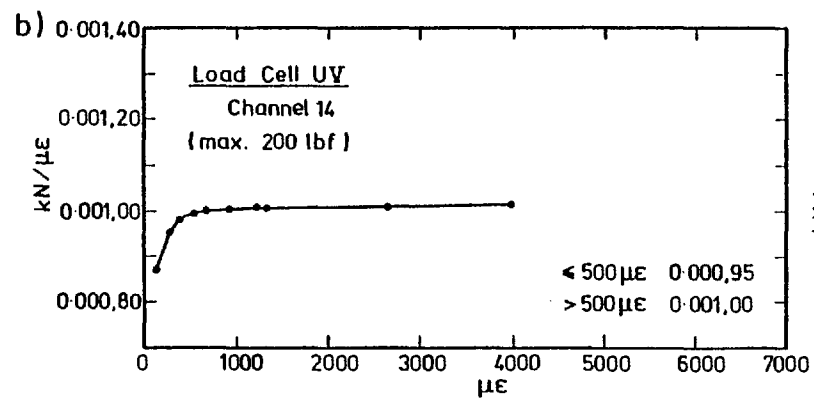


Fig.4-6 Calibration of load cells



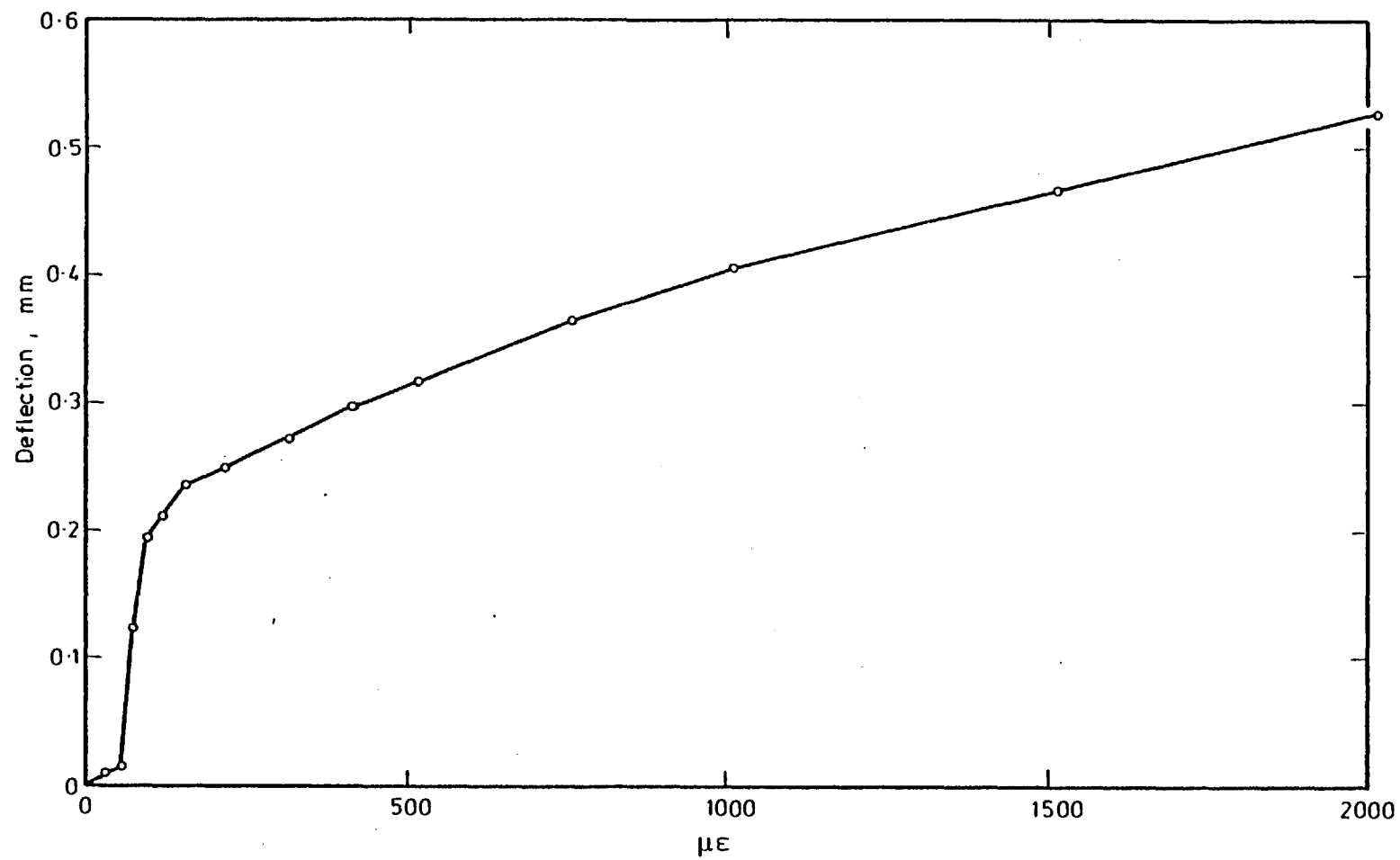


Fig. 4-7 Deflection of load cell UI

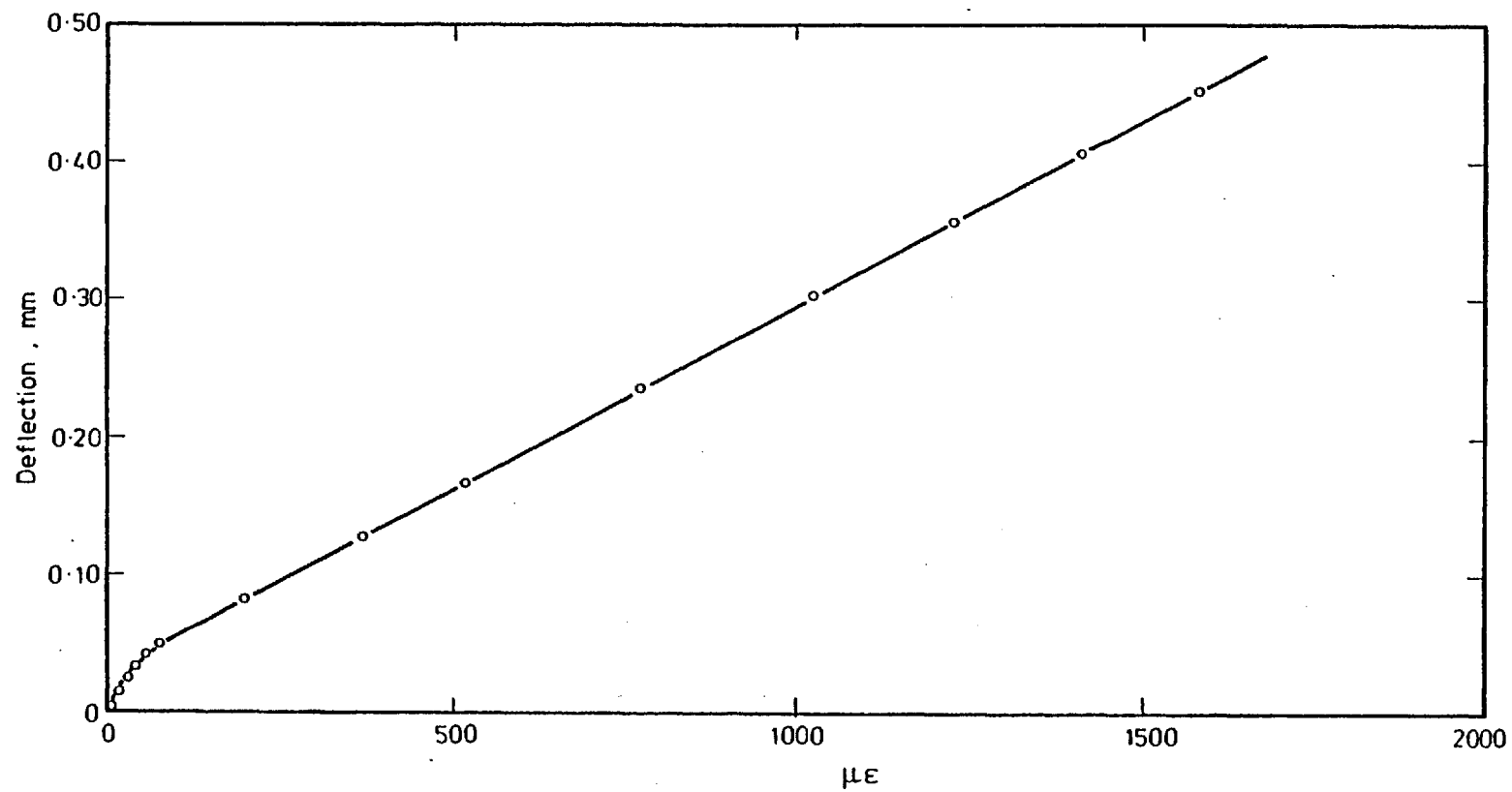


Fig. 4-8 Deflection of load cell UII

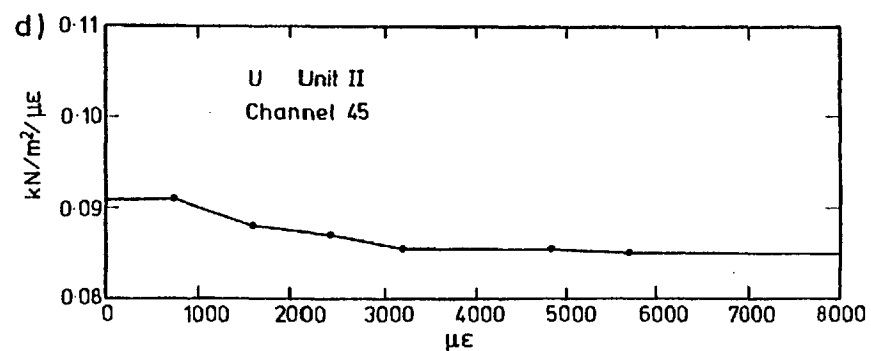
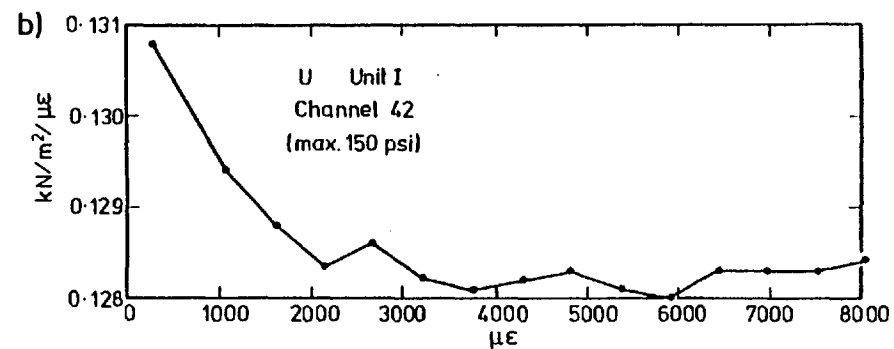
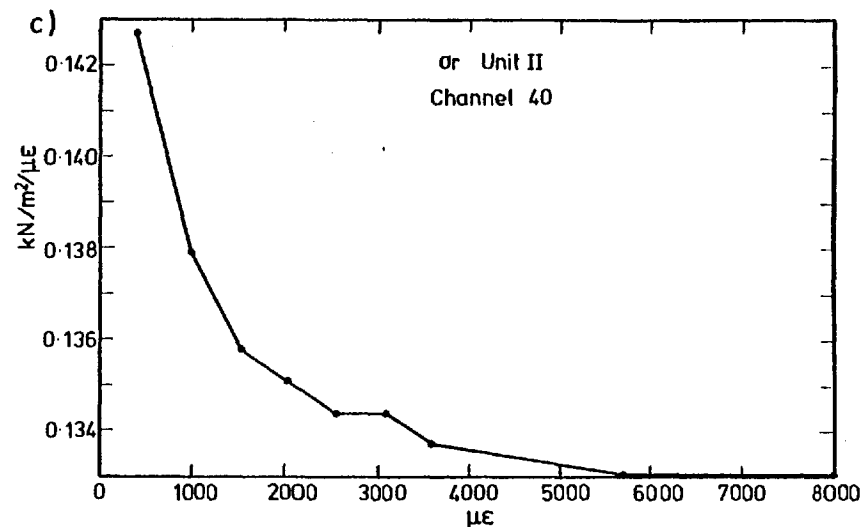
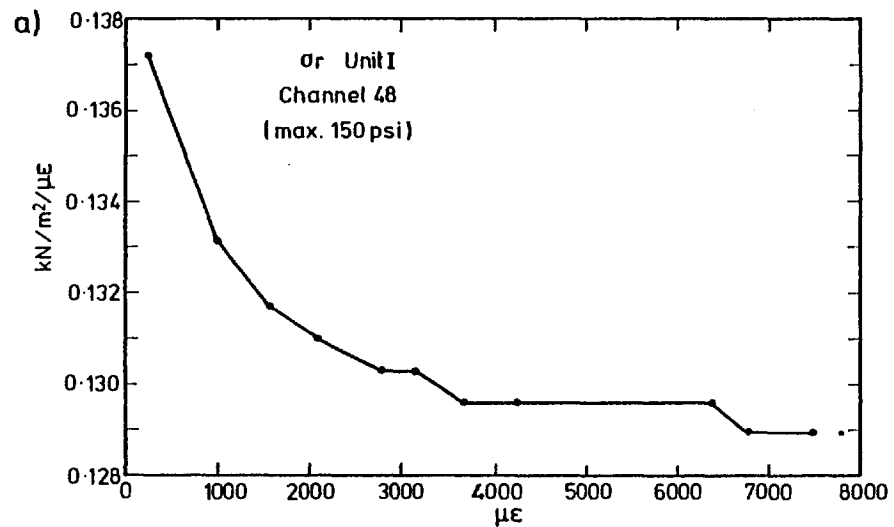
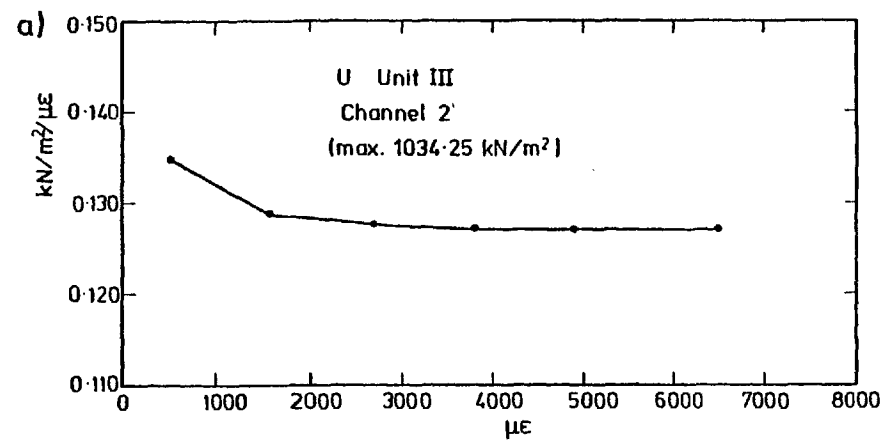
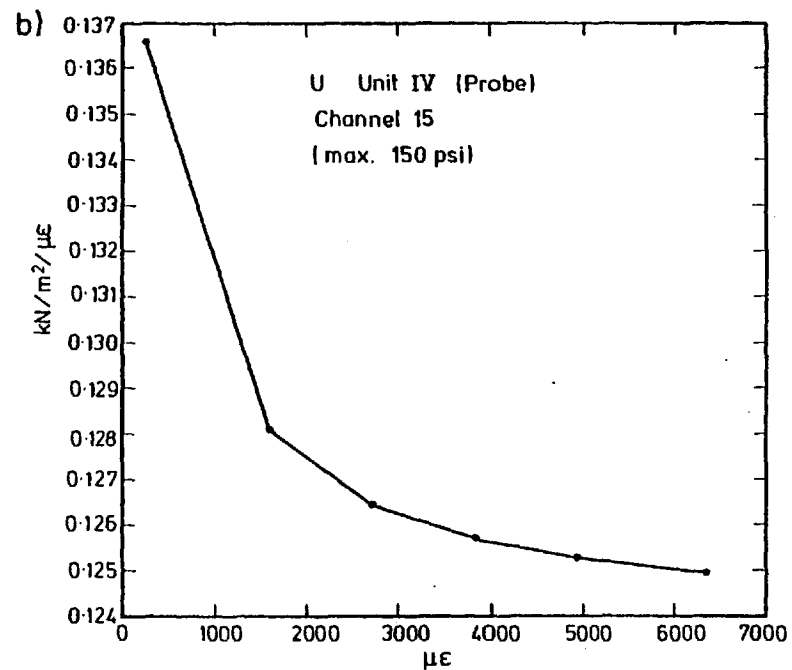


Fig. 4-9 Calibration of pressure transducers.



-16-

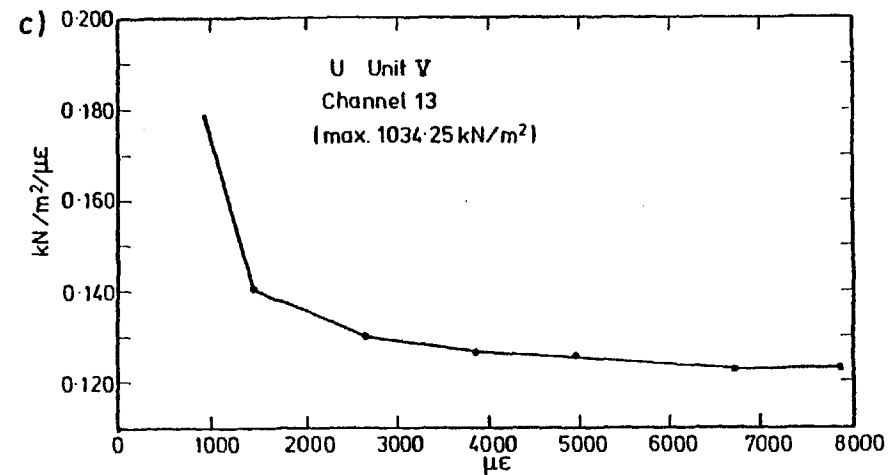
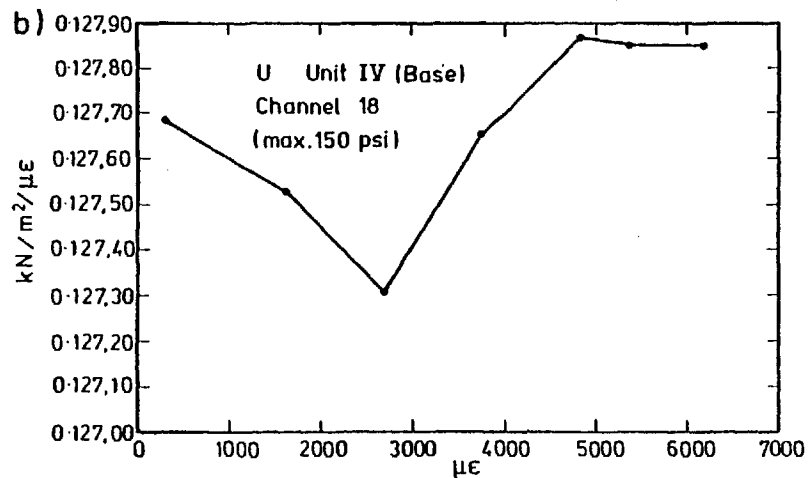


Fig. 4-10 Calibration curves of pore pressure transducers.

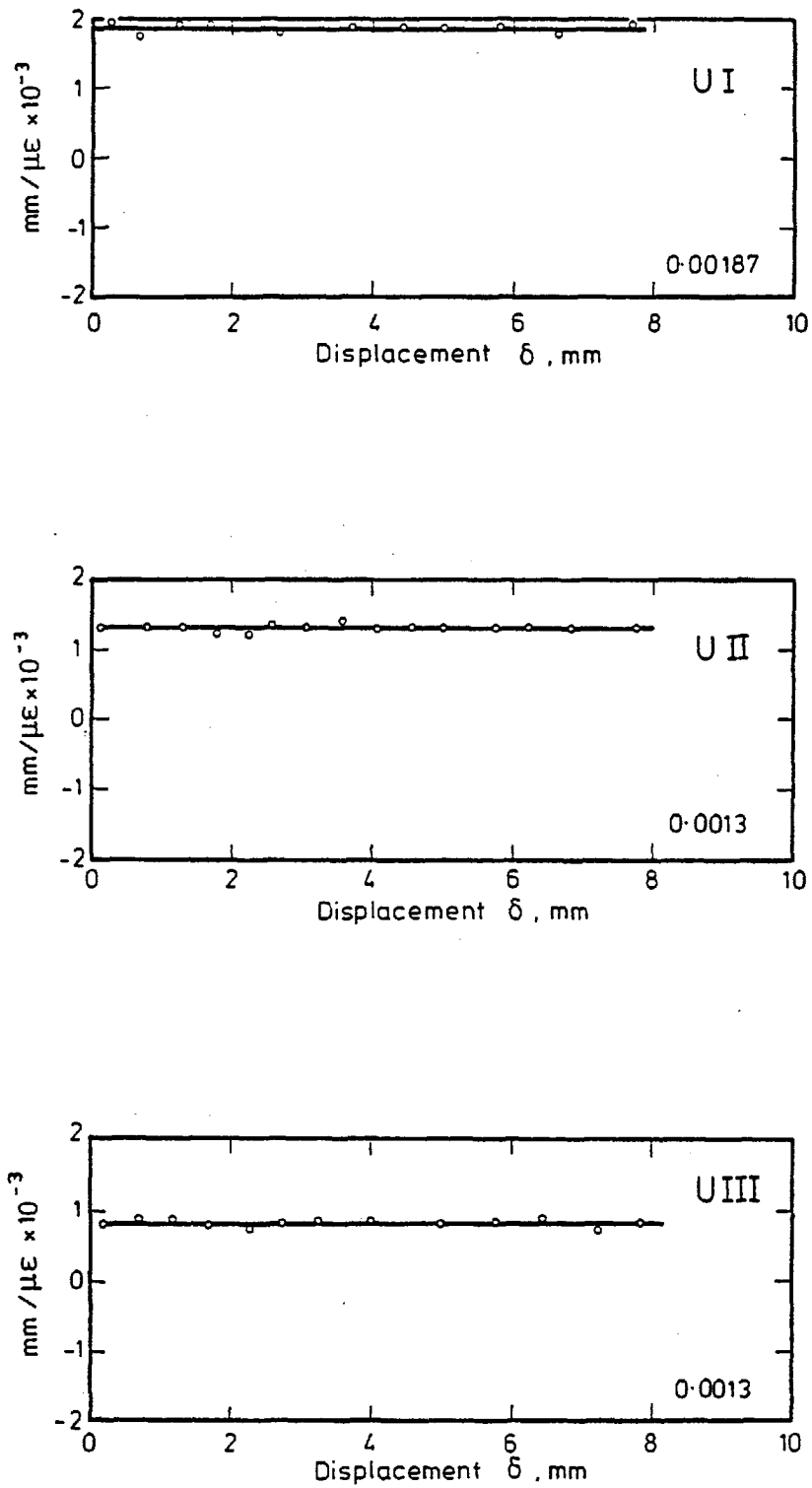


Fig. 4-11 Calibration curves of displacement transducers.

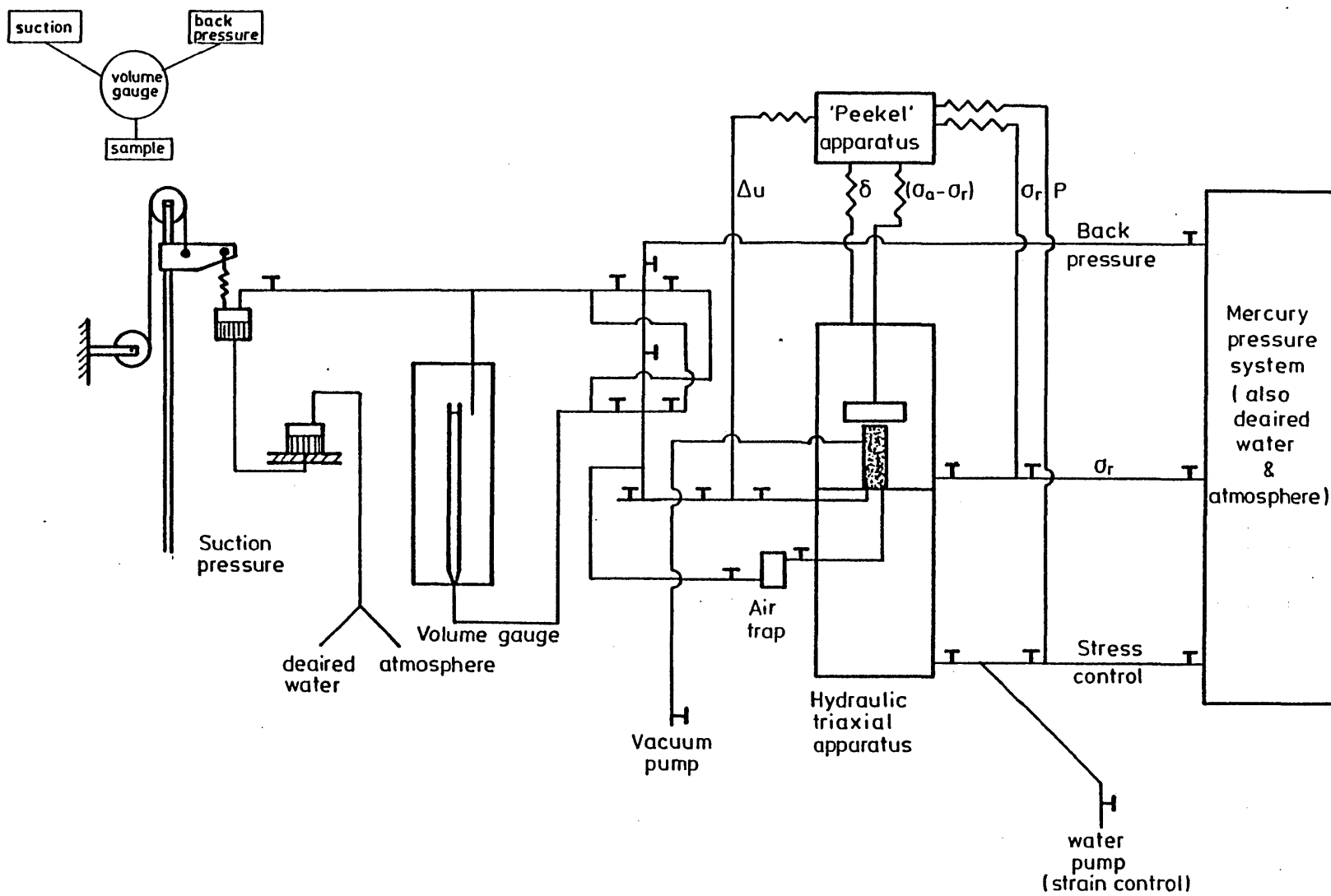


Fig. 4-12 Diagram of the system designed for resedimented samples.

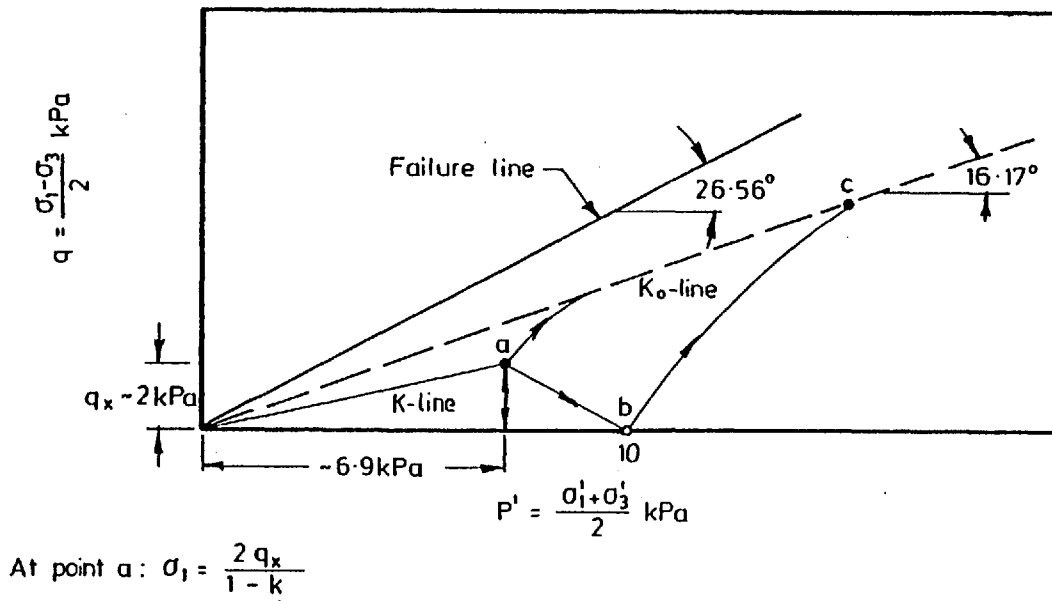


Fig. 4-13 Stress path during early stages of consolidating from slurry.

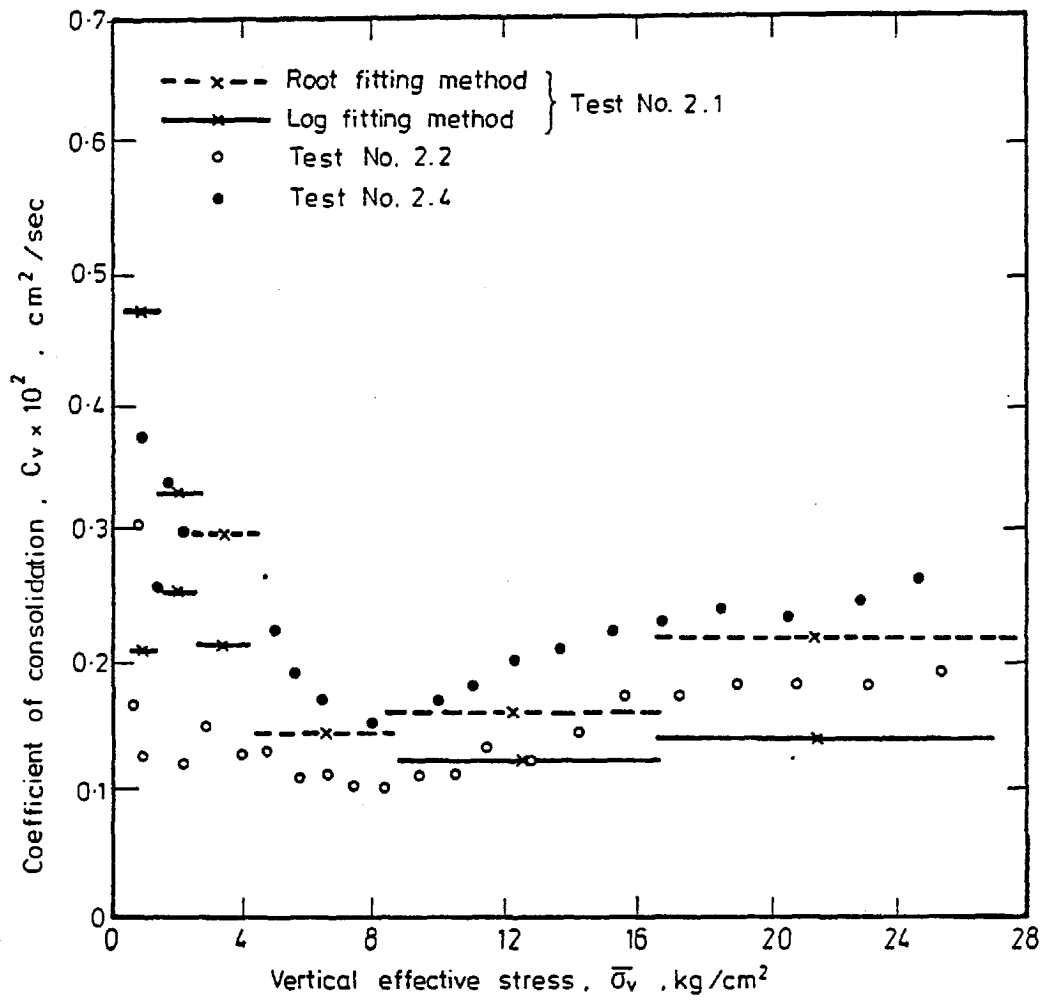
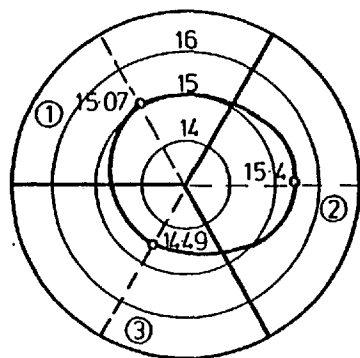
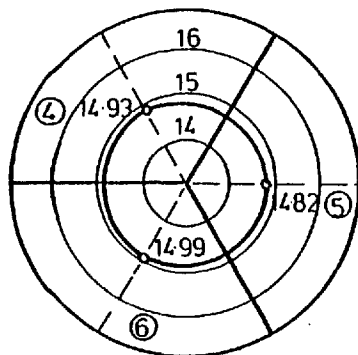


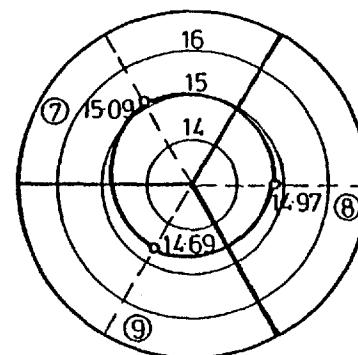
Fig. 4-14 $C_v - \bar{\sigma}_v$ relationship after Wissa (1971)



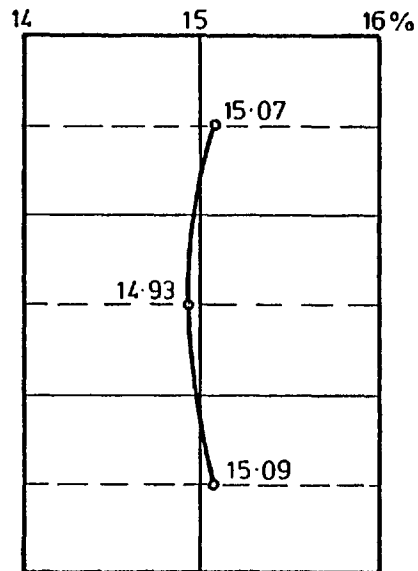
Top horizontal section



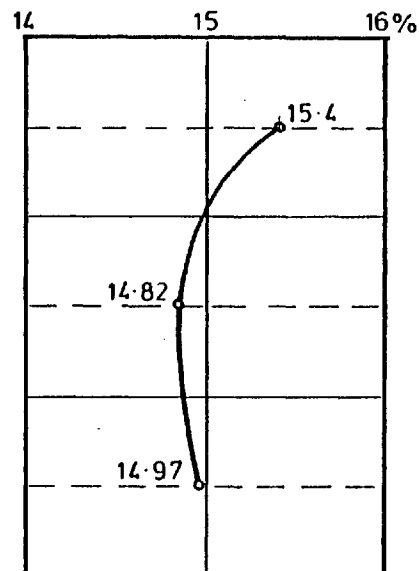
Middle horizontal section



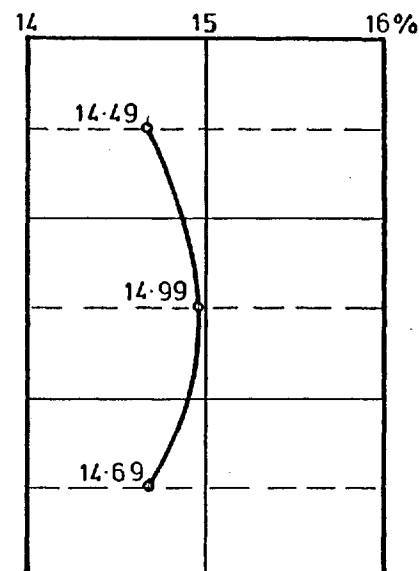
Bottom horizontal section



Vertical section A



Vertical section B



Vertical section C

Test A07P80

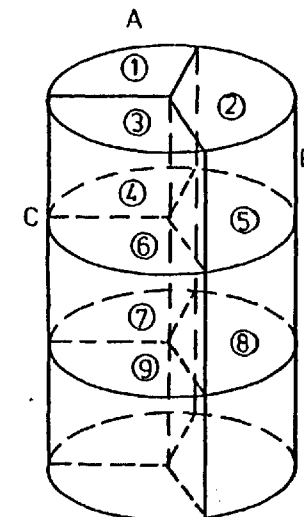
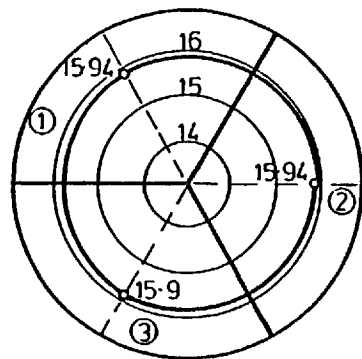
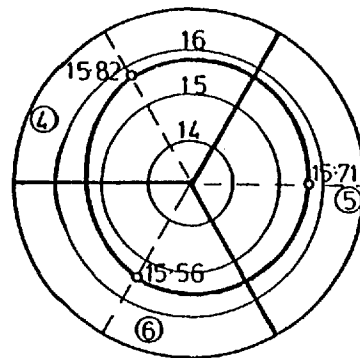


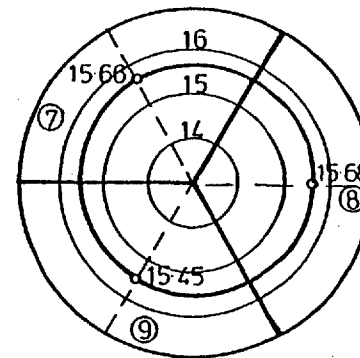
Fig. 4-15 Contours of water contents % after shearing



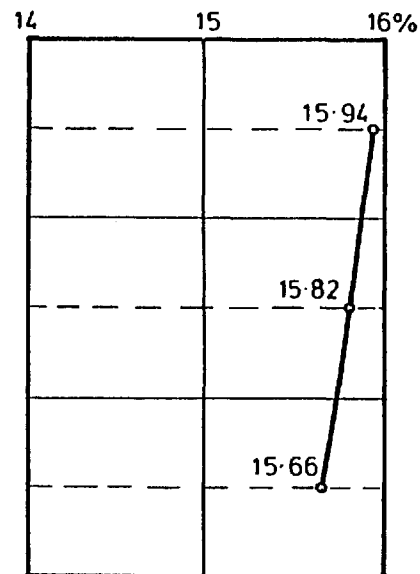
Top horizontal section



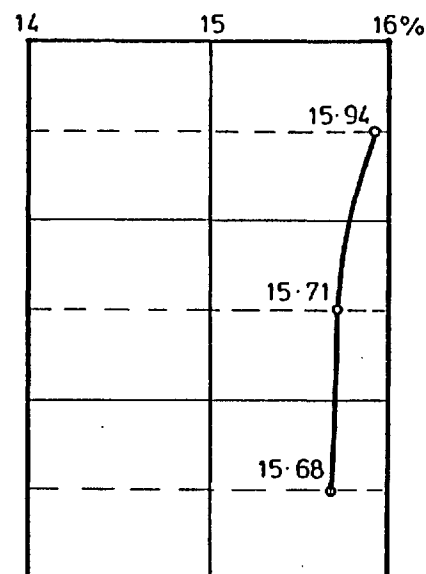
Middle horizontal section



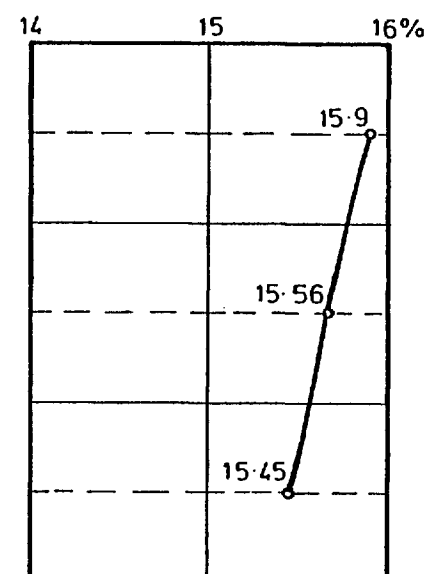
Bottom horizontal section



Vertical section A



Vertical section B



Vertical section C

Test SMP4

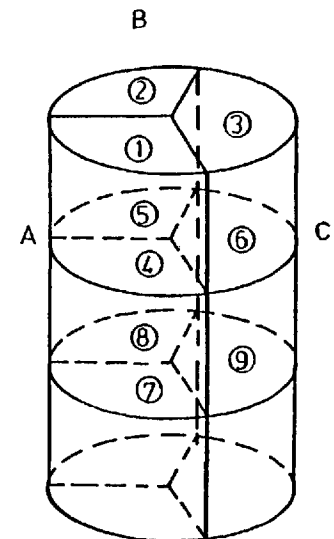


Fig. 4-16 Contours of water content % after shearing.

RING
SHEAR
TEST 2

Thin
Section
⇌

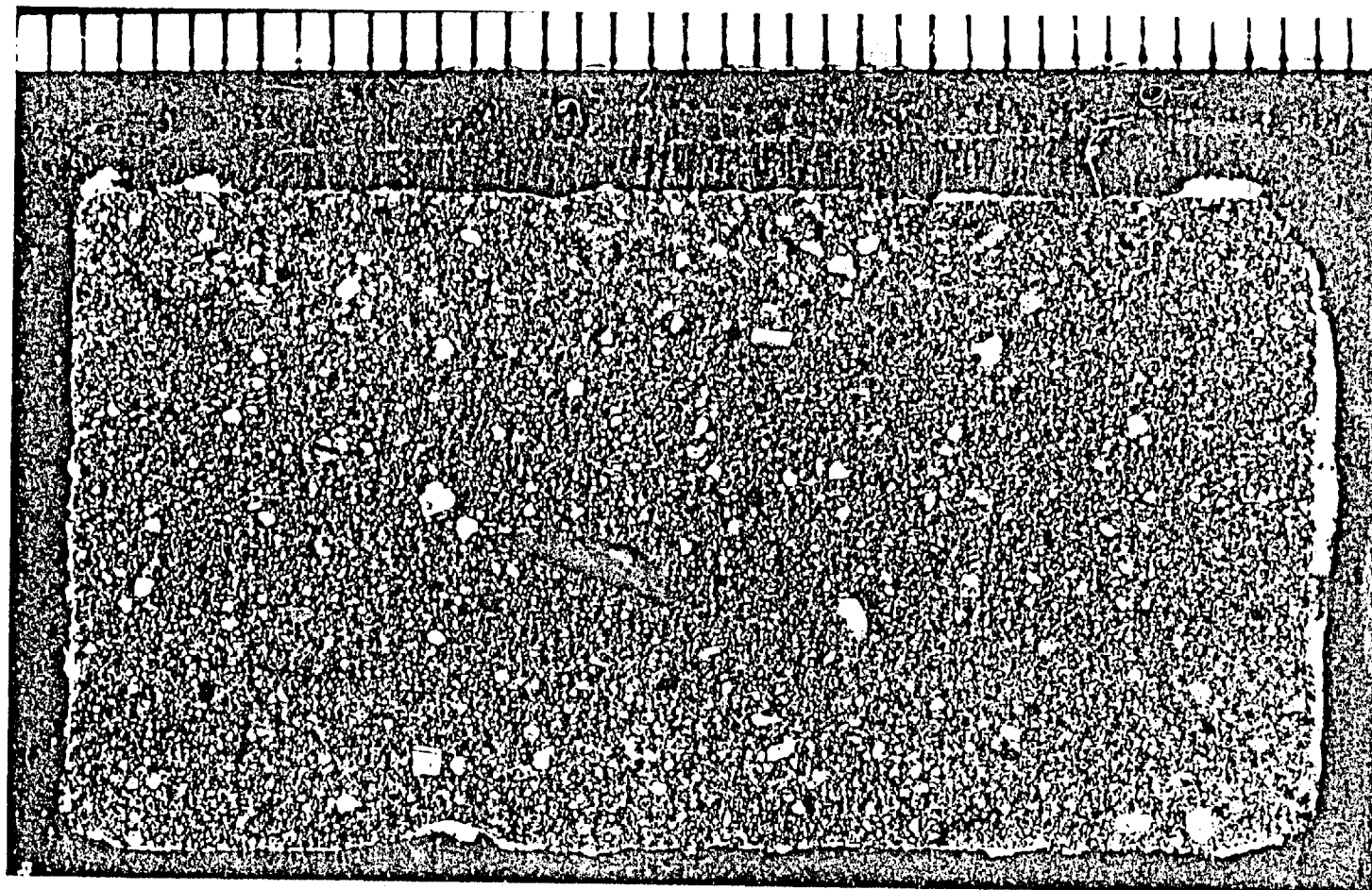


FIG:4-17

CHAPTER 5

TEST RESULTS

5.1 TRIAXIAL TESTS

Some important features of the results of the tests of all groups are tabulated in tables (5-1):(5-10). The complete results are presented graphically in Appendix (3).

In Appendix (3) the test results of each consolidated undrained (CU) and unconsolidated undrained (UU) are presented in six spaces:-

- (i) $W\% - P'$ where $P' = \frac{1}{2}(\sigma_1' + \sigma_3')$
- (ii) $q - p'$ where $q = \frac{1}{2}(\sigma_1' - \sigma_3')$
- (iii) $q - \epsilon_1\%$ where $\epsilon_1 = \epsilon_{v1} = \text{axial strain}$
- (iv) $\Delta U - \epsilon_1\%$
- (v) $\sigma_1'/\sigma_3' - \epsilon_1\%$
- (vi) $\epsilon_1/2q - \epsilon_1\%$ for hyperbolic transformation
(see 6-8)

The only space describing preshear behaviour is the first one (W-P') in which the consolidation history is described. At the start of shearing, parameters of all spaces were recorded simultaneously so that one can relate any point in any of the above spaces to its corresponding parameters in the other spaces.

In case of drained tests, where the variation $\Delta U - \epsilon_1\%$ did not exist, the relationship between the volumetric strain $v\% (= \Delta V/V_0\%)$ has been plotted versus $\epsilon_1\%$. The compressive volumetric strain has a downward direction.

The abscissa of the $(q - \epsilon_1\%)$ space is drawn with two different scales. The first, which is the normal one goes up to about 28%, the second is ten times more sensitive and goes up to about 2.8%. Both scales share the same ordinate. The second and more sensitive scale was for detecting the deformation characteristics at small strains. This detection was, however, missed in some tests.

In those cases when pore pressures were not recorded, as in some CU tests of WCR group ($R_b = 0.0762$ mm/min, $R_c = 0.7620$ mm/min and $R_d = 7.620$ mm/min) and in test ST4 of ST group (shearing rate = 5.0 mm/min), the effective stress paths have not been drawn. It was also not possible to draw $\sigma_1'/\sigma_3' - \epsilon_1\%$.

The data was recorded every minute for the first 30 minutes then every 5 minutes for 90 minutes then every 15 minutes for 4 hours, then readings every 30 minutes for 18 hours, then hourly readings until the end of the test. For a 76.2mm long sample sheared at a rate of 0.005 mm/min. and with an axial strain at failure of 20%, around 126 recordings would have been taken during the test and at each recording all parameters were calculated. The curves

drawn track these points; for clarity not all the calculated points have been plotted.

The above procedure was followed in all CU and UU tests. It is only in UU tests that there was no preshear history and the (W-P') relationship has been drawn only for the shearing stage. In some tests on undisturbed samples the initial pore water tension was equalled by a combination of high σ_r and a back pressure to ensure saturation and to keep the magnitude of the initial effective stress unaltered. In these tests no preshear consolidation history would be apparent in the W-P' plots. Tests ST5 and ST6 were the only exception in the UU tests in that their results were presented in $q-\epsilon_1\%$, $\Delta U-\epsilon_1\%$ and σ_1'/σ_3' spaces for ST5 and only in $q-\epsilon_1\%$ space for ST6. This was because of unconventional steps taken during testing these two particular samples.

Any peculiarities in a test have been noted on its sheet of results (e.g. Failure along a slip plane instead of the usual bulging, missing or doubtful information, etc).

5.2 CONSOLIDATION TESTS

Results of consolidation tests are presented in Fig.(5-1): (5-15). They are all plotted in a (W- σ_v') space, and represent isotropic and anisotropic consolidation behaviour of the soil obtained from oedometer and triaxial tests on both remoulded and resedimented soil from slurry.

Figs.(5-1) to (5-5) represent results of ten oedometer tests. The first four tests OD1: OD4 are presented in Fig. (5-1): (5-4) where loops of unloading and reloading are shown at several stress levels. The rest of the ten tests, which were performed to cover a large portion of the (W-P') space, are presented in Fig. (5-5) (except OD2). Only the normal consolidation behaviour is shown in Fig.(5-5). Oedometer tests OD4,5 and 6 were on resedimented samples from slurry, the rest of the tests were on remoulded samples.

Fig.(5-6) and (5-7) represent the results of seven special oedometer tests in which the magnitude of the increment of loading was changed in each test. The increment $n = P'_m/P'_{m-1}$ was changed from the "usual" value of 2 up to 64. Tests OD11: OD14 have been grouped in Fig.(5-6) and OD15: OD17 in Fig.(5-7).

The range within which the normal consolidation behaviour of residimented soil samples from slurry lies during anisotrpoc consolidation in the triaxial apparatus is shown in Fig.(5-8). It has been calculated from the consolidation stages of tests of groups AN, A0, SMP and some ST tests. On the same Fig.(5-8), results of two tests on anisotropically consolidated remoulded samples (ARI & ARII) are also shown. Details of test ARII before and during undrained shearing are shown in Fig.(5-9) in the way described in item (5-1).

The range within which the normal consolidation behaviour of residimented soil samples from slurry lies during isotropic consolidation in the triaxial apparatus is shown in Fig.(5-10). It has been calculated from the consolidation stages of tests of groups IN, ON and some ST tests.

Fig.(5-11):(5.13) show results of three tests on remoulded samples consolidated isotropically in the triaxial apparatus. One loop of unloading and reloading was performed at high stress level on each of these samples.

Fig.(5-14) shows results of a special test (IS3X) performed on an isotropically consolidated sample which was resedimented from slurry in the triaxial apparatus. The sample was subjected to loops of unloading and reloading at several stress levels. In Fig.(5-15) normal consolidation behaviour of all samples isotropically consolidated in the triaxial apparatus is shown. The loops of unloading and reloading of test IS3X are shown dotted.

5.3 RING SHEAR TESTS

The ring shear tests were performed (RS1 & RS2) the results of which are presented in Fig.(5-16): (5-26). The results are presented in the form of relationships between the circumferential displacement δ and the vertical displacement Δh as well as between δ and $\phi' (= \tan^{-1} \tau/\sigma')$.

The above relationships are shown for each step of loading. At the end of each test, the results are grouped to show relationships between the normal stress σ' and the shear stress τ , $\sigma' - \phi'$ and $\sigma' - I_B\%$ (I_B is the brittleness index = $\tau_{\text{peak}} - \tau_{\text{residual}} / \tau_{\text{peak}}$). See Bishop 1967.

In test RS1 the normal stresses were decreased at each step. Fig.(5-16): (5-19) show the results of each step. In Fig.(5-20) the steps are grouped to give a general trend of behaviour for $\sigma' - I_B\%$, $\sigma' - \tau$ and $\sigma' - \phi'$.

In test RS2 another sequence of loading was performed. The sequence was as follows:-

- (i) step no.1: the soil was sheared under $\sigma'_1 = 176.81 \text{ kPa}$
- (ii) step no 2: the soil was relieved from shear, consolidated to $\sigma' = 353.62 \text{ kPa}$ and sheared.
- (iii) step no 3: the soil was relieved from shear and unloaded to $\sigma' = 176.81 \text{ kPa}$ and sheared.
- (iv) step no 4: the soil was relieved from shear, reconsolidated to $\sigma' = 353.62 \text{ kPa}$ and sheared.

These steps are presented in Fig.(5.21):(5.24) respectively. These steps are also presented in Fig.(5.25a) and the whole $\delta - \tau$ behaviour is summarized in Fig. (5.25b).

The final results of all steps of this test are grouped in Fig.(5.26) where trends of behaviour in terms of $\sigma' - I_B\%$ and ϕ' are shown.

TABLE 5-1

RESULTS OF TESTS OF GROUP G

Group	Test	Soil	Type	W _o * %	P' _o kPa	≠ OCR	Shear Rate mm/min	ε ₁ flr %	σ' ₁ σ' ₃ flr	q _{max} kPa	A _{flr}
G	T10	U	UU	13.96	206.85	-	.0005	15.47	2.83	60.26	1.17
	T12	U	UU	13.00	220.64	-	.0005	7.76	3.26	145.17	.32
	T1	U	UU	14.00	47.88	-	.005	14.48	3.37	134.27	-.25
	T2	U	UU	14.20	208.00	-	.005	14.60	2.96	132.00	.23
	T14	U	UU	12.91	206.85	-	.076	26.09	3.10	180.89	.10
	T24	R	CU	14.72	206.85	-	.0005	7.46	3.27	87.65	.73
	T31	R	CU	16.42	14.39	-	.0005 .0025	30.54	(6.55)	27.3	.06
	T27	R	CU	-	11.30	-	.0005	27.53	(5.16)	34.04	-.07
	T4	R	CU	14.23	343.00	-	.005	13.99	(4.09)	136.62	.94
	T21	R	CU	14.05	206.85	-	.005	15.55	3.00	91.31	.73
	T16	R	CU	14.20	206.85	-	.005	15.55	3.07	80.24	.80
	T18	R	CU	14.00	206.85	-	.005	17.10	3.03	91.88	.63
	T20	R	CU	13.45	206.85	-	.005	12.04	3.21	99.47	.59
	T30	R	CU	14.86	88.95	-	.0025	9.40	2.96	66.00	.16
	T302	R	CU	15.37	214.76	-	.0025	19.27	2.80	47.79	1.69
	T32	R	CU	17,18	12.34	-	.0125	24.57	3.6	31.33	-.19
	T13	R	CU	16.40	(43.69)	-	.0760	28.48	(33)	37.98	.34
	T19	R	CU	13.56	206.85	-	.0580	19.28	3.48	119.62	.46
	T26	R	CU	14.55	206.85	-	.1115	24.14	3.39	101.26	.60

TABLE 5-2

RESULTS OF TESTS OF GROUPS NS

Group	Test	Soil	Type	W _o * %	P' _o kPa	≠ OCR	Shear Rate mm/min	ε ₁ flr %	σ' ₁ σ' ₃ flr	q _{max} kPa	A _{flr}
NS	T17	U	UU	10.03	205.00	-	.005	4.89	3.67	358.04	-.08
	T23	R	CU	16.21	206.85	-	.0050	14.16	3.88	122.39	.50
	T25	R	CU	13.50	206.85	-	.5029	22.02	3.80	102.19	.73

* W_o% denotes the water content at the beginning of shearing

≠ OCR denotes the overconsolidation ratio

- q_{max} denotes (σ₁-σ₃)_{max}.

TABLE 5-3

RESULTS OF TESTS OF GROUP WCR

Group	Test	Soil Type	W_O^* %	P_O' kPa	$\neq$ OCR	Shear Rate mm/min	ϵ_1 Flr %	σ_1' σ_3' flr	q_{max} kPa	A_{flr}
WRC	Ra19	R	CU 19.00	6.10	-	0.00762	31.20	4.25	9.22	.02
	Ra18	R	CU 18.00	16.50	-	0.00762	28.60	3.49	16.95	.08
	Ra14	R	CU 14.10	103.18	-	0.00762	30.32	3.07	85.25	.12
	Ra12	R	CU 12.30	629.49	-	0.00762	23.51	2.99	221.63	.92
	Ra11	R	CU 11.00	800.00	-	0.00762	25.00	3.03	292.00	.88
	RUU19	R	UU 19.50	5.90	-	0.00500	29.13	2.63	10.10	-.32
	RaUU14	R	UU 14.85	204.90	-	0.00500	29.37	2.96	82.90	.73
	RaUU13	R	UU 13.49	334.90	-	0.00500	29.00	2.58	103.8	.98
	Rb19	R	CU 19.42	10.00	-	0.07620	32.18		12.0	
	Rb16	R	CU 16.55	80.00	-	0.07620	27.71		26.25	
	Rb15	R	CU 15.57	85.00	-	0.07620	29.37		47.58	
	Rb14	R	CU 14.48	100.00	-	0.07620	31.90		71.00	
	Rb12	R	CU 12.93	320.00	-	0.07620	31.23		182.50	
	Rb11	R	CU 11.33	430.00	-	0.07620	22.13		255.00	
	Rb9	R	CU 9.32	800.00	-	0.07620	26.32		357.50	
	Rc20	R	CU 20.91	8.00	-	0.76200	35.50		8.00	
	Rc18	R	CU 18.98	14.00	-	0.76200	35.80		15.00	
	Rc16	R	CU 16.50	80.00	-	0.76200	35.17		32.00	
	Rc15	R	CU 15.87	90.00	-	0.76200	35.57		40.25	
	Rc15 ₁	R	CU 15.05	96.53	-	0.76200	30.00		48.00	
	Rc14	R	CU 14.96	98.00	-	0.76200	35.27		66.55	
	Rc13	R	CU 13.39	206.85	-	0.76200	35.50		118.10	
	Rc11	R	CU 11.03	430.00	-	0.76200	34.57		198.00	
	RcUU19	R	UU 19.13	7.40	-	0.50000	29.53	4.67	11.60	.04
	RcUU15	R	UU 14.88	127.50	-	0.50000	29.53	3.66	81.50	.41
	RcUU15 ₁	R	UU 15.40	33.68	-	0.50000	29.53	3.58	63.07	-.12
	RcUU15 ₂	R	UU 15.00	12.24	-	0.50000	29.53	4.52	60.80	-.18
	RcUU12	R	UU 12.85	432.50	-	0.50000	29.53	3.21	179.88	.75
	Rd21	R	CU 21.95	6.00	-	7.62000	35.00		5.80	
	Rd19	R	CU 19.96	10.00	-	7.62000	35.00		10.45	
	Rd16	R	CU 16.58	80.00	-	7.62000	35.00		39.26	
	Rd15	R	CU 15.46	90.00	-	7.62000	35.00		47.63	
	Rd14	R	CU 14.49	100.00	-	7.62000	35.00		74.20	
	Rd12	R	CU 12.35	380.00	-	7.62000	35.00		159.00	
	Rd9	R	CU 9.46	780.32	-	7.62000	35.00		321.20	

TABLE 5-4

RESULTS OF TESTS OF GROUP V

Group	Test	Soil	Type	W _o %	P' _o kPa	OCR	Shear Rate mm/min	ε _i flr. %	σ' ₁ σ' ₃ flr.	q _{max} kPa	A _{flr.}
V	V1U	U	D	14.00	243.37	-	.0001	1.20	3.78	69.69	
	V1R	R	CD	15.30	175.00		.0001	5.20		62.20	
	V1R'	R	CD	15.30	158.00		.0001	23.00	3.00	160.40	
	V2U	U	D	12.00	155.00		.0001	2.40	4.48	58.09	
	V2R	R	CD	15.20	182.00		.0001	6.00	3.80	59.30	
	V3U	U	D	12.90	150.95		.0001	2.80	5.50	64.28	
	V3R	R	CD	15.60	155.00		.0001	10.70	3.40	49.80	
	V4U	U	D	12.50	155.00		.0001	0.80	3.24	44.89	
	V4R	R	CD	16.10	155.00		.0001	11.83	-	42.70	

TABLE 5-5

RESULTS OF TESTS OF GROUP IN

Group	Test	Soil	Type	W _o %	P' _o kPa	OCR	Shear Rate mm/min	ε _i flr. %	σ' ₁ σ' ₃ flr.	q _{max} kPa	A _{flr.}
IN	P20	S	CU	19.98	20.00	1.0	0.005	13.51	4.36	12.91	0.54
	P40	S	CU	19.17	40.00	1.0	0.005	11.34	5.81	12.62	1.38
	P60	S	CU	18.99	60.00	1.0	0.005	12.59	3.50	21.51	1.00
	P80	S	CU	17.80	80.00	1.0	0.005	13.27	4.25	32.60	0.93
	P160	S	CU	16.66	161.78	1.0	0.005	17.36	2.72	45.23	1.35
	P200	S	CU	16.70	200.00	1.0	0.005	16.40	2.89	52.40	1.38
	P300	S	CU	15.15	300.00	1.0	0.005	11.16	3.34	81.64	1.44
	P630	S	CU	13.50	629.00	1.0	0.005	11.00	3.00	159.60	1.31
	P800	S	CU	13.08	800.00	1.0	0.005	19.96	3.14	197.90	1.55
	P40	S	CU	19.15	40.00	1.0	0.500	27.54	3.27	14.67	0.92
	P50	S	CU	17.85	50.00	1.0	0.500	30.50	4.07	18.77	1.01
	P80	S	CU	17.70	80.00	1.0	0.500	29.96	3.86	24.66	1.27
	P200	S	CU	15.50	200.00	1.0	0.050	17.13	3.27	53.97	1.41
	P200	S	CU	15.26	200.00	1.0	0.500	30.46	3.43	63.21	1.17
	P800	S	CU	13.26	800.00	1.0	0.500	28.15	3.29	240.51	1.23

TABLE 5-6

RESULTS OF TESTS OF GROUP IO

Group	Test	Soil	Type	Wo %	P' _o kPa	OCR	Shear Rate mm/min	ϵ_1 flr. %	σ'_1 $\frac{\sigma'_1}{\sigma'_3}$ flr	q _{max} kPa	A _{flr.}
IO	O2P40	S	CU	18.56	40	2	0.005	18.49	2.95	22.90	.36
	O4P40	S	CU	17.51	40	4	0.005	25.11	2.88	36.17	.02
	O10P40	S	CU	15.40	40	10	0.005	15.07	3.10	72.43	-.20
	O20P40	S	CU	14.60	40	20	0.005	16.72	2.87	100.00	-.34
	O7P50	S	CU	15.42	50	7	0.005	28.93	3.05	70.81	-.13
	O2P80	S	CU	17.20	80	2	0.005	18.02	4.07	43.51	.6
	O4P80	S	CU	15.45	80	4	0.005	22.28	3.17	78.21	.04
	O10P80	S	CU	14.40	80	10	0.005	22.01	2.87	133.91	-.24
	O2P200	S	CU	14.95	200	2	0.005	10.88	2.89	82.47	.68
	O4P200	S	CU	13.78	200	4	0.005	10.66	3.06	159.78	.14
	O2P40	S	CU	16.97	40	2	0.500	30.44	3.37	28.71	.27
	O4P40	S	CU	15.96	40	4	0.500	31.27	3.45	43.67	.05
	O2P50	S	CU	17.54	50	2	0.500	30.46	3.64	27.04	.55
	O4P50	S	CU	16.11	50	4	0.500	30.15	4.14	59.68	.10
	O7P50	S	CU	15.20	50	7	0.500	30.35	3.34	76.86	-.10
	O2P200	S	CU	14.50	200	2	0.500	31.17	3.49	110.90	.50
	O4P200	S	CU	13.46	200	4	0.500	28.73	3.27	161.96	.18

TABLE 5-7

RESULTS OF TESTS OF GROUP AN

Group	Test	Soil	Type	Wo %	P' _o kPa	OCR	Shear Rate mm/min	ϵ_1 flr. %	σ'_1 $\frac{\sigma'_1}{\sigma'_3}$ flr.	q _{max} kPa	A _f CU v% CD max
AN	P40	S	CU	21.50	44.29	1.0	0.005	26.23	3.17	12.34	.98
	P80	S	CU	18.10	80.78	1.0	0.005	0.40	3.13	32.10	.37
	P160	S	CU	16.45	155.06	1.0	0.005	0.10	2.18	57.25	.05
	P320	S	CU	15.00	320.00	1.0	0.005	0.07	2.43	133.16	.03
	P40	S	CD	19.70	40.00	1.0	0.005	16.16	3.21	30.50	+3.27
	P80	S	CD	17.50	80.00	1.0	0.005	19.62	3.04	64.40	+3.30
	P160	S	CD	16.20	161.20	1.0	0.005	17.28	2.91	110.50	+2.98
	P320	S	CD	15.20	(320)	1.0	0.005	26.06	3.41	265.90	+3.27

TABLE 5-8

RESULTS OF TESTS OF GROUP AO

Group	Test	Soil Type	Wo %	P' _o kPa	OCR	Shear Rate mm/min	ϵ_1 flr %	$\frac{\sigma'_1}{\sigma'_3}$ flr.	q _{max} kPa	A _f CU % CD
AO	O2P40	S CU	17.54	40.00	2	0.005	15.15	2.63	24.70	0.13
	O4P40	S CU	16.30	40.00	4	0.005	16.00	3.13	38.31	0.14
	O1OP40	S CU	16.32	40.00	10	0.005	31.60	3.38	36.41	0.43
	O4P80	S CU	15.75	76.56	4	0.005	7.83	3.02	87.49	-0.06
	O7P80	S CU	14.97	80.00	7	0.005	19.05	3.42	97.24	0.09
	O2P160	S CU	15.49	146.51	2	0.005	15.87	2.70	91.90	0.15
	O2P40	S CD	18.00	40.00	2	0.005	22.74	4.18	55.03	+2.30
	O4P40	S CD	18.00	40.00	4	0.005	24.76	3.15	50.60	+2.40
	O1OP40	S CD	17.50	40.00	10	0.005	29.23	3.03	63.00	+5.60

TABLE 5-9

RESULTS OF GROUP SMP

Group	Test	Soil Type	Wo %	P' _o kPa	OCR	Shear Rate mm/min	ϵ_1 flr %	$\frac{\sigma'_1}{\sigma'_3}$ flr	q _{max} kPa	A _f flr.
SMP	SMP1	S CU	16.70	70	-	0.005	11.71	3.42	55.31	0.21
	SMP2	S CU	15.67	(172.8)	-	0.005	8.79	2.26	65.64	0.10
	SMP3	S -	16.30	160	-	0.005	16.74	3.08	85.25	0.46
	SMP4	S -	15.74	160	-	0.005	16.54	(5.71)	65.00	0.33

TABLE 5-10

RESULTS OF GROUP ST.

Group	Test	Soil Type	Wo %	P' _o kPa	OCR	Shear Rate mm/min	ϵ_1 flr %	$\frac{\sigma'_1}{\sigma'_3}$ flr	q _{max} kPa	A _f flr.
ST	ST1	S (I)CU	15.63	200.00	-	0.005	31.80	3.35	61.77	1.19
	ST2	S (I)CU	16.65	99.42	-	0.005 ⁽¹⁾	32.89	3.04	48.07	0.55
	ST3	S (A)CU	15.50	189.61	-	0.005	25.18	2.40	78.70	0.72
	ST4	S (I)CU	15.50	200.00	-	5.000	30.37	-	61.86	-
	ST5	R UU	13.30	129.92	-		32.03	3.48	109.40	0.19
	ST6	R UU	14.38	-	-	variable	21.65	-	62.58	-
	ST7	S (I+A)CU	15.45	162.50	-	0.005	32.36	2.10	60.09	0.03

(1) increased from 0.0001: 0.005 mm/min.

FIG: 5-1
 TEST: OD1
 SOIL: R
 GROUP: C

W/C %

25
24
23
22
21
20
19
18
17
16
15
14
13
12

5

10

100

σ'_v KPa

800

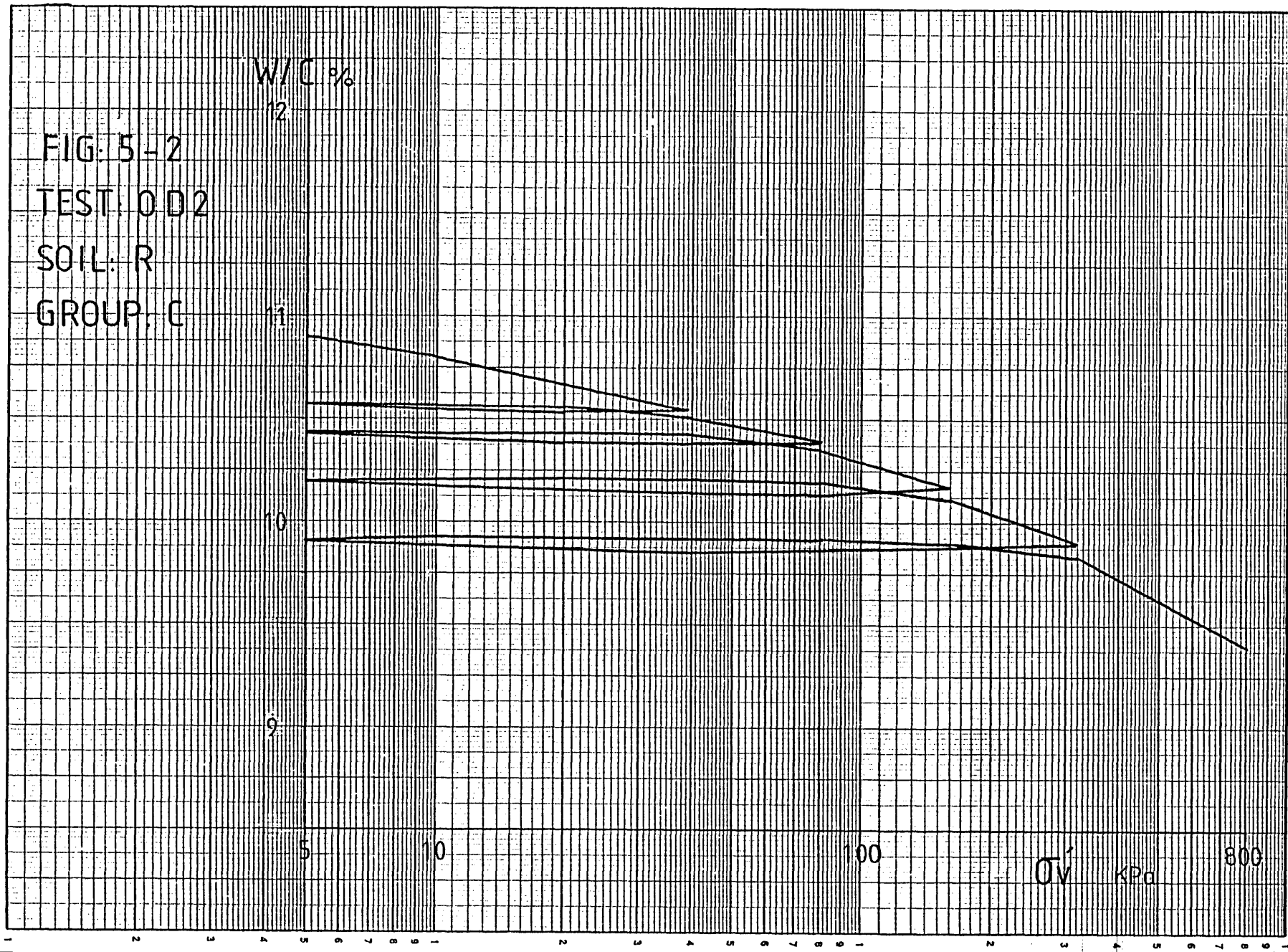


FIG: 5-3
 TEST: OD3
 SOIL: R
 GROUP: C

W/C %

25
24
23
22
21
20
19
18
17
16
15
14
13
12

5

10

100

σ_v'

kPa

800

FIG: 5-4
TEST: OD4
SOIL: S
GROUP: C

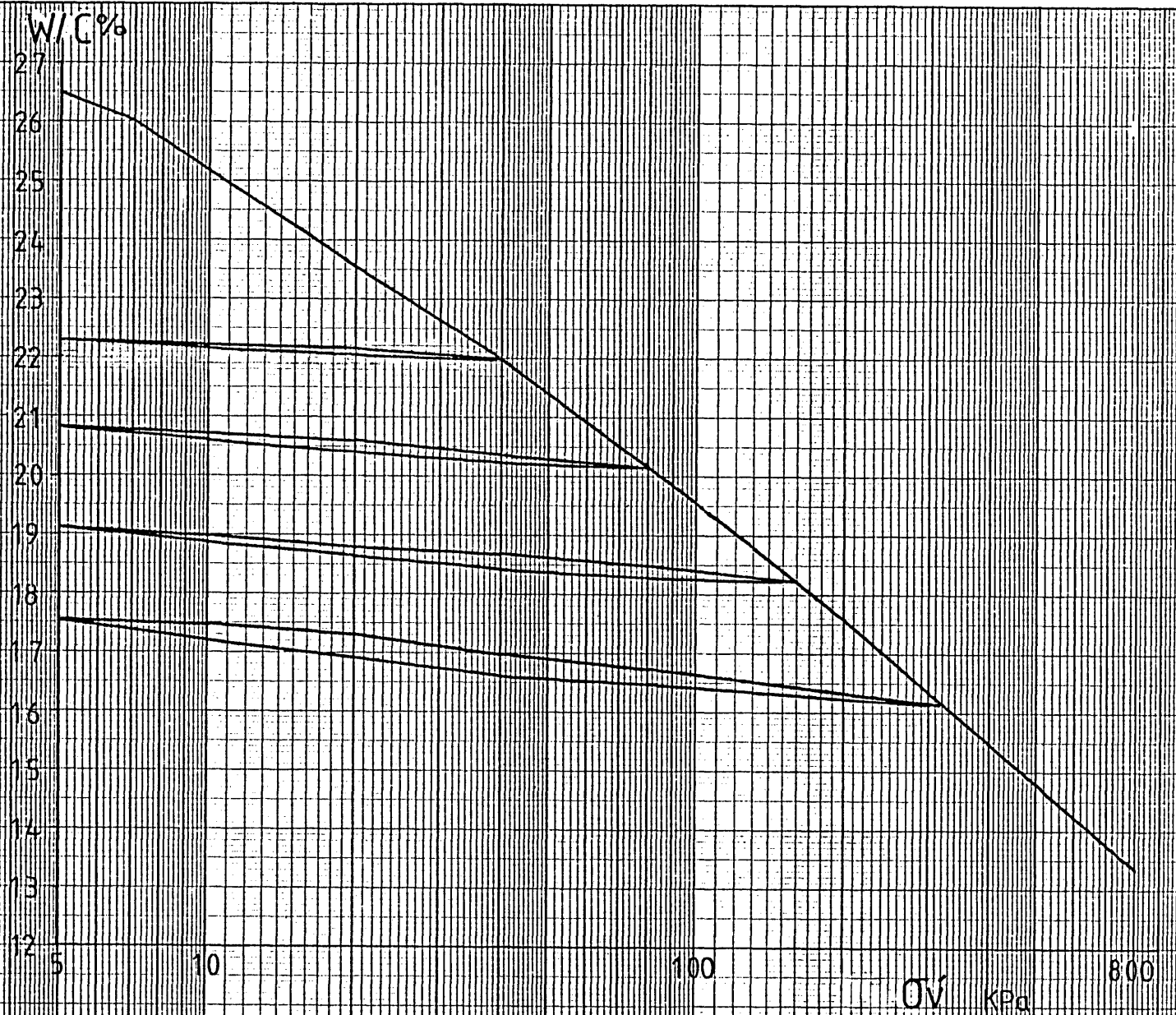
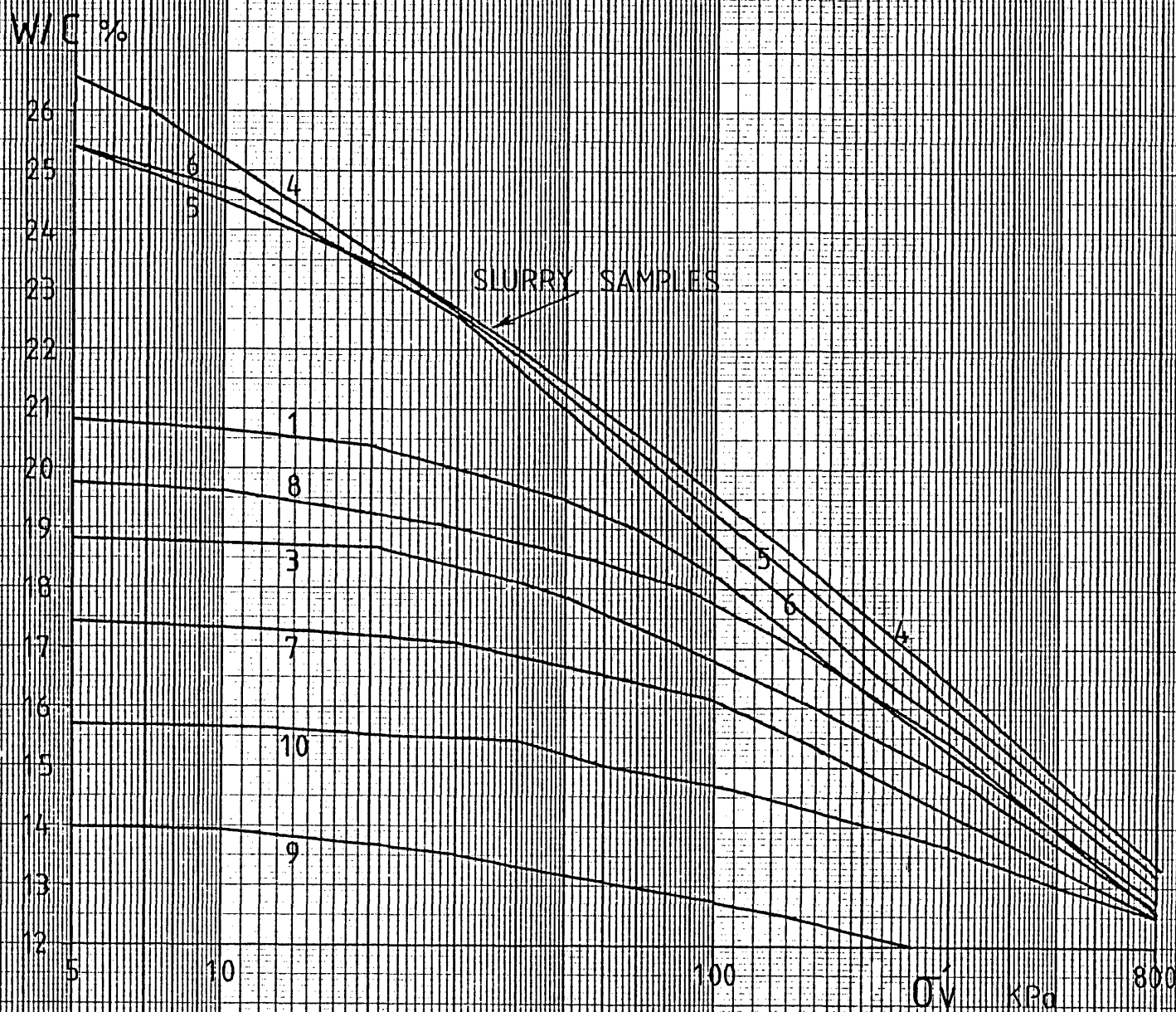


FIG. 5-5
N.C. LINES
OF TESTS
OD 1:10
(EXCEPT OD 2)



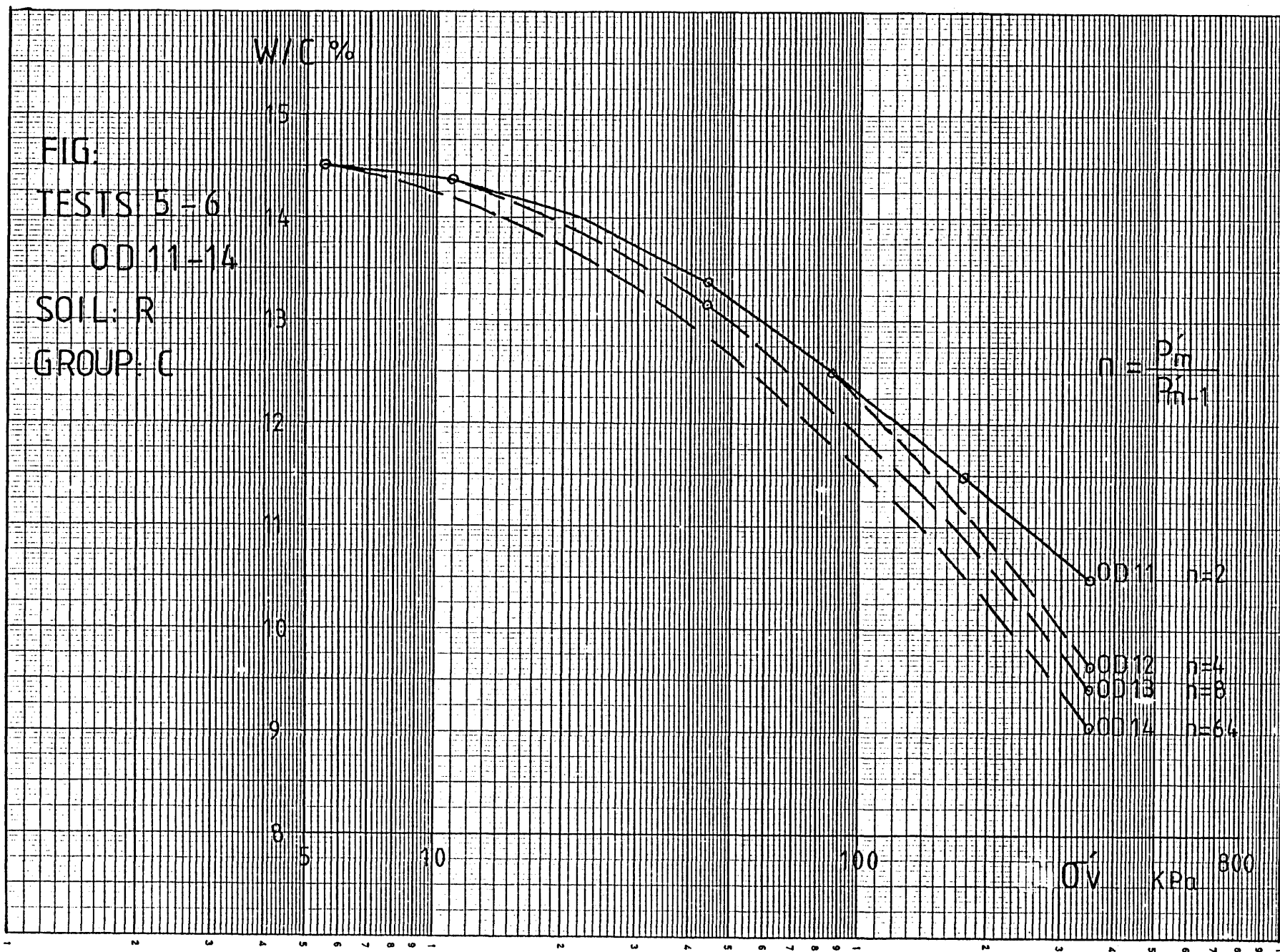


FIG: 5-7

TESTS:

OD 15-17

SOIL: R

GROUP: C

W/C %

18

17

16

15

14

13

12

11

10

9

8

7

6

5

5

10

100

800

$$n = \frac{P_1}{P_{n-1}}$$

$n=20$

OD 17

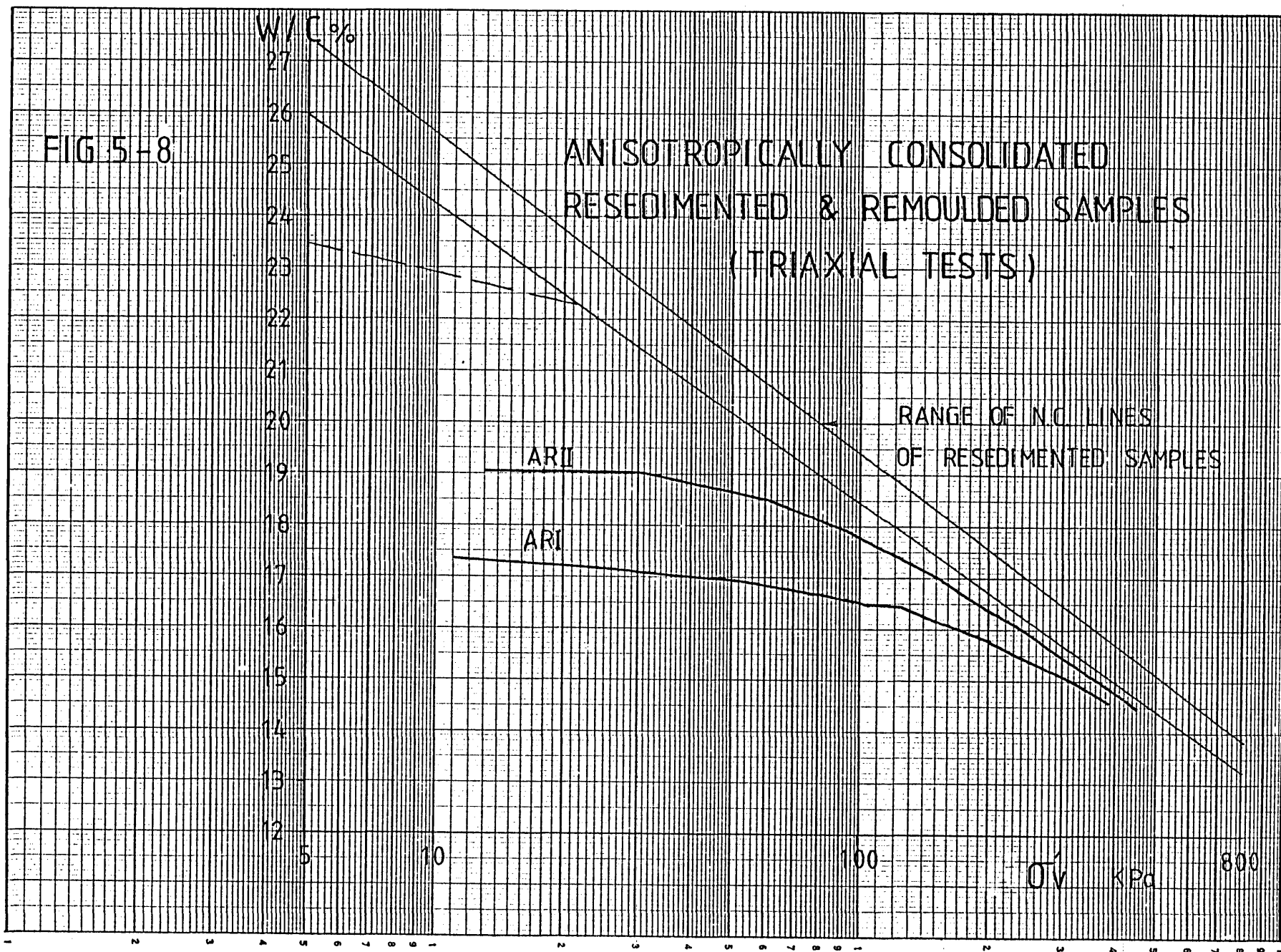
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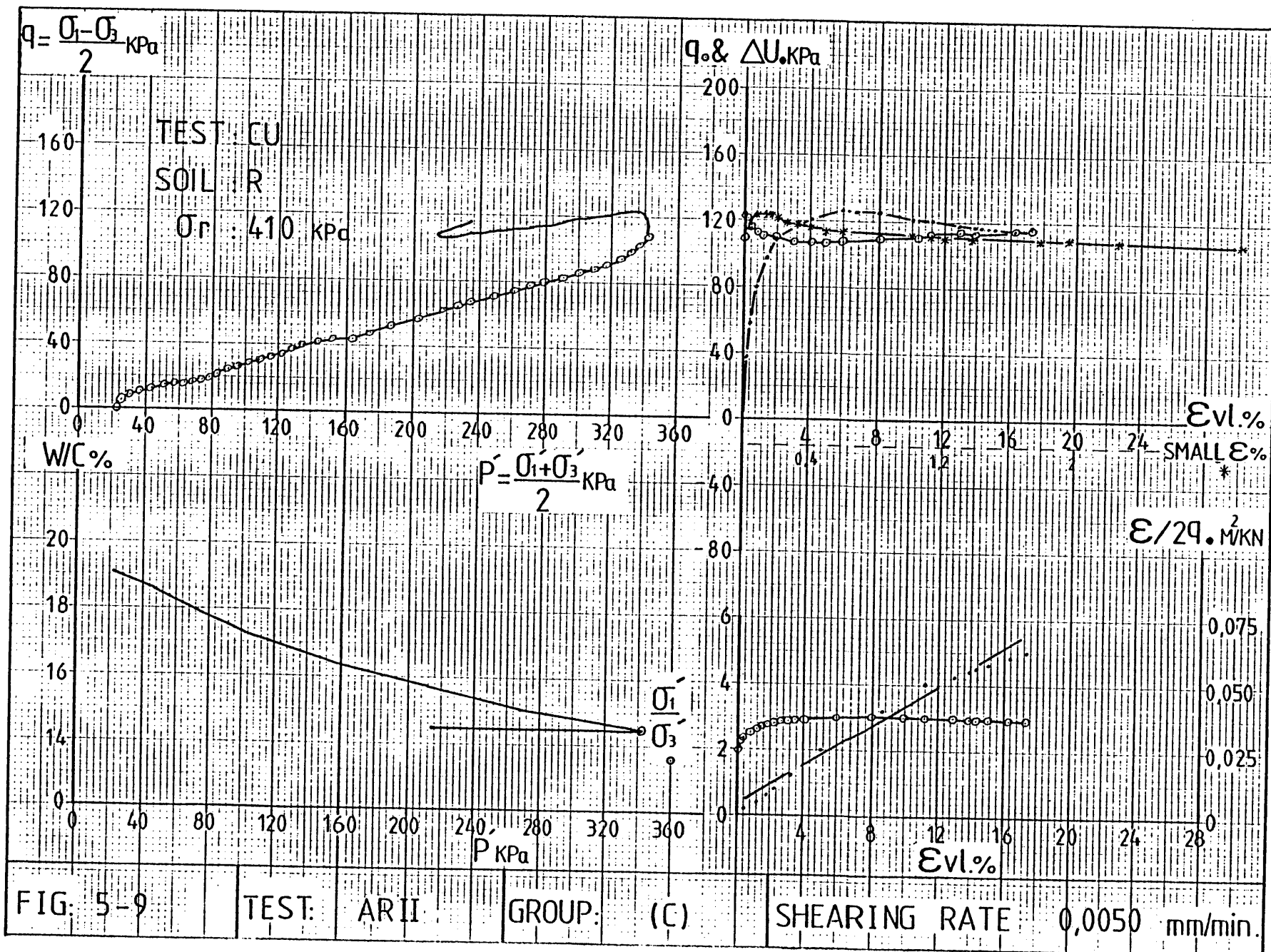
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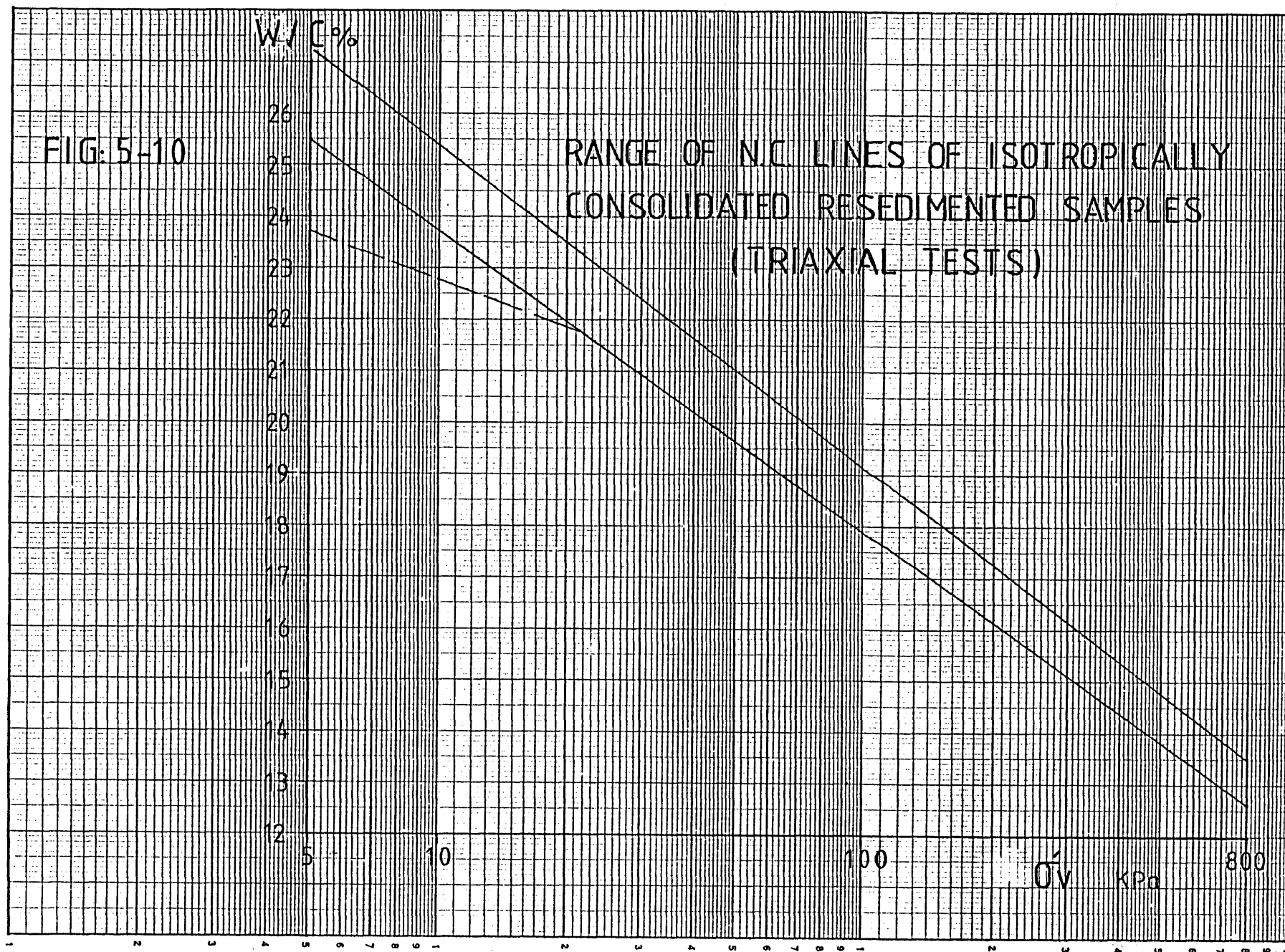
OD 16

OD 15

σ_v KPa







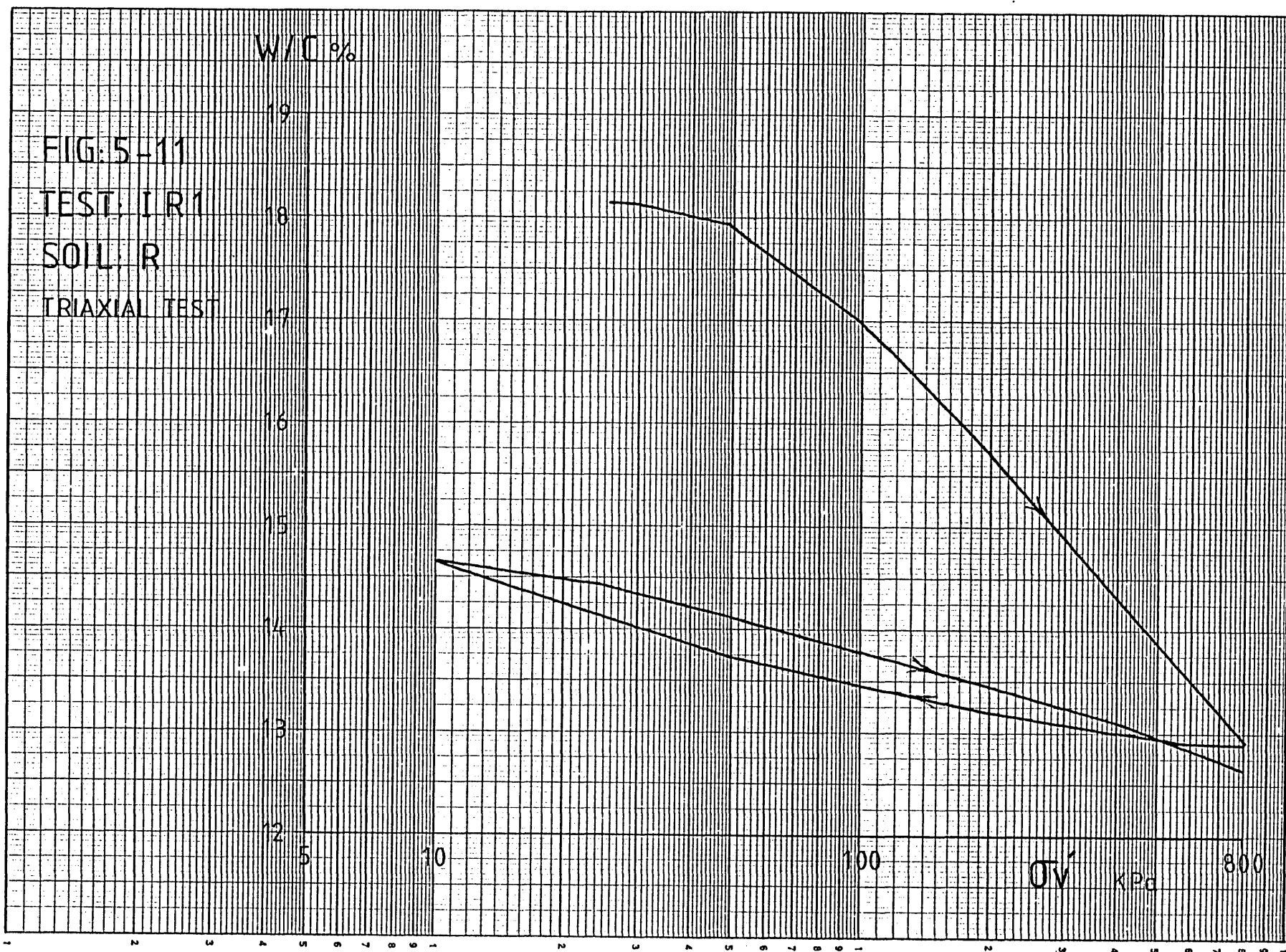


FIG: 5-12
 TEST: I R 2
 SOIL: R
 TRIAXIAL TEST

W/C %

19

18

17

16

15

14

13

12

5

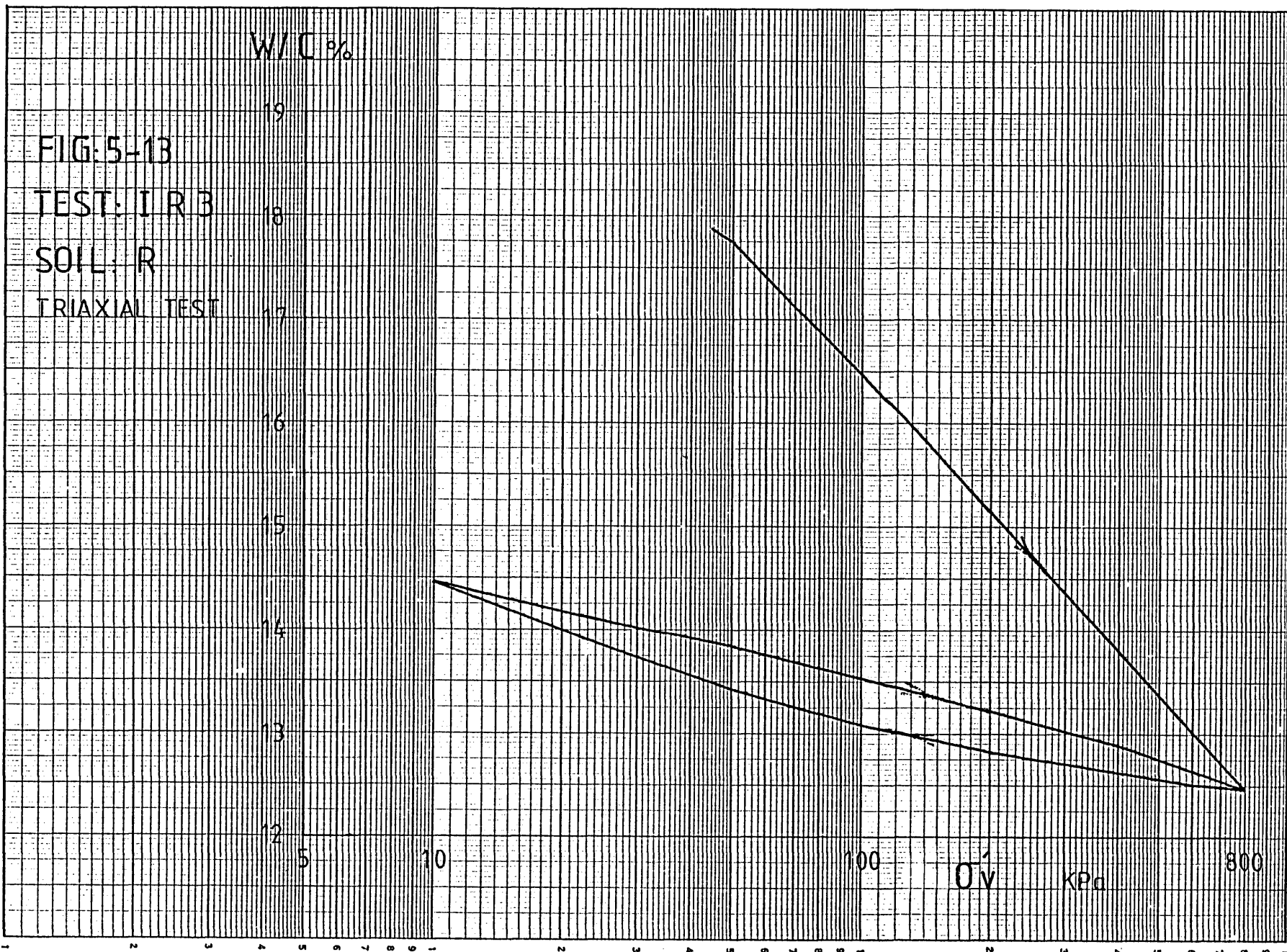
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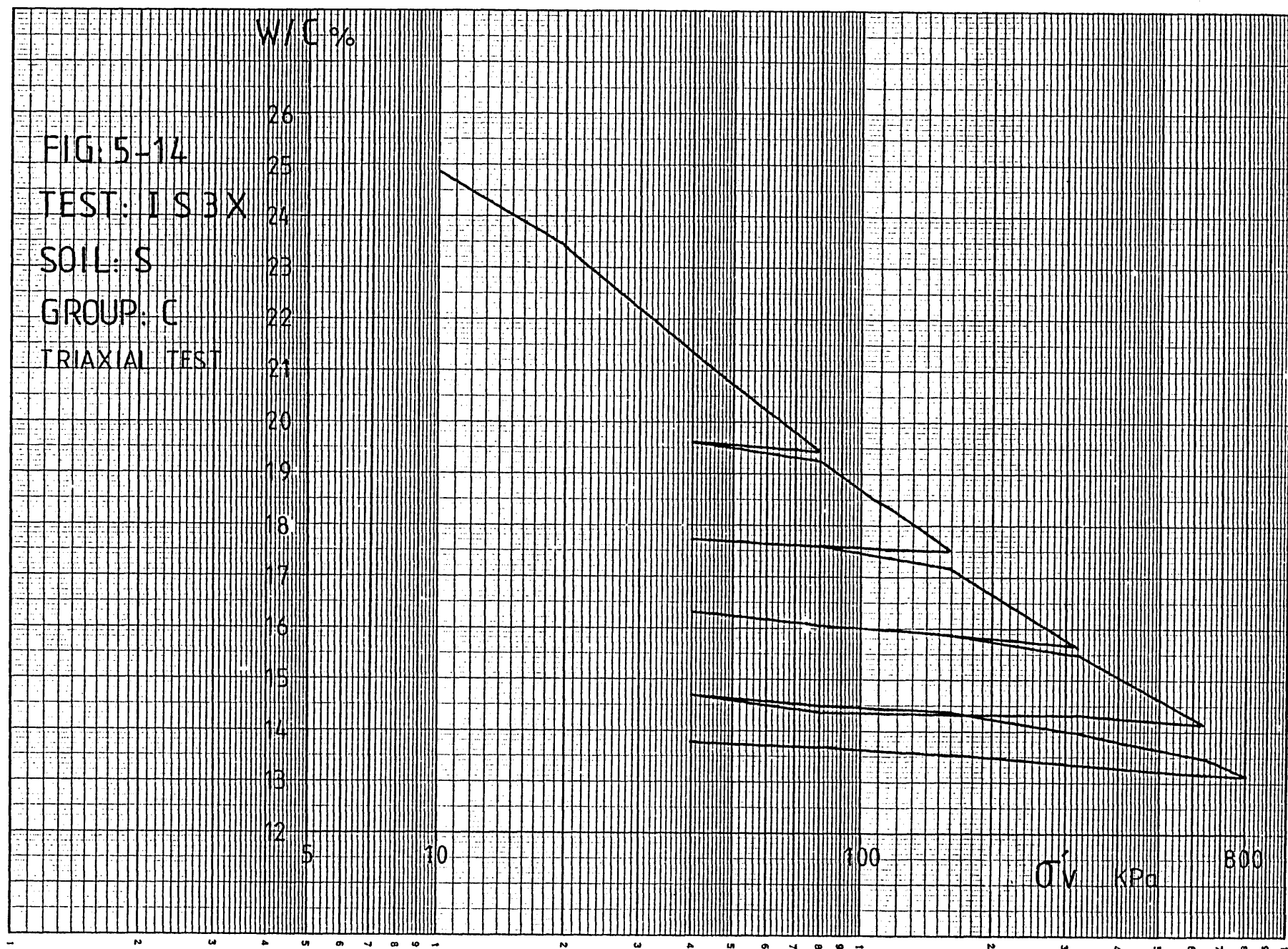
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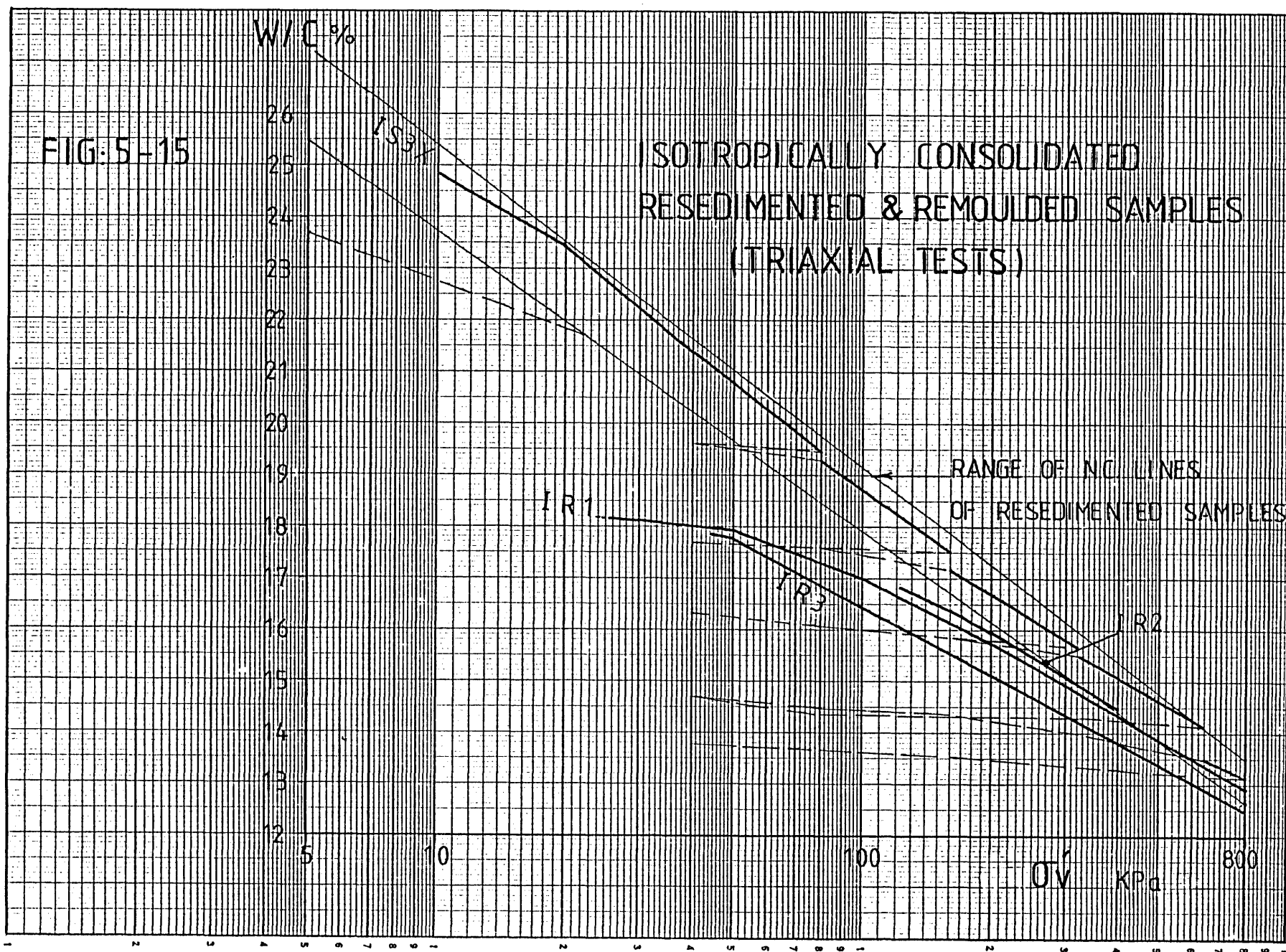
σ_v

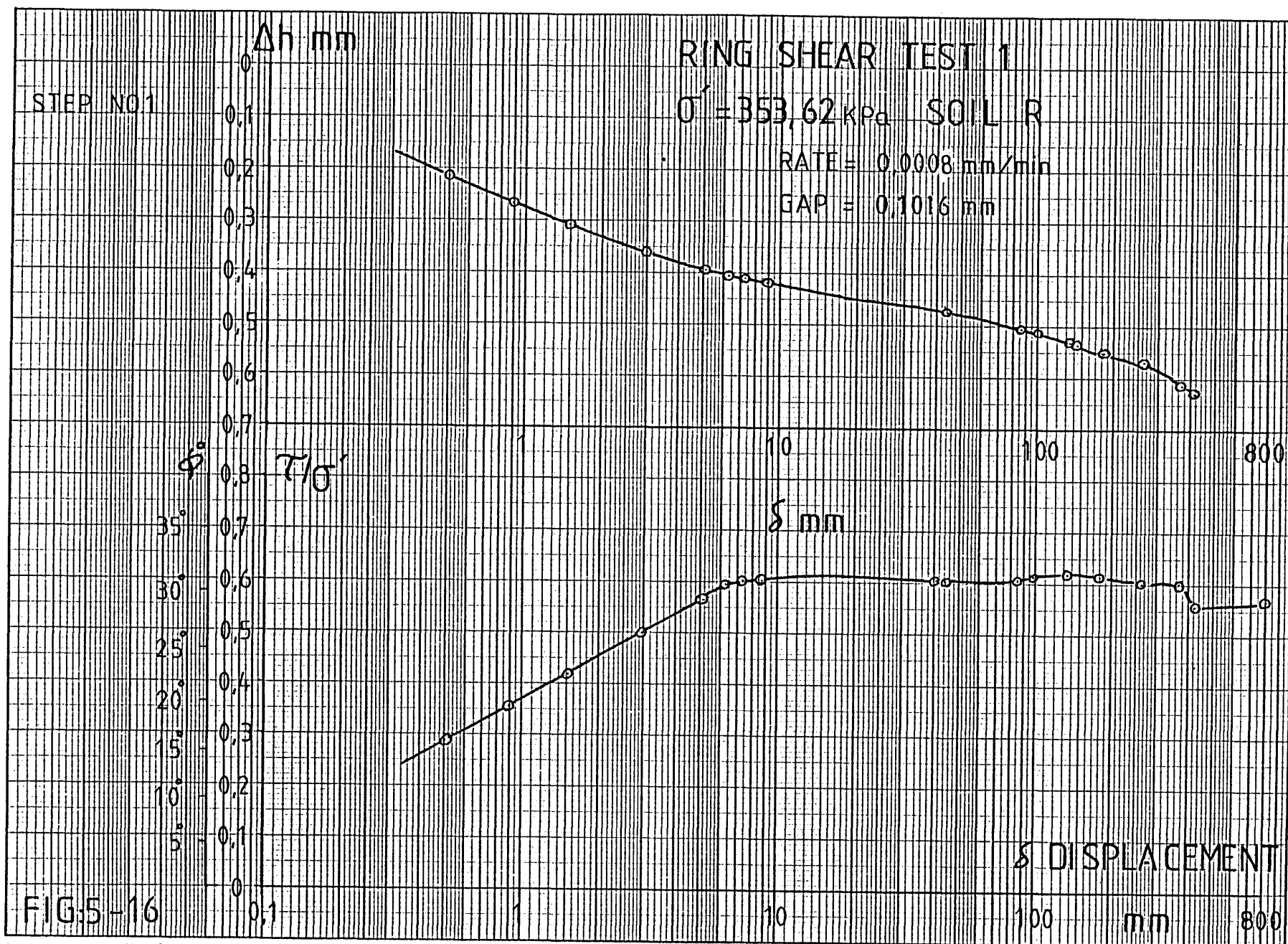
KPa

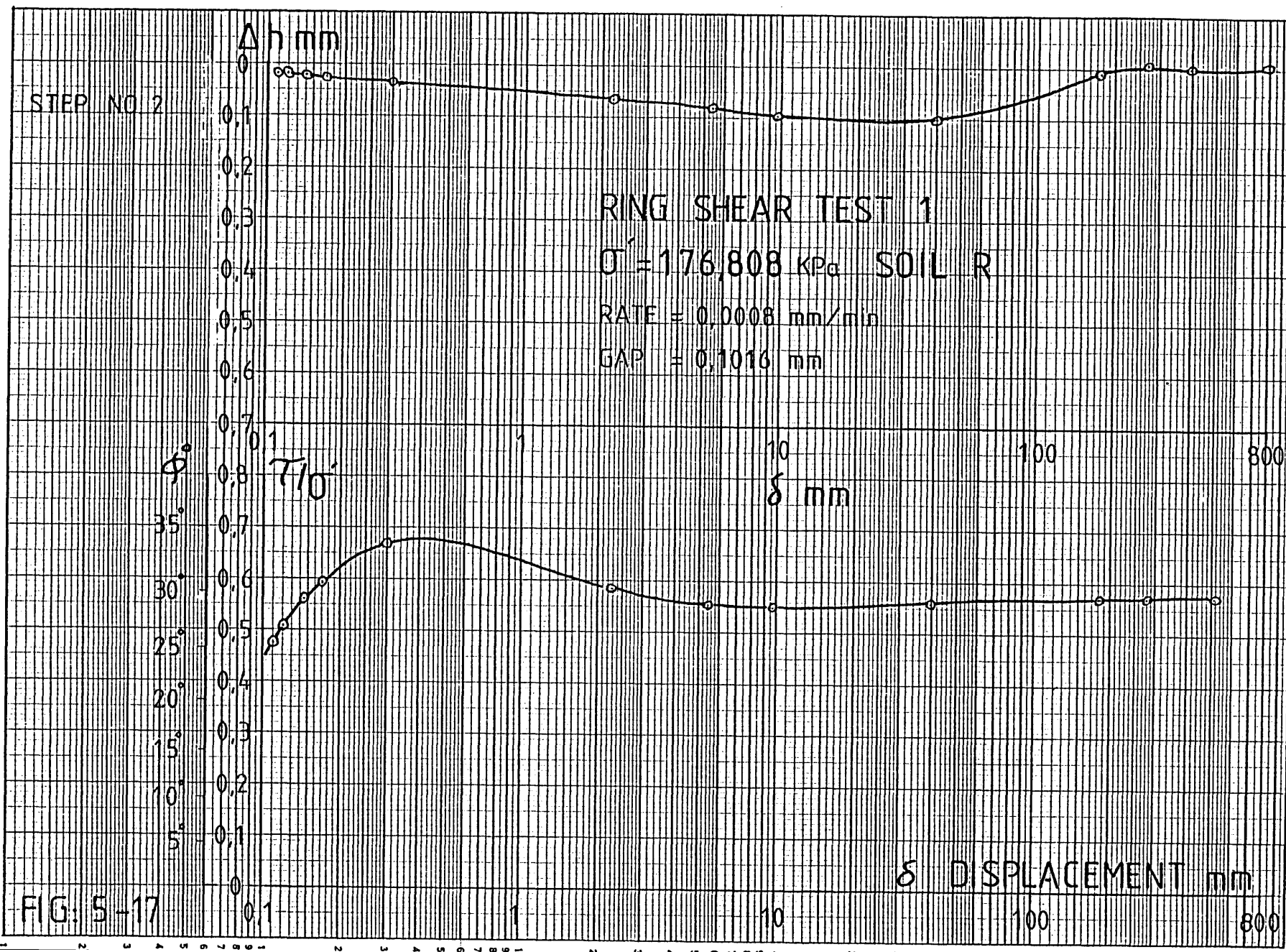
800

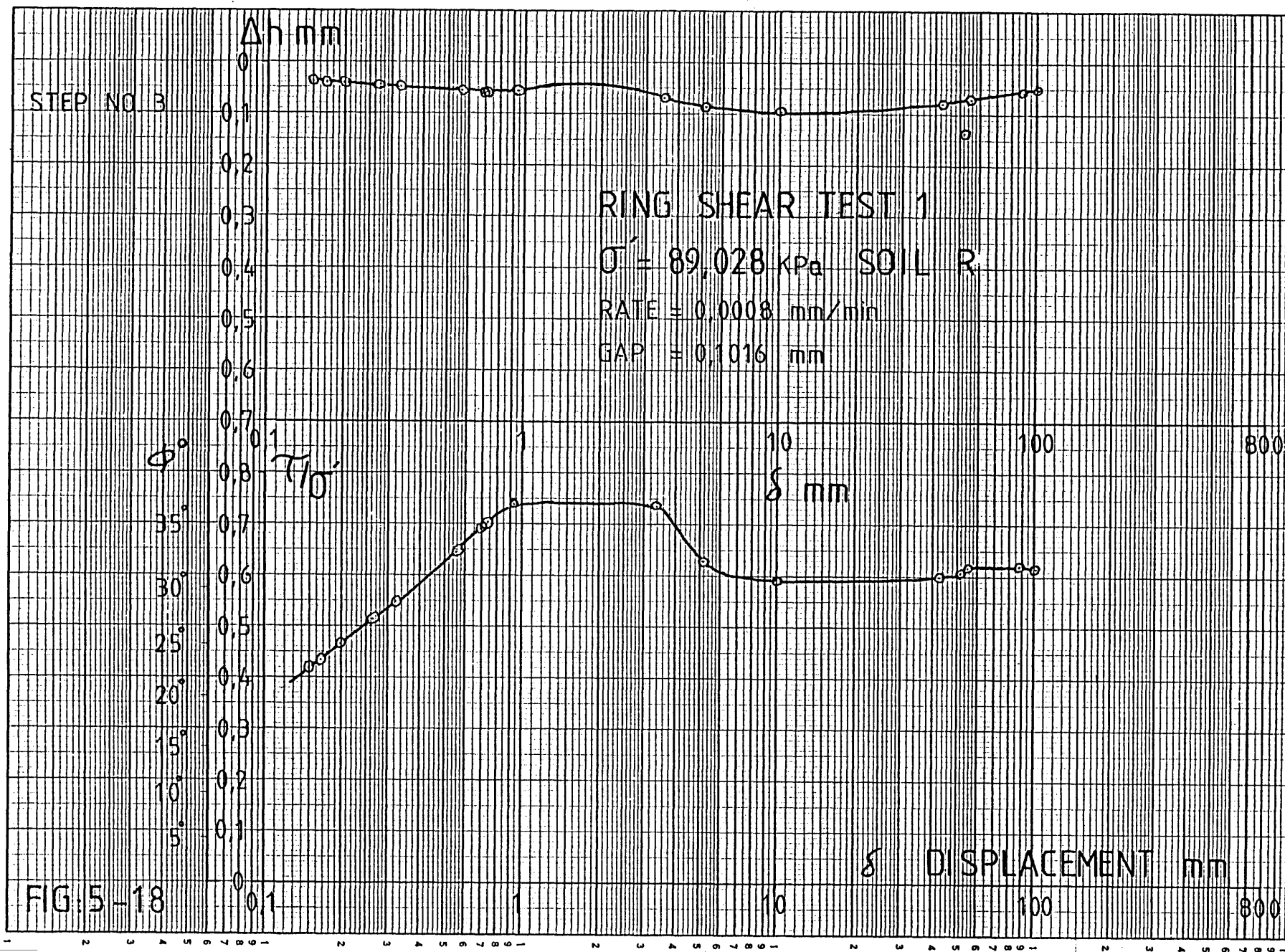


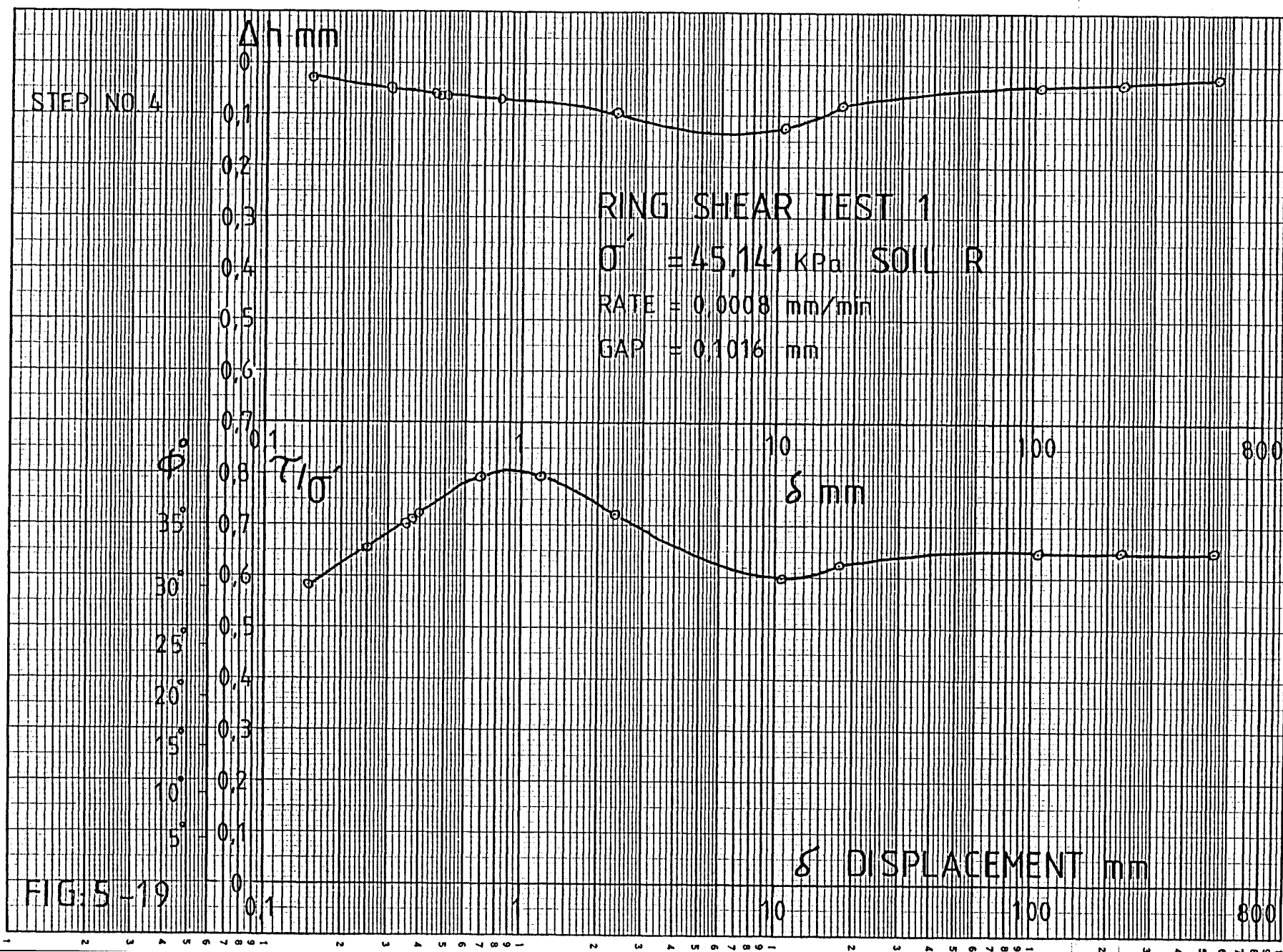


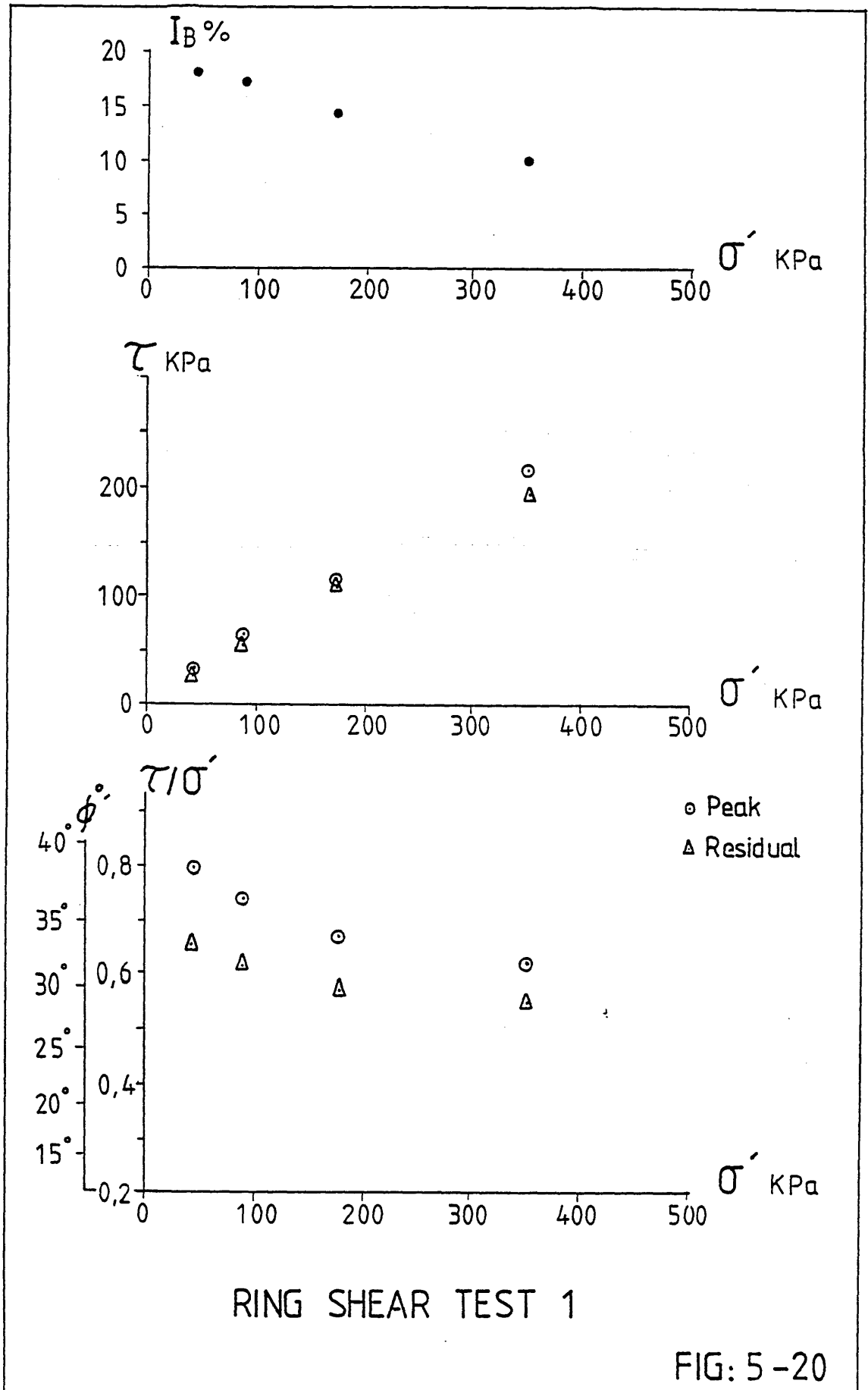


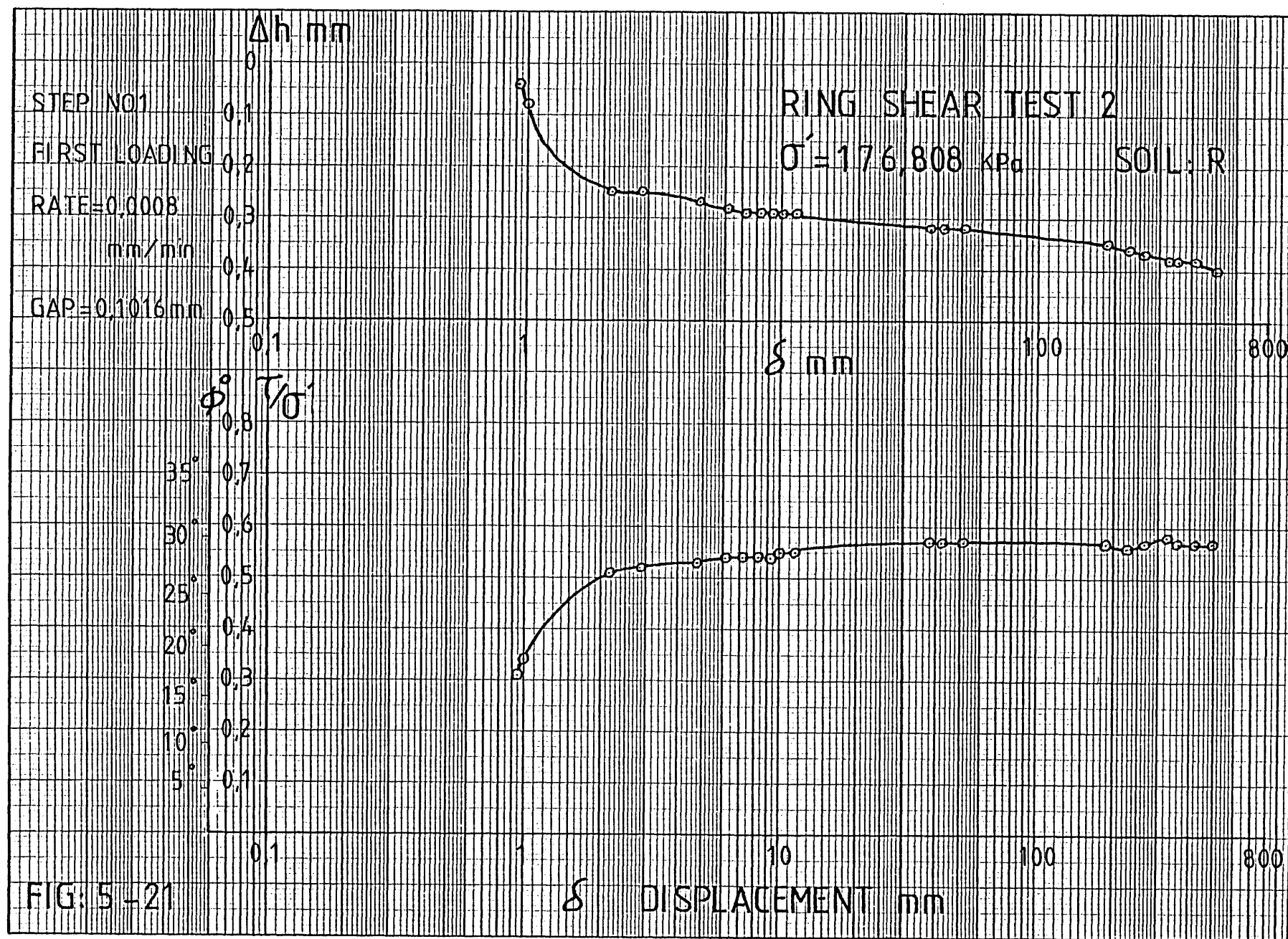


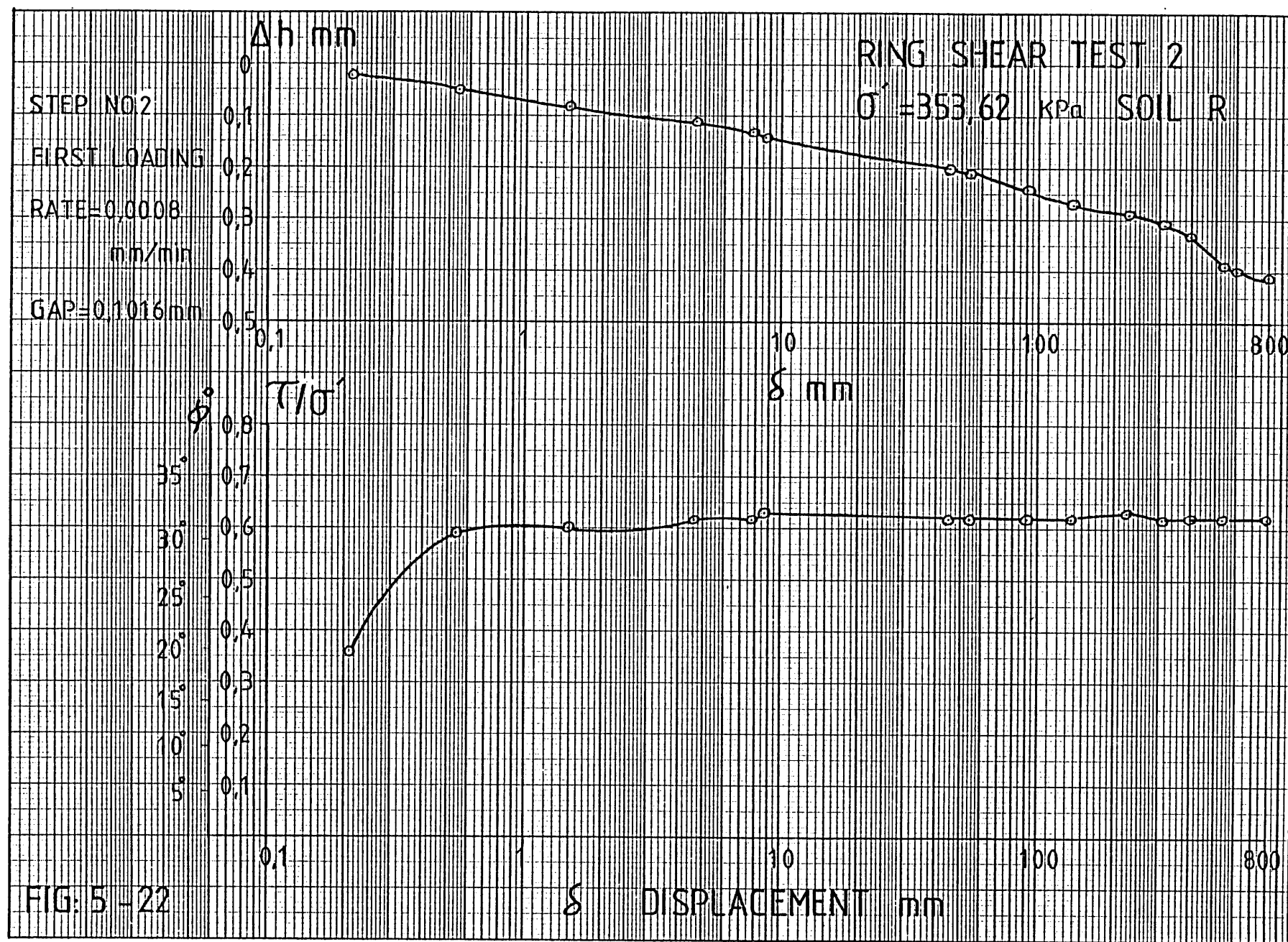


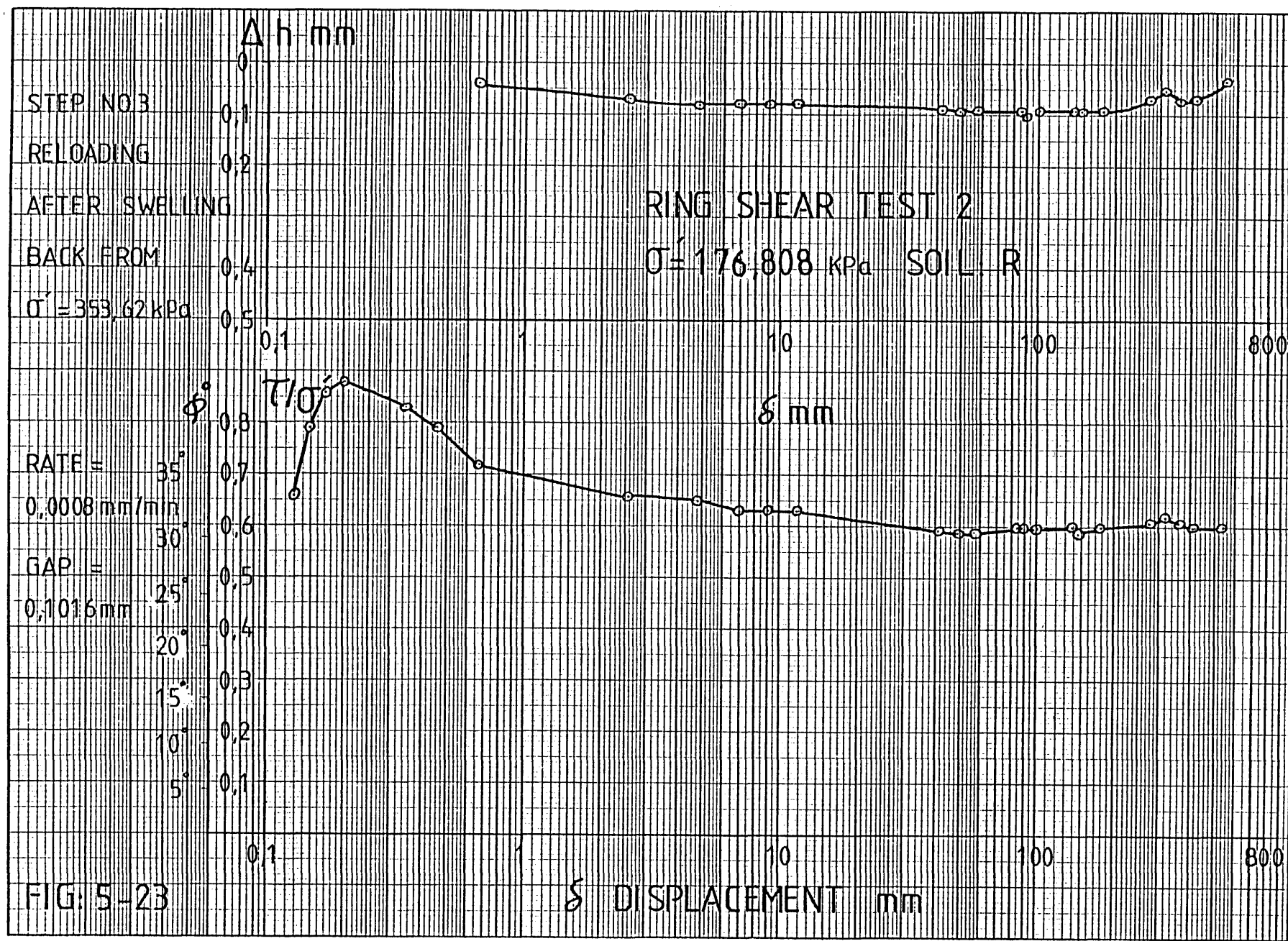


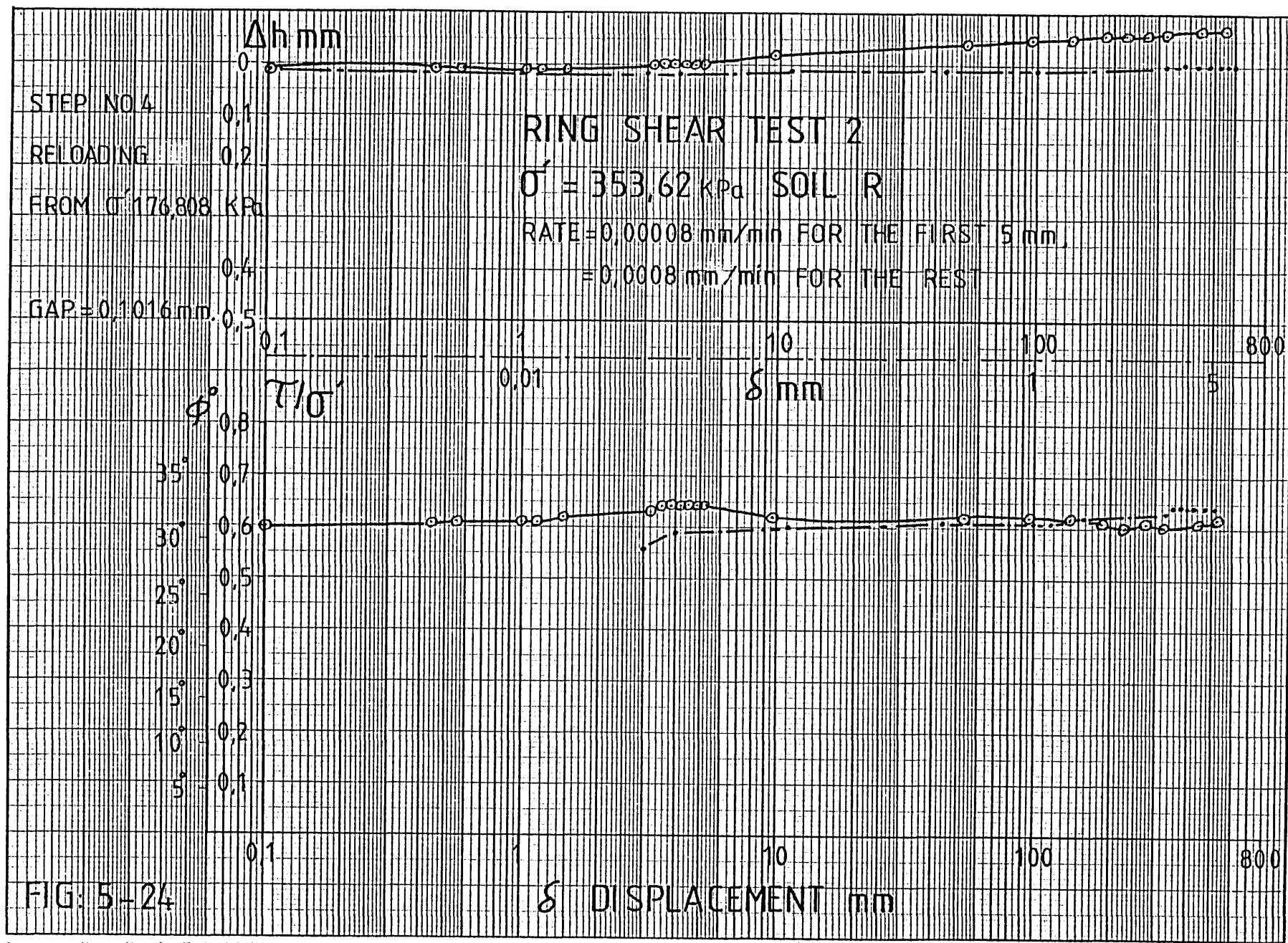


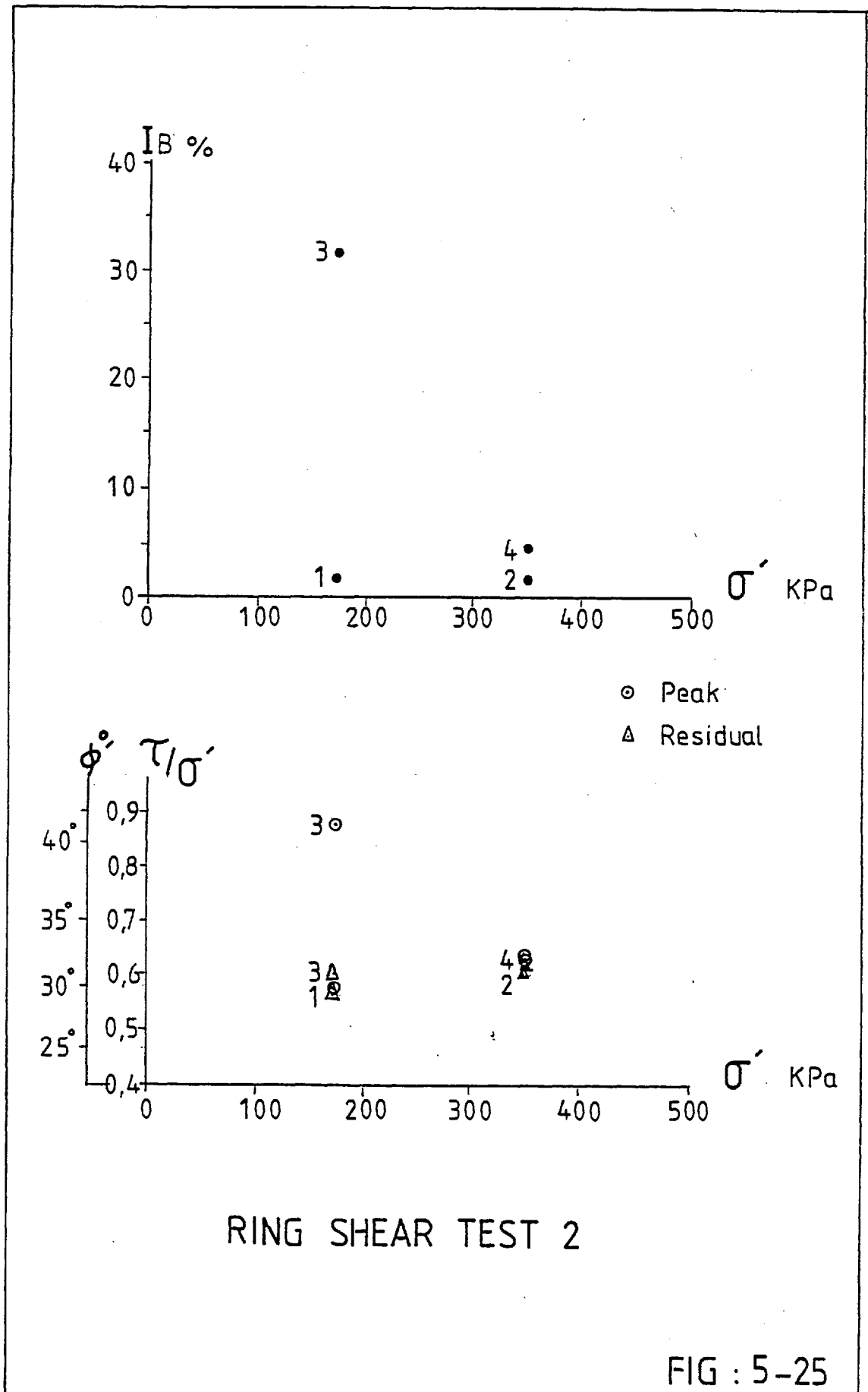












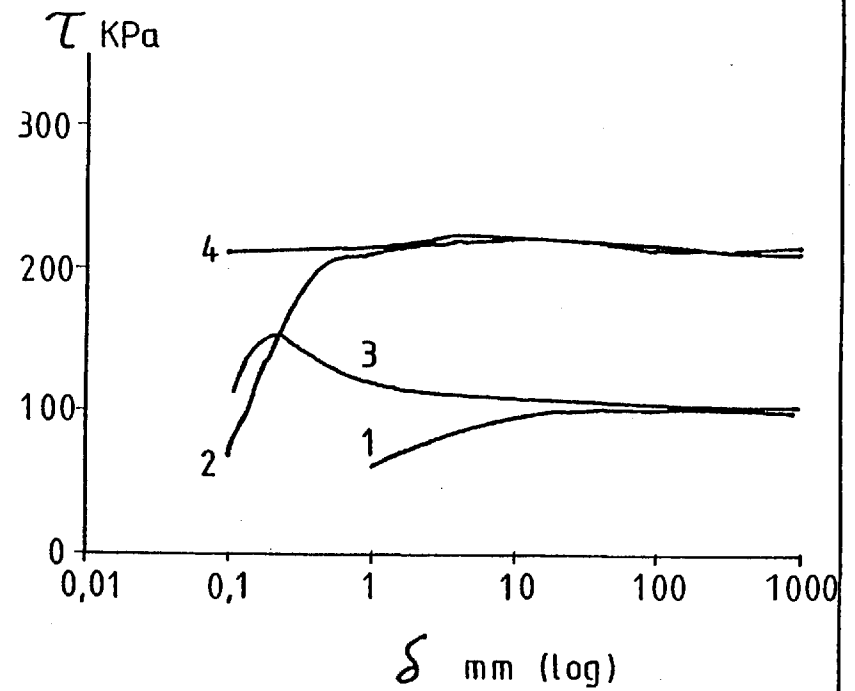
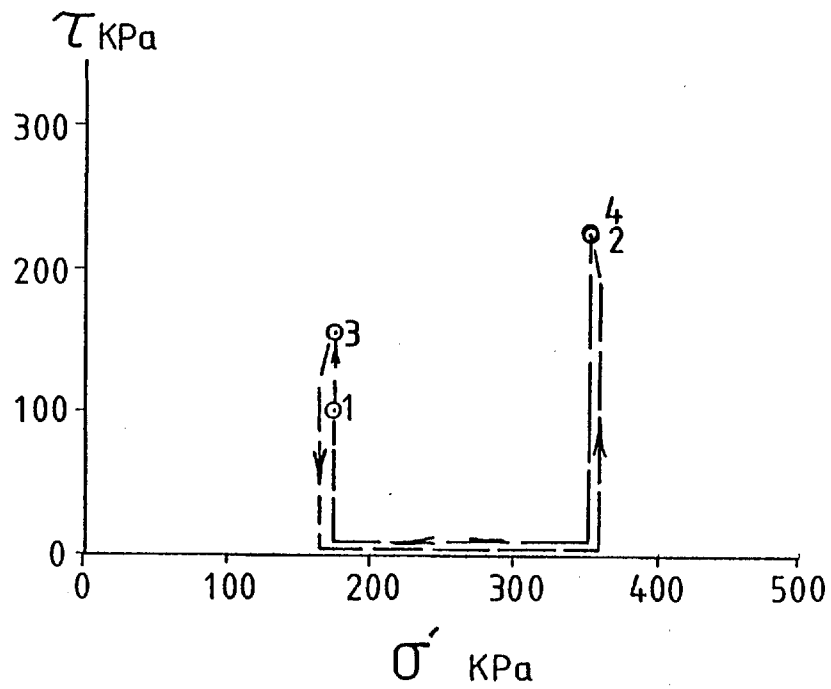


FIG: 5-26

RING SHEAR TEST 2

CHAPTER 6

DISCUSSION OF TEST RESULTS

6.1 INTRODUCTION

In this chapter the results of tests presented in the previous chapter (and Appendix 3) are analysed to investigate the various effects described in Chapter 3 (item 3-1).

Each section of this chapter deals with one of the aforementioned studies. The test results are presented in such a way as to facilitate understanding and interpretation and one of the prime aims is to minimise the effect of scatter of results.

6.2 SCATTER OF RESULTS

The research programme involved testing a natural soil. It was appreciated that there was a danger of introducing scatter into the results due to variability in the material. The variability of the Lower Till was therefore examined, firstly by measuring liquid and plastic limits on samples taken from each block; these results were presented in table (2-2) and they indicate very little variation in both plasticity and grading.

Scatter was found in the results on undisturbed samples, compare VIU: V4U* and T10, T12, T1 and T2.†

* Figs.(A3.58), (A3.61), (A3.63) & (A3.66).

† Figs.(A3.1) - (A3.4)

Assuming minimal variation in material type, this scatter must have arisen from natural differences in bonding, structure etc. or must have arisen as a result of sampling, handling and testing or both. Such differences should have been removed when material was remoulded. Limited confirmation of this point is provided by the results from the V group of tests (item 3-3 and Appendix 3). In this group differences in behaviour between the remoulded samples V1R: V4R^{*} are less than those between the undisturbed samples V1U: V4U. To eliminate as much as possible material variation when forming reconstituted samples, block V2 was used as the material source in this research.

It is rare to find two tests carried out under similar conditions which yield identical results especially when tested and measured with sensitivities such as the ones used in this research. Even if the end results of these identical tests are to reach the same numerical values their effective stress paths and deformation curves might not coincide.

Statistical methods of measuring scatter (e.g. standard deviation, mean deviation, variance) are impractical to be applied in this work because of the relatively small number of samples in each group of representative samples which decreases the significance of the tests to an unreasonable degree (Bishop 1978).

* Figs. (A3.59), (A3.60), (A3.62), (A3.64) & (A3.66)

It is worthwhile considering the magnitude of this scatter and the reasons for it. The scatter of results from apparently similar samples has been discussed by Bishop & Al-Dahir (1969), Wernek (1975), Al-Dahir (1967) and De-Mello (1977) and will be discussed here no further.

The largest scatter in test results might be expected from remoulded samples, since in each of the several steps taken during preparing remoulded samples (see item 4-3-1-1) the chances of exact repeatability are not great. The two methods used to bring about the required water contents in remoulded samples in this research (item 4-3-1-1) might have also contributed to the observed scatter. The first method is analogous to normal consolidation and the second method to overconsolidation. The manual way by which the soil was moulded also involved unknown energy (or compacting effort), the repeatability of which was difficult to achieve.

The scatter in the results of resedimented samples from slurry was expected to be the least. Indeed one of the main purposes of this method was to minimise scatter by forming as closely identical samples as possible. The procedure of preparing a resedimented sample from slurry involved many steps (see item 4-3-1-1) and the scatter observed in the results of tests might be attributed to the following:

- 1) Change in the initial water content of the slurry by slight variations in either the ratios of water to soil or in the water contents of dried soil after being taken from the oven to the atmosphere.
- 2) differences between stabilizing top weights and lengths of top cap (which in turn cause differences in weight)
- 3) differences in initial lengths of the samples mainly because of different depths of penetration of top caps.
- 4) item 3 will cause possible differences in initial cross-sectional area of the samples, these differences would progress as the tests proceed.
- 5) the suction pressure might have varied slightly from one test to another because although care was exercised to start all tests against the same suction, it was difficult to maintain exactly this suction throughout the first stage of consolidation because of the varying volume of air entrapped inside the samples and also because if initial densities of the samples were not the same the volumetric change during suction would vary.
- 6) differences in the magnitude and ratios of successive increments of loading in isotropic consolidation or in the rate of loading in anisotropic consolidation which cause notable discrepancies in the measured volumetric changes and cause consequent scatter (see item 6-9).

The above factors could, however, be controlled to a notable degree and the resulting scatter in the results

of the tests on resedimented samples from slurry can be seen to be minimal. To assess the possible effects of experimental factors on scatter, a brief literature review has been made.

Unfortunately no reference has been found on cohesive soils prepared in the way described in item 4-3.1.1. However information on granular soils has been presented by Reades (1972) on the effects of different methods of compacting a sample in the mould during preparation. He reported slightly different strengths for samples compacted during formation in various ways. Oda (1972) reported the effects of different compacting methods on the secant modulus, volumetric strains and rate of dilatency.

The possible differences in initial lengths and cross-sectional area mentioned in points 3 and 4 above might also have altered the proper height to diameter ratio of 2 recommended by Bishop & Green (1965). The various effects of end restraint on measured parameters have also been reported by Rowe & Barden (1964). Barden and McDermott (1965), Blight (1965) and Green (1969).

Daramola (1978) reported in detail on the expected bedding errors and he interpreted their effects during different types of tests. The bedding errors result from compressibilities of measuring equipment and from direct bedding between different elements of samples and apparatuses in contact. The former was accounted for in this research.

After finishing each test the rubber membrane was checked for leakage. However in long-term tests, as in most tests on resedimented samples from slurry, leakage of the cell fluid through the membrane itself or from around the rubber rings might have occurred. The leakages, if they took place during undrained shearing, would affect the water contents (and effective stresses) inside the samples and might also be responsible for some of the scatter, see Figs. (4-15) & (4.16). The effects of leakage through the membrane have been reported by Barden (1969) and Walker (1969). For drained tests. In undrained tests they were reported by Bjerrum et.al. (1958), Crawford (1959) and Whitman (1960).

From the above review it seems that the practical compromise is to find out the ranges between which the majority of the results would be. These ranges would have an upper and lower bound the difference between which would decrease as the quality of testing improves. To construct these ranges an adequate number of tests has to be done to establish various trends of behaviour (the long-term success of methods such as Casagrade's "Plasticity Chart", see Casagrande (1932), might be attributed to the practicability of using reasonable ranges instead of a single line or a curve). In cases where critical values are to be considered for design, the relevant boundary values should be taken instead of the usual mean value.

6.3 INFLUENCE OF SOIL STRUCTURE ON BEHAVIOUR

Undisturbed sands in nature are almost all stratified with slight anisotropy in compressibility (being less compressible in the direction perpendicular to the direction of bedding). Many undisturbed sands are weakly cemented. The relative density of some natural sands is greater than could be artificially obtained in the laboratory whereas other natural sands have very unstable structures to which only extremely loose laboratory samples can approximate (see Terzaghi and Peck 1967, Mitchell 1976 and Skinner 1975).

In natural clay soils, structure refers to both fabric and interparticle forces. The two extremes of fabric arrangements are the highly flocculated and the dispersed ones, with numerous combinations in between. The former arrangement generally has higher strength, higher permeability and lower compressibility than the latter. Two soils having similar fabrics might not behave identically because the forces between their particles and their groups of particles are not the same, i.e. they have different fabric stability (see Olson 1963). The fabric stability in clay soils is affected by chemical environment in the same way that the sand fabric is affected by the presence of cementing agents.

The structure of a clay reflects to a great extent its composition and history. Structure determining factors and processes are summarised by Mitchell (1976), Fig. (6.1).

The fabrics of clays are not uniquely related to their sedimentary environment but also to the electrochemical environment, rate of accumulation, turbulence during sedimentation and post depositional changes among other factors (see Yong and Warkentin 1975 and Lambe 1953 and 1958).

In boulder clays, such as the Lower Till from near Happisburgh, deposition took place with varying rates giving the opportunity for various fabrics to form (Collin and McGown 1974). The tills also have wide grain size distribution (see Figs. 2.7 and 2.8) and had been subjected to vertical and tangential stresses as a result of the motions of the ice sheets (see chapter 2). They are poorly sorted and contain large numbers of minerals (Pusch 1973 and Appendix 1 and 2). In boulder clays the flocculated structures described above are unlikely to develop. The natural structures of these clays probably depend on the relative proportion of sands and clays present (Vaughan et al. 1978) in addition to their depositional history.

When the natural sands and clays are remoulded their structure is altered. In clays with an initially flocculated structure, remoulding will break down this structure and this is reflected in the change of shape of $W-\sigma_v'$ curves (Wu 1966 and Capper and Cassie 1969). Remoulding will destroy any shear planes and reduce void ratio and permeability in these clays (see Lambe 1962).

It will also produce more dispersed fabric; owing to the randomness of remoulding, dispersed structures may exist only in localised pockets and within each pocket a parallel orientation might exist.

One of the most important consequences of remoulding plastic clays is the loss of undrained strength of undisturbed soil after being disturbed. Sensitivity which is defined as the ratio of undrained strength of a soil before and after disturbance has long been recognised and its causes were studied over many years. It is generally considered that remoulding sandy clays (boulder clays) does not affect their ultimate shearing strength. However, remoulding may affect their deformation and pore pressure curves during shearing.

Compaction as a method of soil improvement involves some remoulding and causes similar changes to the undisturbed soils as mentioned above for complete remoulding (see Lambe 1958 and 1962, Soil Mechanics for Road Engineers 1952 and Lambe and Whitman 1969)

Pile driving also involves remoulding and extensive field measurements on piles have shown that the strength of clay was reduced to a value approximating to that of the remoulded strength immediately after driving (Cummings et al. 1950; Seed and Reese 1955; Skempton 1959 and Wu 1966).

Isotropic and anisotropic consolidation for remoulded soils produce different fabrics, generally an isotropic fabric for isotropic consolidation and an orientated fabric for anisotropic consolidation (with clay plates aligned with their long axes in the direction of the major principal stress, Tchalenko 1967). This will be briefly dealt with later.

The drop from peak to residual strengths in drained tests on undisturbed soils (Taylor 1948, Crawford 1959 and Skempton 1964) has been attributed to the remoulding that takes place at large strains. Discussion of the results of ring shear tests on the Lower Till are presented in item 6.3.3.

(The drop of strength from peak to residual and its relationships to the plasticity of a soil, progressive failure and pattern of failure is discussed in Bishop 1967 and 1971, Vaughan 1971 and Vaughan and Walbancke 1975. The latter relationship is also discussed in Skempton and La Rochelle 1965 and Sandroni 1977.)

The above review emphasises the differences between undisturbed and remoulded soils which are important in geotechnical engineering. These differences arise from changes in structure and it is, therefore, essential to establish the importance of these structural differences for the soil used in this research. To study these effects the results of tests on undisturbed (U), remoulded (R) and normally and overconsolidated samples from slurry (S) had to be presented in a comparable form and it was decided to

normalise all the results of isotropically-consolidated tests against their effective mean consolidation pressure at the start of shearing (P'_0) since a wide spectrum of P'_0 was available (see Appendix 3 and item 6.2 also Ladd and Foott 1974 and Ladd et al. 1977). It was also decided not to include tests with P'_0 less than 20 kPa in order to avoid the instabilities of behaviour that take place at these small stresses. The normalised curves best represent the majority of the tests carried out, whose water content lie within the medium range (14%-16%) of the water content spectrum used in this research. Tests on different types of soils (U, R and S) sheared at closely similar rates were grouped together for proper comparison. The majority of tests were sheared at a rate of 0.005 mm/min (Ra) (see item 3.2.6). As discussed previously truly "undisturbed" material could not be used extensively.

It is appreciated that using consolidated samples from slurry may not resemble exactly the in-situ conditions for soils of analogous history mainly because of the lack of time effects in slurry samples and the differences in homogeneity but it was the best available compromise under the present circumstances. Thus when the terms normally consolidated and overconsolidated samples are used in this text it will be referring to samples resedimented from slurry.

6.3.1 Comparison of Unconsolidated and Isotropically-Consolidated Samples

i) Undisturbed samples (U): results of four tests on samples from the Lower Till and one from the North Sea bed are presented in Figs.(6.2) and (6.3). Keeping the reservations on these tests in mind (see items 4.1 and 6.2) it can be seen that the samples of Lower Till (except T10) show reasonably similar effective stress paths (ESP's). The ESP rises nearly vertically because of the small (mainly positive) generation of pore water pressure; it then touches the failure envelope and climbs up it. The distance it climbs may be influenced by the choice of shearing rate (see Vaughan et al. 1976 and Sandroni 1977).

ii) Remoulded samples (R): results of ten tests ($R_a = 0.005$: 0.0076 mm/min) are shown in Fig. (6.4) and they show a clear trend of behaviour. The dotted lines are the average lines representing the trend.

iii) Normally-consolidated samples (IN): results of seven tests (R_a) are presented in Fig. (6.5) and the dotted lines indicate the clear average trends of behaviour.

From the above three groups of test results the following observations are made,

1) normally consolidated soils generate larger pore pressures than the remoulded ones during undrained shearing. The relative values of the pore pressures coefficient A at failure (A_f) are shown in Fig. (6.31). It can be seen that the difference in A_f decreases with the increase in P'_0 . The undisturbed samples generate lesser

pore pressure than remoulded ones. Crawford (1959) suggested that when remoulding an undisturbed sample at constant water content the value of pore pressure is related to the degree of remoulding.

2) if the undisturbed, remoulded and normally consolidated samples were all compared starting from the same P'_0 as above, the undisturbed samples would have the highest value of undrained strength and the normally consolidated ones the lowest, the remoulded values being in between the two, see Fig.(6.17).

3) the normally consolidated soils are slightly stiffer than remoulded ones (the deformation curves of undisturbed samples give no definite trend) and they reach their peak strength at smaller axial strains.

4) For all types of soils tested, the effective stress parameters (failure envelopes) were almost the same.

In order to investigate whether these observed patterns persist at other shearing rates, the results of tests carried out with rates Rb, Rc and Rd are presented in Figs. (6.6) and (6.9). It should be repeated here that in rates such as Rb and Rc, the values of the measured pore pressures are not reliable (see item 4.3.2.1.2). The ESP's however were drawn from the obtained measurements. In tests carried out with Rd, no pore pressures were measured and only deformation curves were obtained, Fig. (6.9) .

In Fig. (6.6) for U, R and IN samples sheared with R_a the trends of behaviour agree well with the ones for tests with R_a . They give a clear and comprehensive picture of the relative behaviours of different types of sample. Tests sheared with R_c (0.50 mm/min, two orders of magnitude faster than R_a) are shown in Figs. (6.7) and (6.8) for R and IN soils respectively (no U samples were sheared either with R_c or $R_d = 7.62$ mm/min). If in Fig. (6.8) the trends for R samples are superimposed on test results for IN and it shows that the relative behaviour of R and IN samples is generally as stated above regarding peak strengths, pore pressure generations and stiffnesses. However it is clear that the initial measurements of excess pore pressure generation, especially for IN tests are unreliable and the ESP's for tests sheared with rate R_c (or faster) should be disregarded as informative because though the later measurements of pore pressures might be correct and consequently the shape and positioning of the ESP's, the initial parts of any ESP could still be in doubt.

In Fig. (6.9), the deformation curves of R and IN samples tested with R_d are shown. Again the R samples gave higher peak strengths and lesser initial stiffness.

It is shown that, by comparing IN and U samples that the latter exhibited a degree of overconsolidation and that the remoulding resulted in a quasi-overconsolidation effect which is of a degree lesser than that of the U samples. In order to attempt to assess the degree of

overconsolidation of U samples the results of tests on slurry samples with different degrees of overconsolidation (sheared with two rates) are presented in Figs (6.10) and (6.16).

Figs (6.17) and (6.18) summarise the above findings for the isotropically-consolidated samples. They are presented schematically to relate different spaces together. In Fig. (6.17) the relative positions of ESP's, deformation and pore pressure curves of IN, IO, R and U are shown. The relevant $W\%-C_u$ and $W\%-\sigma_v'$ relationships are also presented. In the studied soil the structural disturbance will cause the changes discussed before if the samples start undrained shearing from the same effective consolidation pressure. The overconsolidation ratio of the in-situ Lower Till was calculated by the author before from the geological research to be between 1.84:4.22 (see item 2.3.1). In Fig. (6.17) the ESP of the U samples lie between the ESP's of soils of OCR2 to OCR4 which agrees well with the speculation derived from geology.

The undrained shearing strength of this soil depends a great deal on the water content and $W\%-C_u$ relationships are presented for different rates of shearing in item (6.4). The significance of the water content as a parameter made the comparison between U, R and IN soils sheared at the same water content an important item and this is presented in Fig. (6.18). The relative positions of ESP's and deformation and pore pressure curves are

presented together with the $W\%-C_u$ and $W\%-\sigma_v'$ relationships. The $W\%-C_u$ and $W\%-\sigma_v'$ relationships of Figs. (6.18) and (6.19) should be compared with Figs. (5.15) and (6.21).

It is interesting to note that undisturbed and remoulded samples when sheared with the same water content give similar strengths (see Vaughan et al. 1976) and the pore pressure changes seem to compensate for the initial difference in P'_0 . At the same water content the IN sample has a higher P'_0 because of the relative positions in the $W\%-\sigma_v'$ domain and though the values of C_u/P'_0 differ for IN and R samples (see Fig. 6.17), this difference will not compensate for the large difference in P'_0 and IN samples will inevitably yield higher strengths.

An attempt to assess the OCR of remoulded soils has been tried by the author and by Gens (1978), these attempts proved, however, to be unsatisfactory.

6.3.2 Comparison between Undisturbed and Remoulded Samples in Drained Shearing

In the V group of tests, undisturbed samples were taken from four blocks, V1-V4 and sheared in a drained test; then the same sample was remoulded and tested in a consolidated drained test. Drained shearing was done in a stress controlled way using Wissa's (1971) approach for anisotropic loading (see item 4.3.1.2.1) and checked by the method used by Bishop and Henkel (1962) for drained shearing.

During shearing σ_1' was kept "constant" and σ_3' decreased until failure (SP2, item 3.2.5). This stress path was chosen because the samples were dry, ($W_0 = 12:13\%$, PL av. = 13.43%), and considerable time could be saved by adopting it. It has also given an insight into the effect of stress path as will be discussed in section 6.9. An additional test V1R' was carried out by increasing σ_1' and keeping σ_3' constant (SP1, item 3.2.5). Details of all nine tests are shown in Figs. (A3-58) : (A3.66).

Ideally all tests should have started from the same P'_0 and W_0 . In undisturbed soils P'_0 depended on the pore water tension retained within the samples and three of the four undisturbed samples studied did have closely similar values of P'_0 and W_0 (V2U, V3U and V4U). Test V1U, however, had a higher water content than the above three samples, yet, it also retained higher P'_0 which is contrary to the normal behaviour of this material. (Early tests on undisturbed samples in group G also showed no pattern between P'_0 and W_0 .) Each remoulded sample had an initial water content closely similar to the final water content reached by its original undisturbed sample and was consolidated back to P'_0 "equal" that of its original undisturbed sample. The volumetric change during shearing of an undisturbed sample was larger than that during consolidation of the remoulded sample; this led to a difference in W_0 between a sample in its undisturbed and remoulded state. From the tests it can be concluded that

1) Undisturbed samples show brittle behaviour and remoulded samples show non-brittle behaviour. This could be assumed to be the result of some sort of bonding in the undisturbed soil. Whether this bonding exists in the field or is a consequence of the sampling and storage process is difficult to verify without further testing of sufficient numbers of undisturbed samples as well as carrying out "in-situ" tests.

2) Undisturbed samples failed along a failure plane and the remoulded samples failed by bulging in spite of using the same shearing rate in both cases.

The axial strains at failure for all undisturbed samples were much less than those for remoulded samples.

3) Each sample showed higher effective stress ratio at failure, (σ_1'/σ_3') flr., in its undisturbed state than in its remoulded state (in this point V4U was excluded because its failure took place quickly which did not guarantee full drainage). It has to be noted here that detecting where "failure" took place in these tests is aided by the diagram showing the values of both σ_1' and σ_3' during actual shearing. It is most probable that when σ_1' parts from its almost horizontal configuration then failure is imminent. That was reflected experimentally in the lack of control in keeping σ_1' constant and keeping the same shearing rate as in early stages of the test.

It is noticed here that the values of (σ_1'/σ_3') flr. for all tests sheared by reducing σ_3' and keeping σ_1' constant are higher than the values of the similar parameters

obtained from tests (drained as well as undrained) sheared by keeping σ_3' constant and increasing σ_1' , (see section 6.9). High values of (σ_1'/σ_3') flr. obtained when shearing samples after being unloaded have also been reported in item 6.6.

Henkel (1960) reported results of drained and undrained tests on remoulded Weald and London clays. These tests had different stress paths, some of which resemble SP1 (σ_1 increasing and σ_3 constant) and SP2 (σ_1 constant and σ_3 decreasing) and they all shared the same failure envelope contrary to the findings of this study. Henkel (1960) also quoted results from three papers by Rendulic (1936 a and b) and (1937) in which tests on Wiener Tegel shared the same failure envelope when sheared following SP1 or SP2. The water contents in the tests by Henkel and Rendulic (loc. cit.) have been changing in the drained tests and different tests reached the common failure envelope at different water contents. The envelopes for over-consolidated samples were different from those of the normally-consolidated samples for the above tests. In the Lower Till, however, it will be mentioned in item 6.7 (stress history) that isotropically overconsolidated samples share the same failure envelope as normally-consolidated samples whereas anisotropically overconsolidated samples of high OCR (10) have a failure envelope which lies above the normal envelope. In shearing following SP1, both drained and undrained anisotropically normally-consolidated samples of the Lower Till reached the same envelope and shared almost the same contours of water content, item 6.9 .

Treating the phenomenon of high (σ_1'/σ_3') at failure when SP2 is used, resulting from unloading or a quasi-overconsolidation phenomena, is a rather facile solution. However an attempt was tried to use models relating the shear strength parameters, consolidation pressures and water contents to try to explain the observed high values of (σ_1'/σ_3') assuming that these values reflect the values of ϕ' only, such as the models developed by Hvorslev (1937) and explained in his (1960) paper. There are, however, some factors that preclude the use of such models in the present investigation. Hvorslev (1960) stated (p.202) that his developed expressions cannot be applied to clays remoulded at water contents much drier than their L.L. or to undisturbed clays such as the ones in group V. Bjerrum (1954) and Bishop and Henkel (1972) did not recommend the use of these methods for low plasticity soils such as the lower till. The only method by which the model could have been used is one of the methods presented by Bjerrum (1954) (method C, p.50). Unfortunately this method was recommended for remoulded soils on the condition that there would be large differences in the values of their initial moulding water contents which is not the case in remoulded tests of group V. Hight (1978) will be enlarging on this phenomenon as will be mentioned in item 6.6 (Sampling).

4) All undisturbed samples dilated during shearing. The magnitude of their volumetric strain at failure, $v_{flr.}\%$, ranged from 1.0 : 2.5. V2R and V4R compressed during shearing while test V3R dilated during shearing.

5) The value of the "Compression Index" C_c is approximately the same for all remoulded samples during consolidation ($0.075 =$ the change in void ratio per each logarithmic cycle of stresses). No variability is observed between the blocks as regards this point.

6.3.3 Comparison of Anisotropically-Consolidated Samples

The results of test ARII, Fig. (5.9) are plotted together with the results of test ANP320, Fig. (A3.102) in Fig. (6.19). The two tests show that there is close agreement of behaviour between the anisotropically consolidated remoulded and slurry samples. Gens (1978) has results confirming the above. It is also shown that for anisotropically consolidated samples in the triaxial apparatus the $W\%-\sigma_v'$ relationships for remoulded samples merge with the normal consolidation line at pressures much lower than in the case of isotropically-consolidated samples, compare Figs. (5.8) and (5.15). This fact, no doubt, relates to the anisotropic fabric which the remoulded soil assumes during consolidation as opposed to the more unstable isotropic fabric which builds up during isotropic consolidation (see Skinner 1978, Oda 1972 and Mitchell 1976).

The above finding is of importance to further work with this soil especially in the case of the model testing (see Herbertson 1977 and Martins 1977). It will be more economical to use remoulded soils instead of resedimented soil from slurry in terms of time and equipment provided that pressures high enough to ensure this coincidence between remoulded and slurry behaviour is obtained.

6.3.4 Ring Shear Tests

As described in item 5.3 two tests on the ring shear were carried out on the Lower Till as explained. In RSI the deformations were greater at first loading and decreased afterwards and some dilatation was noticed at large displacements, Figs. (5.16) to (5.19). In RS2 the deformations increased slightly at the first two stages, Figs. (5.21) and (5.22), and then the deformations were minimal and pronounced dilatation took place at the last stage of this special test.

It is very important to study the petrological thin section of Fig. (4.17) taken at the end of test RS2. In this test, R-sample was anisotropically consolidated and a special cycle of loading, unloading and reloading were carried out and after each loading (or unloading) stage shearing took place to large strains. No orientation whatsoever was noticed nor any preferred plane of shearing though the sample was forced to shear along the shown plane at the middle height of the sample and to these large strains. This indicates that this soil has no tendency for orientation even under these circumstances

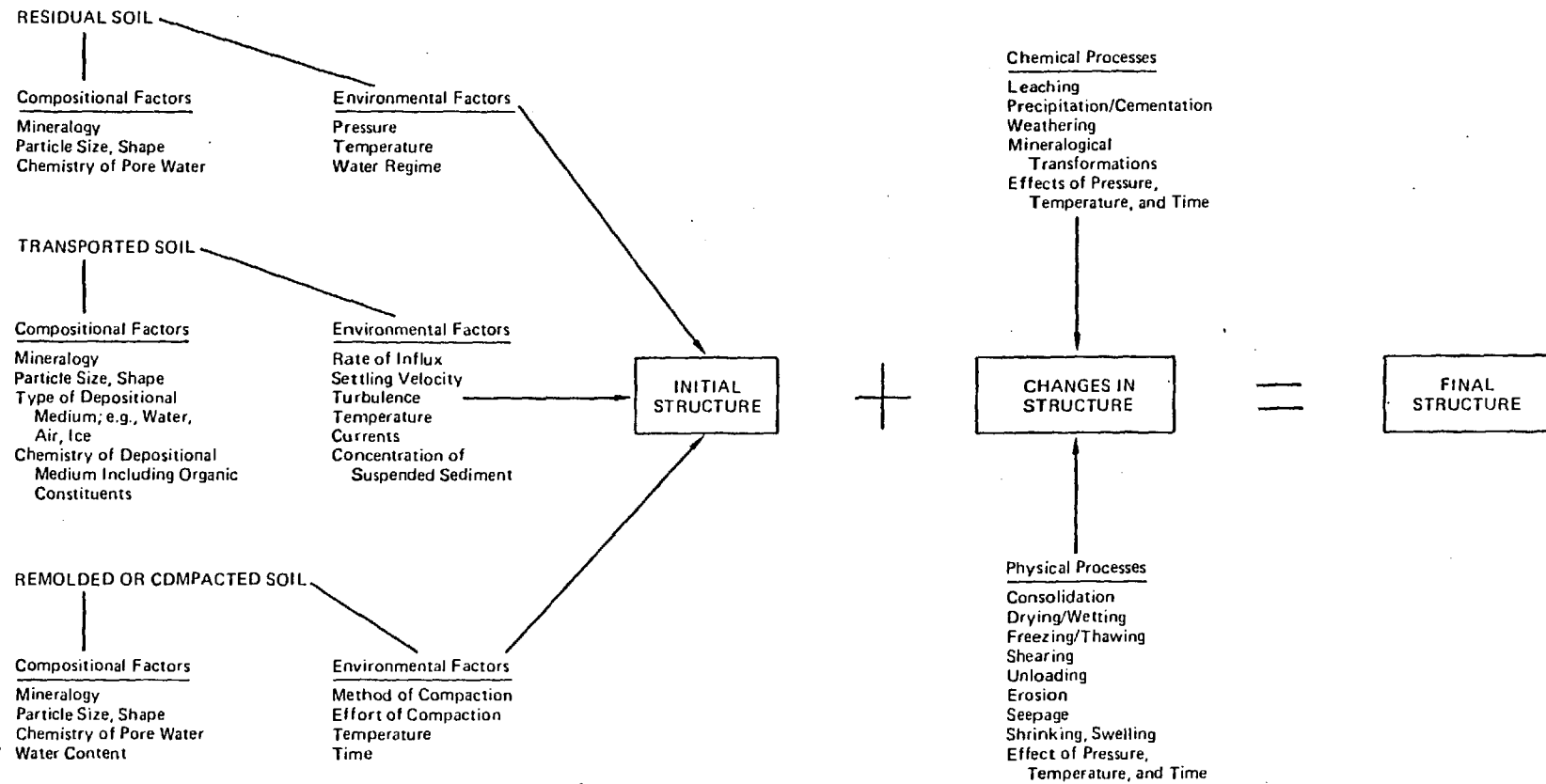
and that the granular behaviour seems to prevail during shearing.

From Fig. (5.20) it is noticed that the brittleness index (I_B) decreased with increasing normal pressure which is also clear from the decrease in the difference between ϕ' peak and ϕ' residual as the normal stress increases.

In Figs. (5.25) and (5.26) the results of RS2 are shown from which the following conclusions could be drawn:-

- i) I_B did not practically change in the first two stages though it showed a minute decrease in stage 2,
- ii) the sample at stage 3 behaved as an overconsolidated sample as far as brittleness was concerned. The residual strength however recorded a value not far from those of stages 1 and 2 regardless of the stress cycle,
- iii) stage 4 being a loading (or rather a reloading) stage, the soil behaved as in first loading, stage 2 and gave closely similar results.

Thus prediction of behaviour of the Lower Till is made easier by realising that the soil has a full memory of deformations and effective stress parameters in reloading and of residual strengths in both unloading and reloading.



Structure-determining factors and processes.

FIG. 6-1 (After Mitchell 1976)

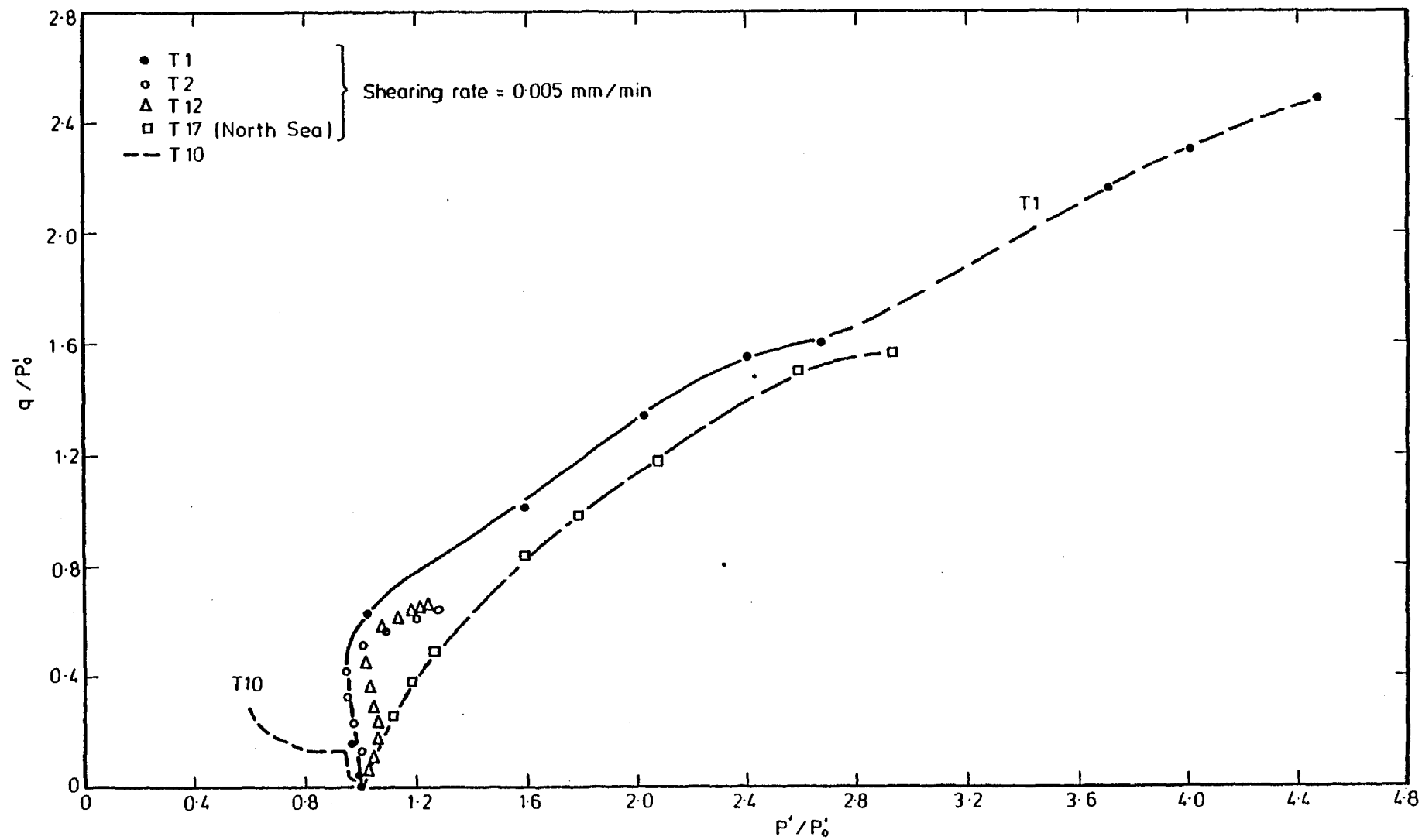


Fig. 6-2 Normalized effective stress paths of undisturbed samples of Happisburgh Lower Till and the North Sea bed, (CU_x tests)

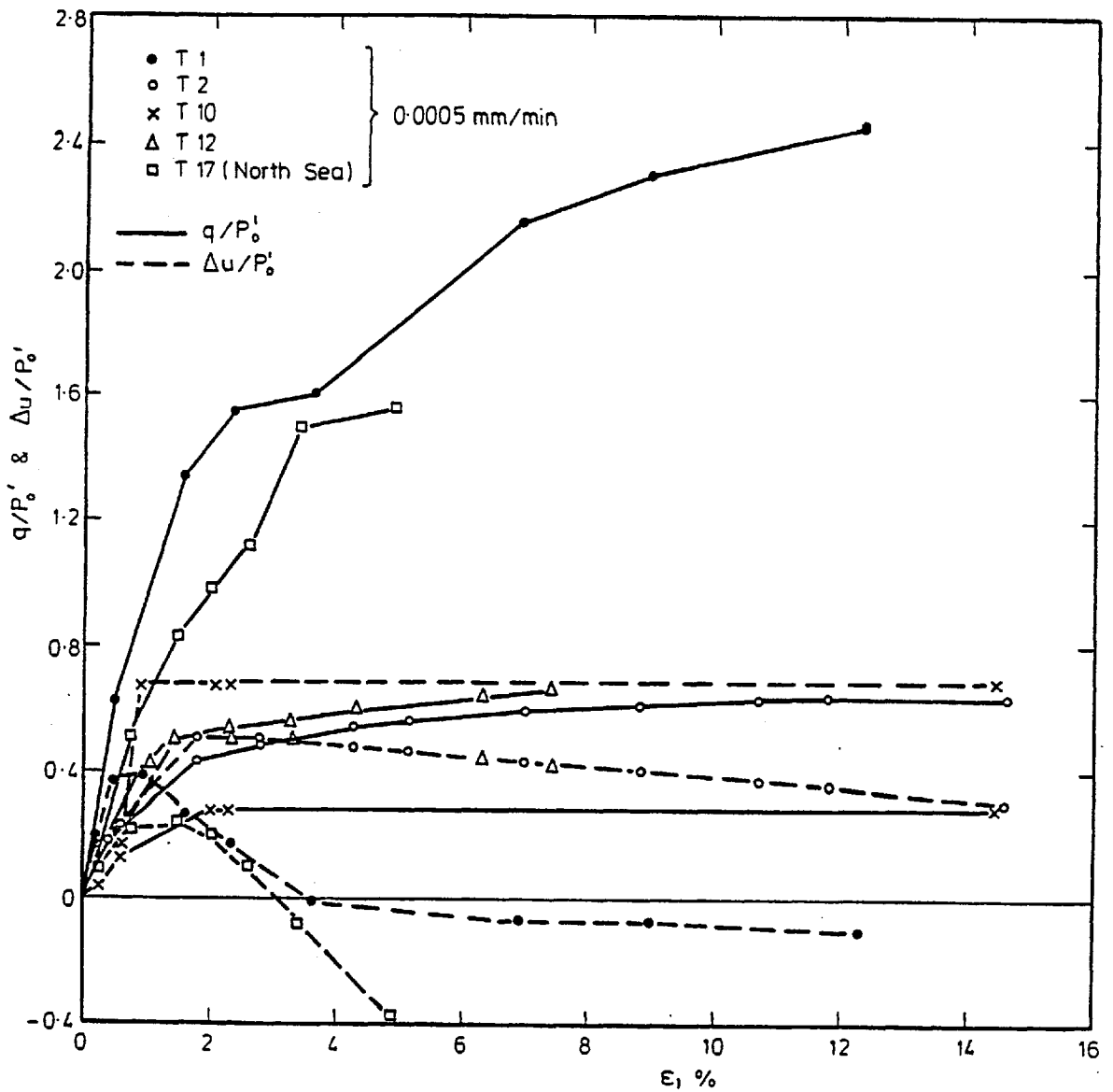


Fig.6-3 Normalized deformation and pore pressure curves of Happisburgh Lower Till and the North Sea bed.
(Undisturbed samples UU tests)

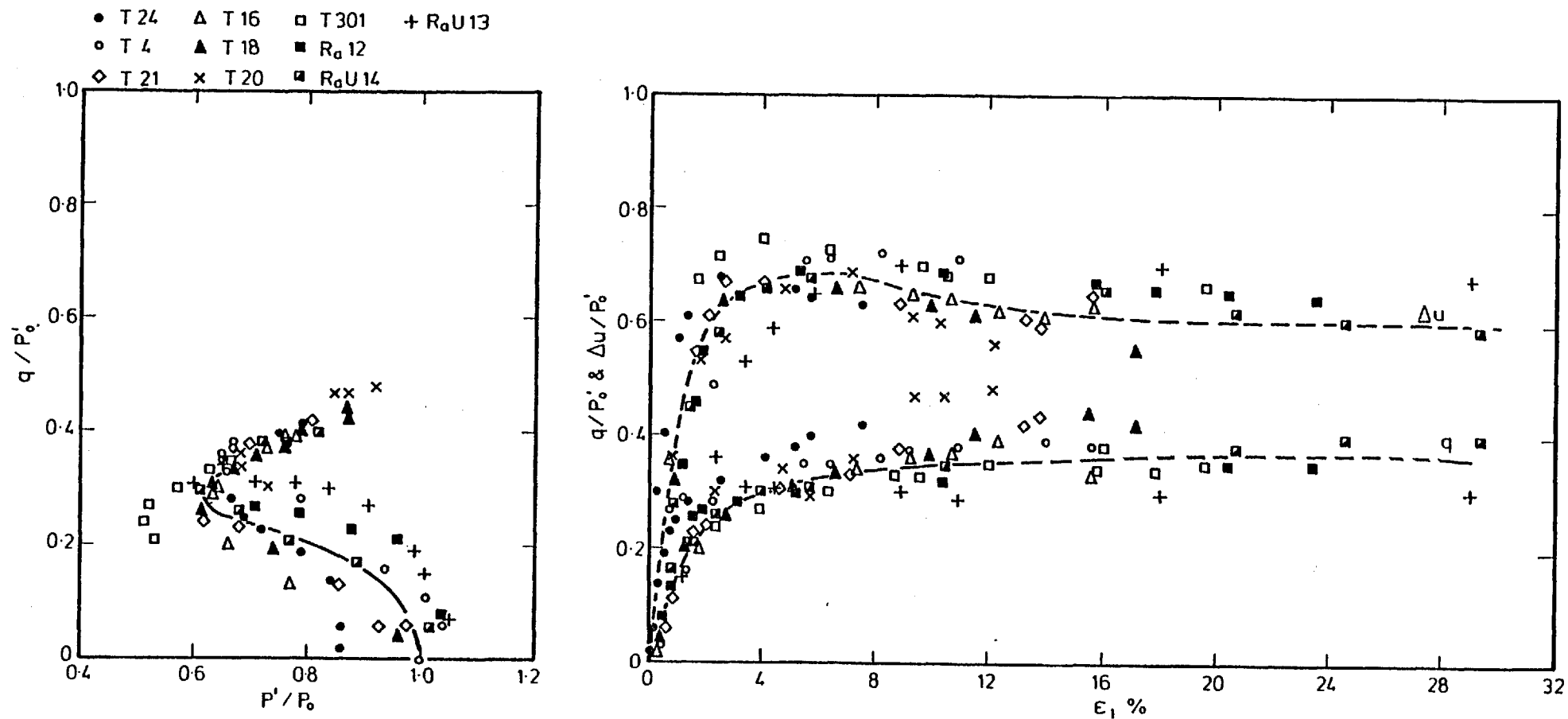


Fig. 6-4 Normalized results of tests on remoulded samples of Happisburgh Lower Till (CU tests - shearing rate 0.005 mm/min)

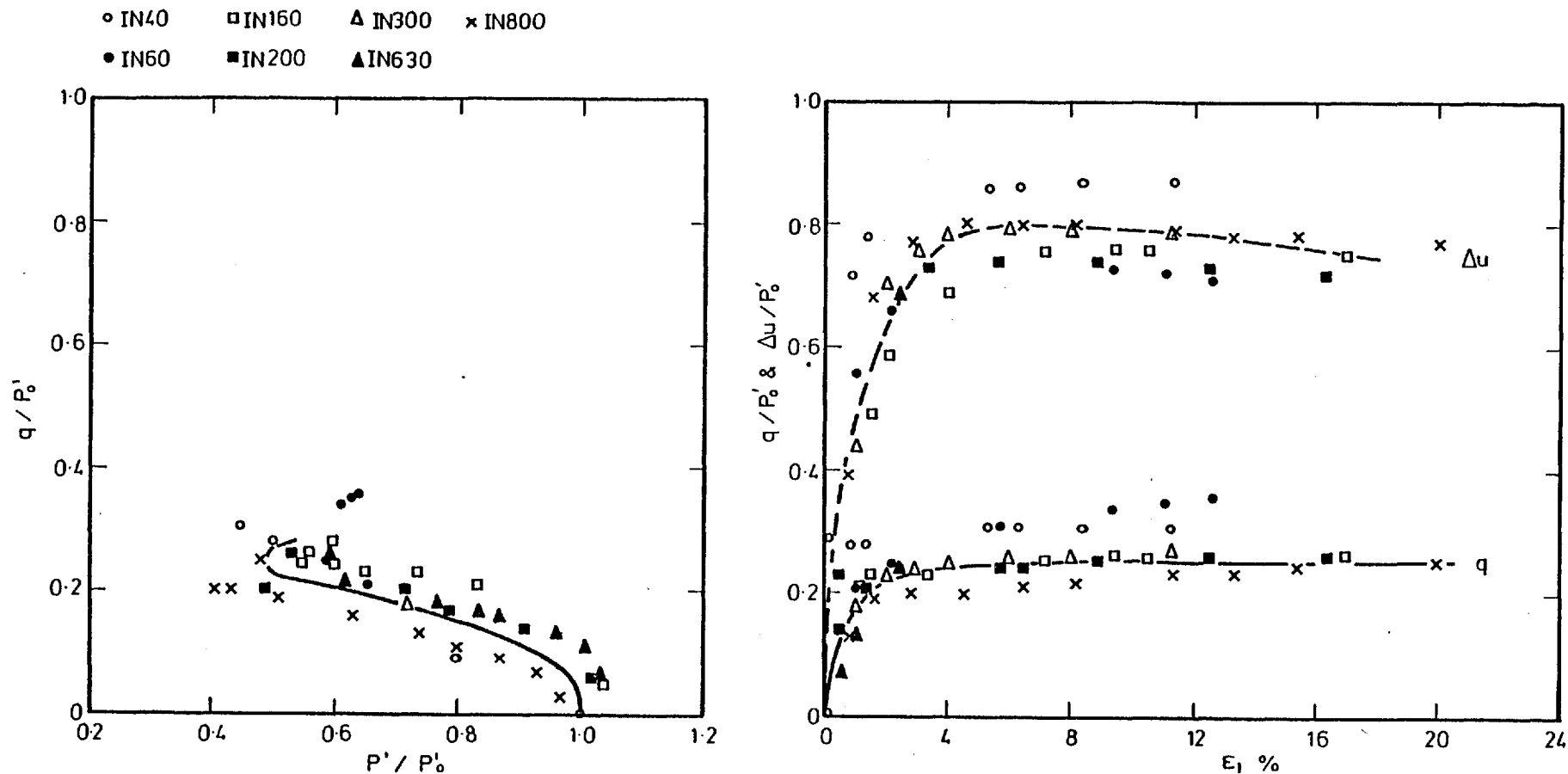


Fig. 6-5 Normalized results of tests on resedimented samples from slurry of Happingburgh Lower Till (CU tests-shearing rate 0.005 mm/min)

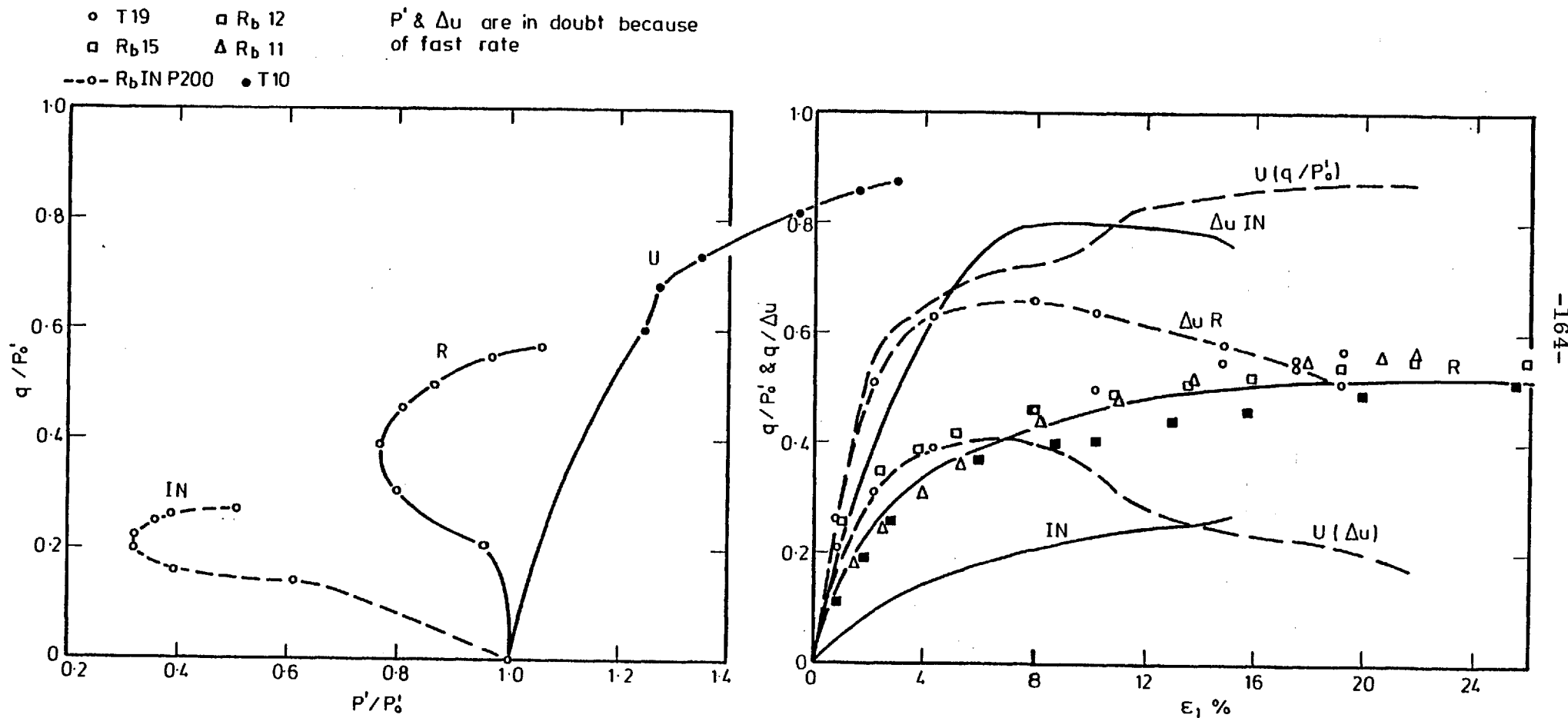


Fig. 6-6 Normalized results of tests on remoulded and resedimented samples from slurry of the Happisburgh Lower Till (CU tests - shearing rate 0.05 mm/min)

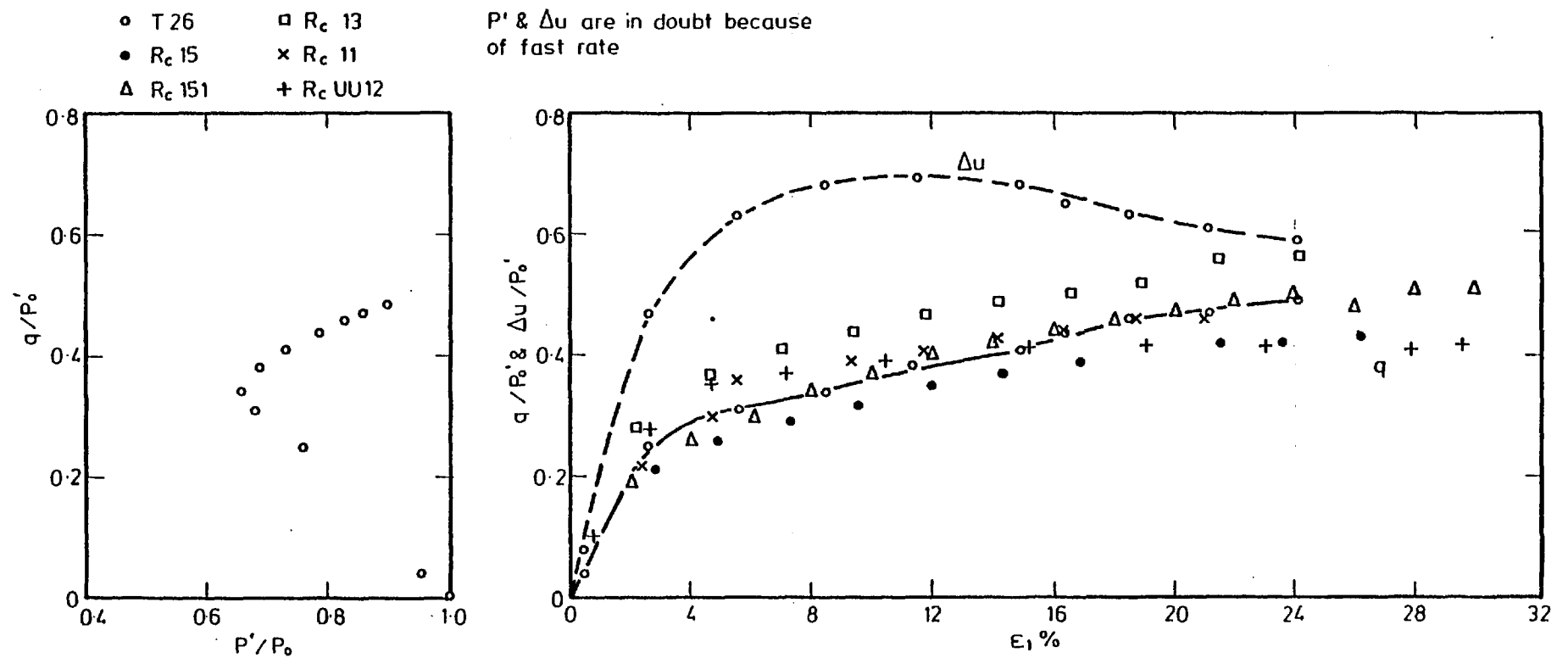


Fig.6-7 Normalized results of tests on remoulded samples of Habbisburgh Lower Till
 (CU tests-shearing rate 0.5 mm/min)

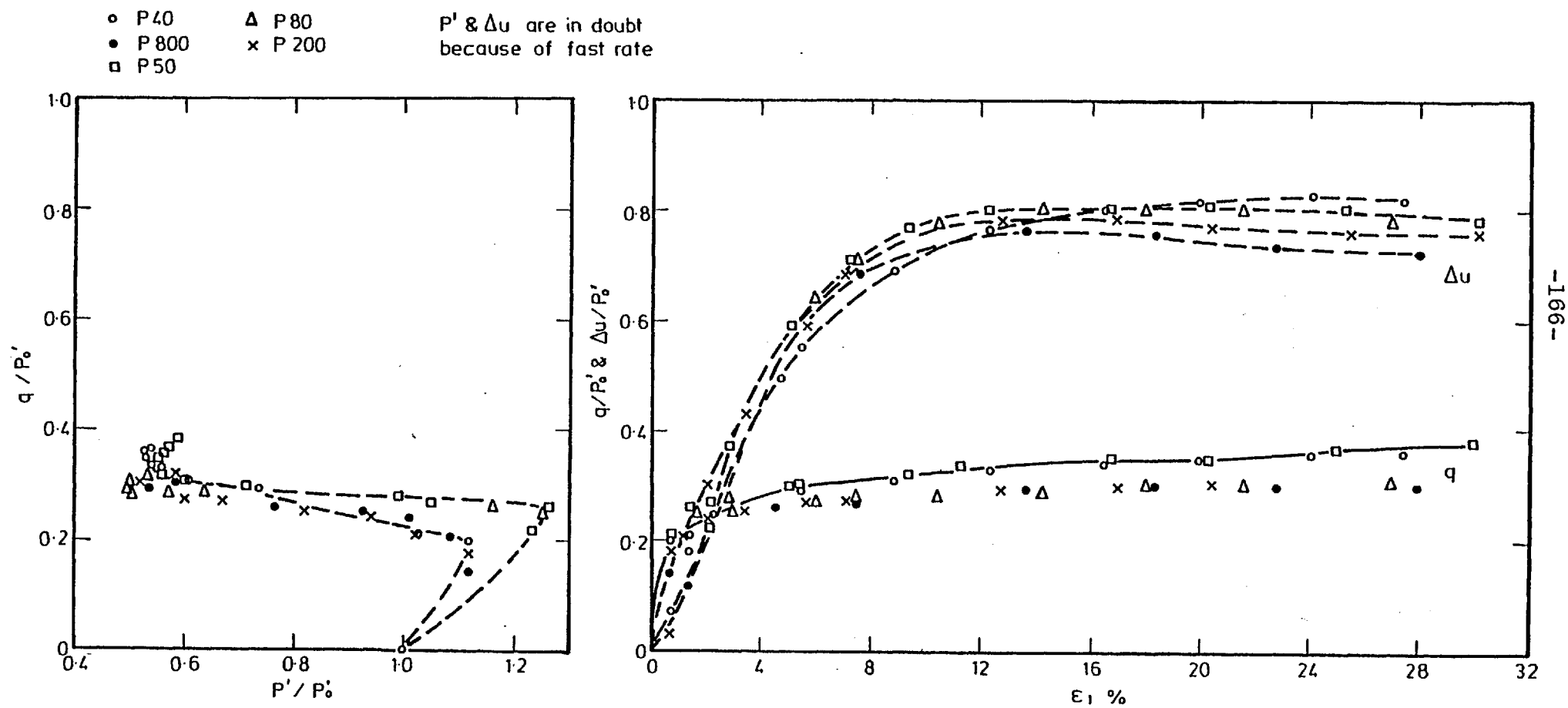


Fig. 6-8 Normalized results of tests on resedimented samples from slurry of Happisburgh Lower Till (CU tests-shearing rate 0.5 mm/min)

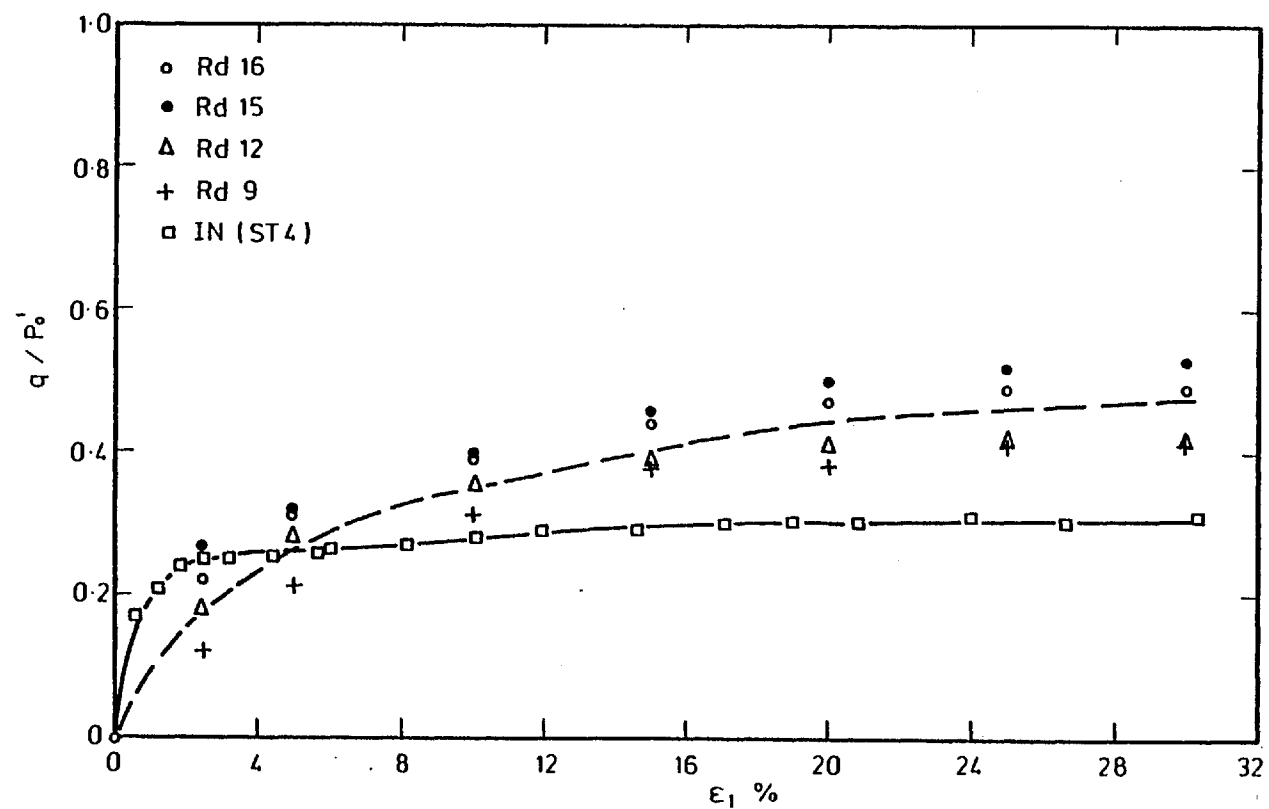


Fig. 6-9 Normalized results of tests on remoulded and resedimented samples from slurry of Happisburgh Lower Till (CU tests - shearing rate 5.0 mm/min)

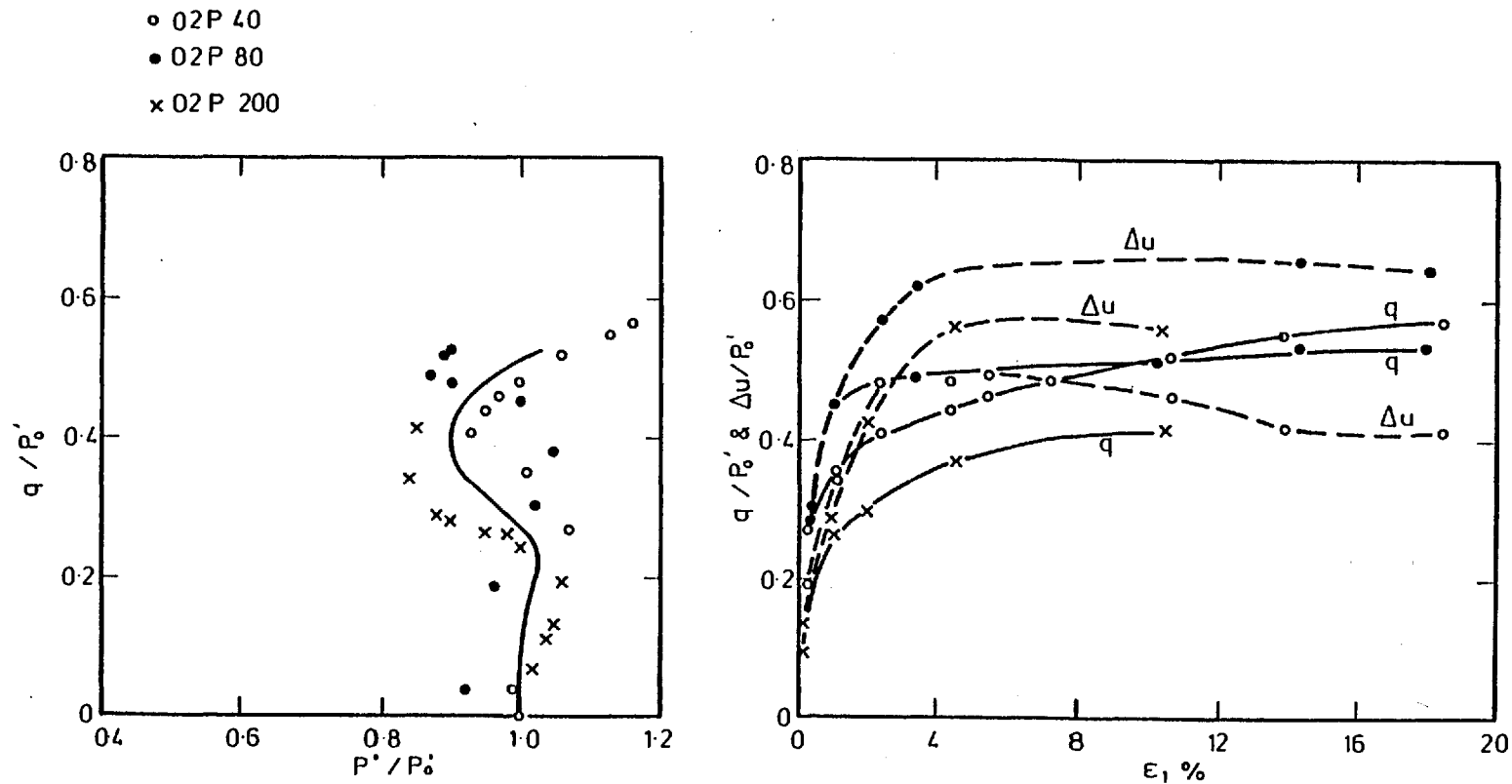


Fig. 6-10 Normalized results of tests on resedimented samples from slurry of Habbisburgh Lower Till (isotropically overconsolidated - OCR = 2 - CU tests- shearing rate=0.005 mm/min).

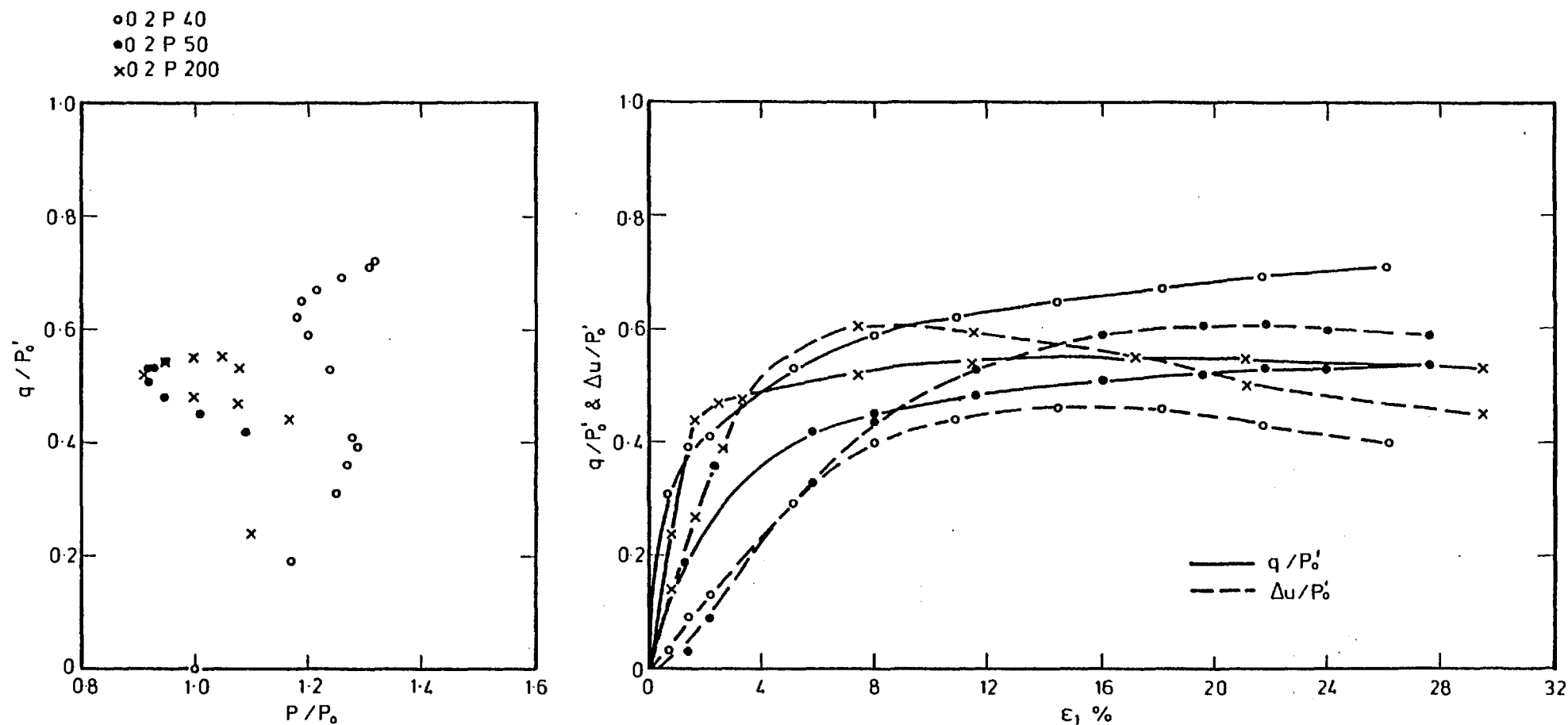


Fig. 6-11 Normalized results of test on resedimented samples from slurry of Happingburgh Lower Till (isotropically overconsolidated - OCR=2 - CU tests - shearing rate 0.5 mm/min).

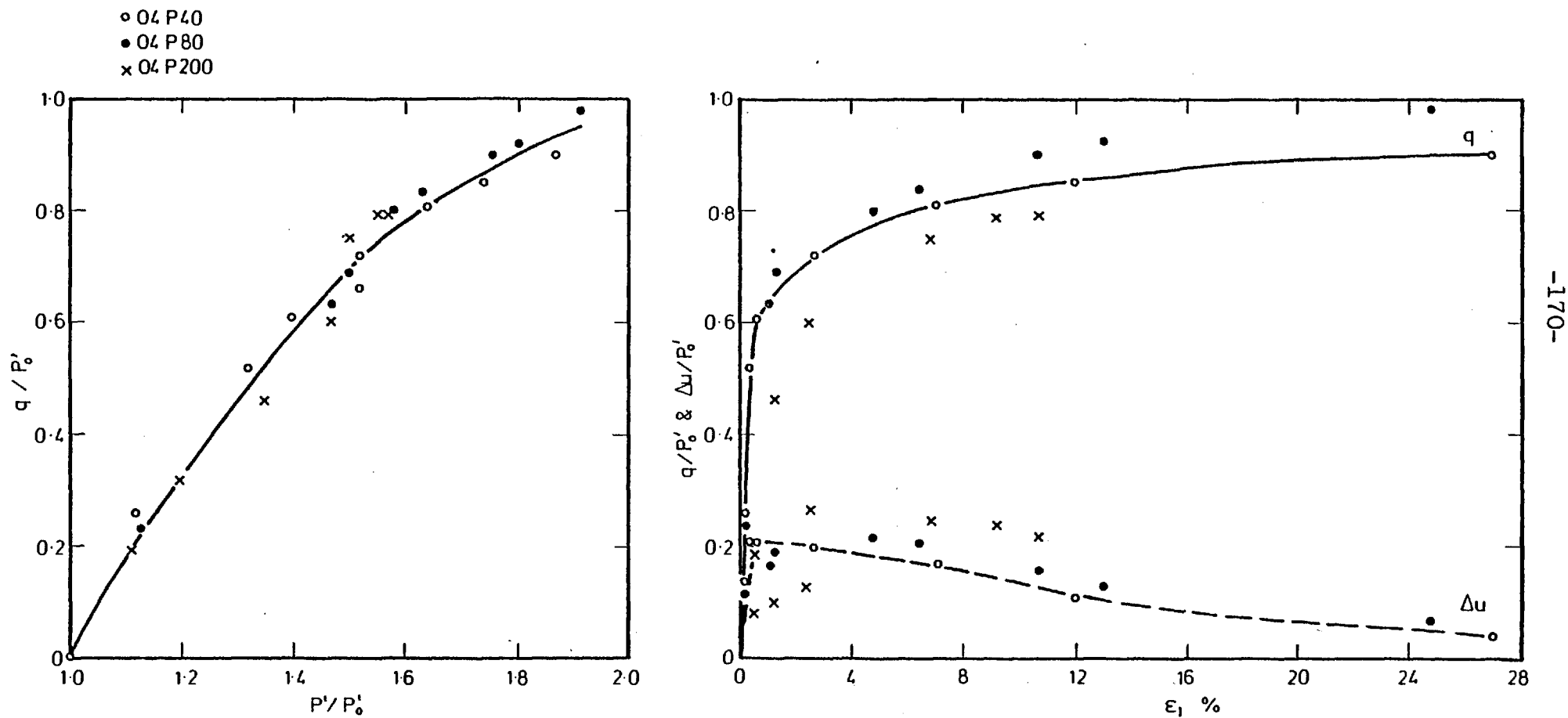


Fig. 6-12 Normalized results of tests on resedimented samples from slurry of Happsburgh Lower Till (isotropically overconsolidated - OCR = 4- CU tests - shearing rate = 0.005 mm/min)

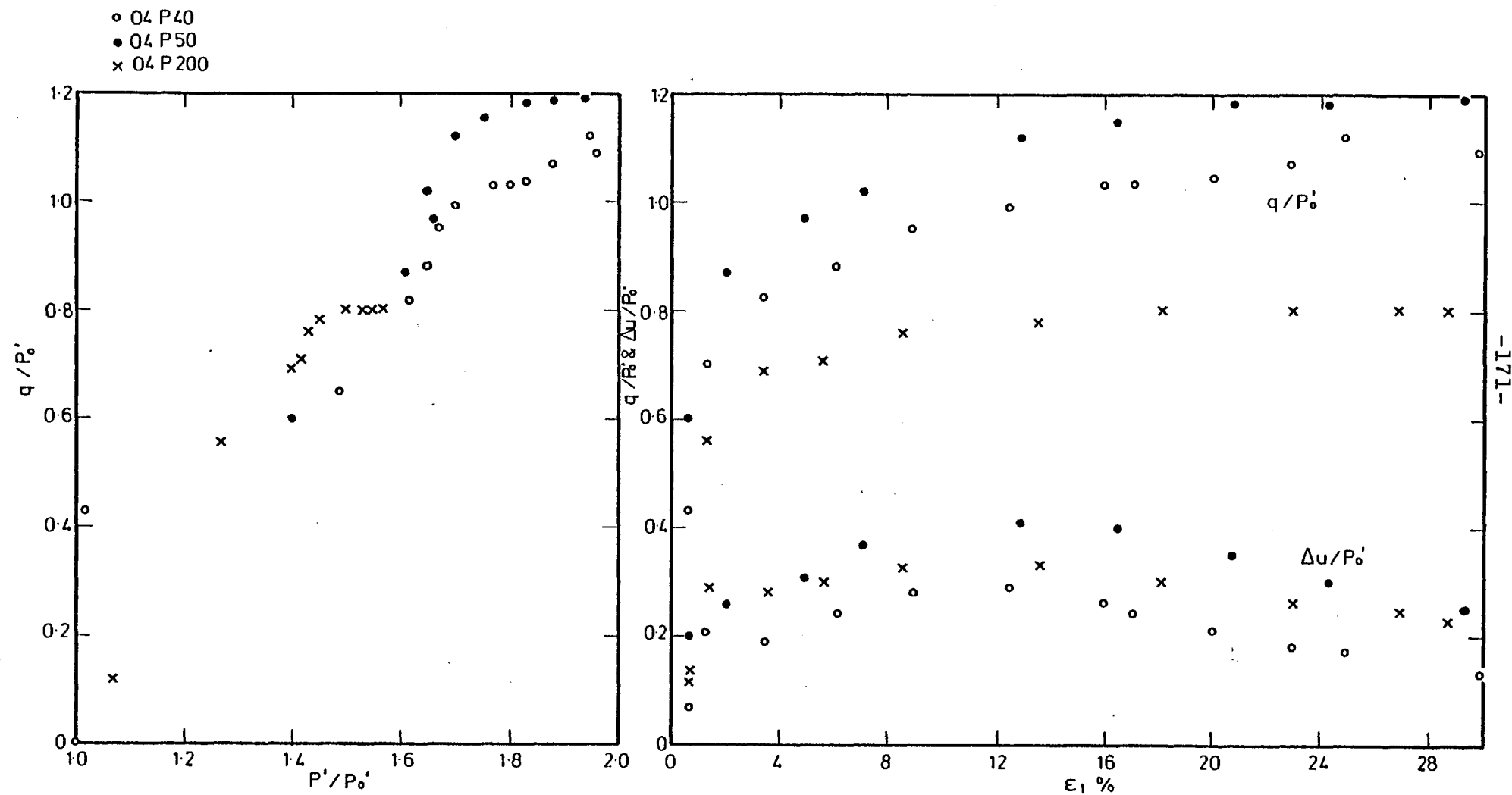


Fig. 6-13 Normalized results of tests on resedimented samples from slurry of Happsburgh Lower Till (isotropically overconsolidated - OCR=4 - CU tests - shearing rate=0.5 mm/min).

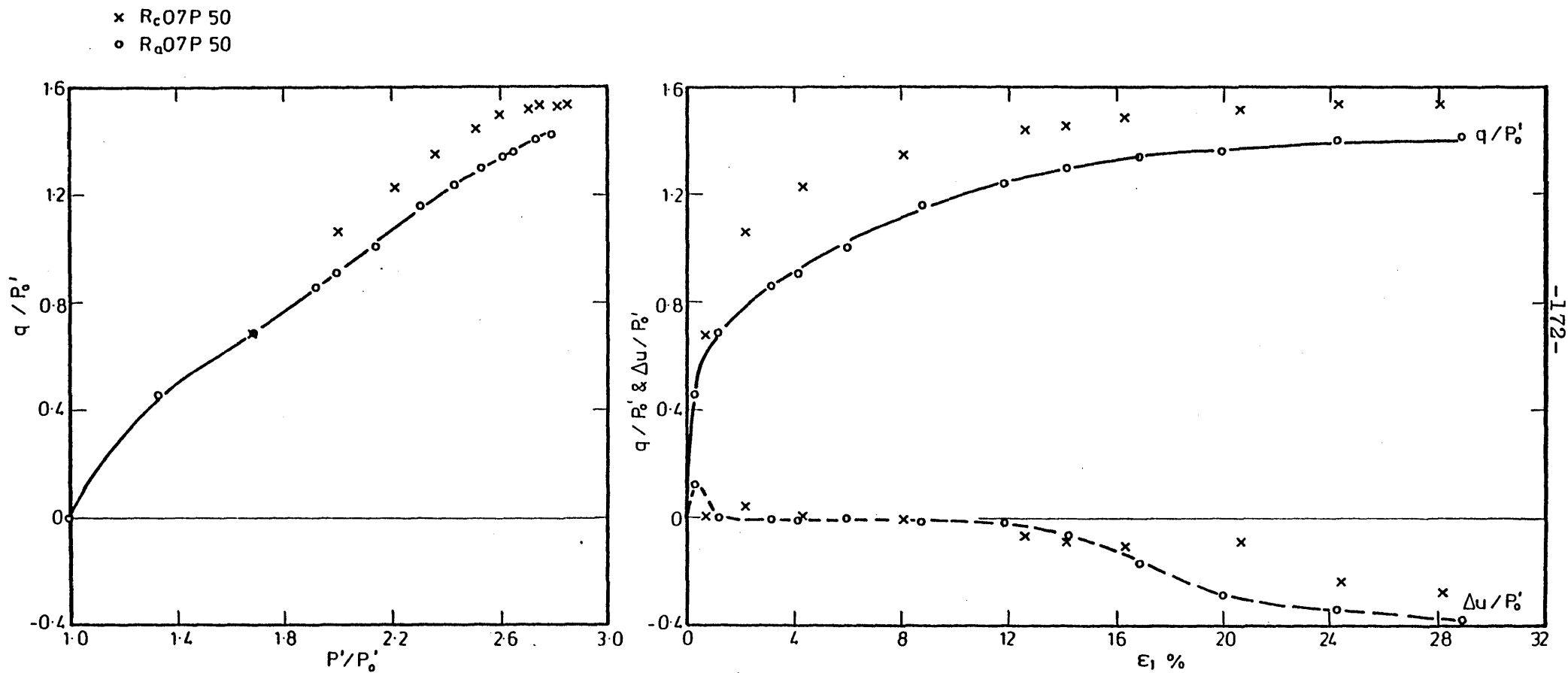


Fig. 6-14 Normalized results of tests on resedimented samples from slurry of Happsburgh Lower Till (isotropically overconsolidated - OCR = 7 - CU tests - shearing rates : $R_a = 0.005 \text{ mm/min}$ & $R_c = 0.500 \text{ mm/min}$)

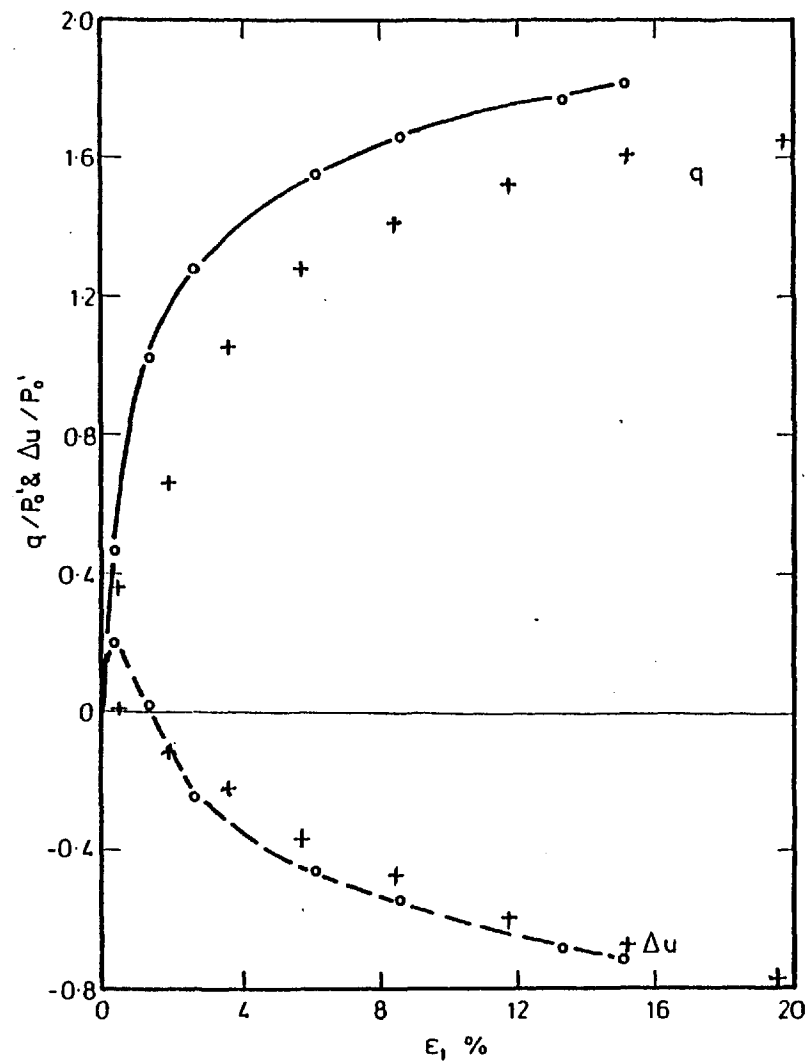
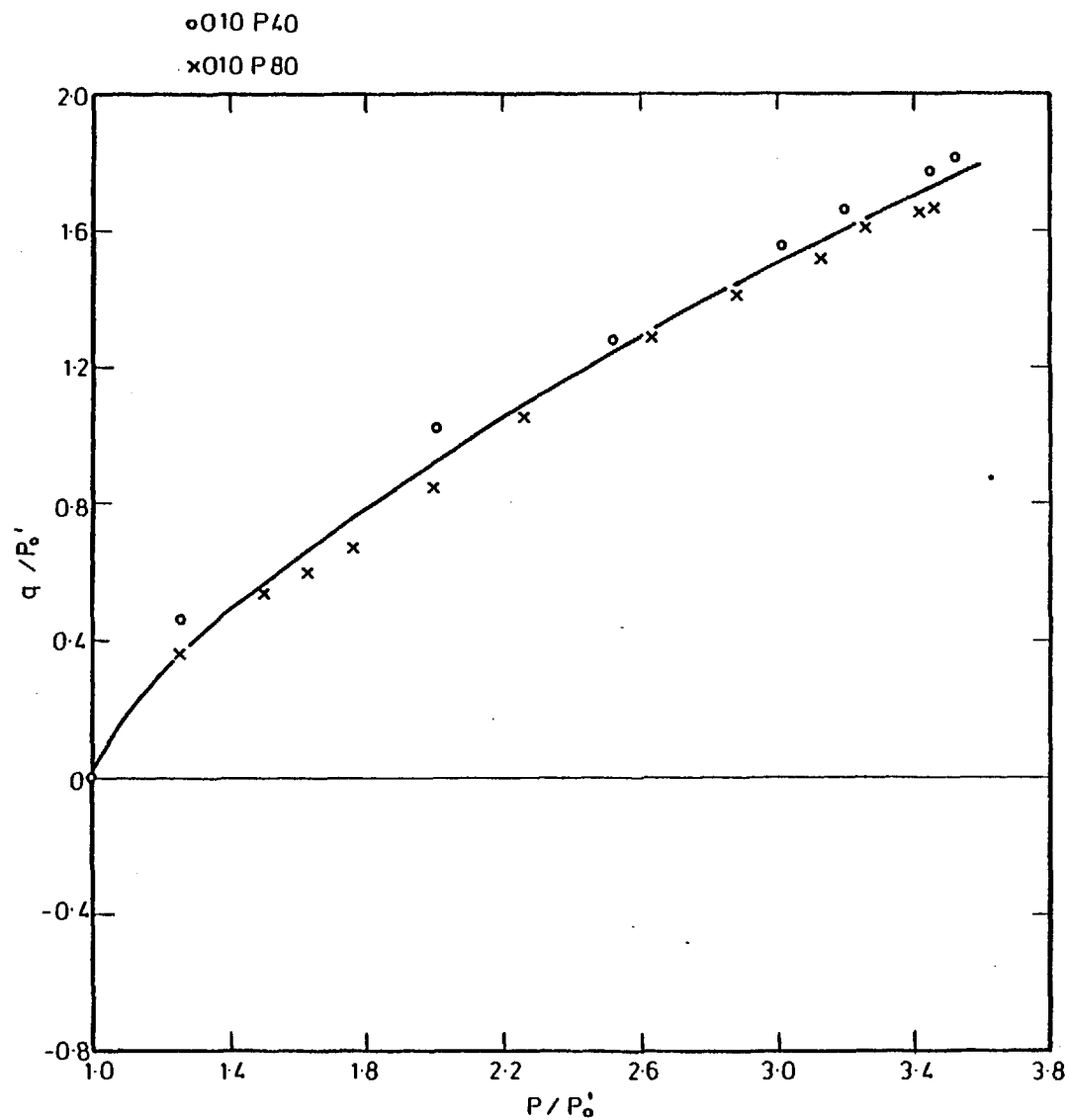


Fig.6-15 Normalized results of tests on resedimented samples from slurry of Happisburgh Lower Till
(isotropically overconsolidated - OCR = 10 - CU tests - shearing rate = 0.005 mm/min)

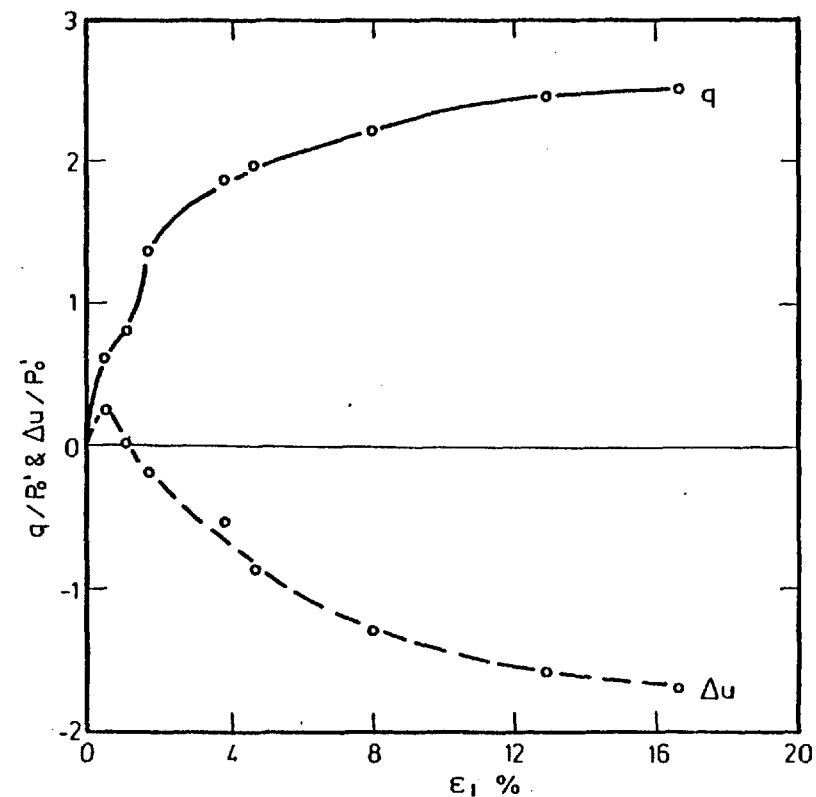
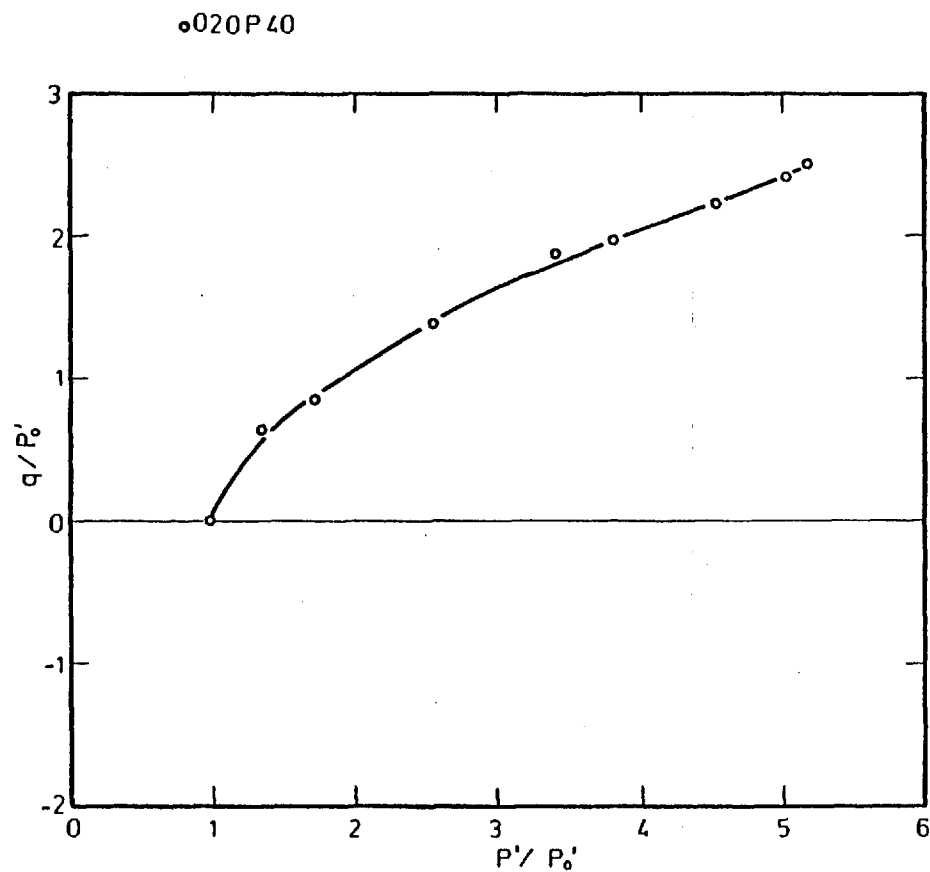


Fig. 6-16 Normalized result of test on resedimented sample from slurry of Happisburgh Lower Till "I020P40" (isotropically overconsolidated - OCR= 20 - CU test - shearing rate = 0.005 mm/min)

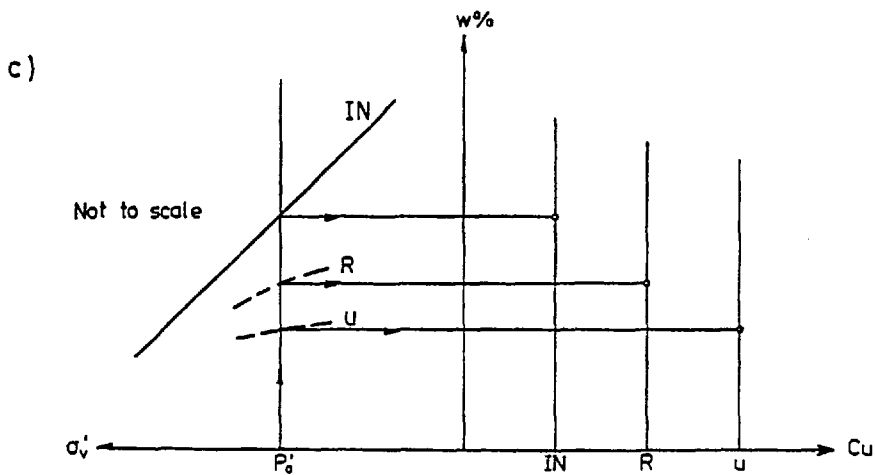
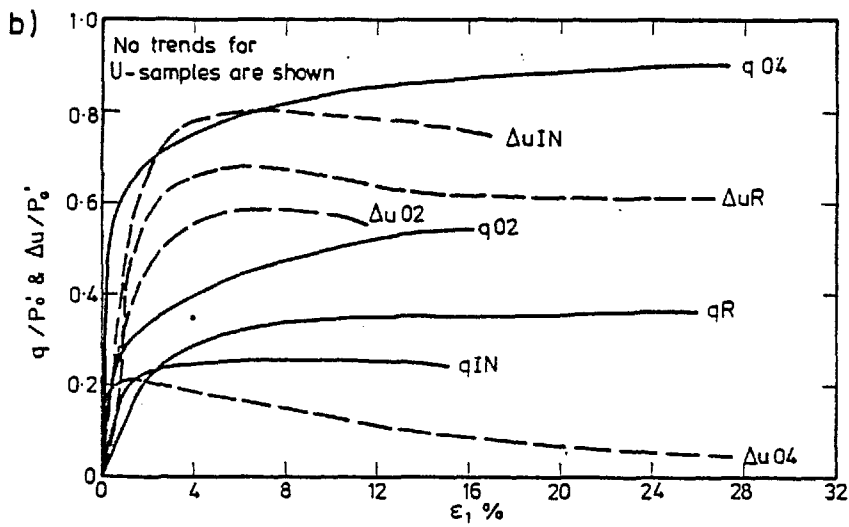
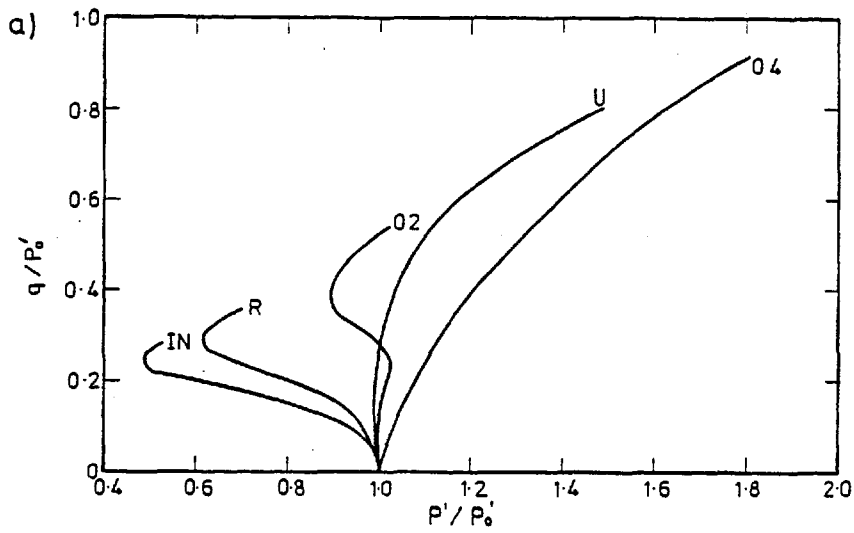


Fig. 6-17 Trends of behaviour of the Lower Till at the same P'_0 (shearing rate = 0.005 mm/min)

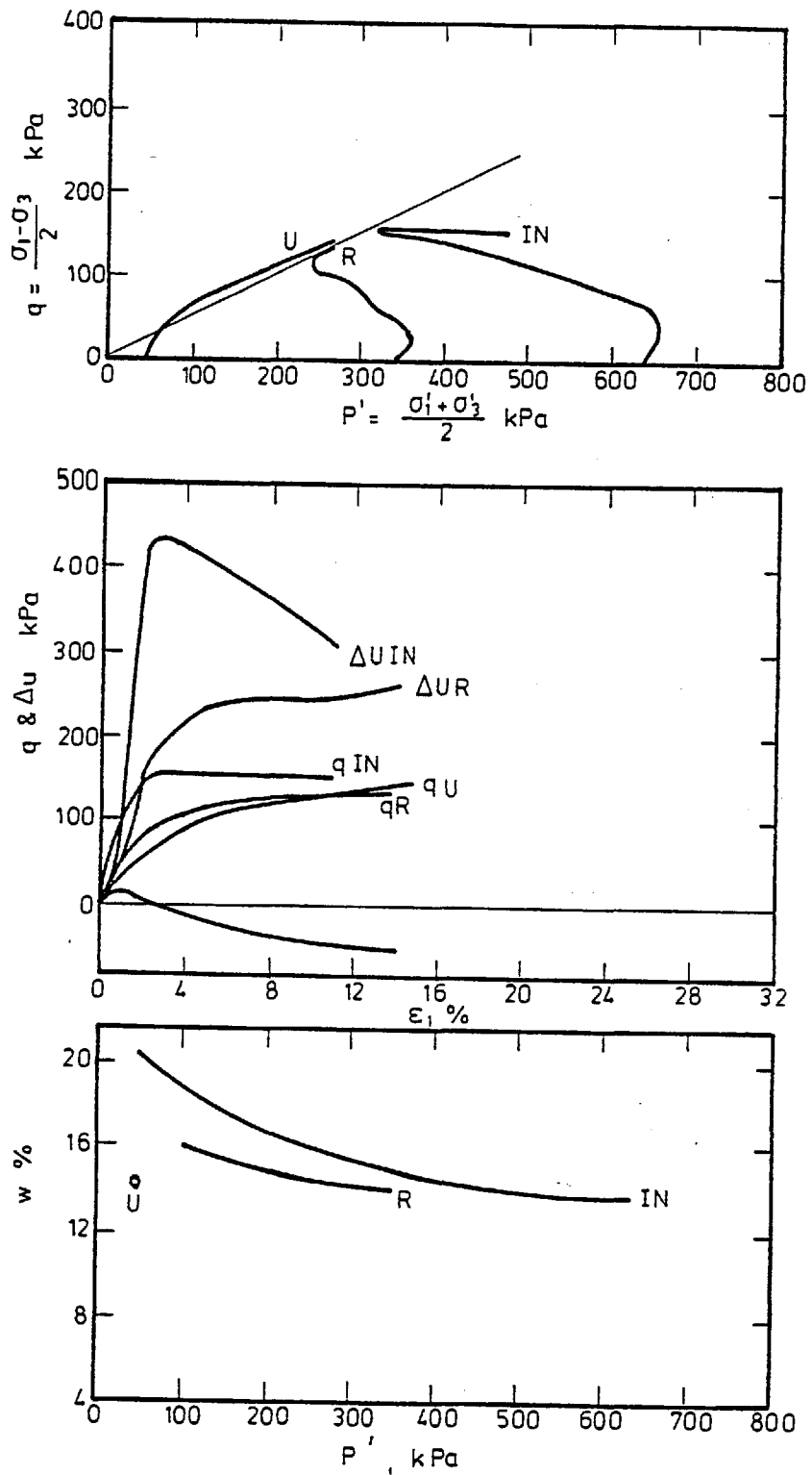


Fig. 6-18 Trends of behaviour of the soil at the "same" water content (R_a)

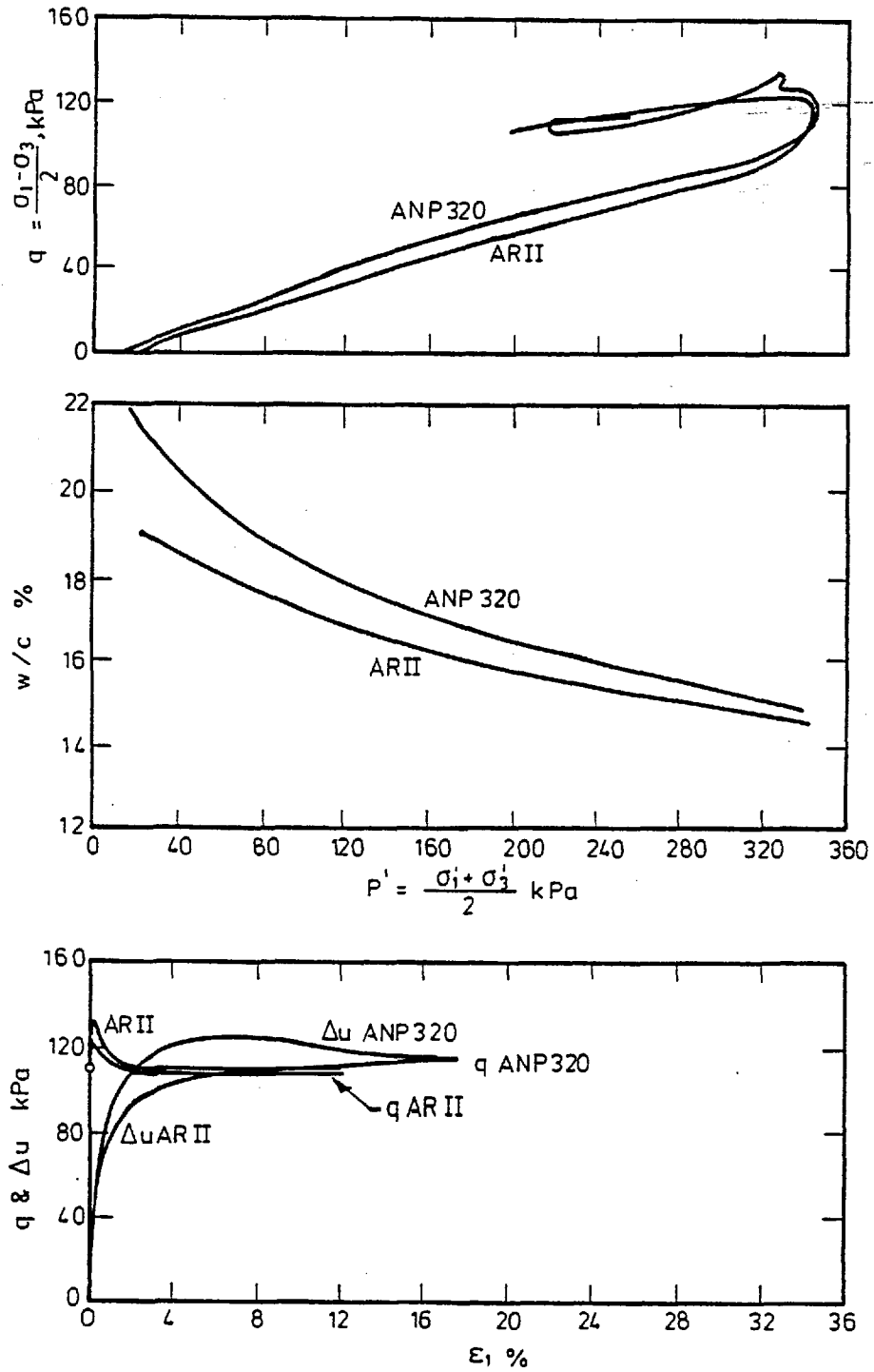


Fig. 6-19 Anisotropically remoulded and normally consolidated samples (R_a)

6.4 EFFECTS OF SHEARING RATE ($\dot{\epsilon}$)

6.4.1 General

In order to study the effects of shearing rates in UU and CU tests, only tests carried out under similar initial conditions should be compared. All the tests studied within this section were isotropically consolidated and sheared in a strain-controlled way. The two parameters whose variation will have the largest effect on any comparison are the water content during shearing (W_o) and the effective consolidation pressure at the start of shearing (P_o').

The studied soil is of low plasticity (P.I. = 13%) and the undrained strength (C_u) is, for a certain shearing rate ($\dot{\epsilon}$), a function of its water content (W_o). This is unlike the more plastic bonded clays where C_u depends on their effective consolidation pressure (P_o'), (e.g. Maguire, 1975, Vaughan et al. 1976 and Vaughan et al. 1978). It was thus decided to study the rate effect on C_u from W_o - C_u relationships. The estimate of C_u for such a non-brittle material will be dealt with in a forthcoming section (6.8).

The effect of rate on the effective stress parameters, pore pressure generation in remoulded (R) and normally-consolidated (IN) samples were studied using the curves normalised for P_o' from item 6.3 since a more comprehensive picture of the relative trends could be drawn by using such curves and because they eliminate the effect of the varying W/C as will be discussed in item 6.4.2.

The above effects of rate in overconsolidated samples (OC) has been studied, enabling some rate effect-stress history relationships to be examined.

6.4.2 The Effect of Shearing Rate on the Undrained Strength ($\dot{\epsilon} - C_u$)

1) Remoulded samples: the test results are presented in a $W_o - C_u$ space (C_u plotted to a $\log_{10}$ scale) in Fig. (6.20), for four rates (Ra 0.005 mm/min: Rd 7.62 mm/min). Also shown on the same figure are results from another boulder clay from Cow Green (Sodha 1974), laboratory vane tests (Sano 1977) and model test results (Herbertson 1977). The vane and model tests were carried out on the same Lower Till. The results give an almost linear relationship and in order to look into the results in more detail, test results of each rate were grouped together and the linear relationship it assumed was calculated by linear regression, Fig. (6.21a).

The correlation coefficient of the linear regression did not fall below 0.984, furthermore the chosen water contents for comparison were between 13% and 18% to avoid any extreme values which might have been caused by too wet or too dry samples.

The shaded area in Fig. (6.21a) shows the range in which comparison is to be made. It is obvious that except for very low water contents where full saturation is harder to ensure the effect of rate on C_u is small and may be considered to be insignificant when account is taken of the scatter in C_u at any particular rate.

It is also shown that at higher water contents the undrained strength increased with increasing shearing rate whereas at the lower water contents the undrained strength decreased with increasing shearing rate. It was decided to elaborate more on the results in order to try to compare values of C_u for different rates and an average line for all rates was drawn and used as a datum for comparison. In Fig. (6.21b) the ratios of strengths are plotted for different water contents at various rates. It is shown in percentages of strength how the wetter samples ($W_o = 16\%$ and 18%) had lower strengths at slow rates and higher strengths at fast rates while the drier samples ($W_o = 13\%$ and 14%) behaved in an opposite way. It is also shown that the shearing rate has least effect in the range between 0.10 mm/min and 1.00 mm/min and in samples with "intermediate" water contents ($W_o = 14\%$ and 16%).

The phenomenon of increased strength at fast rates as observed above at high water contents, has been reported to be the overall behaviour for different soils (e.g. Bjerrum et al. (1958) reported it for CU tests on soft marine clays, Crawford (1959) for sensitive marine clays, Skempton and La Rochelle (1965) and Sandoni (1977) for London clay, Wesley (1975) and Buri (1978) for a soft clay from Mucking, Maguire (1975) for Upper Lias clay and El-Runwayih (1975) for remoulded London clay and Kaolin). Some tests by field vane also gave similar trends, as reported by Wiesel (1973) for tests on normally consolidated soft glacial postglacial clays. The reversed phenomenon

that takes place at lower water contents was reported by Bishop et al. (1960) for compacted London Clay and Sodha (1974) for Cow Green boulder clay.

The discontinuous trends shown in Fig. (6.21b) were observed from other vane tests by Cadling and Odenstad (1950), Flaate (1966) and Pugh (1978). These tests were carried out at different rates ensuring as far as possible undrained behaviour.

Fig. (6.22) shows deformation curves of six remoulded samples, each three of which have "similar" water contents. The three samples with lesser water contents 11.00%, 11.33% and 11.03% gave higher strengths at slower rates (although outside the recommended range of water contents these three tests share almost the same W_o and confirm the trend stated above). For the samples with higher water contents 16.58%, 16.50% and 16.55%, lower strengths were measured at slower rates.

In an attempt to clarify more the rate effect, two multi-stage tests ST5 and ST6 (figs 6.23 and 6.24) were carried out in which the strain rate $\dot{\epsilon}$ was changed during the same test to avoid as many variable parameters as possible. In the first test ST5, $\dot{\epsilon}$ was increased in four steps then decreased in another three steps. The results, Fig. (6.23) showed that the energy absorption by the apparatus and the insufficient time for pore pressure equalisation at each change, masked the rate effect. It was decided to carry out the second test ST6 so that $\dot{\epsilon}$ decreased only in four steps. Sandroni (1977) suggested that a multi-stage test can only be meaningful if the

shearing rates decreased to allow for pore pressure dissipation. The results of ST6 allow quite clearly that although the shearing rate decreased from 0.1168 mm/min to 0.0009 mm/min (129.78 times), the effect of this change was insignificant as far as the changes in shearing resistance is concerned.

2) Normally-consolidated samples: In Fig. (6.25a) test results of two rates (R_a and R_c) are presented as above. It is shown that at higher values of water content the slower rates gave higher strengths and at lower values of water content they gave lower strengths. It should be noted that the average lines of both rates are more convergent than in the case of remoulded soils and that the shaded area showing the chosen range for comparison is smaller than that of fig. (6.21a) for remoulded soils. The overall effect of rate on C_u is insignificant. However a similar elaboration on the results is carried out as for remoulded soil and the percentages deviation of C_u from the values of the average line are shown in Fig. (6.25b) for different water contents.

They show continuous and almost symmetric behaviour confirming the above. The shearing rate has least effect for soils with "intermediate" water contents ($W_o = 14\%$ and 16%) and in the range between 0.01 mm/min and 0.10 mm/min.

Richardson and Whitman (1963) reported a reversed pattern from the one observed above when testing fat clay (LL = 62%) normally-consolidated from slurry. Olson (1963) in his slurry tests on sodium illite also reported a reversed pattern from above.

In order to check on whether the above observations are affected by the scatter of results and the extreme points in the linear regression or whether there are physical trends for these behaviours, Fig. (6.26) was plotted. In Fig. (6.26) only two rates are considered, namely R_a (0.005, 0.00762 mm/min) and R_c which is 100 times faster than R_a . Two regions of water contents are shown for each soil structure, these regions represent "high" and "low" water contents as suggested by Figs. (6.21) and (6.25). From Fig. (6.26) it is shown that the scatter of results is less in the case of normally consolidated samples from slurry than in the case of remoulded samples which is apparent even in these relatively small regions of water contents. There are definite trends confirming the previous findings as regards the normally consolidated samples. In remoulded samples at high water contents there is a confirming trend to the previously described, though the definition of this trend is rather faint. At low water contents the remoulded samples have indeed large scatter and no definite trend could be concluded at this region.

It has to be noted here that the relationship between the water content and the undrained shear strength is subjected to scatter especially in the remoulded samples. It is also observed that the above relationship holds better in case of tests starting from a high value of P_o' where the stiffness is larger than for tests of low P_o' and the "ultimate" strength is reached at relatively low axial strains (see Hight 1978).

3) Overconsolidated samples: Overconsolidated samples with $OCR = 2$ and 4 were also examined at two rates (R_a and R_c) and they gave closely similar trends to normally consolidated samples. Their average lines are shown in fig. (6.25a). It can also be observed from Fig. (6.12) that at a degree of overconsolidation of about 4 the soil is least affected by rate.

6.4.3 The Effect of Shearing Rate at Different Strain Levels

The sometimes confusing and contradicting response of the soil to the shearing rate reflects to a degree the ambiguity by which the value of C_u is determined in a non-brittle material without a pronounced peak, see item 6.8. It was also thought that it was probable that the rate could affect the shear stresses differently at different levels of straining (expressed here by the axial strain ϵ_1), i.e. the pre-failure behaviour of the soil, a factor which would be important in any engineering design.

The same calculations and plottings as in item 6.4.2 were carried out at four different levels of axial strain: 5%, 10%, 15% and 20% for remoulded, normally and overconsolidated samples.

1) Remoulded samples: the results of the above-mentioned calculations are plotted in Fig. (6.27).

The $W_o-q_{mob}^*$ curves show some difference in the relative positions of the lines for different rates, there being more convergence as the straining increases, but no drastic change in the W_o-q_{mob} relationships took place between $\epsilon_1 = 5\%$ and $\epsilon_1 = 20\%$. At all levels of straining a similar pattern to fig. (6.21b) was obtained so that the relative values of q_{mob} obtained at different rates varied slightly with the level of strain. It is also concluded that the rate affects the initial straining (reflected by $E = q/\epsilon_1$ at $\epsilon_1 = 1\%$) in this type of soil so that E decreases as the rate increases.

2) Normally-consolidated samples: the results for this type of soil are shown in Fig. (6.28) for two rates (Ra and Rc). Insignificant changes in the W_o-q_{mob} lines occurred at different strains and in the $\dot{\epsilon}-\% q_{mob}$ plots only small changes in convergence to the average line is noticed as the strain level increased. From these facts it could be concluded that the shearing rate has insignificant effect on the initial straining (ϵ) and that the relative values of $\% q_{mob}$ obtained at different rates remained fairly constant throughout the various levels of straining. The smallest effects of rate were as observed in Fig. (6.25b) for all strain levels.

3) Overconsolidated samples: the samples with $OCR = 2$ and 4 showed similar behaviour to the normally-consolidated samples. Fig. (6.29) shows results of the

* q_{mob} is the mobilised shear strength

tests with $OCR = 4$ in which the $W_o - q_{mob}$ lines show little difference at different strain levels except at high levels where faster rates begin to give higher q_{mob} at lower water contents.

Only a few scholars have studied the effect of rate at different strain levels and those who did confined their reporting to the effects on the pore pressure ΔU . Richardson and Whitman (1963) studied the rate effect at "small" and "large" strains on an isotropically consolidated fat clay from slurry. They concluded that the difference in pore pressure response with rate was more significant at large strains than at small strains. Sandroni (1977), testing undisturbed samples of London clay, which were anisotropically consolidated in the field, in undrained tests, concluded that at small strains the pore pressure is independent of rate. Olson (1963), testing sodium illite samples consolidated from slurry concluded that the pore pressure depends on the strain level and not the strain rate.

6.4.4 The Effect of Shearing Rate on the Effective Stress Parameters $[\dot{\epsilon} - (c', \phi')]$

To assess the effect of shearing rate on the effective stress parameters one is limited to the rates at which pore pressures applicable to the failure zone can be measured reliably. In the case of pore pressures measured at the base, as in this research, only in tests sheared at rate R_a can be measured pore pressures be considered to be wholly reliable. While reference is made in

the following two sections to pore pressures measured in tests at rates Rb and Rc, it must be emphasised that these pore pressures become more reliable as the test proceeds. Conclusions based on the end sections of ESP's are therefore of more value than those based on maximum principal effective stress ratios, since these tend to occur at small strains.

1) Remoulded samples: in Fig. (6.30) the ESP of remoulded samples at Ra and Rb touch the failure envelope at approximately the same inclination $[\psi = \tan^{-1} 0.5 = 26.565^\circ]$ indicating similar values for effective stress parameter $[\phi' = 30^\circ, \text{ and } c' = 0]$. It is noticed here that at Rb, the ESP climbed up above the envelope established by Ra and after some straining descended back and touched it. This shape of ESP may result from pore pressures within the sample not being equilibrated until the final stages of the test (see item 6.3.1). The ESP at Rc suggests a failure envelope with a ϕ' of 33.1° , however, this could be the result of incorrect pore pressures being measured.

Fig. (6.31) shows $(\sigma_1'/\sigma_3')_{\text{max.}}$ plotted against σ_{v_o}' ; it can be seen that the average line representing test results carried out at Ra is almost horizontal, i.e. giving the same value of ϕ' for all values of σ_{v_o}' . Test results of Rc lie above this line and they both tend to converge at high σ_{v_o}' . At an average value of σ_{v_o}' of 200 kPa, the difference between the values of effective stress ratios give a difference in ϕ' of 3.06° .

2) Normally-consolidated samples: from Fig. (6.30) the ESP of normally-consolidated samples sheared at R_a , gives a lower value of ϕ' than that given by R_c . As for R_b the one test shown indicates an even higher value of ϕ' than R_c . The above trend seems to prevail in tests carried out at consolidation pressures higher than 100 kPa as shown in Fig. (6.31a). In Fig. (6.31a) it is clear that the effective stress ratio for tests sheared at R_c is almost the same for all value of σ_{v_o}' . Tests sheared at R_a , however, showed higher values of ϕ' (or c') at low values of σ_{v_o}' (less than 100 kPa).

The general tendency for the increase of ϕ' with increase in rate of shear in undrained tests has been repeatedly stated in the literature (e.g. Bjerrum et al. 1958, Maguire 1975, Richardson and Whitman 1963, Buri 1978 and Wesley 1975). Perloff and Osterberg (1963) reported very little effect of rate on c' and ϕ' for normally-consolidated samples of plastic illitic clay. Kenny (1959) argued that "true" c' and ϕ' may decrease with the increase in rate and he also stated that this effect might be damped in undrained tests by generation of excess pore pressure (and by secondary consolidation in drained tests).

3) Overconsolidated samples: in samples with $OCR = 2$, the data available indicate no particular trend concerning the relationship between the shearing rate and the effective stress parameters (c' , ϕ'), Fig. (6.32a). As for samples with $OCR = 4$ the faster rate R_c gave consistently higher values of effective stress ratio, than

tests sheared at the slower rate R_a , Fig. (6.32a).

6.4.5 The Effect of Shearing Rate on the Generation of Excess Pore Pressure $[\dot{\epsilon} - \Delta U]$

The different trends of curves showing excess pore pressure generation (ΔU) as they vary with different shearing rates are presented in Fig. (6.30) and the relative values of excess pore pressures to maximum shear strength, (represented by the pore pressure coefficient A at failure A_f), for different rates at various consolidation pressures are presented in Figs. (6.31b) and (6.32b).

1) Remoulded samples: from Fig. (6.30), the maximum value of ΔU for tests sheared at R_a and R_c are the same whereas those tests sheared at the intermediate rate R_b gave lower values (5.7% lower than at R_a and R_c). The axial strains at maximum pore pressures increased consistently with increasing rate. In Fig. (6.31b) it is shown that the values of A_f for slower rates are larger than those of the faster rate for consolidation pressures more than 100 kPa. The reverse took place at smaller consolidation pressures.

2) Normally-consolidated samples: from Fig. (6.30) all rates yielded the same value for maximum ΔU whereas the axial strains at which these values were reached increased with increasing rate. As in remoulded samples the normally-consolidated samples gave higher values of A_f when sheared at the slower rate than when sheared at the faster rate for values of P_o' more than

100 kPa, Fig. (6.31b). No reverse pattern occurred at smaller consolidation pressures. It is important to keep in mind that when comparison is carried out at the same water content, Fig. (6.18) and item 6.4.2, the normally consolidated samples started at higher P_o' than the remoulded ones and generated much higher pore pressures.

3) Overconsolidated samples: from the data available at Fig. (6.32b) it is shown that the values of A_f for samples with OCR = 2 were higher for tests sheared with R_a when the consolidation pressure P_o' exceeded 80 kPa. Samples with OCR = 4 had an average line representing R_c test lay just above the average R_a line. The two lines merge at high P_o' . Most scholars reported similar behaviour to the above, namely, an increased pore pressure at slow rates for different types of clay. Bjerrum et al. (1958), however, stated that rate had very little effect on ΔU in CU tests on sensitive marine clay. Wesley (1975) found that A_f increased as the rate decreased for soft clay from Mucking but this finding was not clear cut which he attributed to unequalised ΔU inside the sample.

6.4.6 The Effect of Shearing Rate on ESP and Deformation for Different Soil Structures

A compilation of the previous two items and Fig. (6.30) give the data required for such a study. At the slowest rate R_a , Fig. (6.30a) the maximum pore pressure is reached for remoulded and resedimented soils at smaller axial strains when compared to faster rates. This reflects in the shapes of ESP's.

The value of the axial strain at which a maximum value of $C_u = \frac{1}{2} (\sigma_1 - \sigma_3)$ is reached. (ϵ_f), increased as the rate increased in normally-consolidated soils although the stiffness E , (E is the ratio q/ϵ_1 at $\epsilon_1 = 1\%$, see item 6.8.4) remained fairly constant. As for remoulded soils the rate had no effect on ϵ_f but the stiffness E decreased as the rate increased.

The above observations give rise to the differences in ESP shown in Fig. (6.30) from which it can be concluded that at slow rates the overall differences in ESP's and deformation curves between remoulded and normally-consolidated soil structures are minimal. These differences increase as the shearing rate gets faster.

6.4.7 Epilogue

1) All samples studied within this section failed by bulging and their water contents before and after failure were examined. Figs. (4.15) and (4.16) are examples of the measurement of water contents after testing. This measurement did not reveal any large differences in the value of w/c at different parts of the sample which reassured the author somewhat, that even when fast rates were used, reasonable pore pressure equalisation occurred by the end of the test.

2) It is recommended that more tests on rate effect be done on anisotropically-consolidated samples since this is the normal stress system in most "in-situ" conditions and very little data on this point is available (see Sandroni 1977).

3) It is also recommended that these tests be carried out at a temperature equal that in-situ because the deformations in soils are thermally activated (Mitchell 1976) and large differences in temperature, for instance between an ocean floor and the laboratory might be responsible for smaller values of C_u and unrepresentative results (Kelly 1978).

4) Tests with rates slower than R_a , (T_{10} , T_{12} and T_{24}) gave similar results to tests sheared at R_a and it was decided not to shear at any slower rates since the rate effects will diminish as the strain rates fall below R_a (Terzaghi 1931).

5) In this section the study on C_u has been carried out from the W_o - C_u relationships because of the dependence of C_u on W_o (especially at high values of P_o'). It has been shown that within the "middle" of the workable range of the water contents for this material, the rate effect is minimal. In remoulded samples, however, the rate effect in the above middle range was conventional, i.e. higher values of shear strength for faster rates. In the extremes of the water spectrum the effect of rate had the effects described in item 6.4.3. The study on ΔU ,

ϕ' , ϵ_f and structure was carried out from curves normalised against P_o' , item 6.3. The results from the studies, using W_o or P_o' are consistent. It is unrealistic, in the author's opinion, to study the rate effects on different soil parameters from relationships which have not been normalised as has been done in some of the literature.

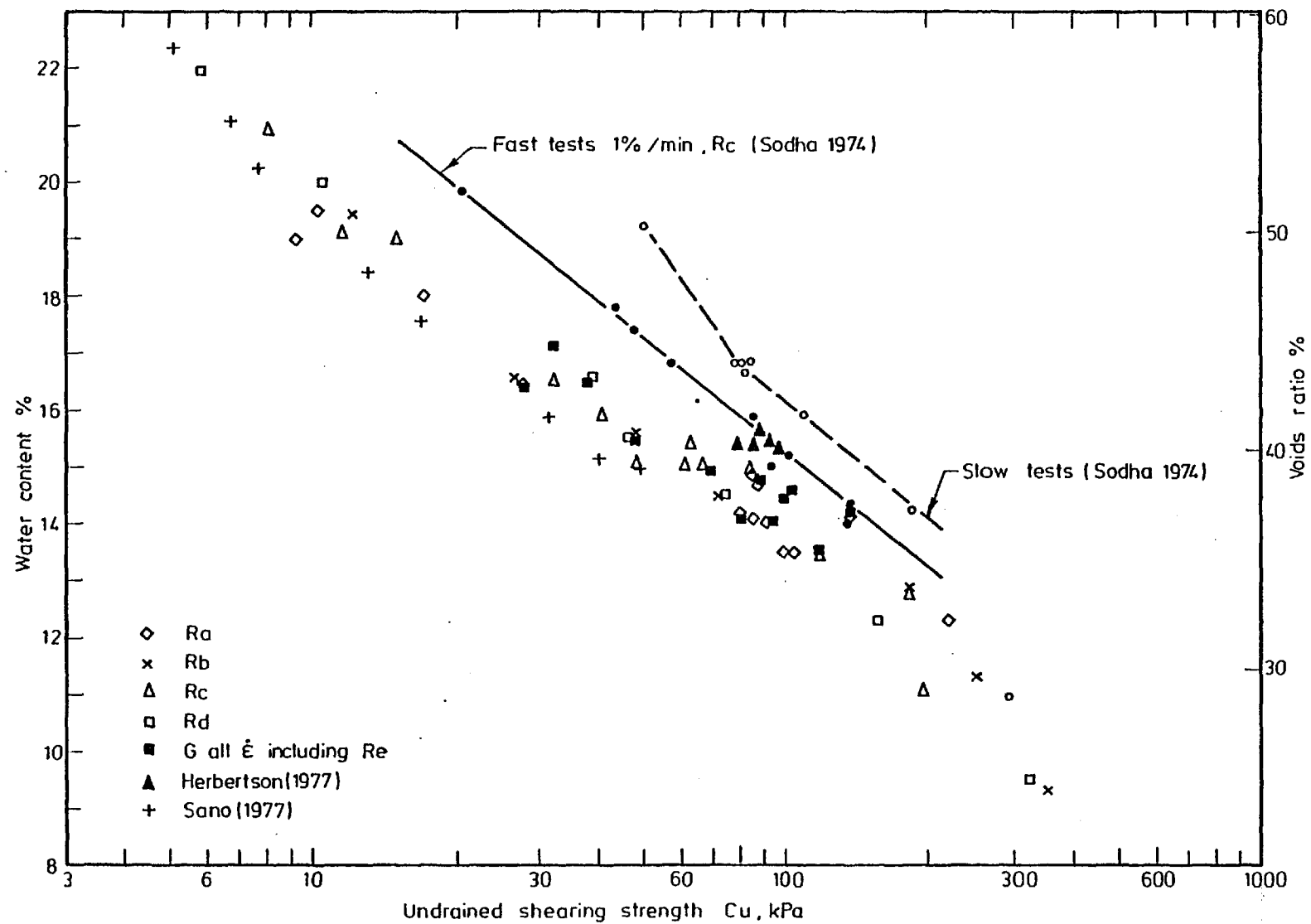


Fig. 6-20 The relationship between undrained shearing strength C_u and the final water contents of remoulded samples of Happingburgh Lower Till.

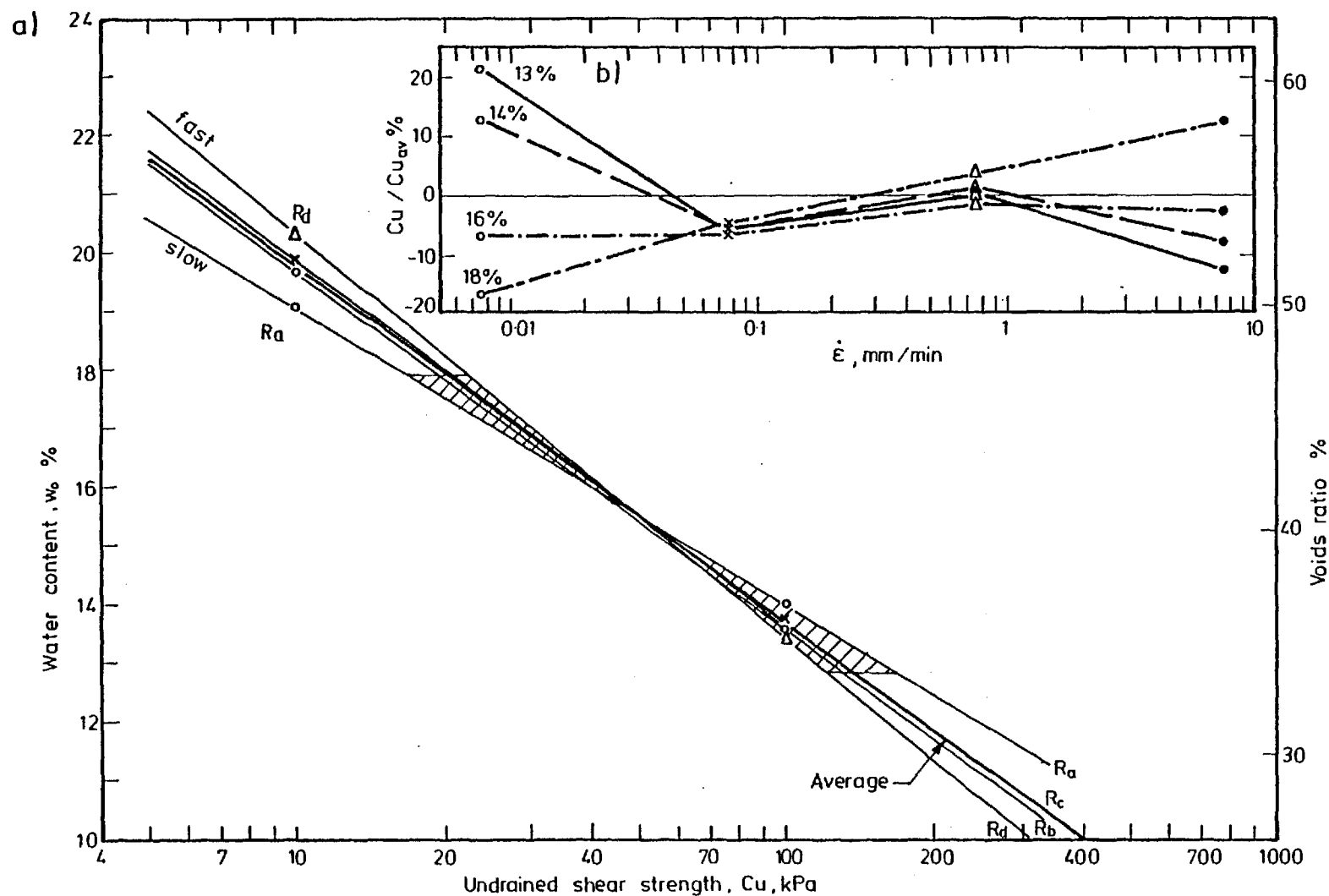


Fig. 6-21 The relationship between undrained shearing strength C_u and the final water contents of remoulded samples of Happingburg Lower Till.

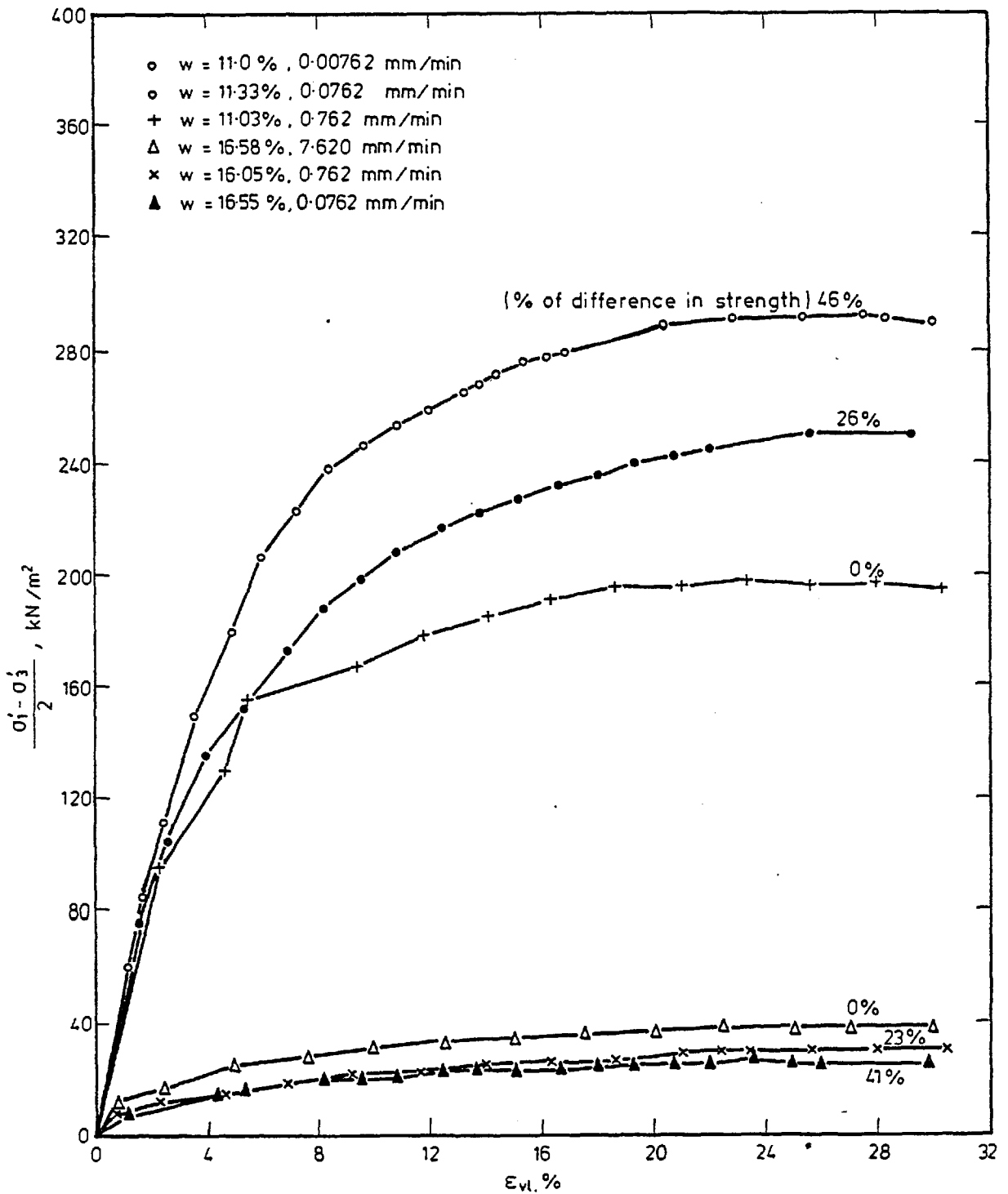
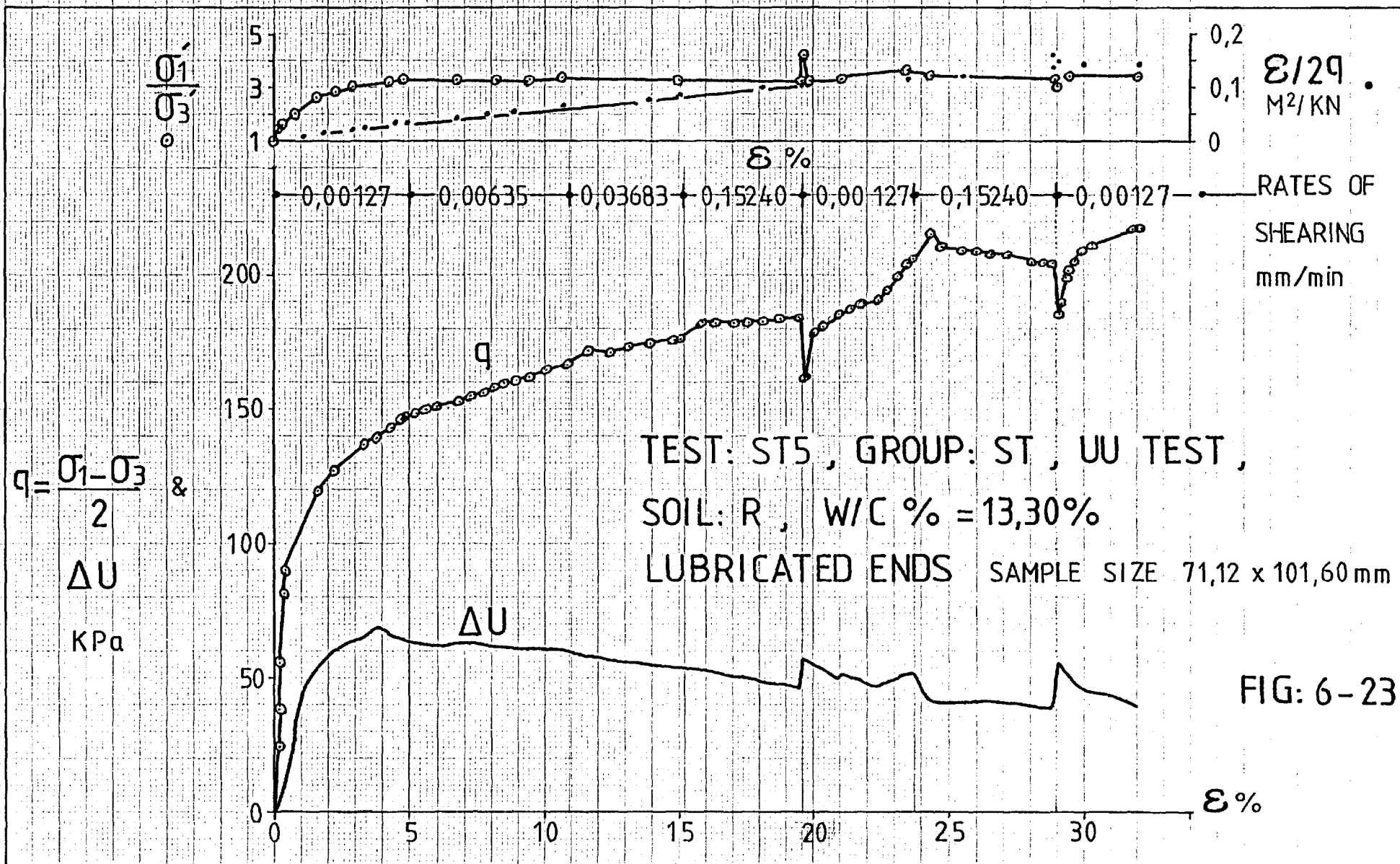
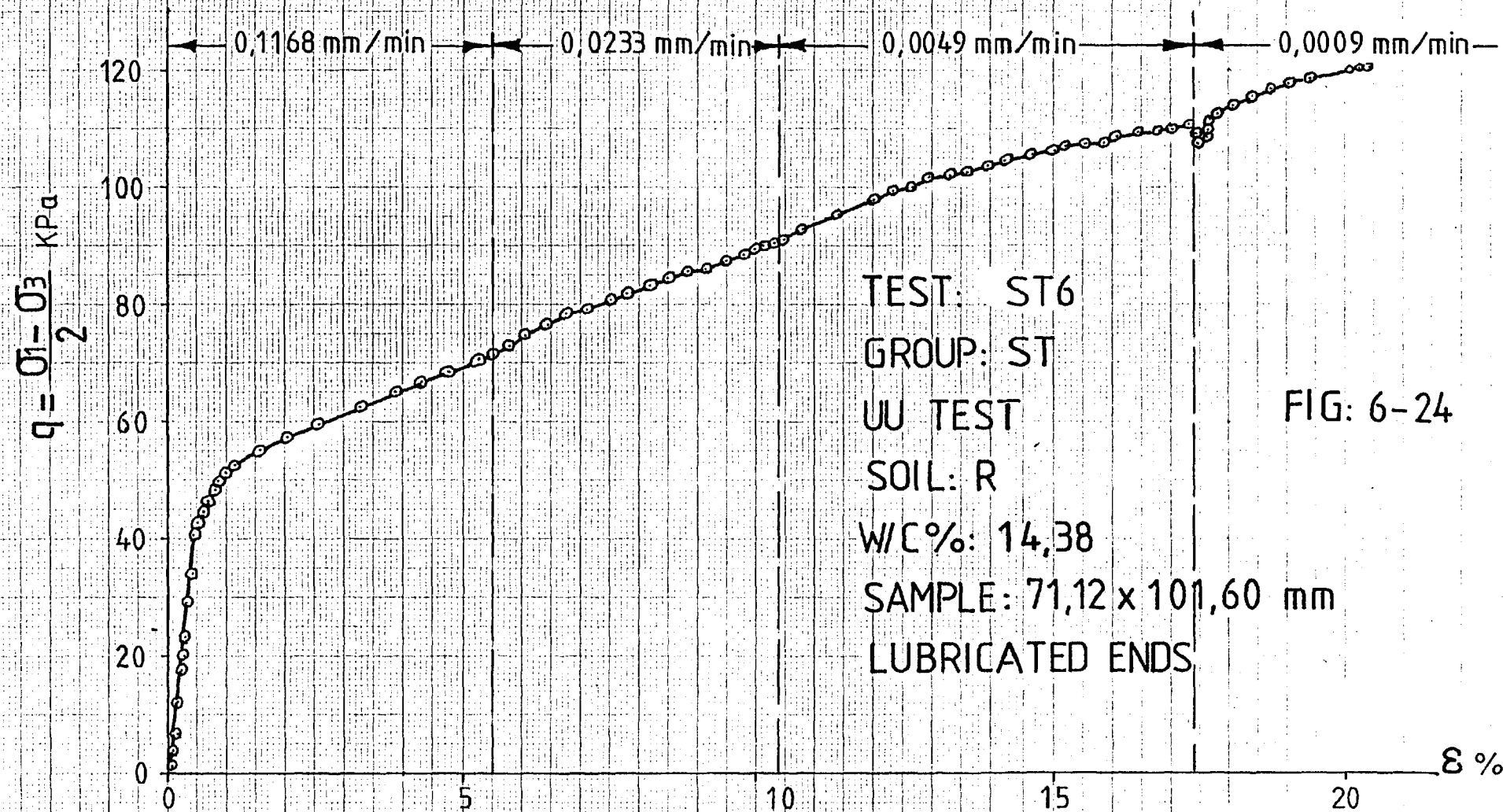


Fig.6-22 Effect of shearing rate (on wet & dry sides of the w/c - Cu plot) on remoulded samples.





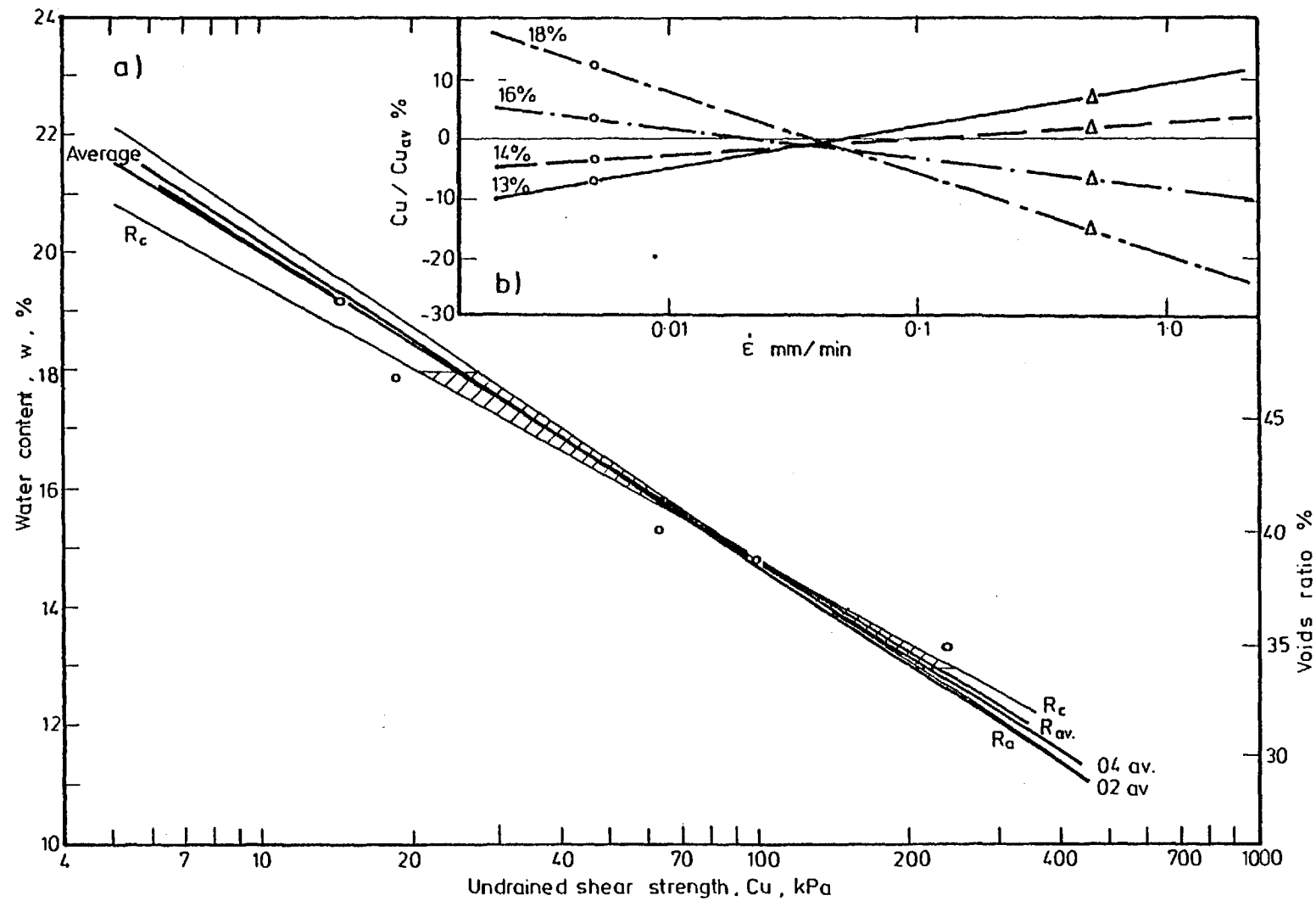


Fig. 6-25 The relationship between undrained shear strength and the final water content of resedimented samples from slurry of Happingburgh Lower Till.

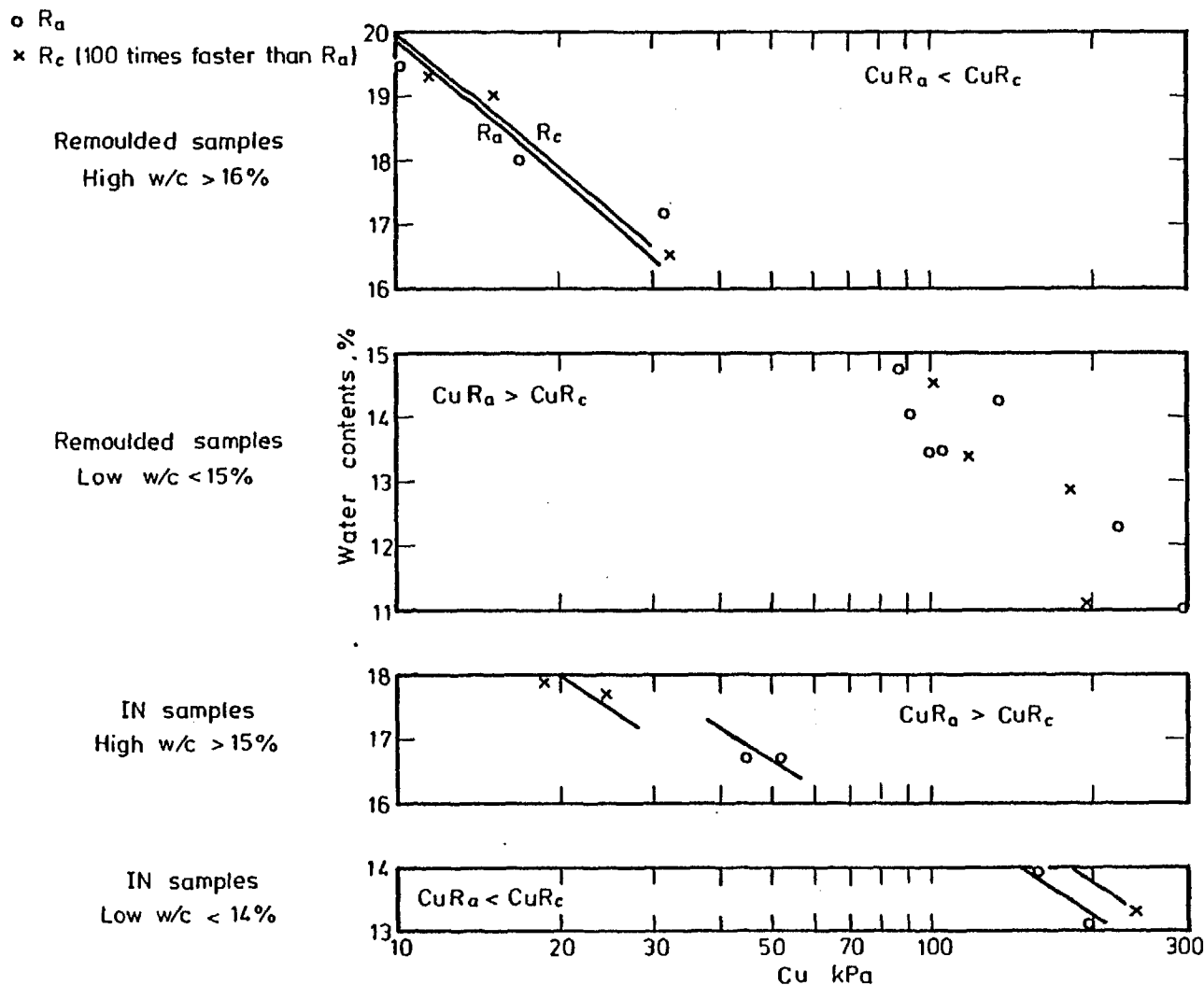


Fig.6-26 Water content % - Undrained shearing strength Cu kPa of remoulded isotropically normally consolidated samples of the Lower Till at 'high' and 'low' regions of water content.

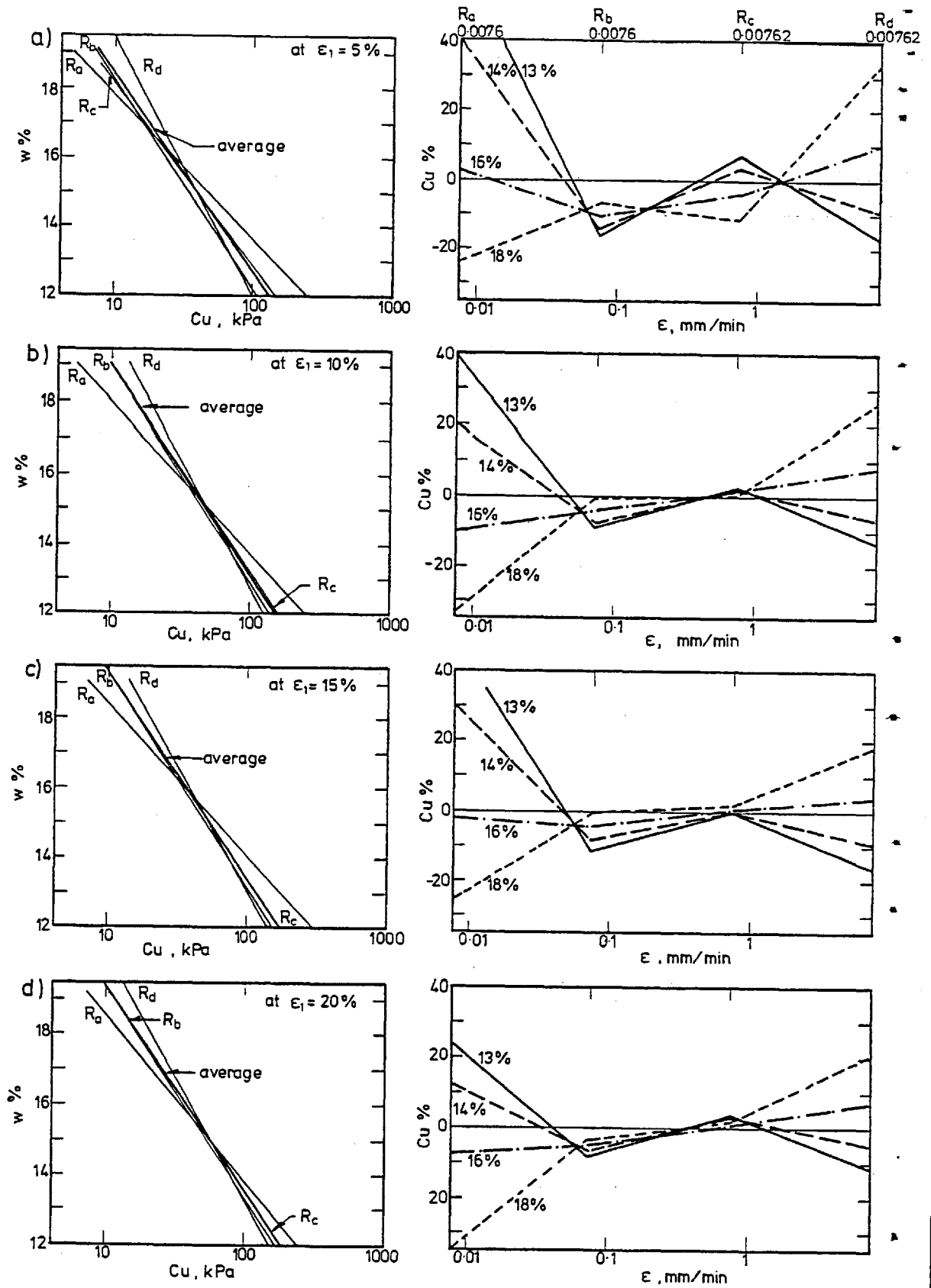


Fig. 6-27 Effect of strain rate at different strain levels (remoulded samples)

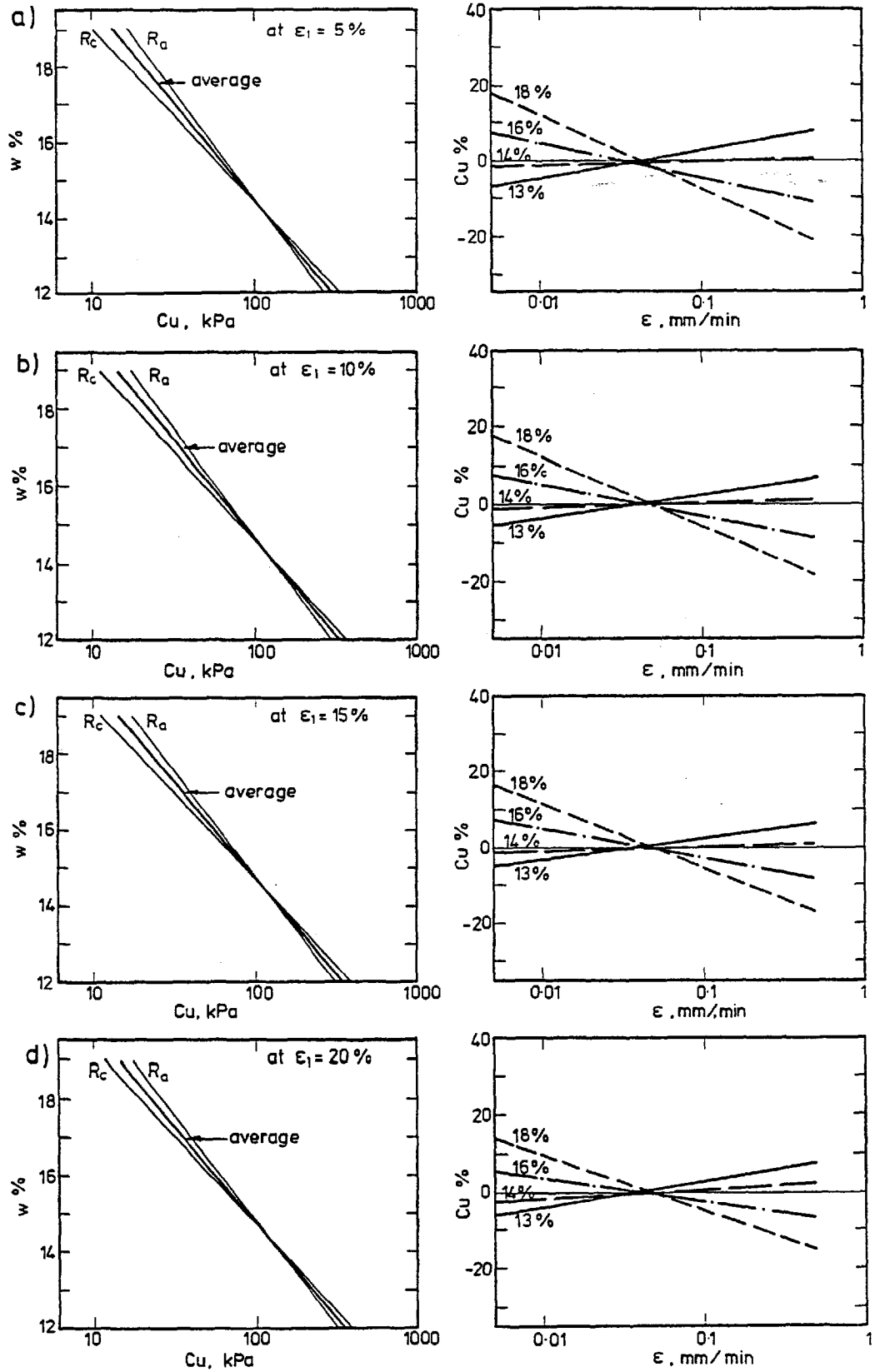


Fig.6-28 Effect of strain rate of different strain levels (N.C. samples) 7

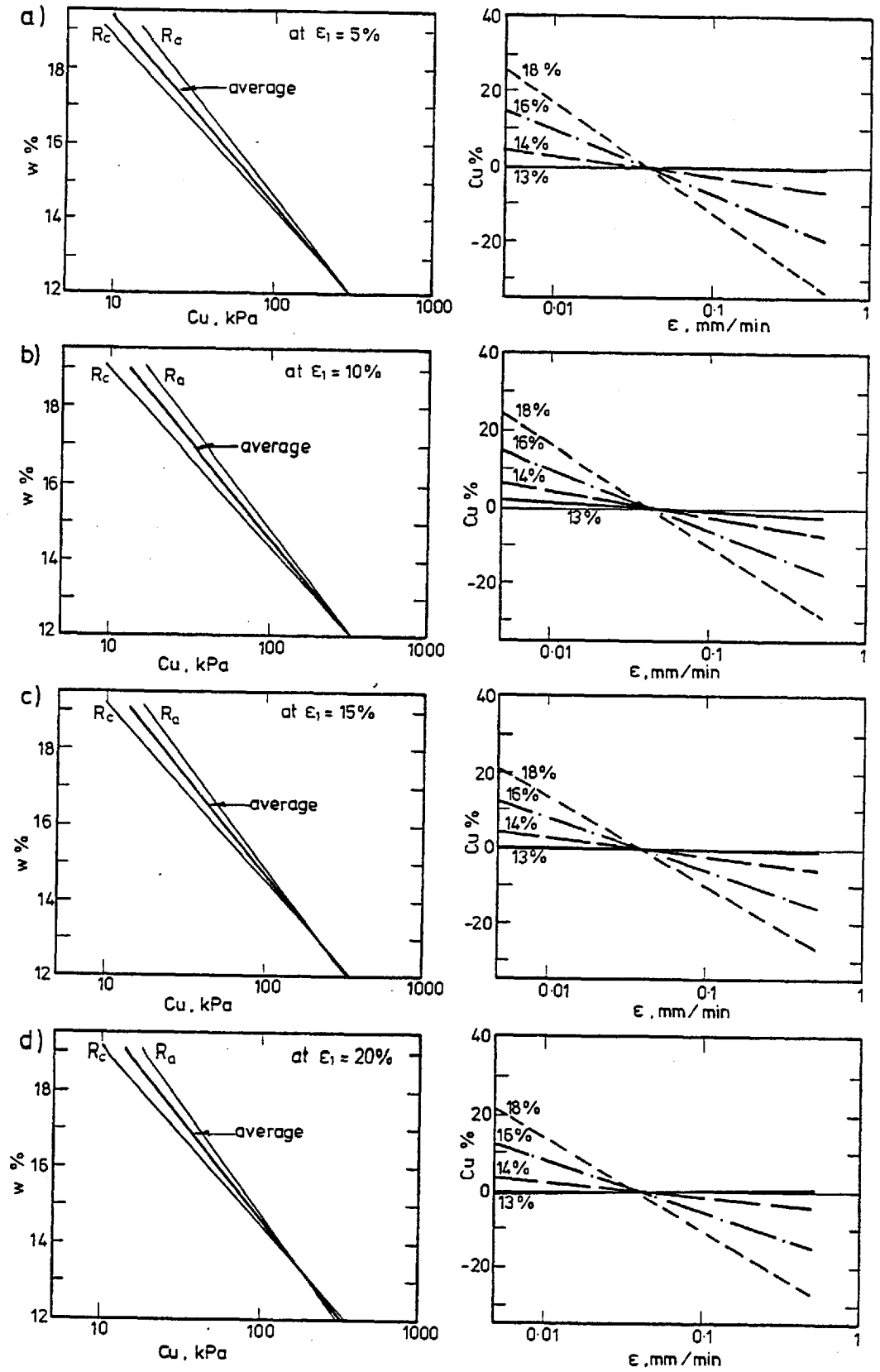


Fig. 6-29 Effect of strain rate at different strain levels (104 samples)

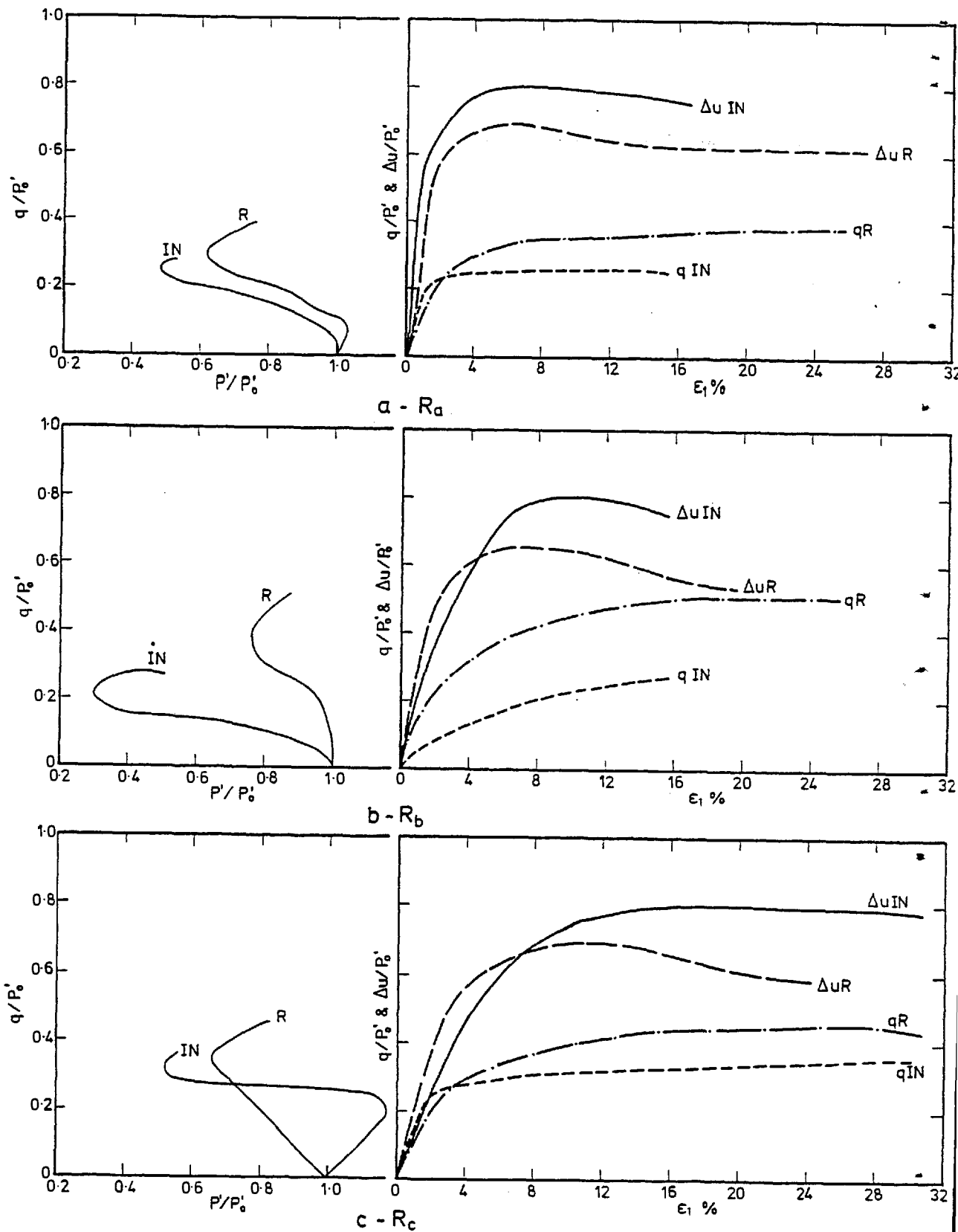


Fig.6-30 Effect of strain rate on relationships between different soil structures

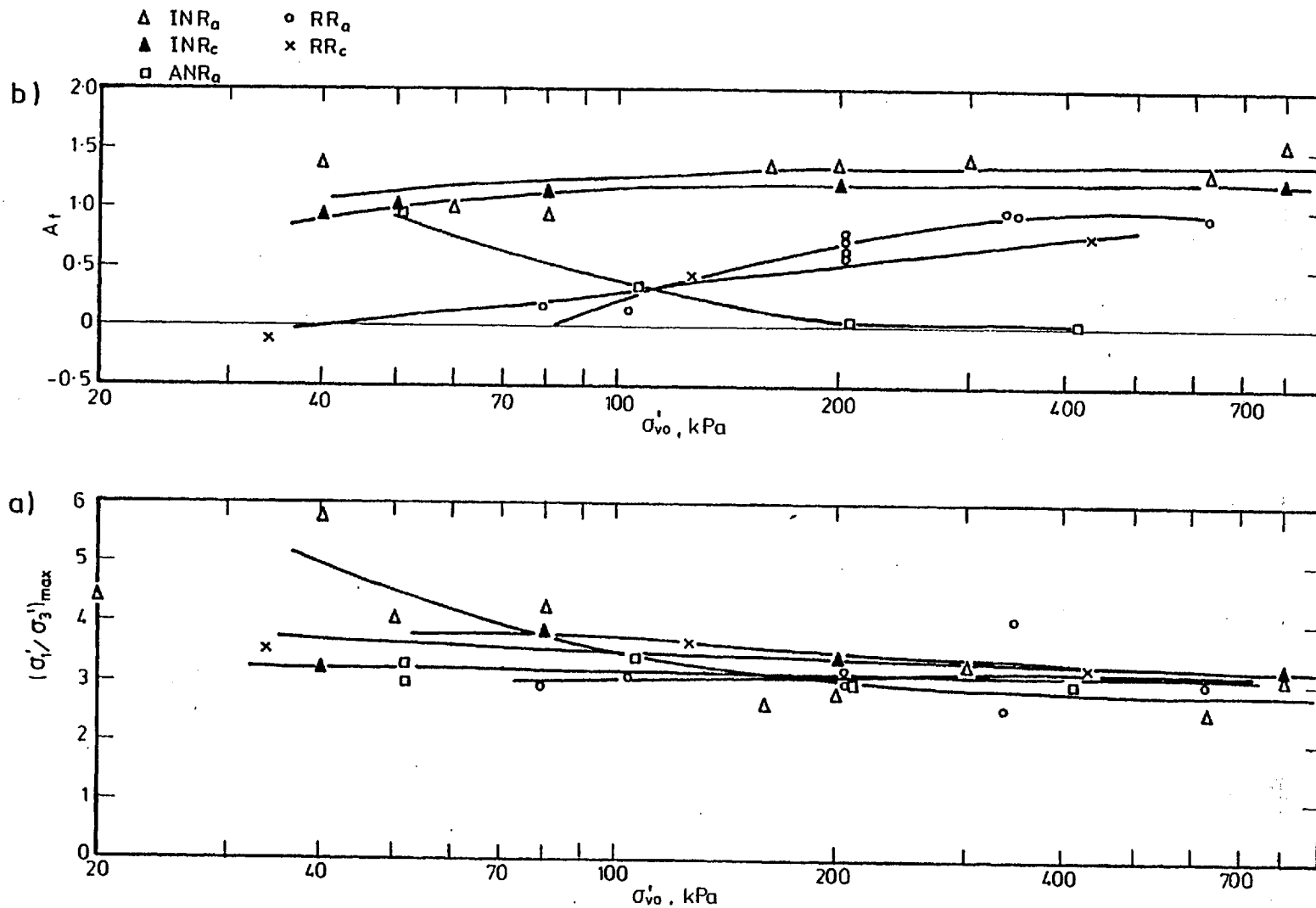


Fig. 6-31 The relationships between the consolidation pressure σ'_{vo} and the pore pressure parameter A at failure and the ratio of the effective stress (σ'_1/σ'_3) at failure for normally consolidated samples of 'Lower' Till.

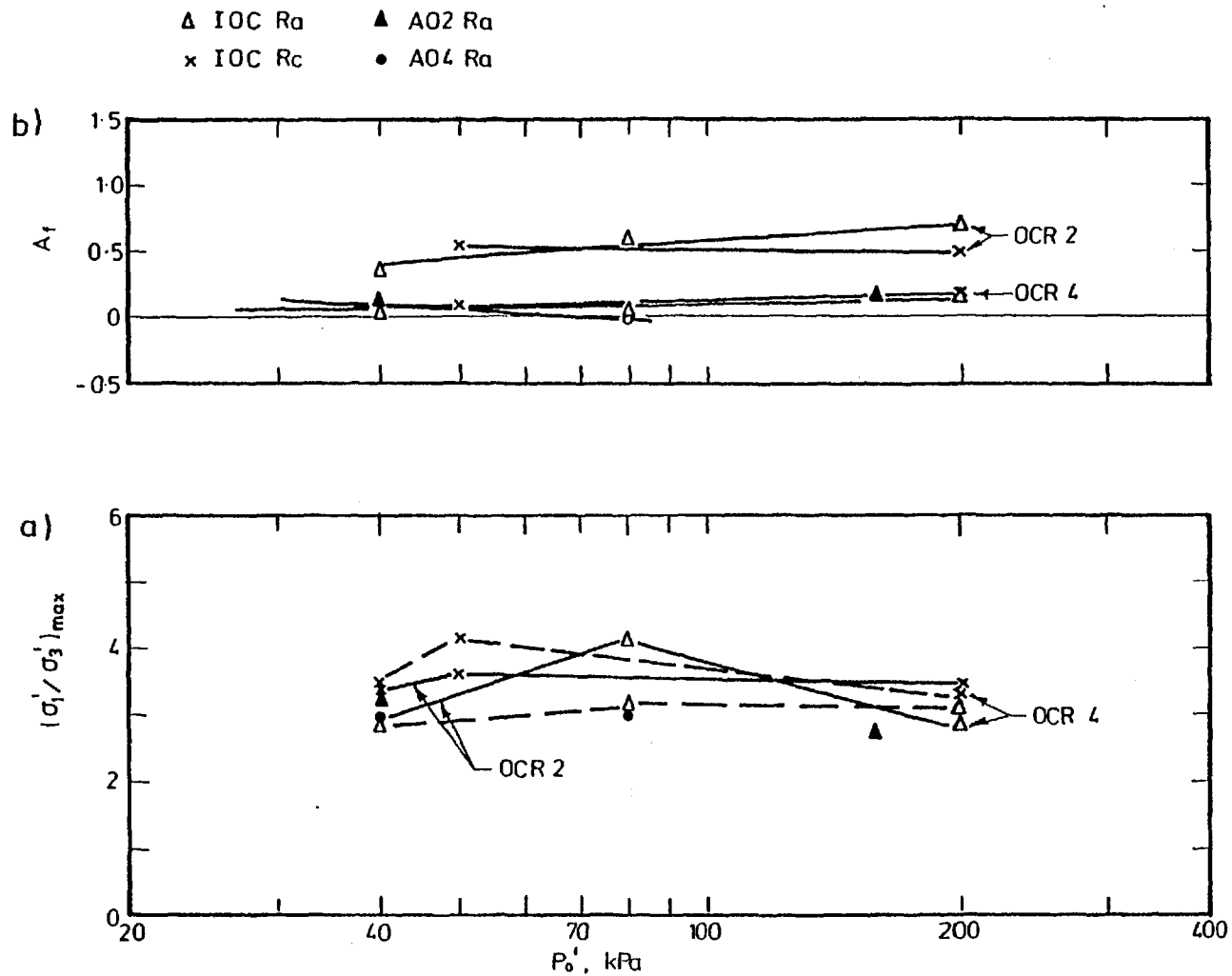


Fig. 6-32 The relationship between the consolidation pressure P_o' and the pore pressure parameter A at failure and the ratio of the effective stresses (σ_1'/σ_3') at failure for overconsolidated samples of Lower Till.

6.5 EFFECTS OF ANISOTROPIC CONSOLIDATION

6.5.1 General

Anisotropy in soils is considered to result from a combination of inherent anisotropy and system induced anisotropy (Hansen & Gibson 1949 and Ladd et.al. 1977). In this particular research a comparison will be made between samples prepared in the same way (resedimented from slurry), and then consolidated either isotropically or anisotropically to comparable states of stresses. The comparison is, therefore, restricted to the effect on compression behaviour of system induced anisotropy.

Different sample preparation methods for granular soils were reported to introduce anisotropy into the behaviour of the tested soils. Green (1969) & (1971) prepared sand samples by vibration and found that ϕ' for horizontal samples was less than that of vertical ones. Reades (1972) found exactly the opposite from the above but his samples were prepared by tamping. Arthur & Menzies (1972) prepared sand samples at different inclinations by their air deposition method and found that ϕ' of vertical samples was 2° greater than ϕ' of horizontal samples.

Restricting the comparison to the system induced anisotropy does not, in the author's opinion, alter the relevance of this study to the in-situ behaviour because the Lower Till is unsorted and unlaminated in nature (refer Chapter 2). Closely similar ESPs and deformation curves

were shown by different structures of the Lower Till when sheared after anisotropic consolidation to relatively high pressures (item 6.3.2). It is experimentally evident, however, from the thin petrological section of ring shear test RS2 (Fig.4.17), that the remoulded soil, although anisotropically loaded and unloaded to pressures comparable to the above ones, stayed unsorted and non-laminated after shearing. The only visible orientation was the vague positioning of the two largest particles which aligned their longer axis perpendicular to the direction of the major principal effective stress.

Unfortunately, a comparison between isotropic and anisotropic consolidation stress systems can not give a direct measure of the scale of the induced anisotropy.

Different methods have been used to study anisotropy in clay soils. Shearing the samples vertically and horizontally and comparing the results has been reported (e.g. Hansen & Gibson 1949). Testing, in cylindrical compression, samples cut at different inclinations has been widely reported (e.g. Lo 1965, Duncan & Seed 1966; Wesley 1975 and Bishop & Wesley 1978). Bishop (1965) has discussed the ambiguity in relating strengths to different planes for the aforementioned method. Analysis of compression and extension tests has also been used to study anisotropy through a difference between these two types of tests should be expected even if the soil was isotropic (Bishop & Eldin 1953; Parry 1960, 1971). In-situ vane tests were also used to

study anisotropy (Aas 1965, 1967) but care should be used when comparing their results with triaxial tests with regard to the different positions of the weakest planes in the ground and inside the cylindrical samples. Indirect methods of studying anisotropy by studying the difference in vertical and horizontal stiffnesses of the soil skeleton from the results of compression tests or steps in these have been best reported by Wesley (1975).

Direct comparison of isotropically and anisotropically prepared samples has been reported by (Ladd 1965, Parry & Nadarajah 1974, Bjerrum & Lo, 1963). In this research direct comparisons will be made because the experimental facilities allowed for such a comparison and also because this soil is sandy clay soil whose anisotropic behaviour is unknown. In a continuation of this work, Gens (1978) extended the investigation to examining samples cut at different inclinations.

In the following study on the effects of anisotropic consolidation, the stress history will be linked to the study by reporting the effects on each studied parameter in overconsolidated samples whenever possible.

6.5.2 Effect Of Anisotropic Consolidation On The Volume Change During Consolidation ($W-\sigma'_{v0}$ relationship).

The ranges of the water content - effective vertical stress ($W-\sigma'_v$) relationships, in which the isotropically and anisotropically consolidated triaxial samples of the Lower

Till lie, are shown in fig. (6.33). (compiled from Figs. 5.8 & 5.15).

The range of the anisotropically - consolidated samples is smaller than that of the isotropically - consolidated ones which is more apparent in the lower bounds. The isotropic system of stresses imposed on the sandy clay when consolidated from slurry does not allow for a unique normally-consolidated line. Samples starting with small differences in water content follow separate consolidation paths. However, the slopes of all normally-consolidated lines are very nearly the same and they all lie within the range shown. The anisotropically-consolidated samples, on the other hand, which start from "nearly" the same point merge in a unique normally-consolidated line at relatively small consolidation pressures. This convergence to a unique normally-consolidated path is helped by the shear component in the anisotropic stress system.

For practical purposes both isotropically and anisotropically normally consolidated samples could be assumed to follow roughly the same path especially when starting from a wet slurry, item 6-5-2. Parry & Nadarajah (1974) found that the isotropic normal-consolidated line lies above the anisotropic line for samples of Spestone kaolin consolidated from slurry.

The unloading-reloading loops in $W-\sigma'_v$ spaces are shown for isotropically-consolidated samples in Figs.(5-11):(5-14) and for anisotropically-consolidated samples in Figs.(5-1):(5-4). For all practical purposes both types of

consolidation give the same slope and shape of unloading and reloading loops.

6.5.3 Effect of Anisotropic Consolidation on the Effective Stress Paths.

1) Normally-consolidated samples: a process of normalising the results of anisotropically normally-consolidated samples has been carried out as explained in item 6-3. The results of this normalisation are shown in Fig.(6-34). The average normalised normally consolidated lines of both isotropic and anisotropic samples are shown in Fig.(6-35). The comparison has been made using the same major principal effective stress (σ'_{vo}).

It is shown that the yield surface created by the isotropic samples does not agree with the path followed by the anisotropic sample except at " failure". It is, therefore, not recommended to deduce pre-failure behaviour of anisotropically consolidated samples from isotropically-consolidated ones and vice-versa (see Ladd 1965). Henkel (1960) reported unique ESPs, for isotropic and anisotropic samples of remoulded Weald and London clays sheared at the same water content. Parry & Nadarajah (1974) reported unique ESPs, for isotropic and anisotropic samples of Spestone kaolin consolidated from slurry provided they shared the same consolidation history (OCR).

2) Over-consolidated samples: Comparison has been carried out between isotropic and anisotropic samples having the same OCR, in terms of vertical effective stresses, and the comparisons started from the common mean effective stresses. These comparisons are shown in Figs. (6-36): (6-38). In Fig.(6.36a) the ESPs of samples of OCR=2 show similar initial and final positions. They part in the intermediate stage of shearing. In Fig.(6-36c) the samples have OCR of 4 and their common P'_0 is 40 kPa which causes the ESP of the anisotropic sample to start from below the $q=0$ line. Overcoming this rotation, the anisotropic ESP only meets the isotropic ESP near "failure". Both configurations of ESPs are, however, quite similar. The point of starting with rotated principal planes as above disappears, in the comparison of ESPs. of the same OCR = 4 but with P'_0 of 80 kPa, the ESPs. Fig.(6-37a); show remarkable similarity. Fig. (6-37c) compares tests having maximum vertical consolidation pressure of 350 kPa and minimum vertical pressure of 50 kPa at the start of shearing. These values were especially chosen to simulate stress levels in the model used by Herbertson (1977) and being used by Martins (1978). The choice of these particular stresses caused the P'_0 of each test to differ from the other but the two ESPs still resemble each other in shape.

Overcomming the rotation of principal planes for samples with OCR = 10 Fig.(6-38a) was so large that the ESP of

the anisotropic samples did not merge with the isotropic ESP. It has to be noted that initial differences in water content and the subsequent differences in the $W - \sigma'_v$ relationships during consolidation (see Appendix 3 and item 6-5-2) will cause differences in the ultimate strength and tend to shift the comparable ESPs away from one another. Test 104P80 had a water content of 15.45% during shearing as compared to test AO4P80 which had a water content of 15.75% during shearing. These two tests had the most comparable water contents of all tests presented and they also showed the closest similarity.

6.5.4 Effect of Anisotropic Consolidation on the Pore Pressure Behaviour

1) Normally-consolidated samples: the anisotropically consolidated samples generate less pore pressures than the isotropically consolidated ones as shown in fig.(6-35b) (see Ladd 1965). Both isotropically and anisotropically consolidated samples have almost the same ultimate q/σ'_{v0} and owing to the difference in the positioning of each sample at the start of shearing a large compensation by the pore pressure generation is naturally provided to neutralise this difference. Bishop and Wesley (1973) attribute the anisotropic behaviour of Mucking clay to the anisotropy of pore pressure.

Bjerrum & Lo. (1963) proposed a formula to correlate pore pressures from isotropically CU and anisotropically CU tests (at maximum obliquity):

$$\Delta U_I = \frac{\Delta U_A}{\sigma'_{v0}} + (1-K) \quad (6-1)$$

where

ΔU_A = excess pore pressure in anisotropically CU test.

ΔU_I = excess pore pressure in isotropically CU test.

σ'_{v0} = vertical effective consolidation pressure pressure.

K = ratio of effective stresses σ'_3/σ'_1 during anisotropic consolidation.

Substituting a value of $\Delta U_A/\sigma'_{v0}$ of 0.3675 and K of 0.55 the resulting ΔU_I would be 0.8175 as compared to the measured 0.8050.

The values of the pore pressure parameter A at failure (A_f) of anisotropically consolidated samples are shown in Fig.(6-31b). They show rapid decrease to near zero as the consolidation pressure exceeds 200 kPa. (see Ladd 1965 & Ladd et.al. 1977 and item 6-7).

2) Overconsolidated Samples: the pore pressure behaviour of anisotropically and isotropically overconsolidated samples depend on the relative positions in q-p' space of each of the compared tests. Since the ESPs. of samples compared in the manner described in item 6-5-3 tend to coincide, the pore pressure behaviour of each case has to be studied separately and no generalisation is feasible (see Figs. 6-36 b & d, 6.37b & d and 6-38b).

The A_f of anisotropically overconsolidated samples are shown in Fig.(6-31b). The line joining the samples with OCR=2 lies horizontal and just above the $A_f=0$ line. The line joining the two tests with OCR=4 crosses the $A_f=0$ line and gives an average A_f of zero. The small positive values or the negative values of the pore pressures at failure could be observed in Figs. (6-36b & d), (6-37b & d) and (6-38b).

6-5-5 Effect of Anisotropic Consolidation on Deformations and Axial Strains at Failure.

1) Normally-consolidated samples: In Fig.(6.35b) it is observed that the deformation curve of isotropically consolidated samples is a strain-hardening non-brittle curve which gently reaches a maximum value at relatively large axial strains (15.00%). The deformation curve of the anisotropically consolidated samples showed a different trend. It reaches its peak at very small axial strains (0.05%) and undergoes strain-softening followed by a faint strain-hardening recovery until it reaches very high axial strains (up to 26% in some tests). The small failure strains of anisotropically normally consolidated samples as compared to isotropically normally consolidated ones have been observed by Ladd (1965) and Simons (1960) & (1963). For several normally consolidated clays, Bishop & Wesley (1978) for Mucking clay, and by Conlon & Isaacs (1970) for a Glacial-Lacustrine clay. It has to be emphasised that

predicting deformations from one type of test to the other is not feasible.

2) Overconsolidated samples: the above mentioned observations are valid for samples with $OCR = 2$ & 4 . They are not true, however, for samples with OCR of more than 4 (7 & 10), the comparison is difficult because of the problems in the initial positions as stated before.

6-5-6 Effect of Anisotropic Consolidation on
 ϕ' and (σ'_1/σ'_3) .

1) Normally-consolidated samples: anisotropically and isotropically-consolidated samples share the same effective stress parameter ϕ' , Fig.(6-35). Scholars such as Simons (1963) and Ladd (1965) stated the same fact but only in the case when failure is taken at (σ'_1/σ'_3) max. Maguire (1975) reported similar ϕ' if anisotropy is looked at in terms of drained strength. Sowa (1963), Henkel (1960), Bishop & Wesley (1978) and Sandroni (1977) all reported similar ϕ' for both stress systems as the author.

The maximum values for (σ'_1/σ'_3) for anisotropic samples are plotted in Fig.(6.31b), where they lie almost horizontal and near to the value of 3.00 . It is noted that the apparent cohesion that appears at low values of σ'_{vo} in case of isotropically consolidated samples, does not exist in low values of σ'_{vo} in case of anisotropically consolidated samples.

2) Overconsolidated samples: examination of individual ESPs. of anisotropically overconsolidated samples from Appendix 3 shows that most tests have more or less the same ϕ' . Fig.(6-31a) shows that the average line joining the $(\sigma'_1/\sigma'_3)_{\max}$ points coincide with the value of 3 ($\phi'=30^\circ$).

6-5.7 Effect of Anisotropic Consolidation on the Water Content-Strength Relationship.

1) Normally-consolidation samples: the average $W-C_u$ line of anisotropic samples is shown together with the average isotropic line Fig.(6-39). The anisotropic line lies above the isotropic one which means that for a given water content the anisotropic samples will give higher shear strength. This ties up with the fact that the upper and lower bounds of the anisotropic ($W-\sigma'_{vo}$) relationship, Fig. (6-33) both lie above the relevant bounds of the isotropic range. Hence, for average lines in ($W-\sigma'_{vo}$) relationship, the anisotropic line will give higher σ'_{vo} for the same W . Reference to Fig. 6-35(a) shows that a higher σ'_{vo} will yield a higher shear strength. Sowa (1963) reported similar observations on studying anisotropy of remoulded Weald clay.

2) Overconsolidated samples: the average line of water content-strength relationship is shown in Fig. (6-39b) together with average lines of isotropically overconsolidated samples of OCR = 2 & 4. The average anisotropic line coincides with the isotropic lines and the anisotropy shows no effect.

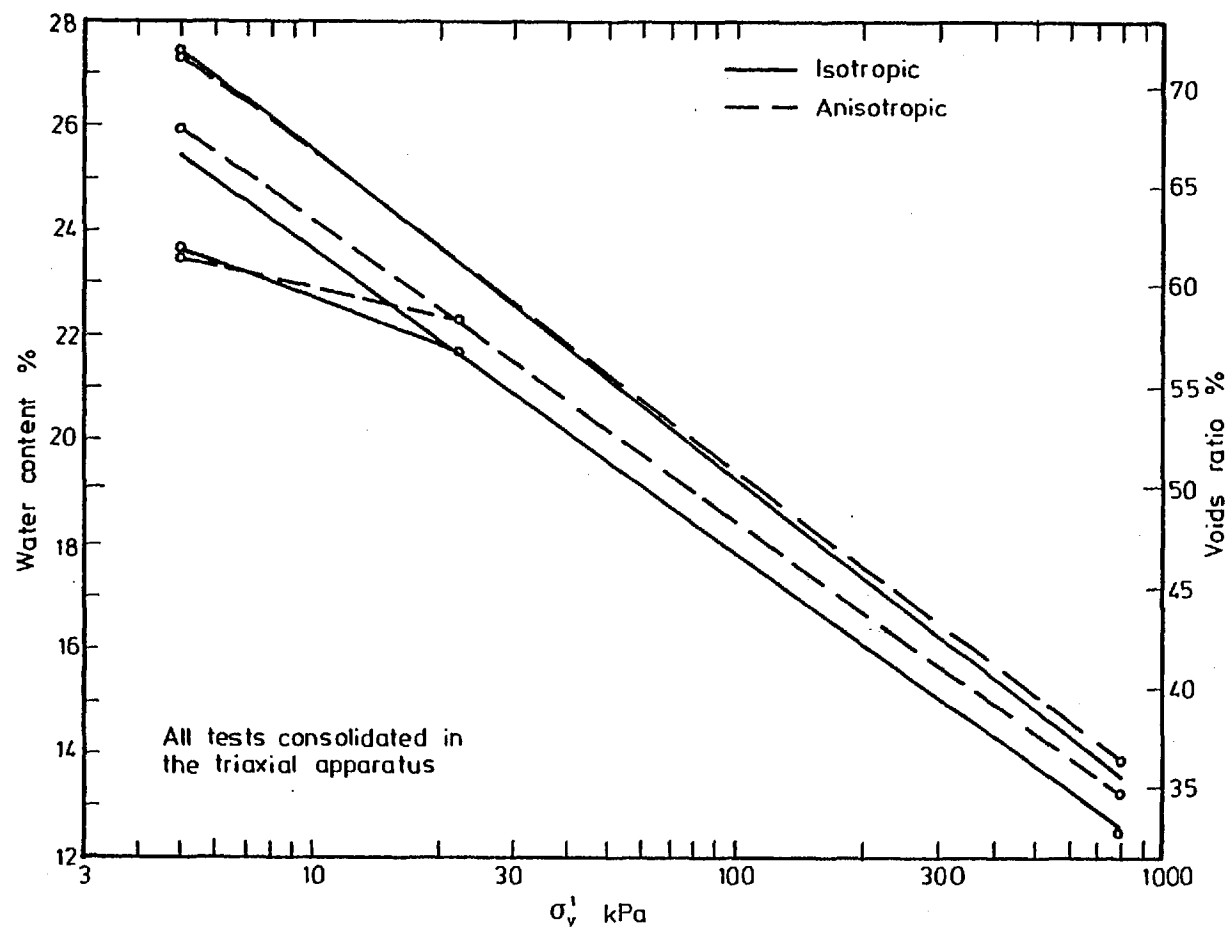


Fig. 6-33 Effect of anisotropic consolidation on the volume change during consolidation.

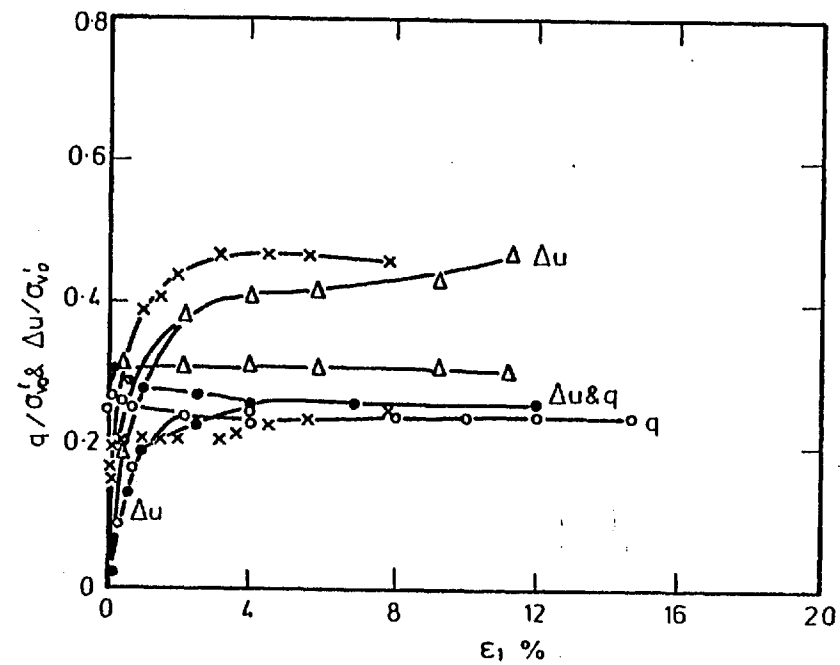
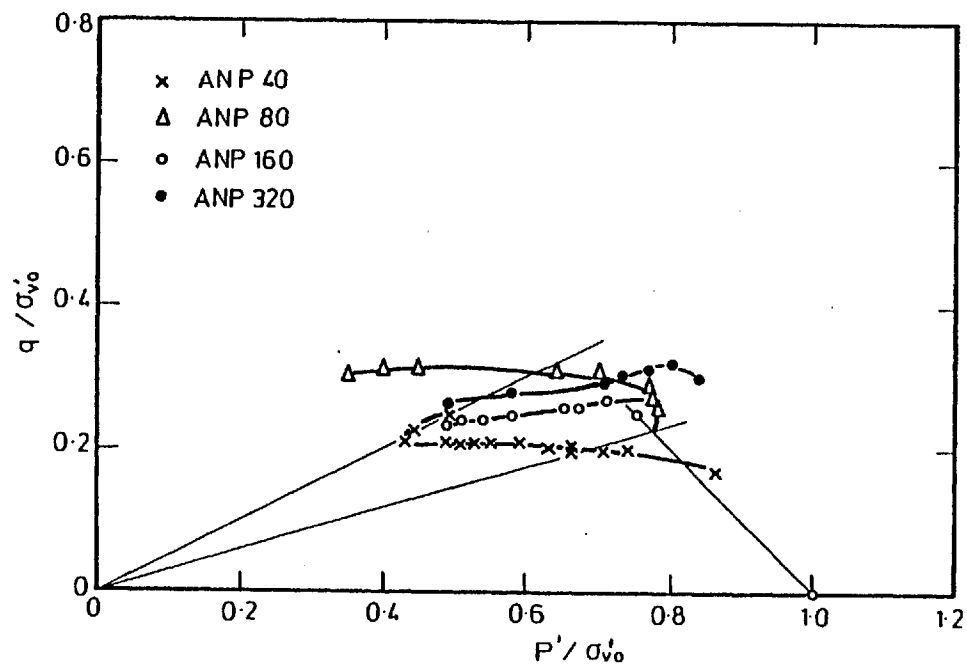


Fig.6-34 Normalized behaviour of anisotropically normally consolidated samples of Lower Till.

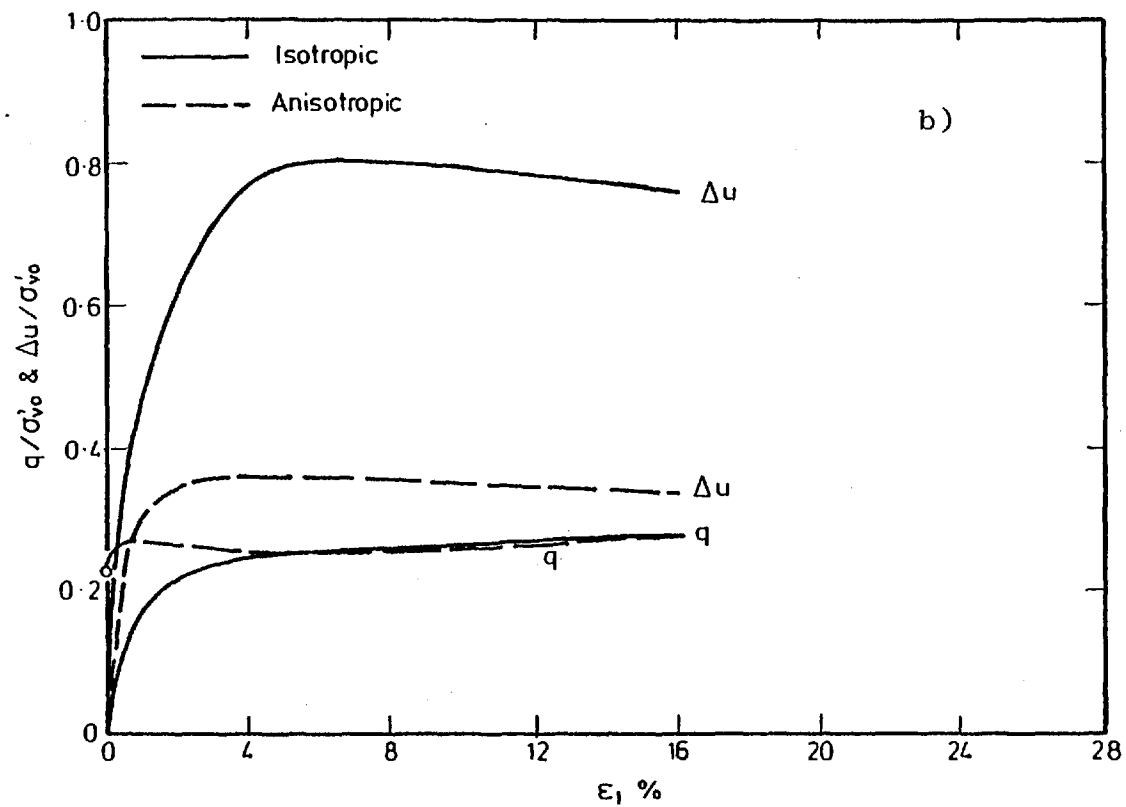
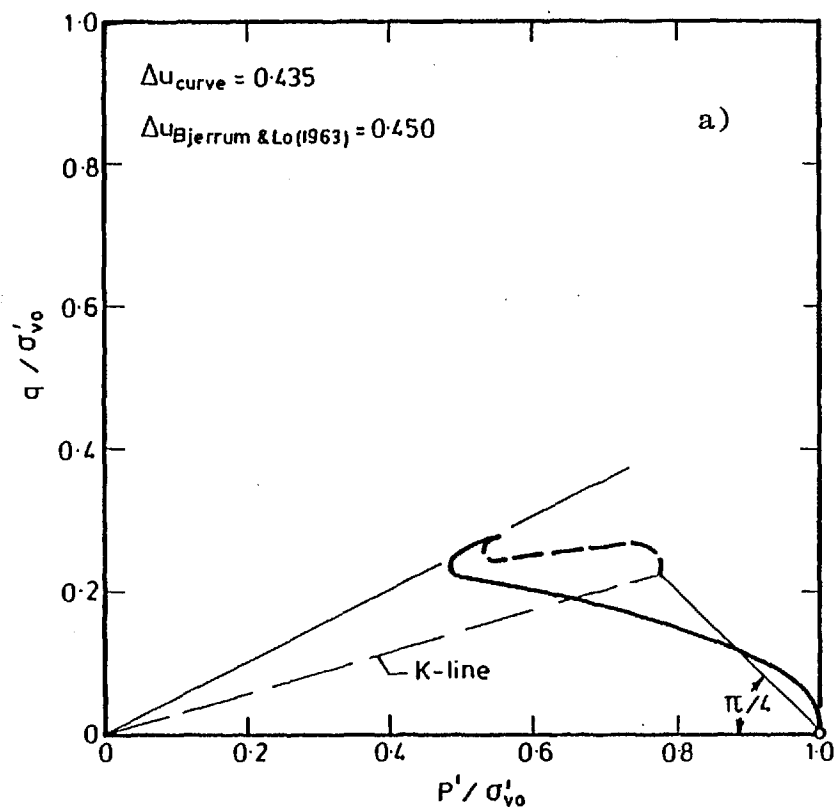
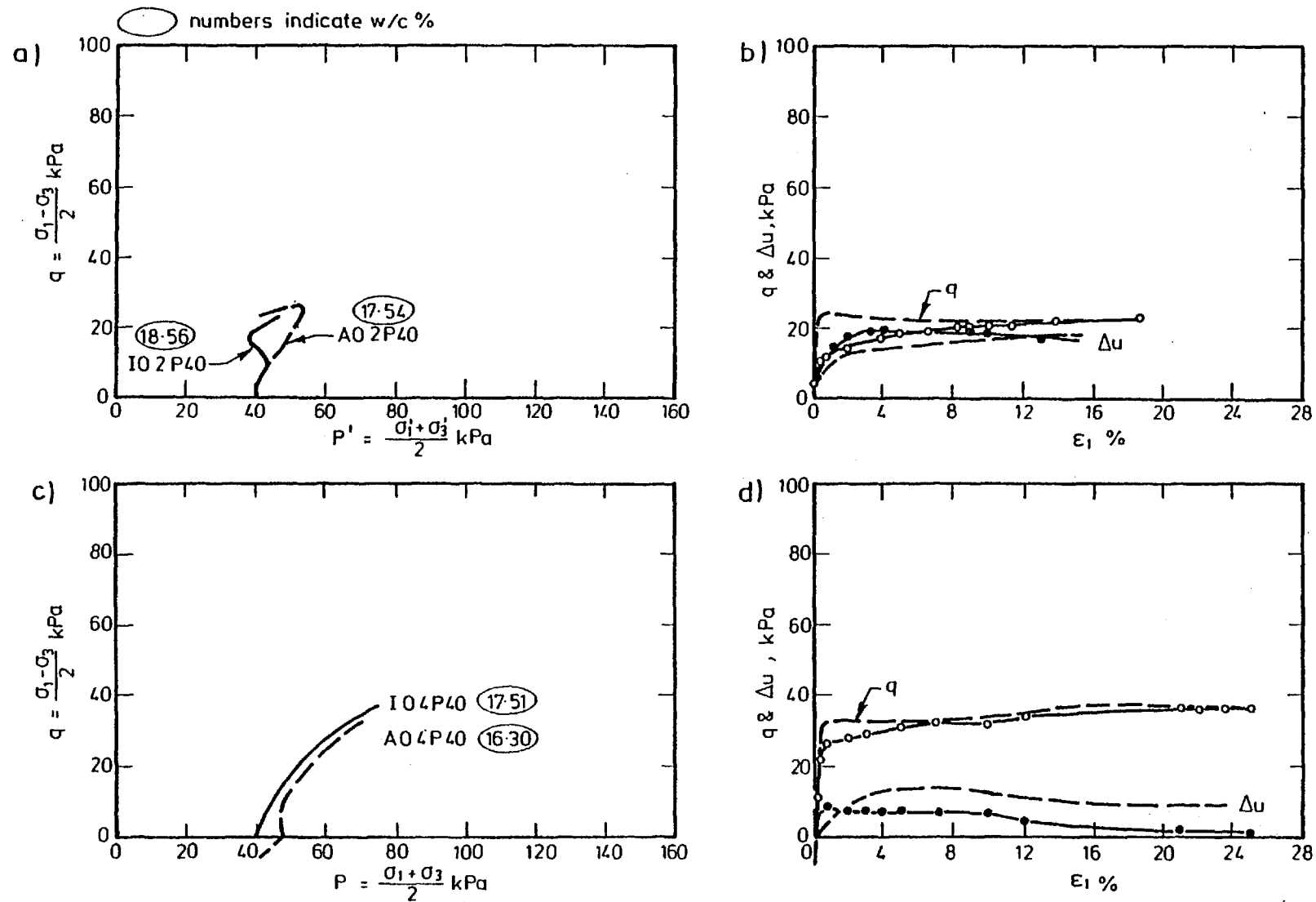


Fig. 6-35

Behaviour of isotropically and anisotropically normally consolidated samples of the Lower Till.



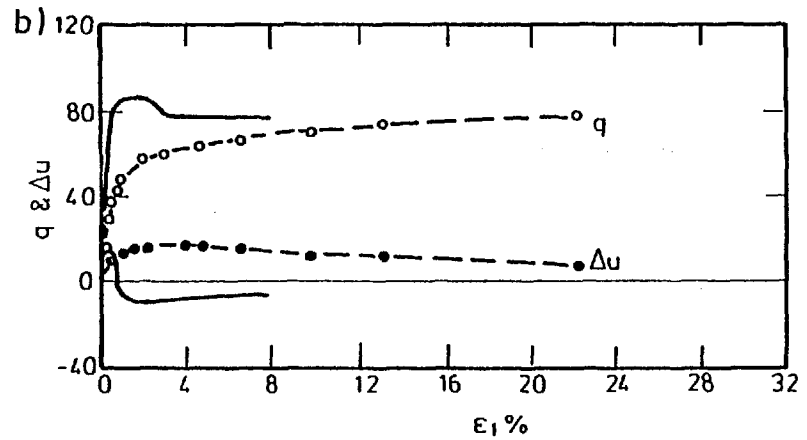
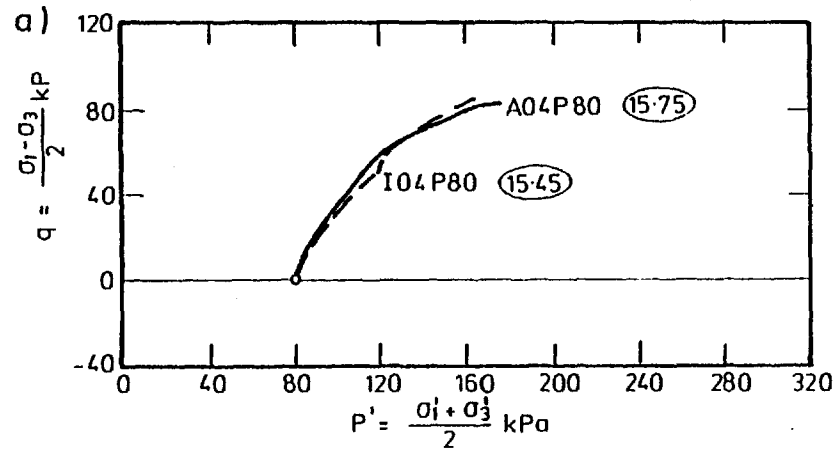
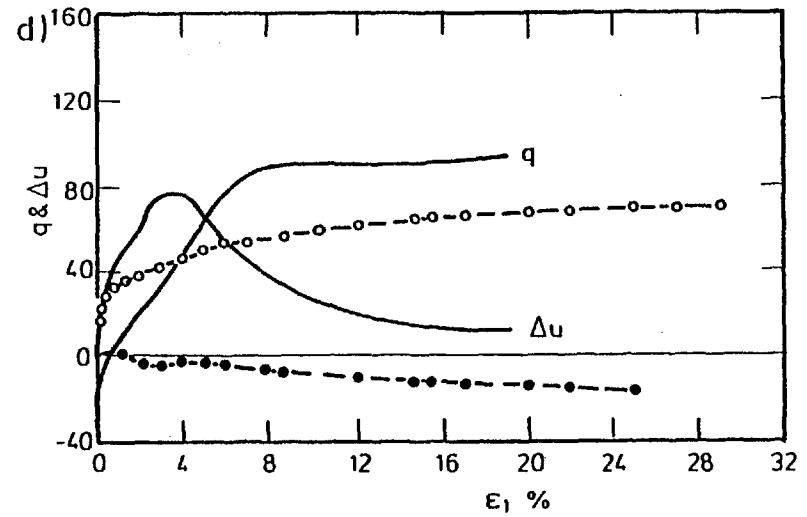
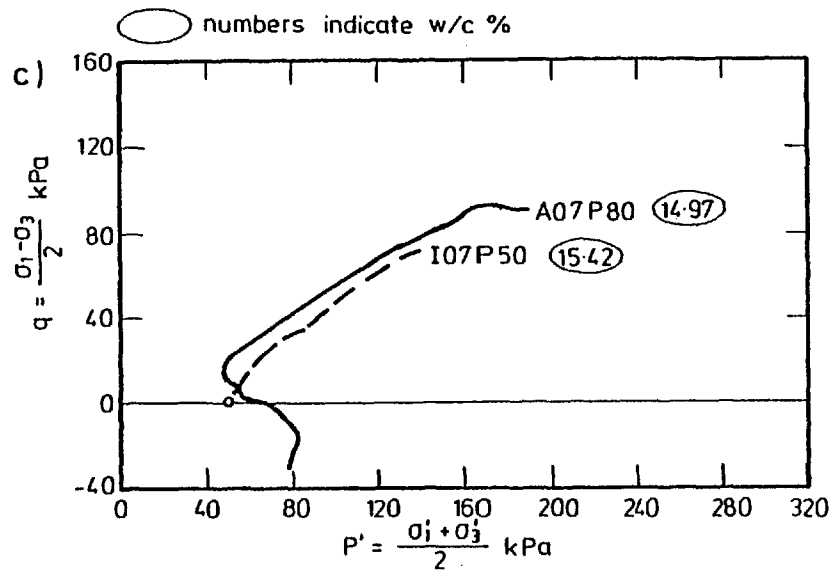


Fig. 6-37 Behaviour of isotropically and anisotropically overconsolidated samples of the Lower Till (OCR = 4.2 & 7),

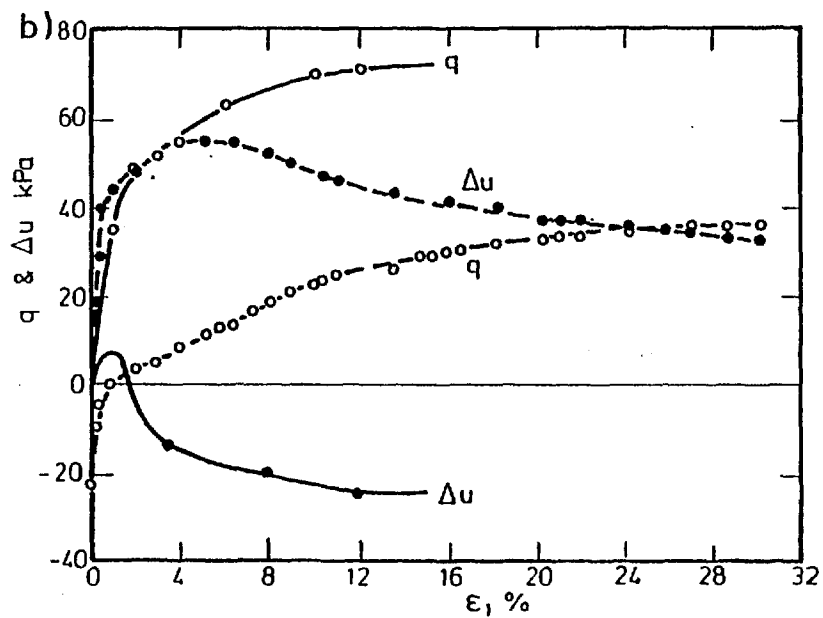
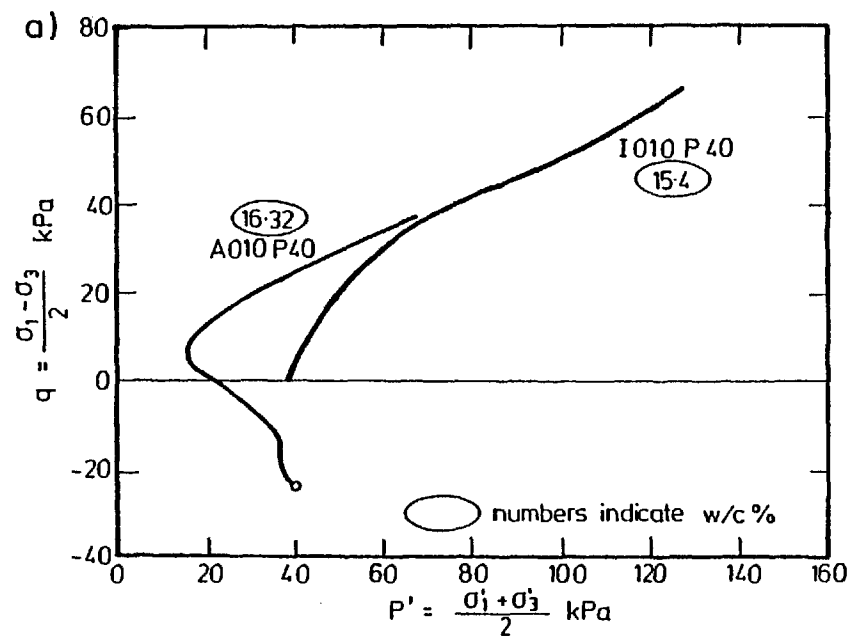


Fig. 6-38 Behaviour of isotropically and anisotropically overconsolidated samples of the Lower Till (OCR=10).

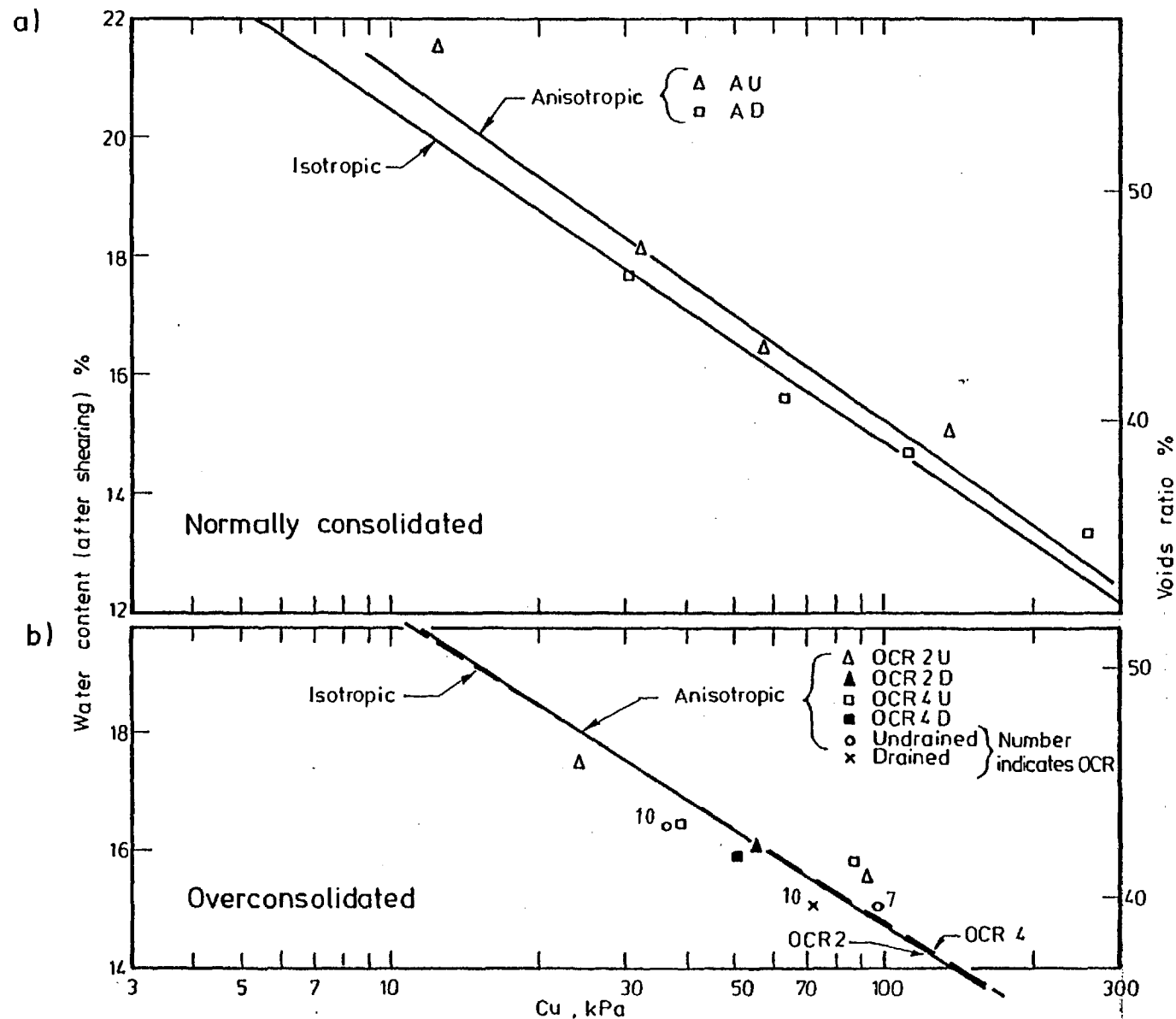


Fig. 6-39 Effect of anisotropic consolidation on the $w_0 - C_u$ relationships

6.6 SAMPLING DISTURBANCE

6.6.1 General

A group (SMP) of four tests was carried out to investigate the effect of sampling disturbance on the Lower Till under simulated field and laboratory conditions. All samples were anisotropically normally-consolidated from slurry to reach an assumed "in-situ" condition (see item 3-3 and Figs A3-116: A3-119). From the "in-situ" condition the samples were made to follow the following stress paths:

- 1) perfect sampling of
 - a) a "young" normally-consolidated sample, Fig.(A3-116).
 - b) an "aged normally-consolidated sample, Fig.(A3-119),
- 2) imperfect sampling, Fig. (A3-118),
- 3) perfect sampling followed by reconsolidation to a state of stresses approximating to the original in-situ stresses, Fig.(A3-117).

The above stress paths were followed, for all samples, by undrained shearing with a rate of 0.005 mm/min. (Ra).

The idea of such an investigation is not novel (Hvorslev 1949) and tests similar to some of the above have been carried out for more than a decade (Skempton & Sowa 1963 and Ladd & Lambe 1963). The important points in this particular investigation, however, are firstly that the soil is of low plasticity and this type of soil has not been

sufficiently studied in this context; secondly that the modern testing equipment and technique enabled the author to be in full control of the stresses and strains from the start to the finish of each test. (see item 4-3).

Unfortunately it was not possible to study the following points:

1) the effect of several sampling techniques on
i) the degree of sampling disturbance, ii) the relevance of laboratory data (Conlon & Isaacs 1970 and Apted 1977) and
iii) the discrepancies of parameters (e.g. water content, pore pressure distribution) within the samples (Hvorslev 1949, Lang 1967 & 1970, Sandroni 1977). The one technique used to extract block samples for this research is described in item 4-1.

2) the relationship between the sample size and the degree of sampling disturbance (Conslon & Isaacs 1970; Lo et.al. 1970 & Rowe 1970).

6.6.2 Discussion

Tests SMP1, Fig.(A3-116) and SMP4, Fig.(A3-119) represent perfect sampling for "young" and "aged" normally-consolidated samples, respectively. Comparing these tests with test ANP160 which represents the "in-situ" stresses, Fig.(A3-101) it is observed Fig.(6-40) that the maximum undrained shear strength (C_u) of test SMP1 is 55.31 kPa ($W_o = 16.70\%$) and of test ANP160 is 57.25 kPa ($W_o = 16.45\%$). Allowing the difference in W_o to compensate for the difference in C_u , both tests have practically equal undrained strengths.

(Sowa 1963 puts the difference between strengths to be 1:2%. Ladd et.al. 1977 puts it about 5% and Apted 1977 puts it between 0:5%). ESPs and deformation curves are, however, not comparable because they start shearing from two different positions in the $q-p'$ space and also because they reach their peak shear strength at small strains. It is to be noted that the stiffness of test SMP1 is 3.9 times that of an isotropically normally-consolidated test sheared from the same isotropic stress (the stiffness is taken as the shear stress at 1.0% axial strain and the isotropic test was calculated from Fig.6-5).

The "aged" normally-consolidated sample, test SMP4, when perfectly sampled and sheared from an isotropic state, Fig. (A3-119) had an identical value of pore pressure coefficient A (Skempton 1954) to that of the "young" sample. This is reflected in the same inclination of their ESPs. When normalising for the difference in the water contents in this range of C_u , Fig.(6-39), the "aged" sample will give a C_u of around 56.00 kPa which is almost exactly the same as that of the "young" sample.

The "aged" sample was stiffer than the "young" one and it showed marked recovery, i.e. strain hardening at fairly small strains, causing its ESP to differ in shape from that of the "young" sample. In both tests, the axial strain during shear unloading was small (less than 0.2%) and it is therefore, assumed that both samples did not lose the

structure gained during anisotropic consolidation. The differences in ESPs and deformation curves between these samples and those in isotropically normally-consolidated tests are attributed to this last remark.

In test SMP2, Fig.(A3-117), the sample was reconsolidated to a state of stresses slightly higher than the in-situ stresses to avoid sampling disturbance (Ladd & Foott 1974). The process of consolidating back to the field stresses has been proposed by many scholars (e.g. Berre & Bjerrum 1973, Wesley 1975) to bring about more similarity between the laboratory and field behaviour. This process inevitably involves a decrease in the "in-situ" water content and subsequent increase in the undrained shear strength C_u for sandy clays. In this research, however, it is possible to account for the difference in W_0 and C_u as described previously. The resulting C_u approximates to that of perfect sampling, but the stiffness is higher. The ESPs of this test, SMP2 and of the "in-situ" test ANP160 are similar in shape.

The test representing imperfect sampling, test SMP3, was allowed to swell, after shear unloading, to an isotropic stress equal to the mean effective stress of previous tests (160kPa) then sheared (undrained) Fig.(A3-118). The water content increased during the process of swelling and the pore pressure coefficient A was higher than that of the perfect sample. (test SMP1). The brittleness index I_B

(Bishop 1967 and item 5-3) of the imperfect sample was 8.5% as compared to 5.9% for perfect sampling.

6.6.3 Conclusions and Recommendations

In this low plasticity soil, the undrained shear strength depends to a large extent on the water content during shearing. This idea has been put forward by Vaughan (1971), Vaughan et.al.(1975) and Vaughan & Walbancke (1975). It has repeatedly been confirmed by laboratory experiments (e.g. Sodha (1974) and by the author in Figs.6-20, 6-21, 6-25 and 6-39).

In this type of soil a loss in the water content during the sampling, transportation, storage and trimming would drastically increase the laboratory measured undrained strength above the "in-situ" strength. This soil is also apt to dry up more quickly than the higher plasticity soils (see item 4-1). A change in the state of "in-situ" stresses will not alter the undrained strength as would be expected in more plastic soils (Wesley 1975, Maguire 1975, Apted 1977 and Sandroni 1977).

It is practically difficult to maintain both "in-situ" state of stresses and water content (Ladd & Lambe 1963) and it is recommended that in order to evaluate an undrained strength of a certain field condition of this soil, tests with water contents similar to those in the field should be carried out.

It is also observed from the results of this section that in order to evaluate the ESP of a certain field condition, the ESP of a sample sheared after being anisotropically consolidated from slurry or reconsolidated to the field stresses gives the best resemblance.

The "in situ" deformation characteristics are, however, difficult to simulate in the laboratory although some remarks were made about the relative deformation behaviour of the four cases presented. It is recommended that "in-situ" loading tests (e.g. plate loading tests, see Lo et.al. 1970) should be carried out and the deformation characteristics of the soil "en masse" be properly evaluated. A recommendation about the proper locations at which to carry out such "in-situ" tests has been put forward by the author in item 2-4.

The depth from which the samples are extracted, represented in the tests by the level of P' (or σ_v') does not affect the similarity between the values of the undrained strengths in the field and laboratory of this soil. In soils of higher plasticity, however, sampling depth (proportional to the magnitude of "in-situ" stresses) would strongly affect the measured strength. In these higher plasticity soils the structure stands a higher chance of being destroyed if the depth of sampling is great because the subsequent strains associated with large stress relief would also be great (see item 6.6.2).

The different aspects of the age of the normally-consolidated samples was considered in this section (item 6.6.2), and will be further discussed in section 6-9, but no attempt has been made to study the sampling of the over-consolidated soils. It is noticed from the results of this group of tests especially SMP1 and SMP4 that in the case of undrained shearing preceded by undrained shear unloading, the effective principal stress ratio was higher than that normally observed. This point is studied and enlarged upon by Hight (1978).

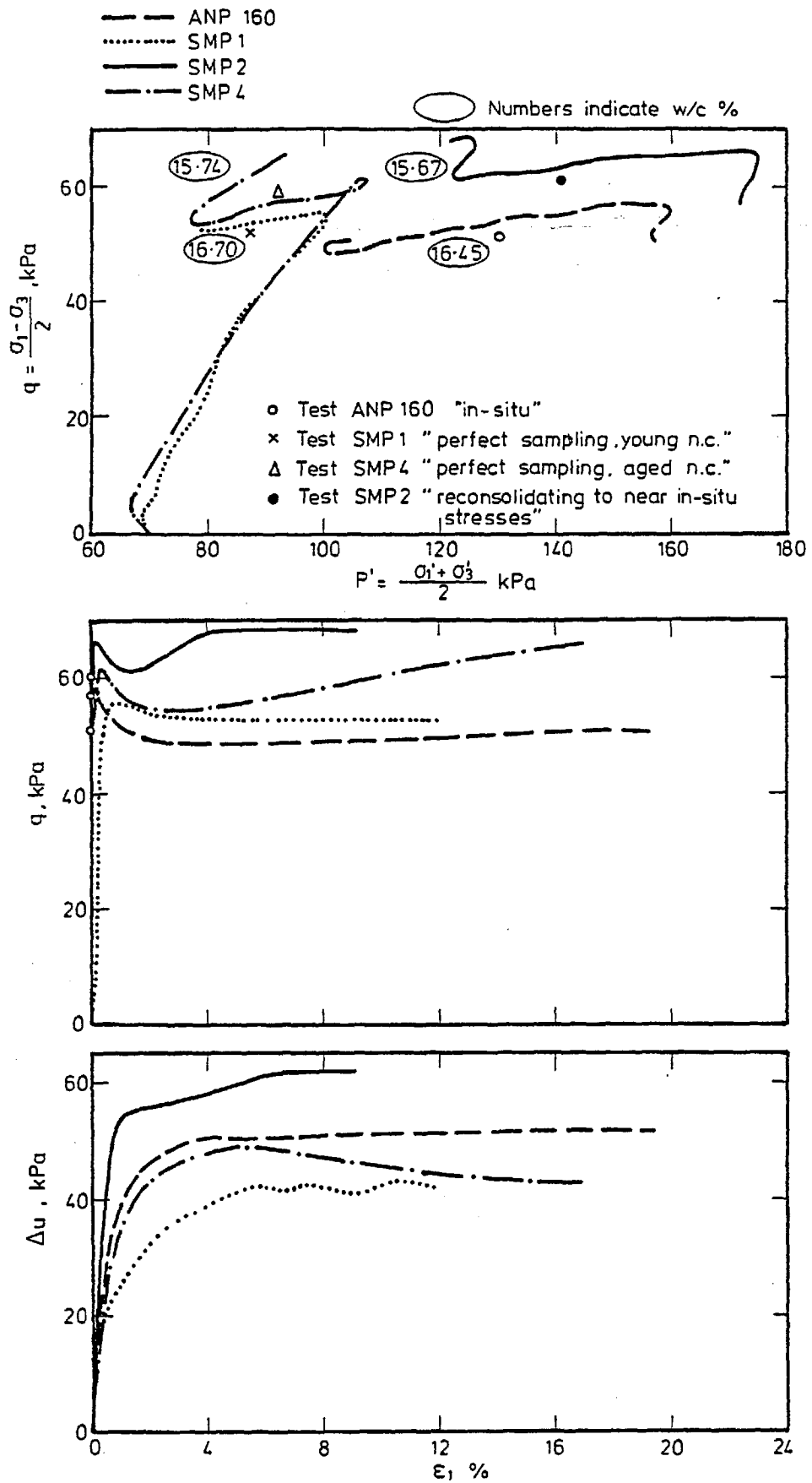


Fig. 6-40 Behaviour of the Lower Till during sampling.

6.7 EFFECT OF STRESS HISTORY

6.7.1 General

The effects of stress history on the Lower Till are studied by observing the comparable trends of behaviour of normally-consolidated and overconsolidated samples with different overconsolidation ratios (OCR). Some of these effects have already been studied in preceding sections.

In section 6-3, the effect of stress history on the influence of soil structure has been studied. The effects of stress history on the various aspects of strain rate and anisotropic consolidation have been studied in items (6-4-2): (6-4-6) and (6-5-2): (6-5-7).

In this section a study is made on samples in which only the stress history is different. All samples are sheared at a constant rate of strain of 0.005 mm/min. (Ra). Anisotropic and isotropic samples are treated separately.

To compare ESPs and deformation curves of samples with different stress history, P'_o (or σ'_{vo}) is taken as a datum for comparison. The OCR is calculated as the ratio between the maximum effective vertical consolidation pressure σ'_{vm} and the effective vertical consolidation pressure at the start of shearing σ'_{vo} (see item 3-2-7).

6.7.2 Anisotropic Samples

Anisotropic samples of different histories (OCR) and sheared from the same P'_o have different values of σ'_{vm}

and σ'_{vo} . In Fig.(6-41) four samples are compared, their OCR ranged from unity to 10. They all have a common P'_O at 40 kPa.

The ESPs of samples of OCR=1:4 seem to share approximately the same failure envelope whereas the sample with an OCR=10 has a failure envelope which lies above that for N.C. samples and those of lower OCR. The deformation curves of samples with OCR=1:4 have the same shape (Fig. (6-41b). A typical elastic-plastic behaviour was shared by these three samples with a small strain-hardening recovery in the sample with OCR=4 and consequently the lowest W_O . Vaughan (1971) reported that, for similar soils, the strain-hardening recovery will increase with the decrease in water content. The pore pressures show corresponding behaviour with the magnitude of pore pressures generated decreasing as the OCR increases (Fig.6-41c).

The sample with an OCR=10 started shearing with its major principal horizontal ($q = -37$ kPa, $P'_O = 40$ kPa). During undrained shearing, the major principal stress switches to the vertical. It is probable that this interchange of principal stress directions gives rise to the large pore pressure generation in this test; other factors may contribute to this large pore pressure, namely the experimental problems this test creates (clearance in the extension top cap and the gap in the load cell) and the deviation from the K_O state during swelling to this high OCR.

In Fig.(6-42a) average values of all tested anisotropic samples are plotted in such a way as to facilitate and generalise the relationship between the shear strength and the OCR (Ladd et.al.1977). The same idea is utilised once more in Fig.(6-43a) to establish the relationship between the ratio of undrained strengths of normally and overconsolidated samples and the OCR. Both anisotropic curves of Figs. (6-42a) and (6-43a) give simple relationships.

Fig.(6-42b) gives the relationship between the pore pressure coefficient A at failure (A_f) and the OCR. This relationship is shown in two ways. Firstly for the average values of all tests, similar to the ideas of the above figures. Secondly for the tests which were specifically studied in Fig.(6-41), i.e. those with a common P'_0 of 40 kPa. It is shown that the values of A_f vary quite largely from one test to the other depending on their relative positions in the q - P' space. The A_f -OCR relationship should, thus, be studied cautiously for each individual case.

The axial strains at maximum shear stresses for anisotropically consolidated samples (ϵ_f) are small (see results in Appendix 3), and care should be exercised during testing especially at the very early stages of shearing where "failure" is apt to occur. This fact also helps in explaining the ambiguity of calculating A_f and might cause some of the large variation in the values of this parameter.

6.7.3 Isotropic Samples

In this section the normalised curves (item 6-3) are made use of and samples with OCR=1:20 can be readily examined. The information obtainable from Figs.(6-41 & 6-43) satisfactorily describes the effect of stress history on the isotropically consolidated Lower Till.

In Fig.(6-44a) it is observed that the ESPs of samples with OCR up to 10, share the same failure envelope with an effective stress parameter $\phi' = 30^\circ$. The ESP of the samples with OCR = 20 climbed above the envelope established by the samples with lesser OCR. After an average axial straining of about 8.5%, the ESP descends to touch a failure envelope of an inclination approximately equal to the one giving a ϕ' of 30° .

The deformation curves, Fig.(6-44b) show steady increases in shear strength and stiffness as the OCR increases. The pore pressure curves of Fig.(6-44c) show compatible behaviour with the above, with the pore pressure generation appropriately decreasing with increasing OCR. All deformation curves show non-brittle behaviour and the relative stiffness are plotted against the OCR in Fig.(6-43b).

The compatible trends of all deformation and pore pressure curves lead to values of A_f decreasing steadily with OCR as shown in Fig.(6-42b). The ratios of maximum $(\sigma_1 - \sigma_3)$ to σ'_{VO} (σ'_{VO} equals P'_O in isotropic consolidation) are shown

in Fig.(6-42a) and the ratios between overconsolidated and normally-consolidated undrained strengths and the OCR are shown in Fig.(6-43a). For the same OCR, the anisotropically-consolidated samples will give higher ratio of strengths than the isotropically-consolidated ones. This conforms with the conclusions previously reached (item 6-5).

- ① = ANP 40 ④ = A0 4 P 40
 ② = A02 P 40 ⑩ = A0 10 P 40

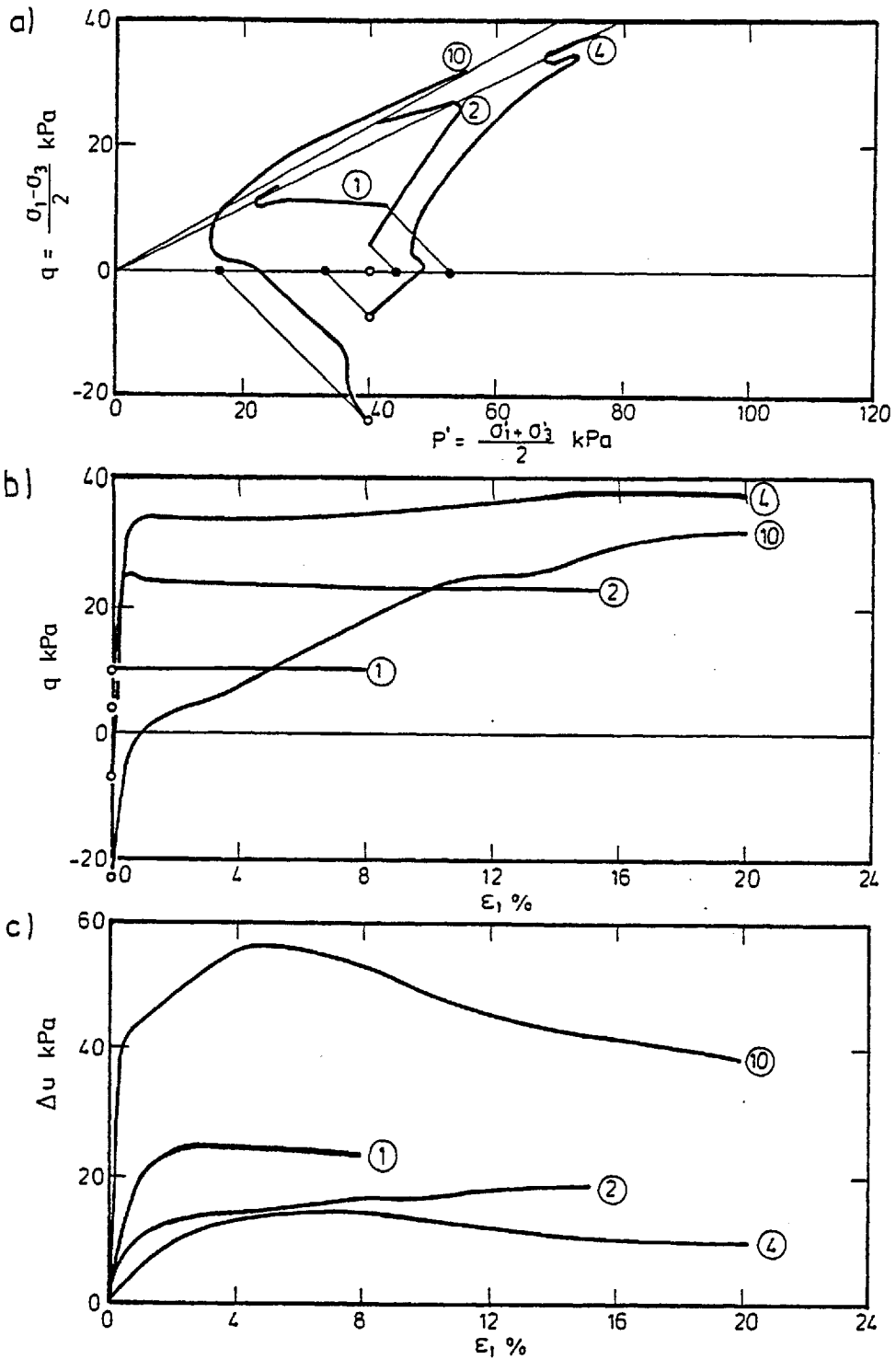
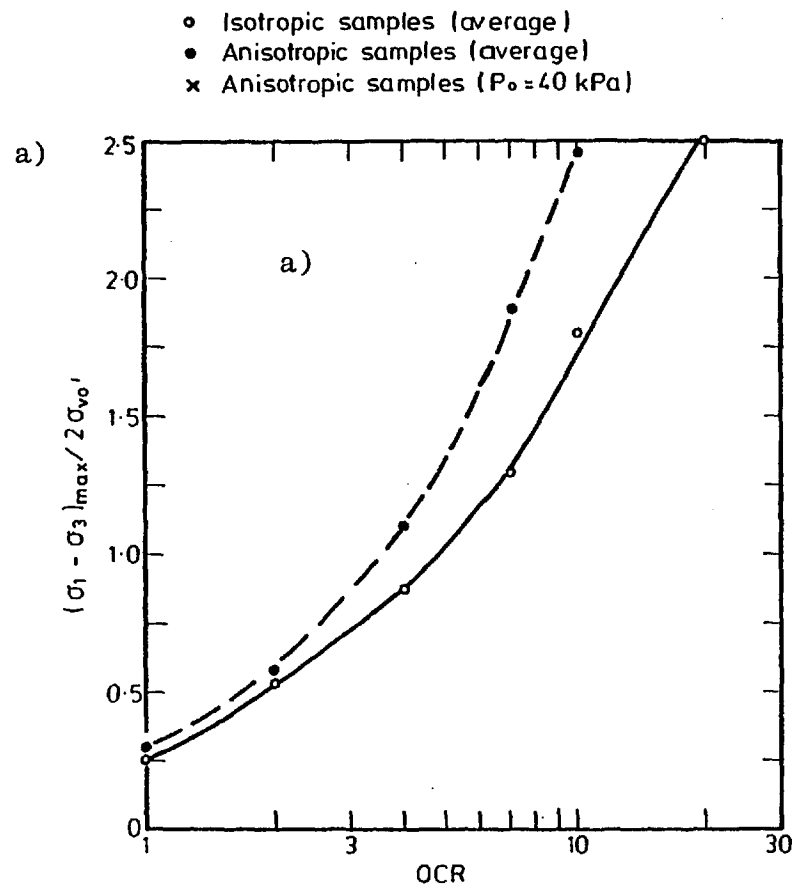


Fig. 6-41 Behaviour of anisotropically consolidated samples
 at $P'_0 = 40$ kPa & $OCR = 1:10$



$\epsilon_{1 \max} \pm 15\%$ for isotropic samples
 $\epsilon_{1 \max} \pm 20\%$ for anisotropic samples

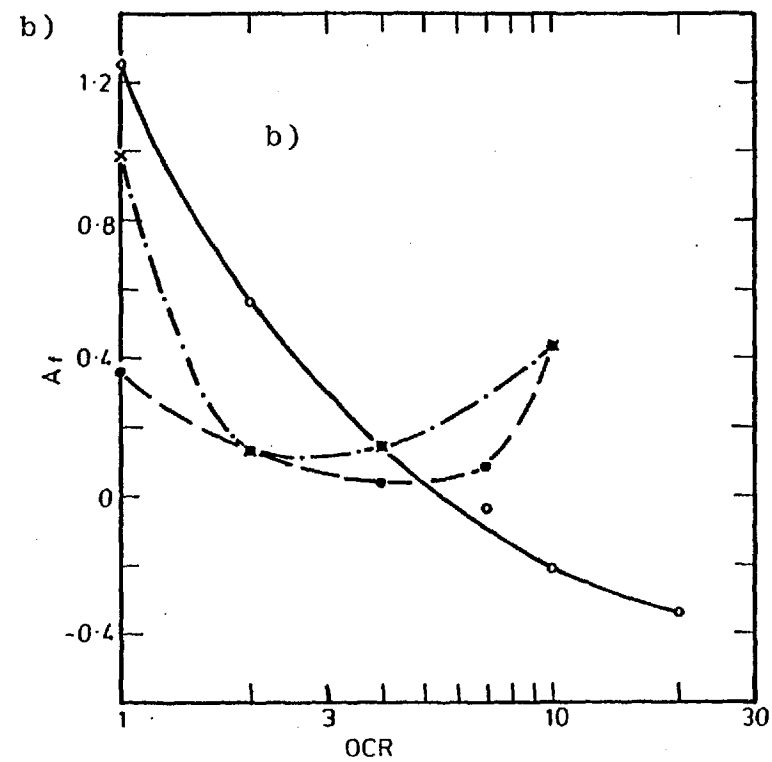


Fig. 6-42 Relationship of mobilized shear strength (at $\epsilon_1 = 15\%$) and the pore pressure coefficient A at failure (A_f) with the overconsolidation ratio of isotropically consolidated samples.

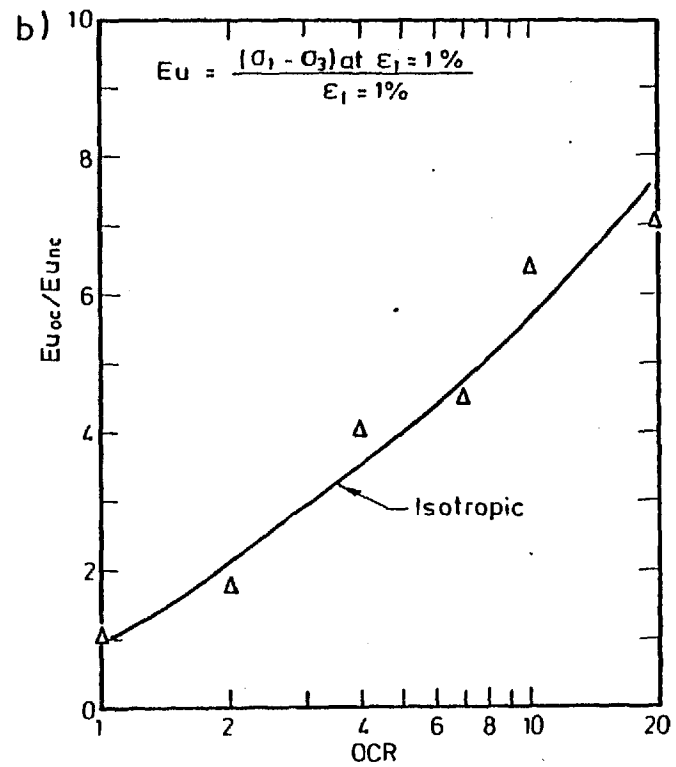
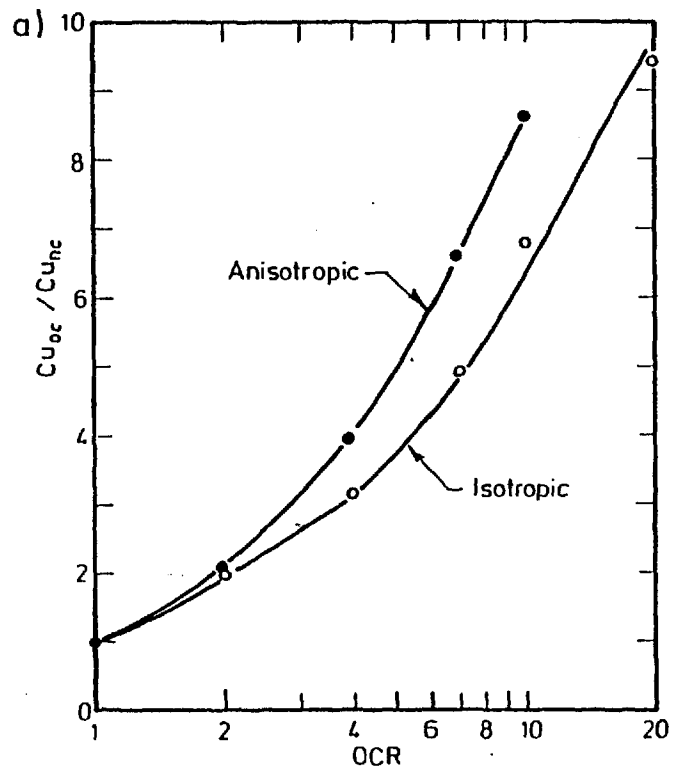


Fig. 6-43 Relative increases in undrained strength and stiffness with OCR in the Lower Till.

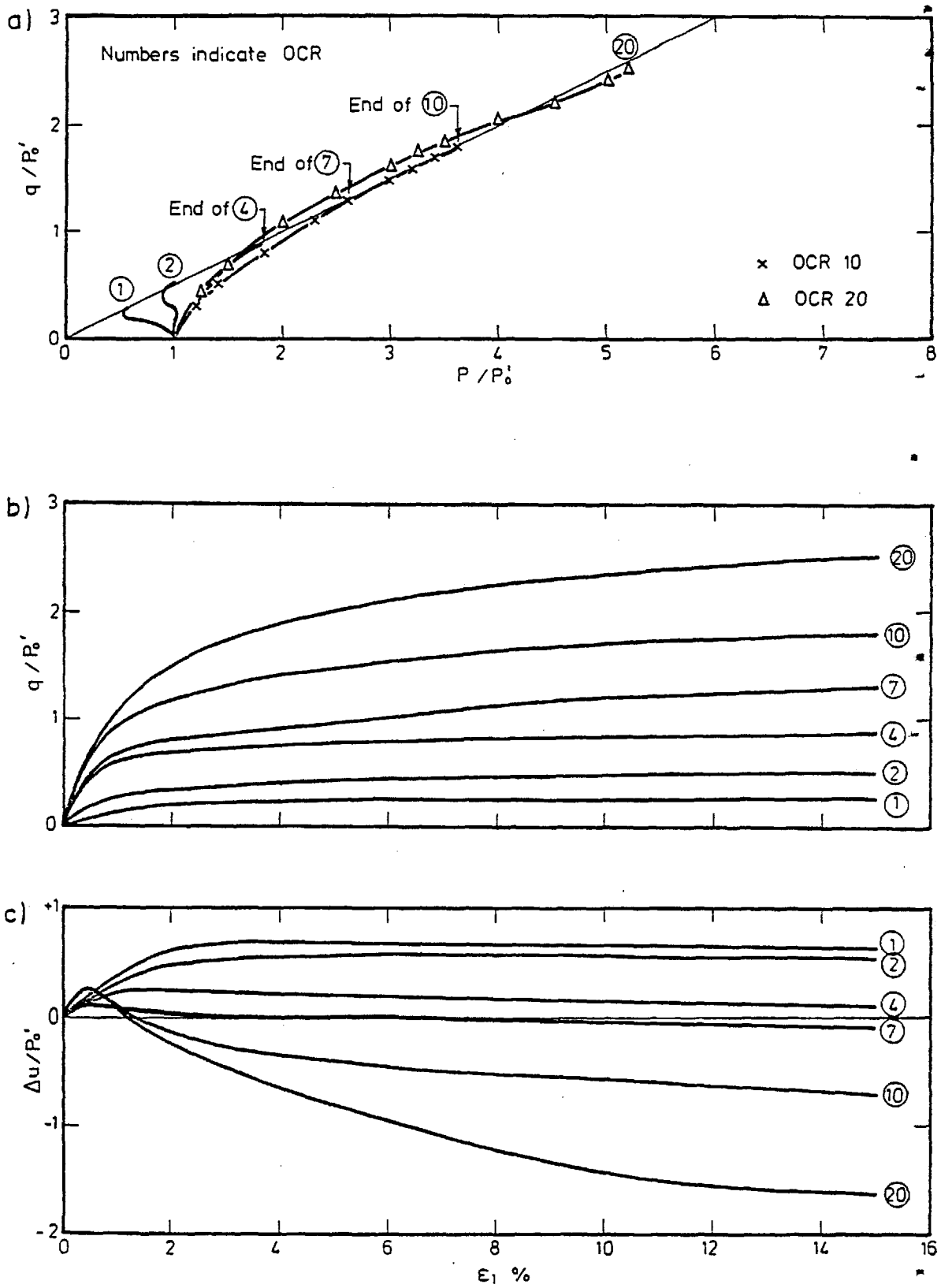


Fig. 6-44 Normalized behaviour of isotropically consolidated samples with OCR = 1 : 20.

6.8 STRESS-STRAIN RESPONSE OF THE LOWER TILL

6.8.1 General

Many of the triaxial tests on the Lower Till in this research have shown non-brittle (i.e. strain-hardening) stress-strain behaviour. The ultimate shear strength has thus been reached (or judged to have been reached) at large strains (more than 20% axial strain). At these large strains the interpretation of the triaxial test results is questionable, mainly because of the large area correction which assumes that the sample remains a right cylinder. The failure mechanism in these sandy clays was bulging except in tests following stress path SP2.

The ability to "estimate" what the ultimate strength would be from early stages of the test, while the strains are small and the area correction is minimal, seemed a desirable goal. This prompted the author to study this problem in the light of what had been done initially by Kondner and his co-authors (e.g. Kondner 1963 and Kondner and Zelask 1963) and later modified and used in finite element analysis by Kulhawy et al. (1969) and Duncan and Chang (1970).

6.8.2 The Presentation of a Non-Linear Stress-Strain Curve as a Hyperbola

The stress-strain relation was developed as part of an attempt by Kondner (1963a and 1963b) to formulate a generalised stress-strain-time relationship for soils.

He suggested that this generalised formulation should contain a conventional failure criteria which would appear in the mathematical limit of the stress as the strains get excessive. He later concluded that such a generalised formulation would be built on one or two-dimensional rheologic situations. Thus it was important to consider simple non-linear mathematical forms which reasonably represented stress-strain behaviour in soils over a large range of response. From his previous research Kondner (loc. cit.) chose the two-constant hyperbolic form of mathematical relationship for stress-strain.

A rectangular hyperbola passing through the origin of the stress σ ($= \sigma_1 - \sigma_3$), strain ϵ (e.g. $\epsilon_1\%$) in a two-dimensional space is shown in Fig. (6.45a). The asymptotes of the hyperbola are represented by the two lines:

$$\epsilon + \alpha = 0 \quad (6.1)$$

and

$$\sigma - \beta = 0 \quad (6.2)$$

The equation of the hyperbola is:-

$$\epsilon\sigma - \beta\epsilon + \alpha\sigma = 0 \quad (6.3)$$

Dividing equation (6.3) by σ and rearranging:

$$\frac{\epsilon}{\sigma} = a + b\epsilon \quad (6.4)$$

where

$$a = \frac{\alpha}{\beta} \quad \text{and} \quad b = \frac{1}{\beta}$$

Equation (6.4) is represented by the straight line of Fig. (6.45b) where the intersection with the $\frac{\epsilon}{\sigma}$ axis represents a and the slope of the line represents b . Rearranging equation (6.4):

$$\sigma = \frac{\epsilon}{a + b\epsilon} \quad (6.5)$$

The ultimate value of σ (σ_{ult}) is obtained by taking the limit of equation (6.5) as ϵ gets very large:

$$\sigma_{ult} = (\sigma_1 - \sigma_3)_{ult} = \lim_{\epsilon \rightarrow \infty} \sigma = \frac{1}{b} \quad (6.6)$$

Differentiating equation (6.5) with respect to ϵ and evaluating the derivative at $\epsilon = 0$

$$\left(\frac{d\sigma}{d\epsilon} \right)_{\epsilon=0} = \frac{1}{a} = E_i \quad (6.7)$$

where E_i is the initial tangent modulus. Kondner further tried to obtain explicit forms for a and b as functions of the various parameters of certain sets of tests (e.g. OCR, σ_3 and $\dot{\epsilon}$ for clays and relative density R_d and σ_{oct} for sands). Kulhawy et al. (1969) pointed out that the experimental value of σ will always be less than the asymptotic value and they termed the ratio

between the two values the "failure ratio", R_f which should always be less than unity. Kulhawy et al. (loc. cit.) studied the various procedures for fitting a straight line to transformed data and showed that a consistently good match with the actual stress-strain curve may be achieved if the hyperbola is chosen so that it coincides with the stress-strain curve at three points; the origin and the points where 70% and 95% of the strength are mobilised. They endorsed the use of the form of hyperbola suggested by Kondner as a simple and accurate means of representing stress-strain curves for soils.

Kulhawy et al. (1969) used the above concept to supply a stress analysis routine with a formulated non-linear stress-strain curve to be solved by a finite element program. Duncan and Chang (1970) used similar approaches for finite element analysis. They used different relationships for shear unloading-reloading cycles and their model was used successfully to compare field results by Mitchell and Gardner (1971). Modifications for the same model and other models were reported by Hamza (1976). Hamza also reviewed the mathematical adequacy of the functional representation developed from the models he reviewed. An elegant review of the empirical and curve fitting techniques of stress-strain behaviour of soils has been recently presented by Daramola (1978) in the context of studying deformations of granular media.

Unlike the above-mentioned research, the object of this study was, as stated before, to estimate the value of ultimate strength at small values of strain as well as

to estimate the value of E. The above review is to ensure the adequacy of representing the non-linear stress-strain relationships of soils as hyperbolae and consequently the correctness of extrapolating at early stages of the test.

6.8.3 Prediction of Ultimate Shear Strength

The results of all the triaxial tests carried out in this research have been processed to give the transformed relationships between $\varepsilon/(\sigma_1 - \sigma_3)$ versus $\varepsilon_1\%$.^{*} These relationships give straight lines the slopes of which give the inverse of the predicted ultimate shear stress differences (σ_{ult}). In Fig. (6.46) the relationships between the predicted ultimate shear strengths ($\sigma_{ult}/2$) and the ultimate shear strengths calculated as half of the measured maximum stress difference $(\sigma_1 - \sigma_3)_{max}/2$ are shown for most of the triaxial tests performed in this research. The ratios between the measured and the predicted shear strengths are called the "Failure Ratios", R_f (Kulhawy et al. 1969) and they are calculated together with their population standard deviation for each group of tests in table (6.1).

The prediction of ultimate strength for isotropically consolidated samples from early stages of the test (e.g. at ε_1 less than 5%), using the hyperbolic transformation conforms well with the conventionally calculated ultimate strength. The average value of R_f for remoulded samples is 0.93 and for slurry samples 0.95. The following remarks are however important in this context:

^{*}These relationships are shown for each test in Appendix 3

1) predictions by this method were poorer for remoulded samples than resedimented samples the standard deviation of remoulded samples, especially those of group G had the highest value among all other samples

2) the ultimate strength of anisotropically consolidated samples which were sheared under undrained conditions, whether normally or lightly overconsolidated, could not be predicted by this method. They showed brittleness and had small values of axial strain at failure, Fig.(6.51), which reflected in the deformation curves not being of hyperbolic shape. This argument is also valid for tests of group V which were sheared following an unloading stress path (SP2). Studying the relationships of $\epsilon_1/2q-\sigma_1$ of anisotropically consolidated samples which were sheared under undrained conditions in Appendix 3, linear relationships are observed. These linear relationships are, however, deceptive since they represent the parts of the stress-strain curves which were subjected to either strain softening or strain hardening recovery past the peaks. The lower part of table 6.1 shows the values of R_f of these tests to exceed unity which contradicts with the fundamental philosophy of the theory previously described. This method should therefore be dismissed as representative of the non-linear stress-strain curves for the anisotropically consolidated samples and should also not be used to simulate deformation behaviour of these samples in any finite element analysis.

3) anisotropically overconsolidated samples which had high OCR started shearing with a negative value of q in q - P' space. The straight line of the transformed hyperbolic relationship started when the deformation curve crossed the $q = 0$ line. This presented difficulties in predicting ultimate strengths of samples of high OCR because the strains at which the deformation curve crossed the $q = 0$ line were sometimes excessive. This invalidates the prime purpose of this investigation, namely to predict ultimate strengths at small strains, although the values of R_f for OCR = 10 are less than unity (see also test AO4P4OD, Fig. A3.114 and test AO4OP4OD, Fig. A3.115).

4) the deformation curves of the Lower Till which had a near hyperbolic configuration were non-linear from the start of shearing. The non-linearity is apparent from two pieces of evidence; firstly from the measured small strains given for every test, secondly from the lack of horizontal lines at the start of the transformed hyperbolic relationships (Kondner 1963 and Kulhawy et al. 1969 predicted that for deformation curves with a linear start, e.g. in the test results by Frydman and Zeitter (1969) the points at the start of the transformed hyperbolic plot will assume a horizontal line whereas for deformation curves with a non-linear start the points will tend to concave towards the origin of the (ϵ/σ) - ϵ space, Fig. 6.45 c and d).

5) stress-strain response of the Lower Till in isotropically consolidated samples sheared undrained and in anisotropically consolidated samples sheared drained had a hyperbolic shape. This was observed regardless of the strain rate, stress history or the confining pressure, Fig. (6.46).

6.8.4 Choice of Modulus of Deformation E

The non-linearity of the deformation curve at the start of shearing, as described above, leads to uncertainties as to where exactly the transformed hyperbolic lines intersect the $\epsilon/(\sigma_1 - \sigma_3)$ axis and hence the prediction of the initial tangent modulus (see item 6.9.2 and Fig. 6.45d). Thus the use of hyperbolic transformation is not recommended as a tool for estimating the initial tangent modulus in the Lower Till.

There remains, however, the need to evaluate a stiffness parameter to compare, and hopefully predict, the deformational behaviour of the Lower Till under different conditions (e.g. stress paths, stress histories, boundary conditions and rates of shearing). This deformation modulus should be useful in relating laboratory and field results and in predicting settlements. The methods for evaluating a deformation modulus are presented in Fig. (6.47) (see also table 6.2a). The first method, Fig. (6.47a) is for the initial tangent modulus E_i and the second method is for the secant modulus E_{sec} . The secant modulus could be evaluated from the origin,

Fig. (6.47b) or from other locations in the stress-strain curve, Fig. (6.47c). There is no practical preference as to which of the above methods is better in engineering application as long as the evaluation and comparison are consistent. Theoretically E_i facilitates comparison because it bypasses the critical problem of choosing a stress increment $\Delta(\sigma_1 - \sigma_3)$ or a strain increment $\Delta\epsilon_1$ for a secant modulus and measures directly from the origin. The evaluation of E_i , however, imposes practical problems because of the initial errors in any test which might cause an artificial decrease or increase in the value of E_i . These initial errors were discussed from an analytical view by Atkinson (1973). Lewin (1970) also discussed this point in studying deformation of remoulded clays and Som (1968) in studying deformation of undisturbed London Clay.

The simple evaluation of E_{sec} by direct measurement from test results attracted most researchers. Here the problem is in choosing the boundaries within which E_{sec} should be evaluated. A comparison was made by Souto Silveira (1967) for the different methods used for evaluating E_{sec} , table (6.2b). Webb (1966) and Agarwal (1967) used E_{50} as did Casagrande and Hirschfeld (1962) their tests were all isotropically consolidated and using E_{50} did not impose any problem whereas this might have caused problems if the samples were anisotropically consolidated especially if the initial deviator stress exceeded 50% of the peak deviator stress (Atkinson 1973). Som (1968) used a secant modulus E which related to the stress ratio during

consolidation K' , i.e. his deformation indicator was a two-parameter one. Reades (1972) used the ratio between the deviator stress and axial strain at 1% as his E and Atkinson (1973) used two secant deformation moduli in his research; one for interpreting the test data was calculated as the ratio between the stress increment and the strain increments at 10% and 30% of the peak deviator stress (this modulus he named E_s and measured approximately E_{50}). The other modulus was for comparing results of tests from a variety of sites in the London Clay and was measured over an increment of the stress-strain curve close to the origin (he named this modulus E_i and was calculated from an assumed mathematical algorithm for a non-linear stress-strain curve for undisturbed London Clay). Wesley (1975) used a secant modulus at $\epsilon_1 = 0.25\%$. In recent years at Imperial College the analytical research on finite element analysis gave rise to the use of a deformation modulus that is progressively calculated from a non-linear stress-strain relationship as the stress level increases. The above method was developed by Hamza (1976) and used by Davachi (1978) and Lopez (1978). Back in (1964), Lambe and Whitman calculated the deformation modulus as the ratio $\Delta\sigma/\Delta\epsilon$, which varied with ϵ_1 . In measuring the effect of stress history on the deformation of granular soils, Daramola (1978) used three parameters to indicate deformations. These parameters were a modulus $M_t = \Delta\sigma_v'/\Delta\epsilon_a$ and a stiffness number $S_N = M_t/\sigma_{va}'$ (where σ_{va}' is the average vertical stress during which $\Delta\sigma_v'$ took place) and

lastly a strain ratio $\dot{\epsilon} = \Delta\epsilon_h / \Delta\epsilon_a$. Simons and Menzies (1975), section 3.6 and 3.7 presented useful study and references regarding the factors affecting the choice of E.

In this research the deformation is indicated by a secant modulus defined, as Reades (1972), at 1% axial strain for all isotropically consolidated samples and for anisotropically consolidated samples under drained shearing. It was decided not to use a stress increment, (although practically preferred in engineering practice), as a datum because of the apparent non-linearity of the deformation curves which might cause the measurement to lie beyond the very start of the test where elastic behaviour is most likely to take place, Davis and Poulos (1968). The measurement of small strains in the triaxial tests helped in the proper evaluation of this parameter.

Care was exercised in chapter 5, to correct initial errors of each test especially those due to the deformation of the load cells and the displacement of the load transducers inside the load cells within their gaps in the loading frames. The measured values of E from the results of isotropically consolidated undrained triaxial tests were plotted against P'_0 (P'_0 is the mean effective consolidation pressure prior to shearing) (in a log-log scale) in Figs. (6.49) and (6.50) for resedimented and remoulded samples respectively.

From the above figures the following observations are made:

1) The scatter of results of remoulded samples, Fig. (6.50), is large and masks the effect of different shearing rates. The scatter of the results of resedimented samples is less as would be expected from the discussion in items 6.2 and 6.3. Atkinson (1973) reported large scatter in the value of E in his tests and in tests from other sources. He also reported a larger scatter from drained tests than from undrained ones.

2) E increases with P'_0 which is valid for both types of soil structures and for different consolidation histories of the resedimented sample. This finding only confirms what previous researchers have reported for different types of soils and tests. Terzaghi and Peck (1967) and Lambe and Whitman (1969) also reported earlier findings of the above relationship. Ladd (1964) and Som (1968) reported unique values of E/P'_0 provided that normally and overconsolidated samples were treated separately. Wroth (1971) emphasised this relationship and Marsland (1971) reported it measured from field tests. Atkinson (1973) also reported the same relationship. The interesting finding is that for anisotropically consolidated drained tests the value of E could be assumed to be constant, although the number of tests are small and caution has to be taken in drawing a general conclusion at this stage.

3) In remoulded samples the scatter masked the rate effect as mentioned in point 1) above. It was not feasible to separate the relationships between E and P'_0 for different rates although it was expected from the results of section 6.4 that E would increase with the increase in rate, Fig. (6.30). In resedimented samples, however, the rate had no effect on the E - P'_0 relationship and tests of the same stress history lay on the same average line. This conforms with the interpretation of the results of item 6.4, Fig. (6.30). The relationship between E - P'_0 and the shearing rate should be considered a material property of the Lower Till (Atkinson 1973). Bjerrum et al. (1958) found that E increased with increasing rate, Berre (1973) reported the same but also discovered that for a clay (Fornebu Clay) the rate had no effect on the value of E . (See Simons and Menzies 1975).

4) The value of E/P'_0 increased linearly with the OCR in resedimented samples as shown in Fig. (6.49). The slope of the lines in the $\log E$ - $\log P'_0$ plot is around unity giving an inclination of about 45° for "all" isotropic samples. Wroth (1971) suggested that, for London Clay, E/P'_0 is a function of OCR. Atkinson (1973) and (1974) reported a linear function for this variation and emphasised the large difference in the slopes of this function between measurements from the field and laboratory tests.

5) The value of E/C_u has been used as a basis for settlement calculations because of the difficulties in measuring E from test results. The values of the parameter E/C_u of the Lower Till are presented in table (6.3) for different rates and structures. It is observed that this soil parameter tends to decrease with the decrease in W and, consequently, the increase in OCR. Low values of E/C_u for highly plastic clays (and high values for low plasticity clays) have been reported by Ladd (1964) and Bjerrum (1972). Simons (1974) presented a list of 15 sites, reported by himself and others, for which E/C_u was calculated. The value of E/C_u in this table varied between 40-3000.

6) In spite of not using undisturbed samples in this section a point of speculation should be put forward regarding the effect of the sampling disturbance on the value of E . Many scholars recommended reconsolidating the undisturbed sample to a state of stresses simulating that of the field (e.g. Ladd 1965, Davis and Poulos 1968, Simons and Som 1969 and Raymond et al. 1971). The evidence from the present work, limited as it may be, suggests that for "perfect" sampling followed by reconsolidation to the field stresses (as demonstrated by test SMP2, Fig. (6.40), item 6.6), the deformation behaviour during subsequent undrained shearing resembles closely that of the "in-situ" case. The E moduli were not measured due to the difficulties previously described for anisotropically consolidated samples but the evidence seems to agree with the general convention.

Perfect sampling is, however, impossible to attain especially for this low plasticity till and the author recommended that in order to evaluate proper "in-situ" deformation behaviour plate loading field tests should be carried out, item 6.6.3. Bauer et al. (1973) reported good correlation between plate loading results and results from a full scale rigid footing. They also found the laboratory results were generally low and inconsistent. Marsland (1971) found reasonable correlation between triaxial tests of two sizes of samples and results from "in-situ" plate loading tests in London Clay.

In anisotropically normally consolidated samples sheared under undrained condition, the peak shear stresses were reached at small strains especially when the tests started at high values of consolidation pressure as shown in Fig. (6.51). Intensive recordings of the test parameters in the early stages of each of these tests were necessary in the laboratory least the early peaks should be missed. Evaluating E at $\epsilon_1 = 1\%$ as above (for isotropically consolidated samples and anisotropically consolidated samples sheared under drained conditions) is, thus, not representative of the measured stiffness. The best evaluation of the modulus in this case has been found to be at the maximum shear stresses (q_{max}) reached at very test. In table (6.4) the values of E measured at q_{max} and at $\epsilon_1 = 1\%$ are shown together with the values of $E_{q_{max}}/C_u$.

In Fig. (6.52) the $E_{q \max} - P'_0$ relationships (in log-log scale) are plotted for anisotropically consolidated samples with different histories. The $E_{q \max}/P'_0$ values were found to be non-linear for the available data despite plotting on a log-log scale. However calculating the arithmetic means for the E/P'_0 of each history the relationship shown in Fig. (6.48) emerges. From the above the following conclusions are made:-

1) care should be taken in measuring E for anisotropically consolidated soils when subjected to undrained shearing. The best measure is found to be the ratio of the stress ($q_{\max} - q_0$) to the strain $\epsilon_1\%$ at $q_{\max}$. If for some reason another modulus should be used as a stiffness parameter, the strain increment of this modulus should be small enough to ensure that the measurement is carried out before the peak shear stresses are reached.

2) anisotropically consolidated samples which are sheared under undrained conditions are, generally, stiffer than isotropically consolidated samples. The slope of the $\log E - \log P'_0$ plot of the average points in ANU group is 68.4° as compared to 45° for isotropically consolidated samples.

3) the increase in OCR does not automatically mean an increase in stiffness as in isotropically consolidated samples, see item 6.7.2, as this will depend on several factors, e.g. position of P'_0 in the $q - P'_0$ space and the value of the OCR. This fact is reflected in the $E/P'_0 - \text{OCR}$ relationship for anisotropically consolidated samples in Fig. (6.48) (where E/P'_0 increases with OCR then decreases

at $OCR = 10$). The value of the parameter E/C_u , table (6.4), decreases with the increase in the value of the OCR at a certain consolidation pressure, P'_o .

The above study on the evaluation of E should be of some use in trying to predict the field behaviour should the Lower Till be used as a foundation material for a certain structure. Coupled with the normalised relationships presented in section 6.3 and with a proper method of stress analysis, ample data should be available for design as will be discussed in chapter 7.

Group	Soil	Consolid- ation System	R_f	Standard Deviation	R_f (average)
G	R	I	0.95	0.09	0.95
WCR Ra	R	I	0.87	0.16	
Rb	R	I	0.94	0.05	
Rc	R	I	0.91	0.08	0.90
Rd	R	I	0.90	0.06	
IN Ra	S	I	0.95	0.04	
Rc	S	I	0.96	0.04	0.96
IO Ra	S	I	0.94	0.04	
Rc	S	I	0.96	0.05	0.95
AN D	S	A	0.91	0.03	0.91
AO D	S	A	0.96	0.08	0.96
AN U	S	A	(1.16)	0.14	(1.16)
AO2 U	S	A	(1.12)	0.03	(1.12)
AO4 U	S	A	(1.04)	0.11	(1.04)
AO7 U	S	A	0.88	0	0.88
AO10U	S	A	0.76	0	0.76

TABLE (6-1) Failure Ratios (R_f) Between Measured and Predicted Shear Strengths.

TYPE OF MODULUS OF ELASTICITY		REFERENCE
Tangent	at the origin	Parkin, 1961
	related to a strain	Burmister, 1948 Hobbs, 1963
Secant	related to a stress	Casagrande, 1947
		Casagrande, 1948
		Casagrande, 1960
		Casagrande, 1964
		Ladd, 1964
		Palmer and Barber, 1940
		Yoder, 1959 Whitman et al, 1964
Ratio of change of stress to change of strain	varies with ϵ_1	Lambe and Whitman, 1964

(a)

COMPARISON CRITERIUM, STRESSES CORRESPONDING TO	REFERENCE
1% strain	Seed, Chan & Monismith, apud Alais, 1960.
1%, 2% strains	Seed, McNeill and Guenin, 1958 Seed and Chan, 1957
1%, 2.5%, 5%, 10% strains	Mitchell, 1964
2%, 6%, 12% strains	Olson and Hardin, 1963
5% strains	Seed, Chan and Monismith, 1955
5%, 20%, 25% strains	Seed and Chan, 1959 and 1959 A
50% maximum stress	Casagrande, 1962
1, 5 and 3 of Safety Factor	Ladd, 1964

(b)

Table (6-2) Evaluation of The Modulus of Elasticity- After Souto Silveira (1967).

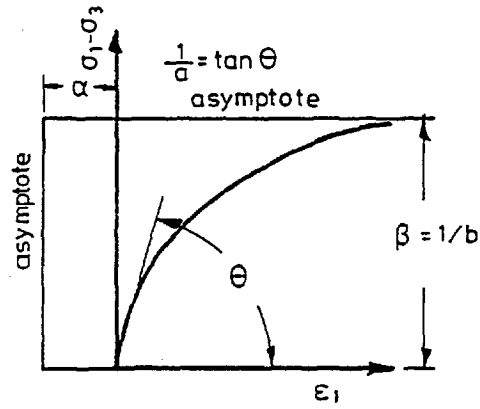
TABLE (6-3) Values of the ratio of Deformation Modulus
to Undrained Shear Strength of the Lower Till.

Structure	OCR	rate mm/min	$\frac{C_u^*}{P'_o}$	$\frac{E}{P'_o}$	$\frac{E}{C_u}$
Resedimented Isotropically From Slurry	1	0.005	0.28	41.35	147.68
	1	0.500	0.36	41.35	114.86
	2	0.005	0.52	57.90	111.35
	2	0.500	(0.60)	57.90	90.50
	4	0.005	0.90	117.08	130.09
	4	0.500	1.12	117.08	104.53
	7	0.005	1.42	142.00	100.00
	7	0.500	1.52	142.00	93.42
	10	0.005	2.79	175.00	62.72
Remoulded	not known	0.005	0.33	38.00	115.15
		0.500	0.49		

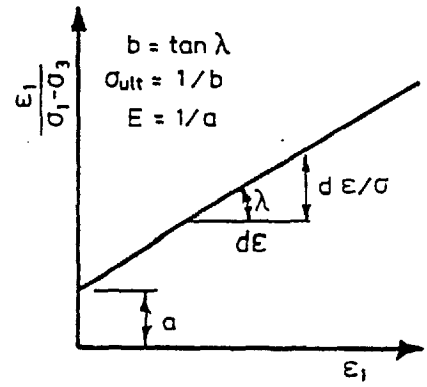
*calculated from the normalised deformation curves of
section 6-3.

Test	q_o kPa	q_{max} kPa	$q_{max} - q_o$ kPa	ϵ_1 % at q_{max}	$E_{q_{max}}$ kPa	$\frac{E_{q_{max}}}{C_u}$	$E_{\epsilon_1=1\%}$
ANP40	8.79	13.11	4.32	7.840	110.2	8.93	100.0
ANP80	27.38	32.31	4.93	0.342	2883.0	89.80	482.0
ANP160	51.50	57.25	5.75	0.069	16670	291.20	100.0
ANP320	122.97	133.16	10.19	0.073	27920	209.70	negative
AO2P40	3.50	24.86	21.36	0.399	10710	433.60	4100.0
AO4P40	-7.00	36.85	43.85	12.262	720	18.80	8000.0
AO10P40	-23.20	36.39	59.59	31.603	388	10.70	5040.0
AO4P80	1.45	87.49	86.04	1.677	10260	117.30	17310.0
AO7P80	-33.04	95.74	128.78	19.047	1350	13.90	7208.0
AO2P146	22.75	91.90	69.15	0.324	42690	464.50	12650.0

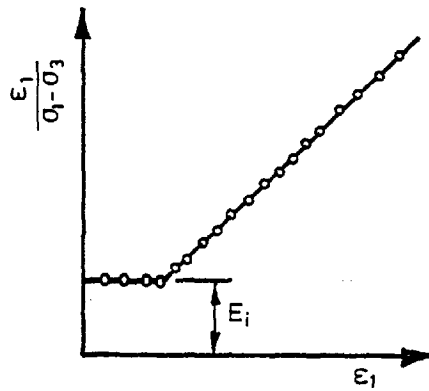
TABLE (6-4) Evaluation of Deformation Parameters for
Anisotropically Consolidated Samples.



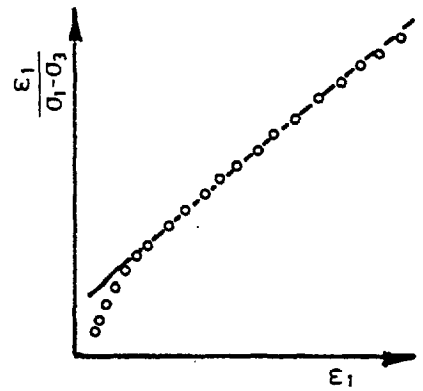
a) Rectangular hyperbolic representation of stress-strain.



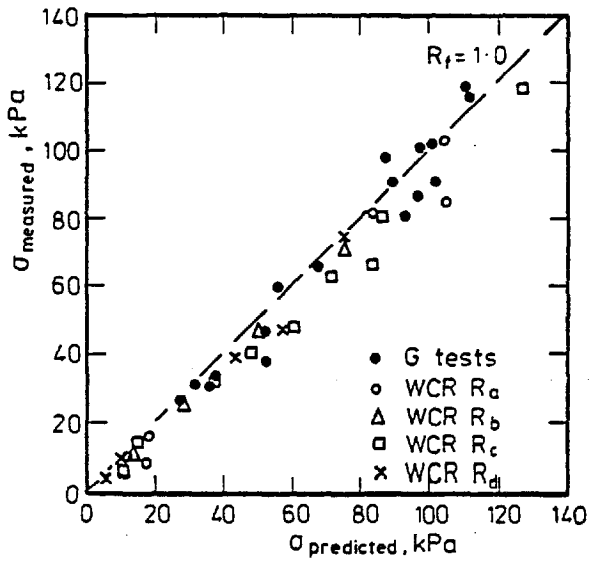
b) Transformed hyperbolic representation of stress-strain.



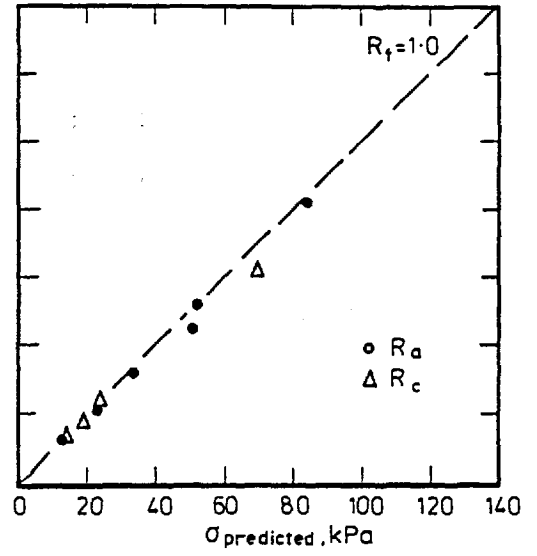
c) Representation of a linear stress-strain curve.



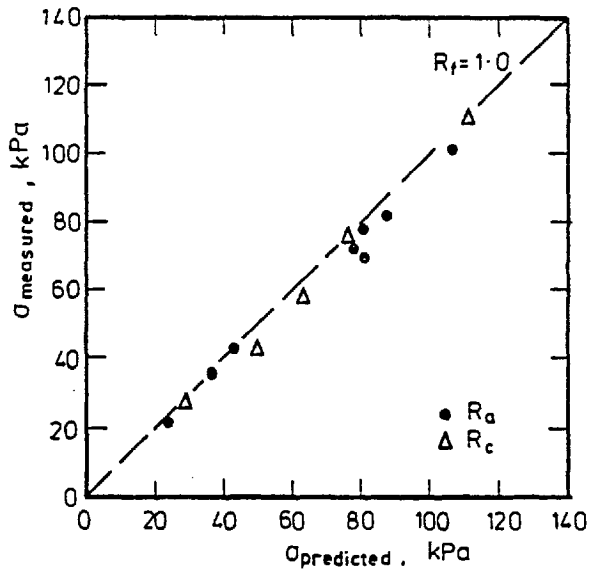
d) Representation of a non-linear stress-strain curve.



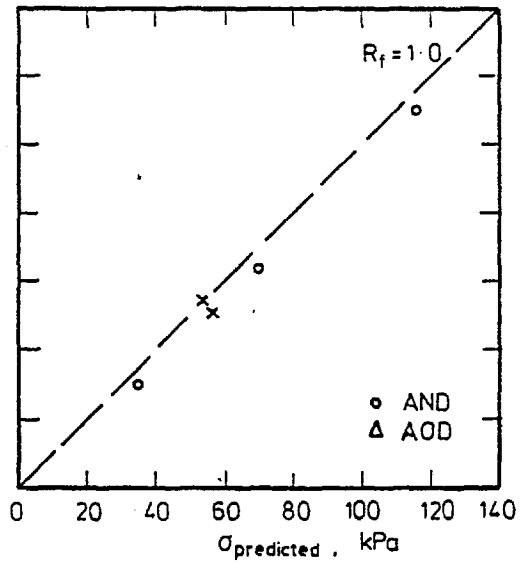
a) G & WCR Group



b) IN Group



c) IO Group



d) Anisotropic tests

Fig. 6-46 Correlation between measured and predicted ultimate values of shear strength ($\sigma = \frac{1}{2} (\sigma_1 - \sigma_3)_{\text{max}}$).

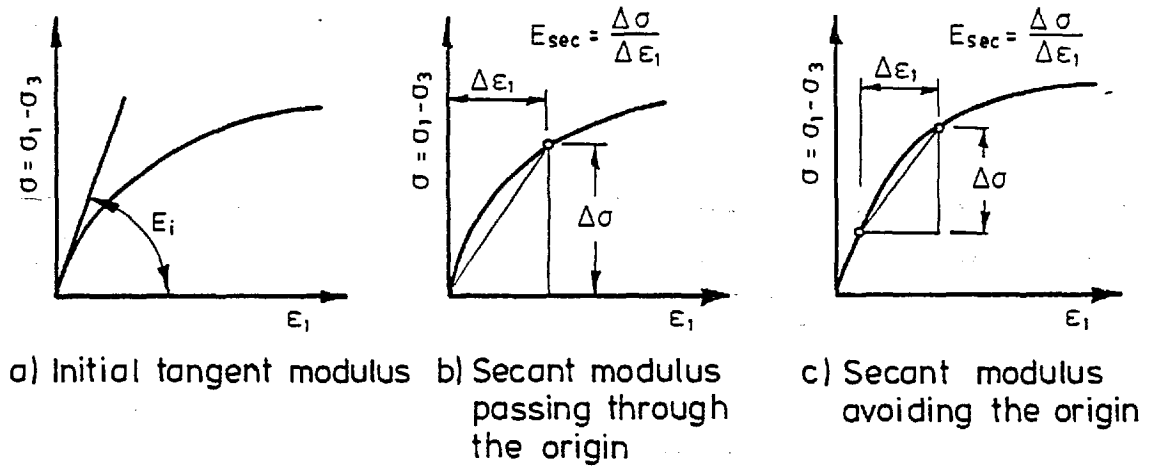


Fig. 6-47 Methods of evaluating deformation moduli.

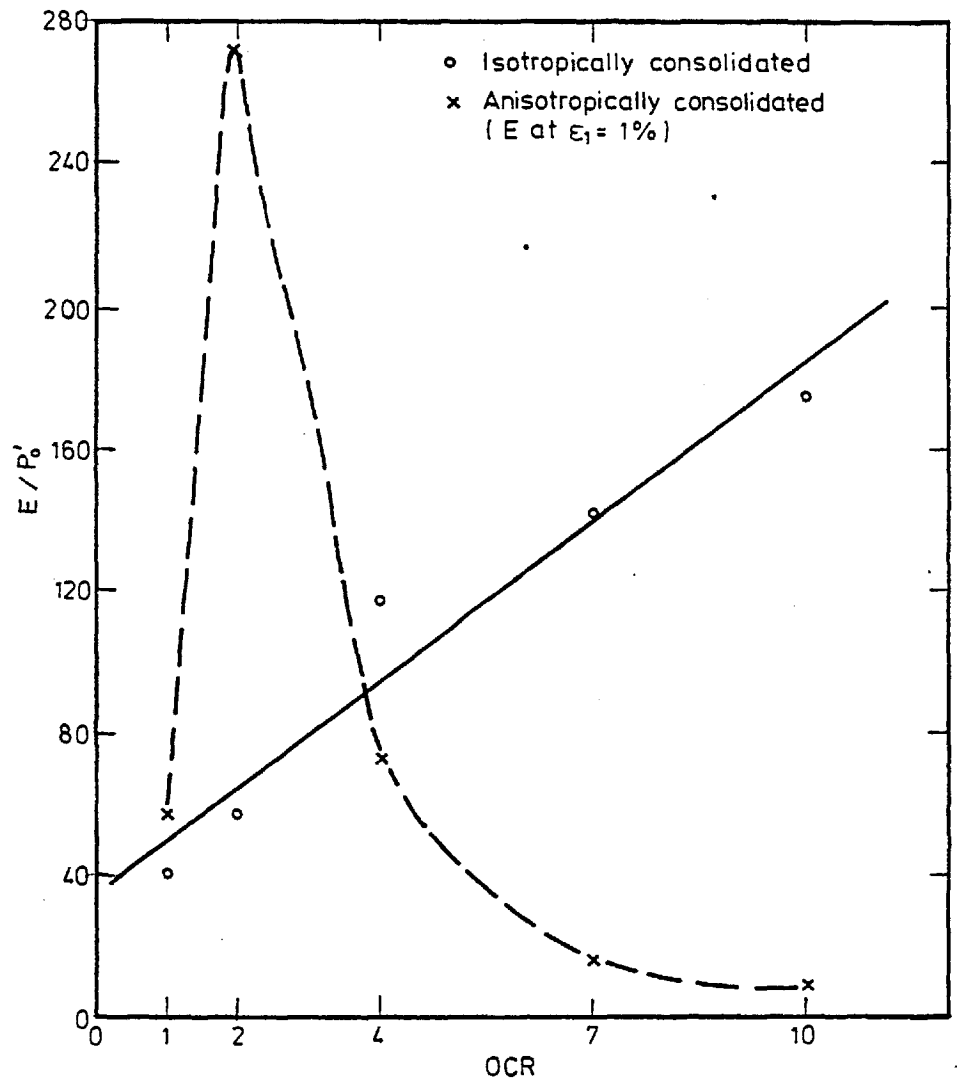


Fig. 6-48 The relationship between stiffness, consolidation pressure and overconsolidation ratio (resedimented samples).

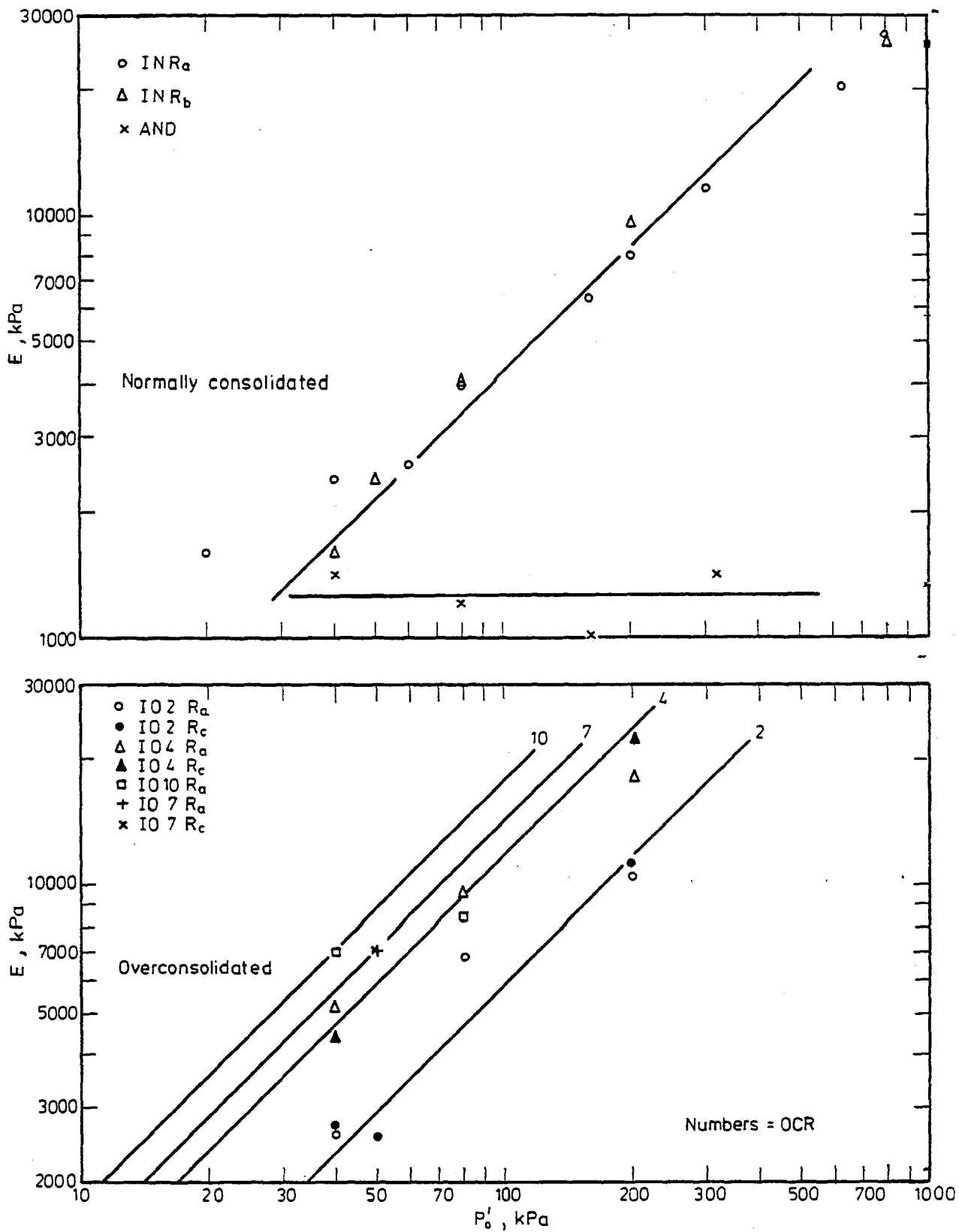


Fig. 6-49 The relationship between stiffness and consolidation pressure (resedimented samples)

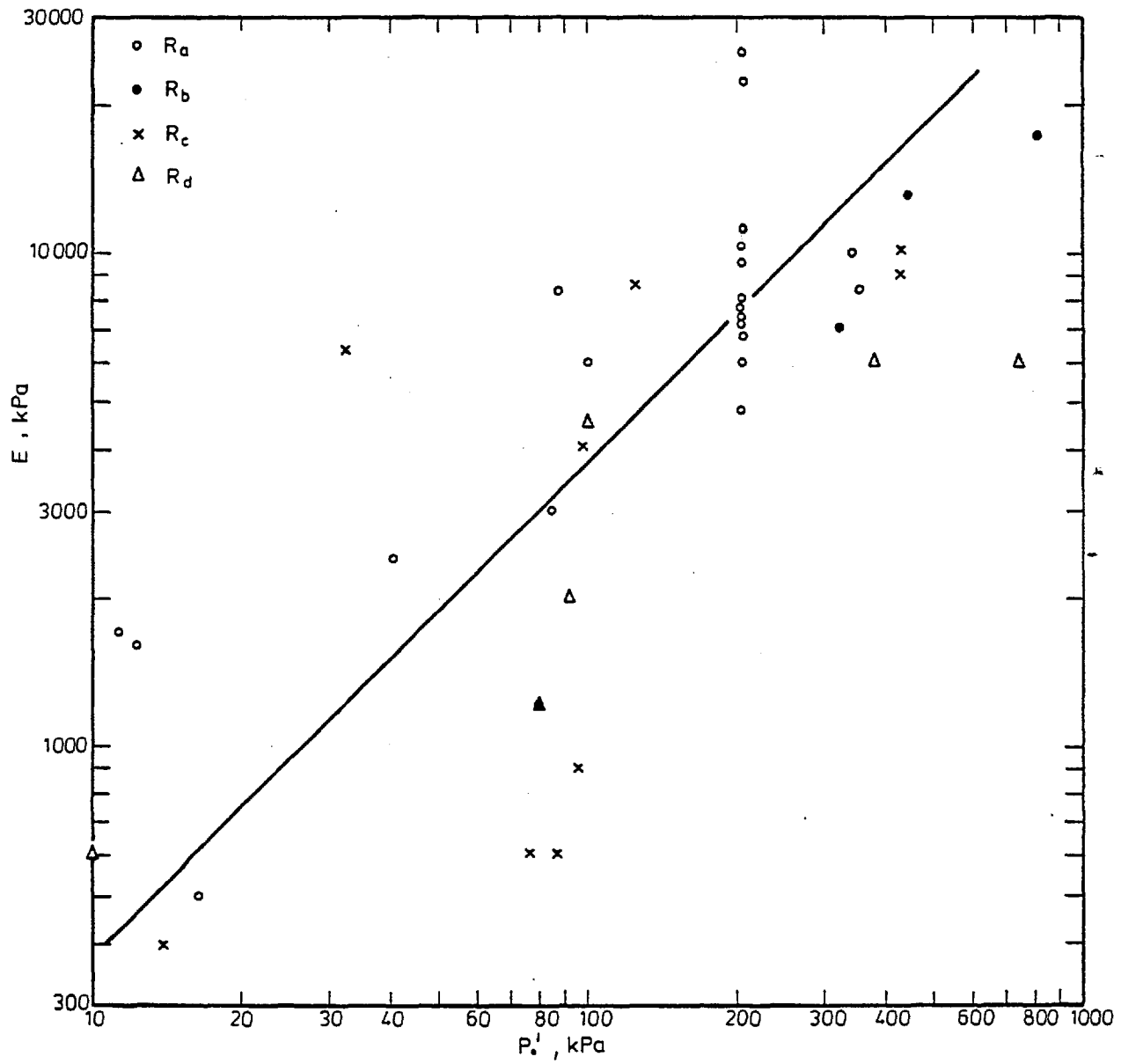


Fig. 6-50 The relationship between stiffness and consolidation pressure (remoulded samples),

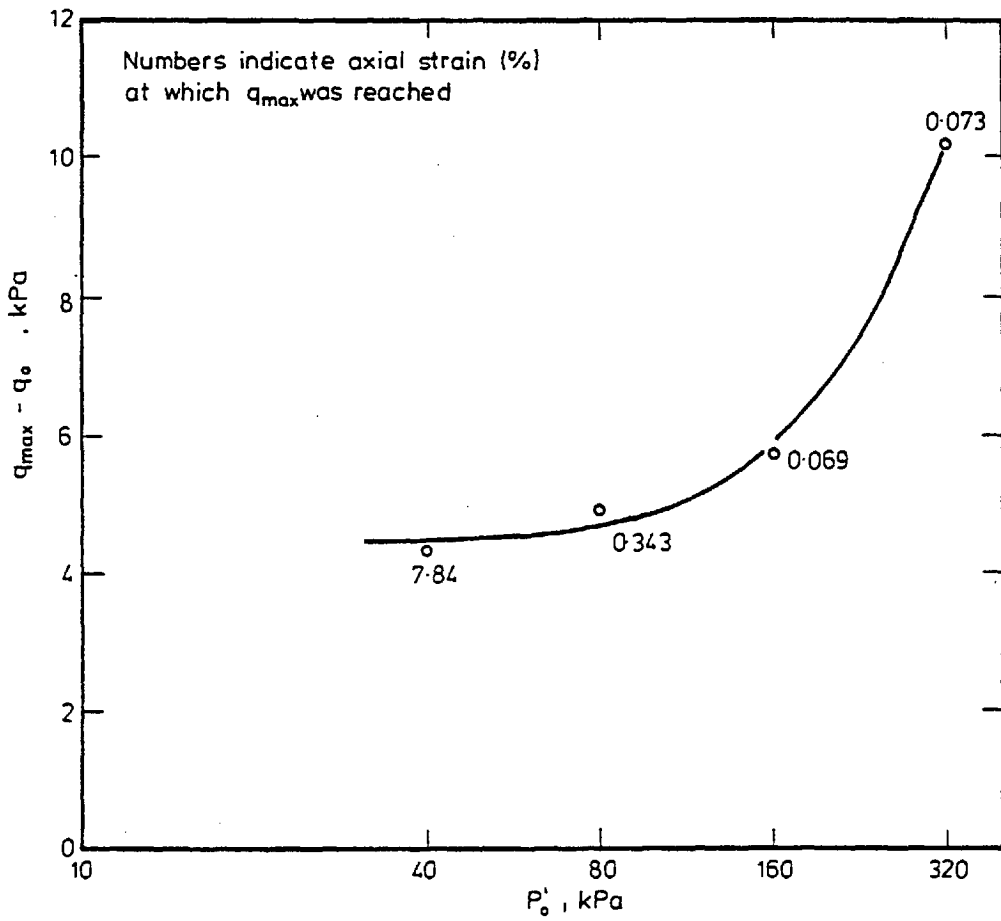


Fig. 6-51 The relationship between the consolidation pressure P'_0 and the difference between the initial (q_0) and maximum ($q_{\max}$) shear stresses in anisotropically consolidated tests (undrained).

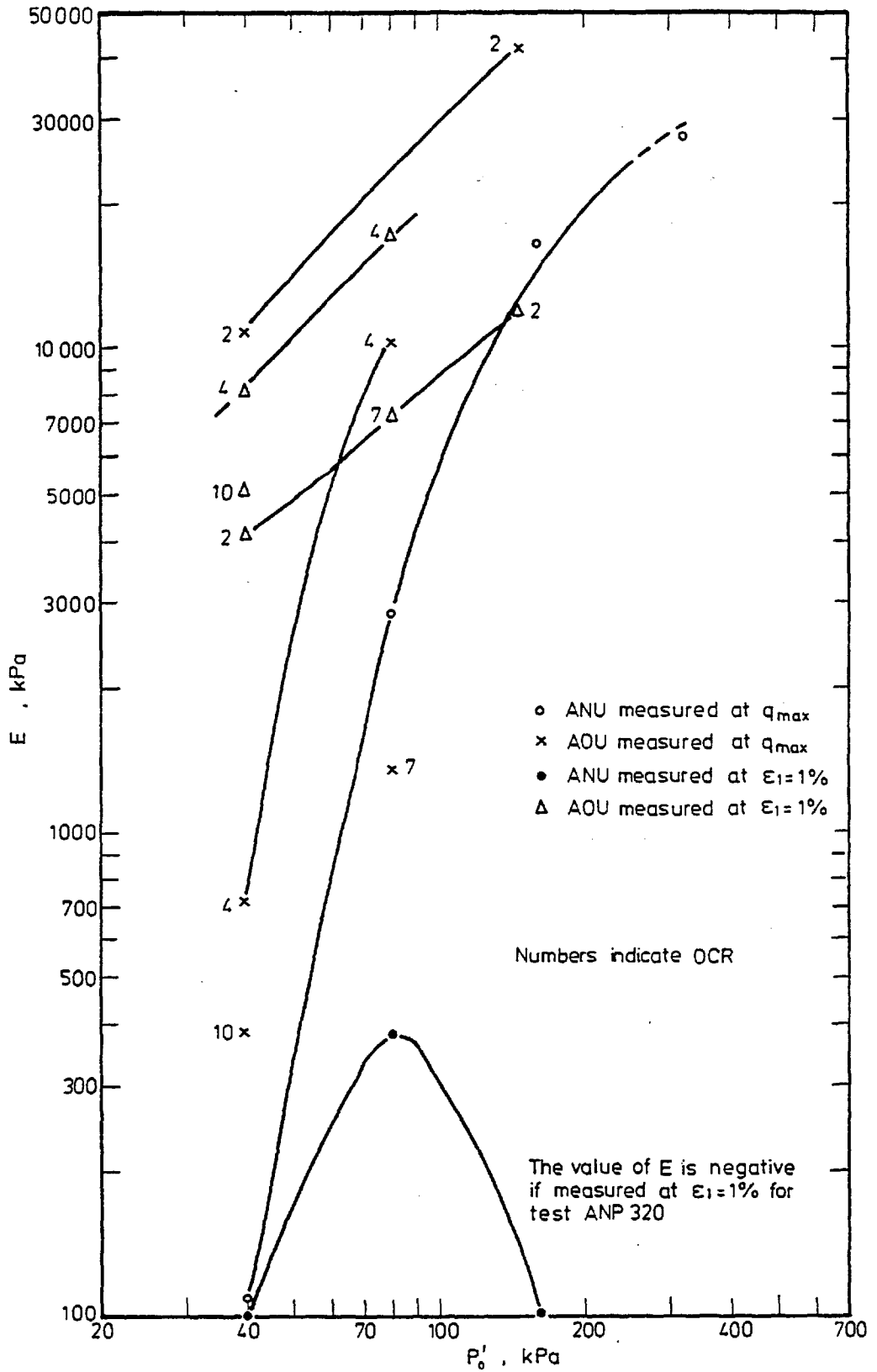


Fig. 6-52 The relationship between stiffness and consolidation pressure for anisotropically consolidated samples in undrained shearing.

6.9 OTHER OBSERVATIONS

6.9.1 General

The items that follow, namely the effects of:

- 1) fissures (discontinuities)
- 2) age of consolidation
- 3) creep
- 4) increments of loading
- 5) stress path, and lastly
- 6) drained and undrained behaviour

have been studied in the light of a limited number of tests. Thorough study or an elaboration on the existing results of each of these items is beyond the scope of this work. However, it was found useful to report the available findings and further research might hopefully cover these points in more depth.

6.9.2 Effect of Fissures

To assess the effect of discontinuity on the behaviour of the Lower Till, three samples with artificial fissures were tested (Tl6, Tl8 and Tl9, Figs. A3.11, A3.12 and A3.18). Each of these samples was mixed in the same way as described for remoulded samples in item 4.3. The sample was formed in two halves which were assembled to provide a plane between them at 60° to the horizontal, refer Apted (1977). The two halves were just gently placed over each other then the usual procedure of testing took place. It was impossible to introduce artificial fissures in

resedimented samples because the testing technique developed did not allow for such a provision, item 4.3

Test T18 (fissured) is compared with test T21 which is a normal remoulded test. These two tests resemble each other in P_o' and the rate and their water contents during shearing differed by only 0.05%, table 5.1. The results of these two tests are given in Figs. (A3.12) and (A3.10) and plotted together in Fig. (6.53). Apart from the end part of the ESP of T21, and allowing for a reasonable scatter, the results of the two tests lay on one another in every space. Tests T16 and T19 behaved in every space as normal remoulded tests, although unfortunately no normal remoulded tests that resemble them exactly were available. A conclusion which can be drawn out of this investigation is that there is no effect of discontinuity on the behaviour of the Lower Till if circumstances like those of the above tests prevailed. McGown et al. (1974) reported the presence of vertical fissures in some Scottish boulder clays and suggested that they may affect the behaviour of cut slopes. Vaughan and Walbancke (1975) discussed in some detail the relations between the fissured, peak and residual strengths and reported that the strength along the fissures and the peak strengths are extremes and that the undrained strength required for design will be in between these extremes.

6.9.3 Effect of the Age of Consolidation

Testing two samples under similar conditions except for a different age of consolidation has been done in the study of "sampling", section 6.6, tests SMP1, Fig. (A3.116) and SMP4, Fig. (A3.119). The samples were anisotropically consolidated at the same rate to the same point in the q - p' space, then the young sample SMP1 was unloaded immediately while the aged sample SMP4 remained for 216 hours before being unloaded. The results show comparable ultimate strengths (after correcting for the difference in water contents) as explained in item 6.6. In the case of deformations the aged sample showed larger strain hardening recovery than the young one. This recovery could have also been influenced by the fact that the aged sample happened to have a lower water content. It was reported by Vaughan (1971) that, for a low plasticity clay, the lower the water content the larger the strain hardening recovery during deformation.

An isotropically consolidated sample ST1 was consolidated in increments lasting 24 hours each (see item 4.3.1.2) to reach a pressure $P'_0 = 200$ kPa. It was then left for a week (168 hours) under this condition, then sheared under undrained conditions. The results shown in Fig. (A3.120) are compared with the results of test INP200 Fig. (A3.72) which was sheared at the same rate, Fig. (6.54). Again the ultimate strengths are comparable after correcting for the differences in water content. Both deformation curves were typically non-brittle but the normal test INP200 had a higher value of E , as defined in item. 6.9

(8000 kPa) than test ST1 ($E = 7200$ kPa). The marked difference, however, is in the values of the ratio of effective stresses $\sigma_1'/\sigma_3' = 2.89$ while ST1 had a value of $\sigma_1'/\sigma_3' = 3.35$. This might suggest that the long time of secondary consolidation has led to more stable configuration of the structure and consequently higher values of shear strength parameters, see Bjerrum and Lo (1963) and Bjerrum (1967).

6.9.4 Effect of Undrained Creep

The effect of undrained creep was studied from the results of the two tests, ST2 and ST3, Figs. (A3.121) and (A3.122). Creep was observed by first consolidating the samples for longer times than the usual, item 4.3.1.2, to ensure that no visible volumetric changes were occurring (both tests were left under the final consolidation pressure for 72 hours instead of 24 hours). Secondly, the valve to the back pressure was closed and the pore pressures monitored until it stabilised, Fig. (6.55). This pore pressure stabilisation took 192 hours in test ST2 (isotropic consolidation) and for 92 hours in test ST3 (anisotropic consolidation). The increase of pore pressure of test ST2 during creep was 100 kPa which brought P'_0 to 100 kPa. The increase of pore pressure for test ST3, however, was only 19 kPa which brought P'_0 to 181 kPa.

Test ST2 is compared with test INP200, Fig. (6.56). (the slow rate of test ST2 does not affect the comparison as it was observed from test results and reported in item 6.4 that rate effect diminishes below 0.005 mm/min). The

water content W_0 of the two tests is practically the same and test ST2 gave an ultimate strength of 48 kPa and test INP200, 52 kPa. The stress paths of the two tests are different because the pore pressure response of test ST2 during undrained shearing was much less than that of test INP200 which compensated for the difference in the value of P'_0 . Test ST2 gave higher value of σ'_1/σ'_3 during undrained shearing than test INP200. The "failure" values of σ'_1/σ'_3 for both tests were, however, closely similar.

If test ST2 is compared with a normalised stress path of a normally-consolidated sample with the same P'_0 , different behaviour and end results are expected because of the different W_0 (about 1.50% difference) of test ST2 and a normally-consolidated sample sharing its same P'_0 , Fig. (6.56).

The effect of creep on the anisotropically-consolidated sample ST3 is much less drastic than on ST2. Anisotropic consolidation probably created a more stable structure than isotropic consolidation. The pore pressure of ST3 increased slightly (19 kPa) as compared to ST2 (100 kPa) and also in a much shorter time (92 hours) than ST2 (192 hours). The shape of ESP was as normally-consolidated samples but the ratio of C_u/P'_0 was 0.32 instead of 0.29 for normally-consolidated samples. The only unusual behaviour in ST3 is the sudden increase in pore pressure of $\epsilon_1 = 11\%$. A check was made to ensure that no electrical fault caused this increase and the author is convinced that this was a genuine event during

shearing. The destruction of a certain level of bonding created during consolidation and creep could be a plausible explanation for this phenomena.

The undrained creep of sandy clay embankment dams has been reported to be slight. Vaughan and Hamza (1977) reported that at Cow Green (see Vaughan et al. 1975) the deformation ceased at the end of loading and at Muirhead (see Banks 1948) movement ceased within a few days of the end of loading. It is interesting to note that in the above investigation on the Lower Till no movement (vertical deformations) at all was observed, yet the pore pressure rose above its equilibrium value as soon as the back pressure connection was cut off.

Undrained creep has been studied by Mitchell (1976) and among his observations is one that the pore pressure during undrained creep could increase or decrease or remain constant. He also reported that the creep behaviour is sensitive to the pre-creep stress system and, quoting results from Campanella and Vaid (1974) reported that if creep followed isotropic consolidation it will yield larger changes than if preceded by anisotropic consolidation. This behaviour agrees with the findings of the current investigation. Ladd et al. (1977) related the general phenomenon of creep in both drained and undrained conditions and emphasised the difficulties in extrapolating laboratory results to field situations. He also suggested that undrained creep does not usually affect field situations in most cases

(except at low undrained factors of safety with highly plastic and/or organic soils). Fortunately in the case of the studied Lower Till the effect of creep is expected to be slight since most of its field conditions are expected to be due to anisotropic consolidation. Extrapolation of test results could, therefore, be used in "in-situ" situations.

6.9.5 Effect of Loading Increment Ratio

The effect of the magnitude of the loading increment ratio (keeping the duration of loading constant) has been investigated when carrying out the oedometer tests. Figs. (5.6) and (5.7) show the $W-\sigma'_v$ relationships for different magnitudes of increment ratios. In Fig. (6.57) the relationship, in a semi-log plot, between the increment ratio n and the volumetric change is shown. The volumetric strain is expressed, in percentage, as the ratio between the change in the water content due to a certain ratio to the change in the water content due to a ratio of 2 (an increment of 2 being the conventional ratio used in oedometer tests). The results show that an increase in the value of n increases the volumetric strains and that this increase tends to disappear at large values of n where the relationship tends to be asymptotic to the n axis. These findings could be of practical importance when decisions are to be made about the sequence and loadings of any structure founded on a Lower Till.

Som (1968) observed that the volumetric strains were dependent on the stress increment ratio when testing undisturbed London Clay in the oedometer. Atkinson (1973) studied the effect of stress increment ratio in ideally linear and non-linear elastic material and in plastic material. Leonards and Ramiah (1959) reported that for remoulded clay in the oedometer the volumetric strains depended on the increment ratio irrespective of whether secondary consolidation was allowed to occur.

6.9.6 Effect of Stress Path

6.9.6.1 Consolidation stress path

The differences that arise in soil behaviour during undrained shearing preceded by either an isotropic or anisotropic consolidation stress path have been dealt with in item 6.5. An additional investigation has been made to assess the differences in behaviour during undrained shearing between two samples consolidated from slurry following two different consolidation stress paths to the same point in the q - p' space. Test ANP160, Fig. (A3.101) has been consolidated anisotropically from slurry and test ST7, Fig. (A3.124) has been isotropically then anisotropically consolidated from slurry to the "same" point in q - p' space, fig. (6.58a).

The W - P' relationship during consolidation for the two tests is shown in Fig. (6.58c). It is noted that the different consolidation stress system in test ST7 produced a lower W_o than test ANP160. (There was also a difference of 2 kPa in q_o and of 2 kPa in P'_o between the two tests, Fig. (6.58a).)

The two ESP's have similar shapes and the difference in their maximum stress difference (2.5 kPa) is compensated by the difference in their W_0 at this stress level, Fig. (6.39). The two tests also share the same deformation characteristics.

The values of the pore pressure coefficient A at maximum shear stress difference are 0.05 for test ANP160 and 0.03 for test ST7. If the values of A at the maximum effective stress ratio were considered, test ST7 would have higher A value (0.63) than test ANP160 ($A = 0.53$). The difference in the latter case is much larger than in the former. There is a difference of 0.25 between the values of the maximum effective stress ratio (σ_1/σ_3') max. in the two tests, test ST7 having the higher value. If it is assumed that the value of (σ_1'/σ_3') max. reflects the value of ϕ' only, the above difference will result in a difference of just less than 2° in the value of ϕ' . This difference is small and the values of ϕ' usually fluctuate within 2° and it will be considered that this consolidation stress path does not affect the value of ϕ' . Parry (1971) measured almost the same difference in ϕ' when testing remoulded Weald clay and considered ϕ' to be constant. Henkel and Sowa (1963) reported no effect of consolidation stress path on ϕ' ; they were also testing remoulded Weald clay.

6.9.6.2 Shearing stress path

No direct comparison is available for undisturbed or resedimented samples from which a meaningful conclusion could be drawn. However in the case of remoulded samples use could be made by comparing test V1R' which was tested following SP1, Fig. (A3.60) and the general behaviour of tests V1R:V4R, Figs. (A3.59), (A3.62), (A3.64) and (A3.66) which were tested following SP2. All the above tests were sheared under drained conditions. From this comparison the following observations are made.

i) tests following SP1 show lower (σ_1'/σ_3') max. than tests following SP2. Parry (1960) reported that ϕ' was independent of the stress path when testing remoulded London and Weald clays. In his (1974) paper (with Nadarajah) he tested a Spestone kaolin and reported that ϕ' was independent of the stress path for isotropically consolidated samples and that ϕ' was dependent on the stress path for anisotropically consolidated samples.

ii) tests following SP1 showed smaller brittleness, lower modulus (see item 6.10) and higher axial strain at failure, ϵ_f , than tests following SP2. Parry and Nadarajah (1974) reported that at small ϵ_1 , Young's modulus E is independent of the stress path in isotropically consolidated samples. They also stated that E is dependent on the stress path in anisotropically consolidated samples. The dependence of the stress-strain behaviour of soils on the stress path has also been reported in Parry's early

paper of 1960 when he tested remoulded samples of Weald and London clay.

iii) the remoulded samples failed by bulging irrespective of the stress path.

6.9.7 Drained and Undrained Tests

An attempt has been made to investigate whether the drained behaviour of the Lower Till could be predicted experimentally, from its undrained behaviour (and vice versa) as was done by Henkel (1960) for saturated remoulded London and Weald clays (and for Wiener Tegel from the results of three papers by Rendulic 1936a, 1936b and 1937).

Only four samples which were anisotropically normally consolidated from slurry and sheared under undrained conditions (tests NAP40, ANP80, ANP160 and ANP320, Figs. A3.99: A3.102) could be compared with four other similarly consolidated samples which were sheared under drained conditions (tests ANP40D, ANP80D, ANP160D and ANP320D, Figs. A3.103: A3.106). The ESP's of these eight tests are plotted in Fig. (6.59). The ESP's of undrained tests are, by definition, contours of constant water content and the water content along the imposed ESP's of drained tests have been continuously monitored during each test as explained in chapter 4. The water content of each of the eight tests is indicated on their ESP's in Fig. (6.59).

It could be observed from the above figure that within the range of experimental scatter, the constant water content contours of drained and undrained tests agree well together. This fact suggests that a relationship between water content and effective stresses might exist irrespective of the type of test. (see Henkel 1960a and 1960b). This finding of pre-failure behaviour should then be added to the previously established relationship between the ultimate shearing strength and the final water content in this soil (see section 6.4).

The prediction of the relationship between pore pressures and total stresses in undrained tests from a set of drained tests (or indeed the prediction of the relationship between volume change and effective stresses in drained tests from a set of undrained tests) has been sufficiently dealt with by Henkel (1960) and will not be repeated. It has to be noted that Henkel (1960) used a different arrangement of axes from the ones used in this research.

The relationship between the drained and undrained behaviour of soils has been studied by other researchers, e.g. Atkinson (1973) looked at this relationship from an analytical point of view using both isotropic and anisotropic elasticity. Wesley (1975) related the early portions of his stress-strain curves, (elastic portions), of his drained and undrained tests on soft normally-consolidated clay from Mucking. He separated the isotropic tests from the anisotropic ones and the interpretation of his results using the theory of elasticity was not quite successful.

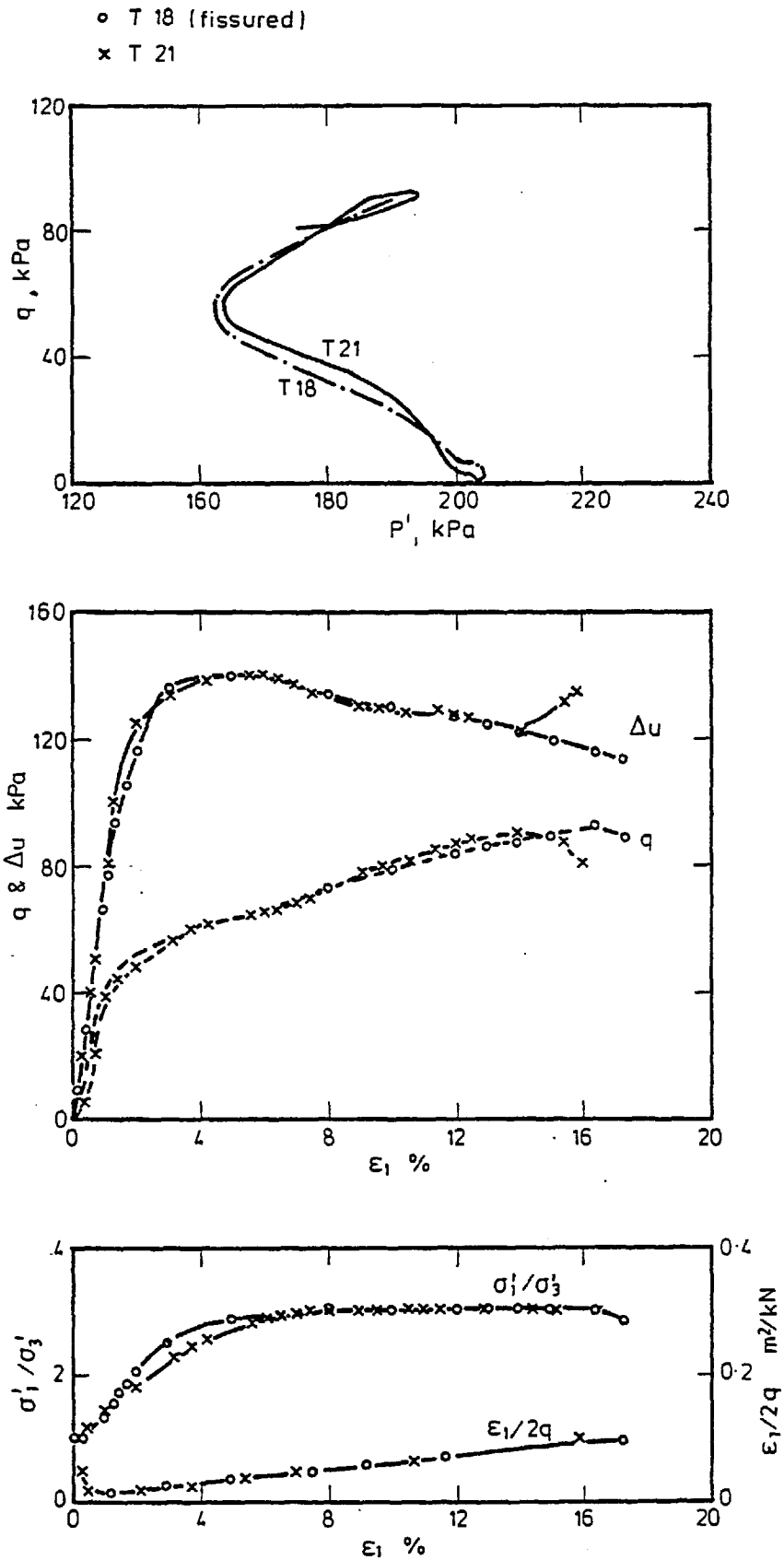


Fig. 6-53 A comparison between a fissured and a non-fissured remoulded isotropically consolidated sample of the Lower Till.

- INP 200 "Young"
- × ST1 "Aged"

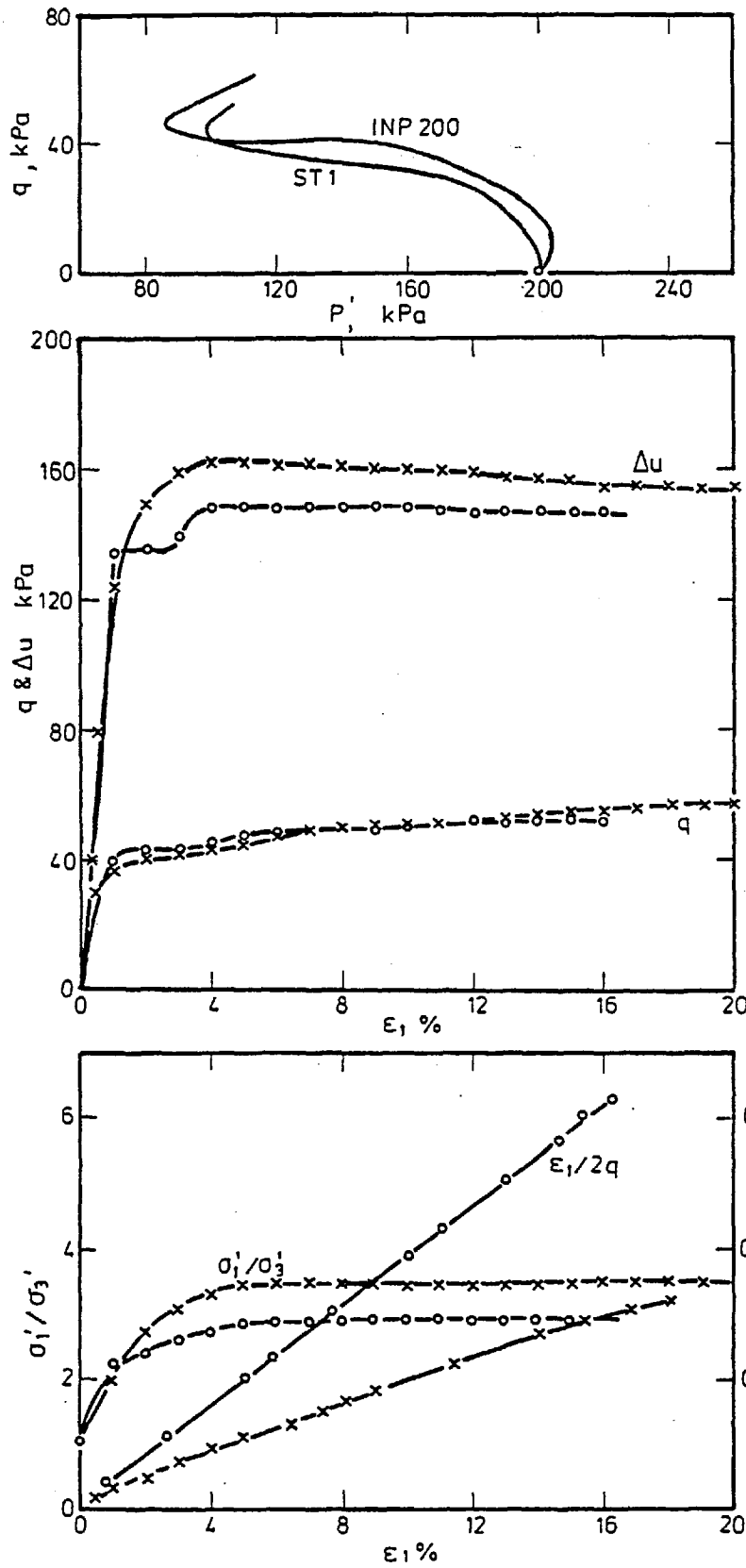


Fig. 6-54 A comparison between a "young" and an "aged" isotropically normally consolidated sample of the Lower Till.

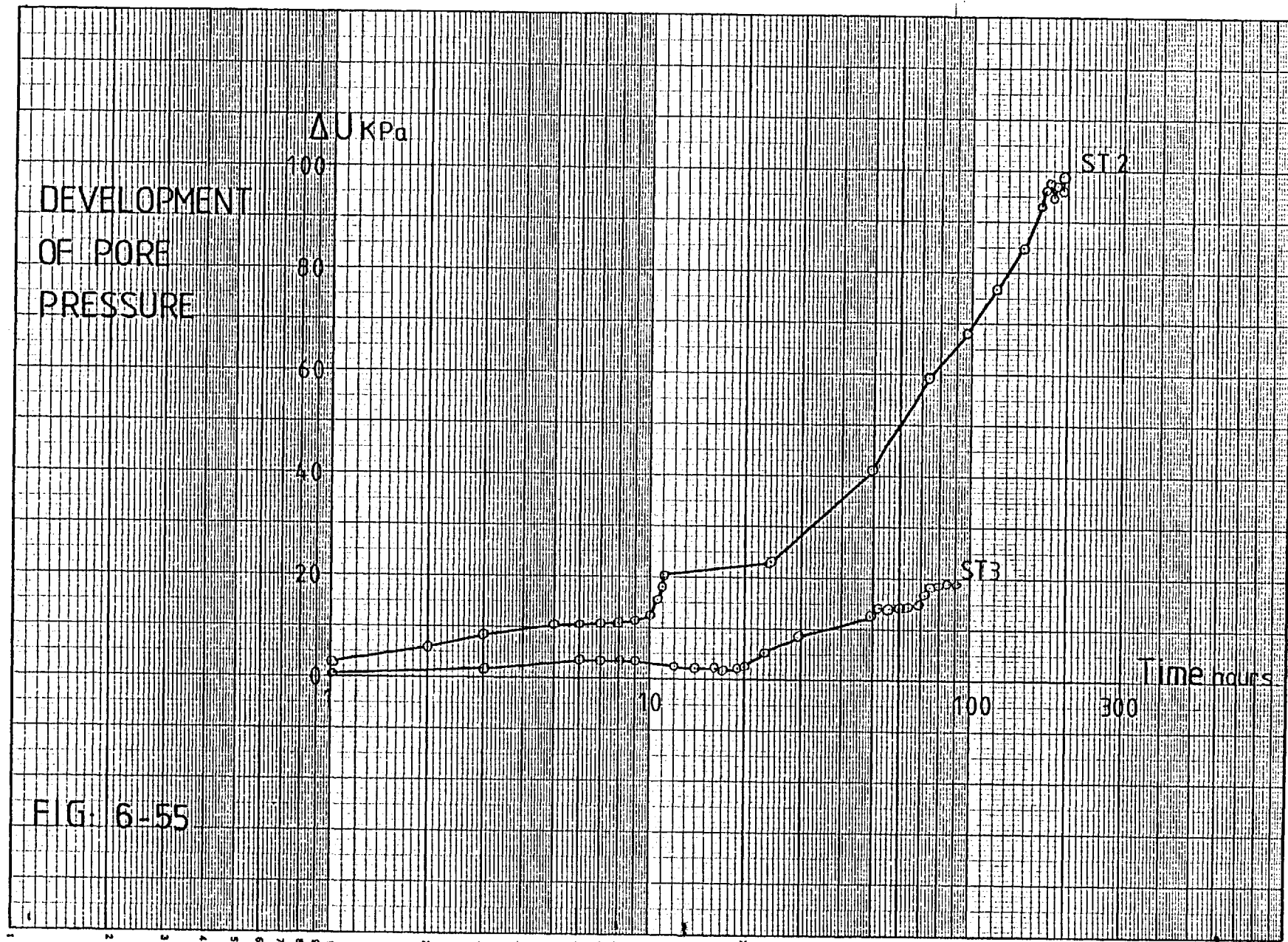


FIG. 6-55

- INP 200
- × ST2
- △ N.C. test (normalized)

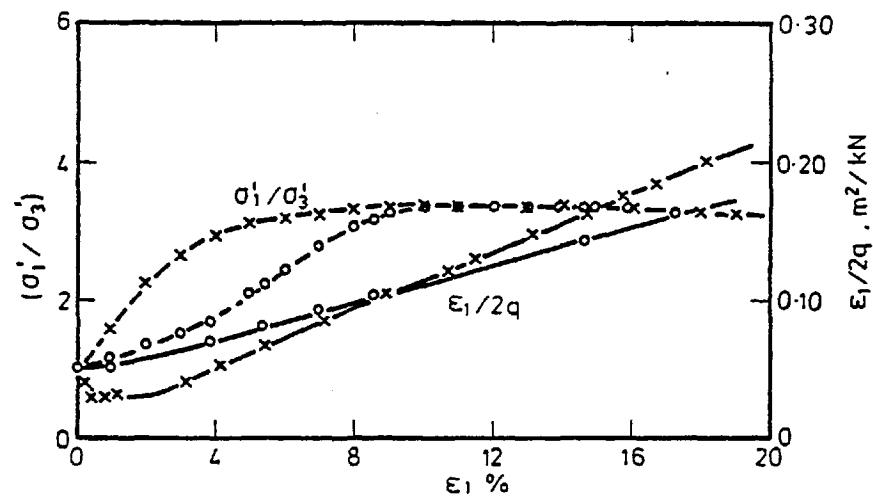
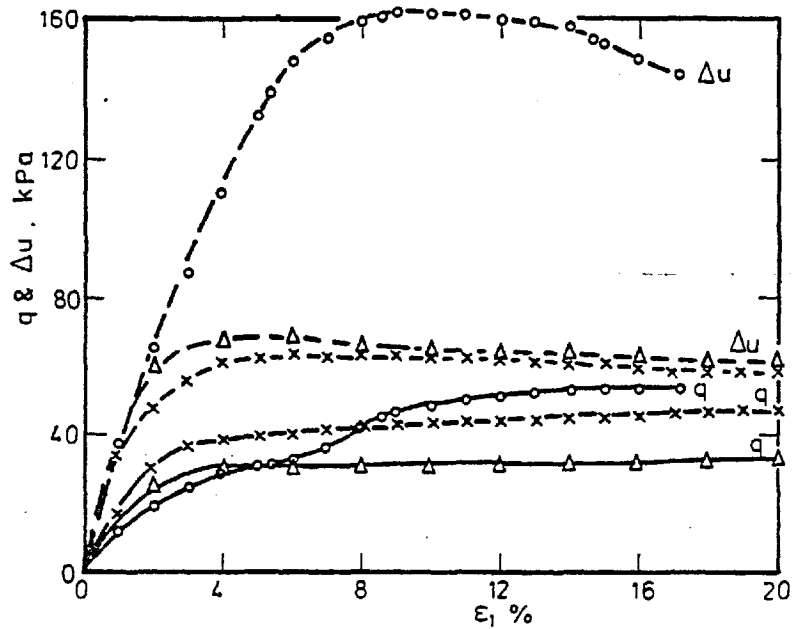
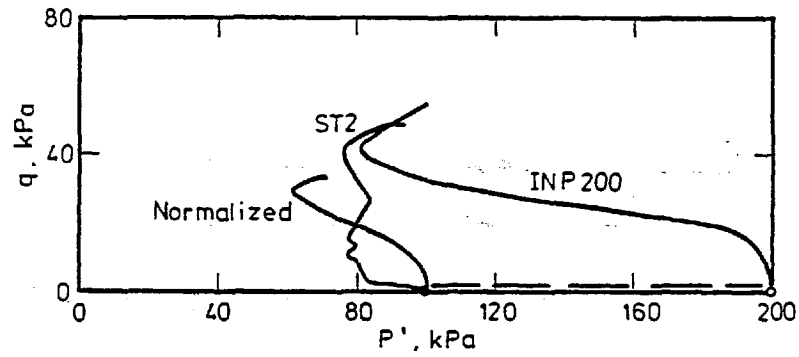


Fig. 6-56 A comparison between the behaviour of an isotropically consolidated resedimented sample which "creeped", normally consolidated sample and a normalized normally consolidated sample of the Lower Till.

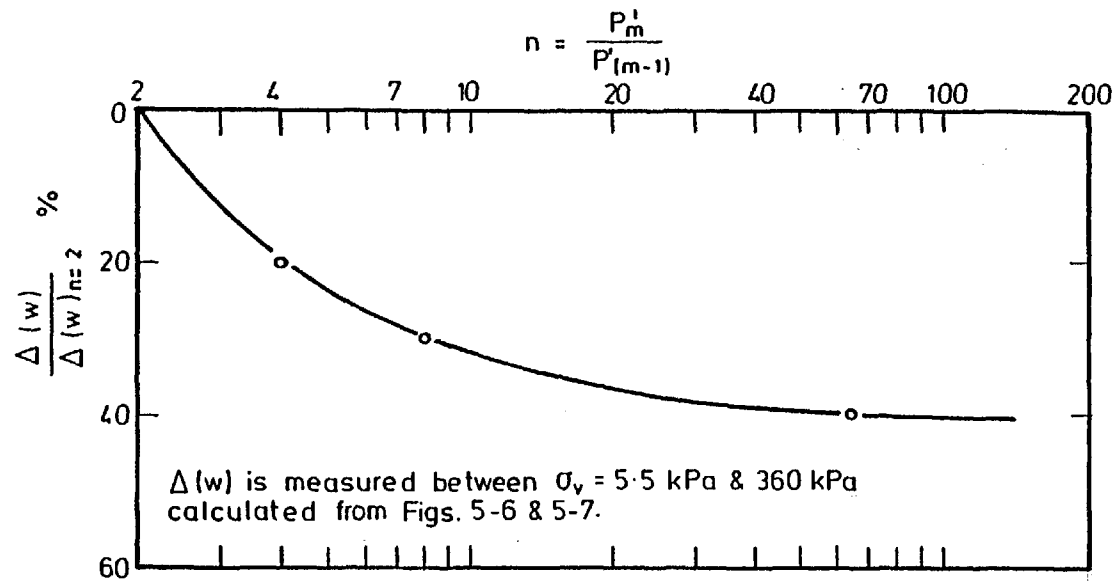


Fig. 6-57 The relationship between the magnitude of the increment of loading (n) and the volumetric change in oedometer tests on the Lower Till.

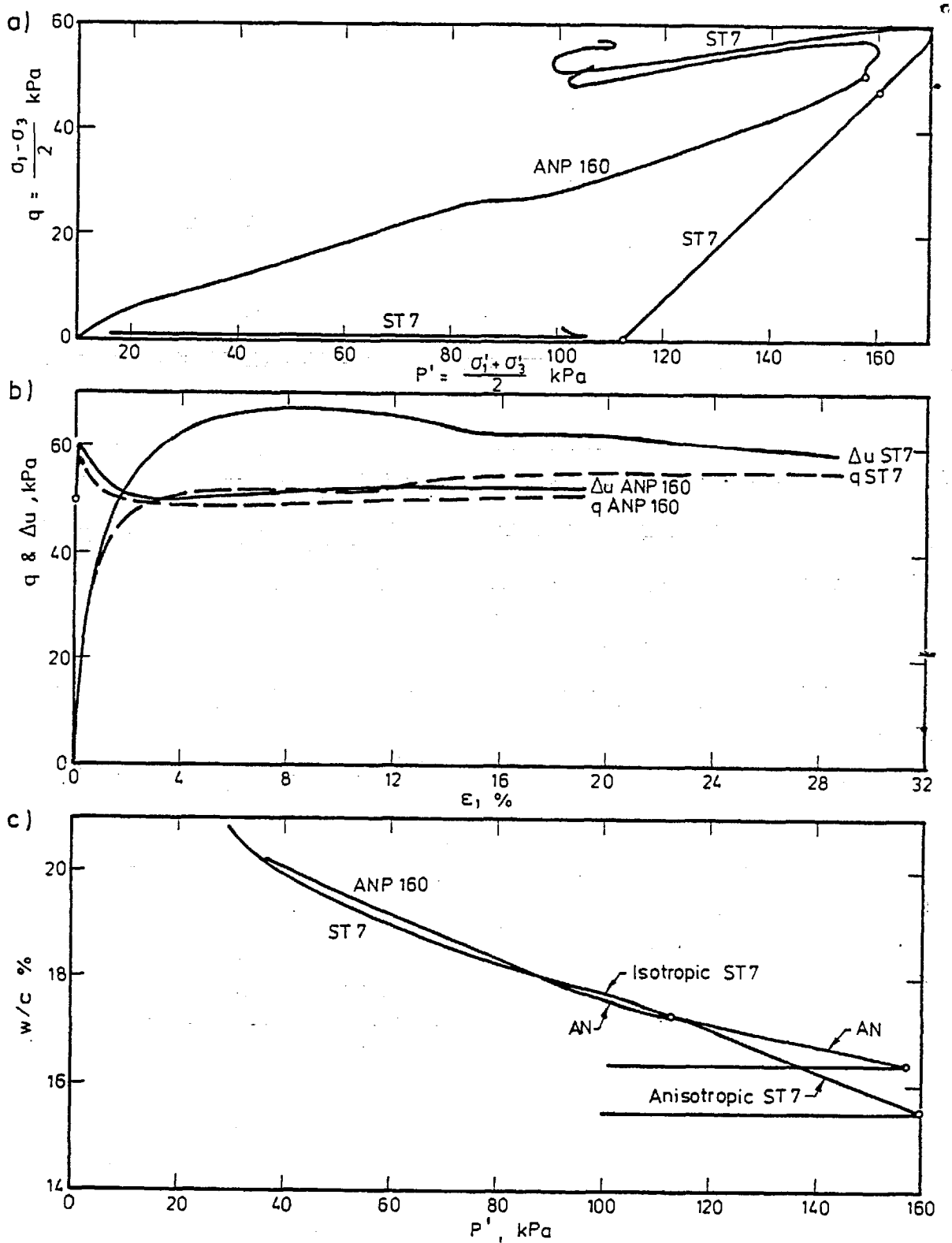


Fig. 6-58 Effect of consolidation stress path

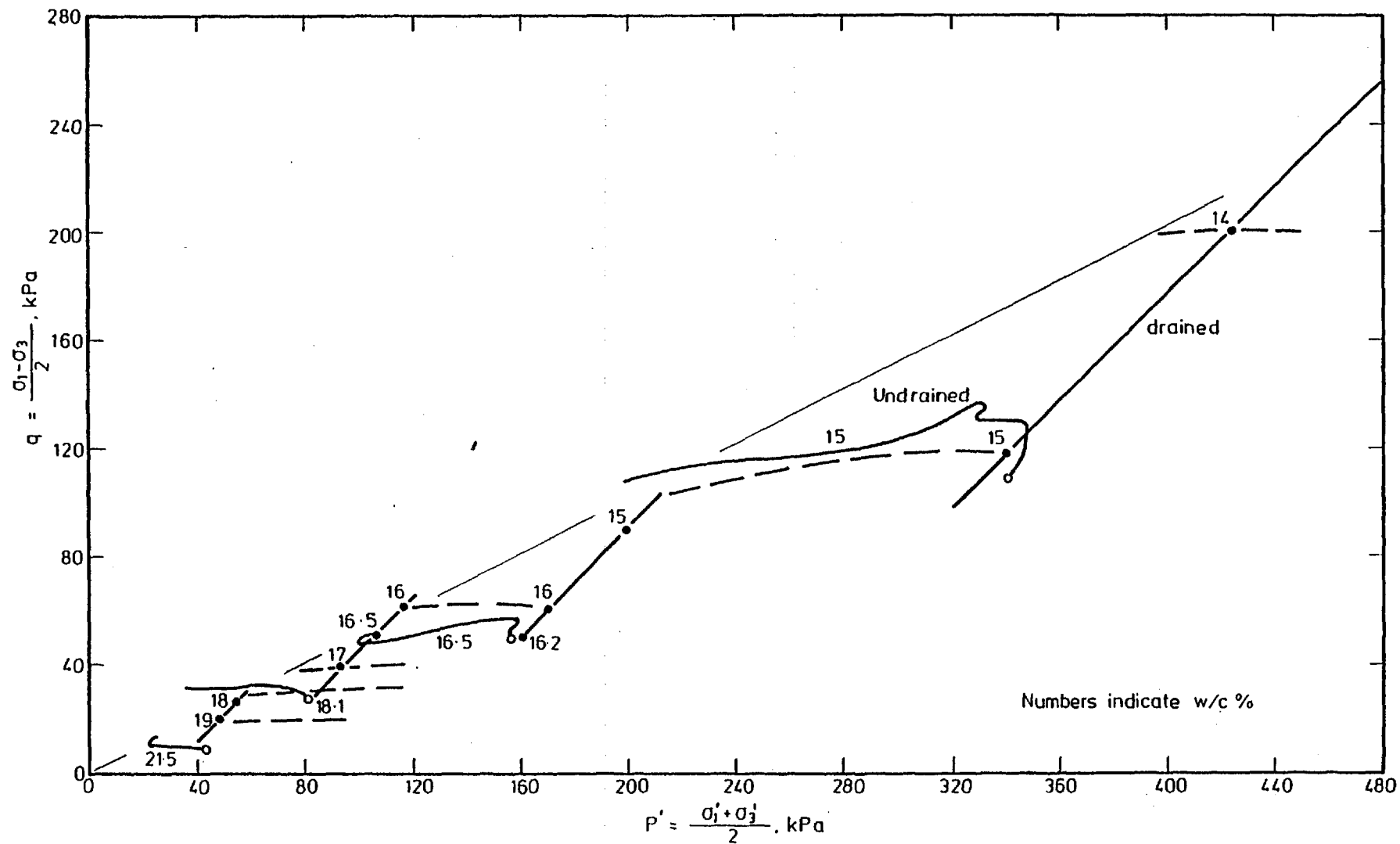


Fig. 6-59 Contours of water contents from drained and undrained anisotropically consolidated samples of the Lower Till.

CHAPTER 7

SOME PRACTICAL APPLICATIONS OF THE RESEARCH FINDINGS

7.1 SOIL PARAMETERS FOR DESIGN

7.1.1 Introduction

In this section an attempt is made to evaluate the required soil parameters for design in a strata of a low plasticity soil similar to the studied Lower Till. This evaluation is based on the information obtained in chapters 2,5 and 6. No attempt is made to study the methods of design per se.

The sedimentary history of the Lower Till in Norfolk is not precisely known, however, the stress history to which these soils have been exposed should lead to a behaviour similar to that of overconsolidated soils (see chapter 2 and Vaughan and Walbancke 1975). A soil sample could have been deposited by an infinite number of ways. The two bounds for sedimentation however, are firstly if it is deposited along a normal consolidation line then unloaded (Figs. 5.1-5.6), secondly if it reached its present state along a normal consolidation line without unloading. Only the first bound is considered in this section. The OCR of the tested Lower Till has been theoretically calculated to range between 1.84-4.22 in item 2.3.1. This range of the OCR has been experimentally confirmed from tests on "undisturbed" samples in item 6.3.1.

The field consolidation system is most likely to have been anisotropic. However the apparent overconsolidation would have caused an unloading that would have brought some of these soils to approach an in-situ isotropic state of stress. The anisotropic unloading path for the above range of OCR, is expected to be linear, as indeed was assumed throughout testing, and to lay within the "elastic" range of behaviour (see item 4.3.2.1.2 and Wroth 1975).

The value of the effective stress parameter ϕ' of the Lower Till tested is assumed to be unique for all practical purposes over the range of stresses of this research. This assumption is based on the information obtained in items 6.4.4, 6.5.6 and 6.7. There are pieces of evidence indicating that values of ϕ' higher than the "unique" values might exist in cases such as

- i) drained shearing by following an unloading stress path (such as SP2 where σ'_3 decreases and σ'_1 remains constant), item 6.3.2
- ii) undrained shearing by following a compression stress path (such as SP1 where σ'_3 remains constant and σ'_1 increases), if preceded by an undrained unloading, section 6.6.

The values of ϕ' in the above two cases are assumed to have been reflected by the values of $(\sigma'_1/\sigma'_3)_F$. However these high values of ϕ' will be disregarded and an

estimate of ϕ' is to be made from results of conventional triaxial compression tests although this might be on the conservative side in cases of unloading, excavations, etc.

To accumulate the required knowledge of the Lower Till parameters needed for design calculations, (mainly bearing capacity and deformation) some steps have to take place in the field and other steps in the laboratory. Guidance during this operation could be made by using the relationships produced in this research as will be subsequently explained, (refer to Fig. 7.1 throughout items 7.1.2 and 7.1.3).

7.1.2 "In-Situ" Investigation

1) Exploratory borings should be made in the proper locations and to adequate depths proportional to the dimensions of the required structure. Stratification and lithological sections obtained from these borings should enable a decision to be made regarding the proper foundation level.

2) It is very important to determine the value of the natural water content W_0 of the foundation soil in the field immediately after obtaining a "representative" sample from this layer. The values of the liquid limit LL, plastic limit PL and the bulk density γ_{bulk} should similarly be determined in the field. The measured value of W_0 could still have been slightly different from the natural W of the foundation strata because of the disturbance during sampling and the possible migration of

water from one level to another. Yet the value of W_0 measured as indicated above still stands as the best representative value for the natural strata. (The swelling of low plasticity soils is less than of high plasticity ones, yet any change in water content may seriously affect the measured strength.)

3) Plate loading tests should be carried out in the field to obtain the deformation characteristics of the soil. Field measurements of deformation are the only realistic measurements as recommended in sections 6.6 and 6.8. Values of the deformation modulus E should therefore be obtained as explained in item 6.8.4.

4) Quick sealing of the undisturbed samples should take place in the field to try to preserve the natural moisture inside the samples, (see item 4.1 and 6.6.3). Transportation and storage times should be minimised as much as possible before testing.

5) From the above information the value of vertical effective stress σ'_{vo} could be calculated. A quick check on the history of the soil could be made by locating the position of the point representing $W_0 - \sigma'_{vo}$ in Fig. 5.18 (Fig. 7.1a). If it does not lie on the normally-consolidated line (NCL) it is likely to be overconsolidated as explained in item 7.1.1. The values of the compression index C_c , coefficient of volume change m_v and the coefficient of compressibility a_v could also be calculated from Fig. 5.18 (see also item 6.3.2). The above parameters should be

calculated for the range of stresses appropriate to the ones to which the soil will be subjected. The value of the coefficient of consolidation C_v is to be estimated from Fig. 2.9 (Fig. 7.1b) and the value of the coefficient of permeability k should be calculated using the equation $k = C_v \cdot m_v \cdot \gamma_w$, where γ_w is the density of water. Should the value of the swelling index, C_s , be required it could be calculated from Fig. 5.1 (Fig. 7.1c).

7.1.3 Laboratory Investigation

This investigation is divided into two phases. In the first phase estimations of different soil parameters are made from the relationships obtained in this research with a limited number of conventional tests. In the second phase recommendations are made regarding stress path testing in the event of the proper equipment being available.

First Phase:

1) The value of the undrained shear strength C_u can be obtained from Fig. 6.39 (Fig. 7.1d) for the value of W_o obtained in the field. The relationship of Fig. 6.39 also holds for the final water content versus the drained shear strength in anisotropically overconsolidated samples, sections 6.5 and 6.9. The evidence of some bonding that exists in the "undisturbed" samples tested in item 6.3.2 does not affect the relationship of $W_o - C_u$ of Fig. 6.39. A slow UU test on a remoulded sample having the same water content as W_o should confirm the correctness of the

value of C_u obtained above, see Fig. 6.39, section 6.5. The sample in this test should be subjected to as large axial strains as is possible to obtain the correct value of C_u corresponding to its W , item 6.4.2. The values of C_u which are quoted correspond to a shearing rate $R_a = 0.005$ mm/min and as it is evident from item 6.4.2 that the effect of shearing rate on the value of C_u becomes more insignificant as the OCR approaches 4 in isotropically overconsolidated soils; it will be assumed here that the value of C_u obtained as above stands for all the practical range of shearing rates.

2) The value of C_u/σ'_{vo} should then be entered in Fig. 6.42a (Fig. 7.1e) to estimate a value for the OCR. If an undisturbed sample could be successfully mounted in the oedometer a confirmation of the value of the OCR could be obtained although the chances of carrying out such a test and getting meaningful results from it is rather slim. The value of σ'_{vm} could be readily calculated as soon as a value of OCR is obtained. The value of the excess pore pressure developed at failure can be calculated from the value of A_f and C_u . The value of A_f can be obtained from Fig. 6.42b (Fig. 7.1f) and later confirmed from Fig. 6.32b (Fig. 7.1i).

3) The value of $(\sigma'_1/\sigma'_3)_f = R'$ corresponding to the OCR obtained above can be estimated from Fig. 6.32a (Fig. 7.1g). The value of ϕ' (assumed unique and reflected by R' , item 7.1.1) is to be calculated from the equation $\phi' = \sin^{-1}(R'-1)/(R'+1)$. The UU triaxial test carried

out to confirm the value of C_u (point 1) should also confirm the value of ϕ' , assuming pore pressures are measured. The anisotropic consolidation line in the field is a K_0 -line where K_0 is the coefficient of earth pressure at rest and it could be estimated from the equation $K_0 = 1 - \sin \phi'$ (Jaky 1948 and Bishop 1958). The line that was followed during anisotropic unloading is defined by the coefficient K_{oc} which can be obtained from the relationship presented by Wroth (1975) and explained in item 4.3, equation 4.3. The value of the mean effective consolidation pressure at the start of shearing P'_0 should therefore be readily calculated, see Fig. 7.1h. From Fig. 6.48 (Fig. 7.1j) a value of the deformation modulus E can be estimated and compared with the value obtained from plate loading test (point 3, item 7.1.2). If the two values of E differ, the value obtained from the plate loading test in the field should be used.

Second Phase:

In this part the possibility of carrying out stress path testing is investigated. It might not be practical to carry out a series of tests simulating all the possible conditions of the soil in the field. A limited amount of carefully selected stress path testing should, therefore, be carried out.

The estimation of the OCR, K_0 , K_{oc} and P'_0 should be carried out primarily from the "first phase". Either undisturbed U samples or artificial samples (remoulded R or reconstituted from slurry S) could be used.

1) If undisturbed samples were used the state of stresses of the sample after mounting on the triaxial apparatus and being subjected to all round pressure would resemble P'_1 at point c of Fig. 7.1k. The sample is then reconsolidated to the "field stresses" at point b of Fig. 7.1k. The process of reconsolidating to the field stresses has been studied in section 6.6 (see Fig. A3-117) and the process described gives the best ESP resemblance to the "in-situ" condition. This has been done for normally consolidated soils and will be also applied in the present case (of overconsolidated soils). The consolidation process can be carried out continuously as explained in item 4.3 and this requires special equipment and control devices such as the "Hydraulic Triaxial Apparatus" described in item 4.2.1.2. If these pieces of equipment were not available, incremental consolidation can be carried out. It is recommended that the consolidation process described in item 4.2.1.3 (for oedometers) is followed. Care should be taken not to subject the sample to a loading increment ratio, n , of more than 2 (see item 6.9.5, fig. 6.57 and Fig. 7.1e). A larger value of n would seriously affect the final value of the water content and lead to misinterpretation of the value of shear strength. The value of C_u should be obtained after correcting for the difference between the final value of the water content in the test and W_0 , from Fig. 6.39. It is always expected that an exaggerated value of C_u would be obtained from such a test if the correction for the water content did not take place.

The shearing process from the "in-situ" stresses, point b, Fig. 7.1k, should follow whatever stress path is expected in the field.

2) If artificial samples were used, it is preferable to use reconstituted samples from slurry S as explained in item 4.3.1.1. If the required equipment was not available consolidating from slurry in an oedometer and then extracting cylindrical samples for the triaxial testing can be done as described by Gens (1978). The consolidation and unloading paths should follow oab in Fig. 7.1h (or Fig. 7.1k) and if the relationships are correct these consolidation and unloading paths should cause the sample to have a water content at point b, Fig. 7.1h (or Fig. 7.1k) equal (or near) to W_0 . Shearing from point b should, as mentioned above, follow the required stress path and correction for any difference in the water contents should also be carried out from fig. 6.39 as explained above. Using samples remoulded R at water contents lower than the Liquid Limit should be dealt with carefully. It is evident from Fig. 5.8 that unless the sample is wet (probably wetter than can stand unconfined during preparation in the triaxial apparatus) or if the value of σ_{vm}' is high no convergence between the remoulded line and the normally consolidated line, NCL, will take place. Without convergence to the NCL any unloading of the remoulded sample to σ_{v0}' will give unreliable results. It is, therefore essential to secure one or both of the above two conditions before using remoulded samples. If the

field "in-situ" stresses were at (or near) the isotropic state of stresses ($q = 0$) and only conventional equipment were available, isotropically overconsolidated samples with the same OCR and P'_o will give fairly good resemblances of ESP as shown in Figs 6.36 and 6.37. However deformation and pore pressure response will most likely differ in these cases from the anisotropically overconsolidated samples.

7.2 RELATIVE VALUES OF LOADS, MOMENTS AND DIMENSIONS SUGGESTED FOR OFF-SHORE GRAVITY STRUCTURES IN THE NORTH SEA AND FOR MODEL TESTING

7.2.1 General

In this section a guide is given to the choice of design loads (including moments) and dimensions of an off-shore gravity structure in the North Sea. The same principles could also be applied to model testing. Model testing experience at Imperial College which simulates the conditions of an off-shore gravity structure at the North Sea is described in Herbertson (1977) and Martins (1977).

This section is not derived from the experimental programme of this thesis. It contains results predicted from known bearing capacity theories, (e.g. Brinch Hansen 1961 and 1970 and Meyerhof 1963), and from relationships of contact pressures beneath off-shore gravity structure which were derived by Vaughan (1974) and by the author. Use is made of the scarcely available arithmetical data of some of the existing off-shore gravity structure in the North Sea, table 7.1.

The distribution of forces, pressures and moments on the foundation level of an off-shore gravity structure in the North Sea is shown in Fig. 7.2. This distribution is based on the assumption that the structure is subjected to a certain design wave, (the factors affecting the change in pressure on the sea bed due to the passage of surface waves are described by Henkel 1970 and the wave loads on structure are described by Hogben 1976). In the case of the North Sea the design wave is 600 m in length, 24-30 m high and its time period 14-18 seconds (Bjerrum 1973).

No attempt has been made, within the scope of this section, to investigate the critical rupture planes under the above-mentioned combinations of straining actions as was done by Bent Hansen (for Bjerrum 1973) by trial and error or by introducing a shear strength which varies with depth (Davies and Booker 1973 and Gibson 1974) as was done by Vaughan et al. (1976). The investigation which is done in this section assumes a known value of the shear strength of the foundation soil which is checked against the loads and moments of a structure with a known foundation base dimensions.

7.2.2 Values Obtained from Bearing Capacity Calculations

The bearing capacity formula used in this section is the revised and extended formula presented by Brinch Hansen (1970). It is assumed here that the structure rests directly on the sea bed and that both its foundation base and sea bed are horizontal. The undrained loading of clays

is taken to be the critical design case (see chapter 2 and Vaughan et al. 1976). The bearing capacity formula is:

$$\frac{Q}{A_e} = C_u N_c (1 + s_c - i_c)$$

where

- Q = ultimate bearing capacity per unit length of the foundation
- A_e = effective area
- C_u = undrained shear strength
- N_c = $(\pi + 2)$ = bearing capacity factor
- s_c = shape factor
- i_c = inclination factor

The principle of using an effective (or equivalent) area instead of the real area in case of eccentric loading has long been used (e.g. Brinch Hansen 1961 and 1970 and Meyerhof 1963). The idea is to establish a rectangular area $B \times L$ whose centre coincides with the point of application of the resultant of all the forces applied to the foundation base. The sides of this effective area should follow as closely as possible the nearest contours of the actual base area. Frimann Clausen (1973) presented two figures, Fig. 7.3a and b, relating the eccentricity of the loads and the diameter of a circular foundation area to the values of B and L of their effective area. Janbu et al. (1976) presented an analytical method to calculate B and L (B and B_0 in his paper). The author found Janbu's method

to be too conservative when its results were compared to the results obtained by the method suggested by Brinch Hansen (1970) and that the results from the relationships by Frimann Clausen to be closer to Brinch Hansen's method. The latter method is used in this section.

The inclination factor i_c is calculated from the equation:

$$i_c = 0.5 - 0.5 \sqrt{1 - \frac{H}{A_e C_u}} \quad (7.2)$$

where H = horizontal component of the resultant of the applied loads

The formula was presented in Brinch Hansen (1961) based on previous findings by himself, Meyerhoff (1953) and others. It has been used in the analysis of off-shore gravity structure by the NGI team and will be used in this section. The critical value of i_c is 0.5 where $H = A_e \cdot C_u$, a condition in which failure by sliding is imminent. If $H/A_e C_u$ exceeds unity sliding will occur. In Figs. 7.4 and 7.5 calculations were made using the relationships of Fig. 7.3 and the critical condition where $H = A_e \cdot C_u$. If the moment M , the vertical load V , the horizontal load H and the diameter of the foundation base D were known, the eccentricity $e = M/V$ and the values of the sides of the effective are B and L are readily calculated from Fig. 7.3, hence $A_e = B \cdot L$. A check on the safety of these loads against sliding could be made from Figs. 7.4 and 7.5 according to the design value of the undrained shear strength C_u . If from this preliminary

check the combination of loads gave an unsafe condition a change in one or more of these parameters could be made in the early stages of the design. Failure by sliding in off-shore gravity structures in the North Sea could be the deciding factor in whether to use gravity structure or other forms of structures. The cost of securing a vertical load that would ensure stability or the choice of a foundation soil of high enough strength could prohibit the use of gravity structure in a particular site. This problem has been recognised by Bjerrum (1973) in his bearing capacity analysis and by Vaughan et al. (1976) in their finite elements analysis.

The shape factor s_c has been given by Brinch Hansen (1970):

$$s_c = 0.2 i_c \frac{B}{L} \quad (7.3)$$

Brinch Hansen (1970) also gave another formula for s_c based on vast data of undrained loading tests on clay:

$$s_c = 0.2 \frac{B}{L} \quad (7.4)$$

Frimann Clausen (1973) used the formula:

$$s_c = 0.2 (1 - 2 i_c) \frac{B}{L} \quad (7.5)$$

The above formula (7.5) reduces to zero in the critical case of sliding where $i_c = 0.5$. Equation (7.5) gives lower values of s_c at high values of i_c and higher values

of s_c at low values of i_c than equation (7.3) .

Equation (7.3) will be used in this section.

The dead and live loads exerted on the foundation are assumed to act statically. However if a dynamic analysis is to be carried out, it would be more economical to include an additive term, at the end of the bearing capacity equation (7.1), which describes the pressure distribution on the sea bed due to the wave action.

The ratios of the vertical loads V , horizontal loads H and the moments M of off-shore gravity structure in the North Sea lie within the ranges, table 7.1:

$$\text{i) } M/V = 0 - 20 \quad (= \text{eccentricity } e) \quad (7.6)$$

$$\text{ii) } M/H = 0 - 60 \quad (7.7)$$

$$\text{iii) } H/V = 0 - 0.5 \quad (7.8)$$

It has to be noted here that in the case of model testing the effective area A_e has to be calculated by trial and error since, without extrapolation, Frimann Clausen's method, fig. 7.3, applies to large diameters only (e.g. if $M/V = e = 5$, the smallest diameter required would be $5/0.5 = 10$ meters).

To summarise this section an example of the calculations which are proposed is given in table 7.2. It is assumed that the data available includes the possible diameter of the foundation base ($D = 100$ meters) and the critical vertical load ($V = 2.3 \times 10^3$ MN). From this information a preliminary check is required to ensure that

the ratios M/V , M/H and H/V are compatible and safe against bearing capacity failure for a chosen value of undrained shear strength C_u . Columns 1-3 in table 7.2 include the likely ratios mentioned above. Columns 4-5 are obtained from Fig. 7.3. Columns 6 and 7 are simple derivations from the data. In columns 8-11 calculations of the term $H/A_e C_u$ are carried out for different values of C_u . The values of $H/A_e C_u$ denoted by an "asterisk" * are greater than unity and the structure in this case will fail by sliding. It is observed that the values of C_u are generally high, however, Bjerrum (1973) and Eide (1974) reported values of C_u of about 500 kPa from tests on clayey strata in some sites in the North Sea.

Pursuing the above results, it is assumed that case number 4 with $C_u = 150$ kPa is to be investigated further. Substituting in equations (7.2) and (7.3), $i_c = 0.41$ and $s_c = 0.07$ and substituting in equation (7.1):

$$q_{ult} = 508.86 \text{ kPa} \quad (7.9)$$

If the functions f_1 and f_2 (see Fig. 7.1) are assumed, 0.2 and 0.5 respectively, the maximum load on the edge of the off-shore structure would be $1.3 P$ where P is the uniform pressure inside the structure $= 4V/\pi D^2$

$$\therefore P_{max} = 380.70 \text{ kPa} \quad (7.10)$$

The factor of safety F against bearing capacity failure in this case would be

$$F = 1.34 \quad (7.11)$$

If the structure is subjected to axial loads only, the ultimate bearing capacity would be

$$q = C_u N_c = 771.00 \text{ kPa} \quad (7.12)$$

and the factor of safety F_1 against bearing capacity failure in this case would be

$$F_1 = 2.63 \quad (7.13)$$

It is noted that the bearing capacity is reduced by almost a half when the structure is subjected to the action of waves. The undrained shear strength mobilised C_{mob} during the action of waves would be at its maximum when $F = 1$

$$C_{mob} = \frac{1.3P}{N_c(1 + s_c - i_c)} \quad (7.14)$$

$$C_{mob} = 112.22 \text{ kPa} \sim 75\% C_u \quad (7.15)$$

These calculations could be repeated until a satisfactory solution is achieved.

7.2.3 Values from Contact Pressure Calculations

When the wave loads are applied on an off-shore structure the foundation soil will respond to the resulting loading, unloading (to "at rest" position) and reloading according to its deformation characteristics. The topography of the site and the configuration of the foundation base will also affect these responses. It is possible to map the sea floor with an accuracy of 0.5 m and Eide (1974) reported an accuracy of 0.1 m from results of soundings carried out by the N.G.I. in the North Sea. In this section critical ratios of M/V as well as values for contact pressures and settlements beneath an off-shore gravity structure are calculated by using linear elasticity. This has been done by Vaughan (1974) for a rigid strip footing (2-dimensional) and will be extended here to a rigid circular footing in 3-dimensions. Vaughan (loc. cit.) assumed a number of assumptions which will also be used in this section.

- 1) linear elastic behaviour for soils and structures,

- 2) a non-reversing modulus of subgrade reaction K_1 and that the subgrade reaction is the same in unloading and reloading K_2 , Fig. 7.6,

- 3) the cyclic moments build up progressively and the rocking remains symmetrical,

- 4) at maximum extent of rocking, the contact stress on the loaded side is just equivalent to the first loading,

- 5) the settlements and contact pressure distributions are always linear,
- 6) the "at rest" settlements and contact pressures after the application of waves may be different from those during the application of waves,
- 7) flat sea bed and flat foundation base.

The conclusions of Vaughan's analysis are given in Appendix (4). From these conclusions the following relationships are extracted, (see Fig. 7.7):

$$\rho_r = \rho_c \max \quad (7.16)$$

and

$$q_{cr} = q_c \max \quad (7.17)$$

where r denotes "at rest"

c denotes centre

The negative pressure at the edge will occur if $q_{e \min} = 0$, the critical values of M/V are therefore:

$$\frac{M}{V} = \frac{B}{3(3 - \frac{K_1}{K_2})} \quad (7.18)$$

$$\frac{M}{V} = \frac{B}{9} \quad \text{if } K_2 \rightarrow \infty \quad (7.19)$$

where B is the breadth of the strip foundation, Fig. 7.7.

The maximum contact pressure difference at rest:

$$q_{cr} - q_{er} = \frac{6M}{B^2} \left(1 - \frac{K_1}{K_2}\right) \quad (7.20)$$

$$q_{cr} - q_{er} = \frac{6M}{B^2} \quad \text{if } K_2 \rightarrow \infty \quad (7.21)$$

where e denotes, edge

The ratio of the "at rest" settlement ρ_r to be static settlement ρ_i (case of vertical loads only, $\rho_i = V/B.K_1$):

$$\frac{\rho_r}{\rho_i} = 1 - \frac{3M}{VB} \left(\frac{K_1}{K_2} - 1\right) \quad (7.22)$$

$$\frac{\rho_r}{\rho_i} = 1 + \frac{3M}{VB} \quad \text{if } K_2 \rightarrow \infty \quad (7.23)$$

The model used to evaluate the relationships in case of a rigid circular foundation which is subjected to waves from all directions is shown in Fig. 7.8. It is assumed in this analysis that equations (7.16) and (7.17) are valid and the settlements and contact pressures will be as shown in Fig. 7.9. The derivations of the equations of this model are given in Appendix (4) and the following relationships are from the general conclusions:

$$\text{At } q_{e \text{ min}} = 0,$$

$$\frac{M}{V} = \frac{3r_0}{4\left(4 - \frac{K_1}{K_2}\right)} \quad (7.24)$$

$$\frac{M}{V} = \frac{3r_o}{16} \quad \text{if } K_2 \rightarrow \infty \quad (7.25)$$

The maximum contact pressure difference "at rest",

$$q_{cr} - q_{er} = \frac{4M}{\pi r_o^3} \left(1 - \frac{K_1}{K_2}\right) \quad (7.26)$$

$$q_{cr} - q_{er} = \frac{4M}{\pi r_o^3} \quad \text{if } K_2 \rightarrow \infty \quad (7.27)$$

The ratio of the "at rest" settlement to the "static" settlement,

$$\frac{\rho_c}{\rho_i} = 1 - \frac{8M}{3Vr_o} \left(\frac{K_1}{K_2} - 1\right) \quad (7.28)$$

$$\frac{\rho_c}{\rho_i} = 1 + \frac{8M}{3Vr_o} \quad \text{if } K_2 \rightarrow \infty \quad (7.29)$$

The model used to evaluate the relationships in case of a rigid circular foundation which is subjected to waves from one direction only is shown in Fig. 7.10. The mathematical analysis is in Appendix (4) and the following relationships are from the general conclusions:

$$\text{At } q_{e \min} = 0,$$

$$\frac{M}{V} = \frac{3r_o}{4(4.75 - 1.73 K_1/K_2)} \quad (7.30)$$

$$\frac{M}{V} = \frac{3r_o}{19} \quad \text{if } K_2 \rightarrow \infty \quad (7.31)$$

The maximum contact pressure difference "at rest",

$$q_{cr} - q_{er} = \frac{4M}{3\pi r_o^3} (2.93 - 3.01 K_1/K_2) \quad (7.32)$$

$$q_{cr} - q_{er} = \frac{11.72M}{3\pi r_o^3} \quad \text{if } K_2 \rightarrow \infty \quad (7.33)$$

The ratio of the "at rest" settlement to the static settlement,

$$\frac{\rho_c}{\rho_i} = 1 - \frac{4M}{3r_o V} (1.28 K_1/K_2 - 1.28) \quad (7.34)$$

$$\frac{\rho_c}{\rho_i} = 1 + \frac{5.12M}{3r_o V} \quad \text{if } K_2 \rightarrow \infty \quad (7.35)$$

In table 7.3 the above relationships are applied to the data of the off-shore gravity structures of table 7.1. A similar exercise could be carried out in other proposed designs of off-shore structures are in choosing proper loads for model testing. In table 7.4 a comparison has been done, by Davachi (1978), between the elastic solution and a finite element solution gave generally more critical values of the maximum and minimum settlements and contact pressures at the edges, $\rho_{e \max}$, $\rho_{e \min}$, $q_{e \min}$ and $q_{e \max}$. However the elastic solution can well give a preliminary idea about these parameters which is the aim of this section.

For the design of off-shore gravity structures, or when loads and moments are chosen to be applied on a model test representing the above structures, use could be made of the studies of items 7.22 and 7.2.3 to choose preliminary ratios of M, V and H. These ratios should ensure safety against bearing capacity failure and should also give acceptable values of settlements and contact pressure distribution. For detailed designs, however, more sophisticated methods of analysis should be used.

Reference	Moment MN.m	Vertical Load V MN	Horizontal Load H MN	Diameter D m
Duncan 1972	-	2×10^3	7.6×10^2	-
Eide 1974	2×10^4	2×10^3	5.0×10^2	-
Clausen 1974	1×10^4	2×10^3	7.5×10^2	100
Shell-Bernt 1974	3.6×10^4	3.78×10^3	13.7×10^2	150 equivalent
Example by Janbu et al. 1976	2.9×10^4	1.9×10^3	5×10^2	~100 equivalent
Bjerrum 1973 (Ekofisk)	2.83×10^4	1.9×10^3	7.86×10^2	~93

Table 7.1 Values of Loads and Moments and Dimensions
of some Off-shore Gravity Structure

Column No.	1	2	3	4	5	6	7	8	9	10	11
Case No.	$\frac{M}{V}_m$	$\frac{H}{V}$	$\frac{M}{H}_m$	$\frac{M}{VD}$	$A_{e_m^2}$	V MN	H MN	$C_u=100$ kPa $H/A_e C_u$	$C_u=150$ kPa $H/A_e C_u$	$C_u=200$ kPa $H/A_e C_u$	$C_u=300$ kPa $H/A_e C_u$
1	5		20	0.05	5530			1.04*	0.69	0.52	0.35
2	10	0.25	40	0.10	4500		575	1.28*	0.85	0.64	0.43
3	15		60	0.15	3500			1.64*	1.10*	0.82	0.55
4	5		14.3	0.05	5530			1.46*	0.97	0.73	0.49
5	10	0.35	28.6	0.10	4500	2.3×10^3	805	1.79*	1.19*	0.89	0.60
6	15		42.9	0.15	3500			2.30*	1.53*	1.15*	0.77
7	5		11.1	0.05	5530			1.87*	1.25*	0.94	0.62
8	10	0.45	22.2	0.10	4500		1035	2.30*	1.53*	1.15*	0.77
9	15		33.3	0.05	3500			2.96*	1.97*	1.48*	0.99

*Failure by sliding will occur

Table 7.2 An Example of the Proposed Calculations for Bearing Capacity
($D = 100$ m and $V = 2.3 \times 10^3$ MN)

(a)

References	Critical M/V from Analysis of Circular Foundations(m)			
	Waves from all directions		Waves from one direction	
	$K_2 = 5K_1$	$K_2 = \infty$	$K_2 = 5K_1$	$K_2 = \infty$
Friman Clausen 1973	9.87	9.38	8.51	7.89
Bjerrum 1973	9.28	8.81	8.00	7.42
Eide 1974	9.87	9.38	8.51	7.89
Shell-Bernt 1974	14.80	14.06	12.77	11.84
Janbu 1976	9.87	9.38	8.51	7.89

(b)

	$(q_{cr}-q_{er})_{max}$ From Analysis of Circular Foundations(kPa)			
	Waves from all directions		Waves from one direction	
	$K_2 = 5K_1$	$K_2 = \infty$	$K_2 = 5K_1$	$K_2 = \infty$
Friman Clausen 1973	81.49	101.86	79.04	99.48
Bjerrum 1973	277.65	347.06	269.32	338.96
Eide 1974	162.97	203.72	158.09	198.96
Shell-Bernt 1974	86.92	108.65	84.31	106.11
Janbu 1976	236.31	295.39	229.22	288.50

(c)

	% increase of ρ_c than ρ_i From Analysis of Circular Foundations			
	Waves from all directions		Waves from one direction	
	$K_2 = 5K_1$	$K_2 = \infty$	$K_2 = 5K_1$	$K_2 = \infty$
Friman Clausen 1973	21.33	26.67	13.33	17.07
Bjerrum 1973	67.61	84.51	42.25	54.09
Eide 1974	42.67	53.33	26.67	34.13
Shell-Bernt 1974	27.09	33.86	16.93	21.67
Janbu 1976	65.12	81.40	40.70	52.10

Table 7.3 Application of the Contact Pressure and Settlement Analysis on Some Off-shore Gravity Structures in the North Sea

Parameter	Elastic Analysis, Vaughan (1974)	Finite Elements Analysis, Davachi (1978)
$\rho_e \text{ max}$	1.00 cm	1.30 cm
$\rho_e \text{ min}$	0.60 cm	0.40 cm
$q_e \text{ max}$	50.00 kPa/m'	65.00 kPa/m'
$q_e \text{ min}$	30.00 kPa/m'	27.00 kPa/m'

assumptions: $K_o = 1$
 $M = 0.48 \text{ MN.m/m'}$
 $V = 0.48 \text{ MN/m'}$
 $B = 12.00 \text{ m}$
 $K_1 = 5 \text{ MN/m'}$
 $K_2 = \infty$

Table 7.4 Comparison between Linear Elastic and
Finite Elements Analysis of Rigid
Strip Foundations

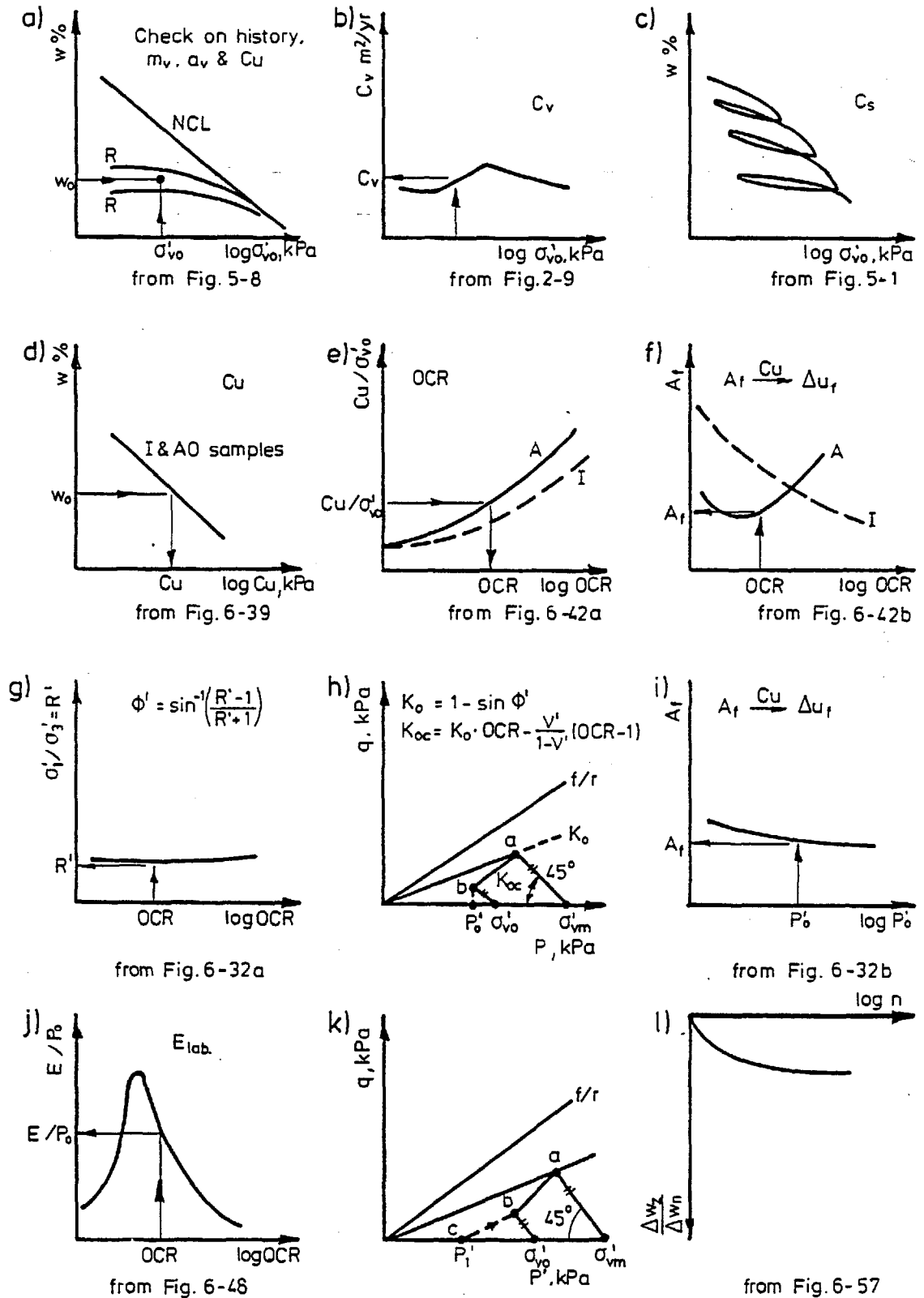


Fig.7-1 Schematic figures showing the sequence of obtaining the soil parameters of the Lower Till.

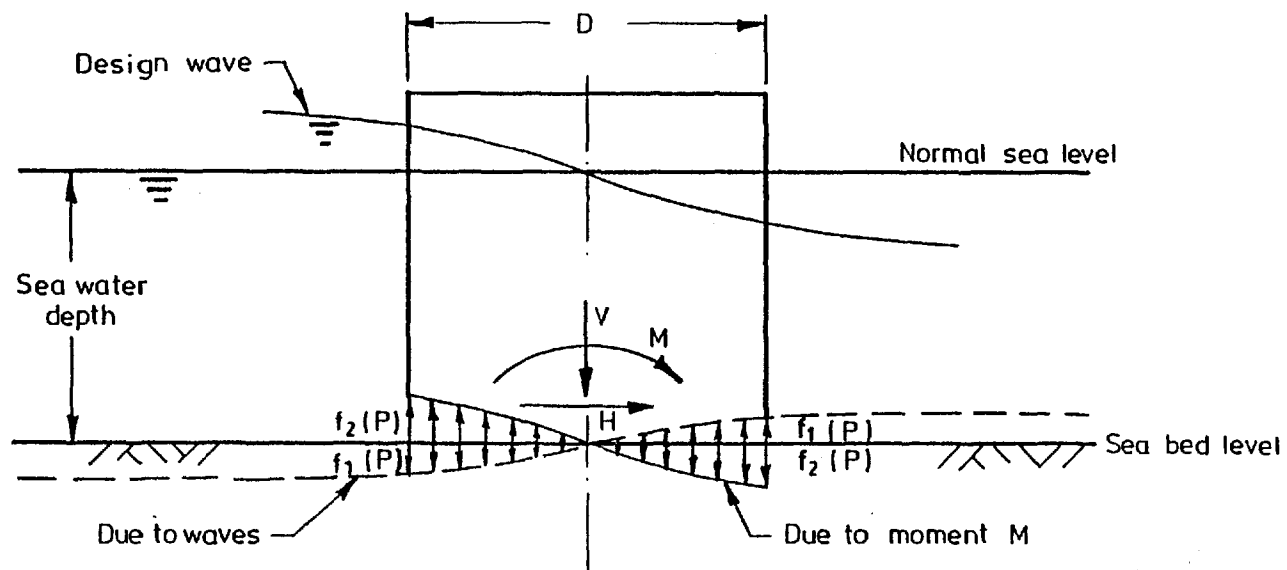


Fig.7-2 Distribution of loads and moments during the application of a design wave in an off-shore gravity structure.

Effective area $A_e = BL$

$e = M/V$

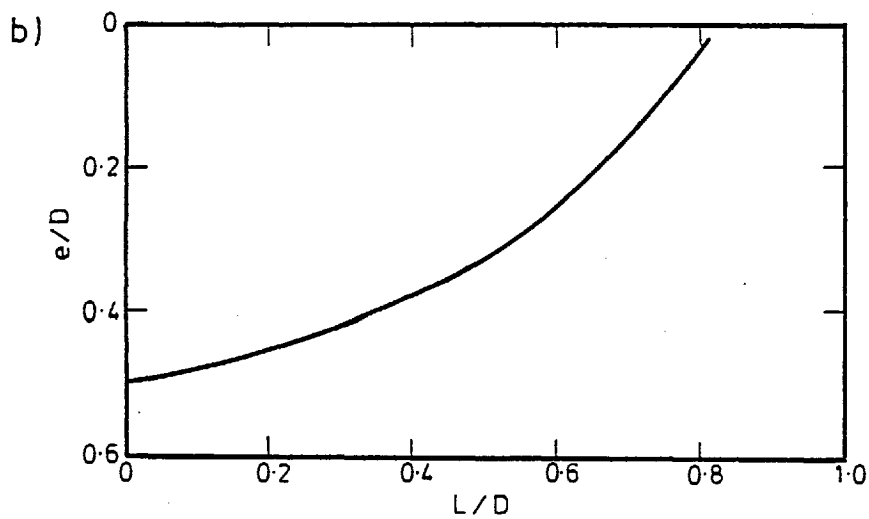
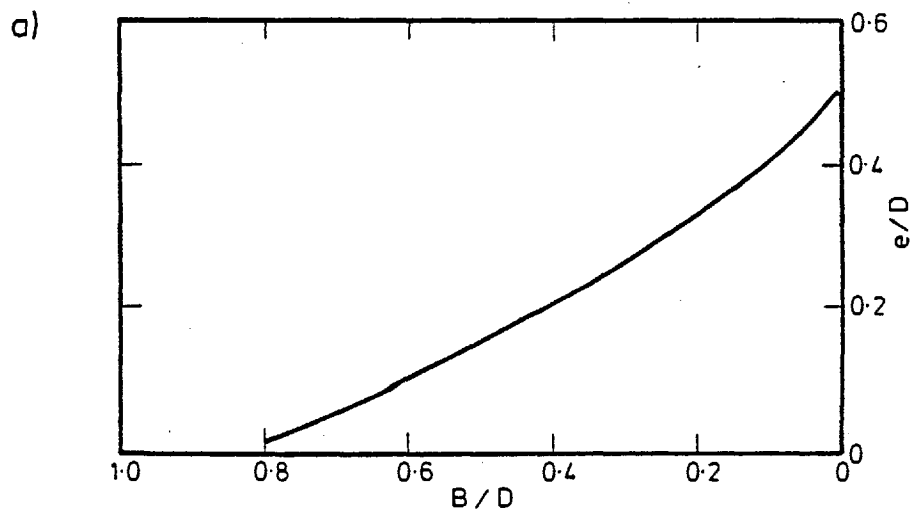


Fig.7-3 Calculations of the dimensions of the effective area
(after Clausen 1973)

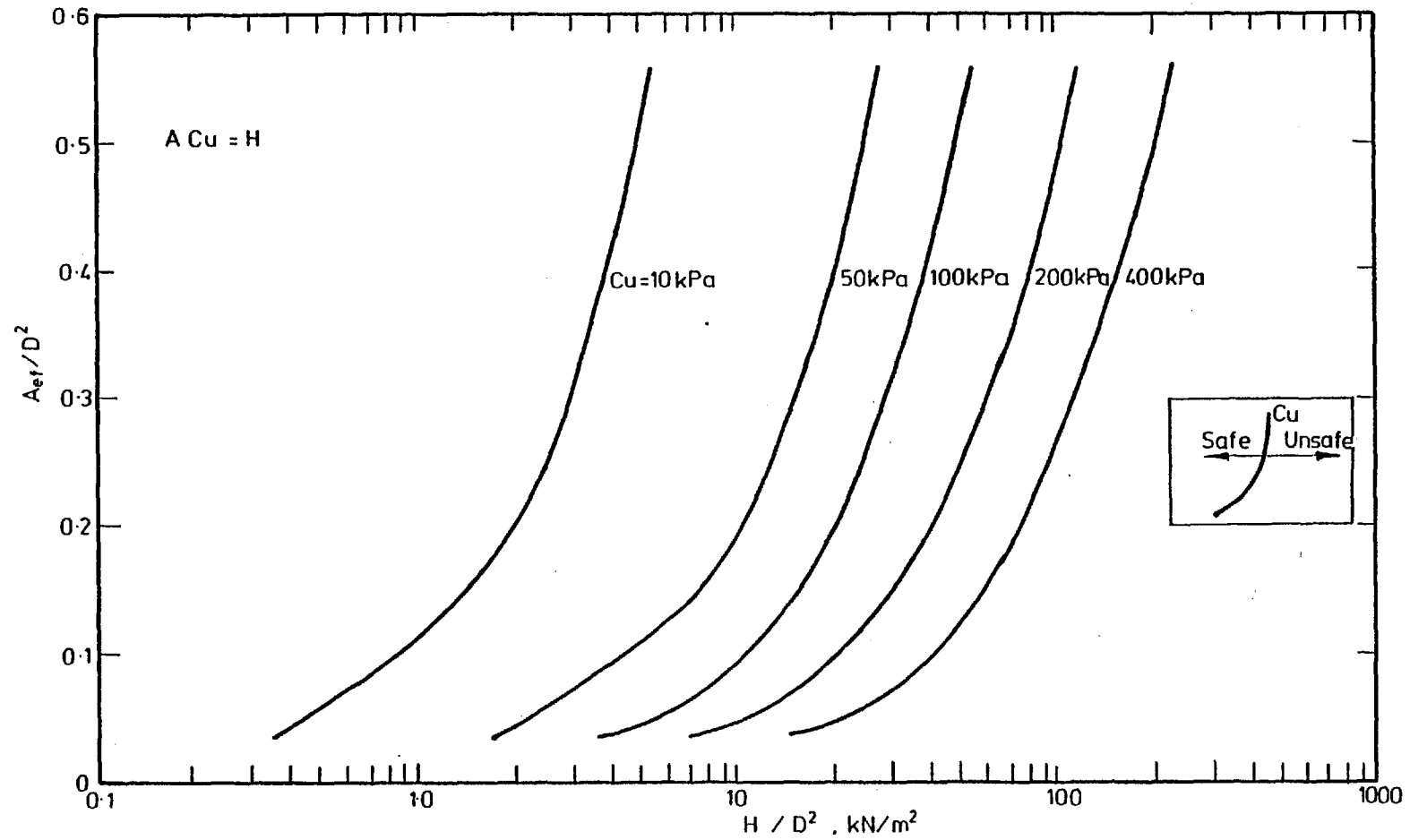


Fig.7-4 Preliminary check against failure by sliding.

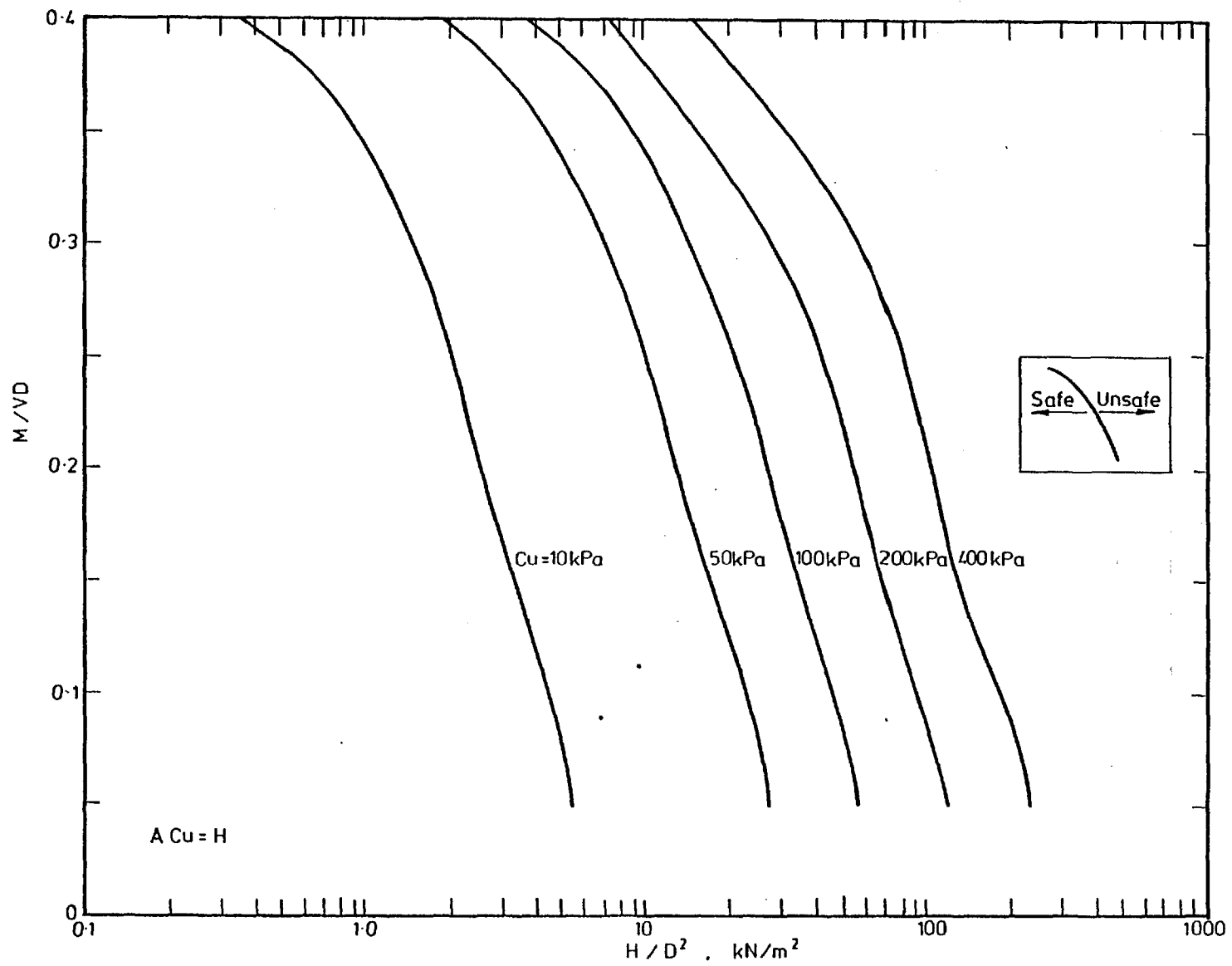


Fig.7.5 Preliminary check against failure by sliding

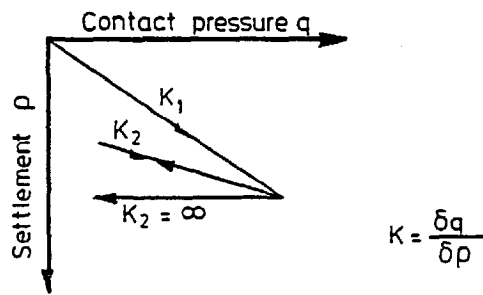


Fig. 7-6 Assumptions of the moduli of subgrade reaction

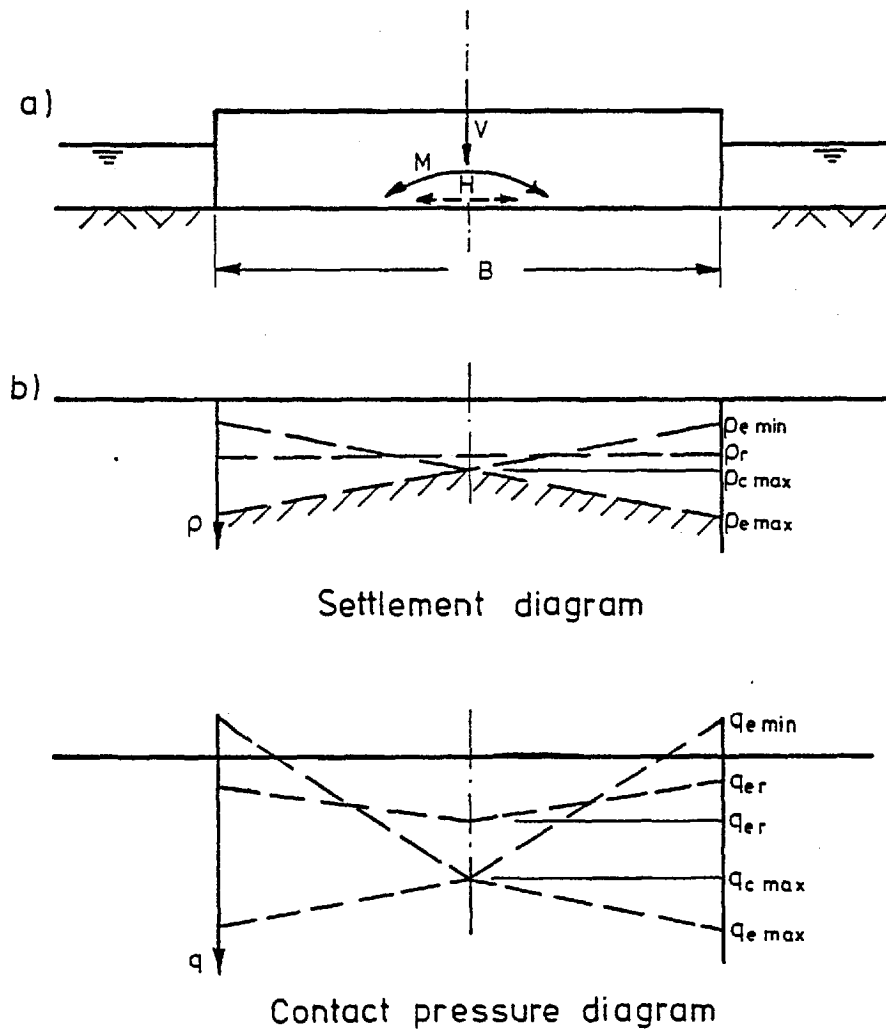


Fig. 7-7 Contact pressures and settlements of a rigid strip foundation subjected to wave loading (after Vaughan, 1974)

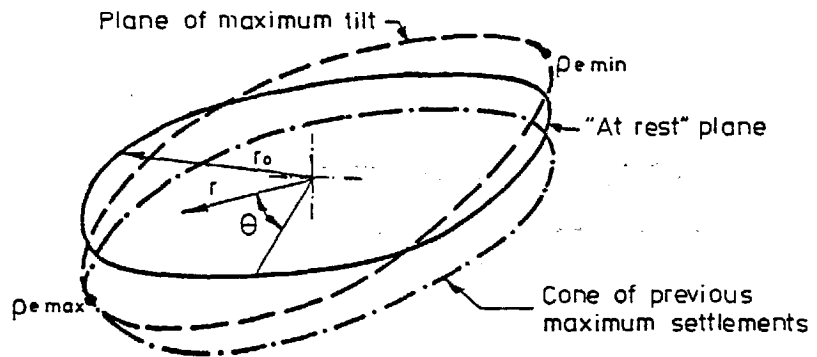


Fig.7-8 A model of a rigid circular foundation subjected to waves from all directions

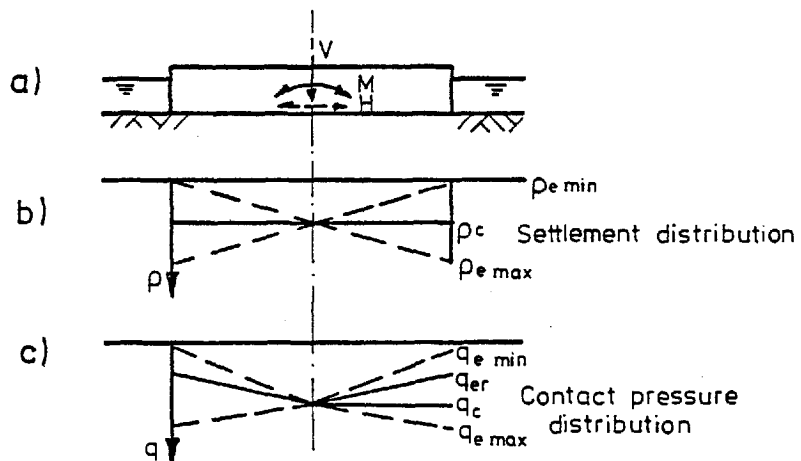


Fig.7-9 Distribution of settlements and contact pressures for a rigid circular foundation subjected to waves from all direction.

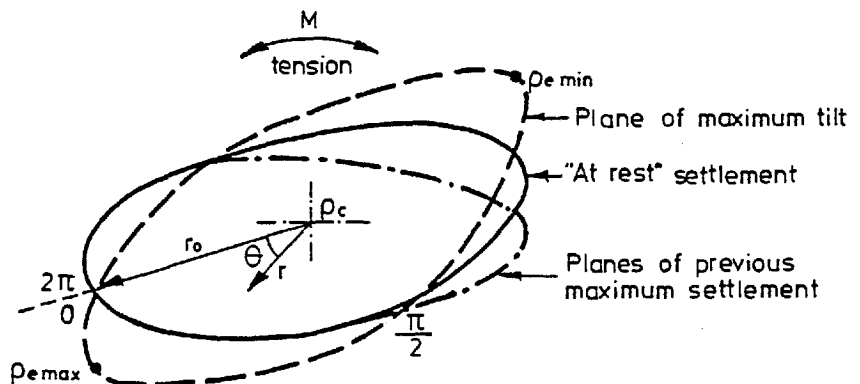


Fig. 7-10 A model of a rigid circular foundation subjected to waves from one direction only.

APPENDIX (1)

Al-1

ROCKS, SHELLS AND MINERALS

OF THE LOWER CROMER TILL

1. LITERATURE REVIEW

1.1 Rocks

Author	Description
Boswell (1914)	igneous rocks, some of which are Scandinavian, dolerites, cheviot rocks, smaller boulders of flint (some fresh and black others weathered and brown), quartz and chalk. The North Sea Drift in its inland sections does not carry Neocomian or Jurassic erratics or bear in its boulders or matrix any noteworthy quantity of hard chalk or grey tabular flint.
Boswell (1923)	<u>Rocks of Scandinavian origin:</u> rhomb-porphyr, nordmarkite laurvikite, cancrinite-syenite (särna), elaeolite-syenite, zircon-bearing granite; quartz-porphry, fine-grained granite (Sweden), hornblende-porphyr, quartz-hornblende-schist, sparagmites and others. <u>Rocks of Northern Origin:</u> granites, dolerites, basalts, porphyrites, andesites, gneisses, schists.

Author	Description
Boswell (1923) cont.	<u>Erratics, the source of which can be determined stratigraphically:</u> Old red sandstone, carboniferous limestone and chert, coal measure sandstone, soft red (Triassic) sandstone, Liassic limestone and sandstone, Red chalk, Grey and hard chalk with belemnites.
Solomon (1932)	Chalk usually as pebles but in places also as matrix, flint often green coated sometimes stained and dark purplish, quartz, quartzite, schist and greiss, igneous rocks both Scandinavian and Scottish in origin, marcasite nodules, lignite, peat, spetaria.
Baden-Powell (1948)	Scottish rocks, small black flint, pink granophyne, schist (several kinds), pink acid gneiss, rocks are erratics not as matrixes, rocks rarely from Rhine Province.
Baden-Powell & West (1960)	In glacial outwash of gravels hornblende syenite of Norwegian origin.
Kazi & Knill (1969)	Flint (often stained), quartz, quartzite, schist, gneiss, igneous rocks of Scandinavian and rarely Scottish origin.

1.2 SHELLS

Mentioned by all to be present mainly in fragmentary form but sometimes found intact. They are derived shells from various Eocene and Pliocene deposits especially found at coast, probably ploughed up from the bottom of the North Sea bed (Boswell (1914 and 1923)).

Baden-Powell and West (1960) gave the following description of the shells:-

- *Cardium edule* (common, point to shallow water condition).
- *Tellina balthica* (common, some valves almost complete, this shell does not represent specially cold conditions).
- *Cyprina islandica*
- *Acila* sp.
- *Mya* sp.
- *Ostrea* sp.

1.3 MINERALOGY (Clay Minerals)

Dealt with by Boswell (1916). The following description is after Solomon (1932) who mentioned the removing and analysis of the argillaceous matter in the Lower Till. The succession in the following description indicates dominance.

1) Magnetic Fraction

Al-4

Mineral	Description	Origin	%
Garnet	occurs briefly in the form of rather small angular fragments of a pale colour and mostly without inclusions.	Tertiary origin, it is scarce in the drift at Hartlepool and cannot therefore be considered a direct Norwegian derivative	27.7
Amphiboles	includes ordinary green hornblende and a considerable proportion of a bluish-green mineral which resembles arfvedsonite.	Probably of Scandinavian origin, and probably partly that and partly from Tertiary deposits on the floor of the North Sea bed.	26.7
Epidote	is abundant in large rounded lemon yellow grains also in smaller rounded grains of a pale colour.	as above	16.5
Elmnite			
Tourmaline	scarce, the brown variety is commonest.		2
Staurolite	scarce.		
Pyroxenes Magnetite Monazite	not calling for special notice.		
Siderite	occurs as small brown rhombs or aggregates of rhombs.		
Iron Ores			25.6

2) Non-Magnetic Fraction

Al-5.

Mineral	Description	Origin and Notes
Zircon		
Hyanite		of Pleiocene affinities
Apatite	occurs in pellucid grains showing some degree of roundings	directly derived from Norway.
Andalusite		of Pleiocene affinities
Sphene		as Apatite
Topaz		
Rutile	is chiefly represented by a variety which shows pleochroism in greys, smoky-yellows and purplish-browns. The greyish mineral occurs in good crystals sometimes showing geniculate twinning.	From lower Tertiary horizons.
Zoisite		
Chlorite		
Marcasite	often occurs in the form of faraminiferal casts and also as little rod-and ball-shaped aggregates which are probably casts of some organic bodies.	
Crystalline Calcite & Rhombs of Dolomite		

APPENDIX (2)

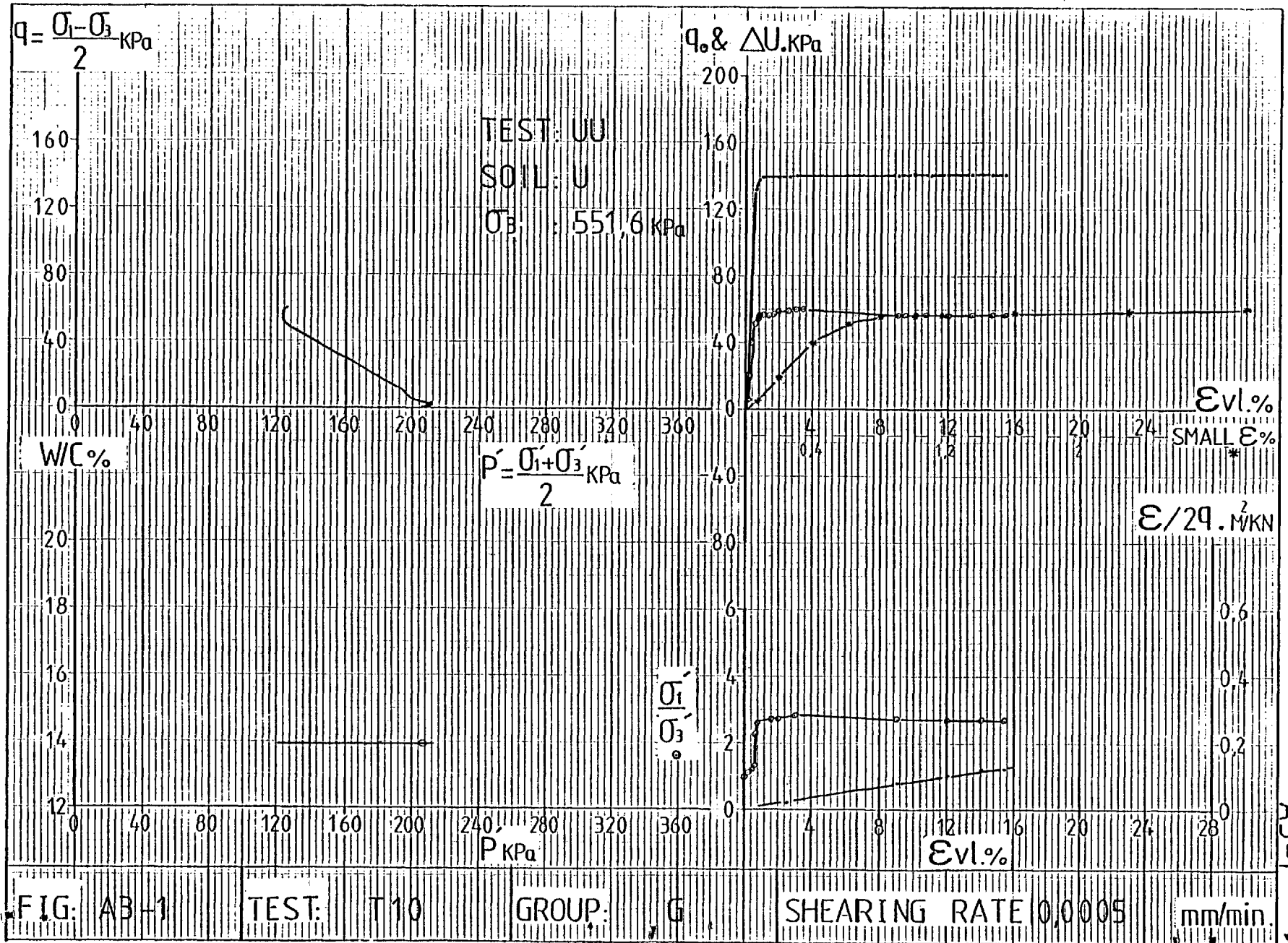
A2.1

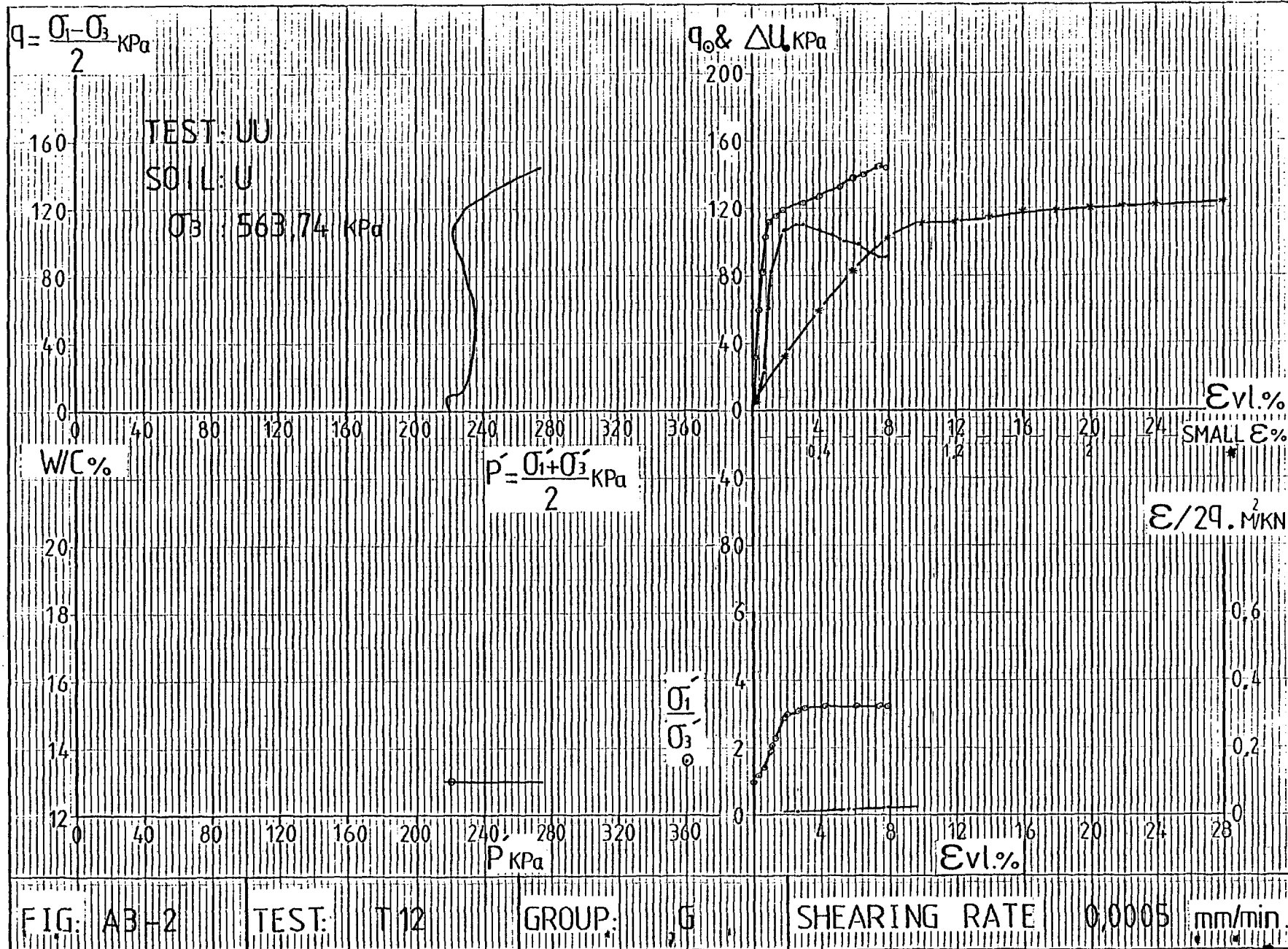
Analysis of Rocks from the Lower Till at Happisburgh
and North Sea Bed.
by XRD Laboratory, Geology Department, Imperial College, London.

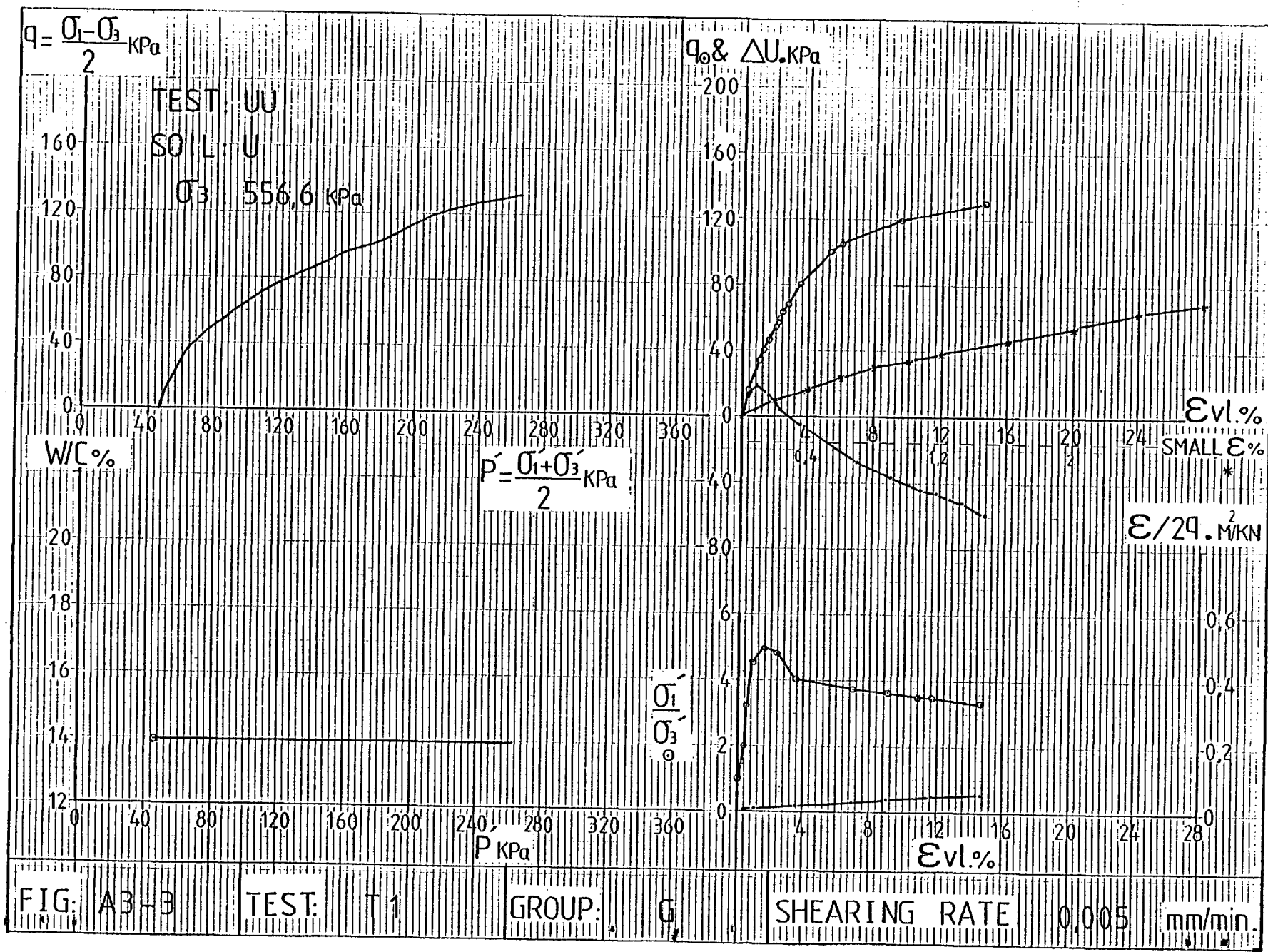
Site	WHOLE ROCK			CLAY FRACTION		
	major	minor	traces	major	minor	traces
Happisburgh	quartz	calcite, Felspar, clay minerals.	mica, muscovite type, unknown possibly including some dolomite.	calcite, illite	Smectite, Kaolin Chlorite	quartz, Felspar possible
Ninian, 5.2A Class 1 (North Sea)	quartz	calcite unknown, Felspar	mica, muscovite type, clay minerals, unknowns including possible dolomite	illite	Chlorite Kaolin Calcite	Smectite Felspar, quartz unknowns
Ninian, 7B. 37 Class 3 (North Sea)	quartz	Felspar calcite	mica muscovite type, clay minerals, unknowns may include dolomite amphibole, etc.	illite	Smectite, Kaolin Chlorite, Calcite	Felspar, quartz unknowns, possibly dolomite
Statfjord, NB2.53 Class 1 (North Sea)	quartz	Felspar calcite	mica, muscovite type, unknown may include dolomite	illite	Chlorite, Kaolin Smectite, Calcite	Felspar, quartz, unknown
Statfjord NB4, 54 Class 2 (North Sea)	quartz	Felspar calcite	possible dolomite mica muscovite type, clay minerals, unknown may be amphibole	illite	Chlorite Kaolin Calcite Smectite	Felspar quartz slight unknown
Statfjord, BA6, S21 Class 3 (North Sea)	as above	as above	as above		as above	as above

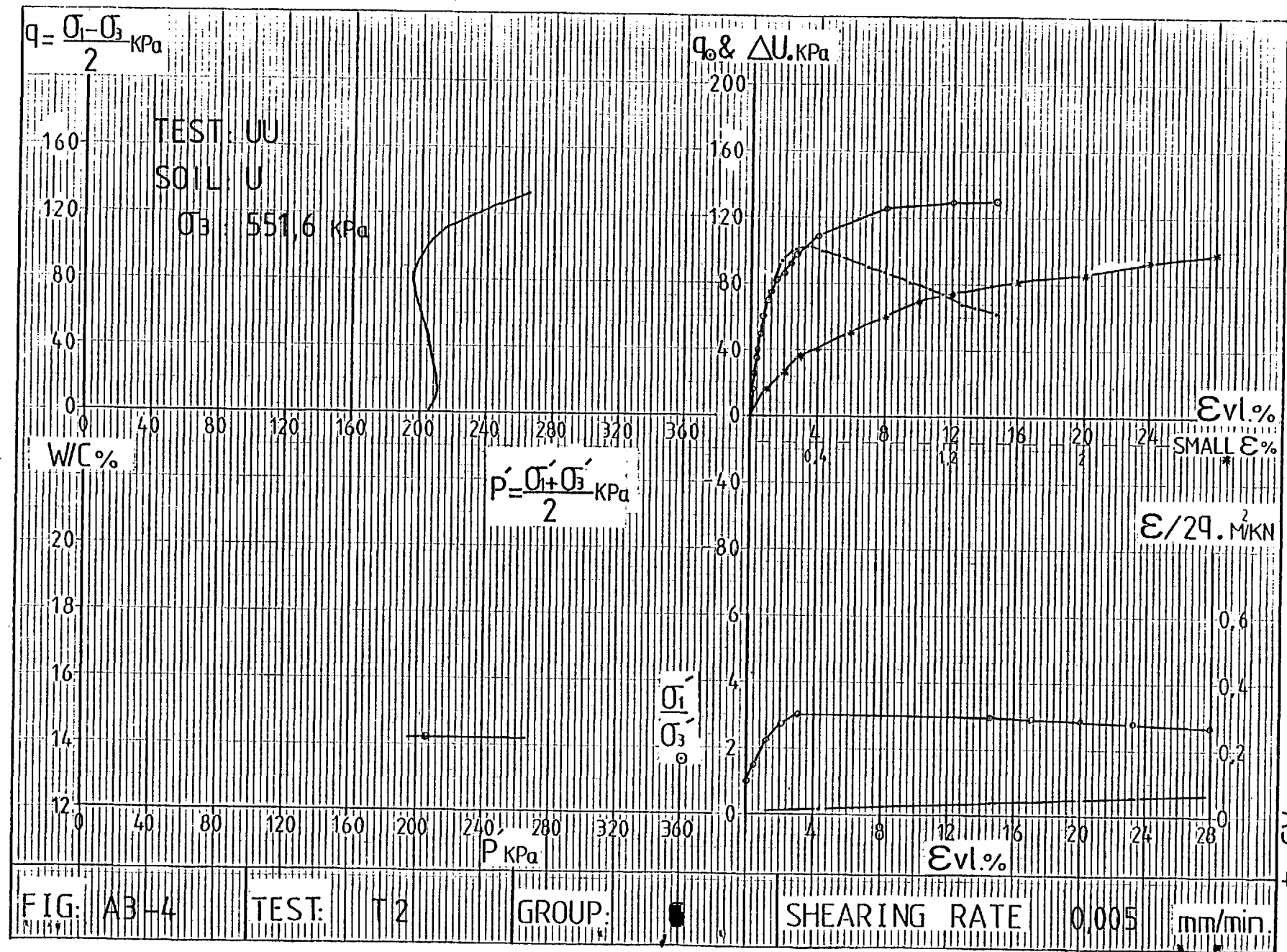
APPENDIX 3

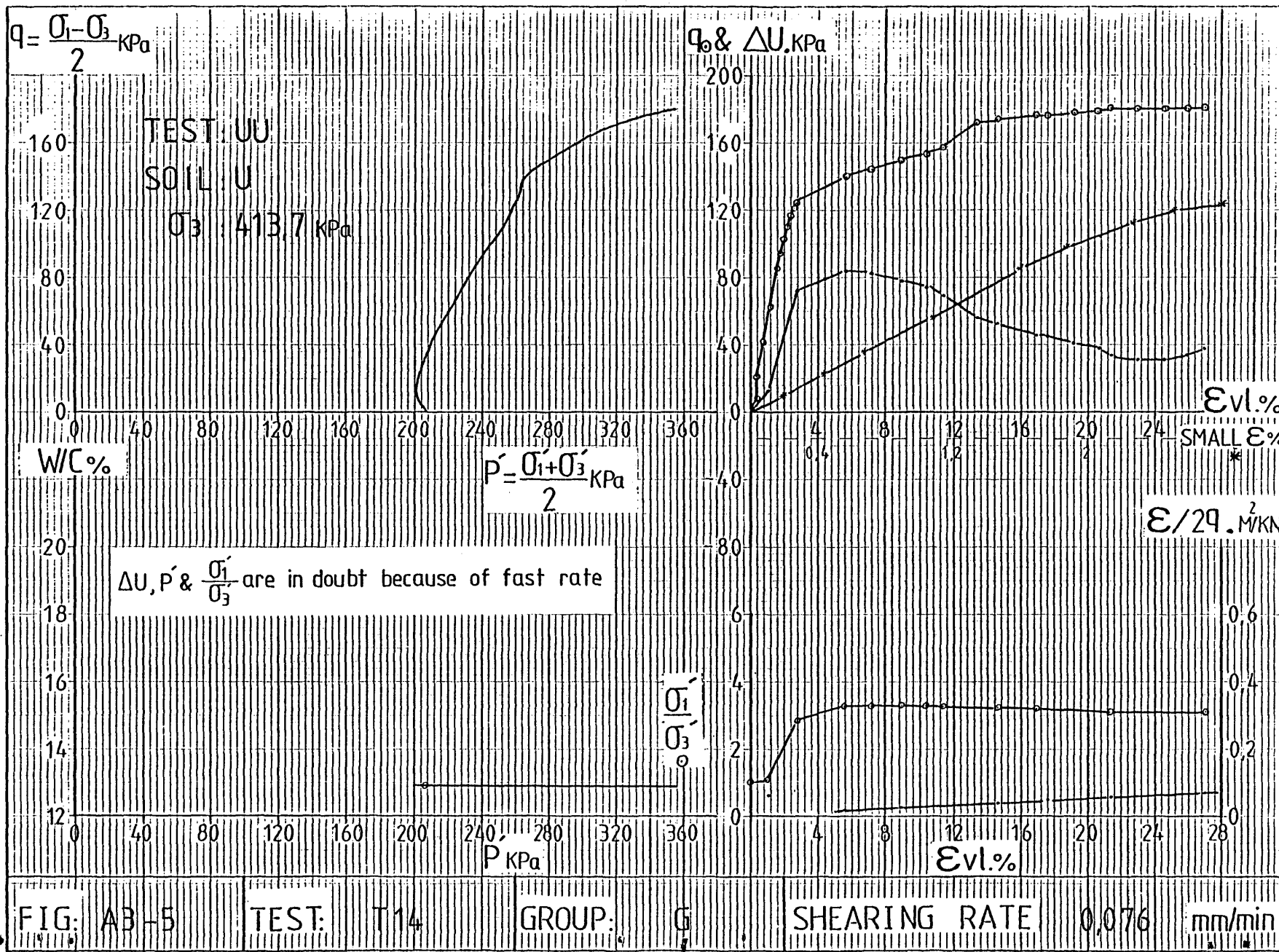
RESULTS OF TRIAXIAL TESTS

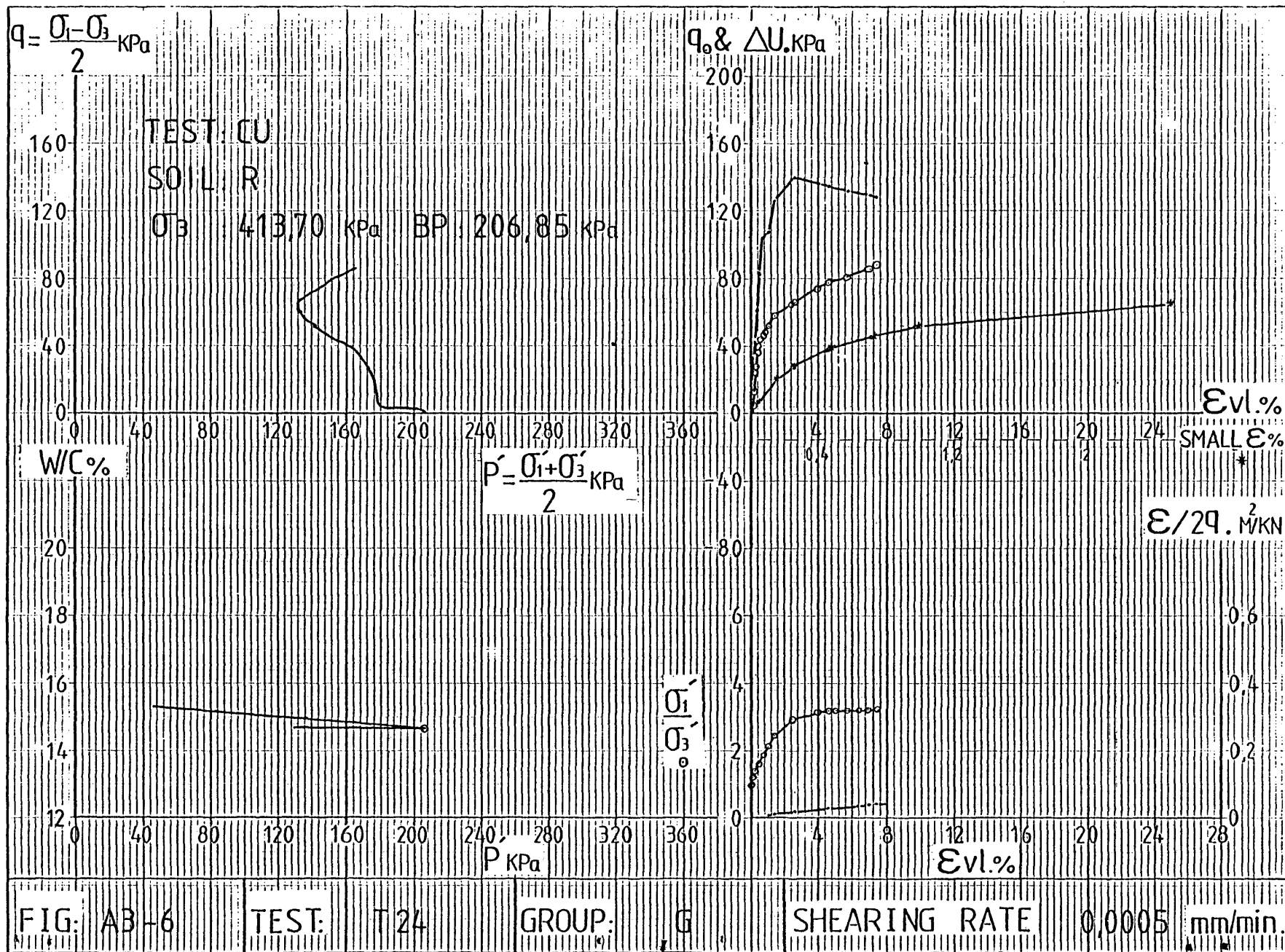












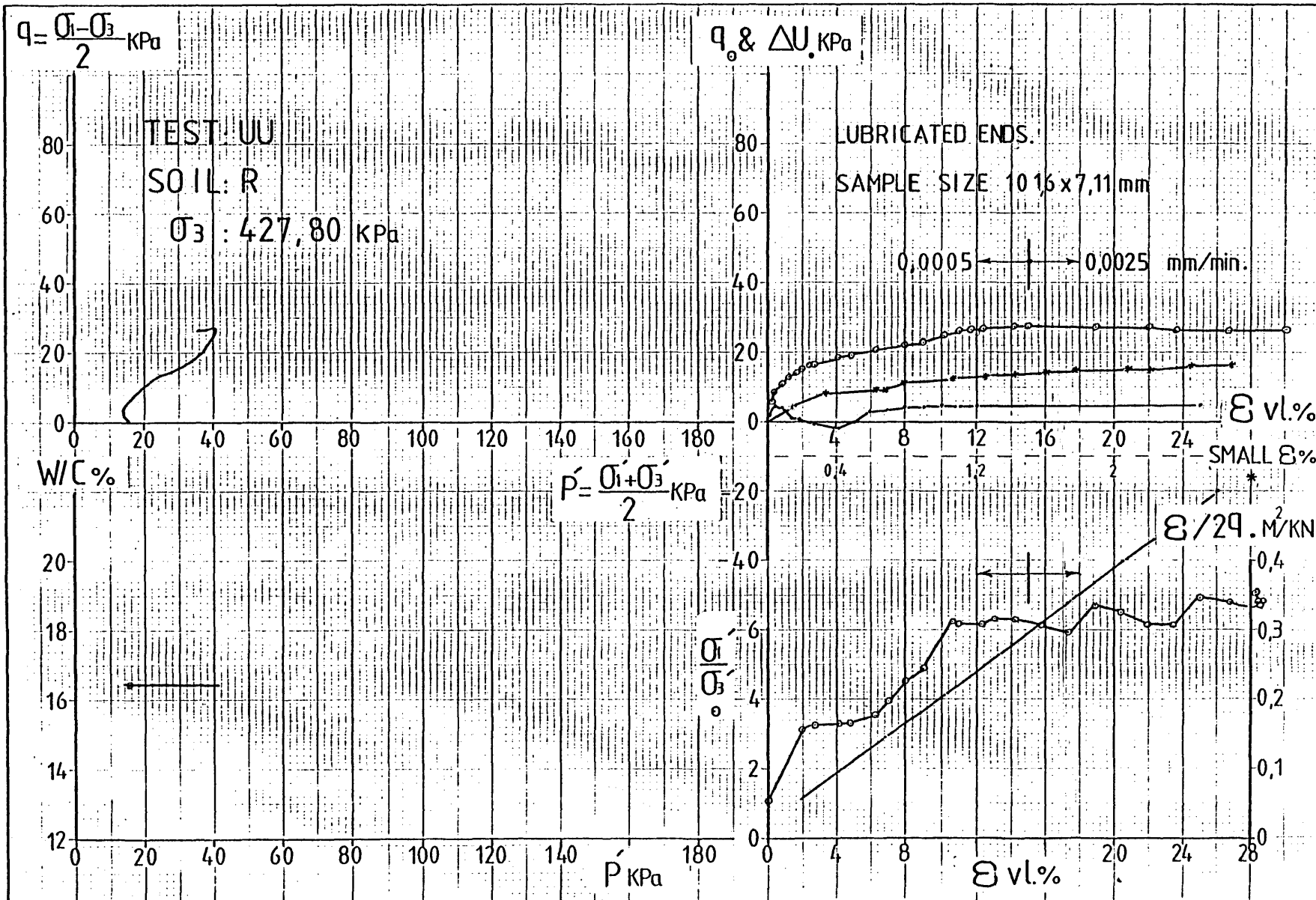
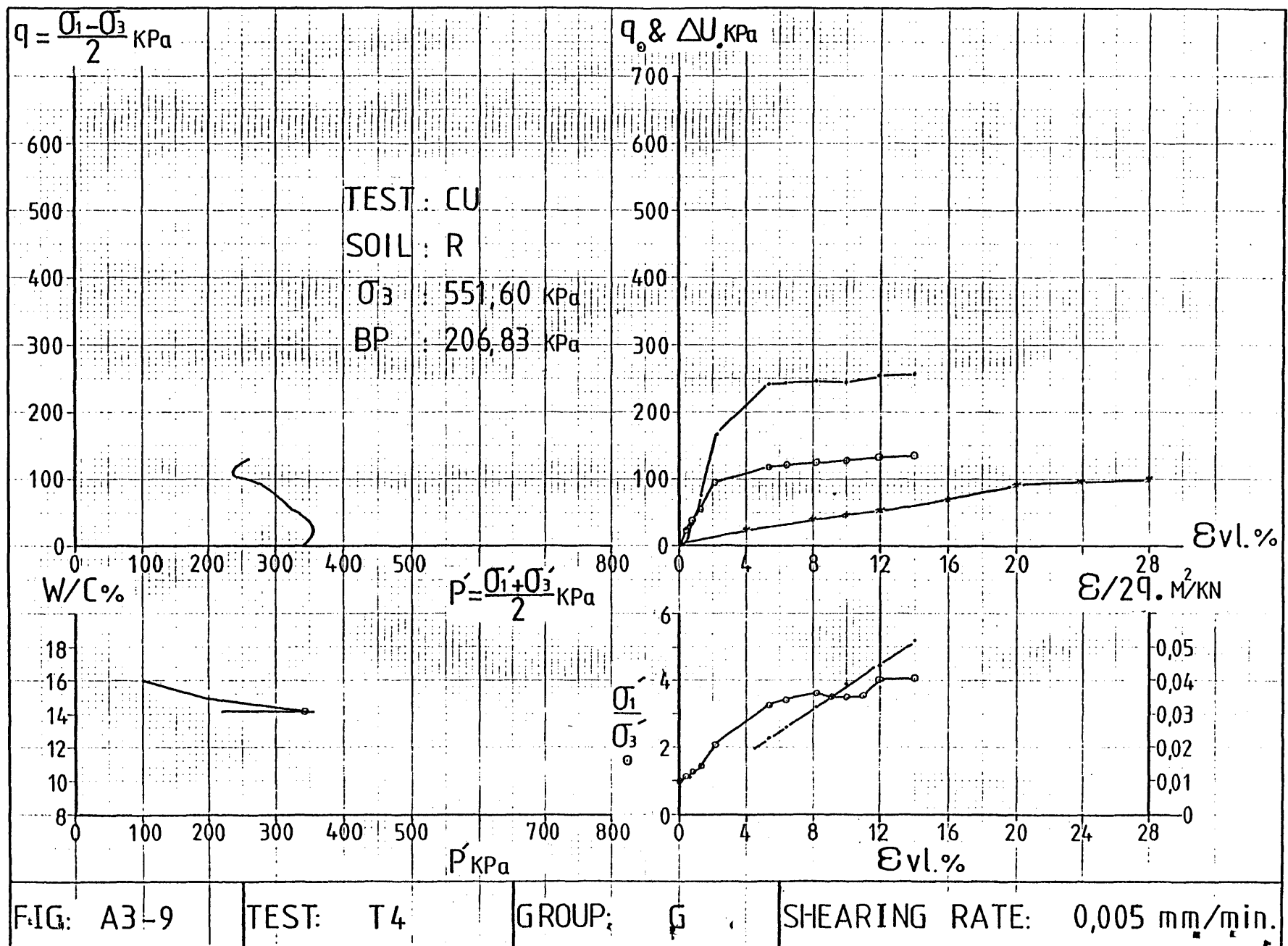


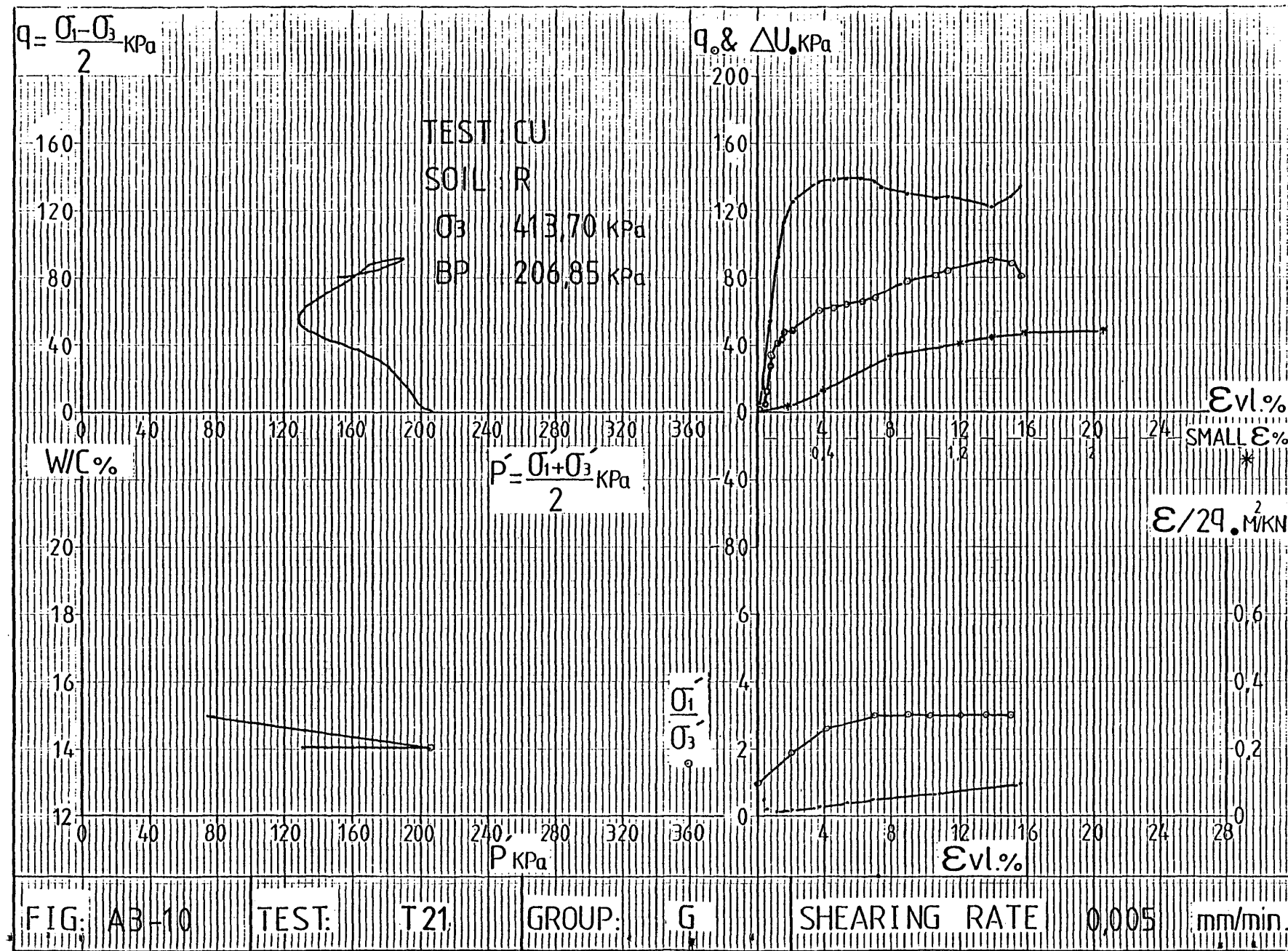
FIG: A3-7

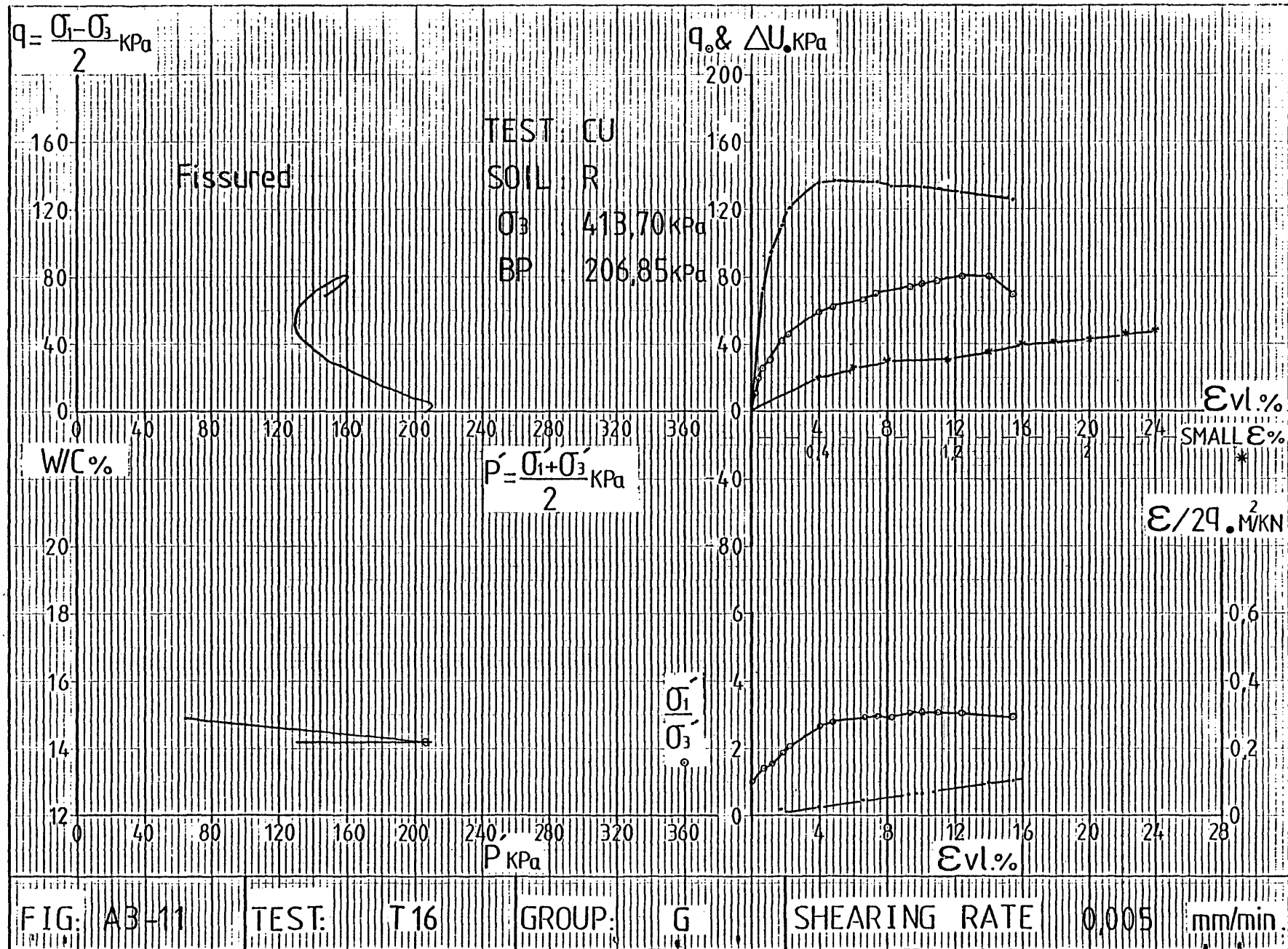
TEST: T 31

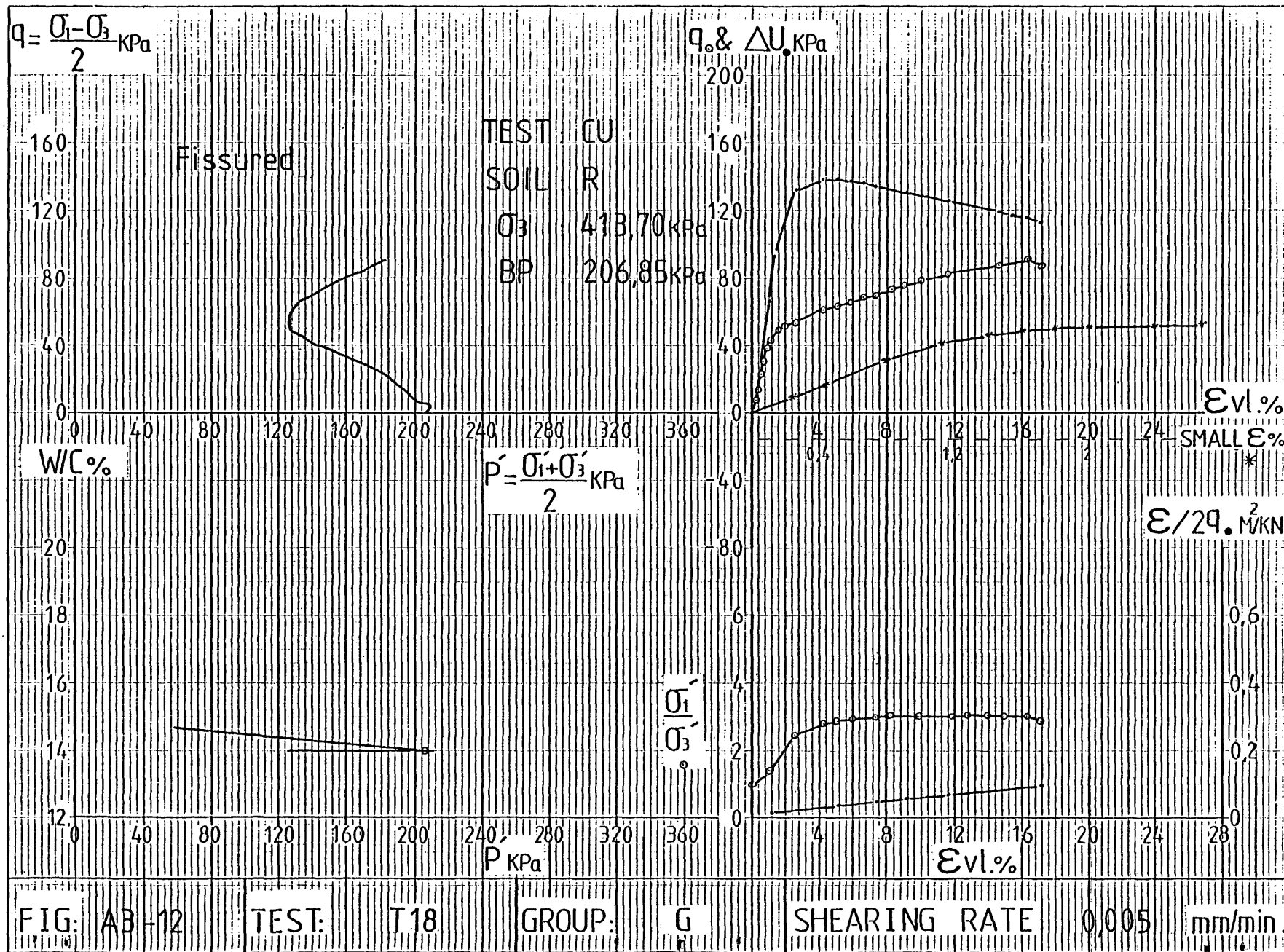
GROUP: G

SHEARING RATE 0,0005; 0,0025 mm/min.









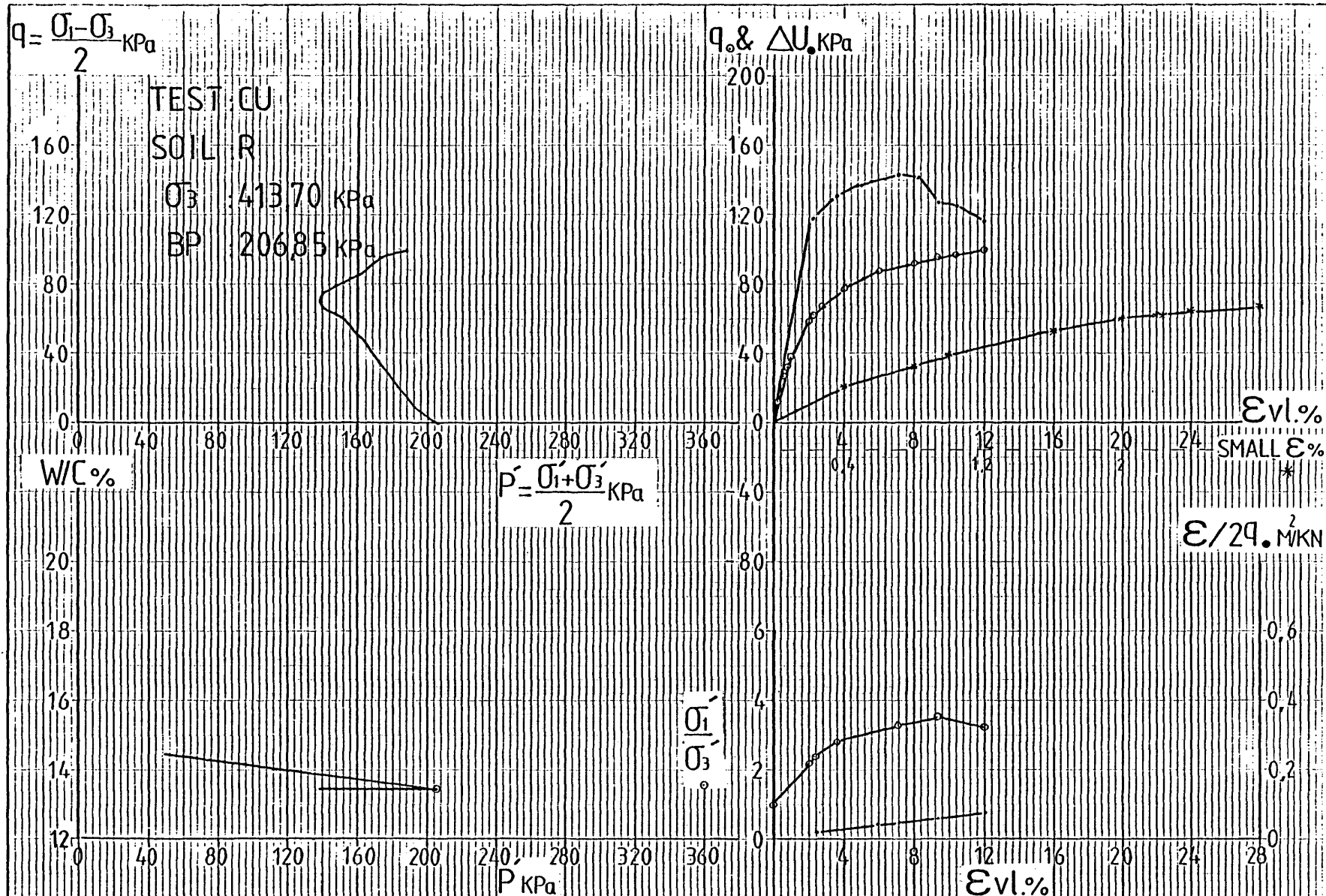


FIG: A3-13

TEST: T20

GROUP: G

SHEARING RATE: 0.005 mm/min.

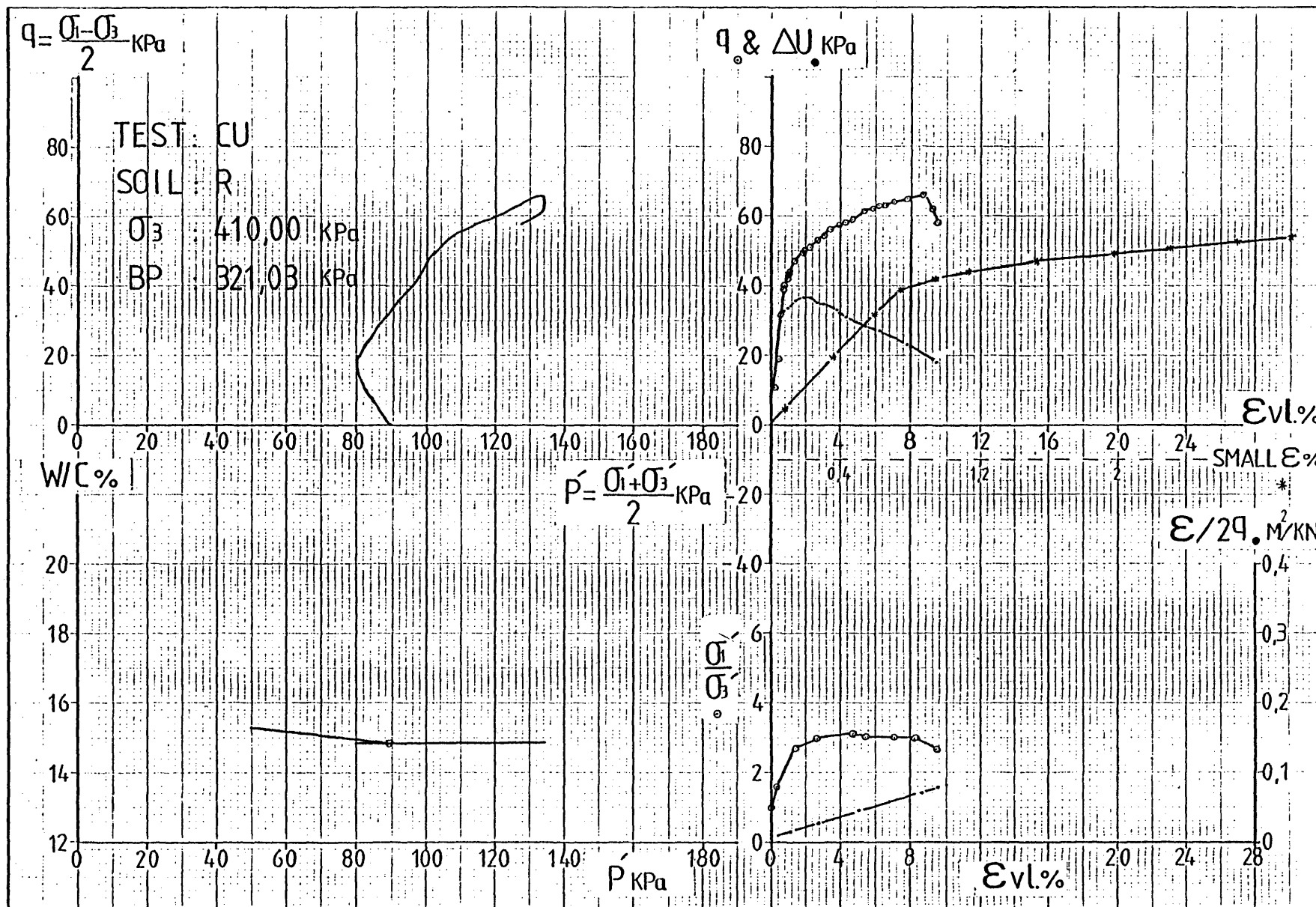
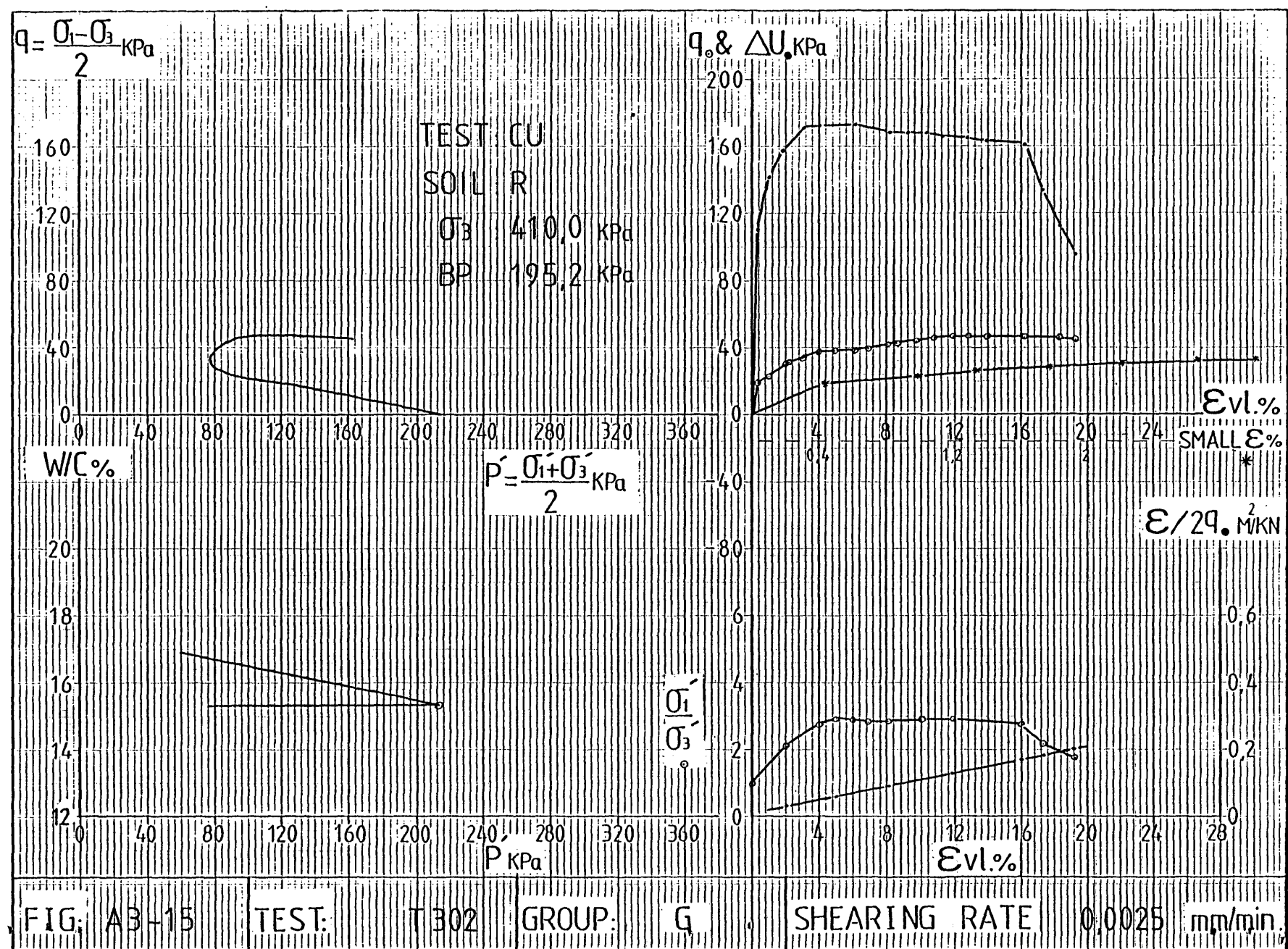


FIG. A3-14

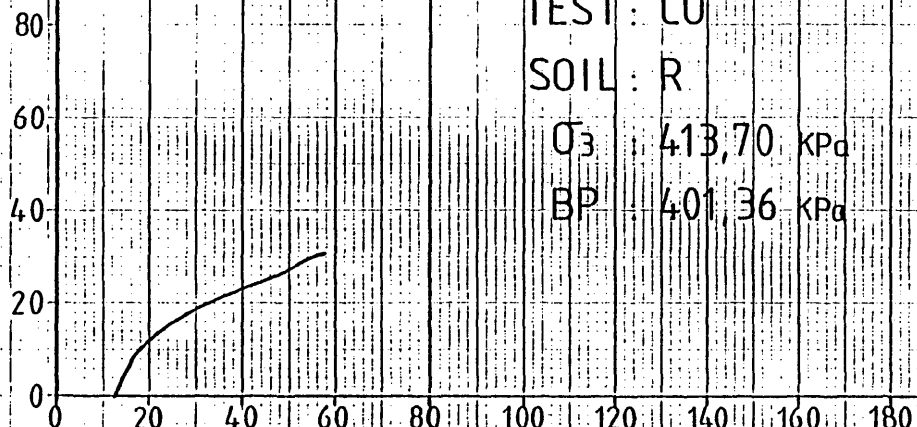
TEST: T 30

GROUP: G

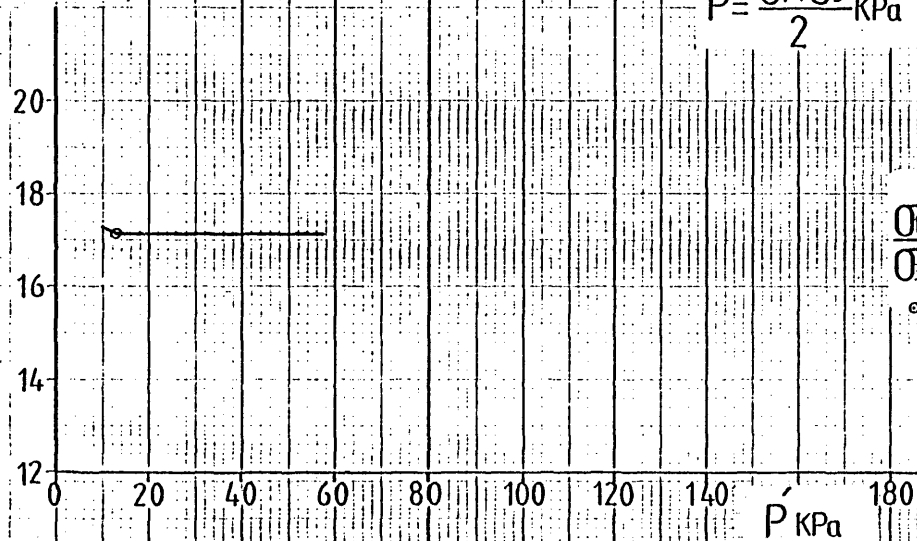
SHEARING RATE: 0,0025 mm/min.



$$q = \frac{\sigma_1 - \sigma_3}{2} \text{ KPa}$$



W/C %



q_0 & ΔU KPa

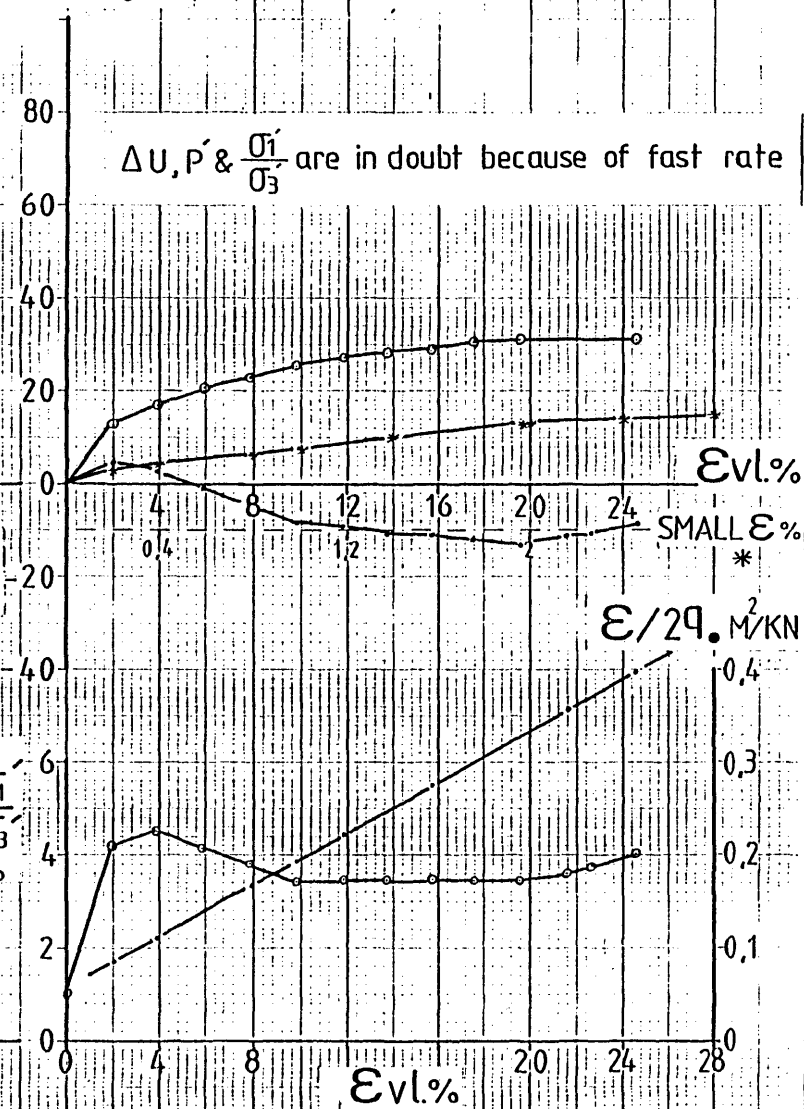


FIG: A3-16

TEST:

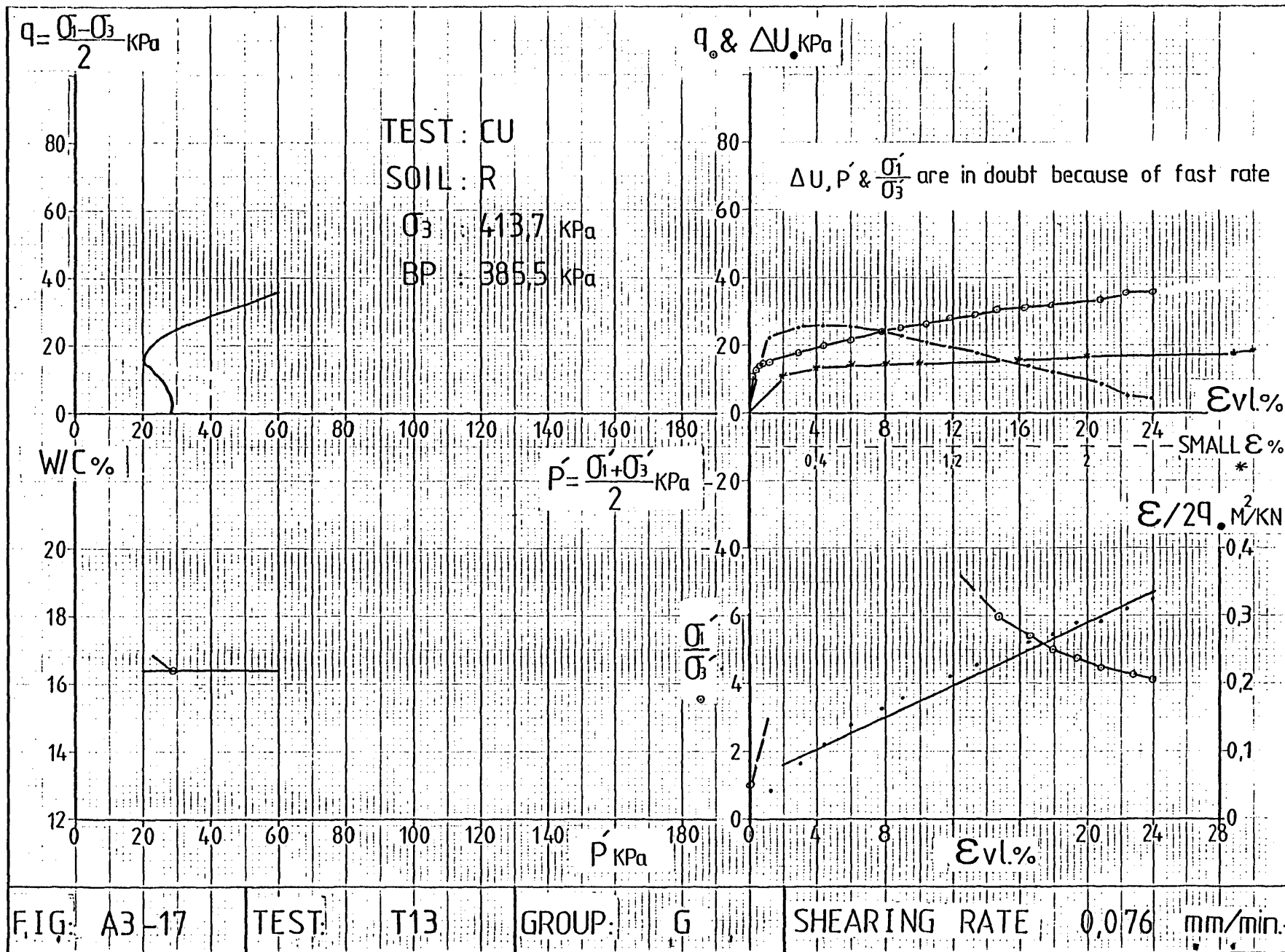
T32

GROUP:

G

SHEARING RATE

0,0125 mm/min.



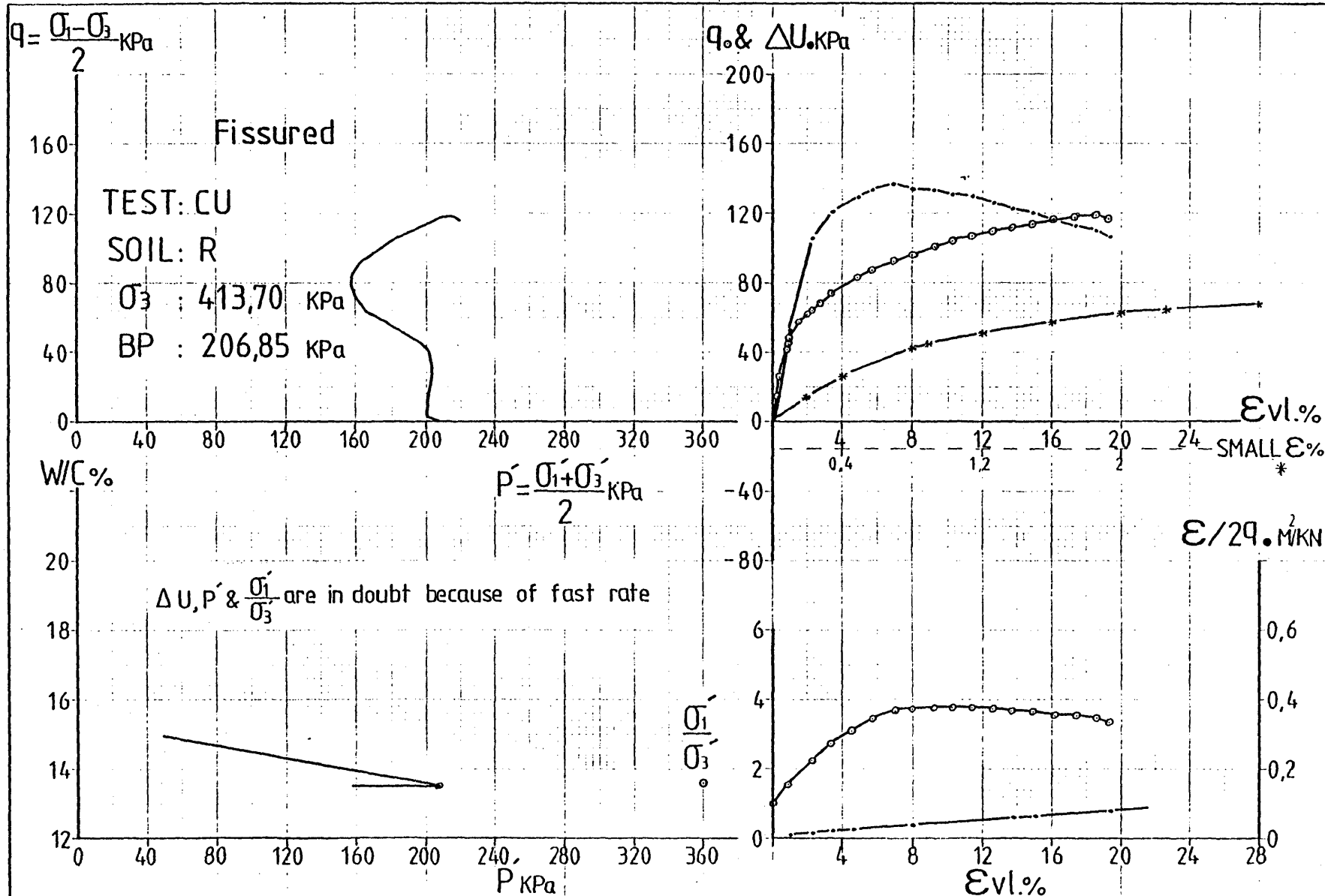


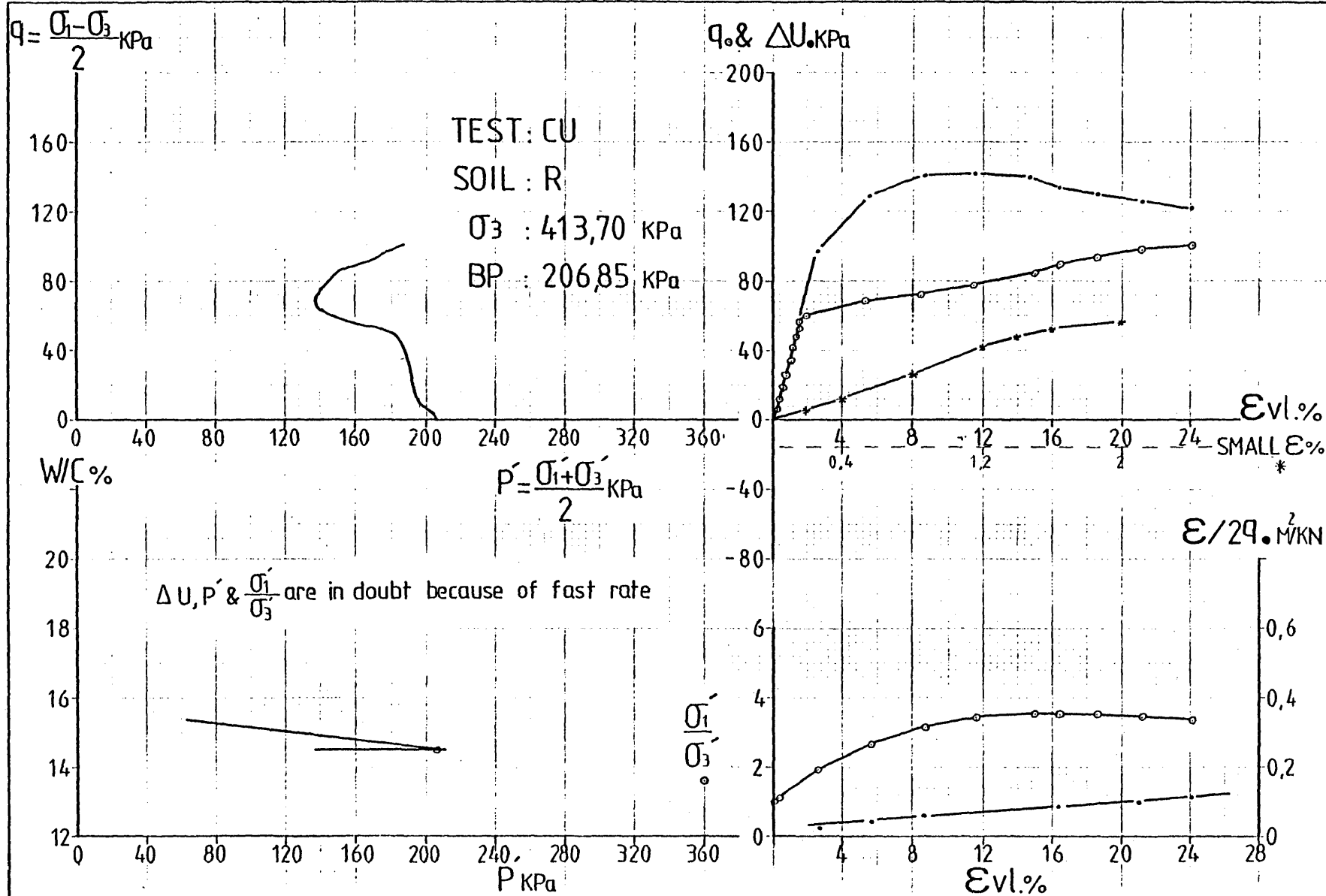
FIG: A3-18

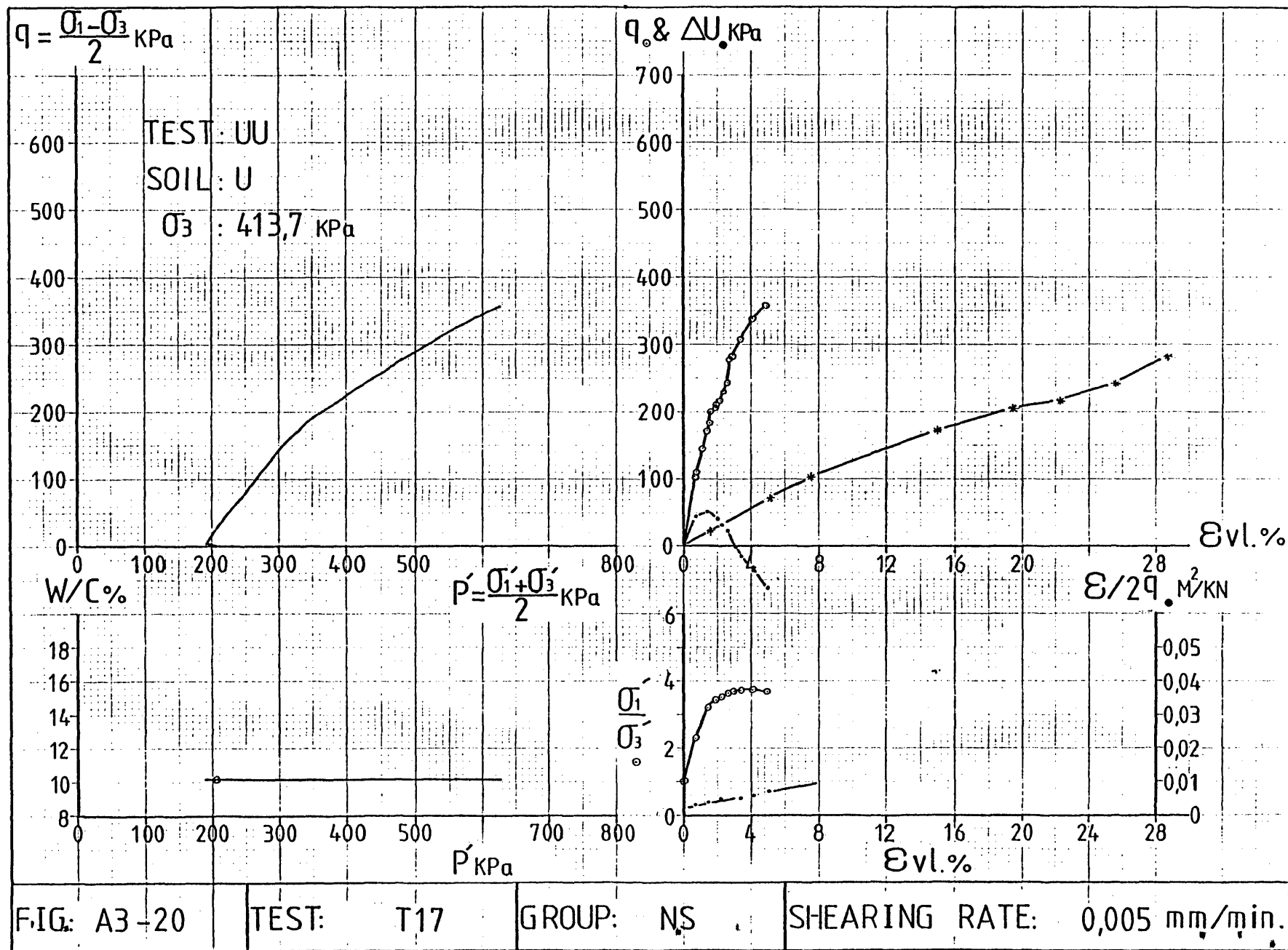
TEST: T19

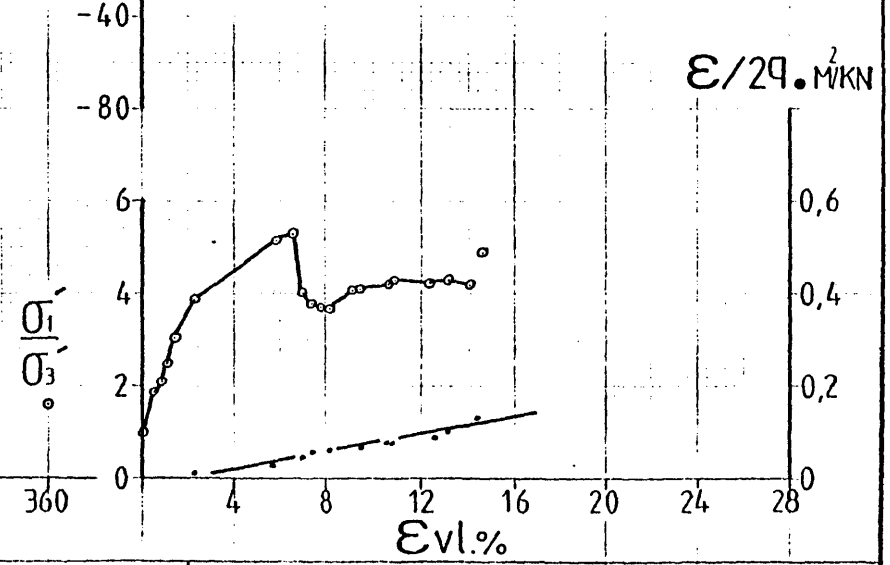
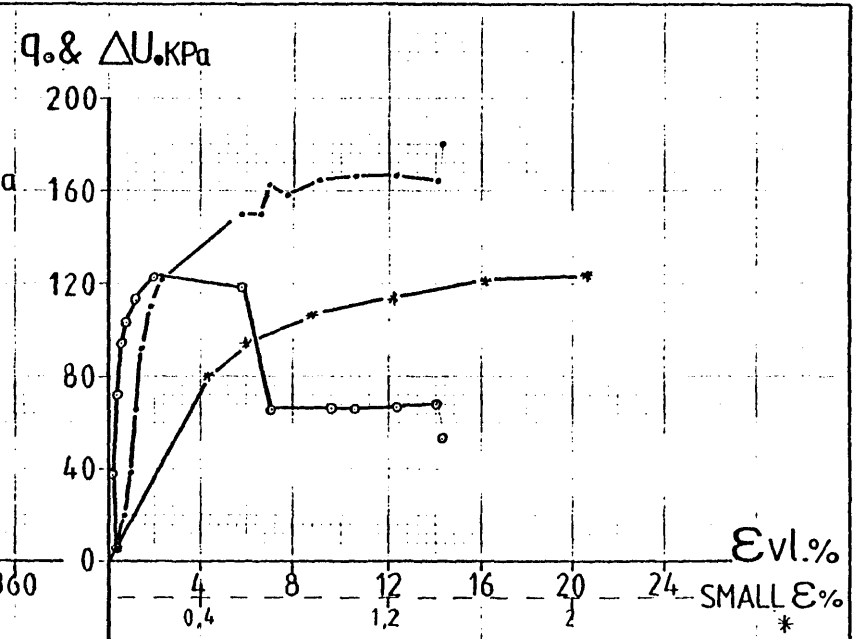
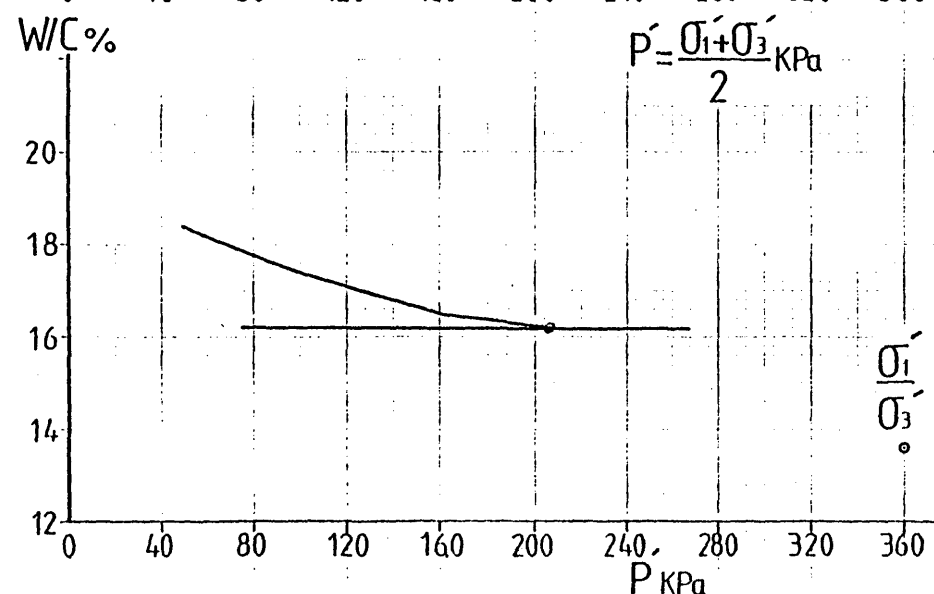
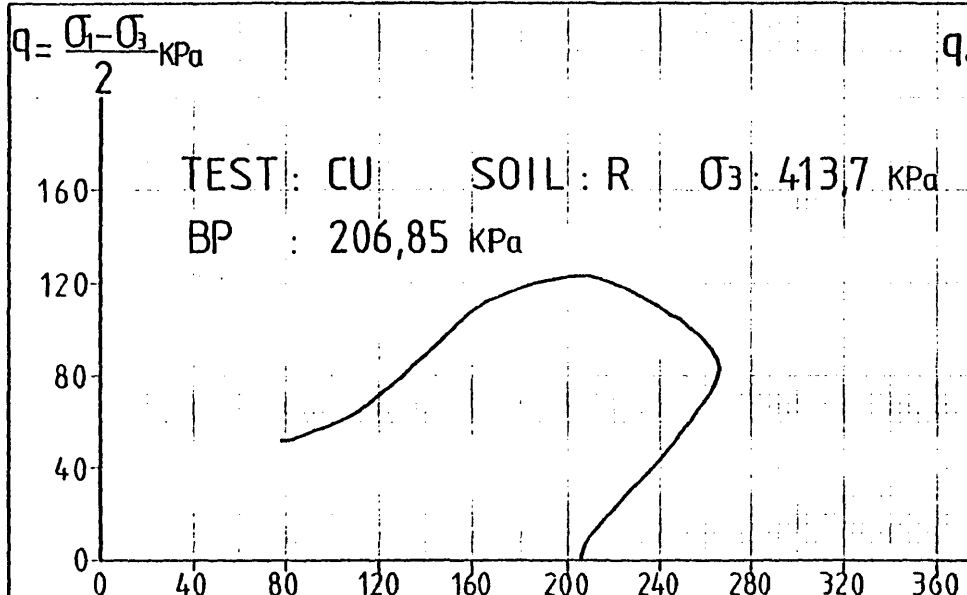
GROUP: G₁

SHEARING RATE: 0,058 mm/min.

A3-18







-350-

A3-21

FIG: A3-21

TEST: T23

GROUP: NS

SHEARING RATE: 0,005 mm/min.

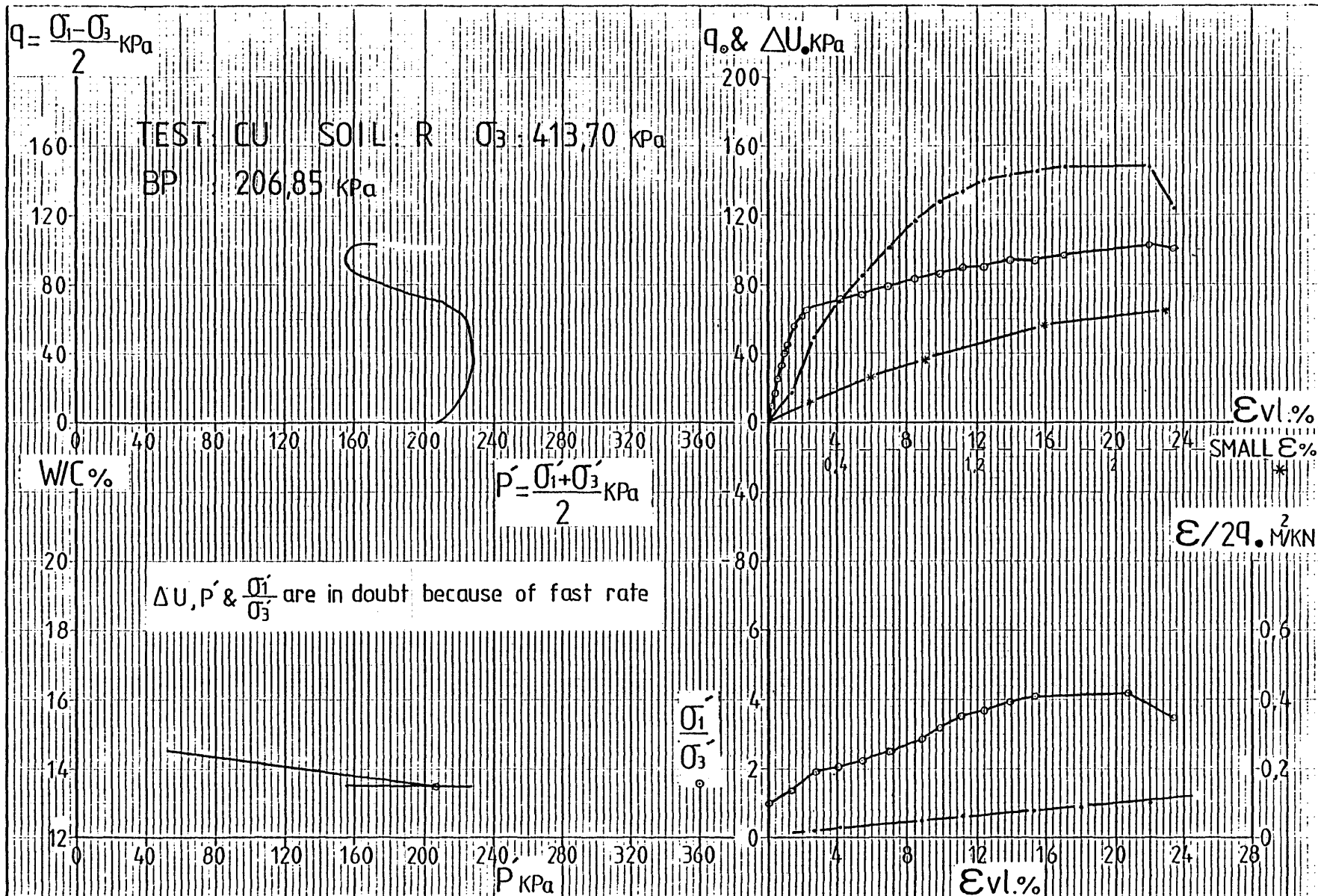
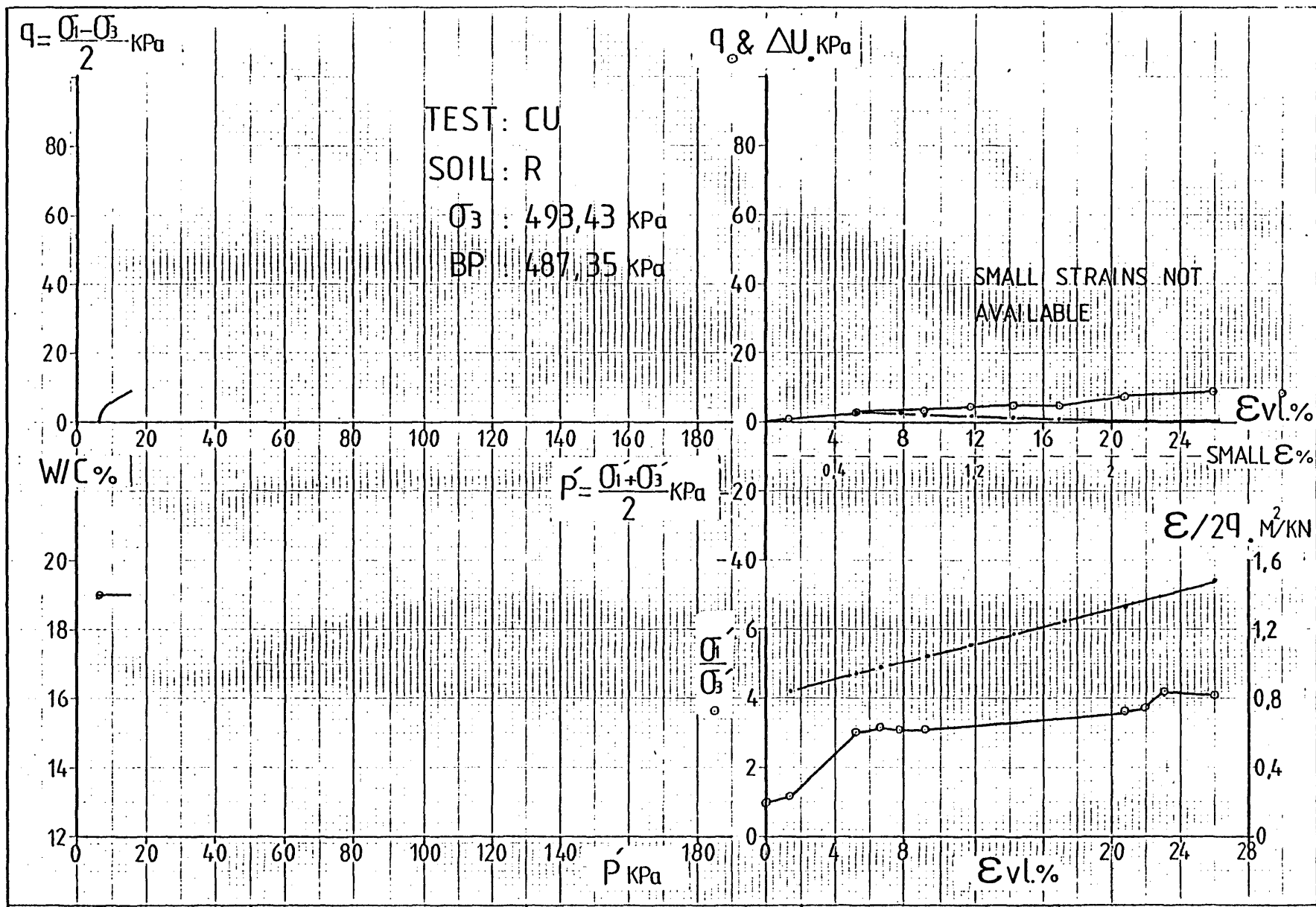


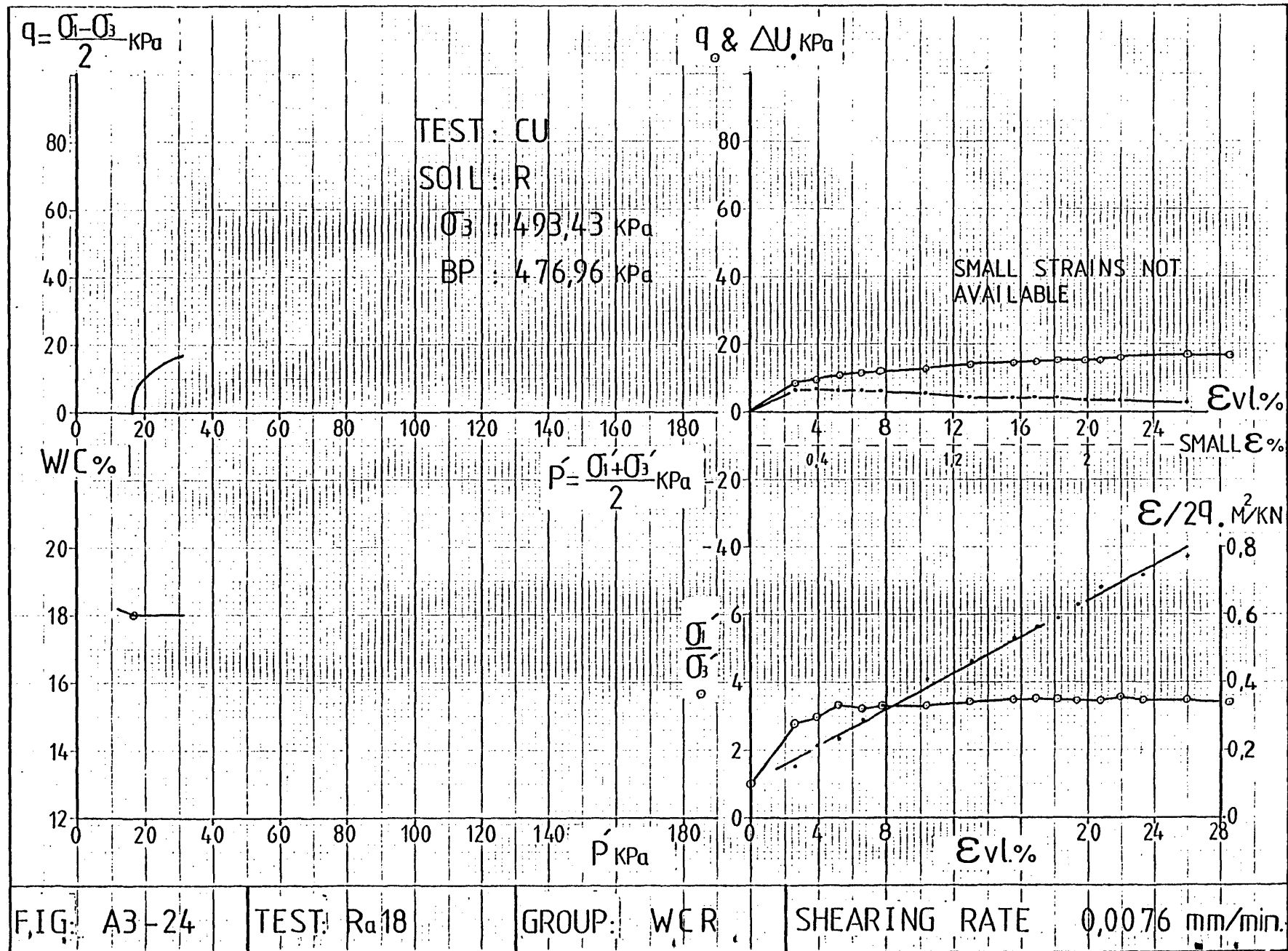
FIG: AB-22

TEST: T25

GROUP: NS

SHEARING RATE: 0.5029 mm/min.





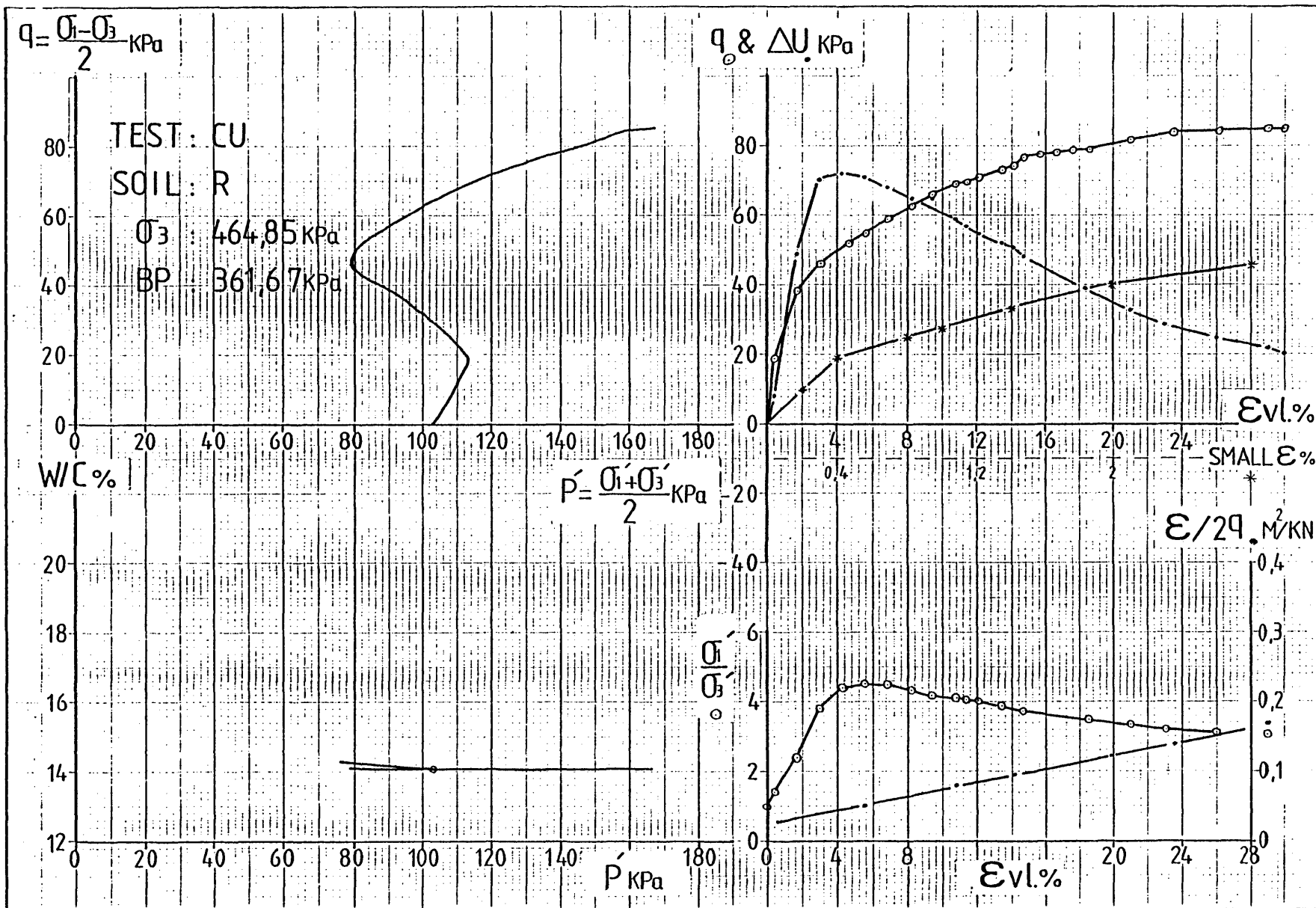
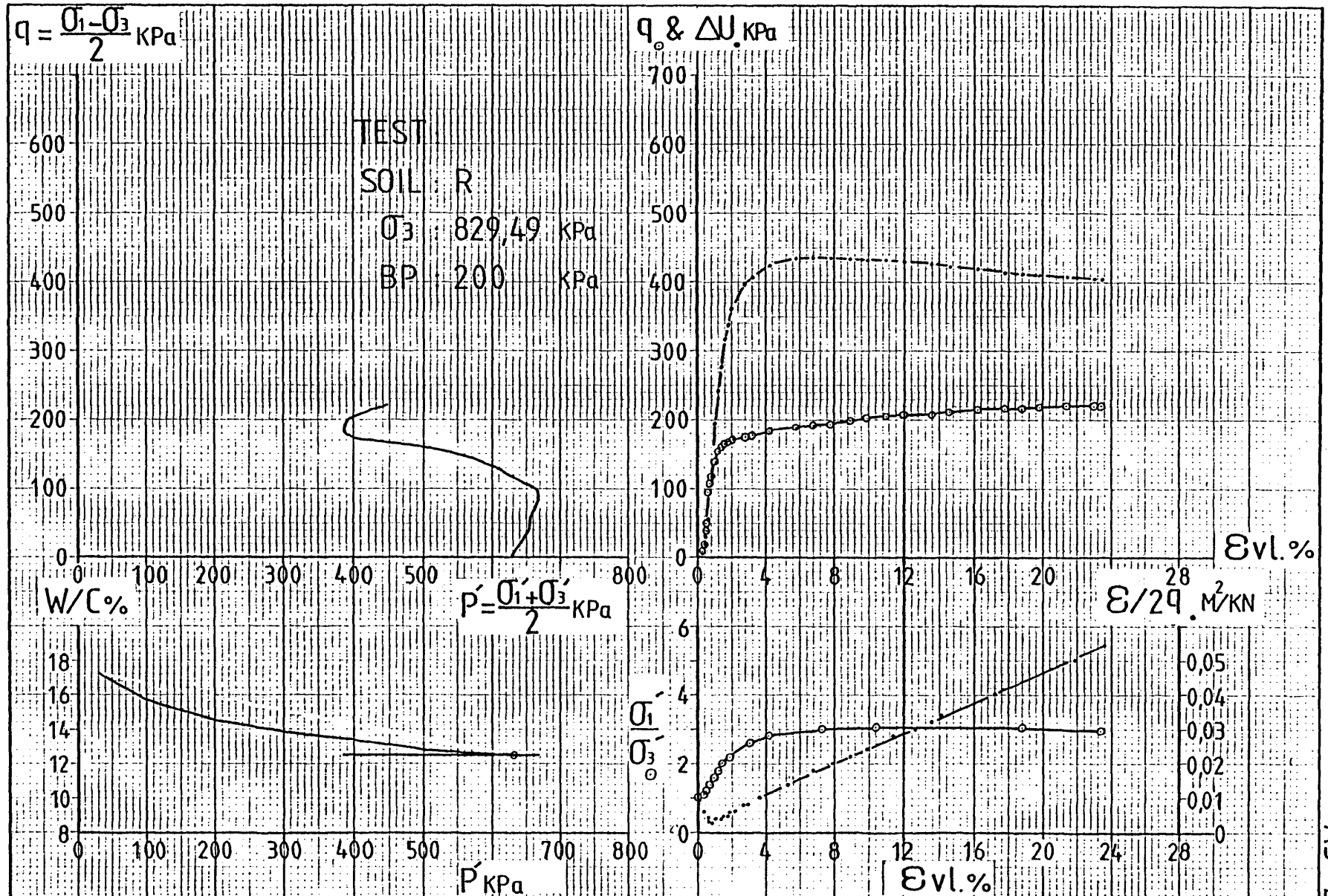
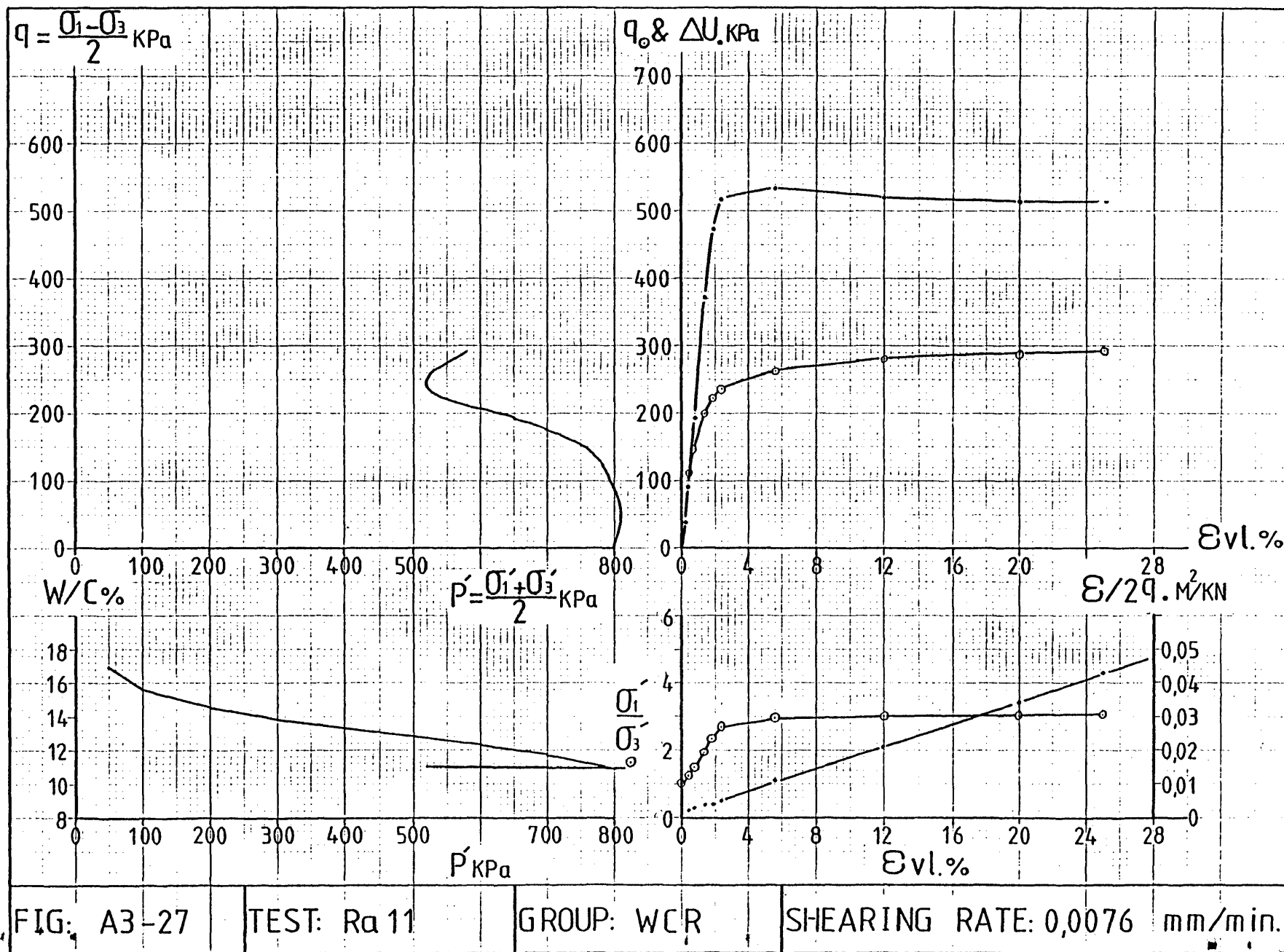
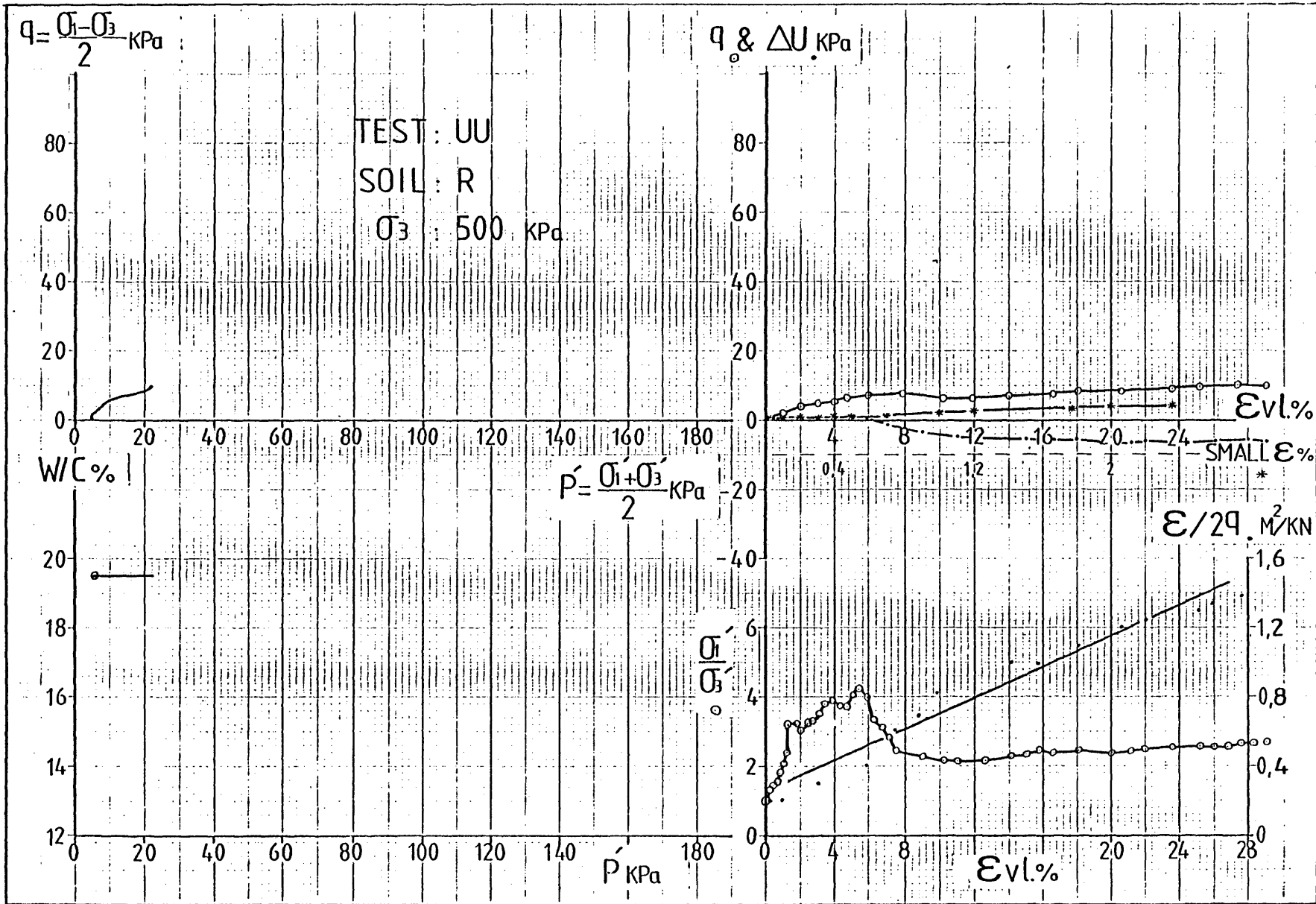


FIG: A3-25	TEST: R _a 14	GROUP: WCR	SHEARING RATE: 0,0076 mm/min.
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A3-25



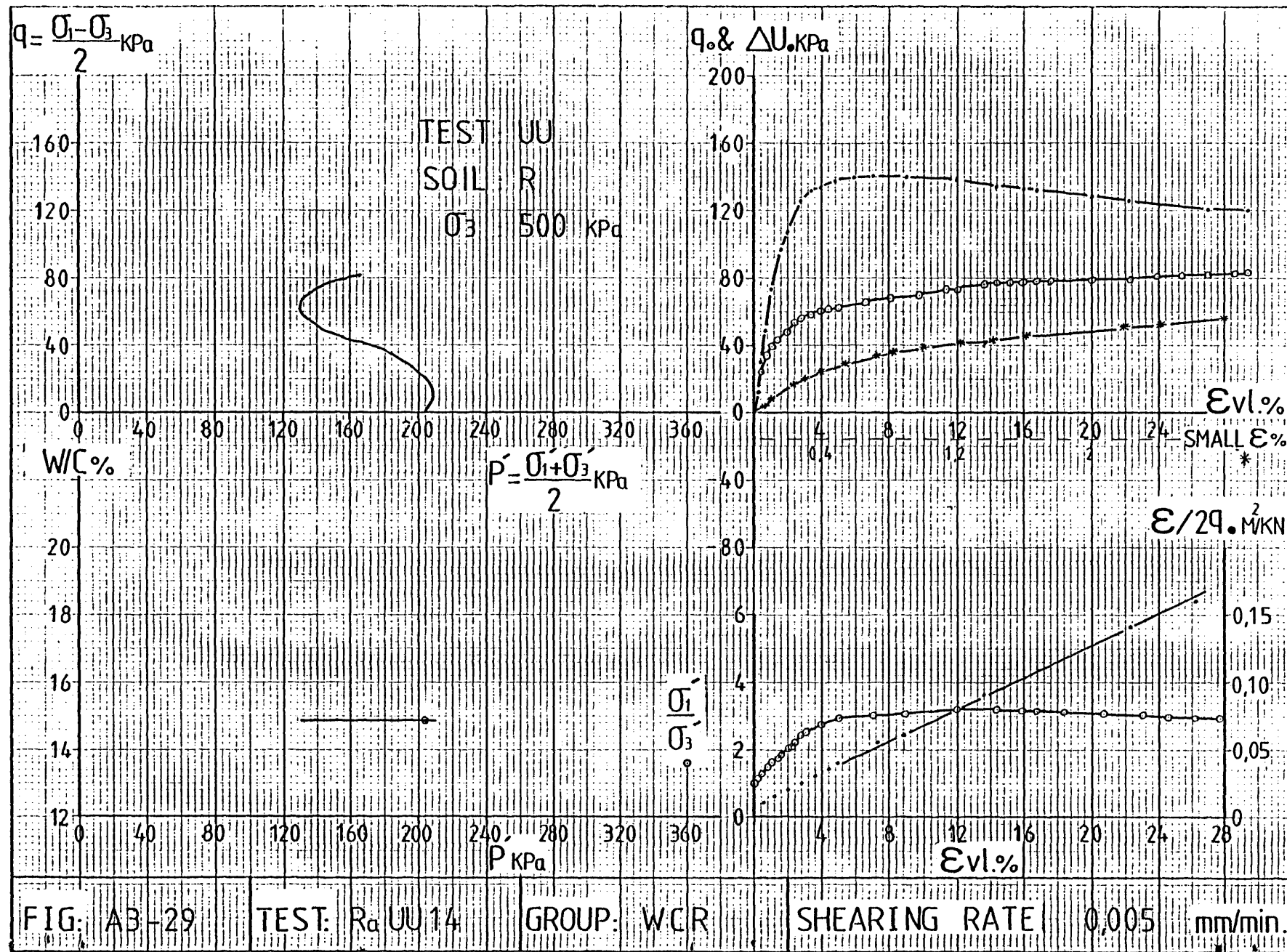


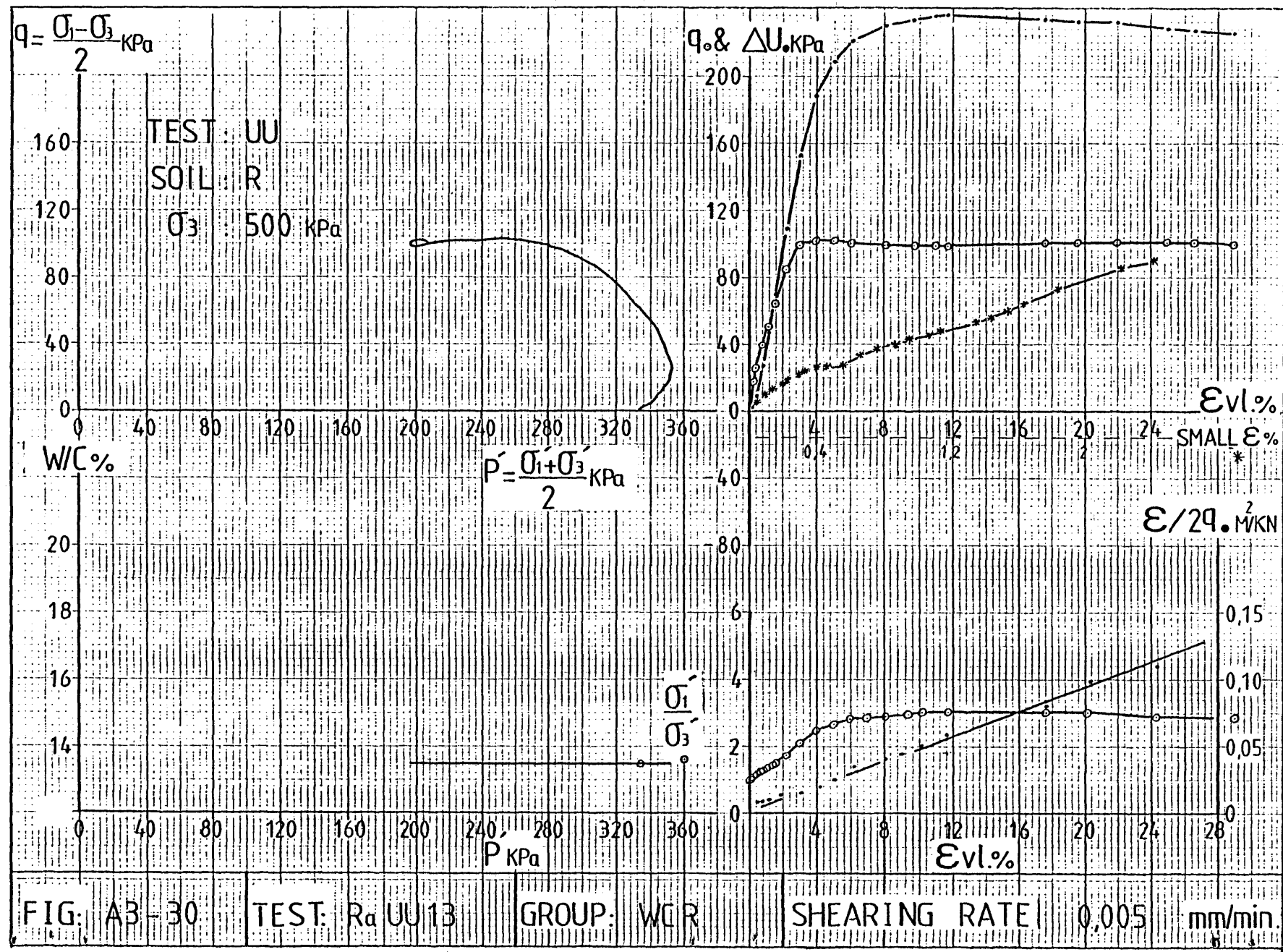


-357-

A3-28

FIG. A3-28 TEST: R_a UU 19 GROUP: WCR SHEARING RATE 0.005 mm/min.





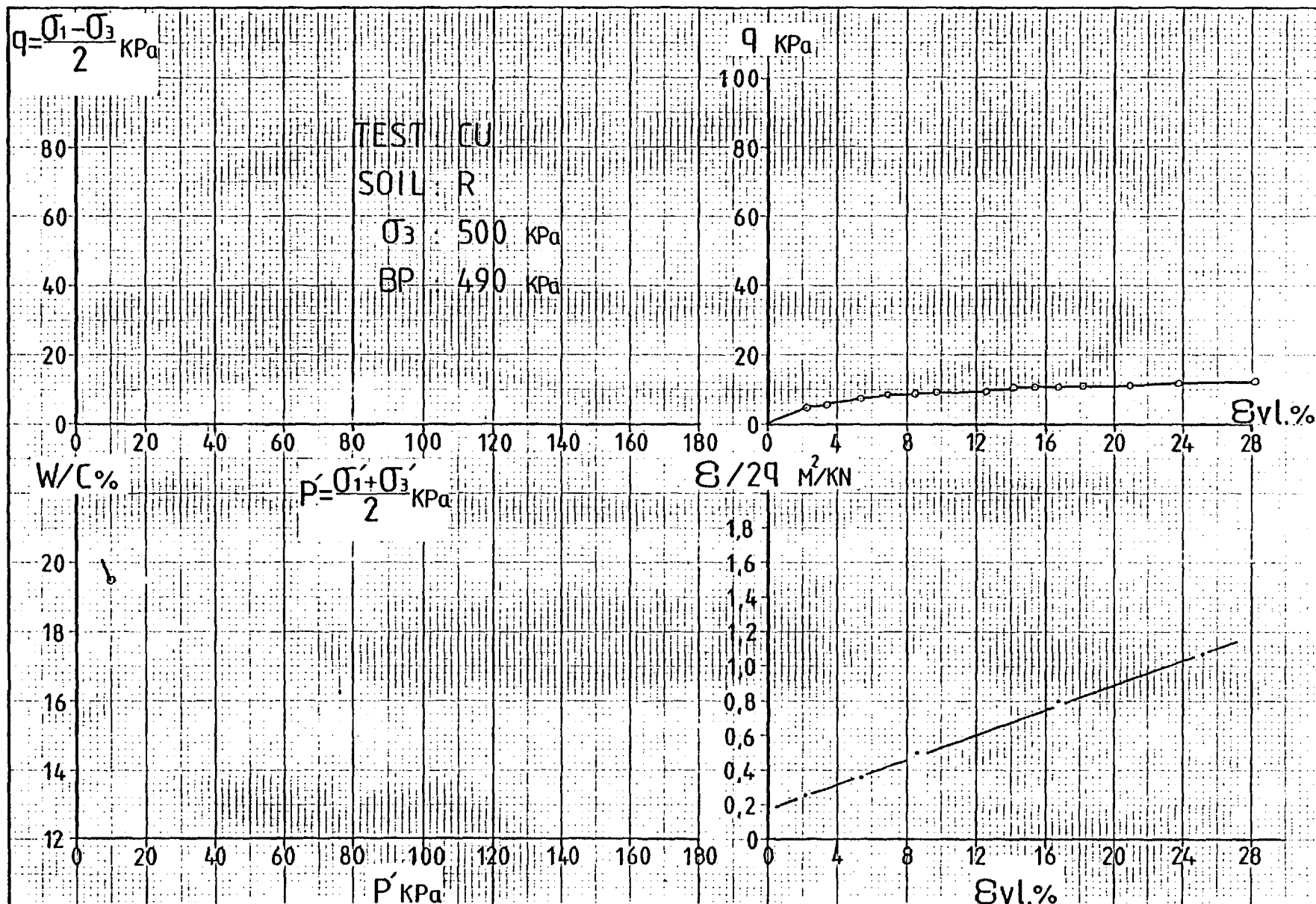
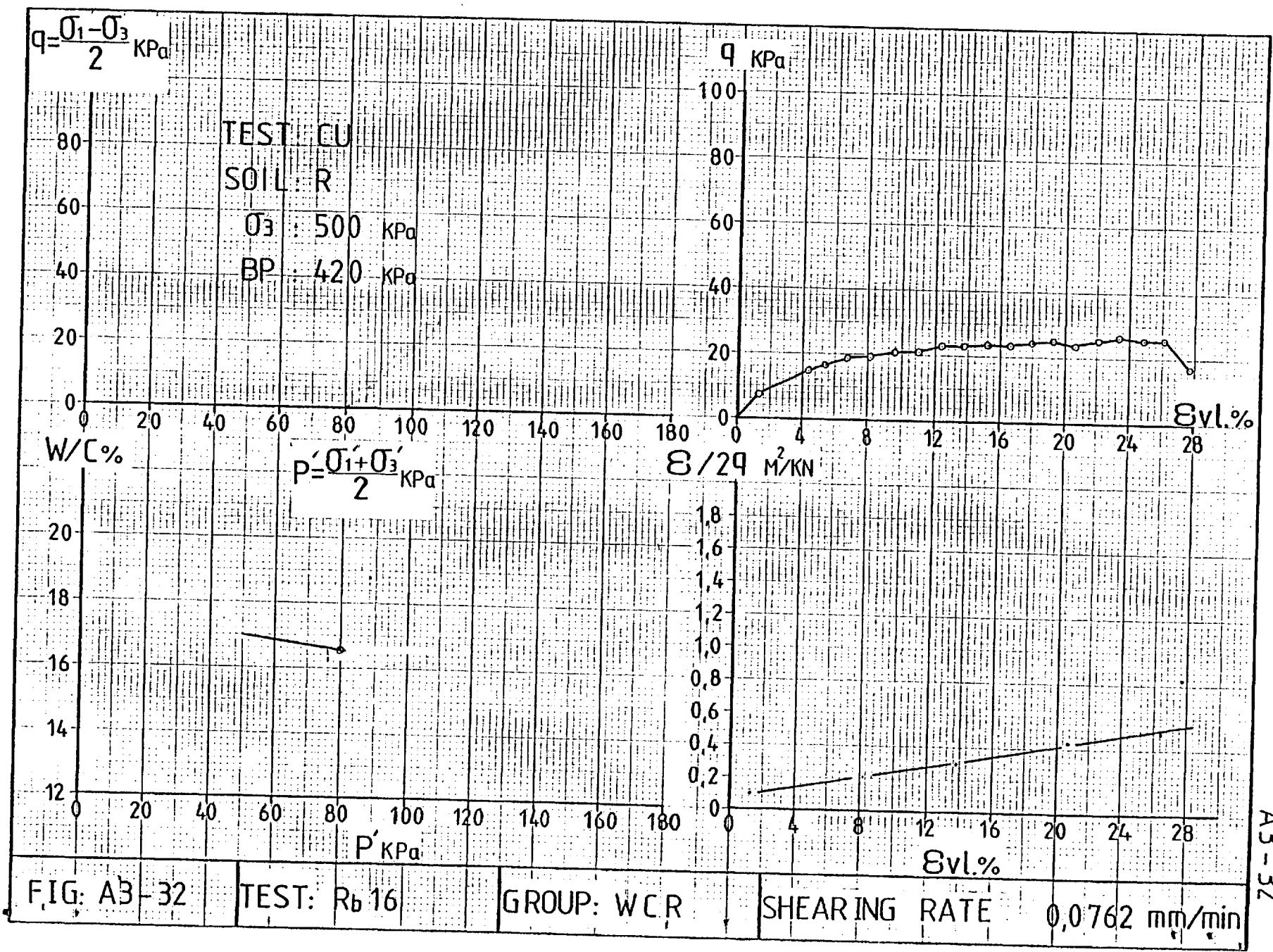


FIG: A3-31 TEST: Rb 19 GROUP: WCR SHEARING RATE 0,0762 mm/min



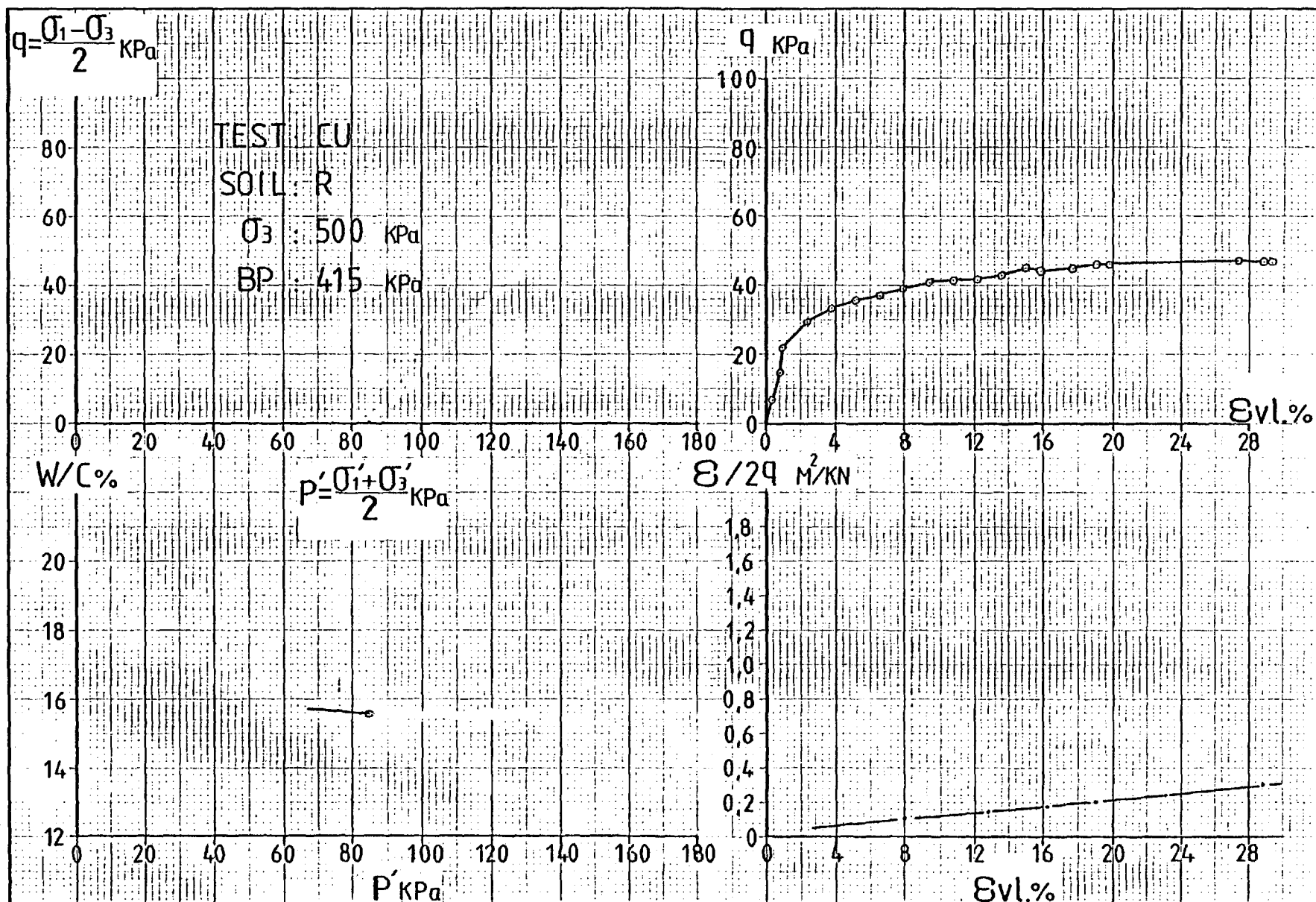
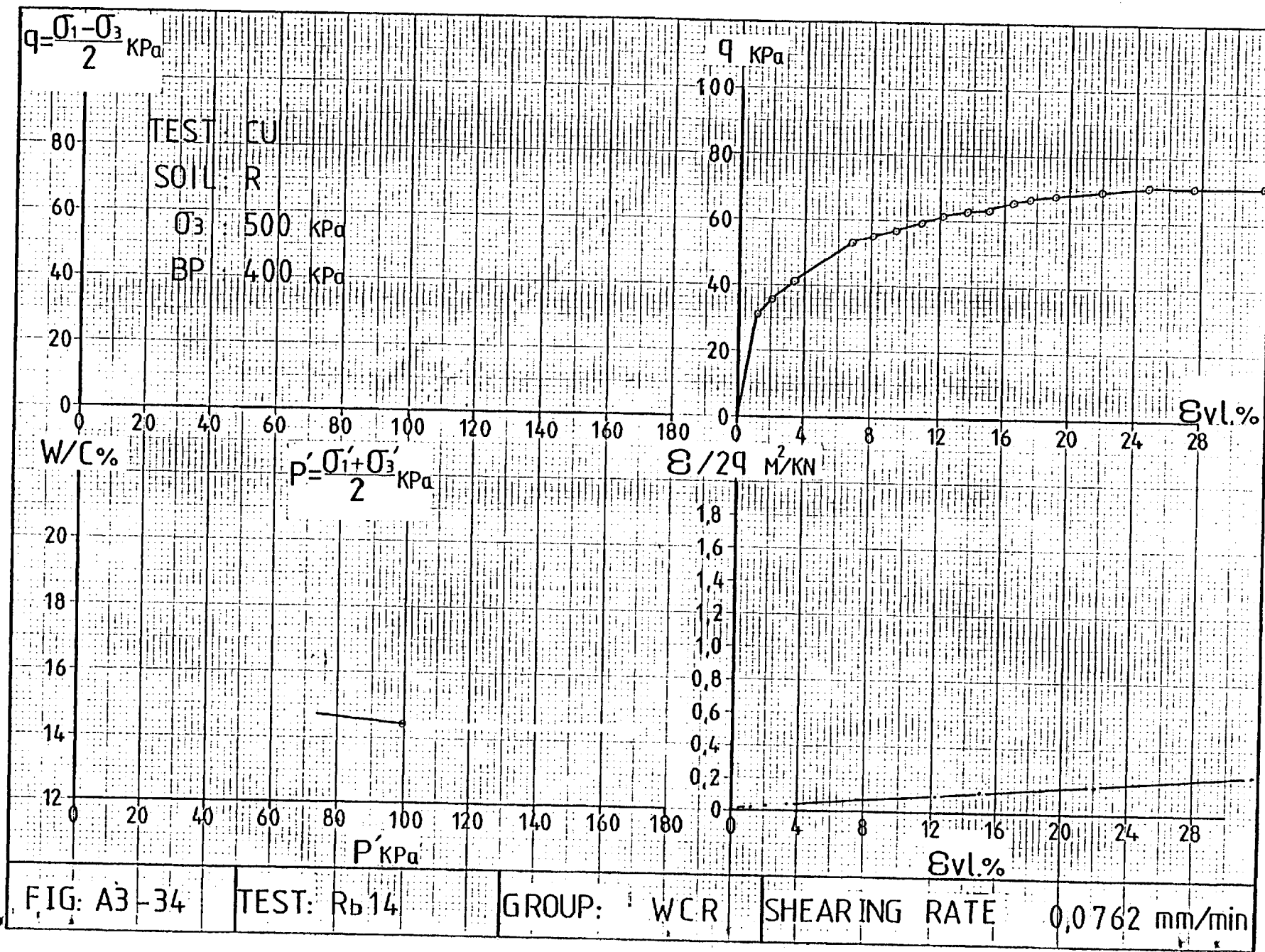
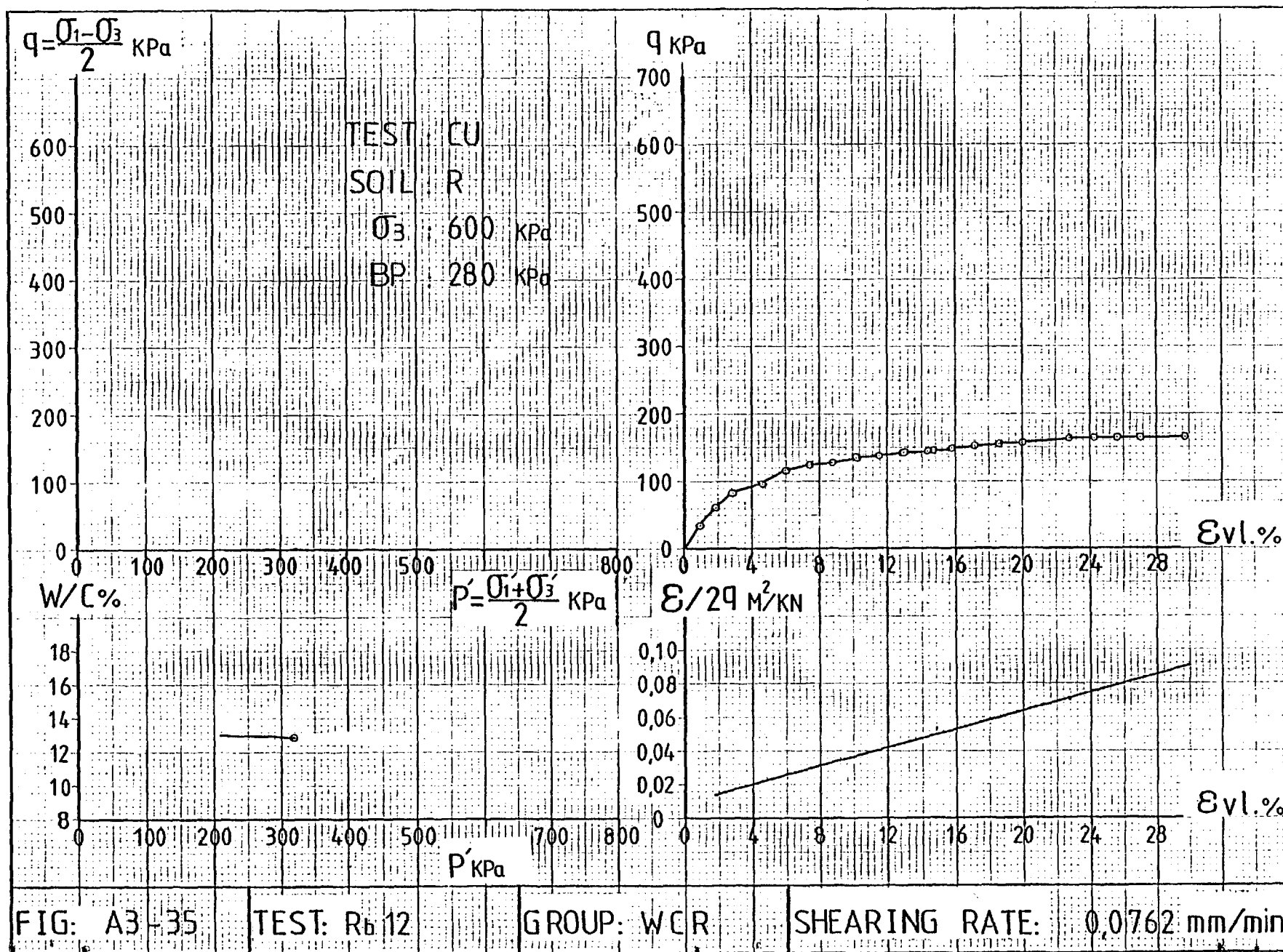
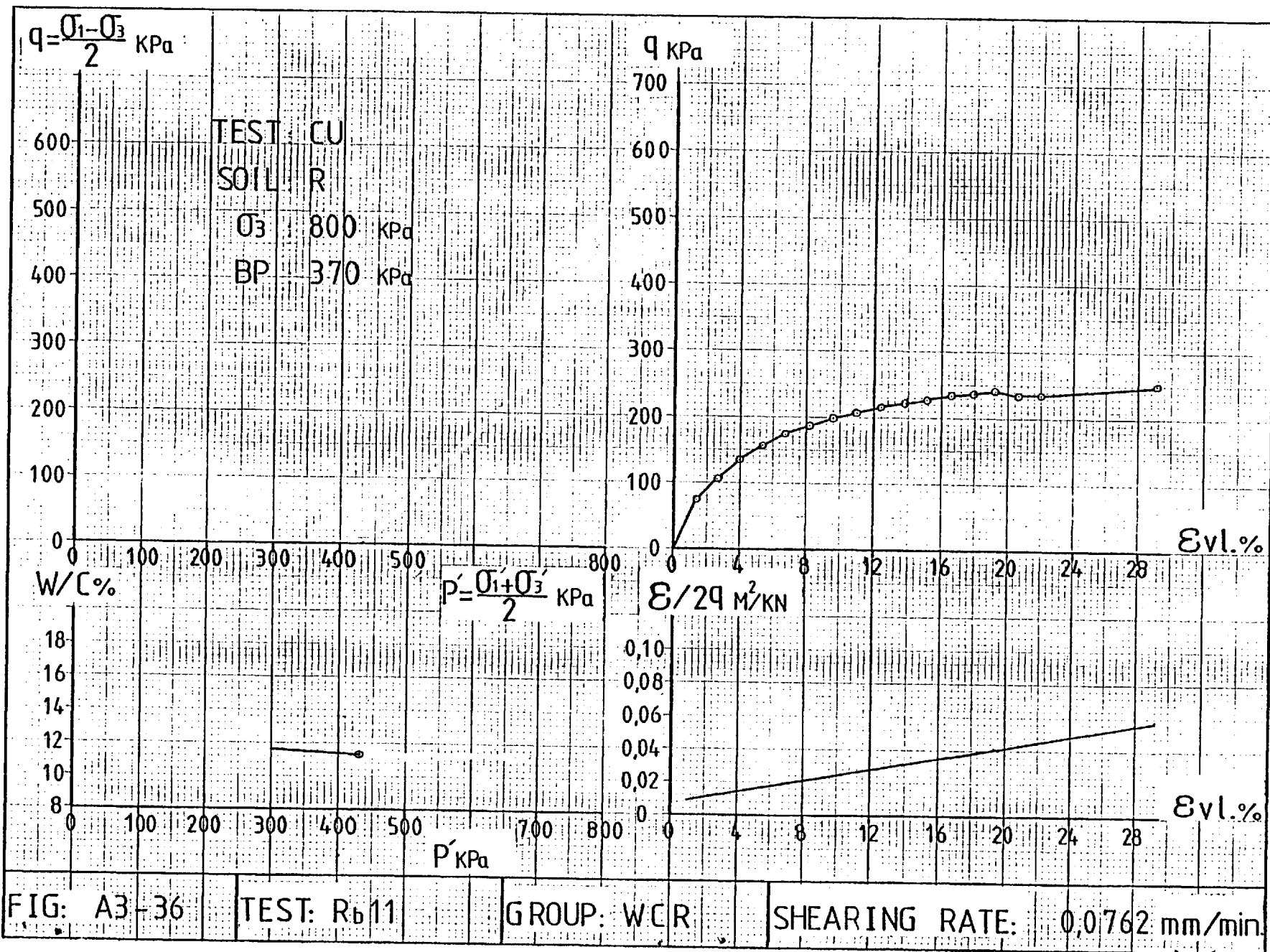


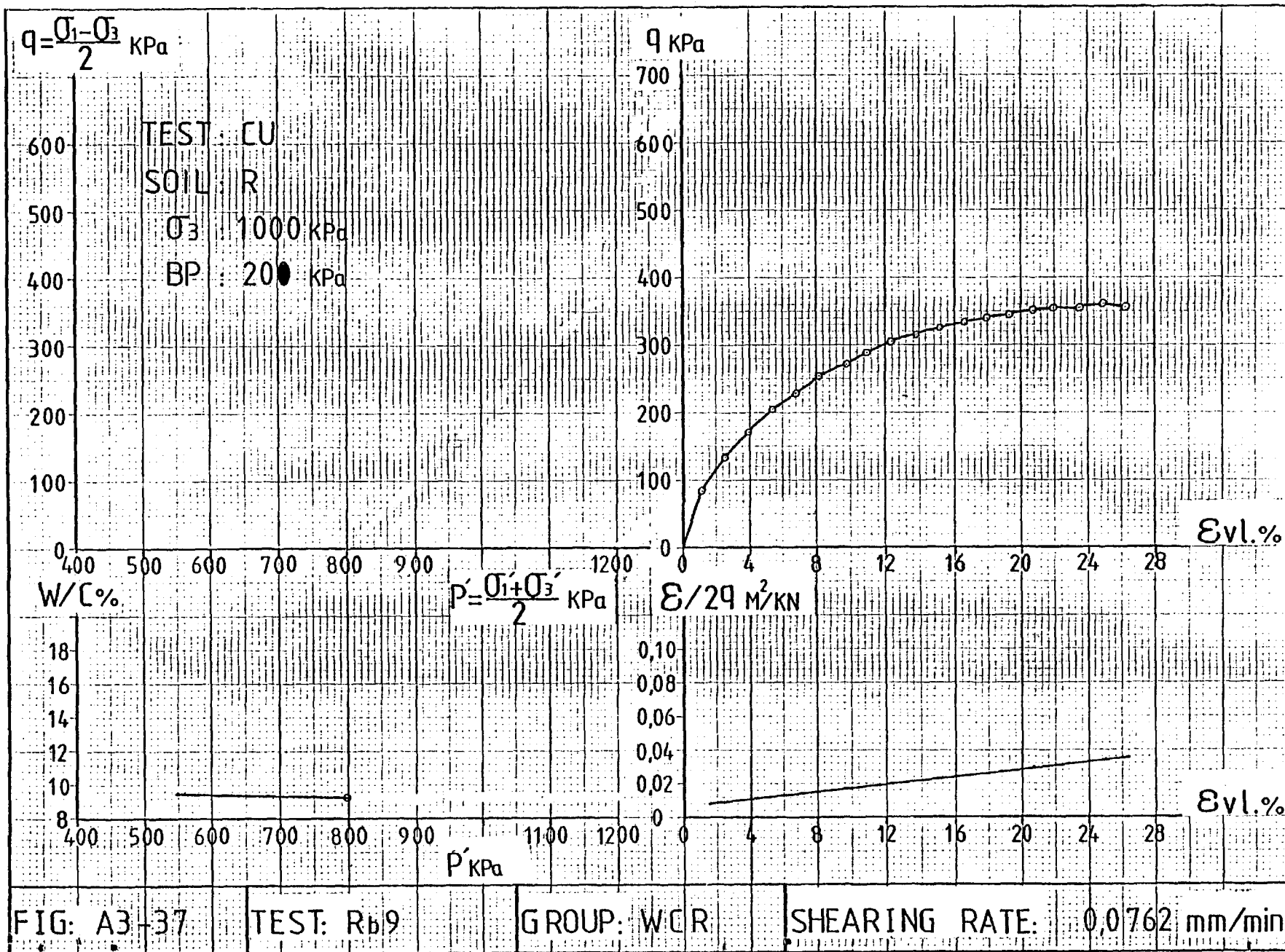
FIG: A3-33 TEST: Rb 15 GROUP: WCR SHEARING RATE 0,0762 mm/min

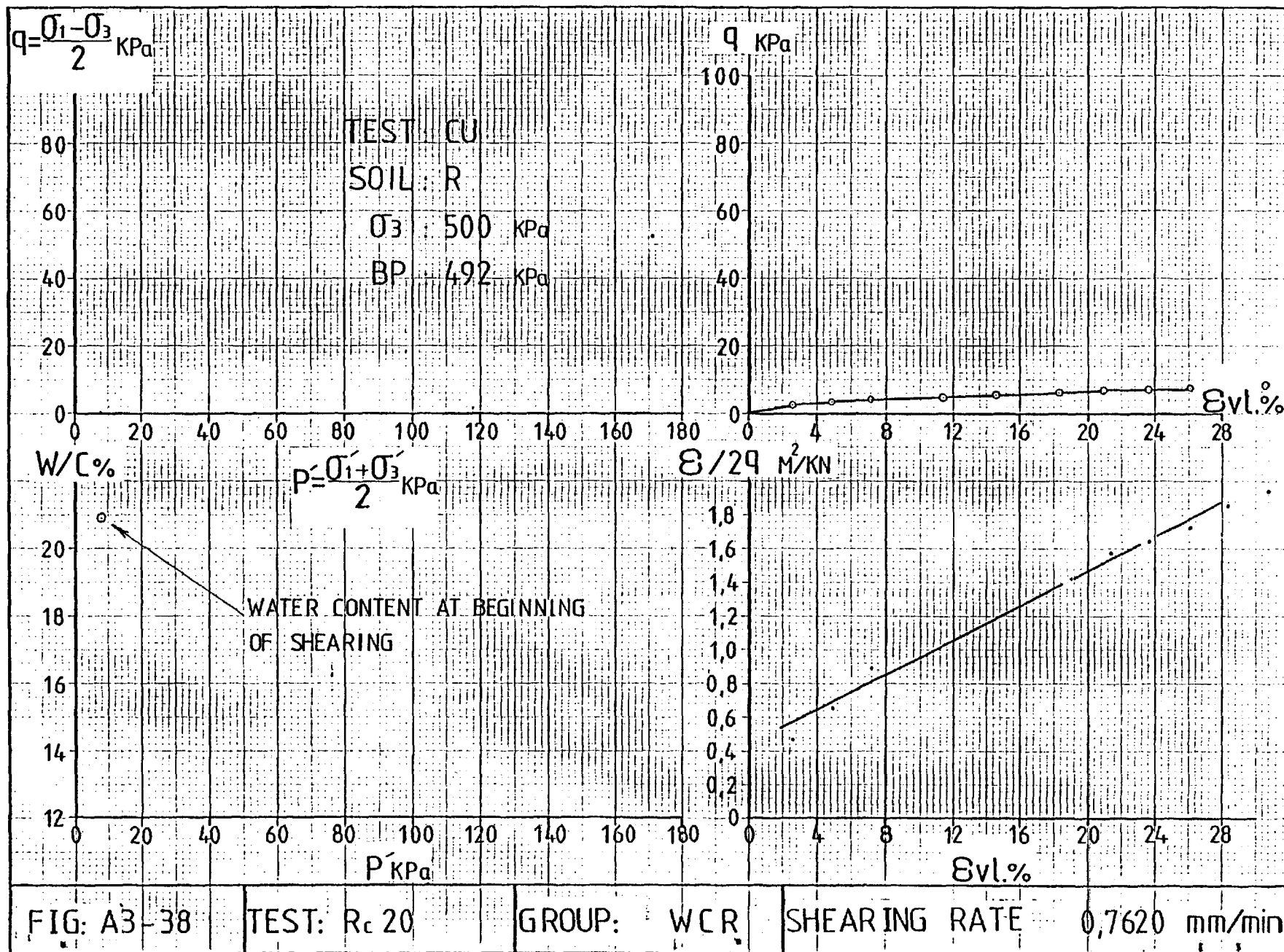
A3-33











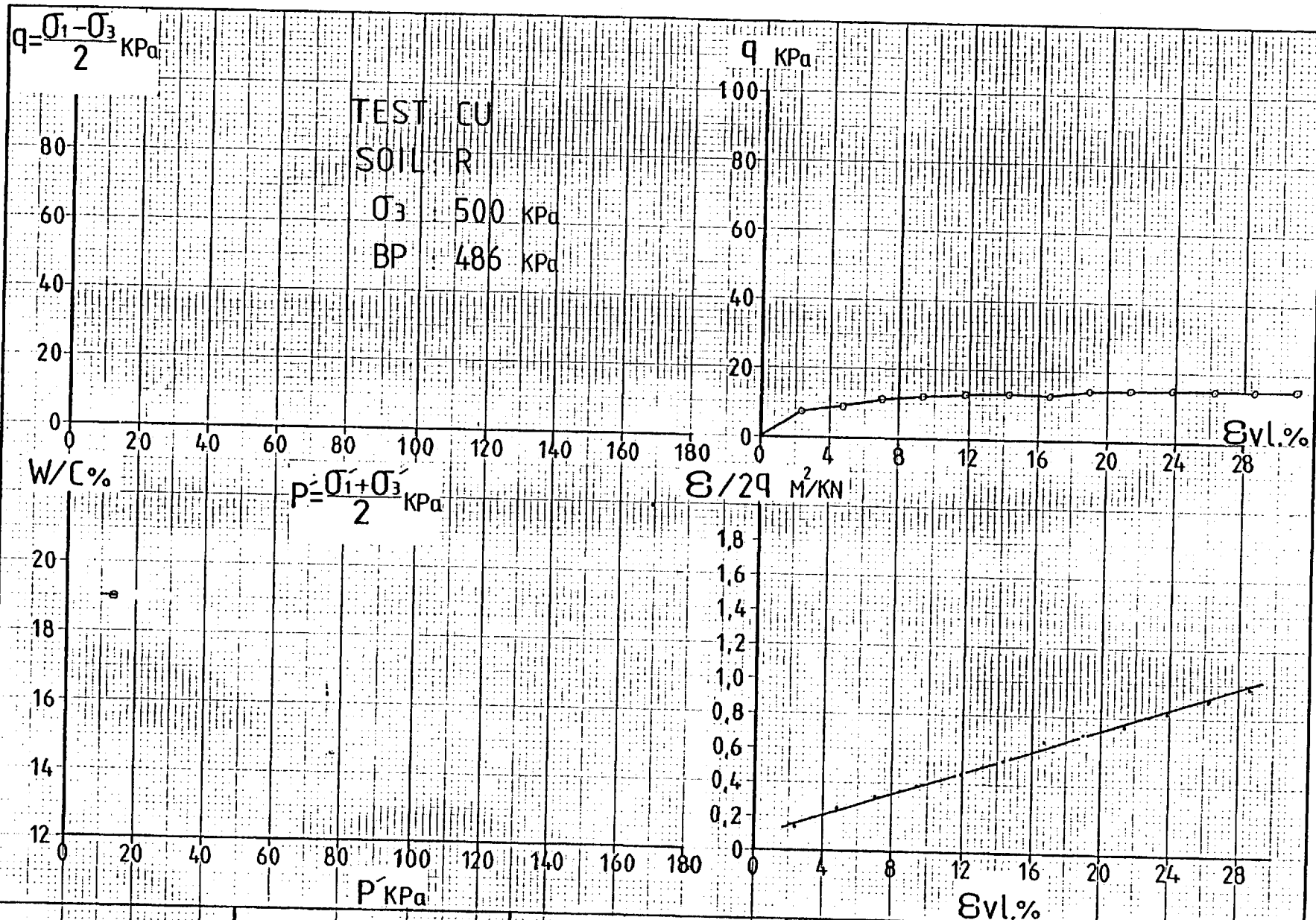
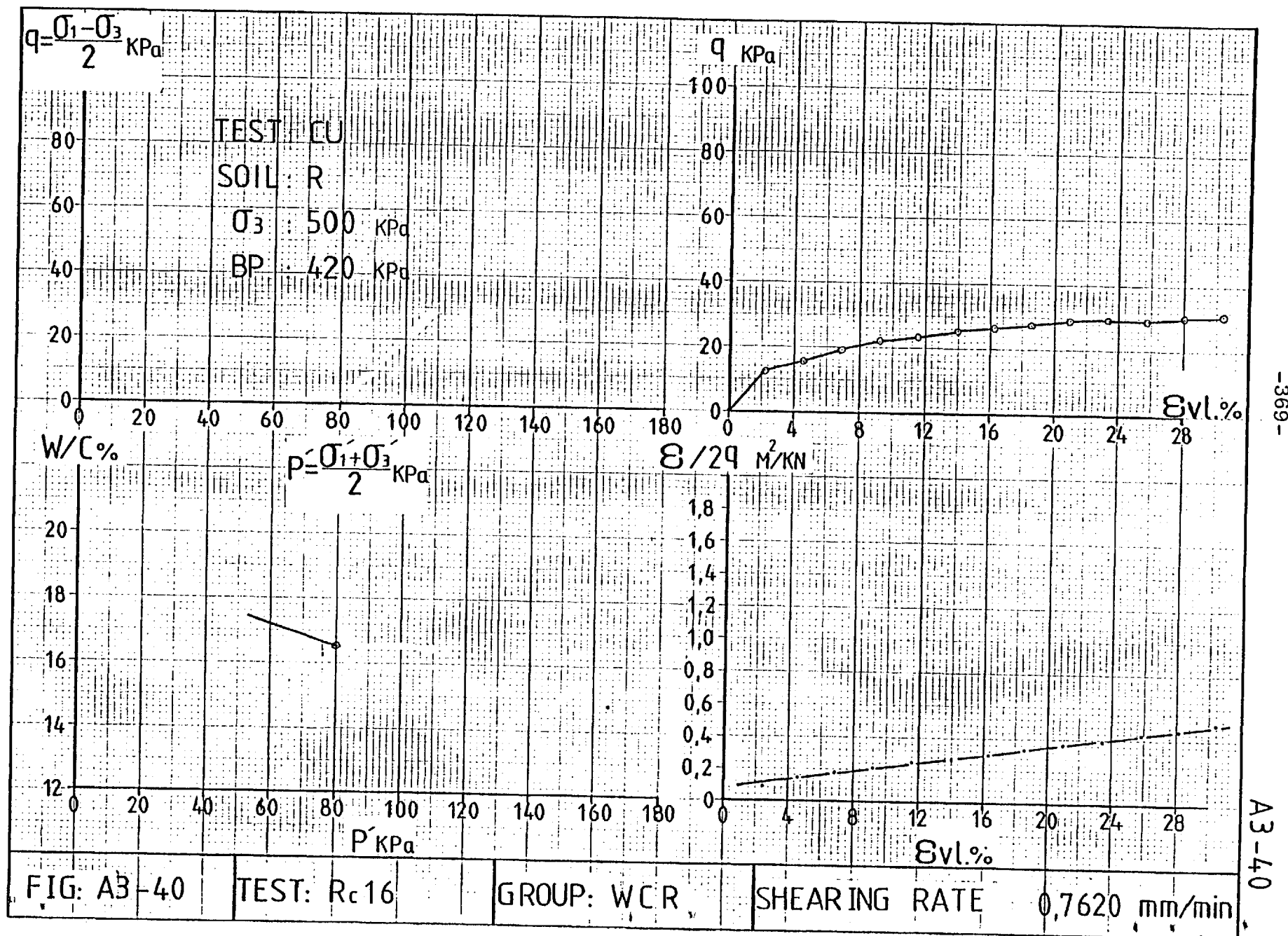


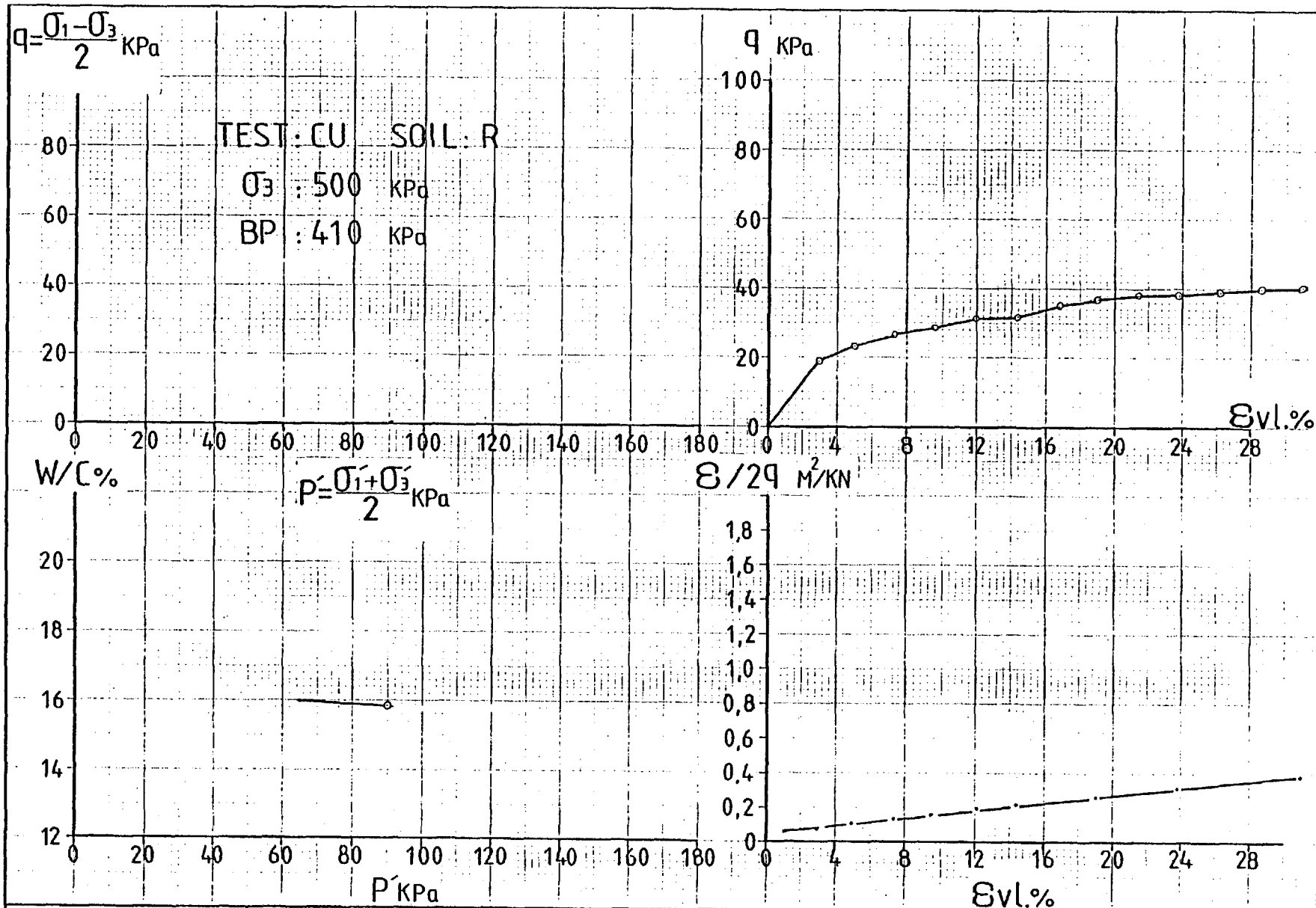
FIG: A3-39

TEST: Rc18

GROUP: WCR

SHEARING RATE: 0,7620 mm/min





-370-

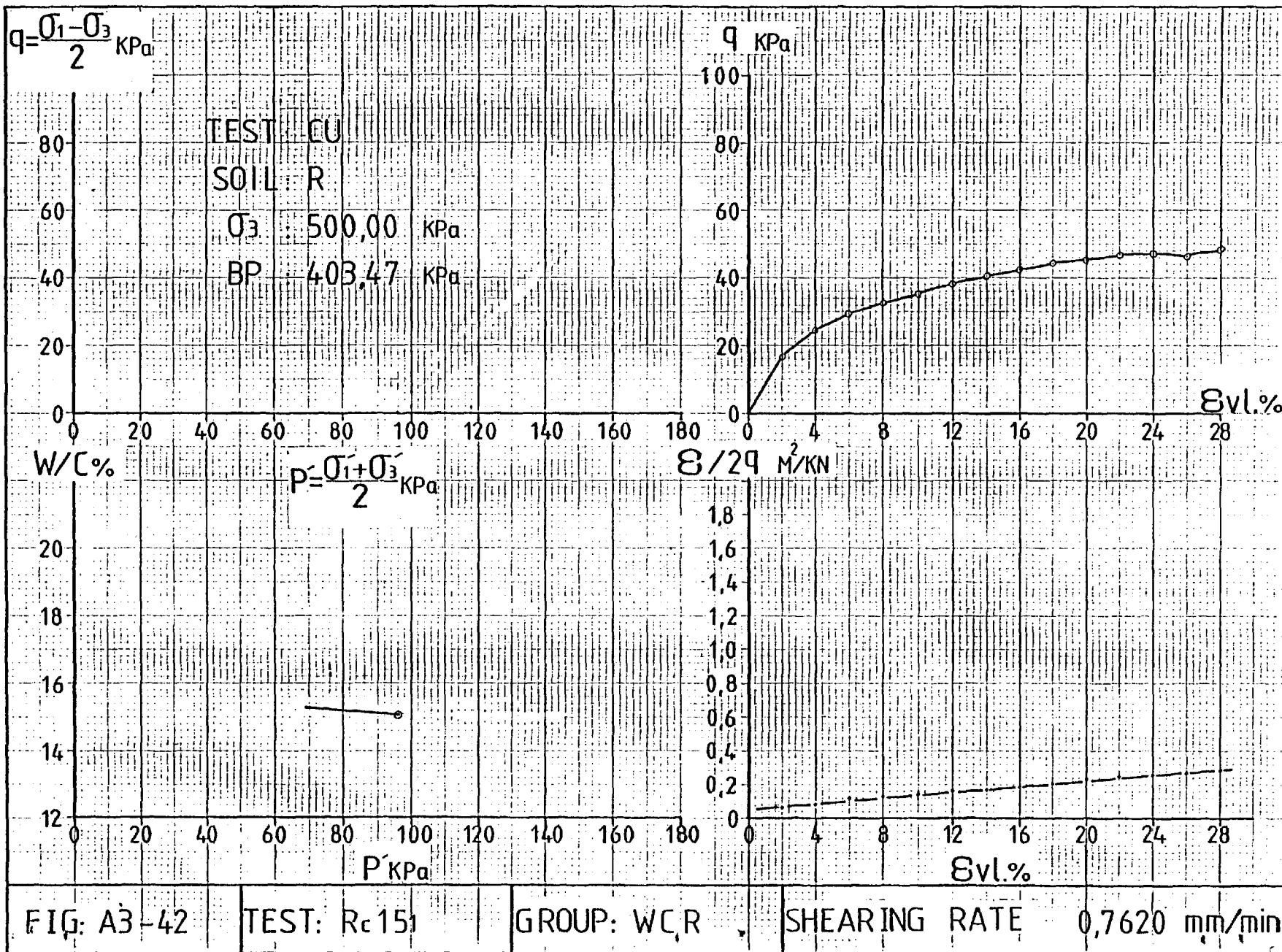
A3-41

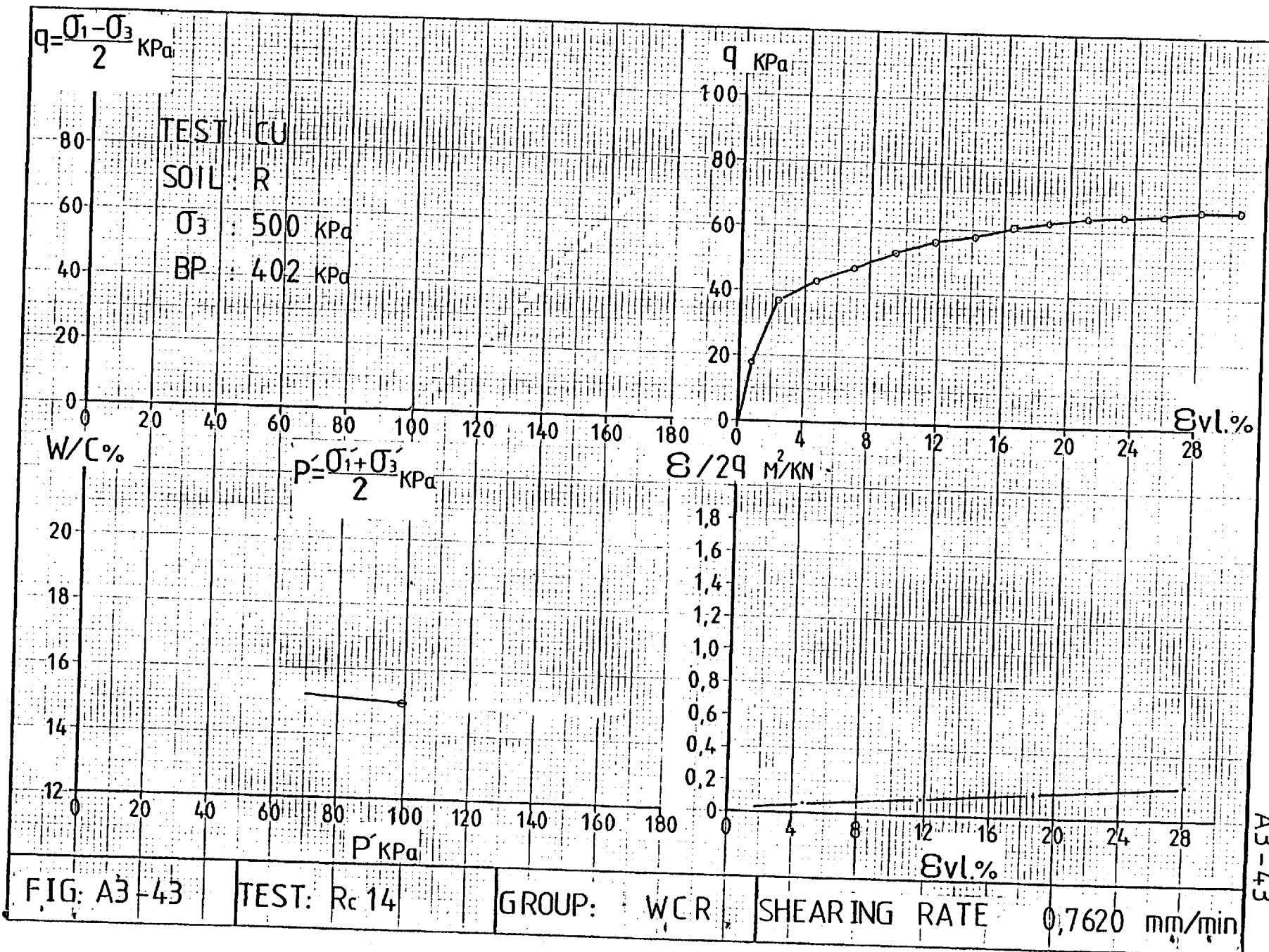
FIG: A3-41

TEST: R_c 15

GROUP: WCR

SHEARING RATE 0.7620 mm/min





A3-43

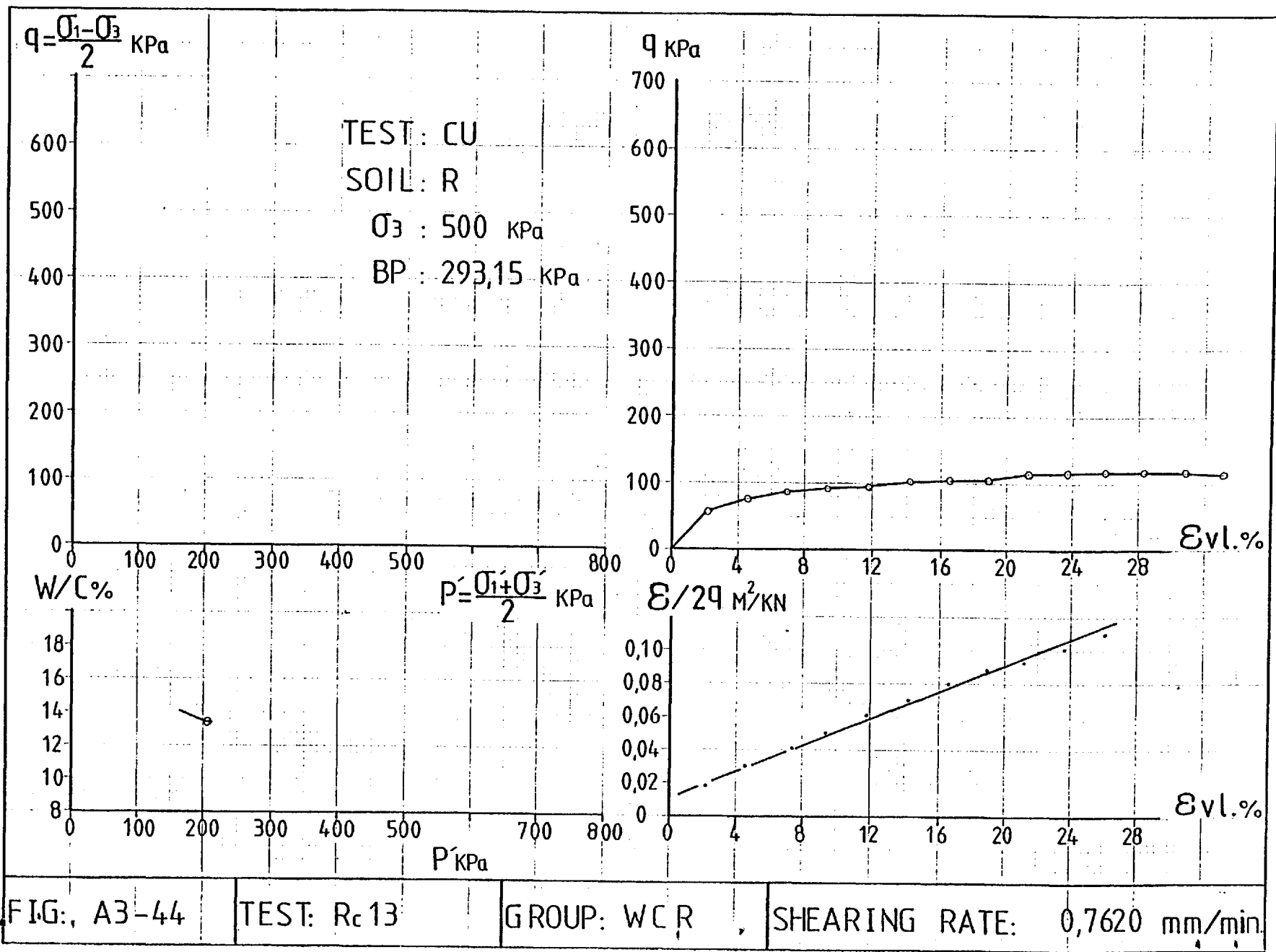
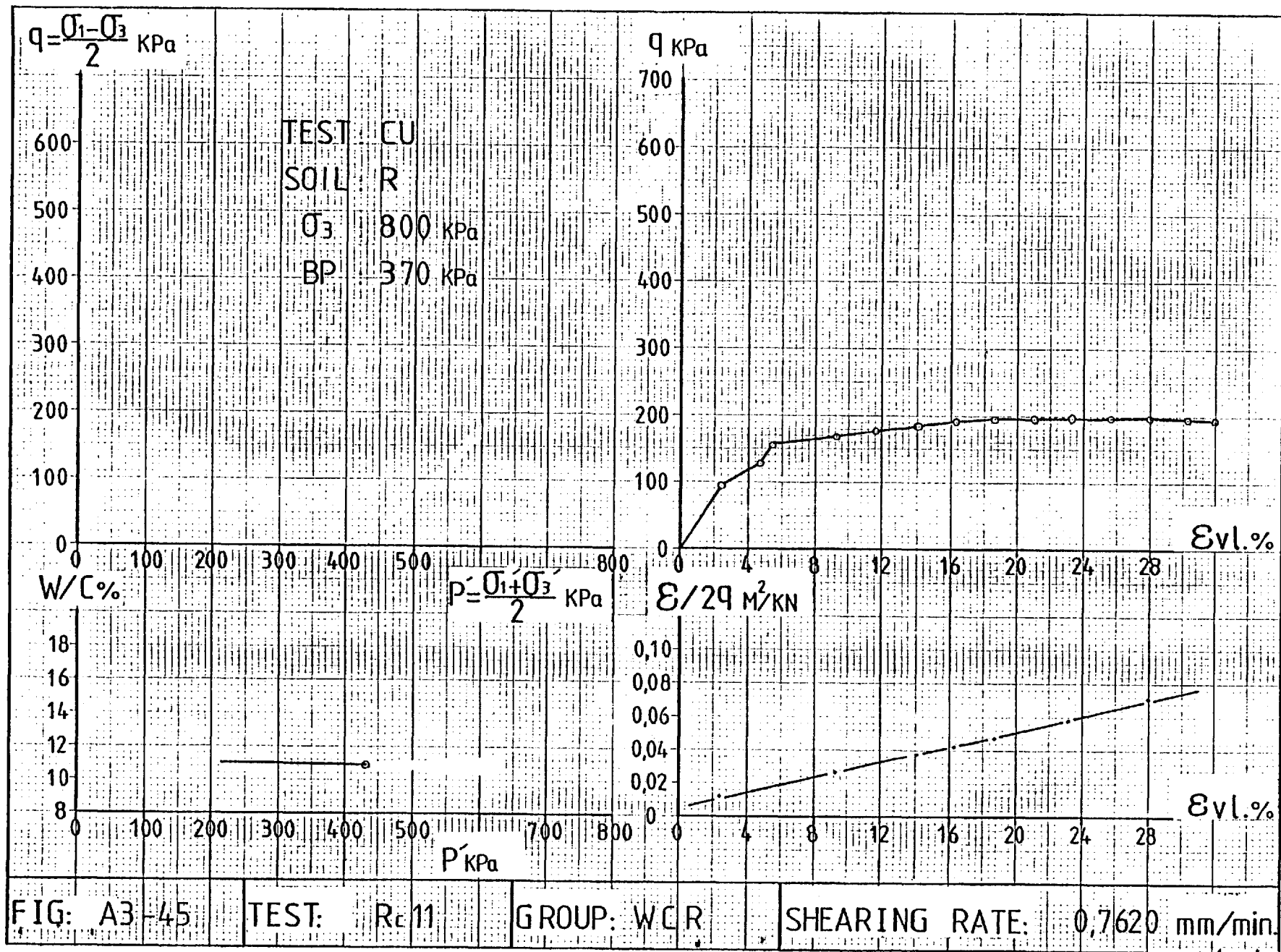


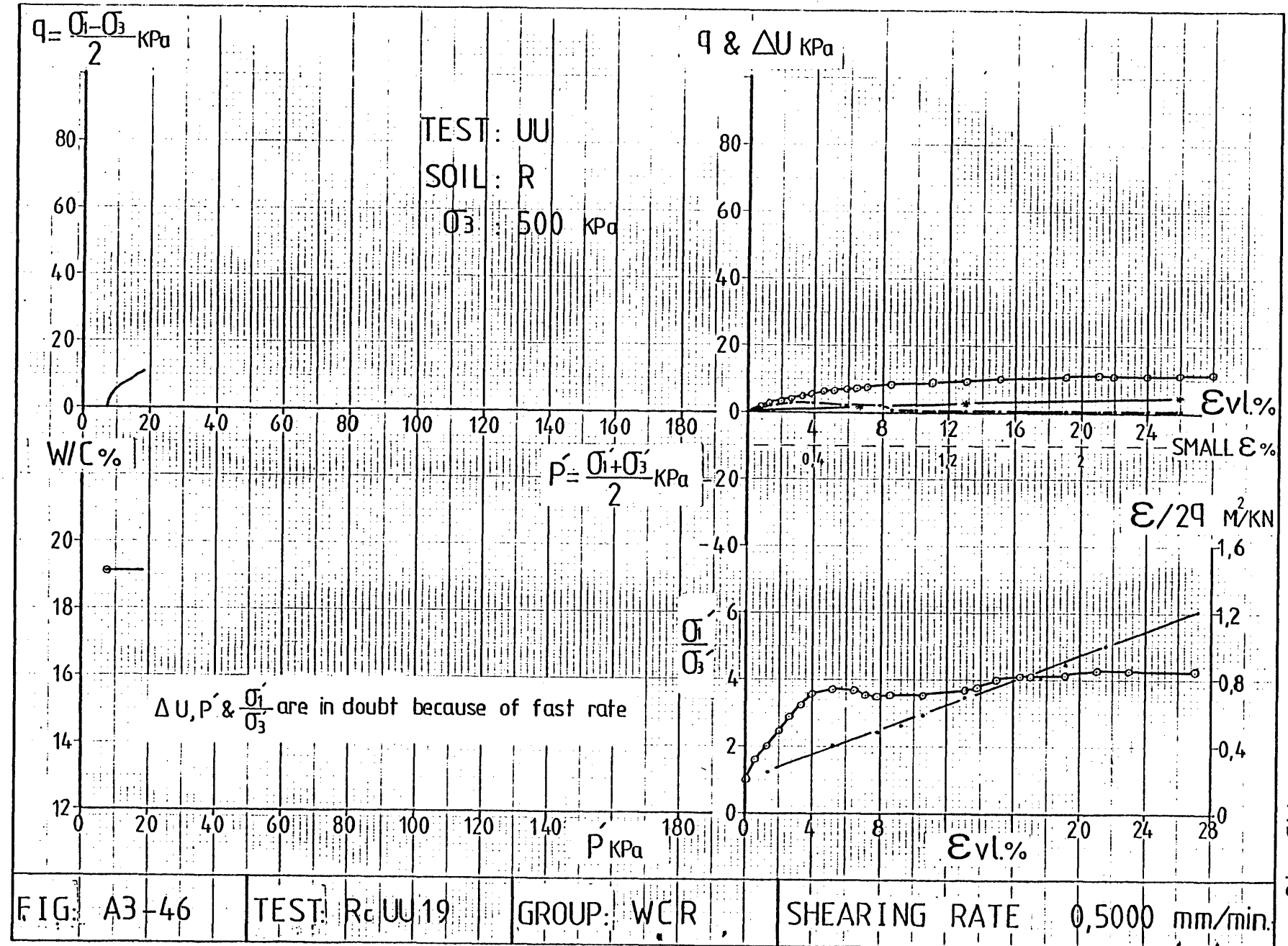
FIG.: A3-44

TEST: R_c 13

GROUP: WCR

SHEARING RATE: 0,7620 mm/min.





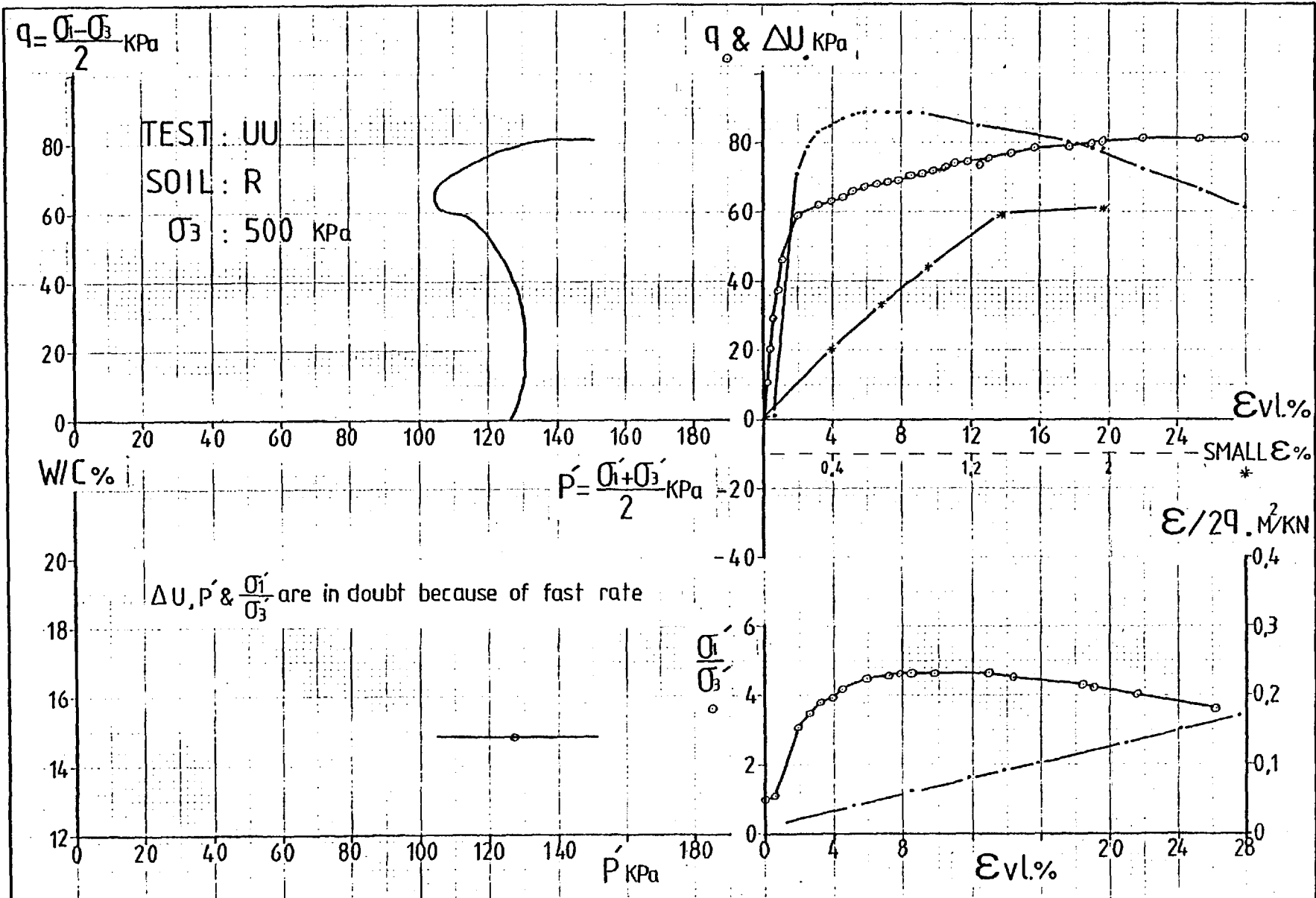
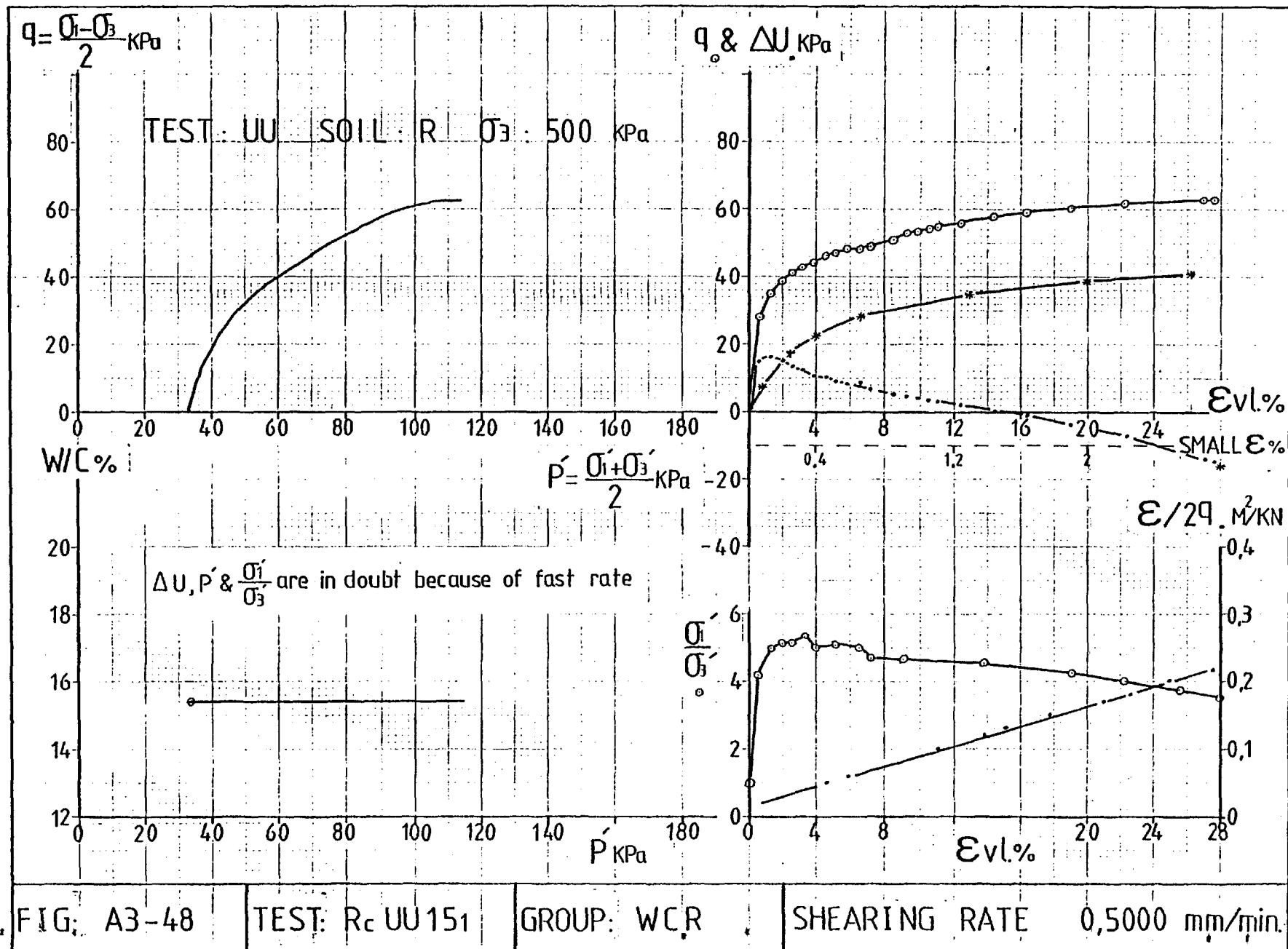


FIG: A3-47 TEST: R_c UU15 GROUP: WCR SHEARING RATE 0,5000 mm/min.



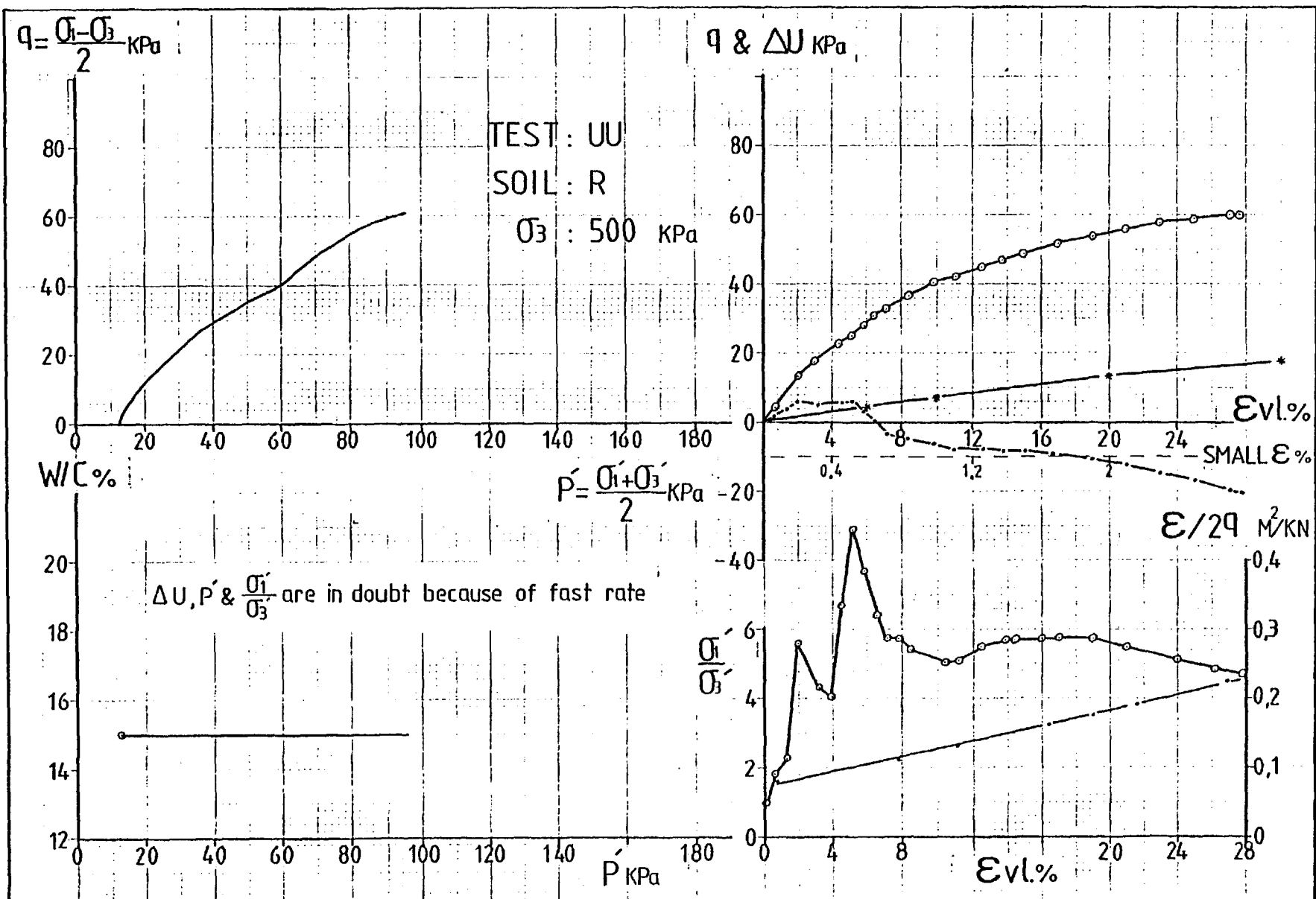
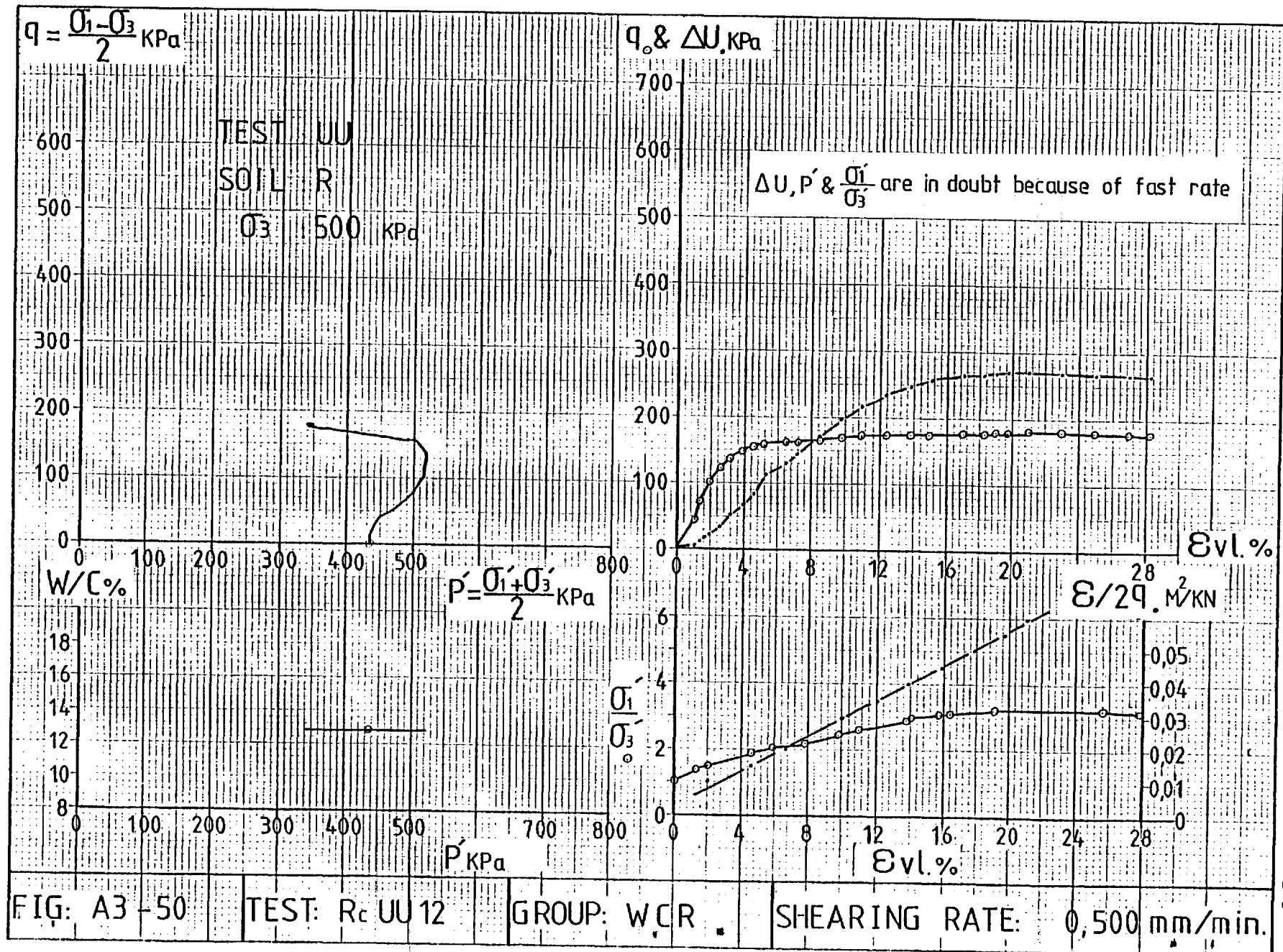
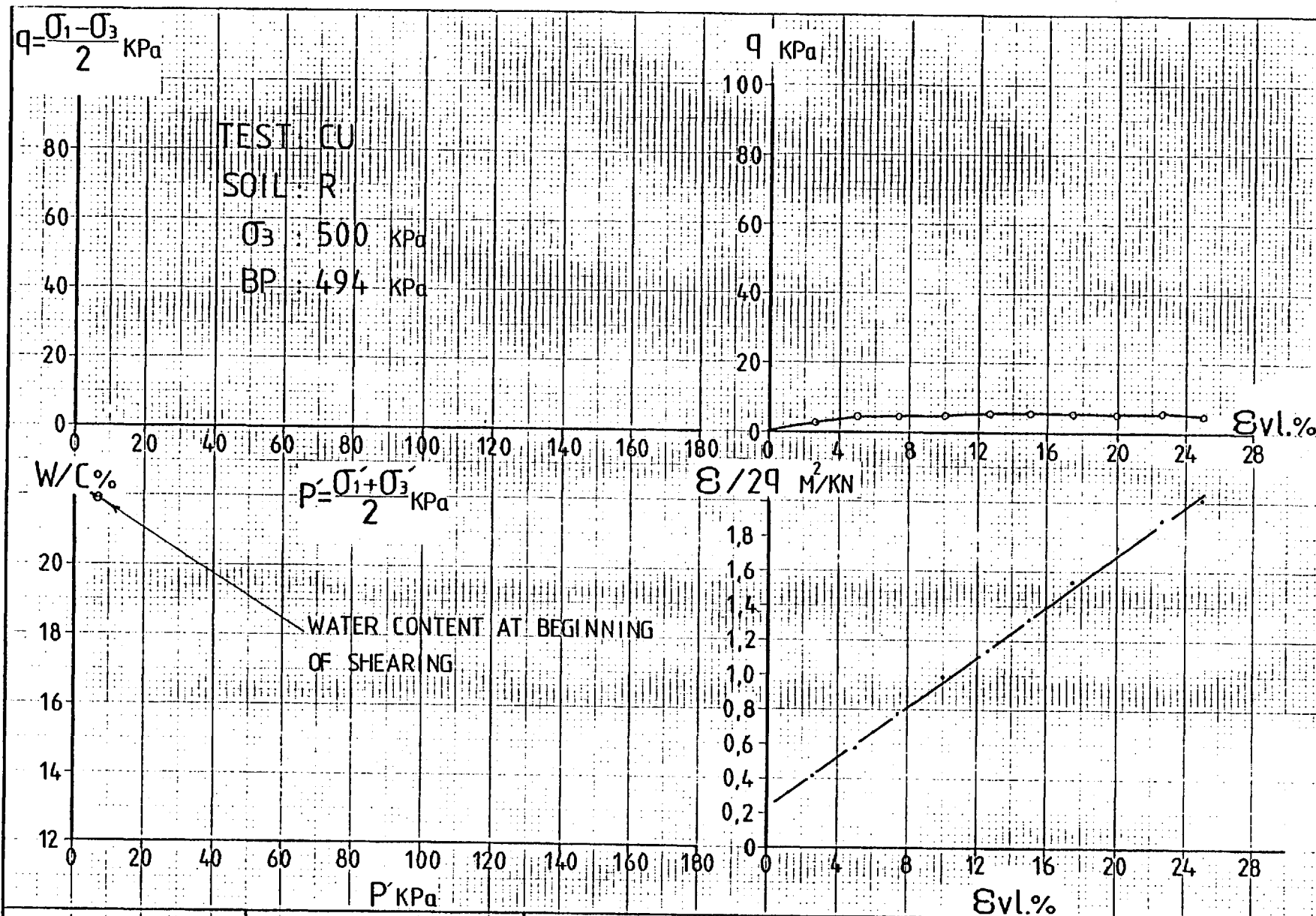
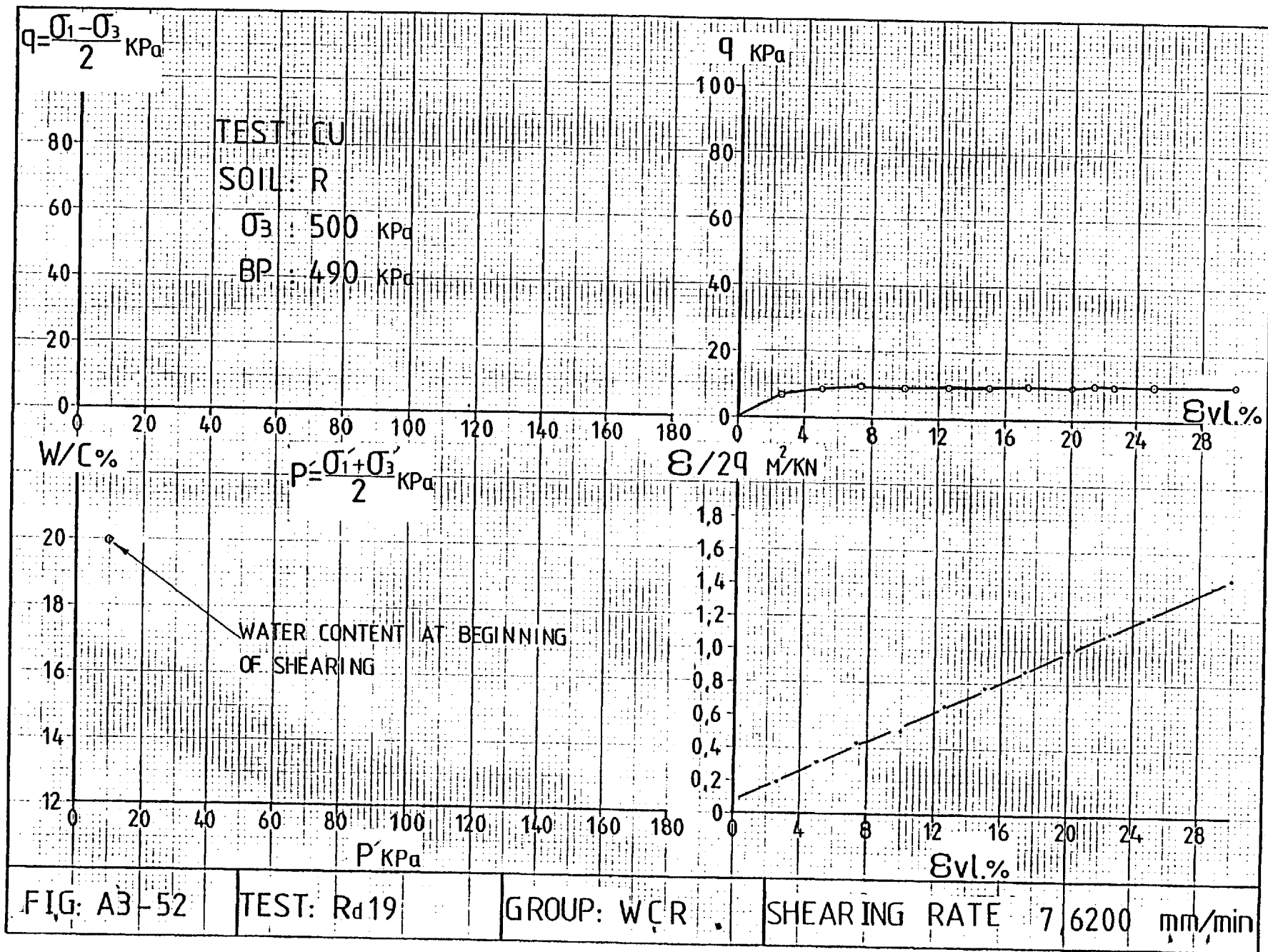
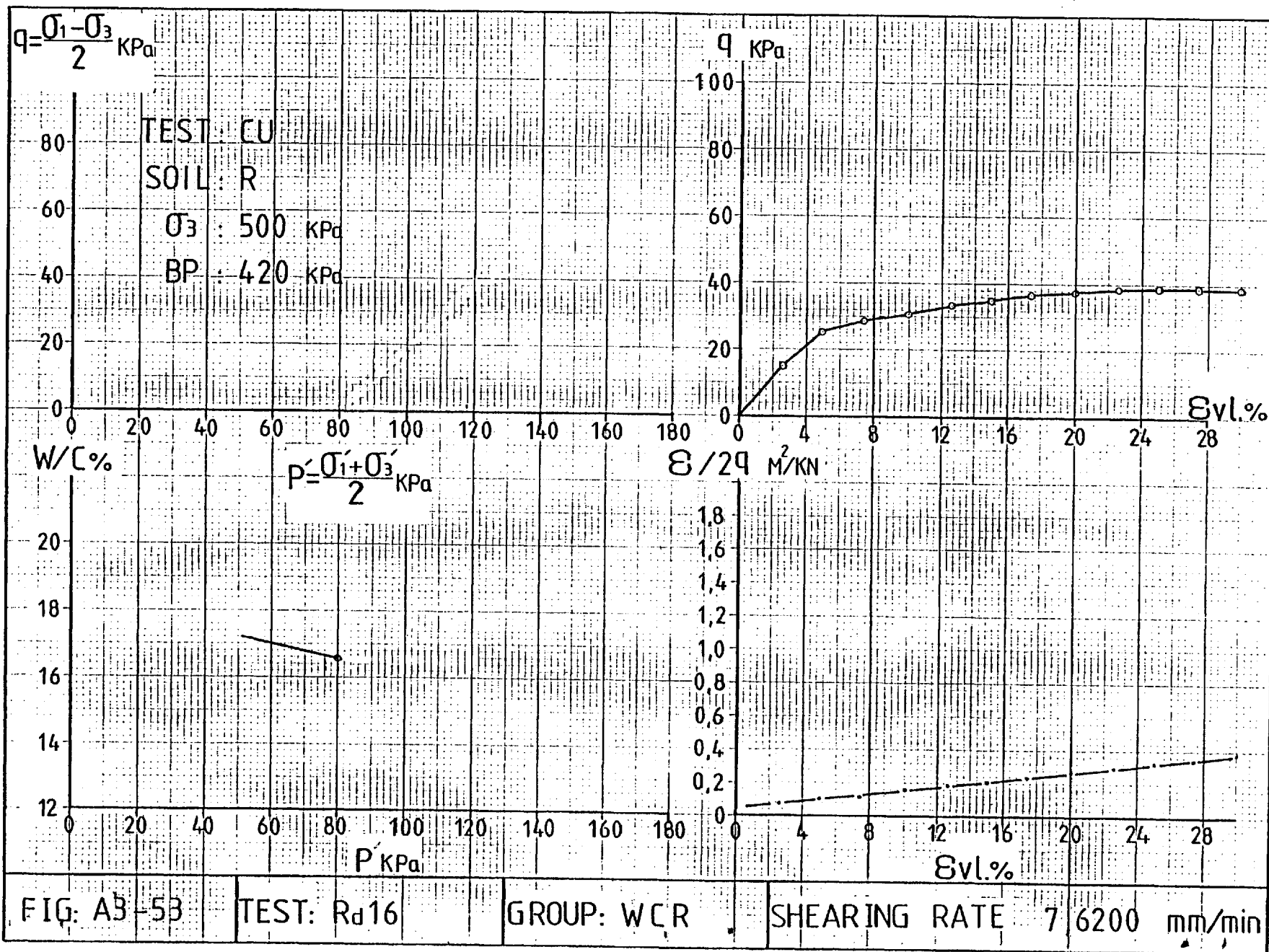


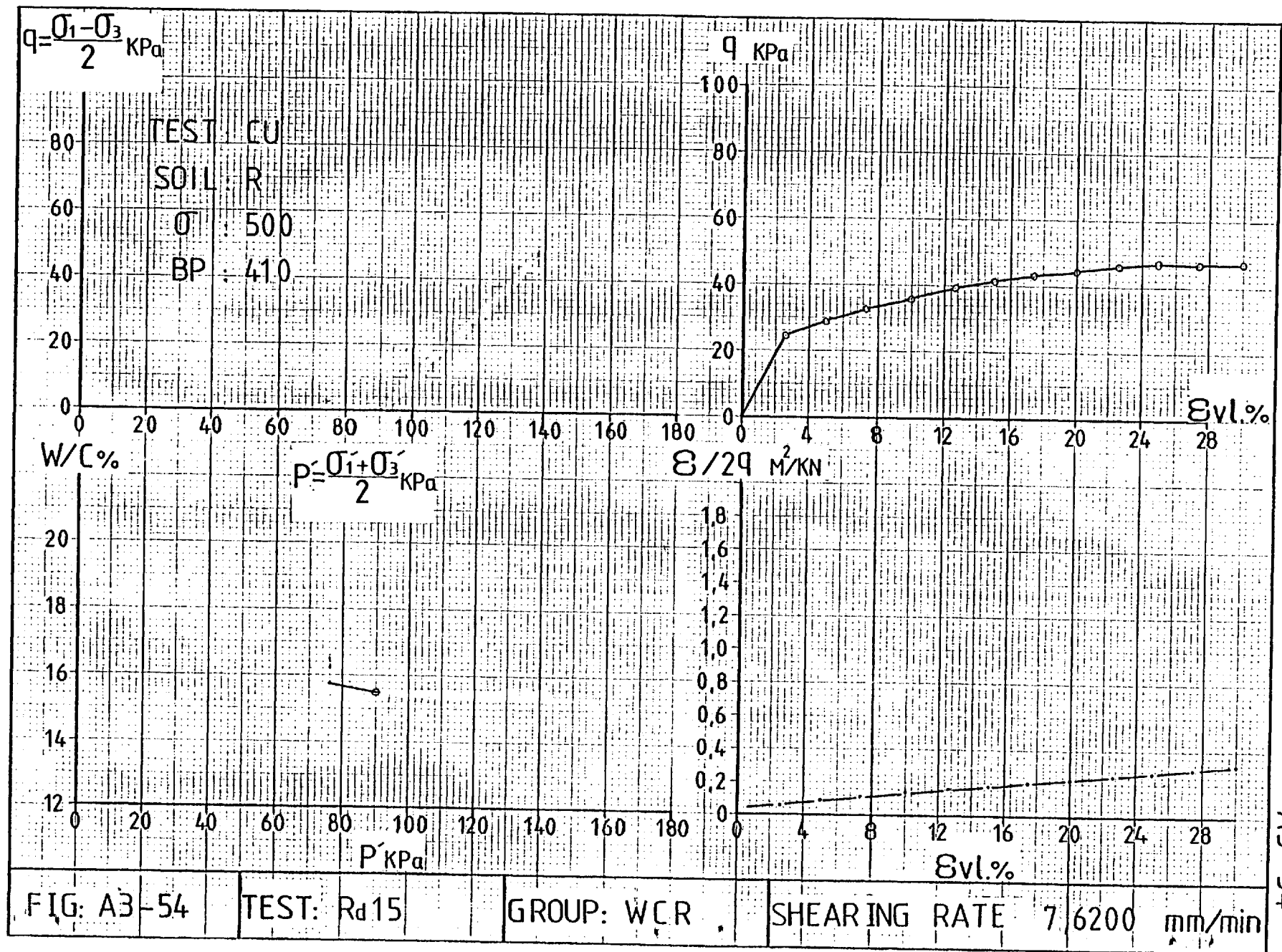
FIG: A3-49	TEST: R _c UU152	GROUP: WCR	SHEARING RATE 0,5000 mm/min.
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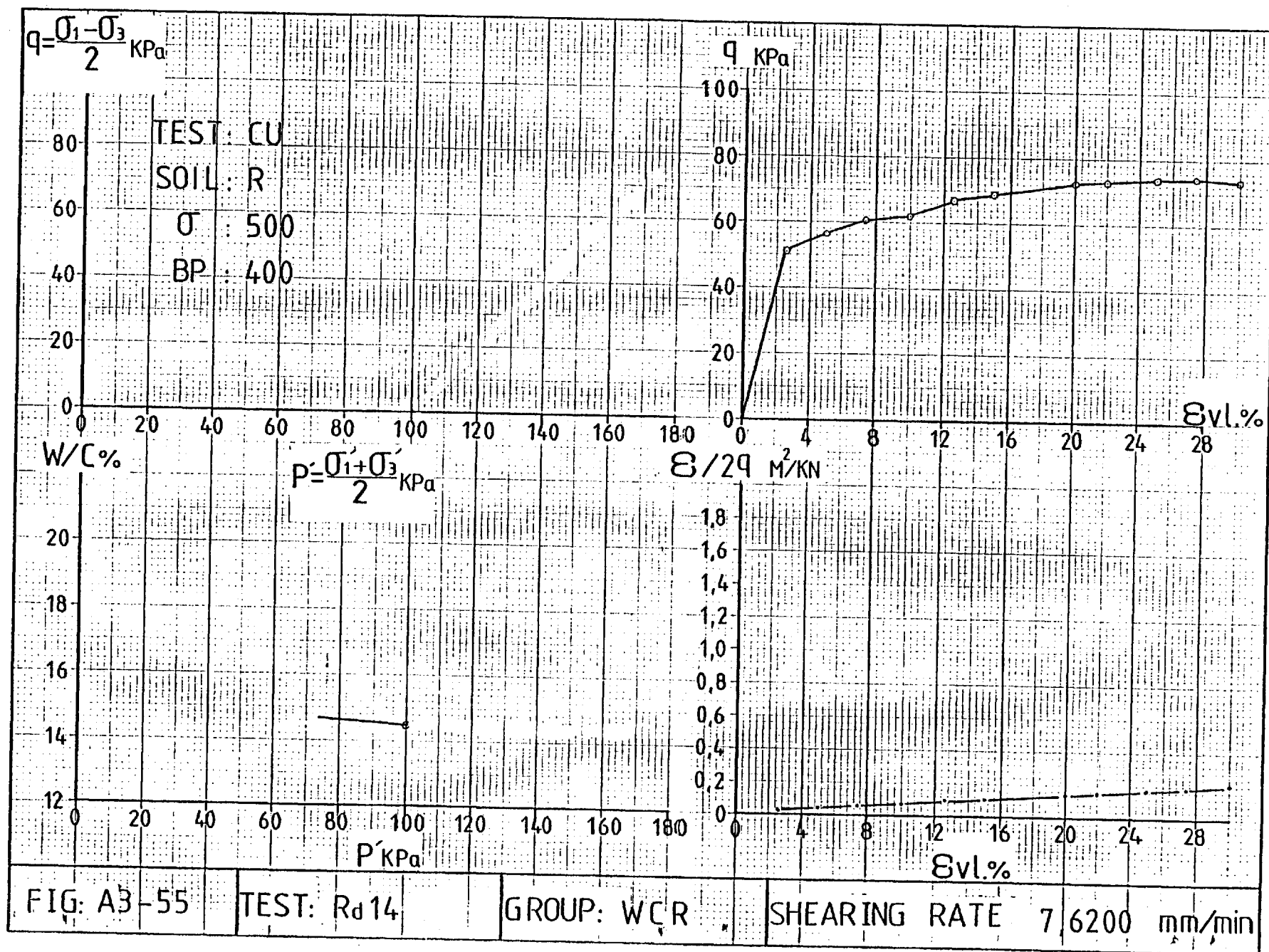


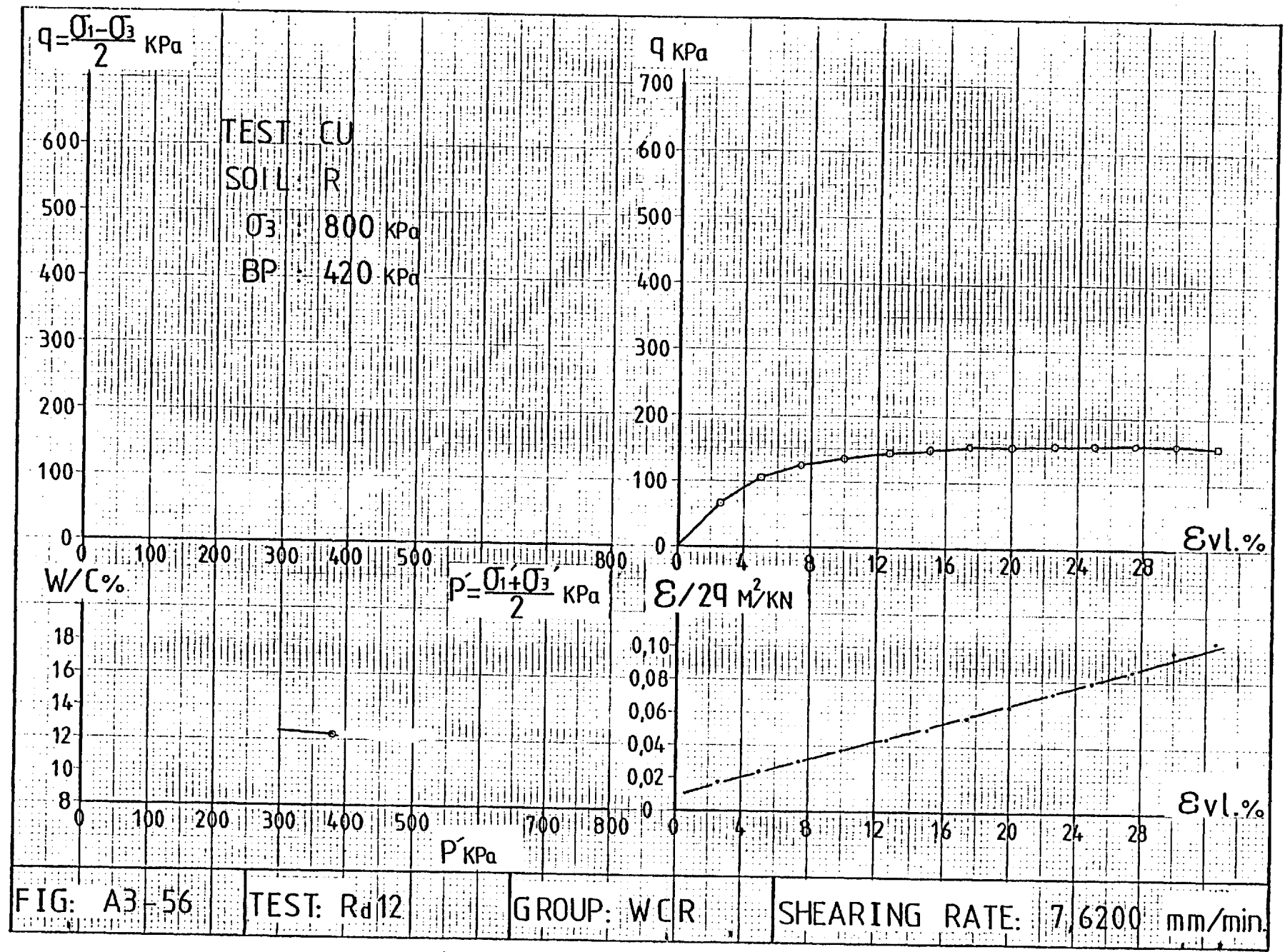


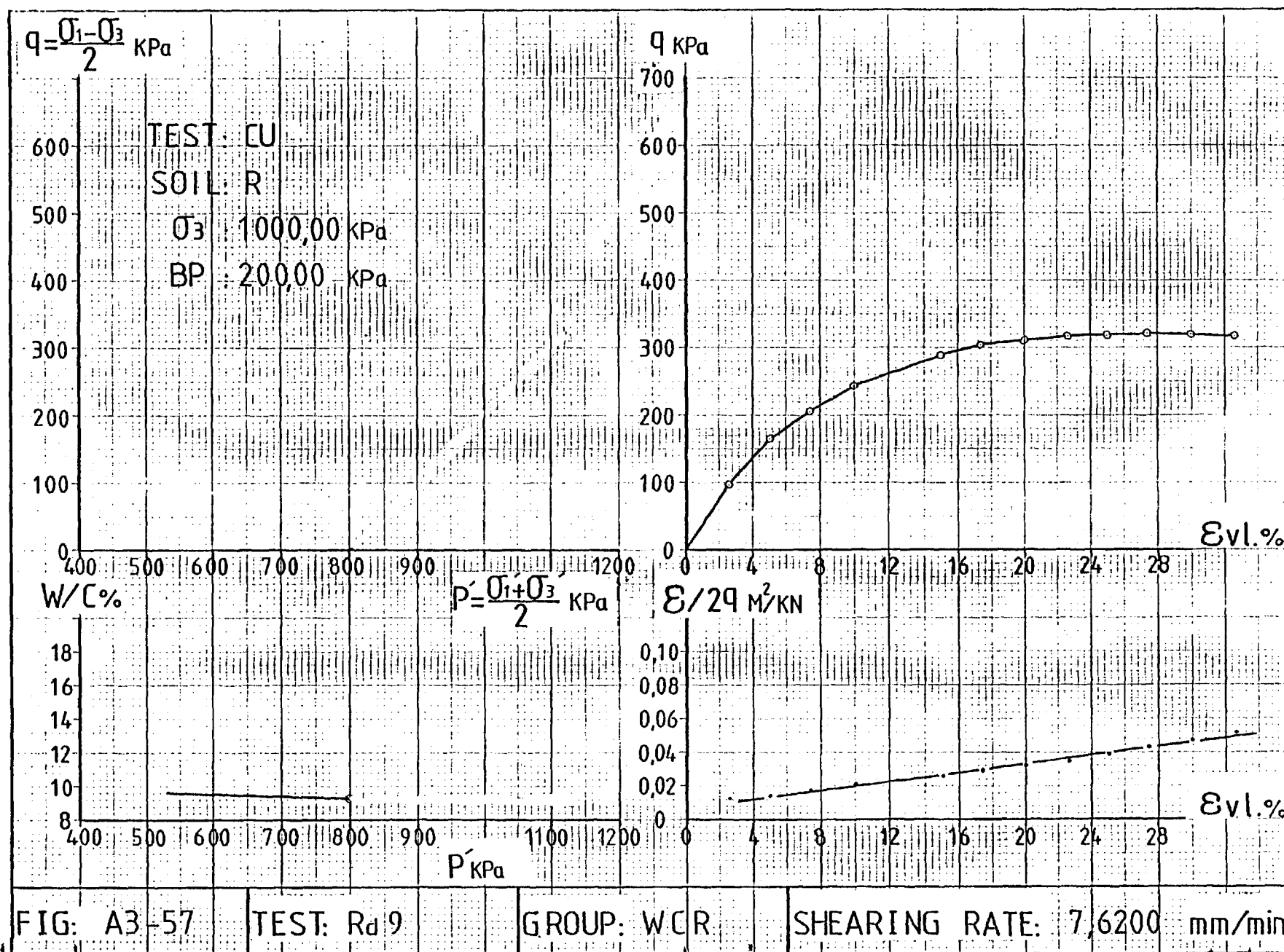












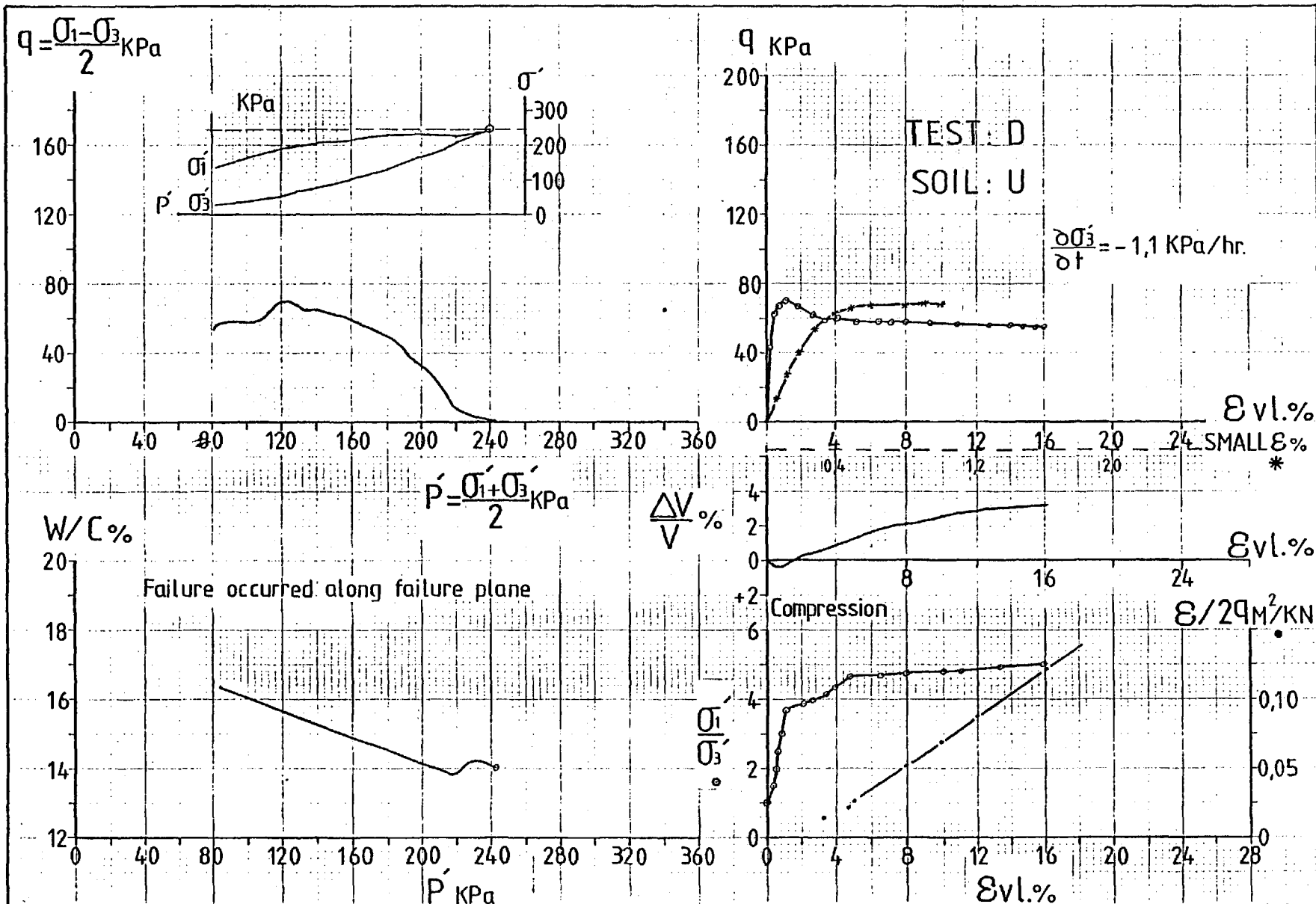
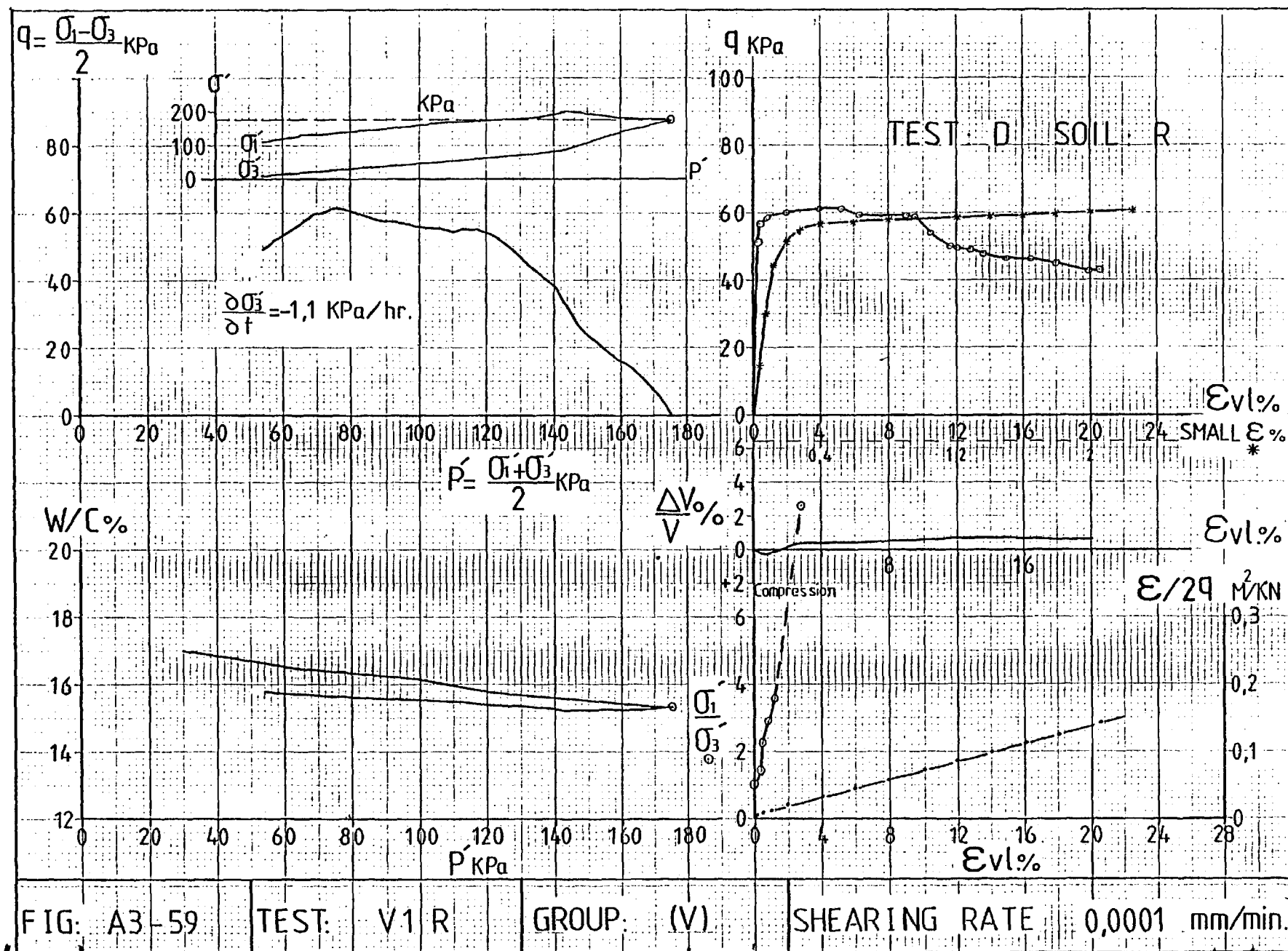
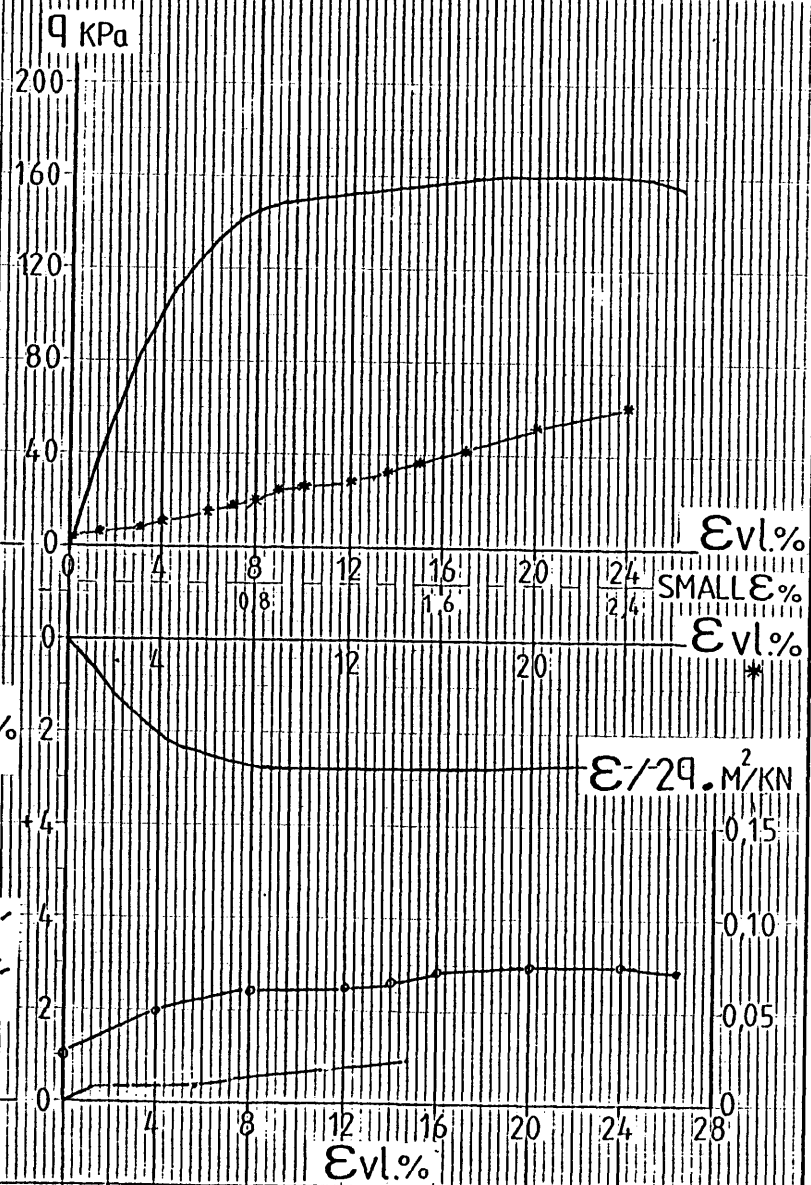
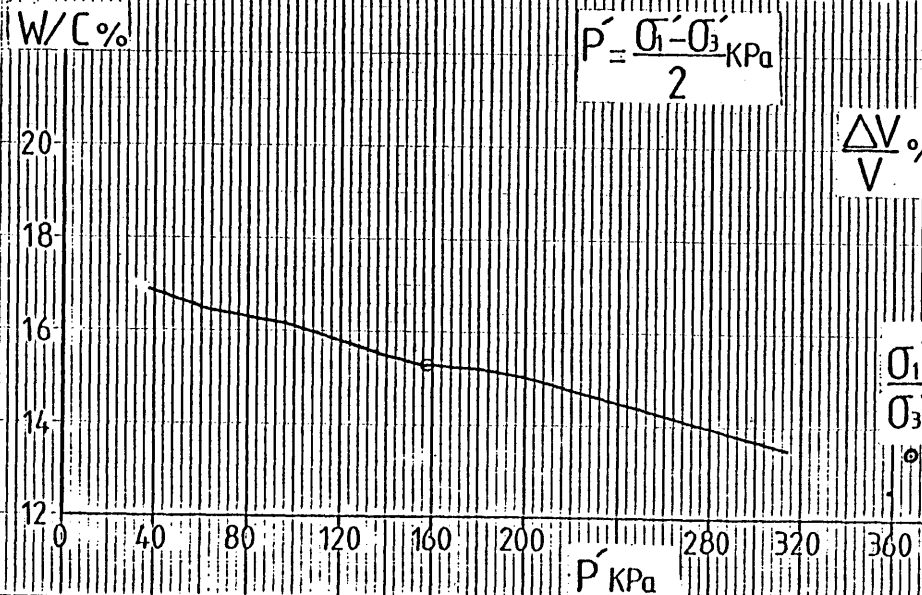
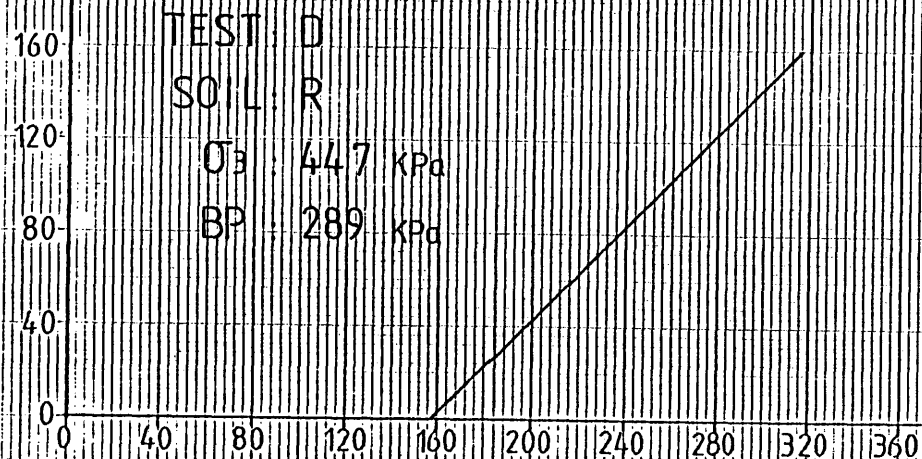


FIG: A3-58 TEST: V1U GROUP: (V) SHEARING RATE: 0,0001 mm/min.



$$q = \frac{\sigma_1 - \sigma_3}{2} \text{ KPa}$$

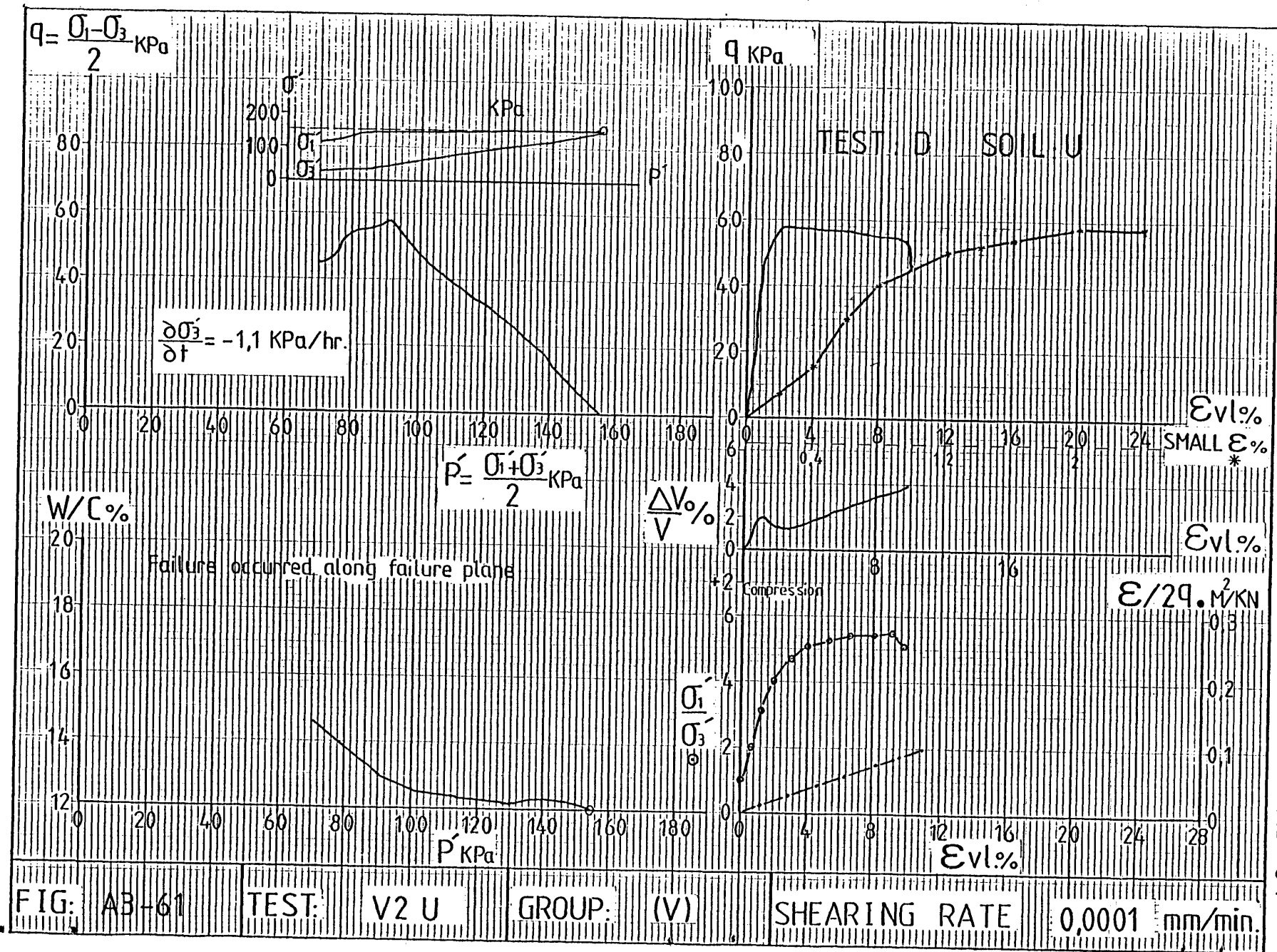


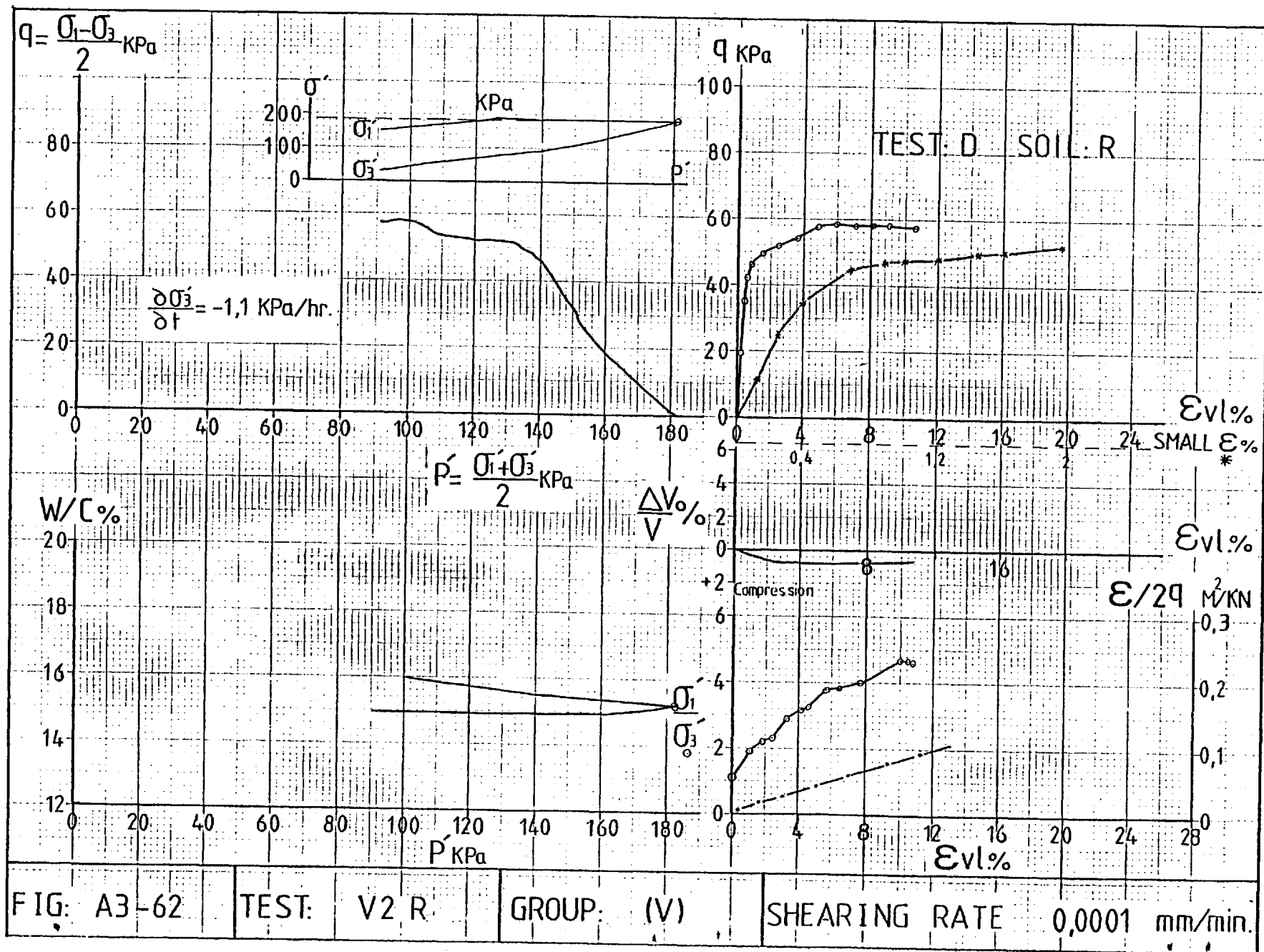
A3-60

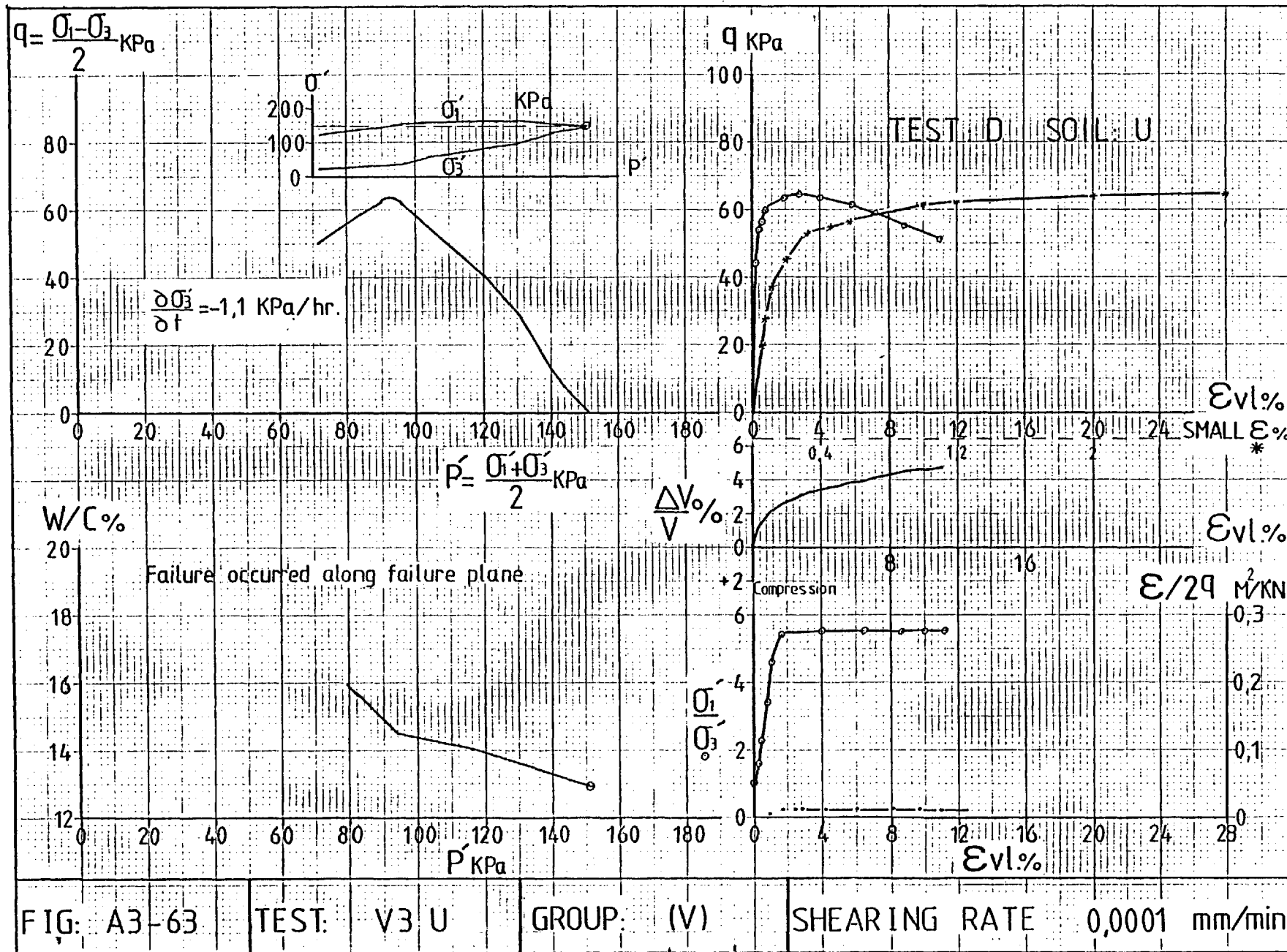
FIG: A3-60

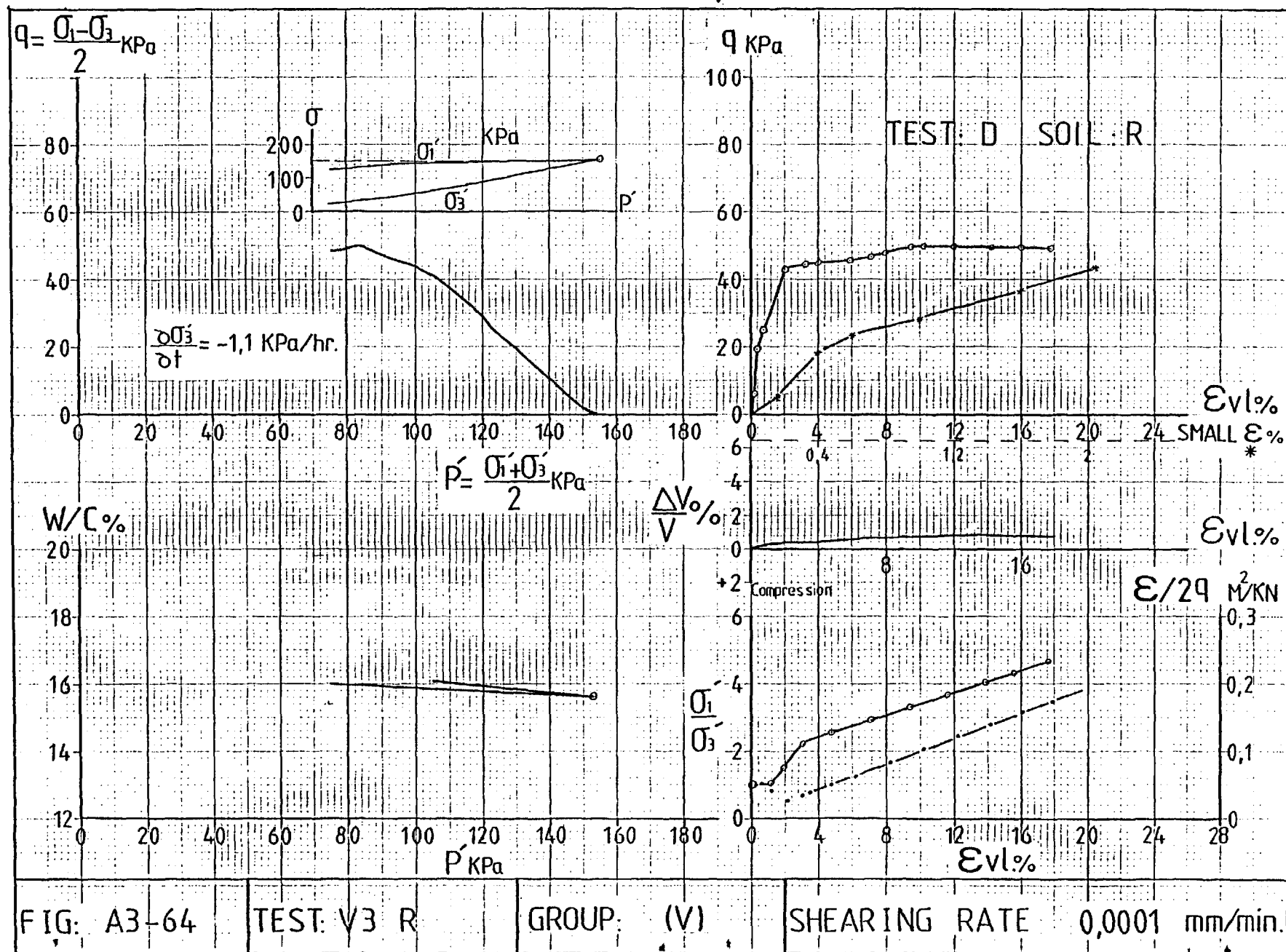
TEST: V1 R' (T28) GROUP: (V)

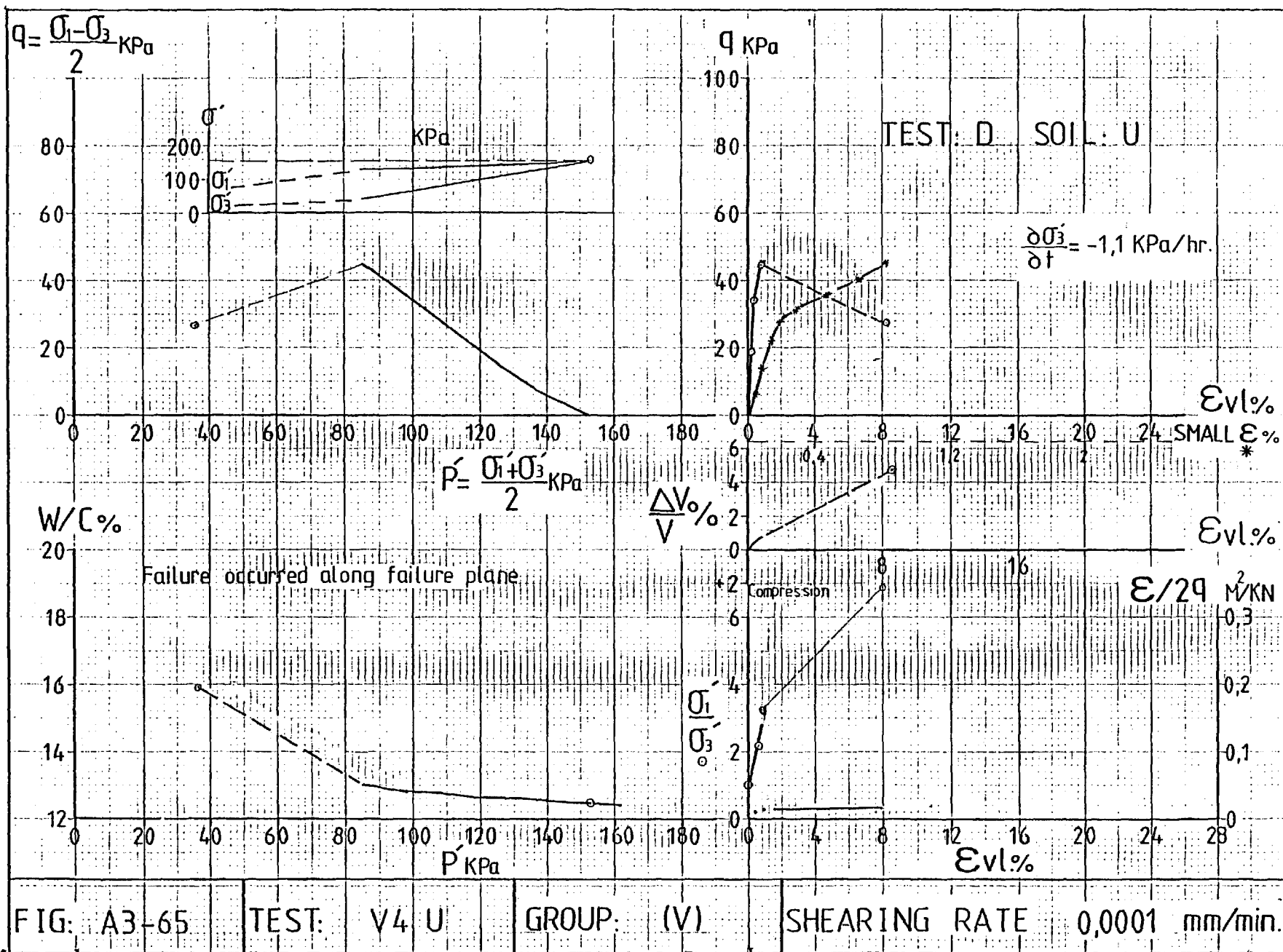
SHEARING RATE: 0.0001 mm/min.

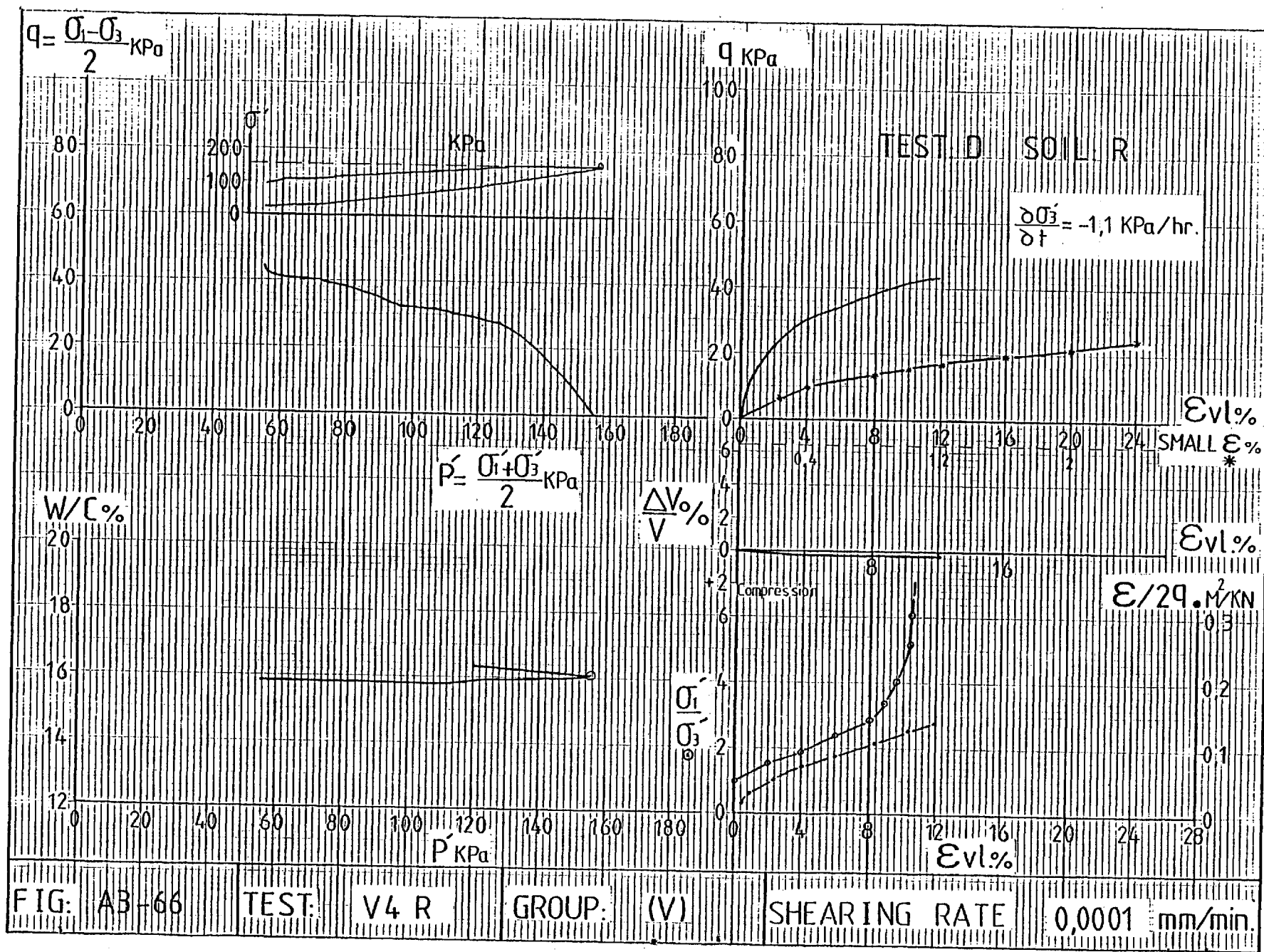












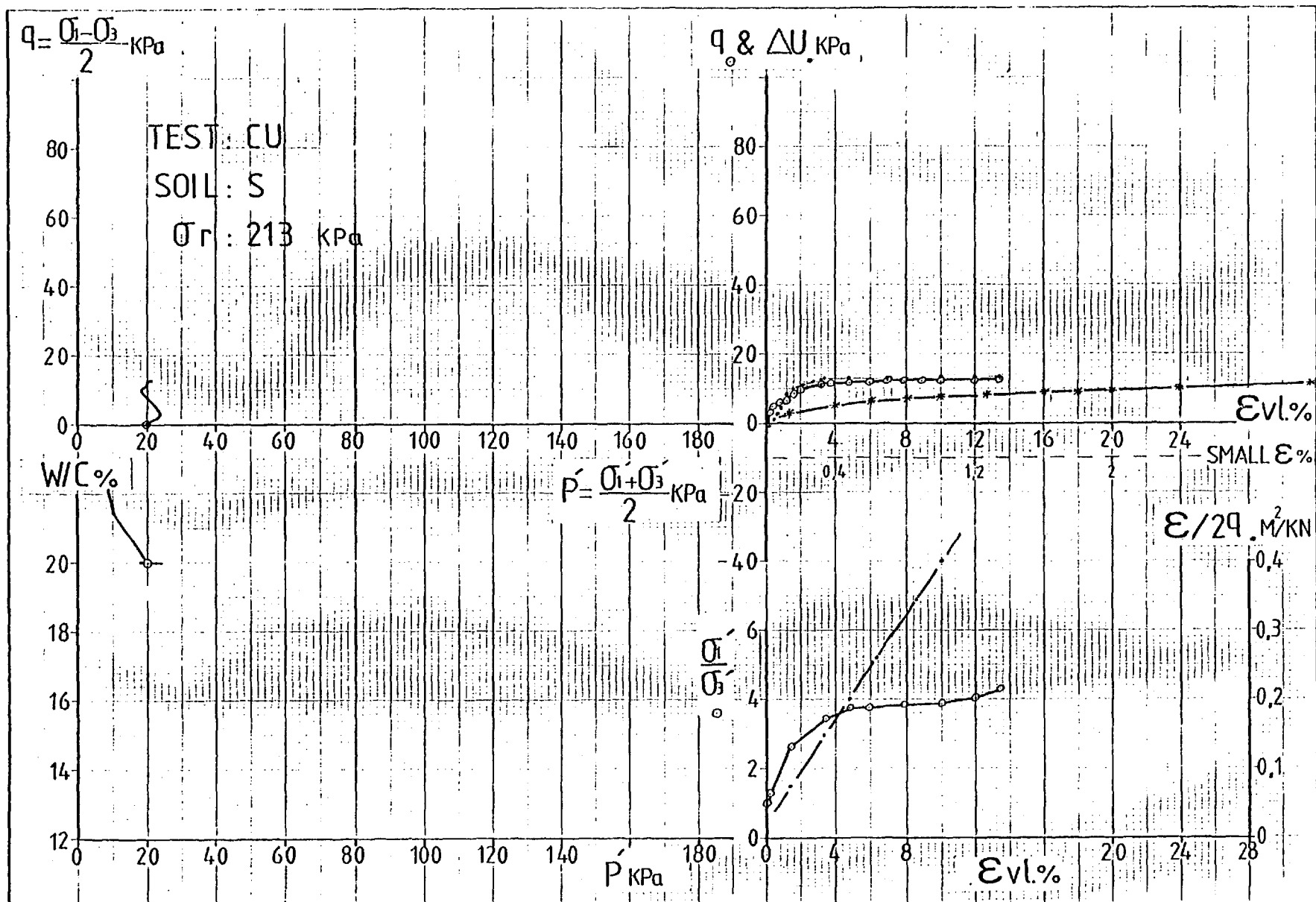


FIG: A3-67 TEST: INP20 GROUP: IN SHEARING RATE 0,0050 mm/min.

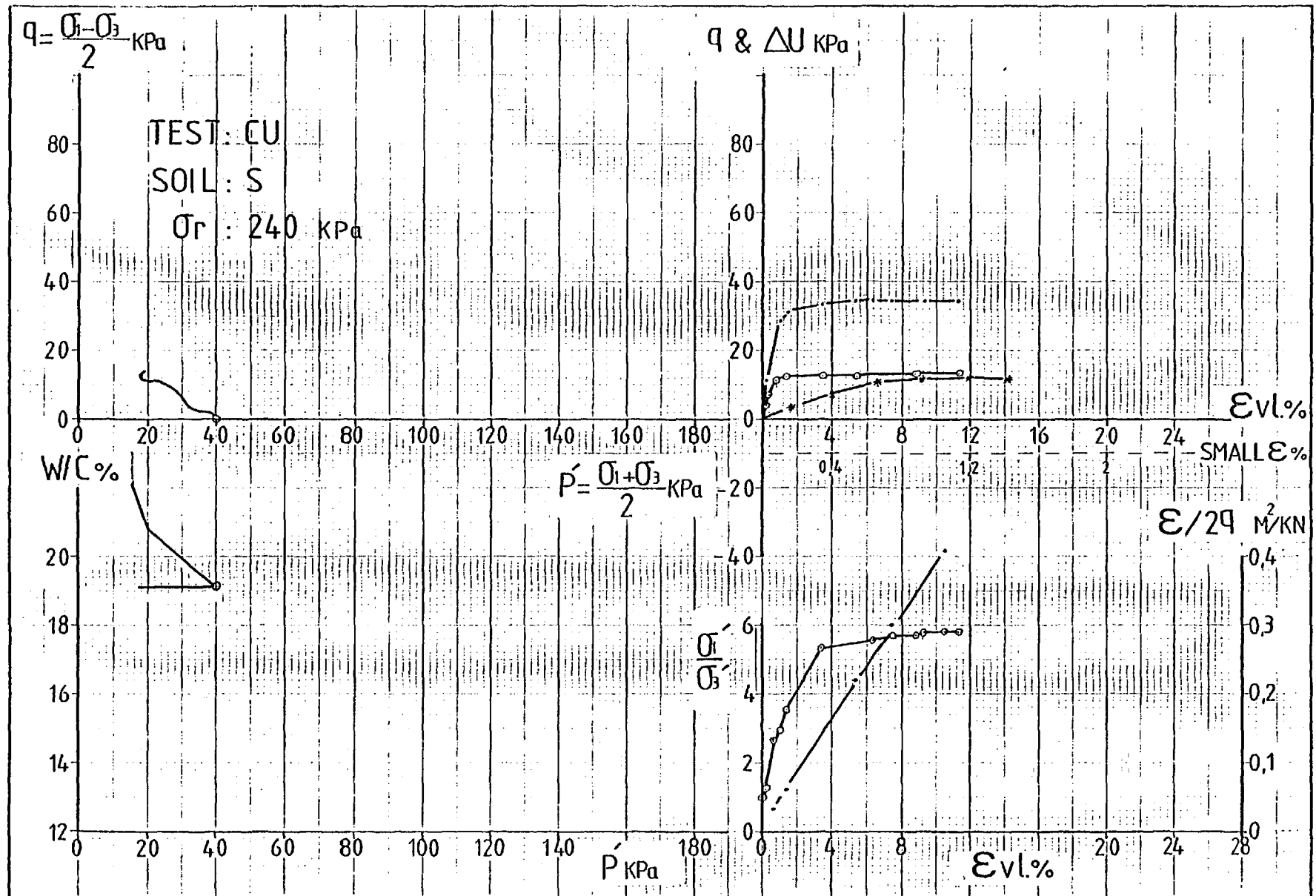


FIG. A3-68

TEST: INP 40

GROUP: IN

SHEARING RATE: 0,0050 mm/min.

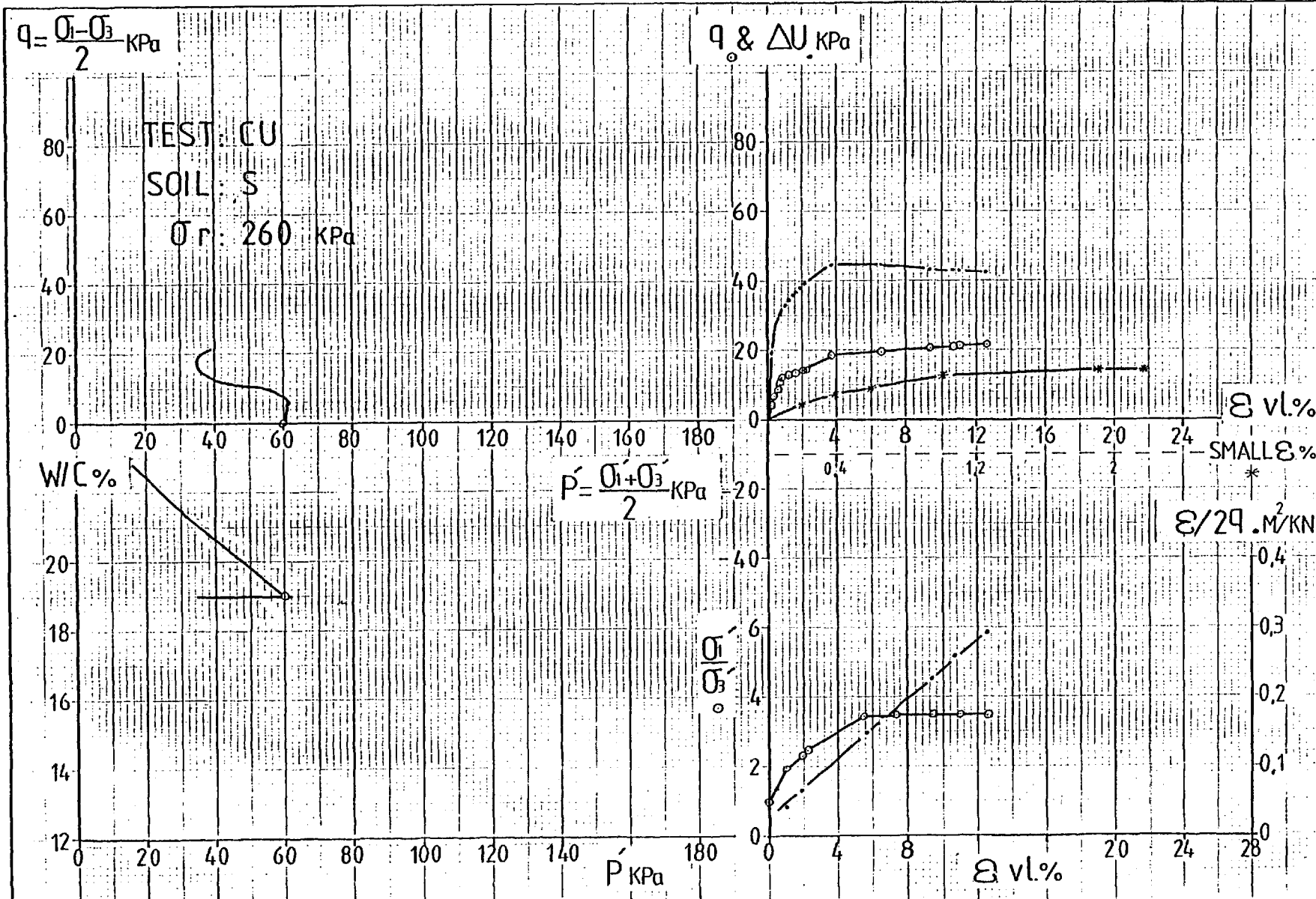
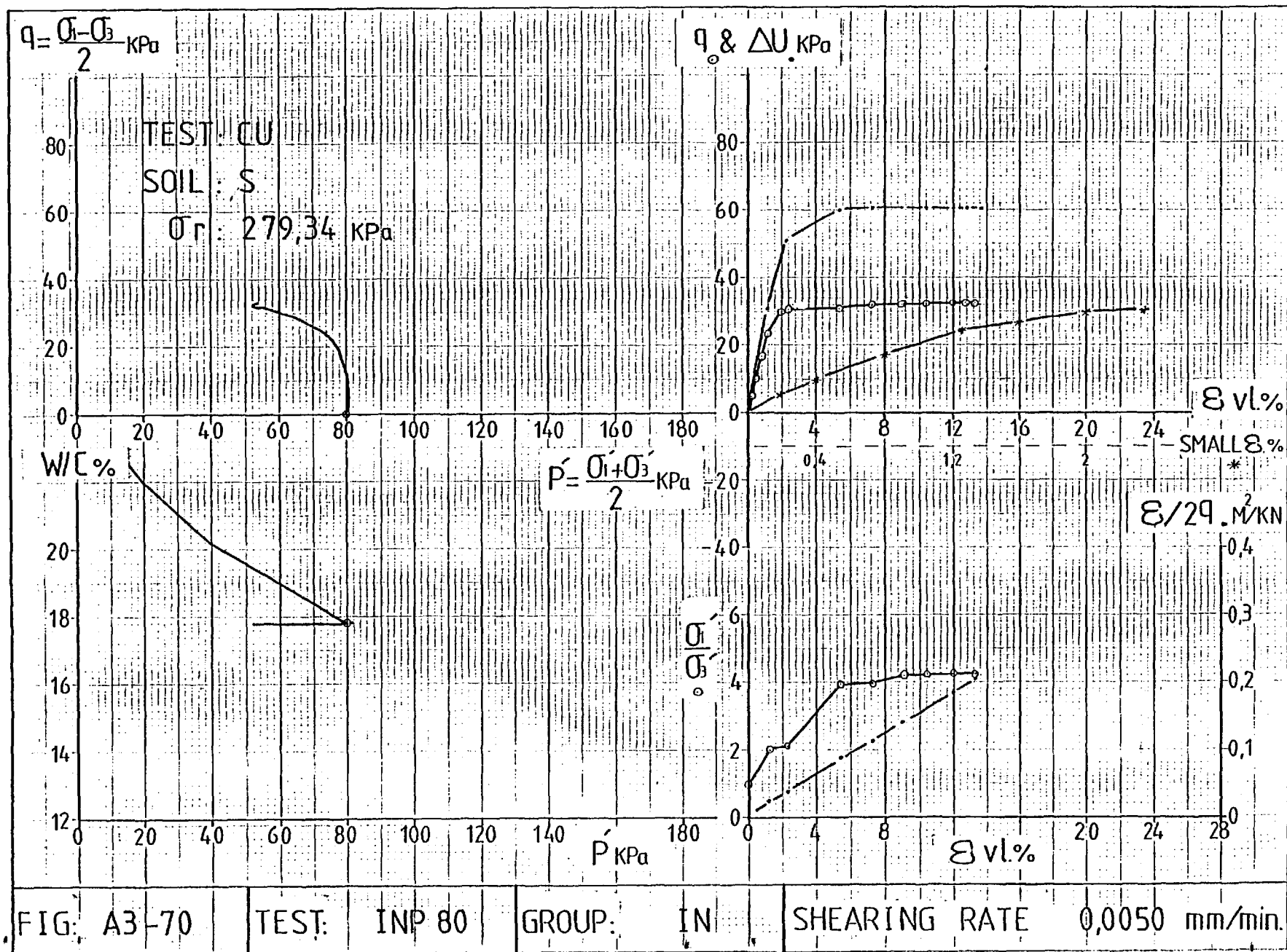


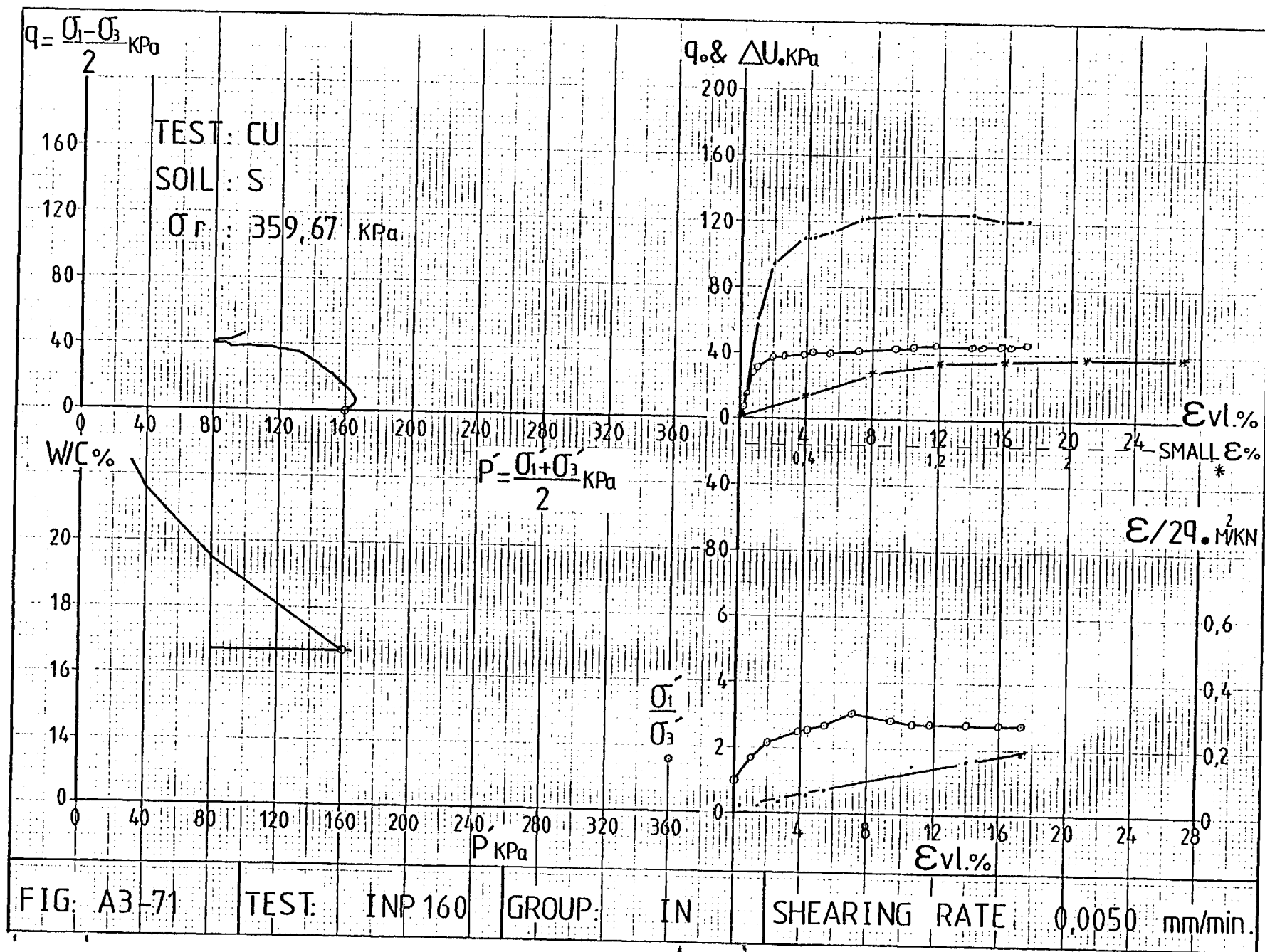
FIG: A3-69

TEST: INP 60

GROUP: IN

SHEARING RATE 0,0050 mm/min.





$$q = \frac{\sigma_1 - \sigma_3}{2} \text{ KPa}$$

160

TEST: CU

SOIL: S

$\sigma_r = 400 \text{ KPa}$

120

80

40

0

W/C %

20

18

16

14

0

0

40

80

120

160

200

240

280

320

360

$$p' = \frac{\sigma_1 + \sigma_3}{2} \text{ KPa}$$

P KPa

0

40

80

120

160

200

240

280

320

360

q & ΔU KPa

200

160

120

80

40

0

-40

-80

6

4

2

0

0

4

8

12

16

20

24

28

32

36

40

44

48

52

56

60

64

68

72

76

80

84

88

92

96

100

104

108

112

116

120

124

128

132

136

140

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148

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728

732

736

740

744

748

752

756

760

764

768

772

776

780

784

788

792

796

800

804

808

812

816

820

824

828

832

836

840

844

848

852

856

860

864

868

872

876

880

884

888

892

896

900

904

908

912

916

920

924

928

932

936

940

944

948

952

956

960

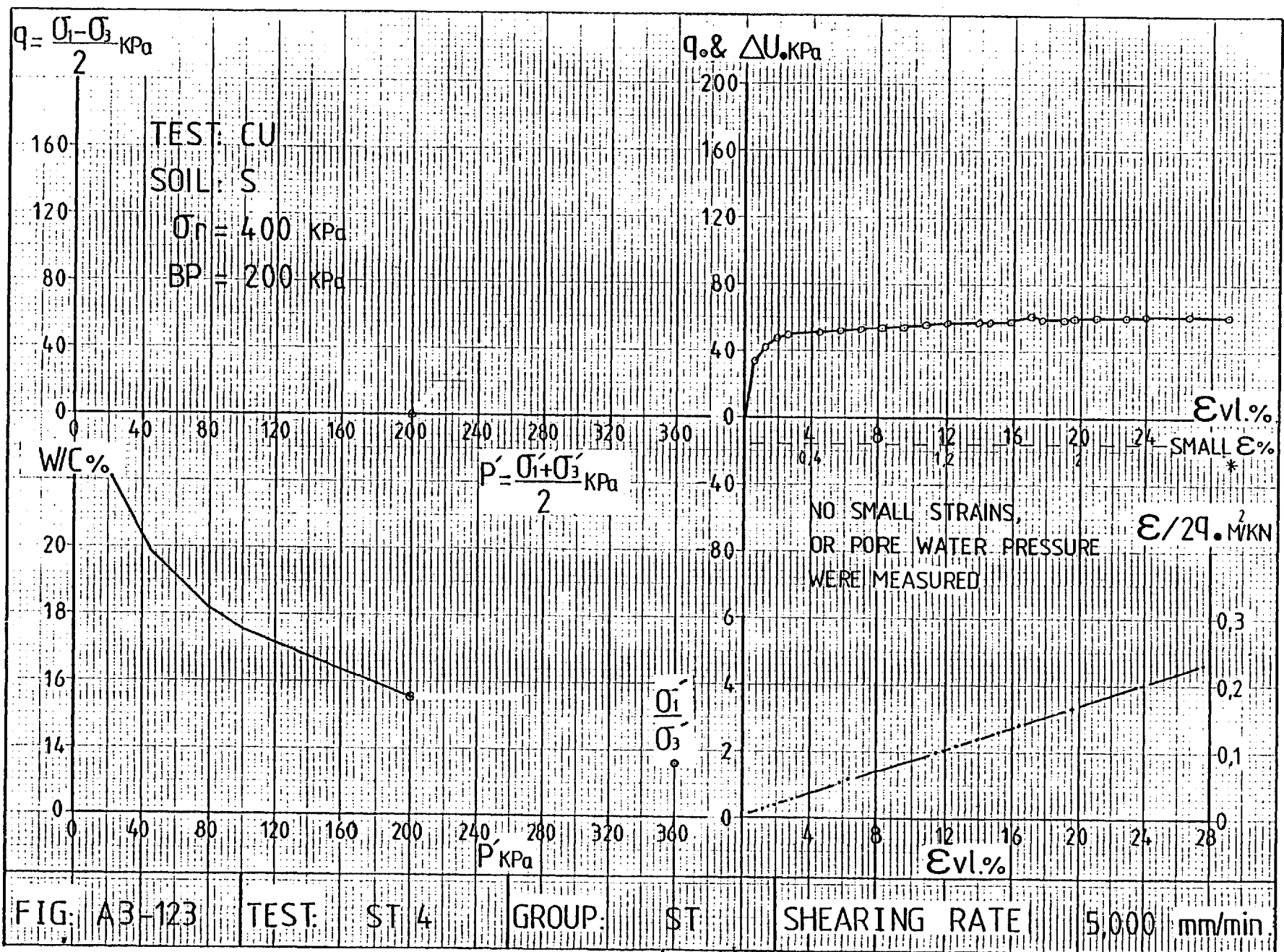
964

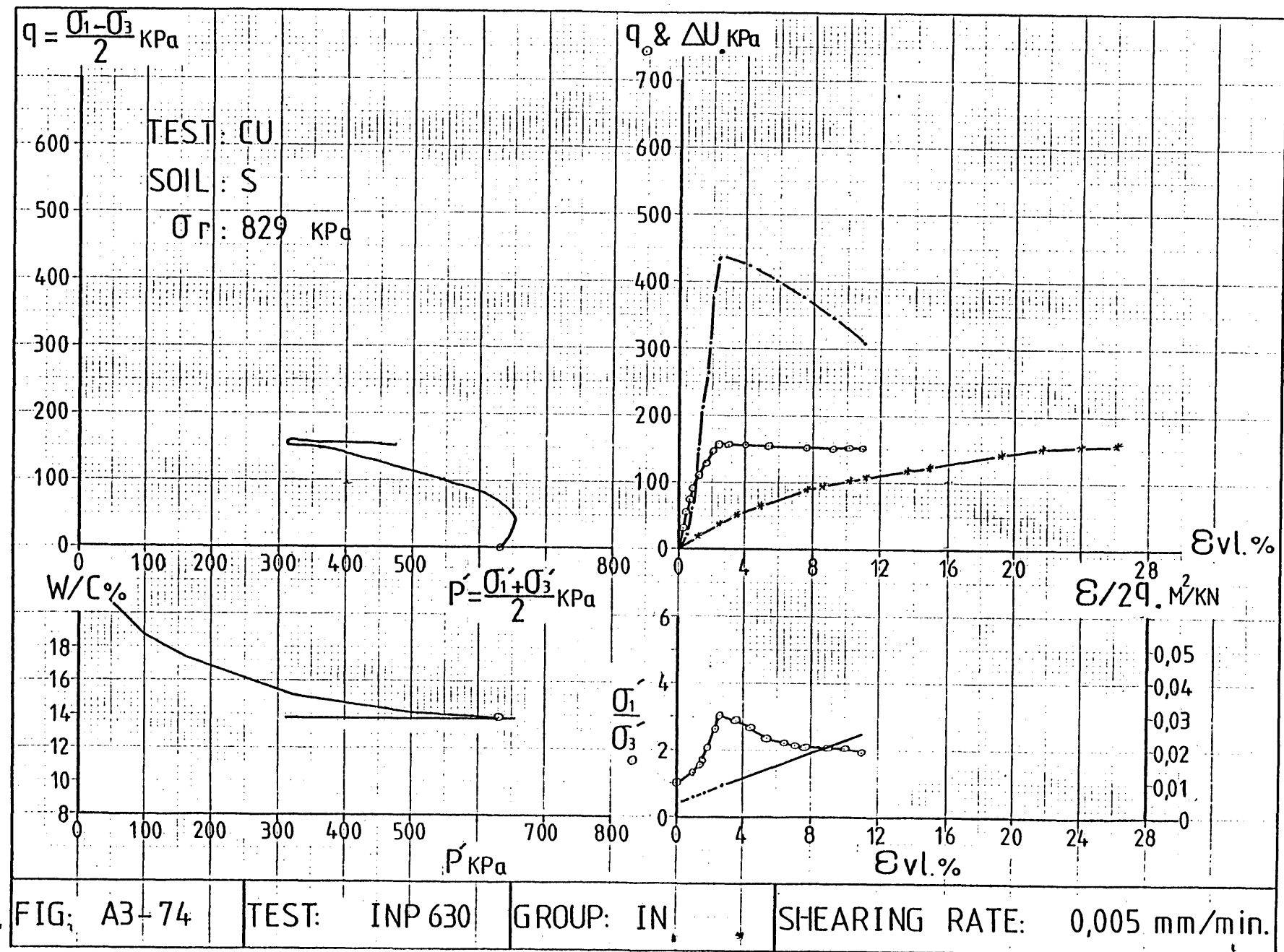
968

972

976

980





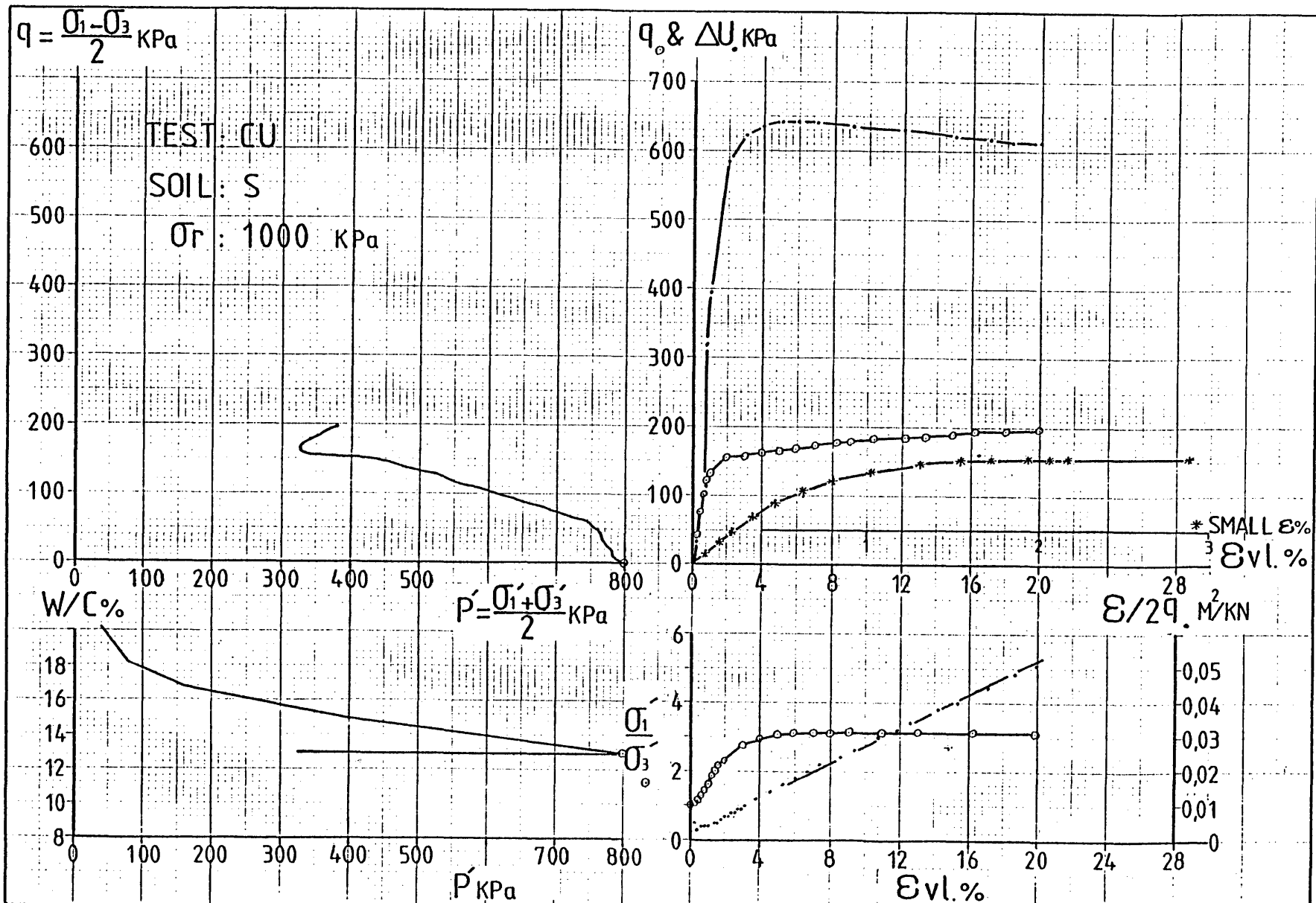
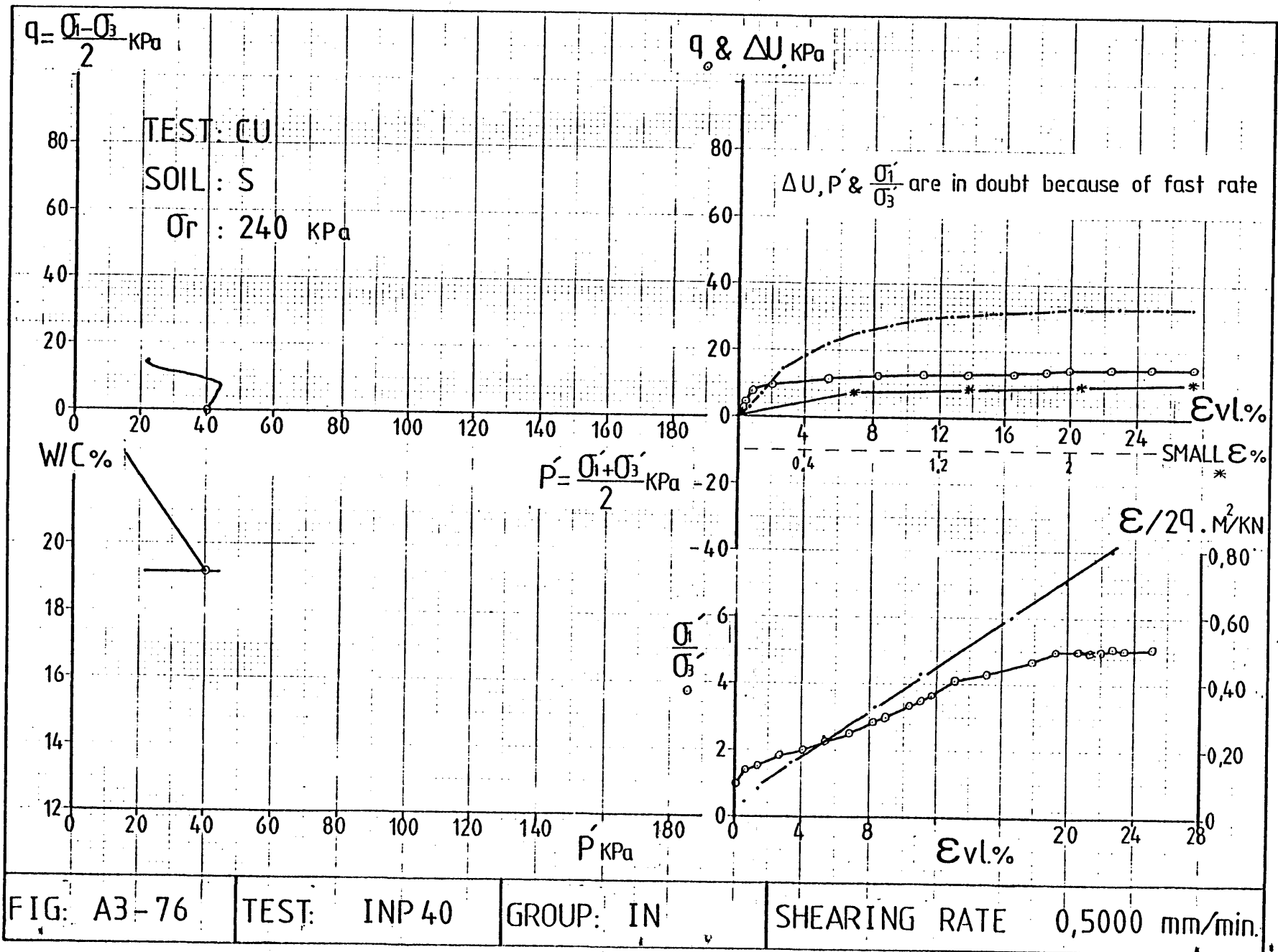


FIG: A3-75

TEST: INP 800

GROUP: IN

SHEARING RATE: 0,005 mm/min.



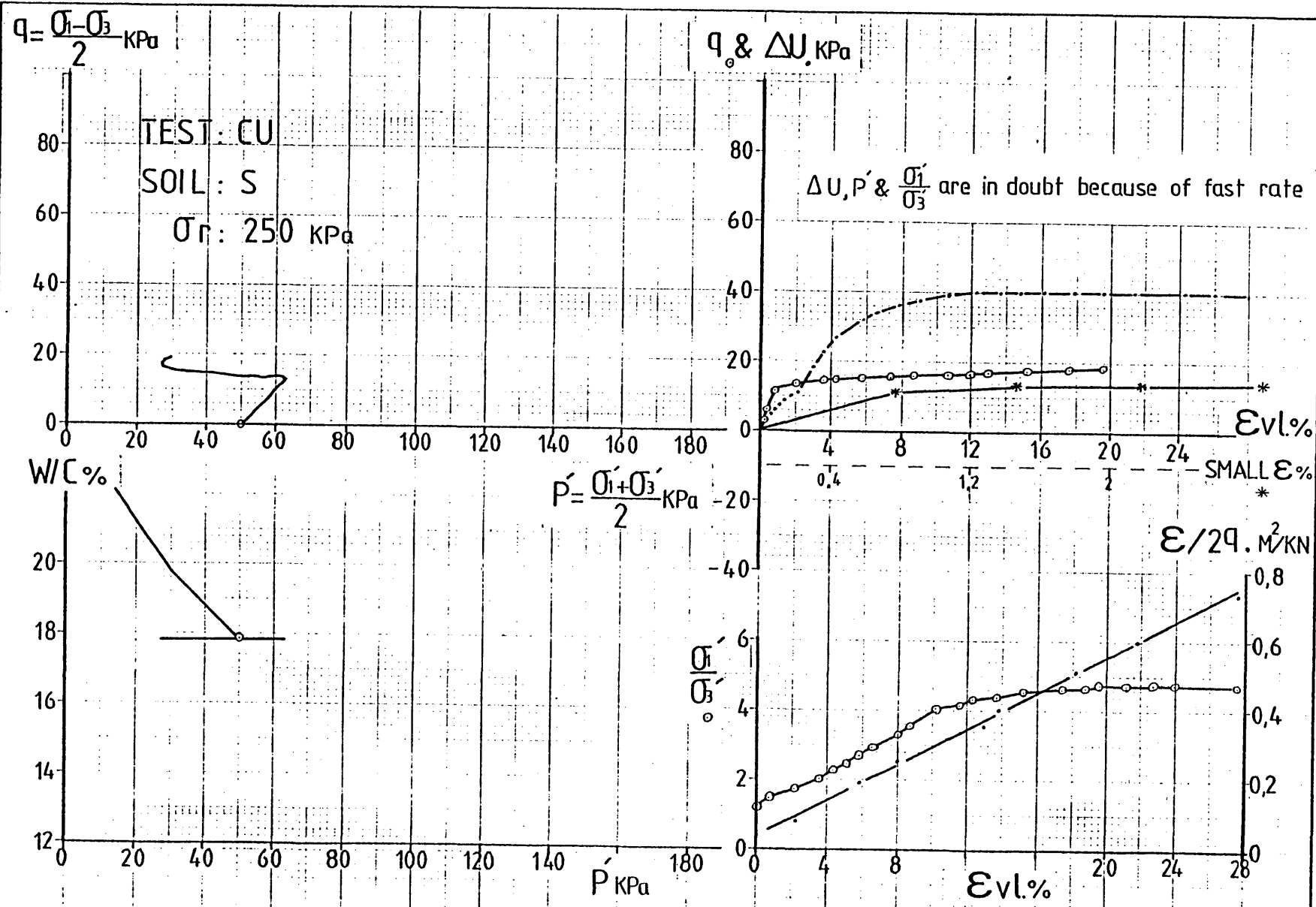
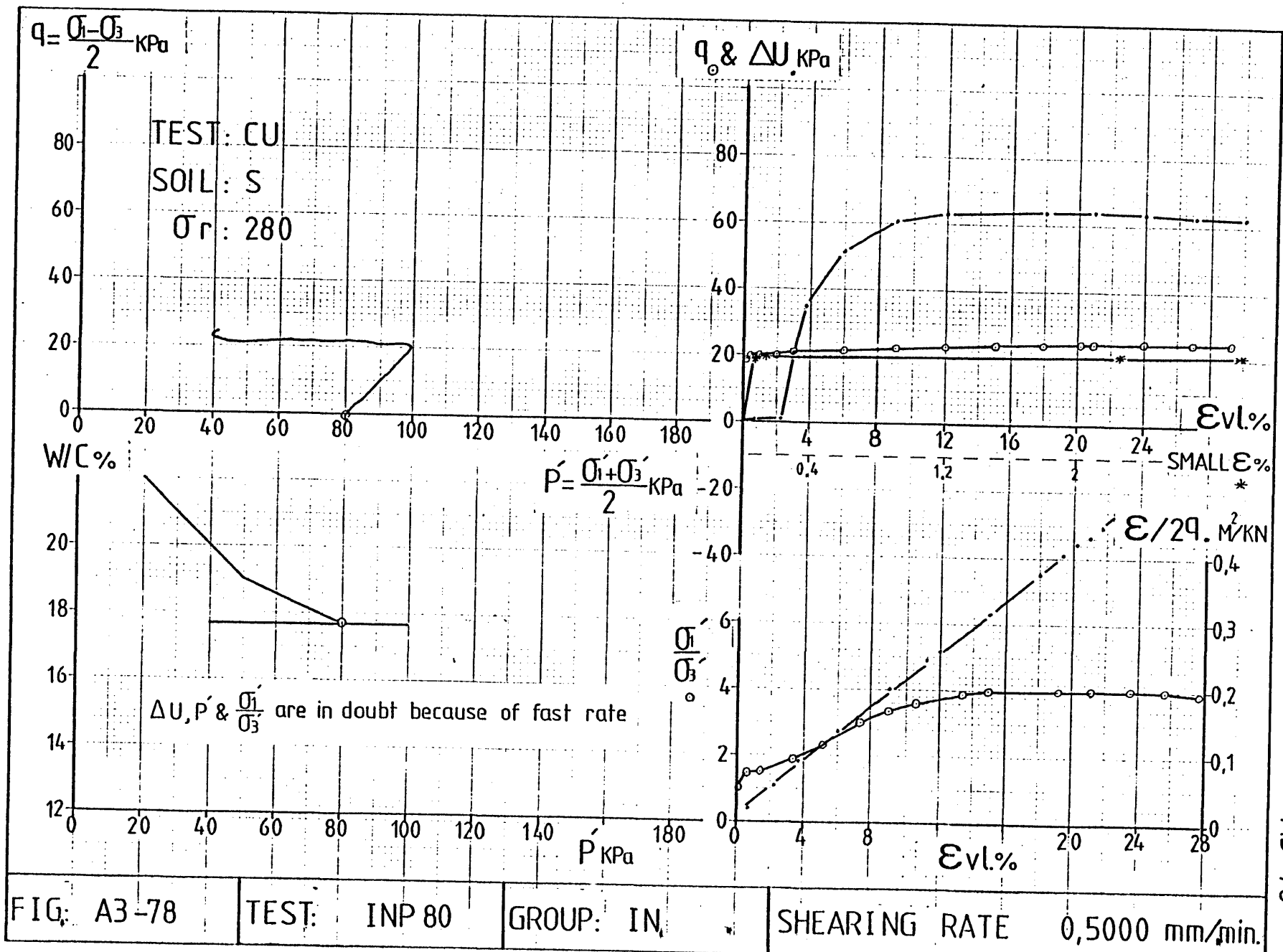


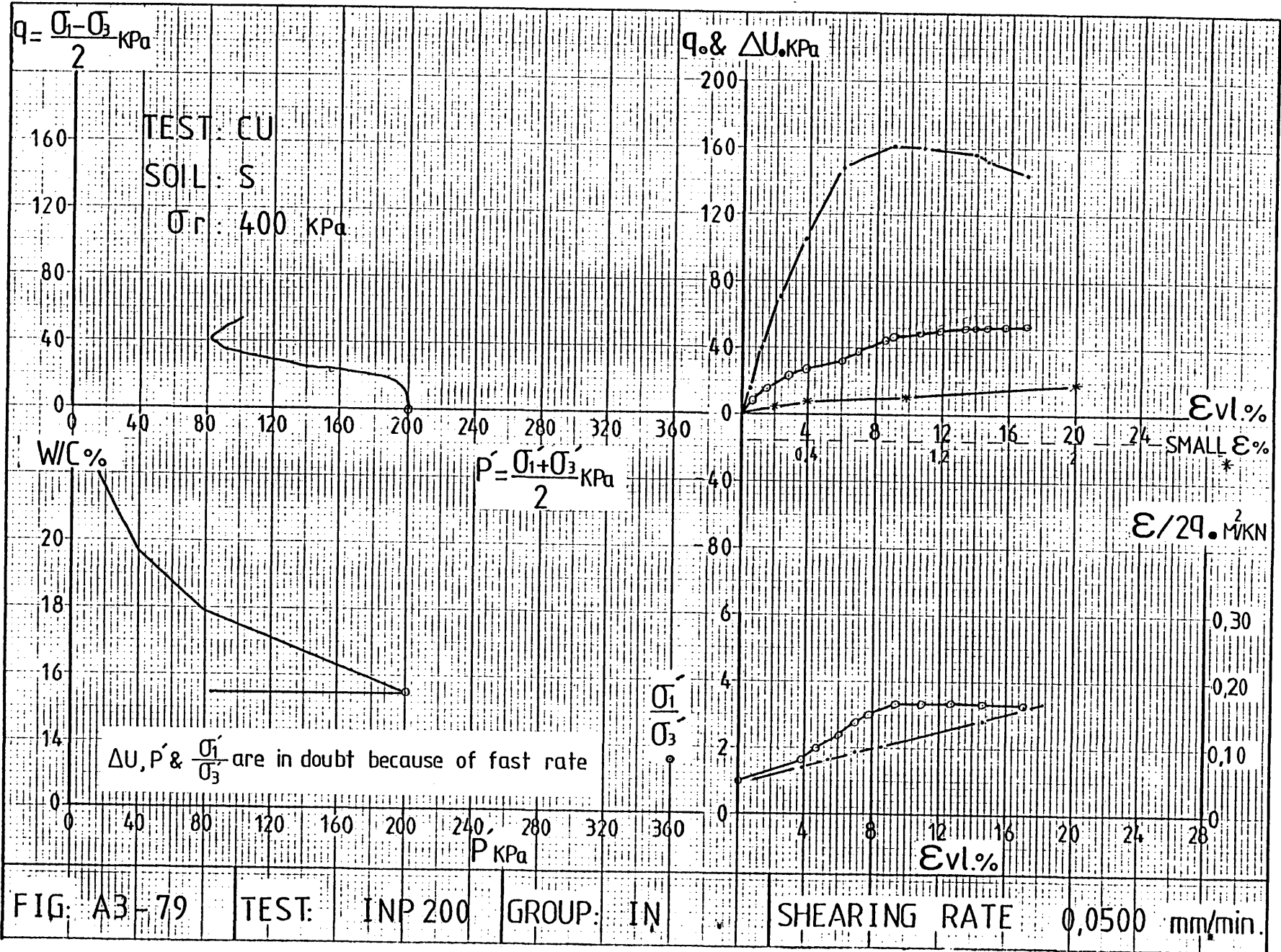
FIG: A3-77

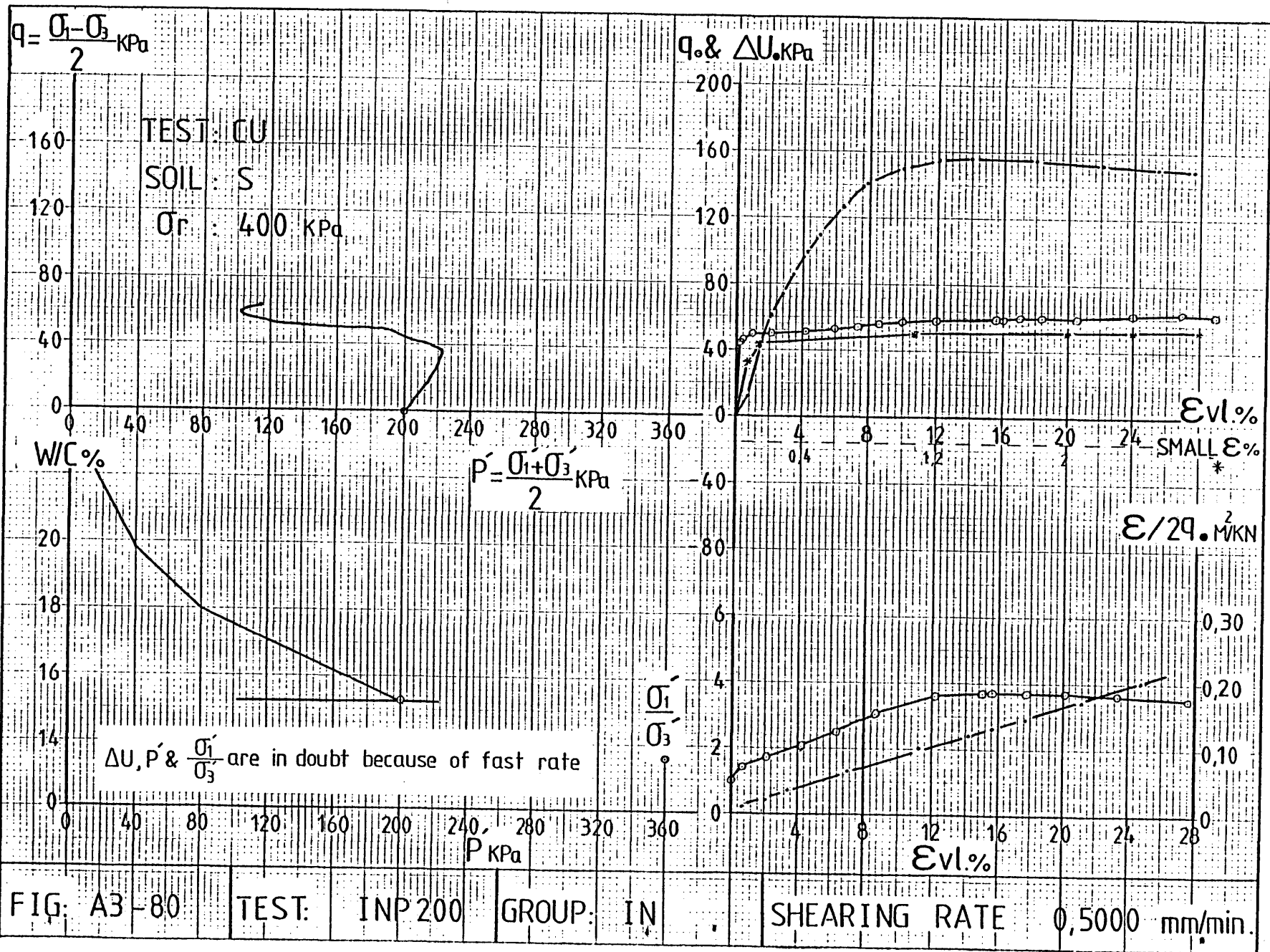
TEST: INP 50

GROUP: IN

SHEARING RATE 0,5000 mm/min.







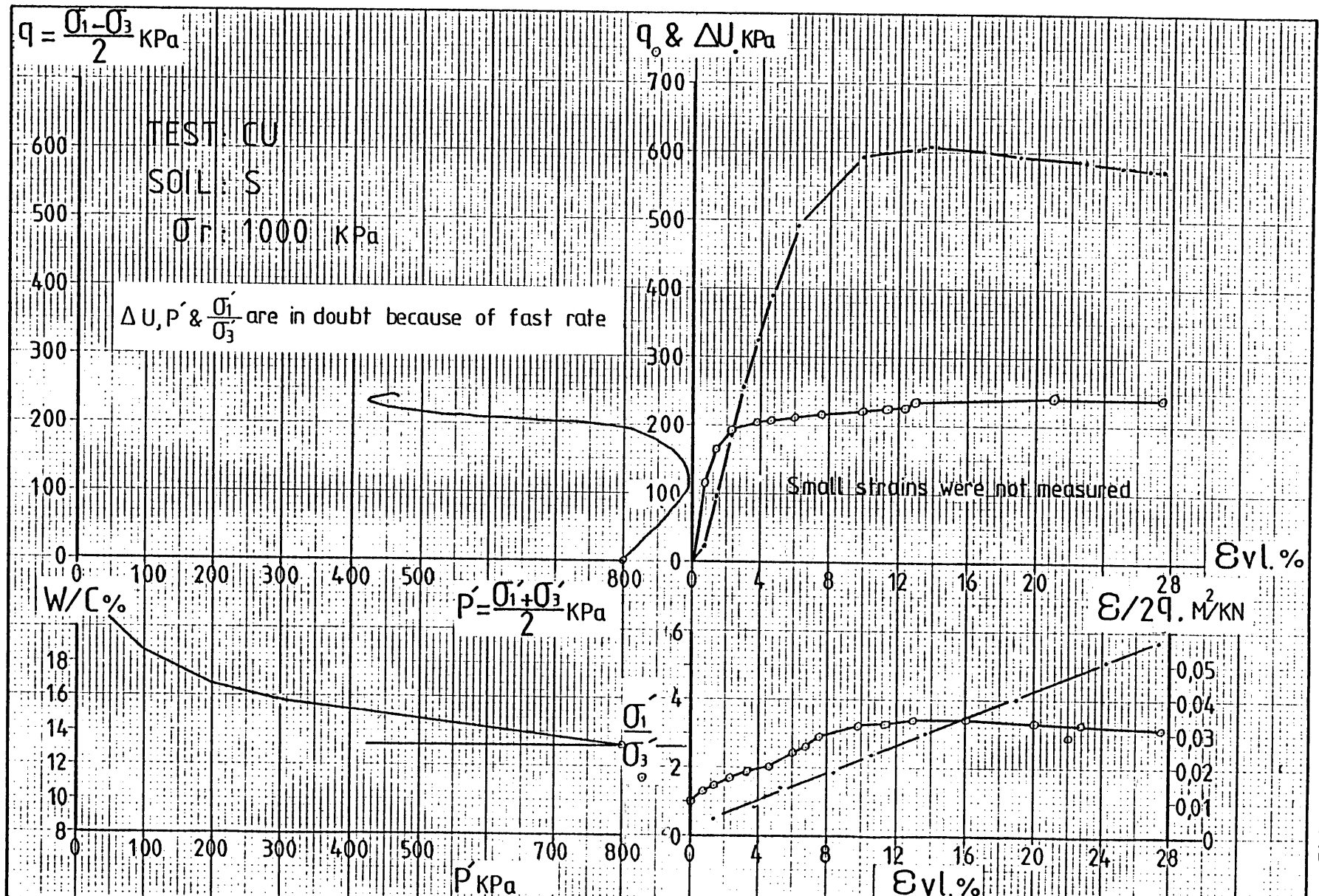


FIG: A3-81 TEST: INP 800 GROUP: IN SHEARING RATE: 0,500 mm/min.

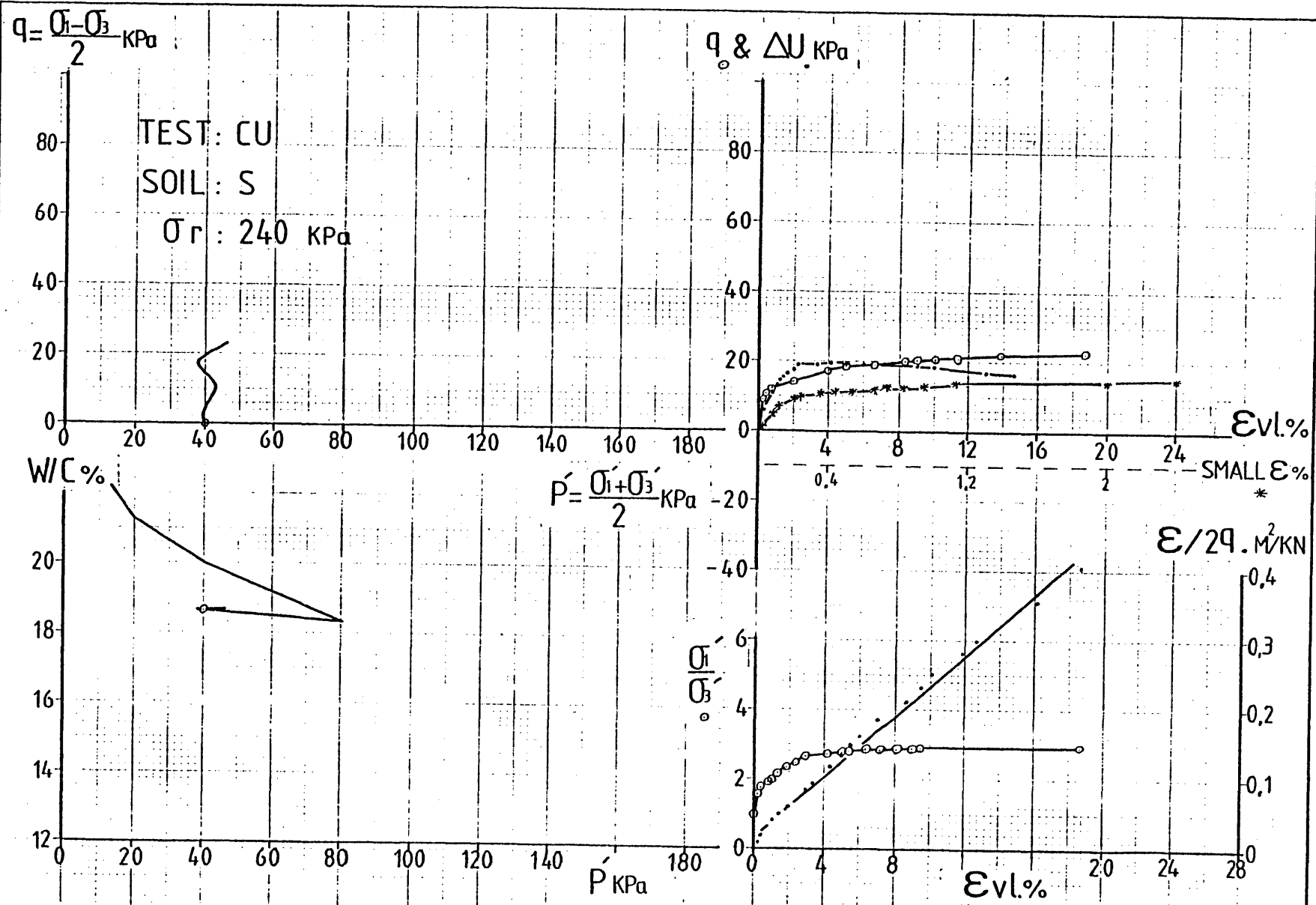
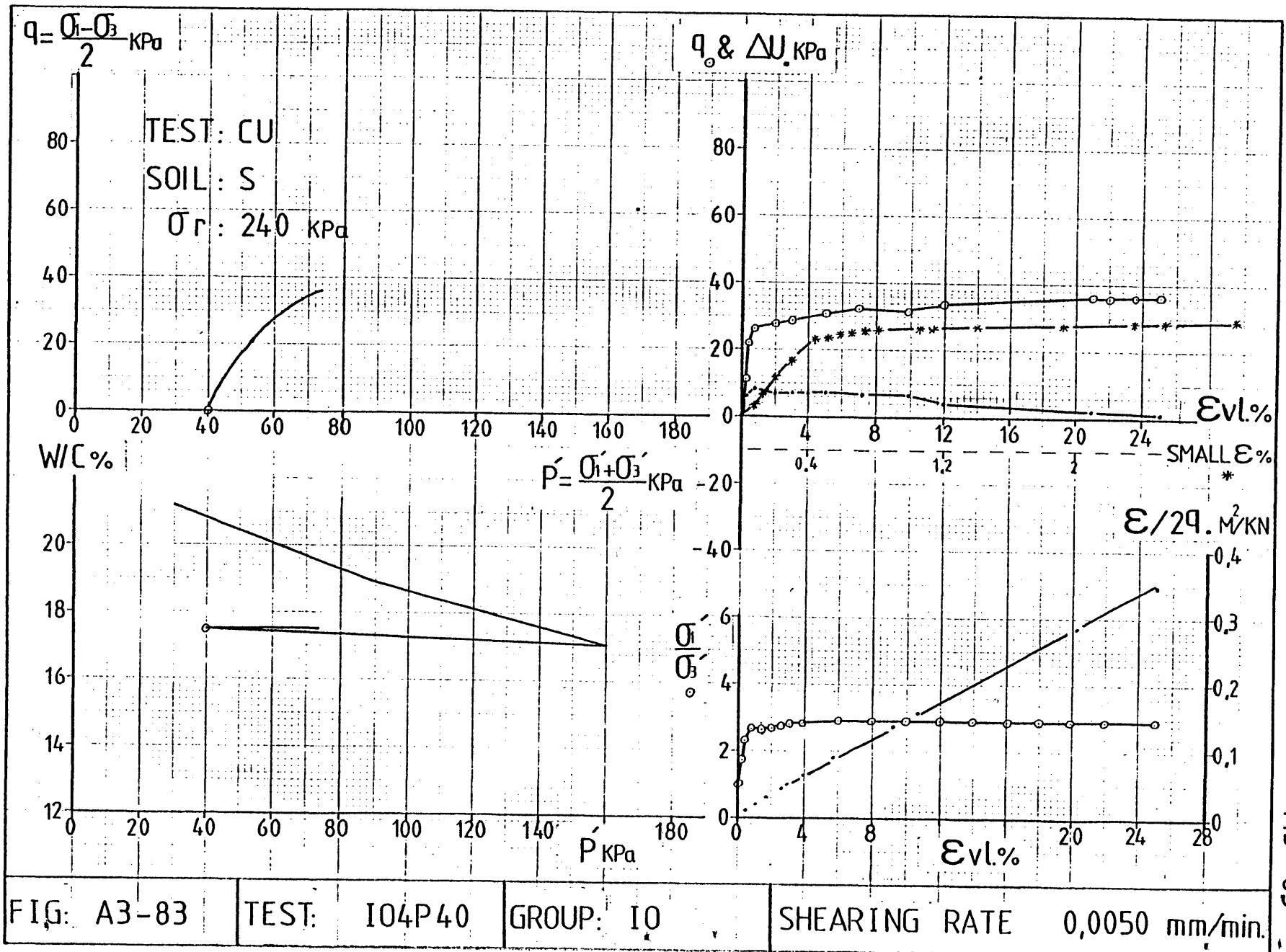


FIG: A3-82

TEST: IO2P 40

GROUP: IO

SHEARING RATE 0,0050 mm/min.



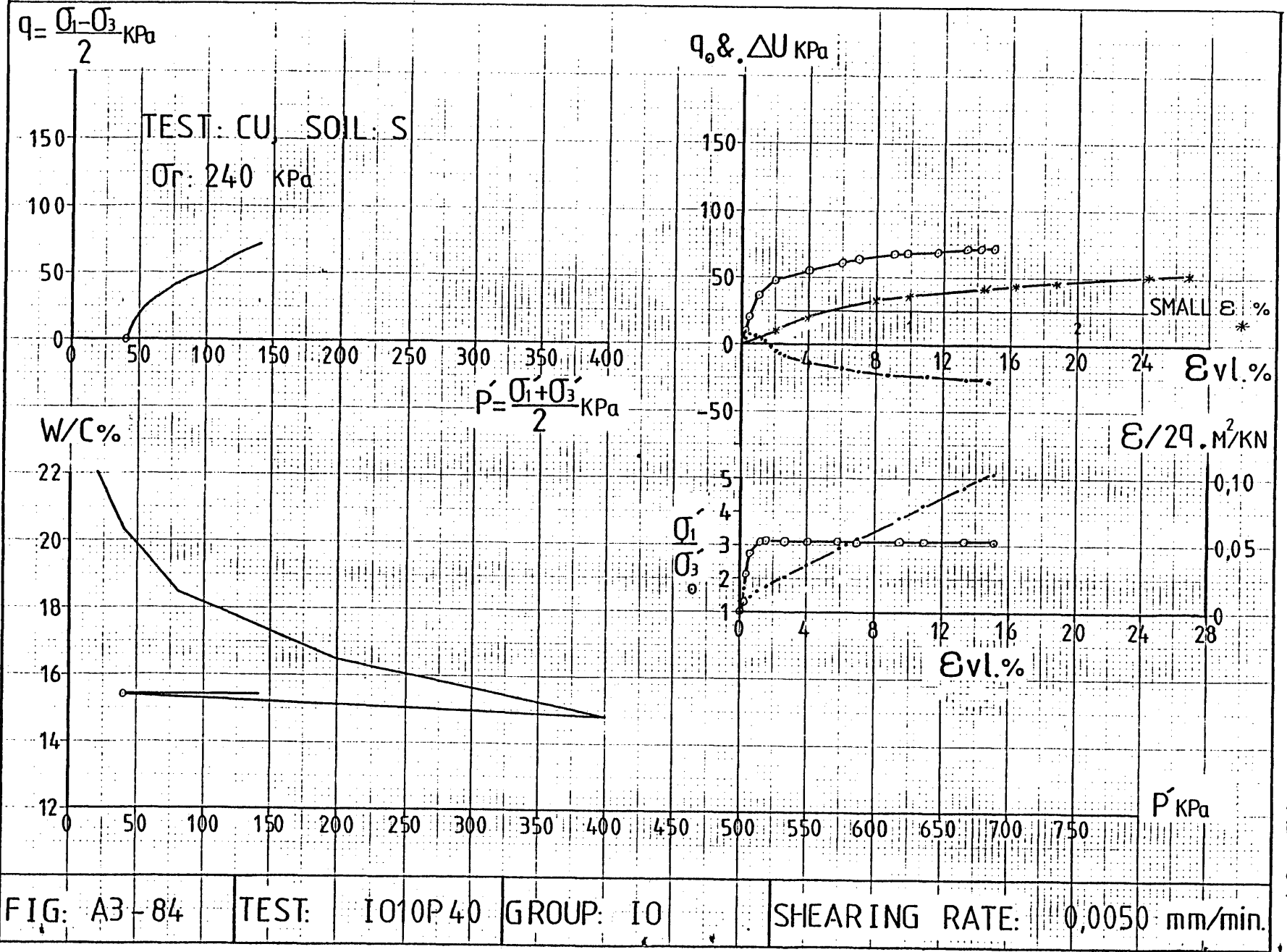


FIG: A3-84

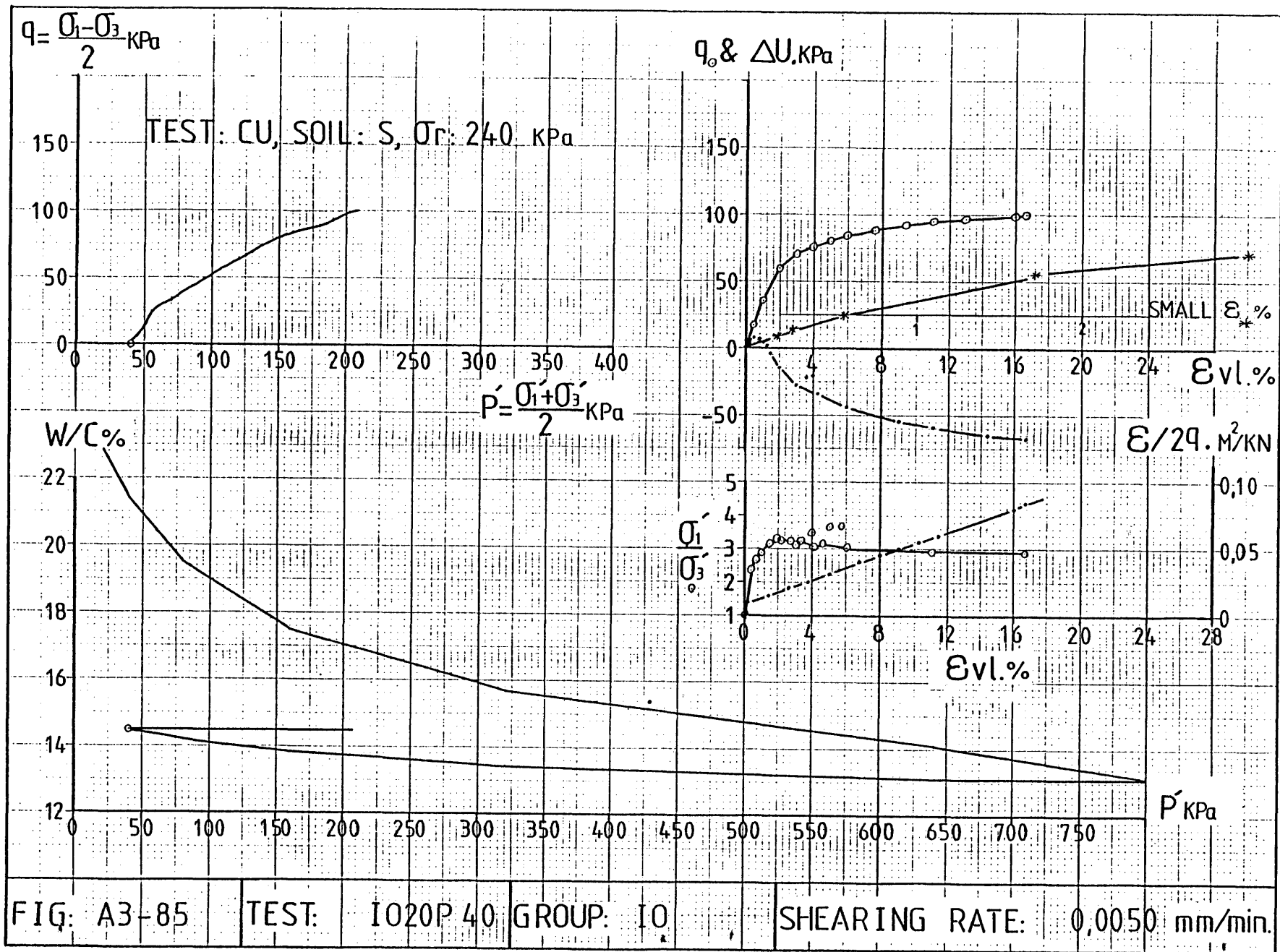
TEST:

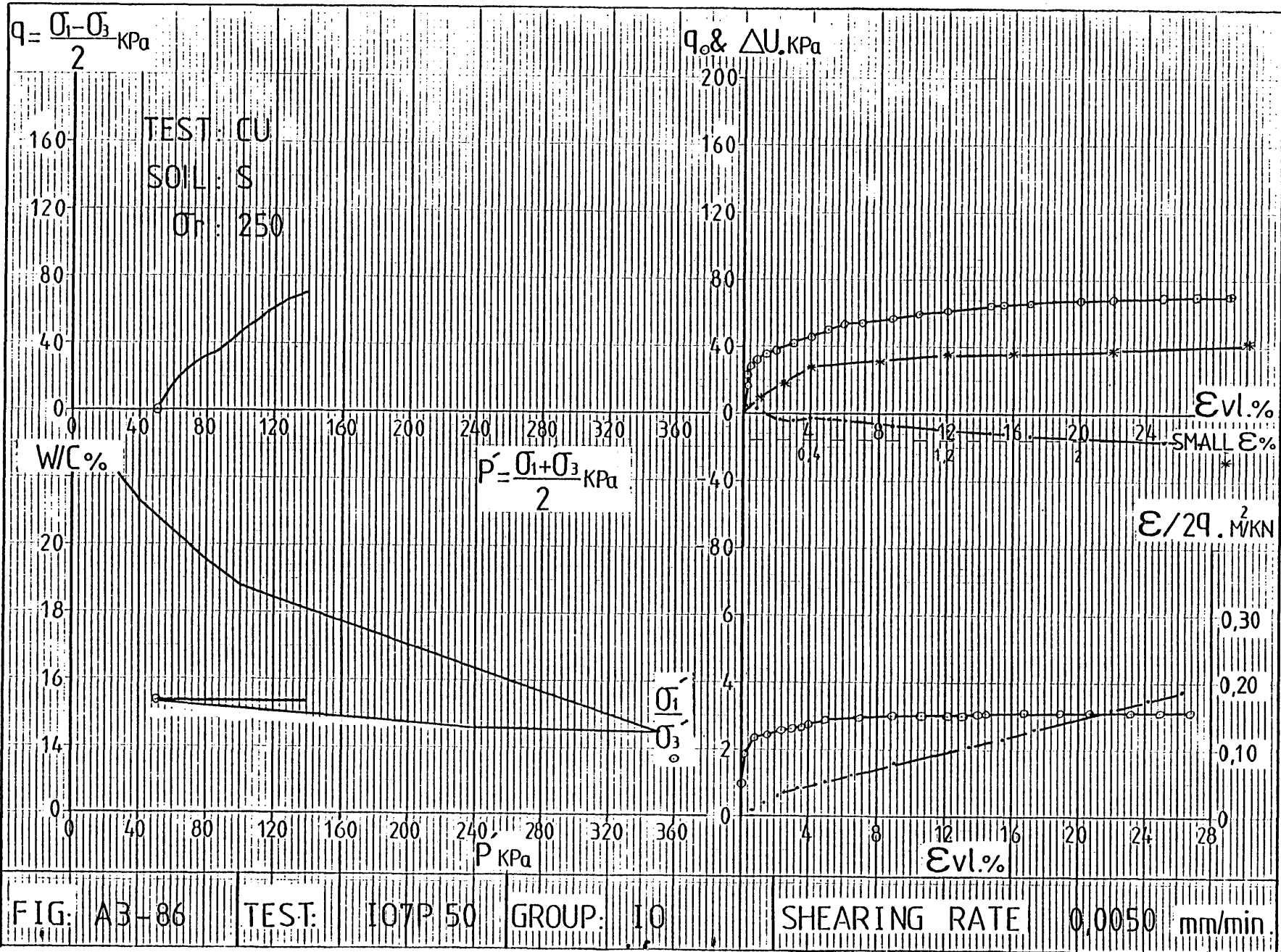
IO10P40

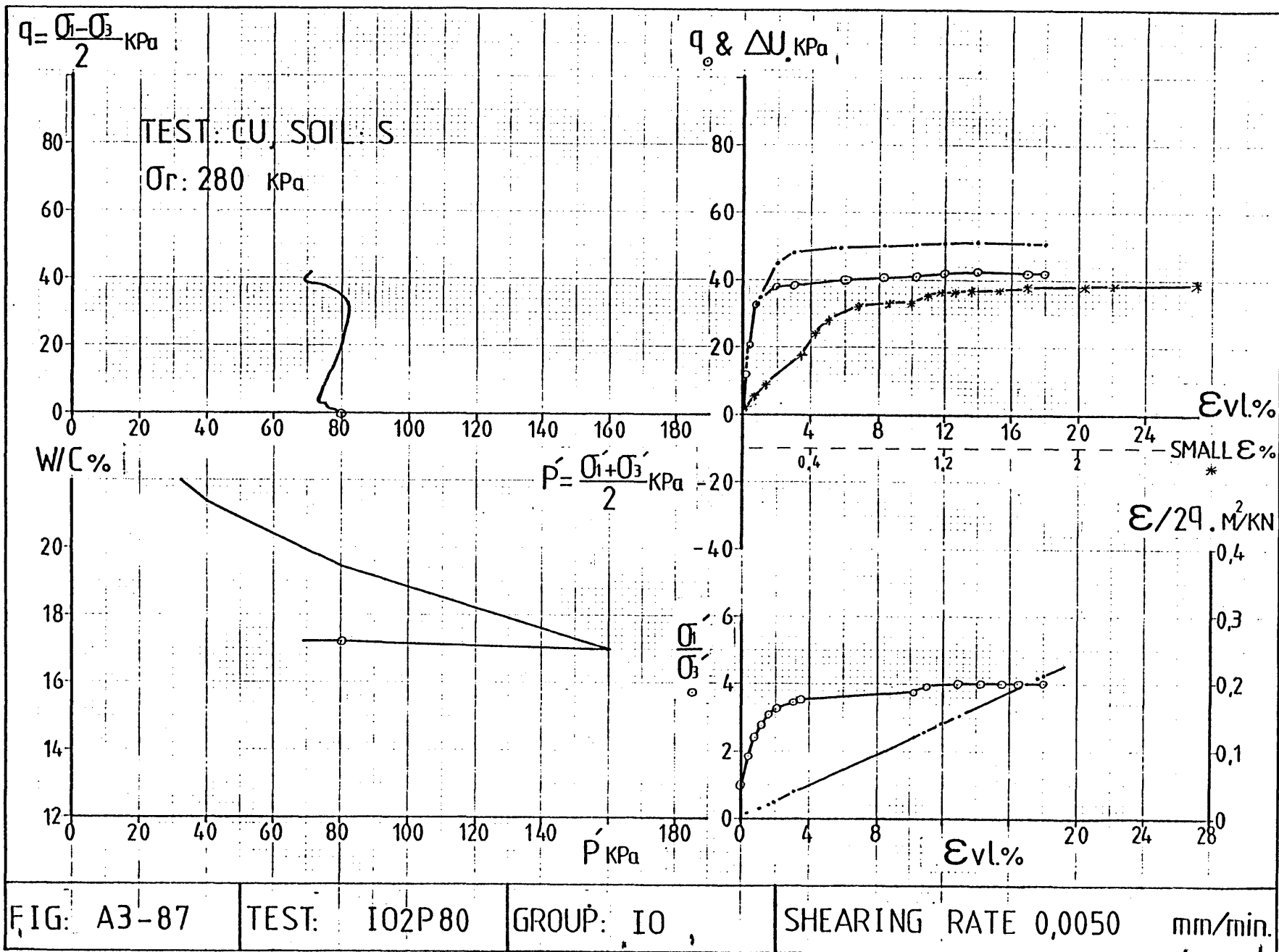
GROUP: IO

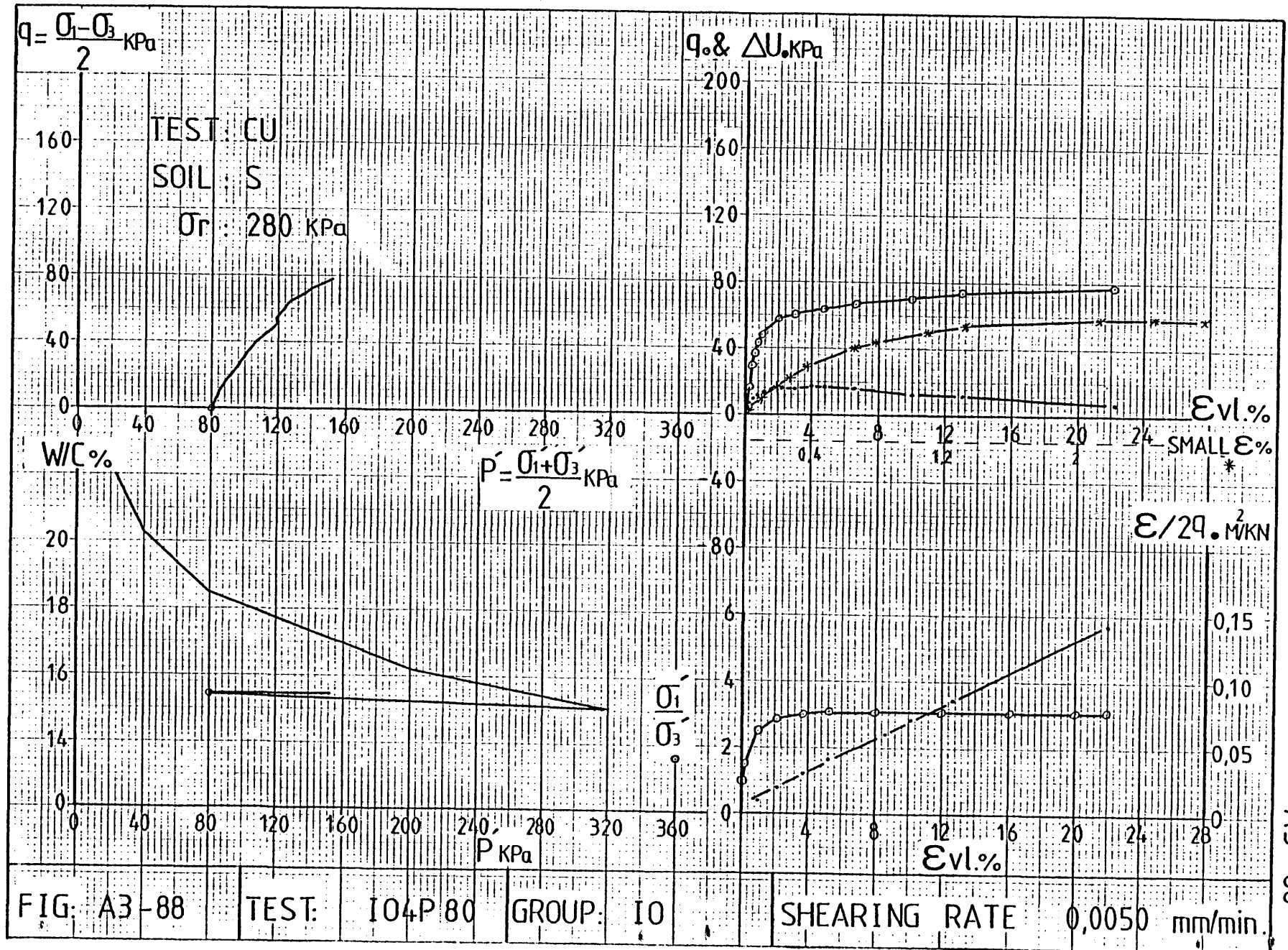
SHEARING RATE:

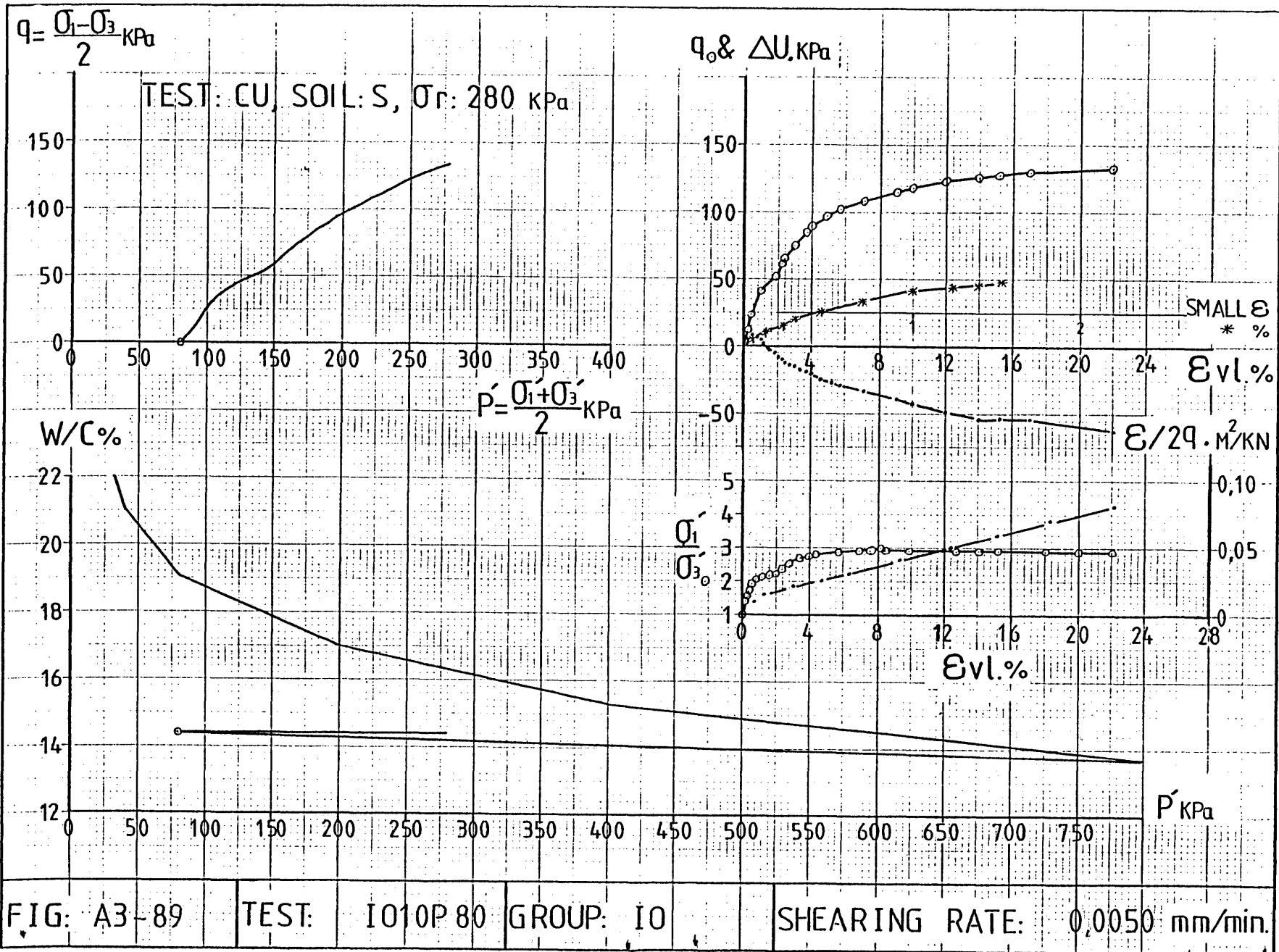
0,0050 mm/min.











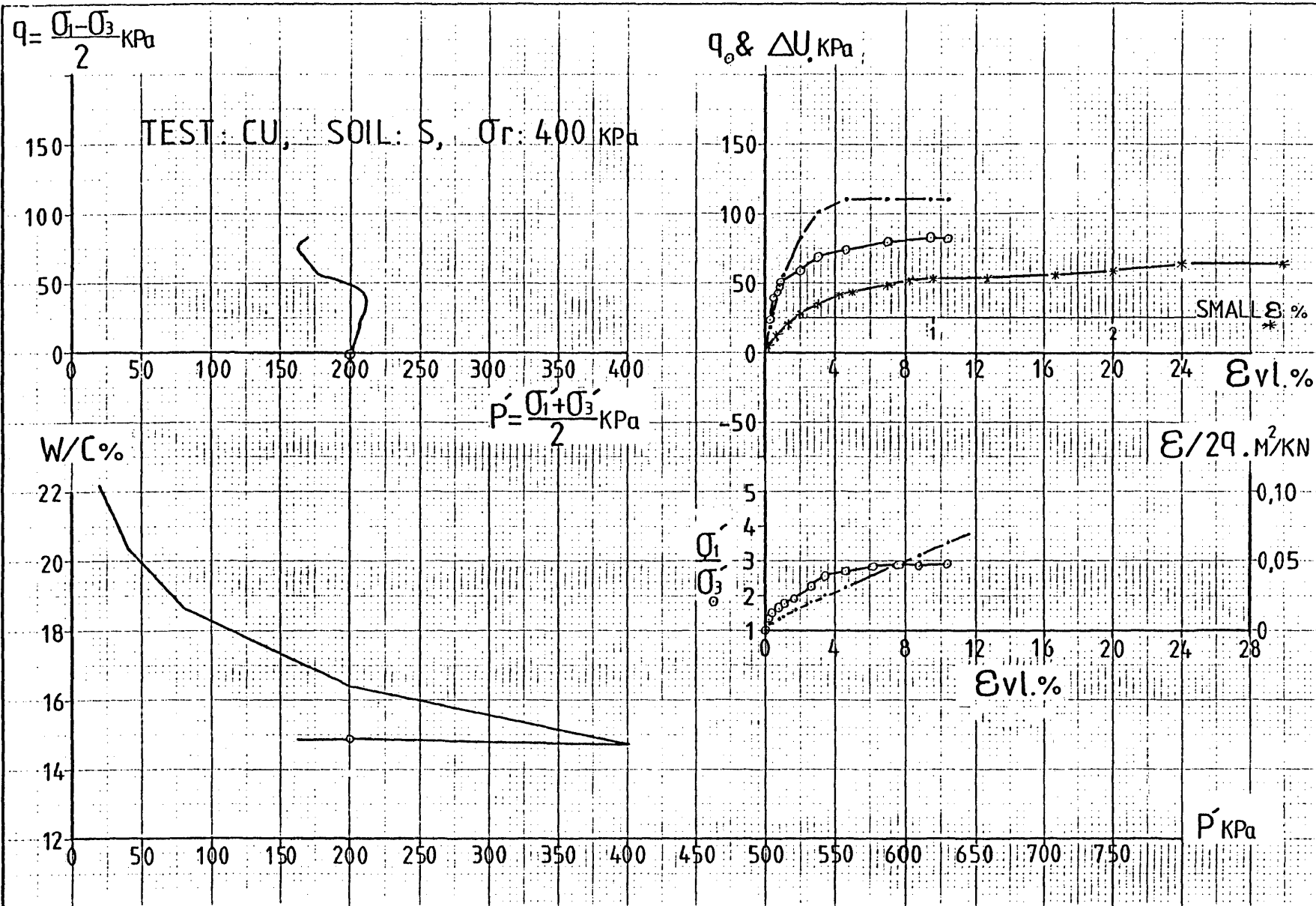
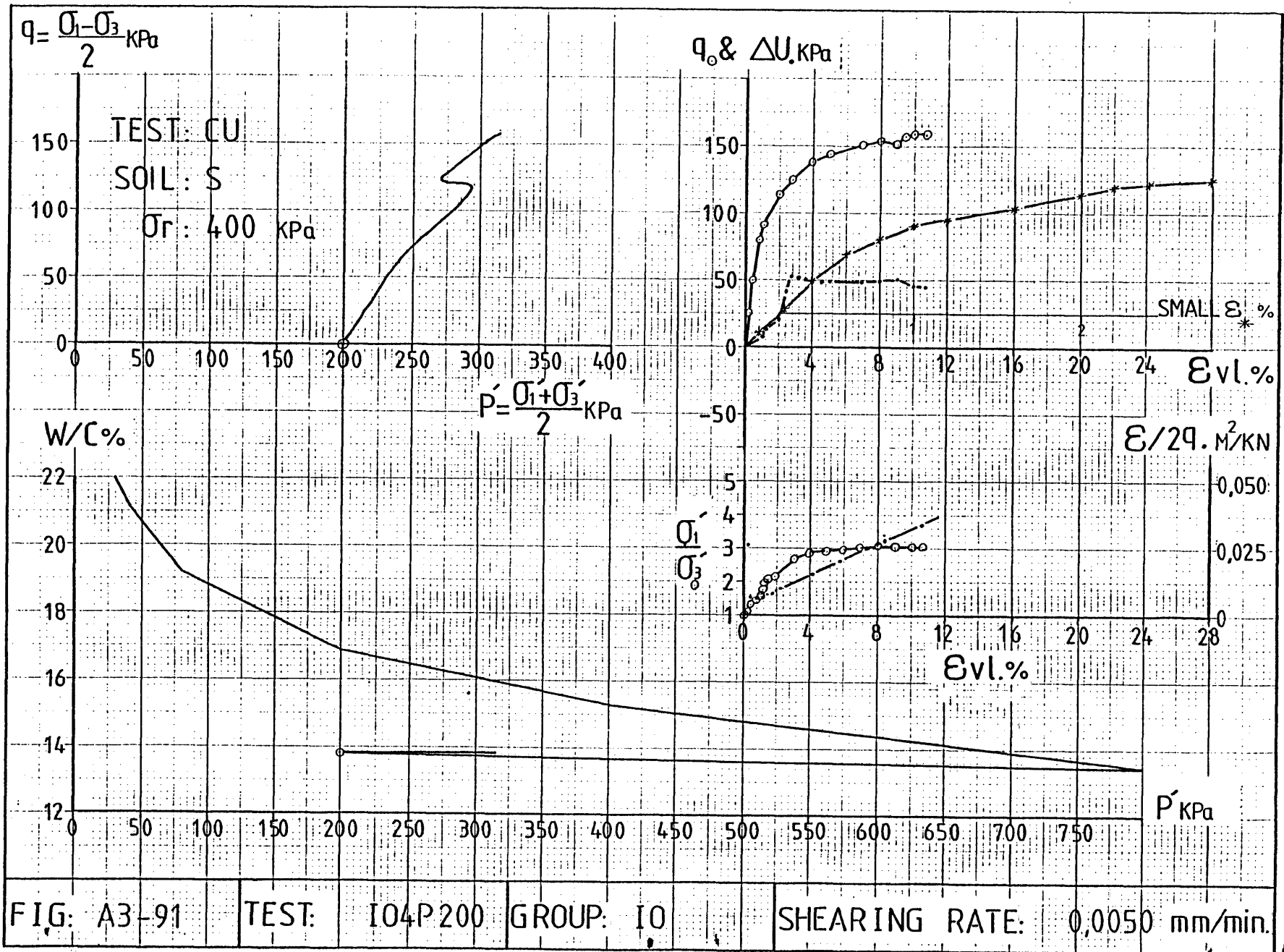
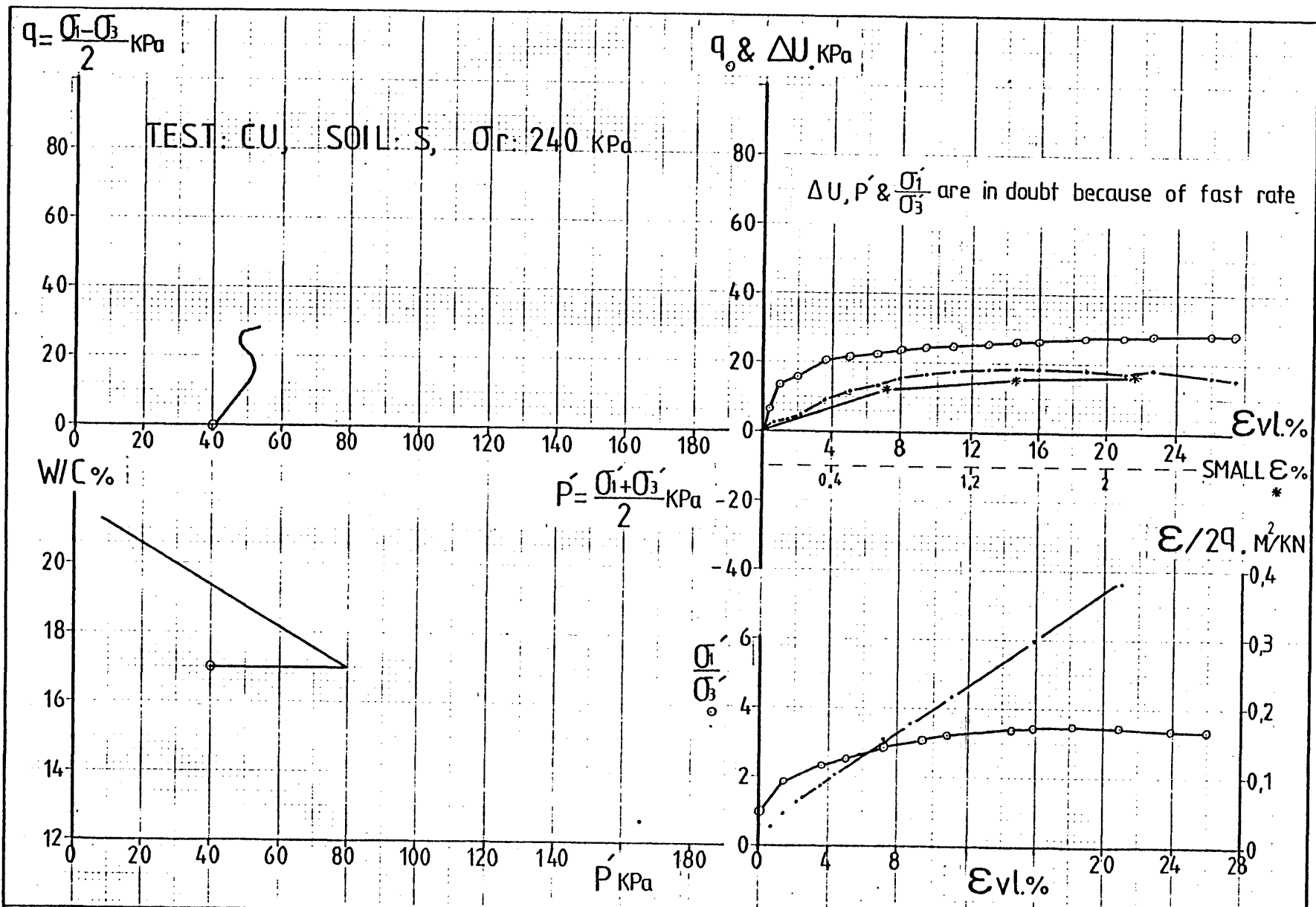


FIG: A3-90 TEST: IO2P 200 GROUP: IO SHEARING RATE: 0,0050 mm/min.





-421-

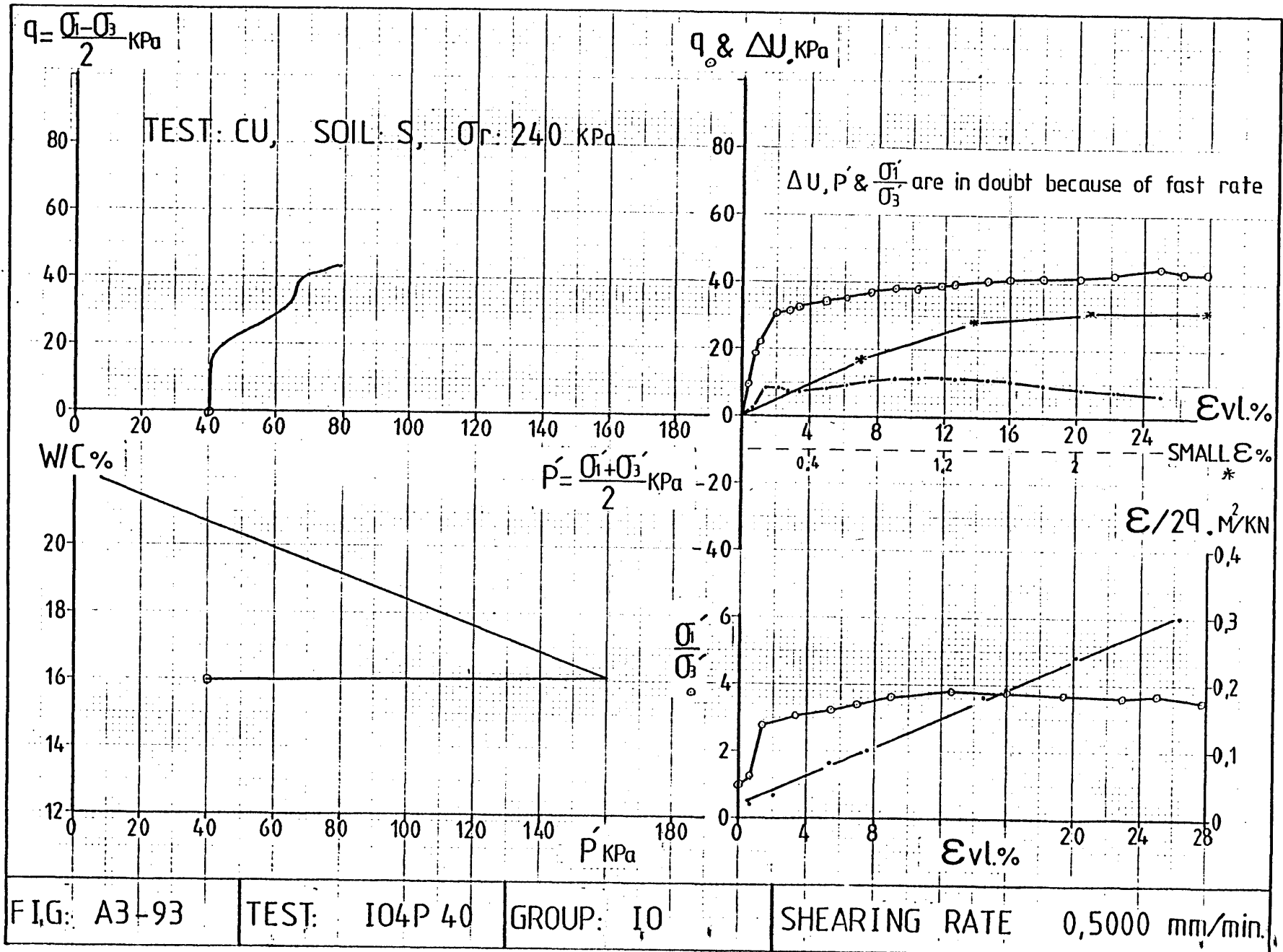
A3-92

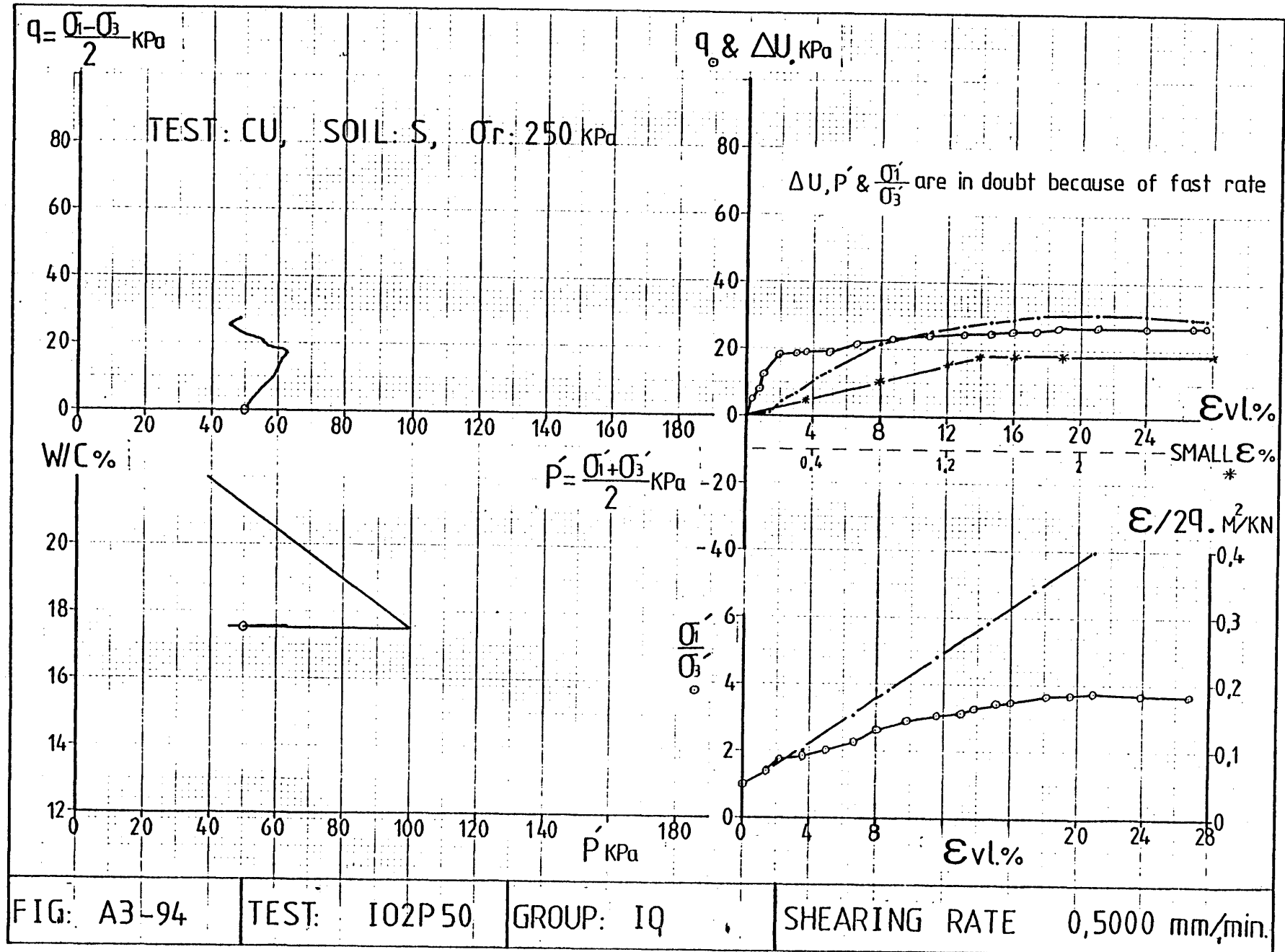
FIG: A3-92

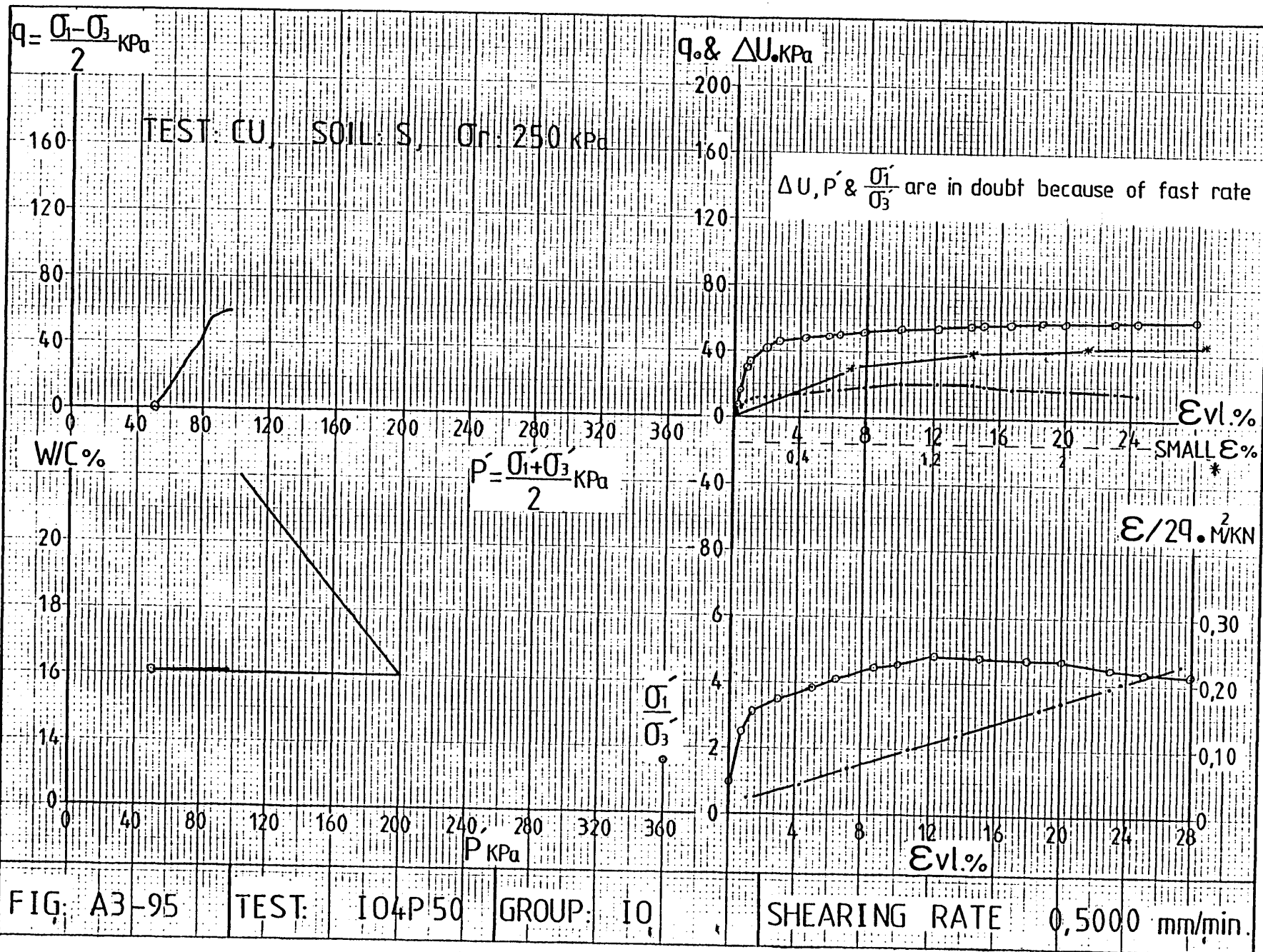
TEST: IO2P 40

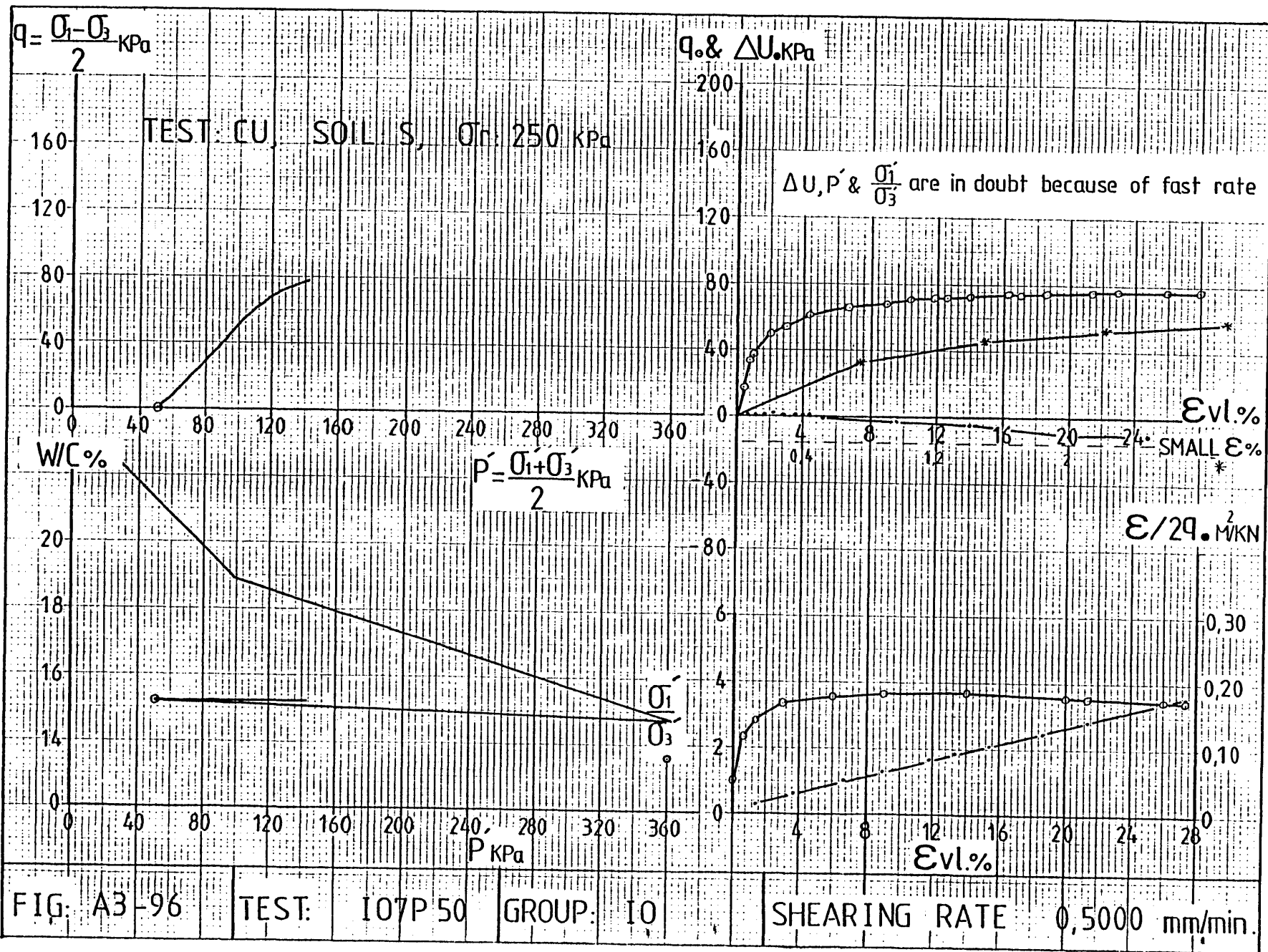
GROUP: IO

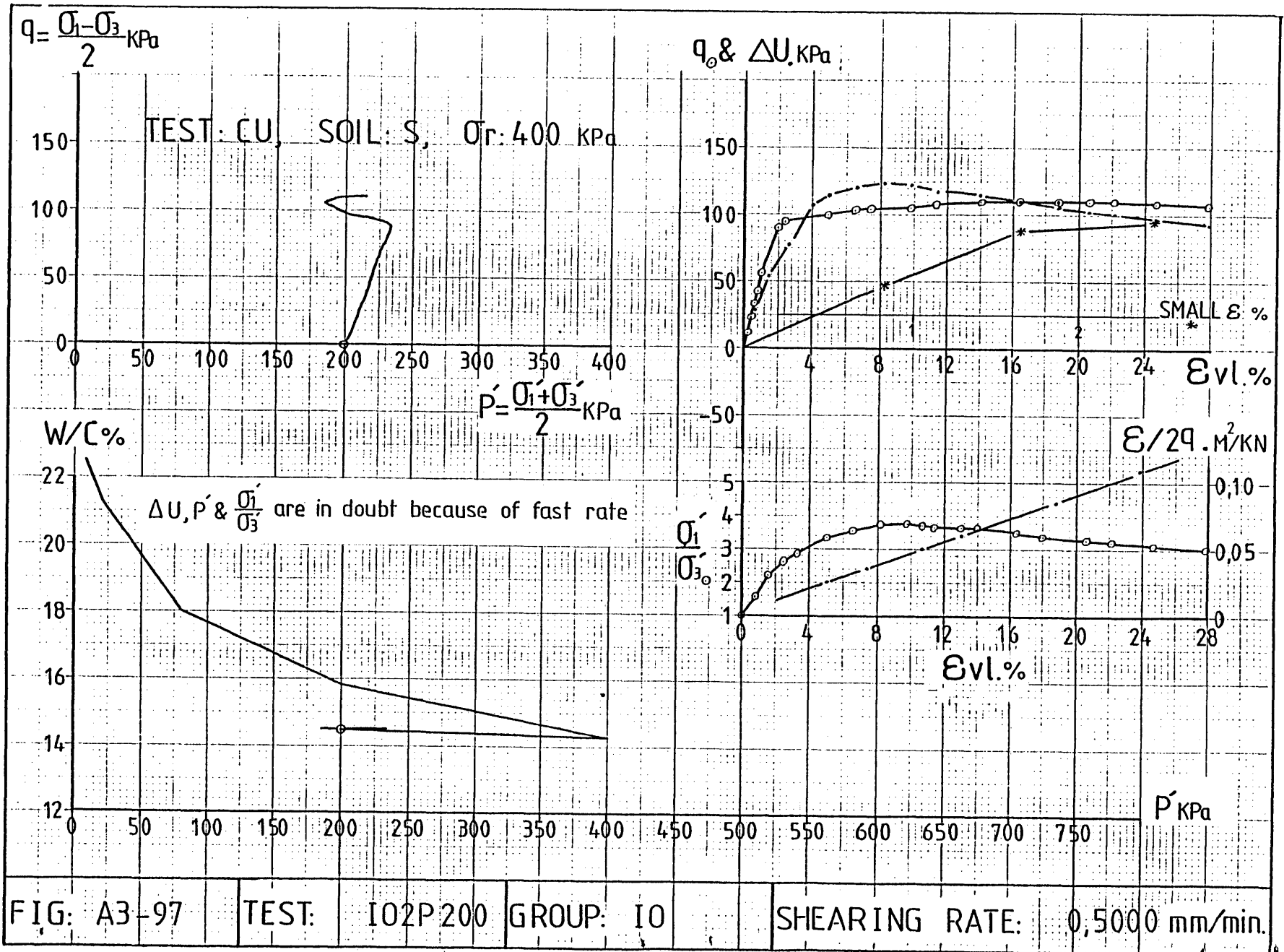
SHEARING RATE 0,5000 mm/min.

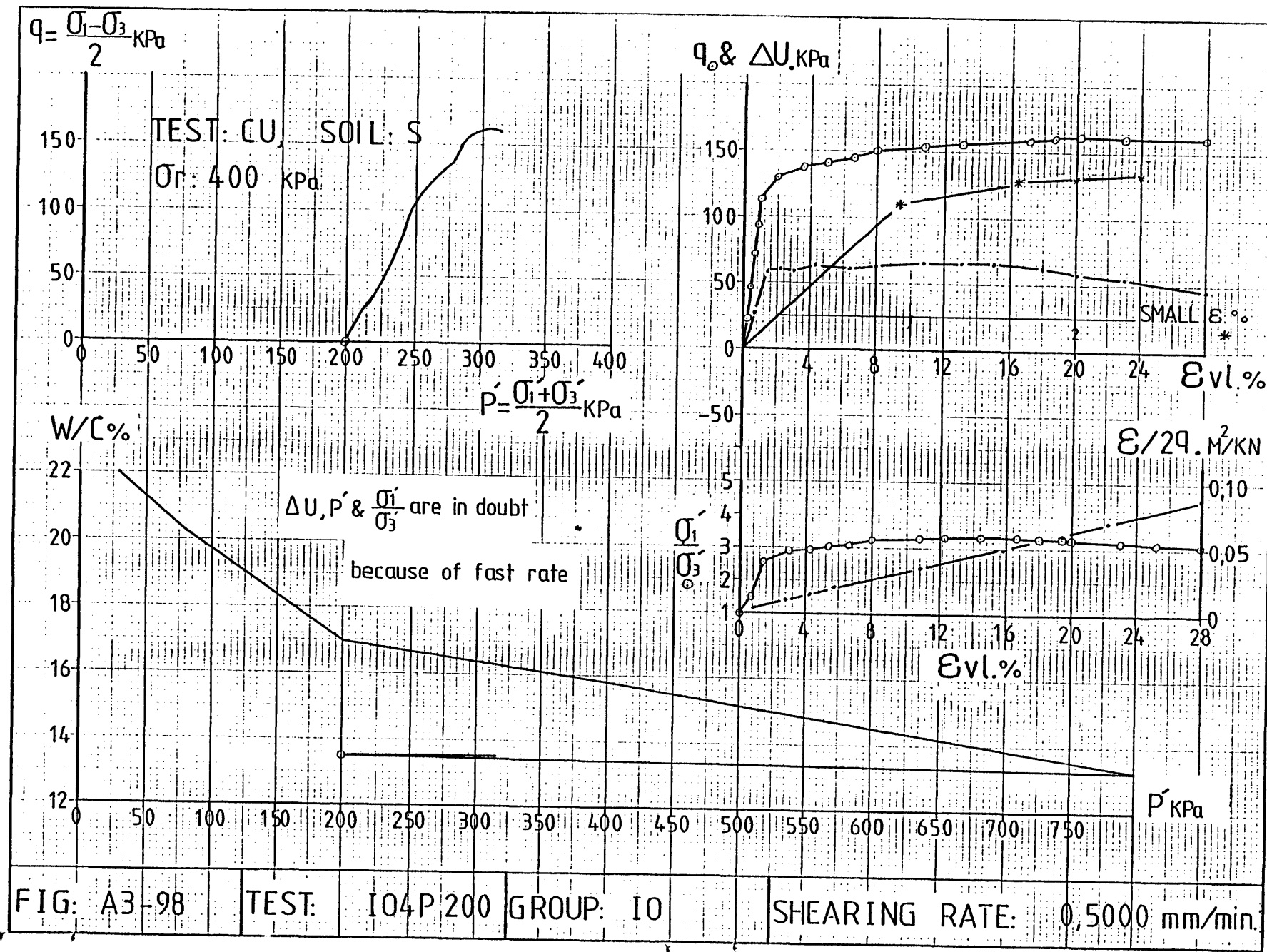


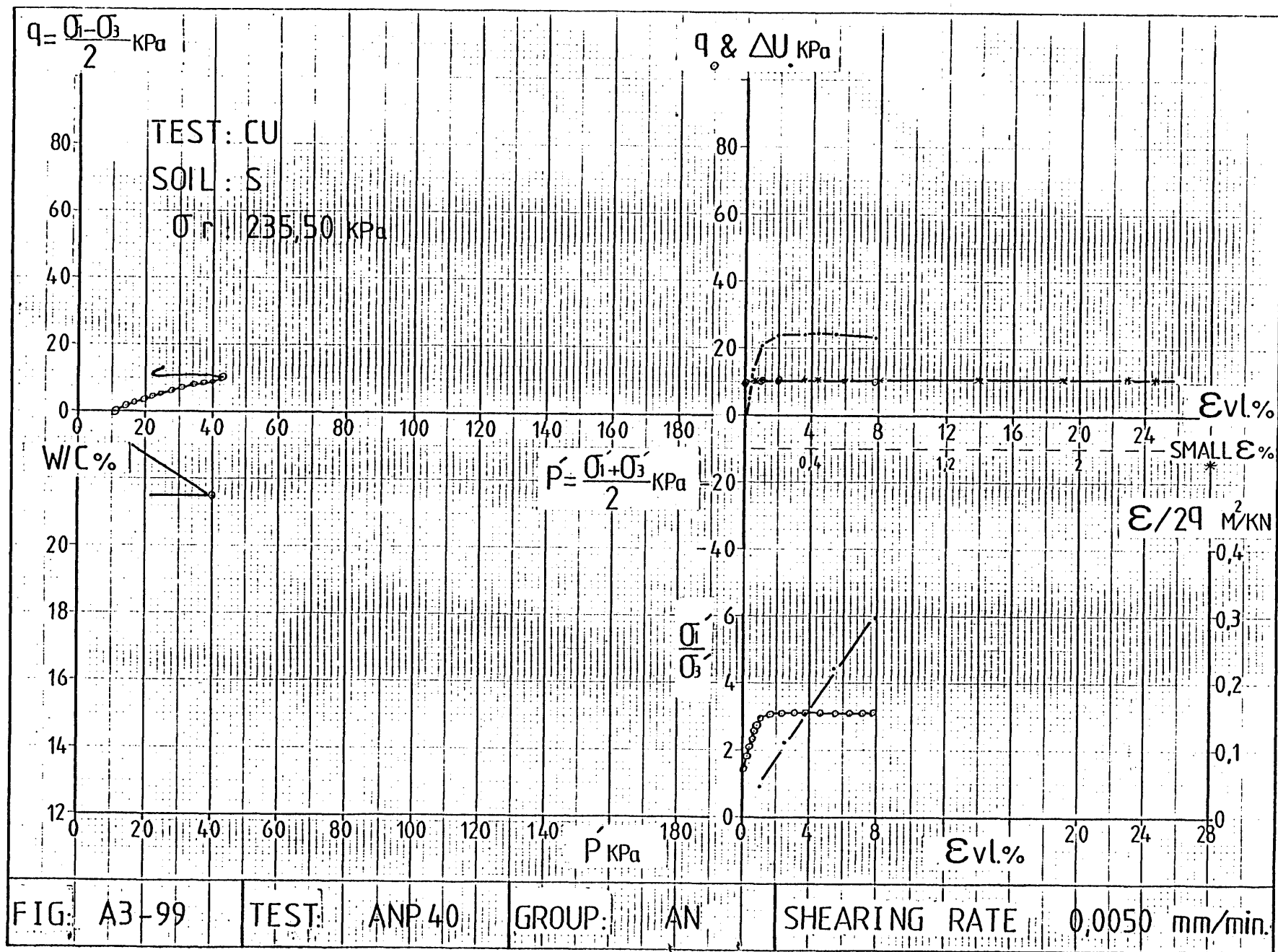


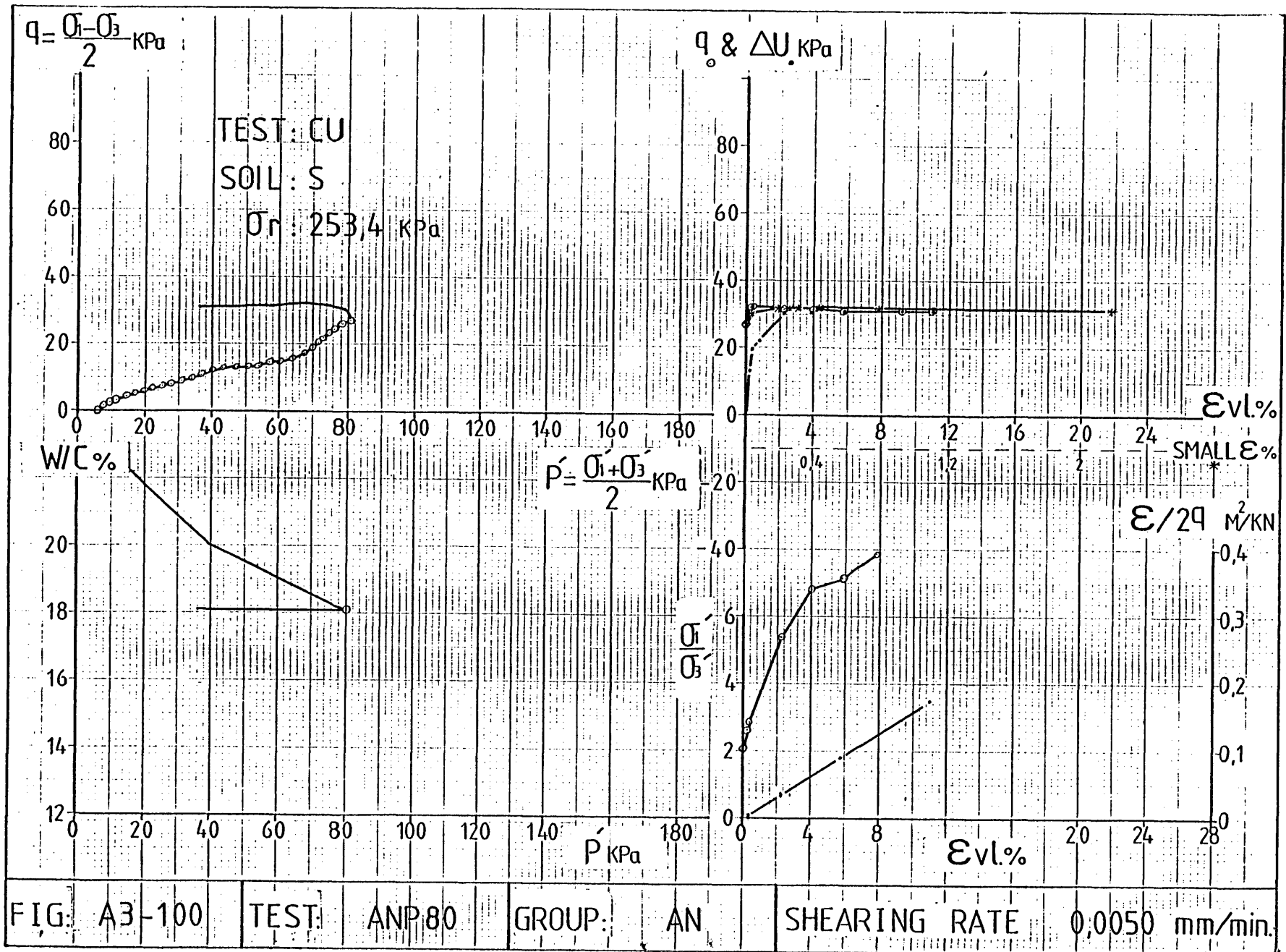












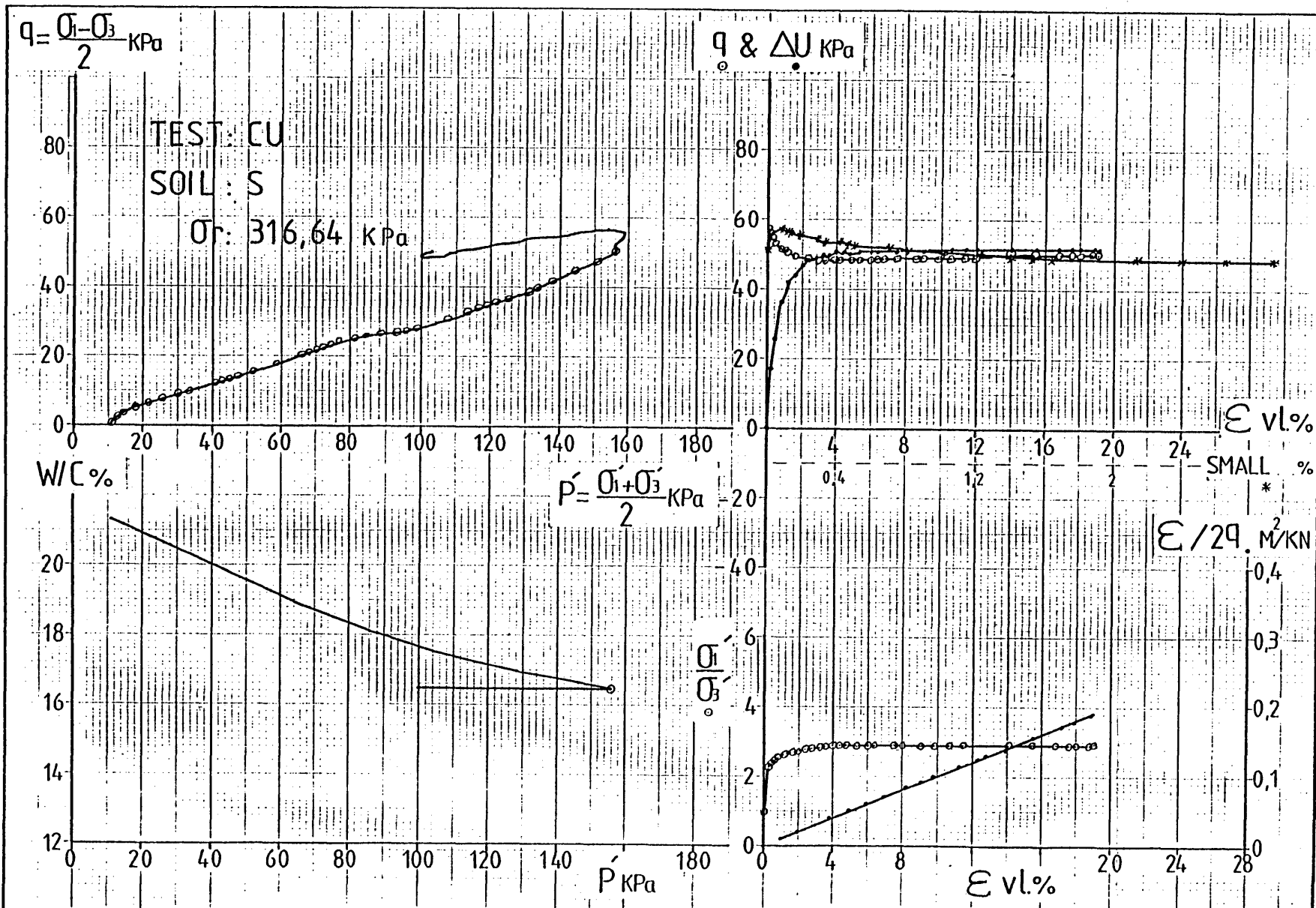


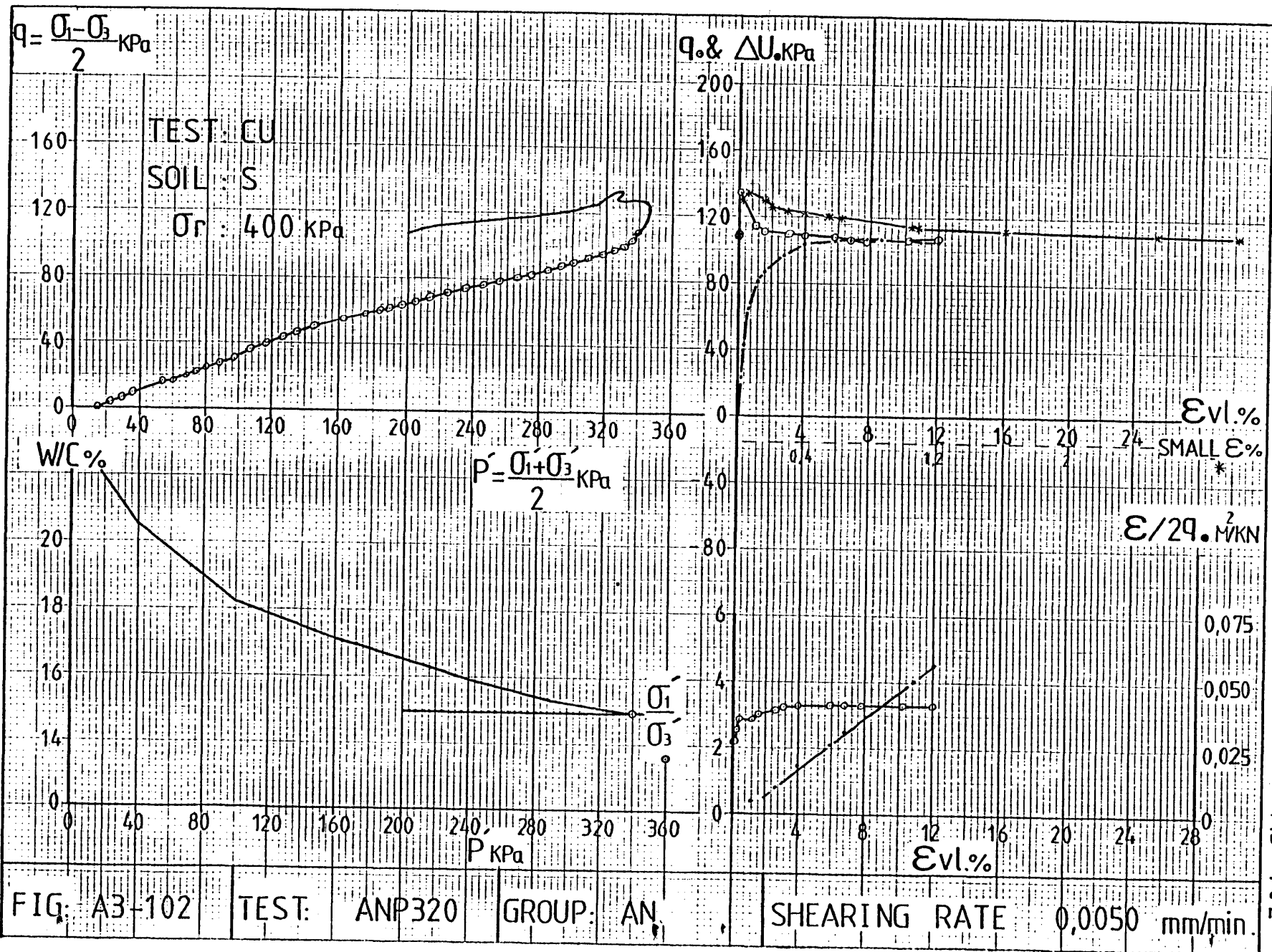
FIG: A3-101

TEST: ANP160

GROUP: AN

SHEARING RATE: 0,0050

mm/min.



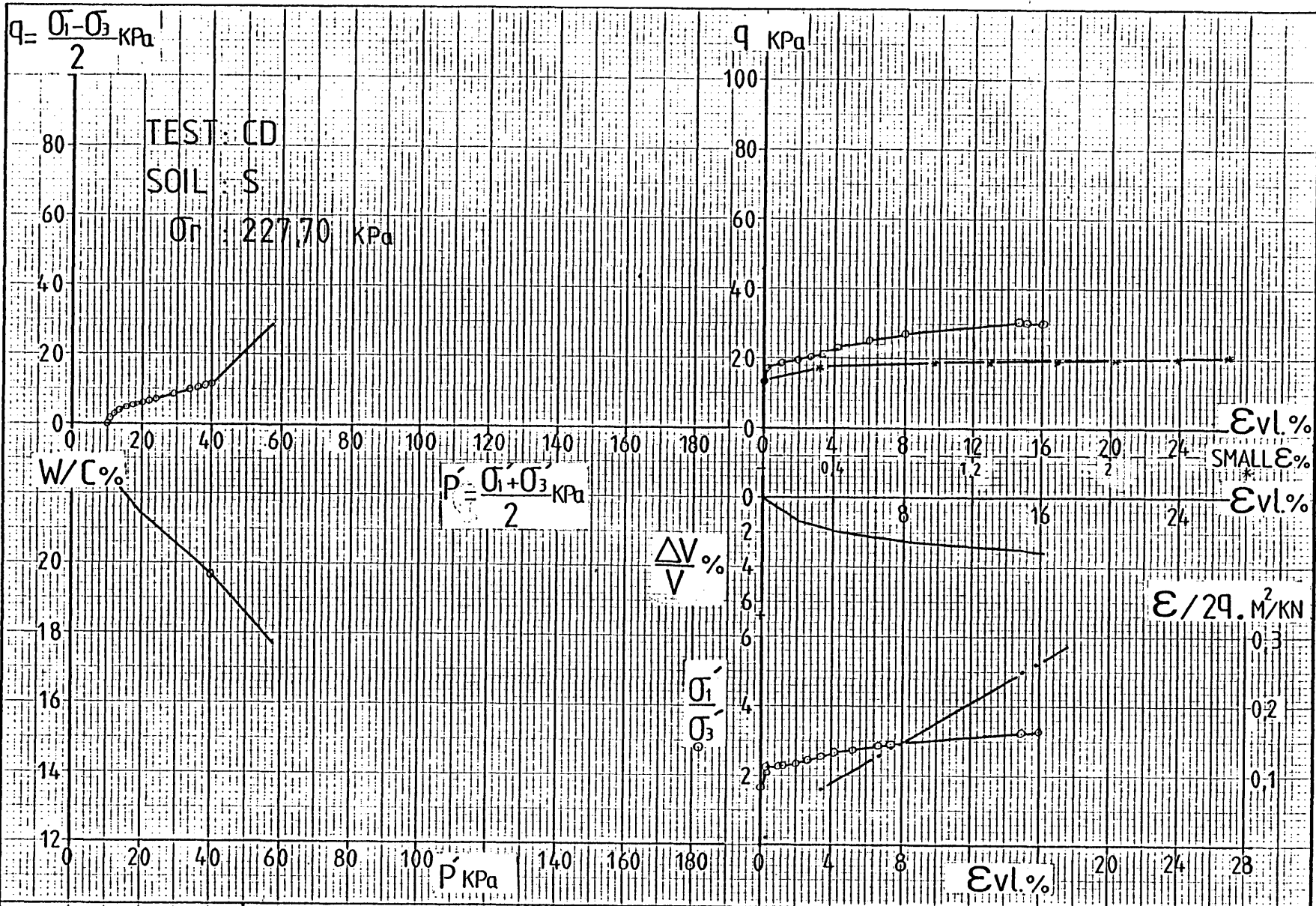
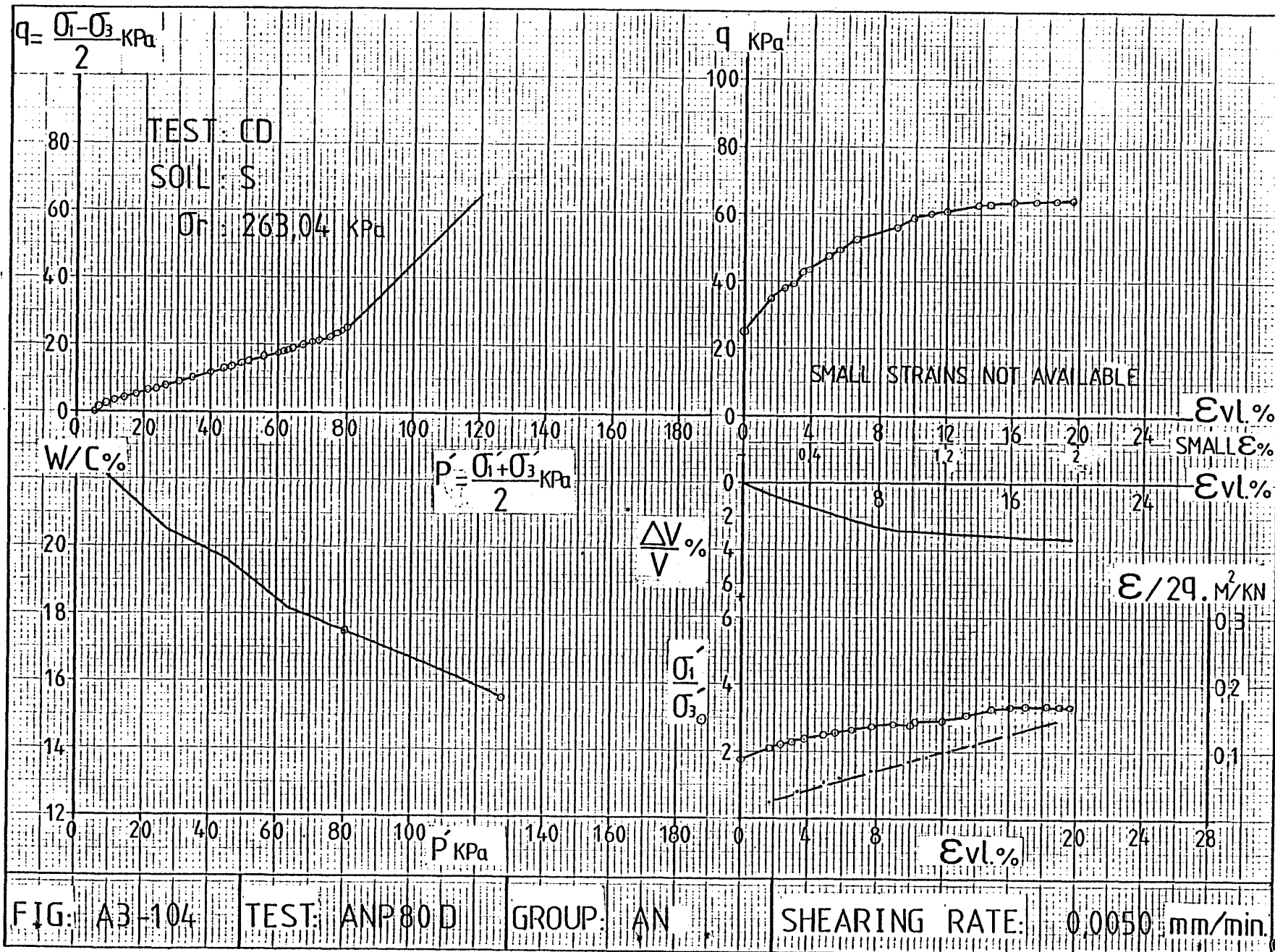
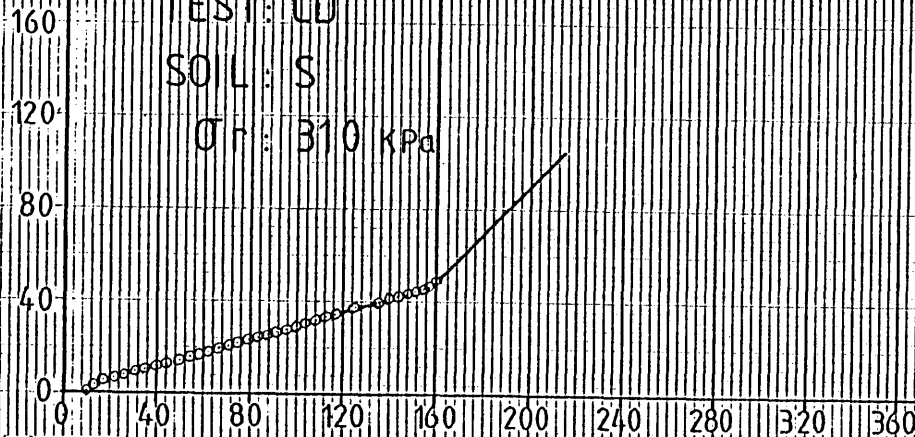


FIG: A3-103 TEST: ANP40 D GROUP: AN SHEARING RATE: 0.0050 mm/min.

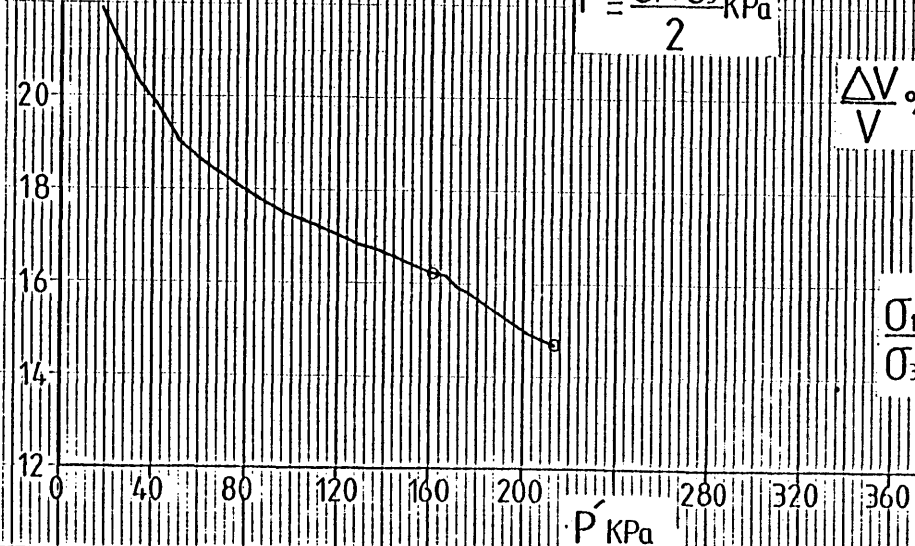


$$q = \frac{\sigma_1 - \sigma_3}{2} \text{ KPa}$$

TEST: CD
SOIL: S
 σ_r : 310 KPa



W/C%



$$\sigma' = \frac{\sigma_1 + \sigma_3}{2} \text{ KPa}$$

$$\frac{\Delta V}{V} \%$$

$$\frac{\sigma_1}{\sigma_3}$$

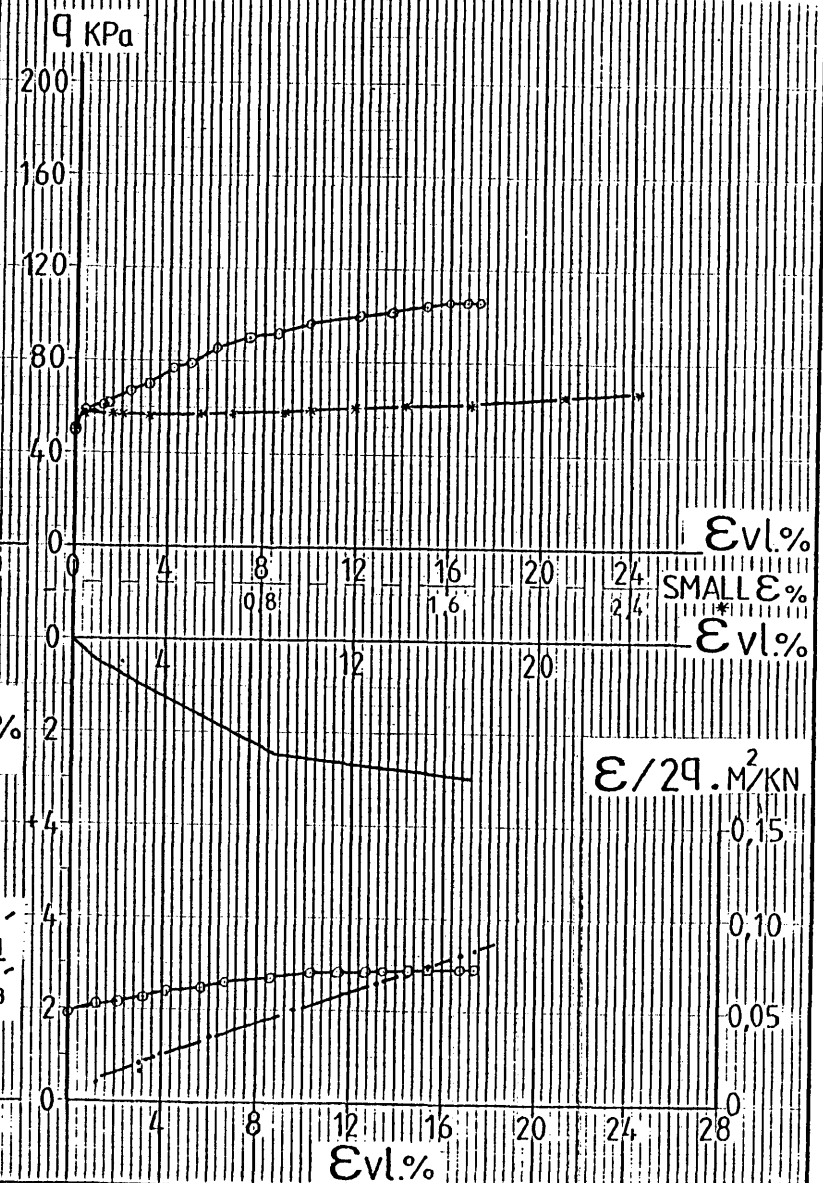


FIG: AB-105

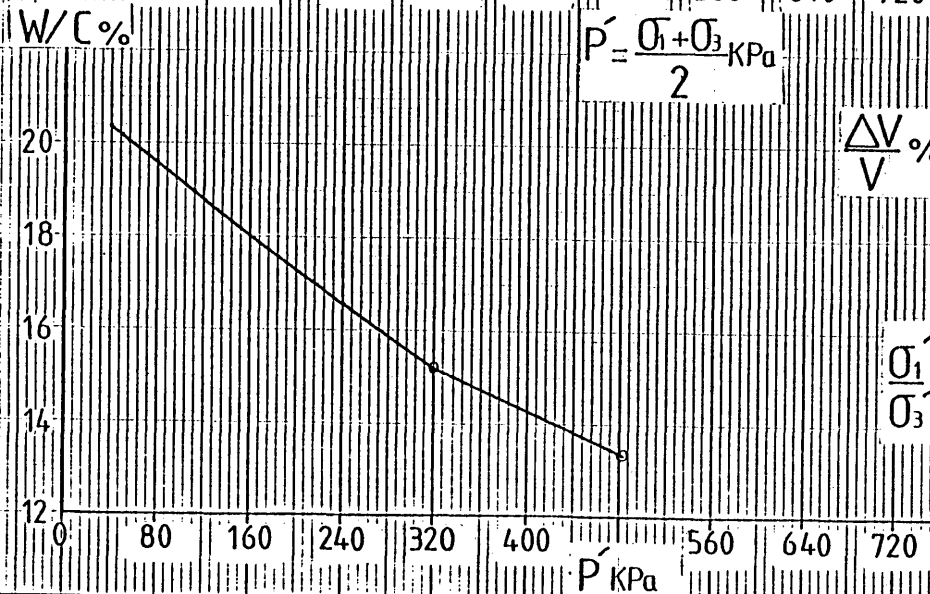
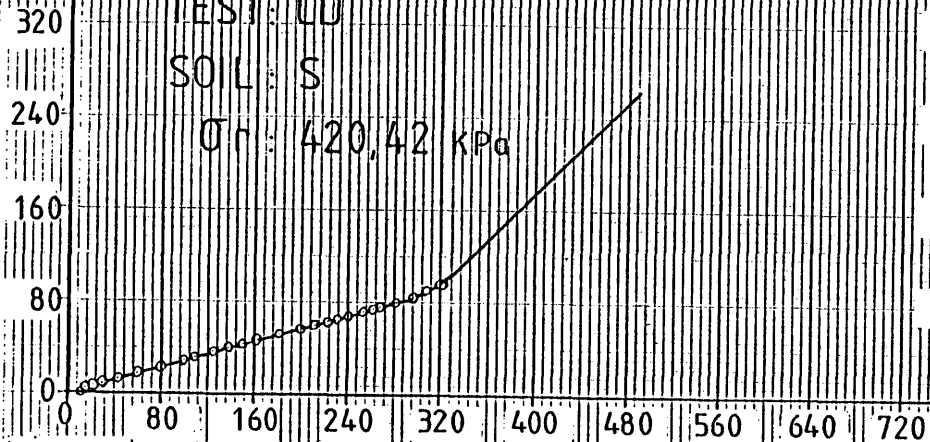
TEST: ANP160D

GROUP: AN

SHEARING RATE 0,0050 mm/min

$$q = \frac{\sigma_1 - \sigma_3}{2} \text{ KPa}$$

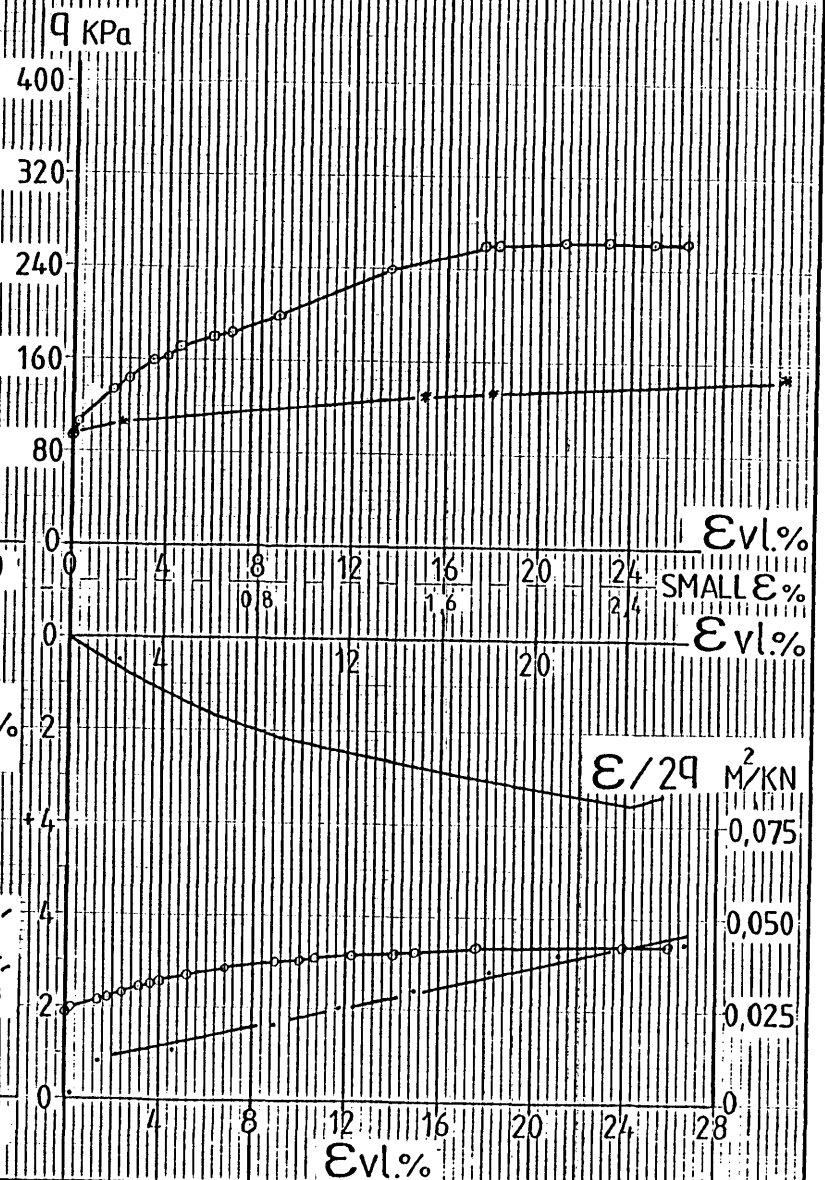
TEST: CD
SOIL: S
 $\sigma_1 = 420,42 \text{ KPa}$



$$P' = \frac{\sigma_1 + \sigma_3}{2} \text{ KPa}$$

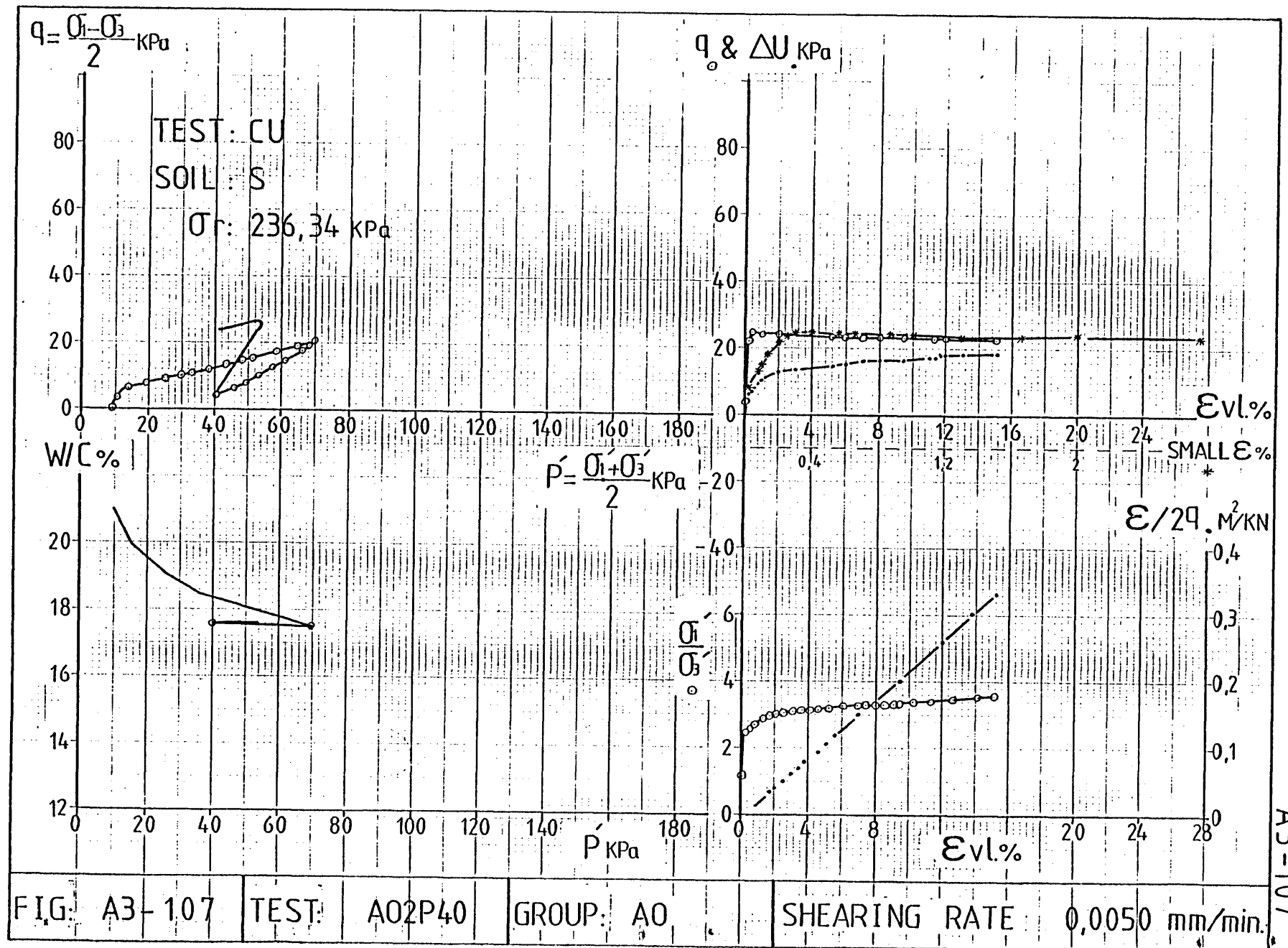
$$\frac{\Delta V}{V} \%$$

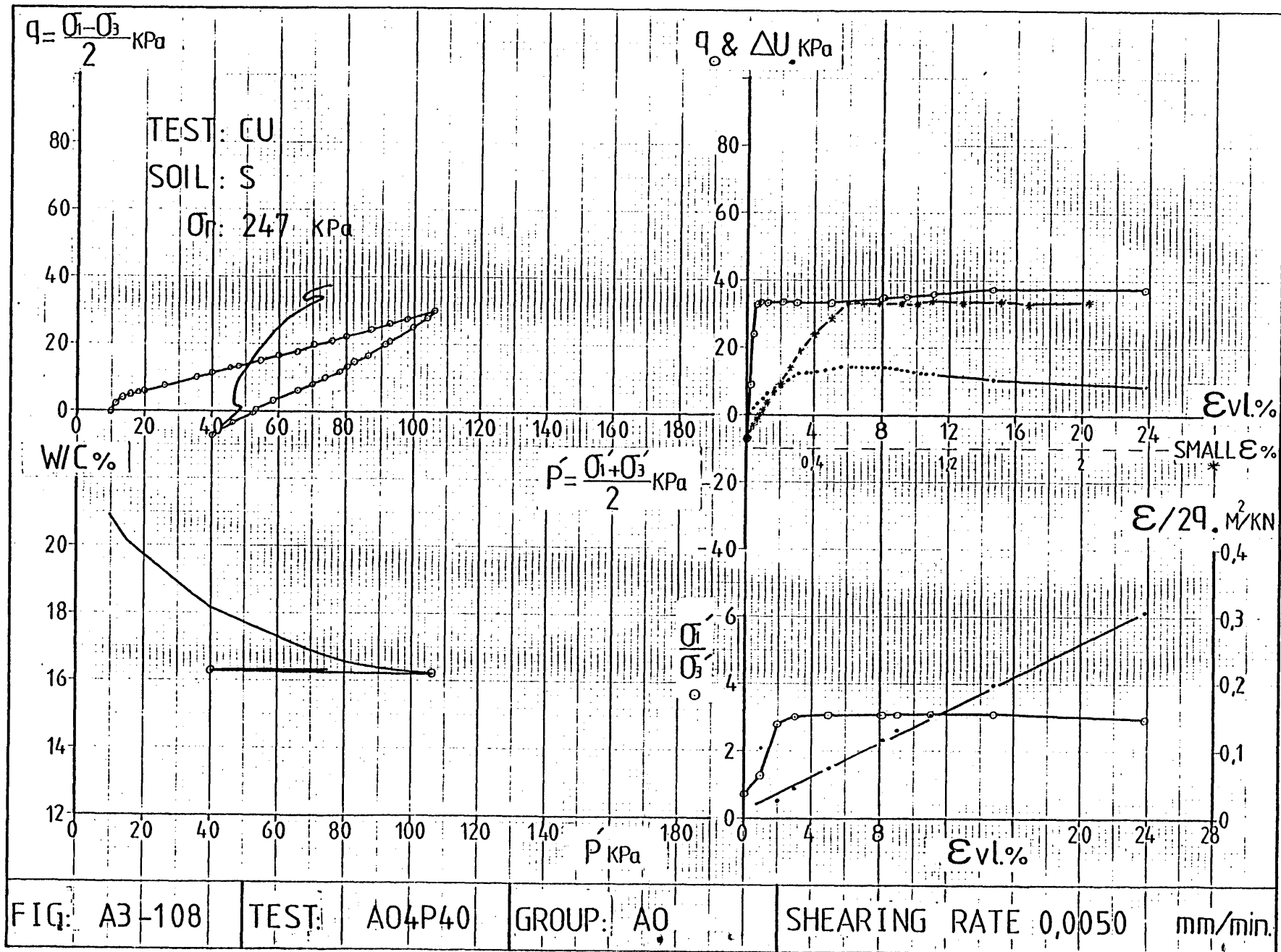
$$\frac{\sigma_1'}{\sigma_3'}$$

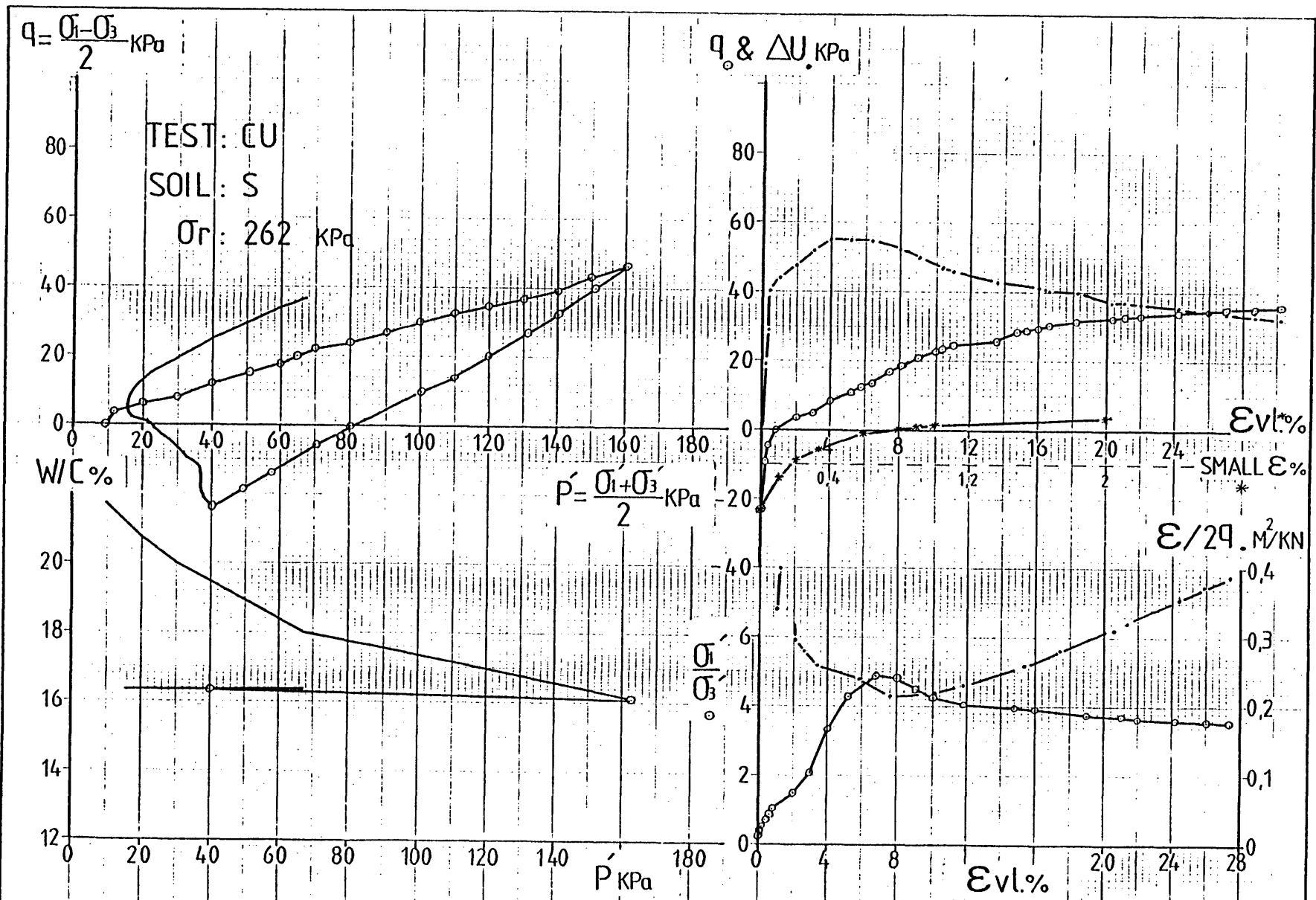


$$\frac{\epsilon}{2q} \frac{M^2}{KN}$$

FIG: AB-106 TEST: ANP320 D GROUP: AN SHEARING RATE 0,0050 mm/min.



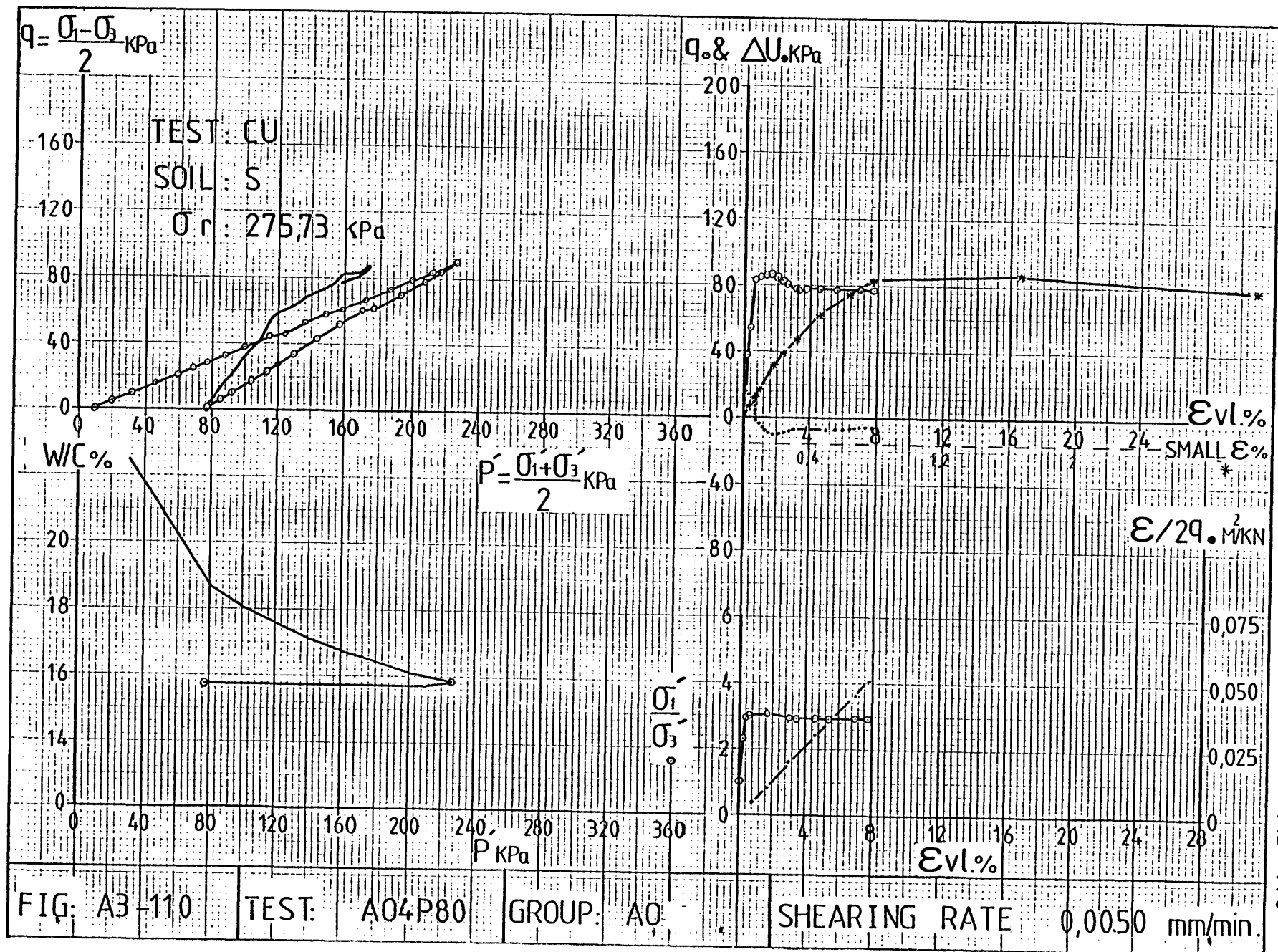


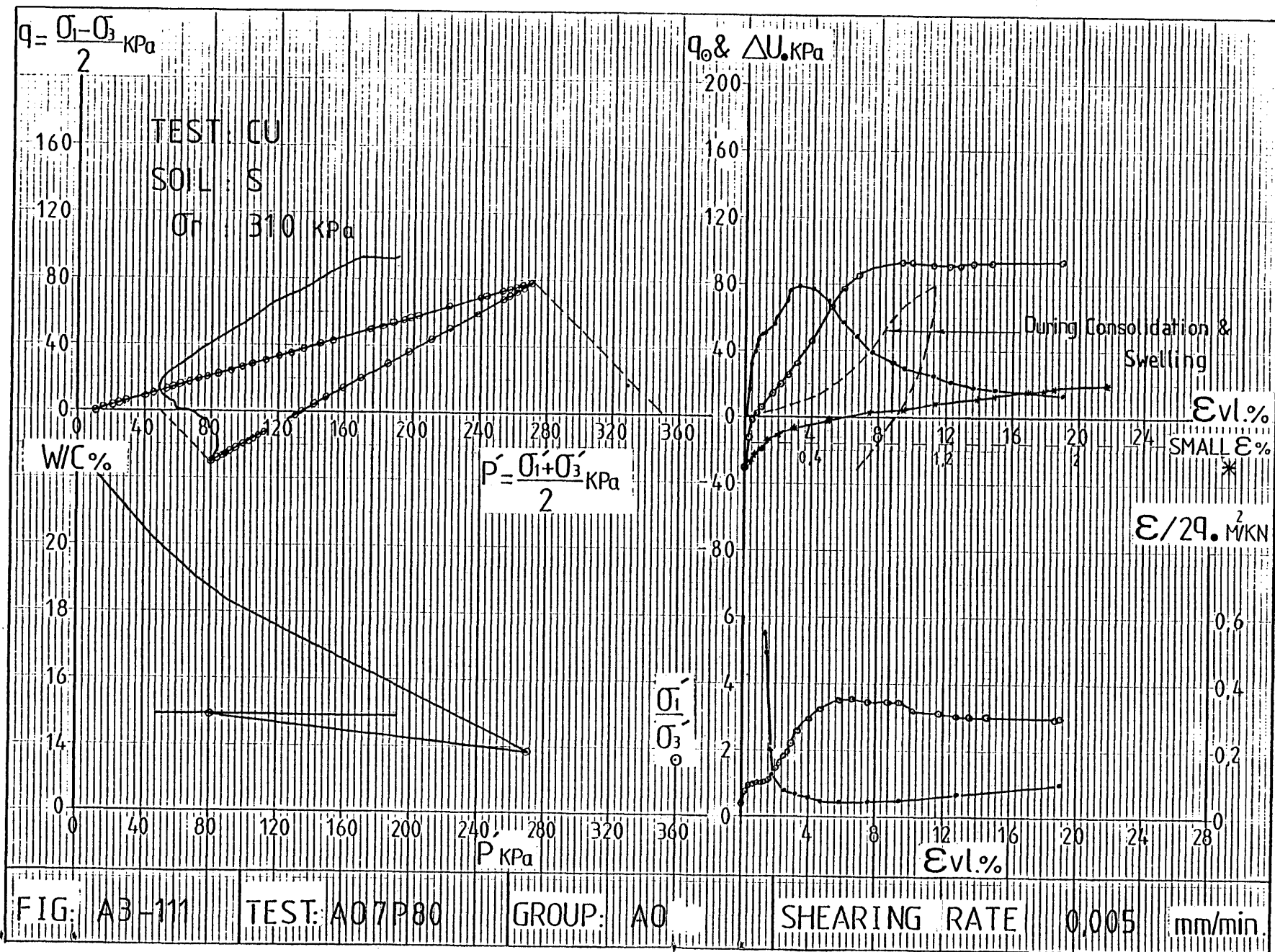


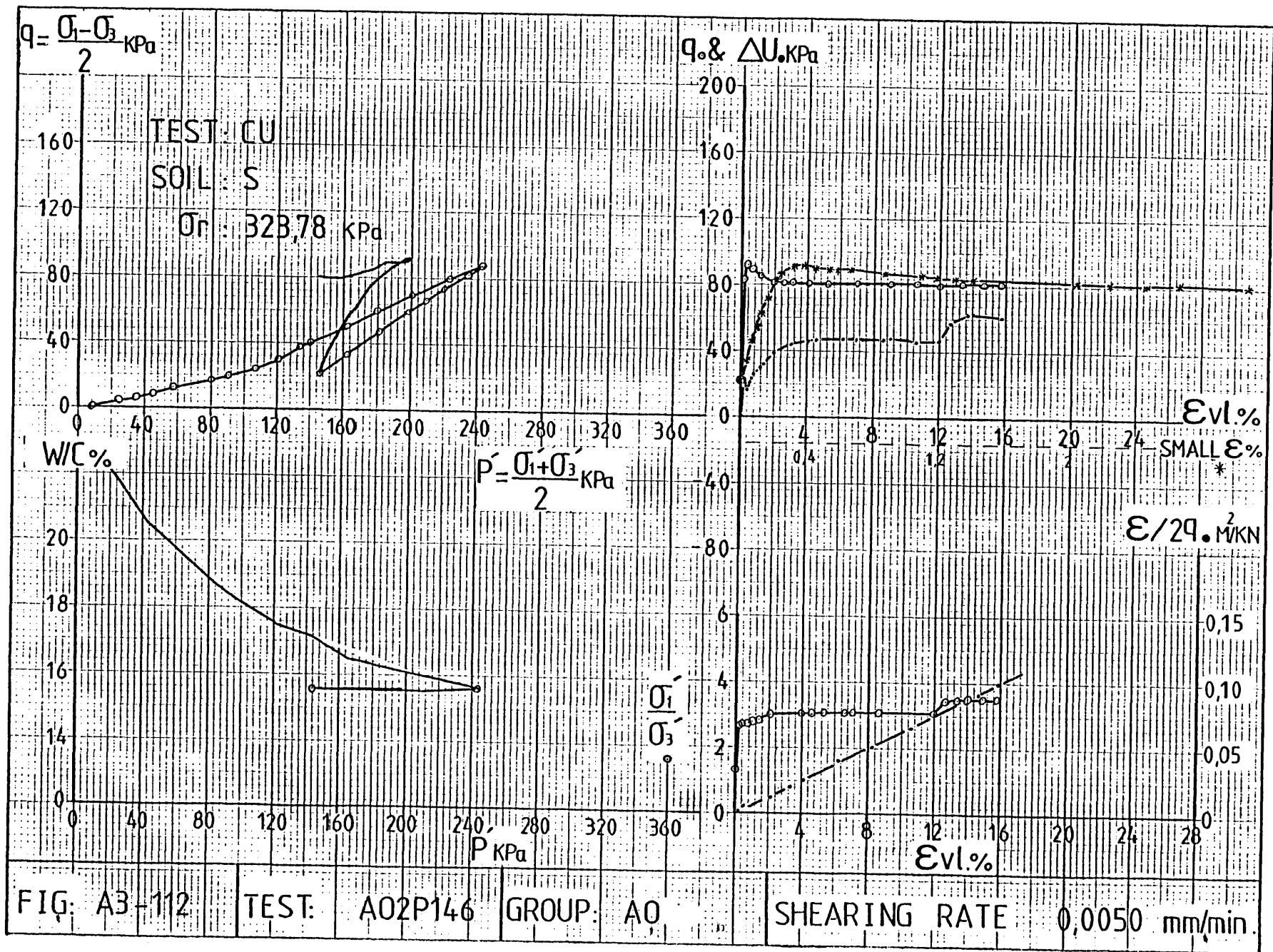
-438-

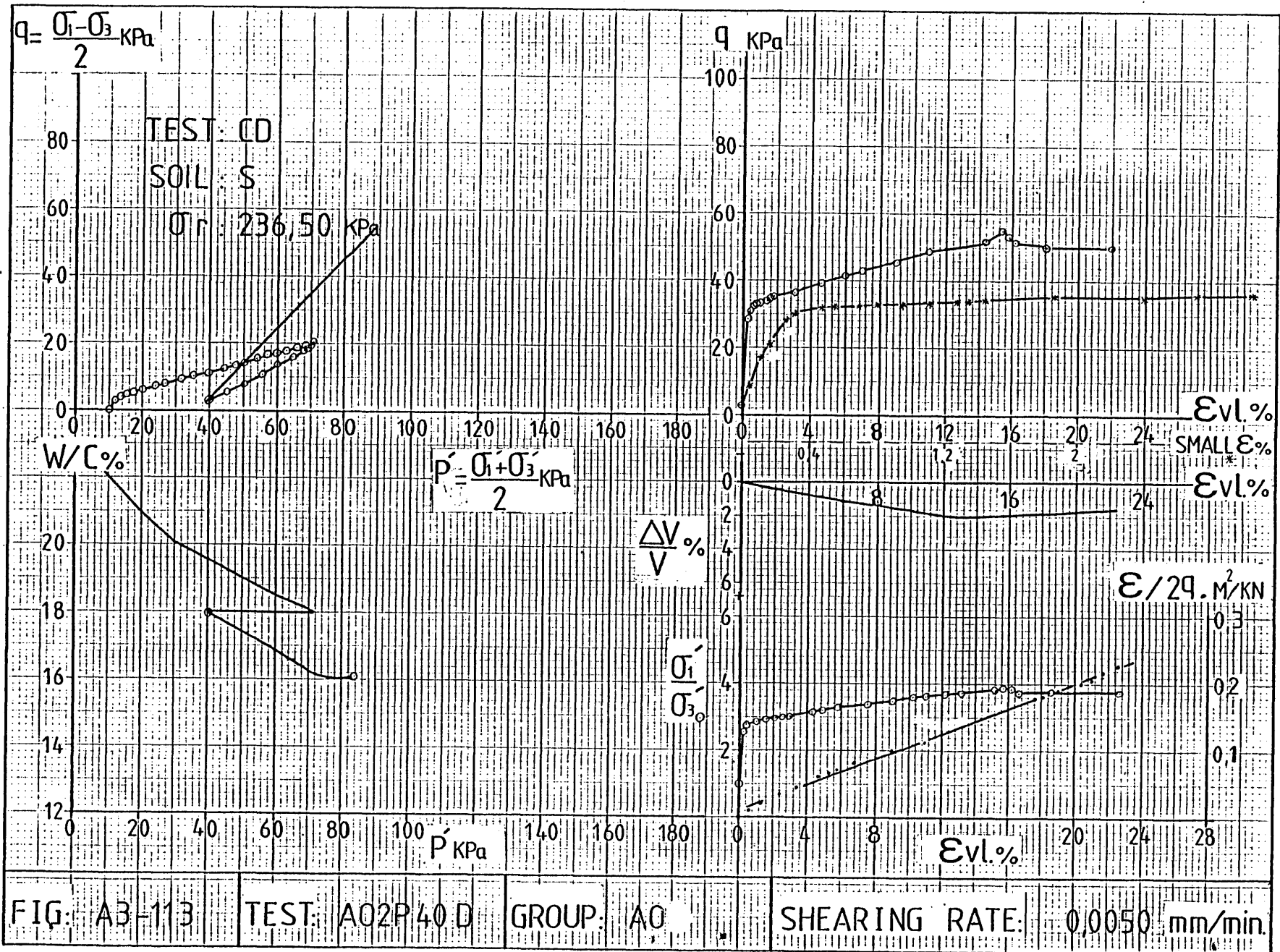
A3-109

FIG: A3-109	TEST: A010P40	GROUP: A0	SHEARING RATE: 0,0050 mm/min.
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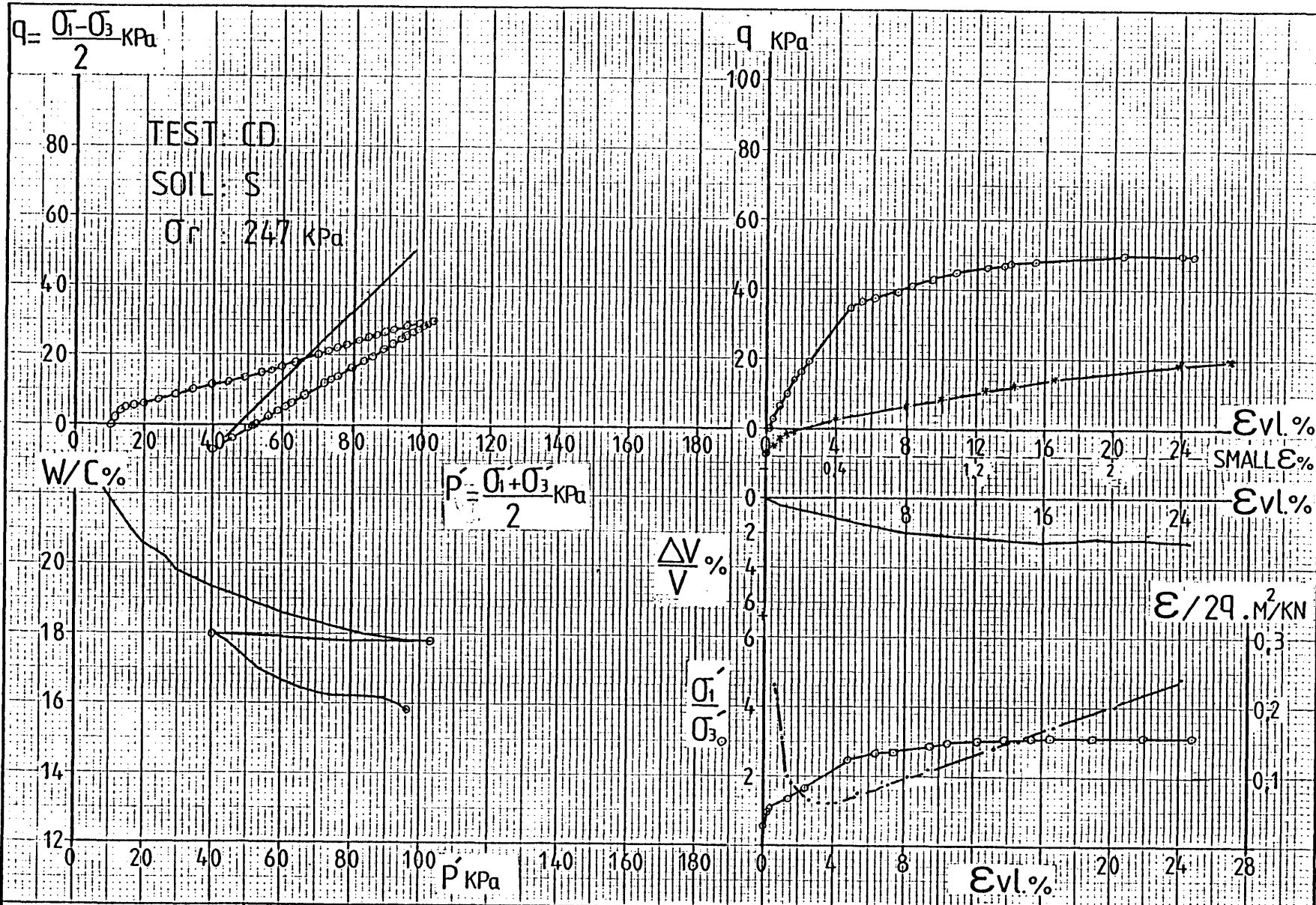


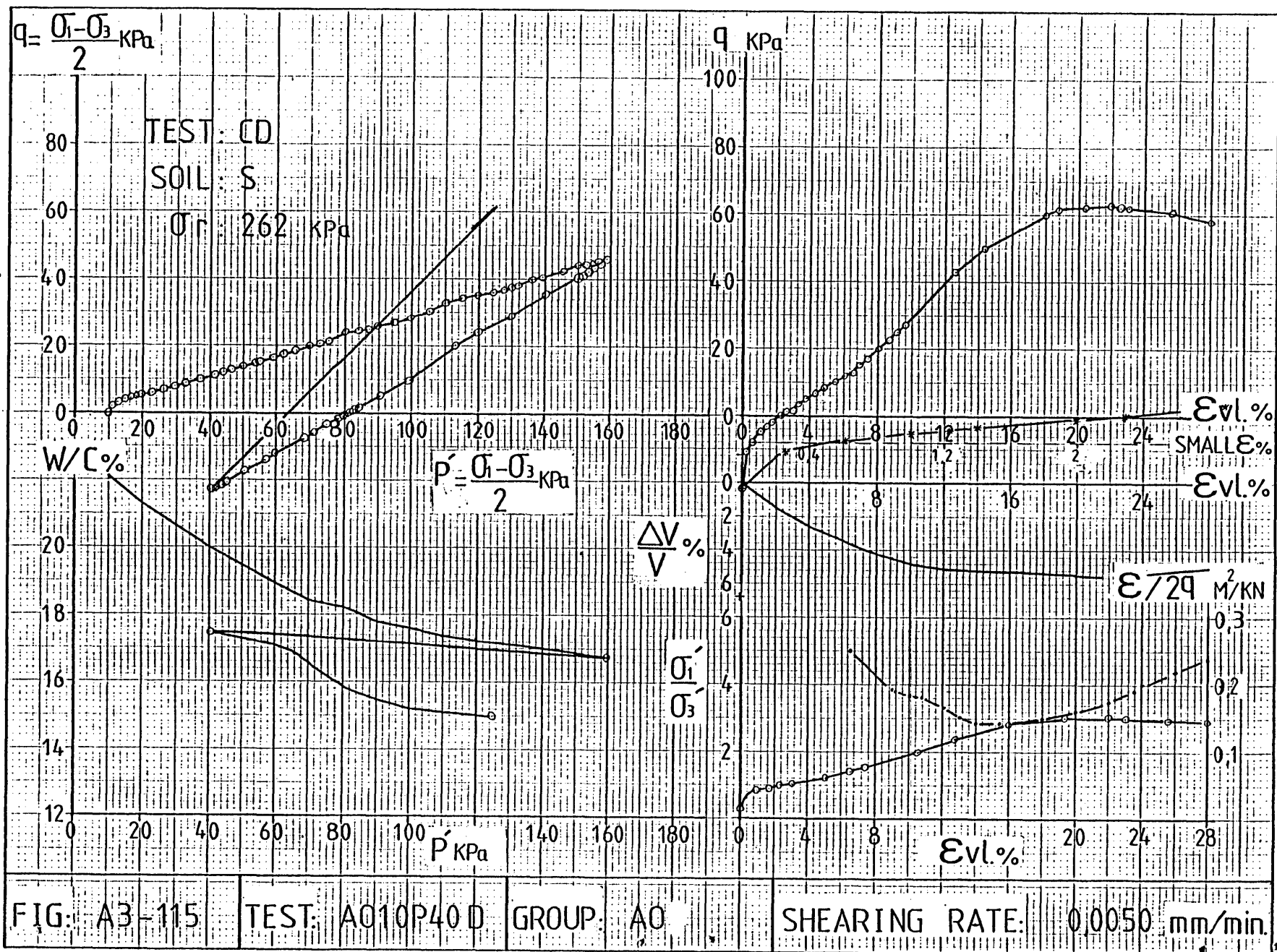
FIG. A3-114

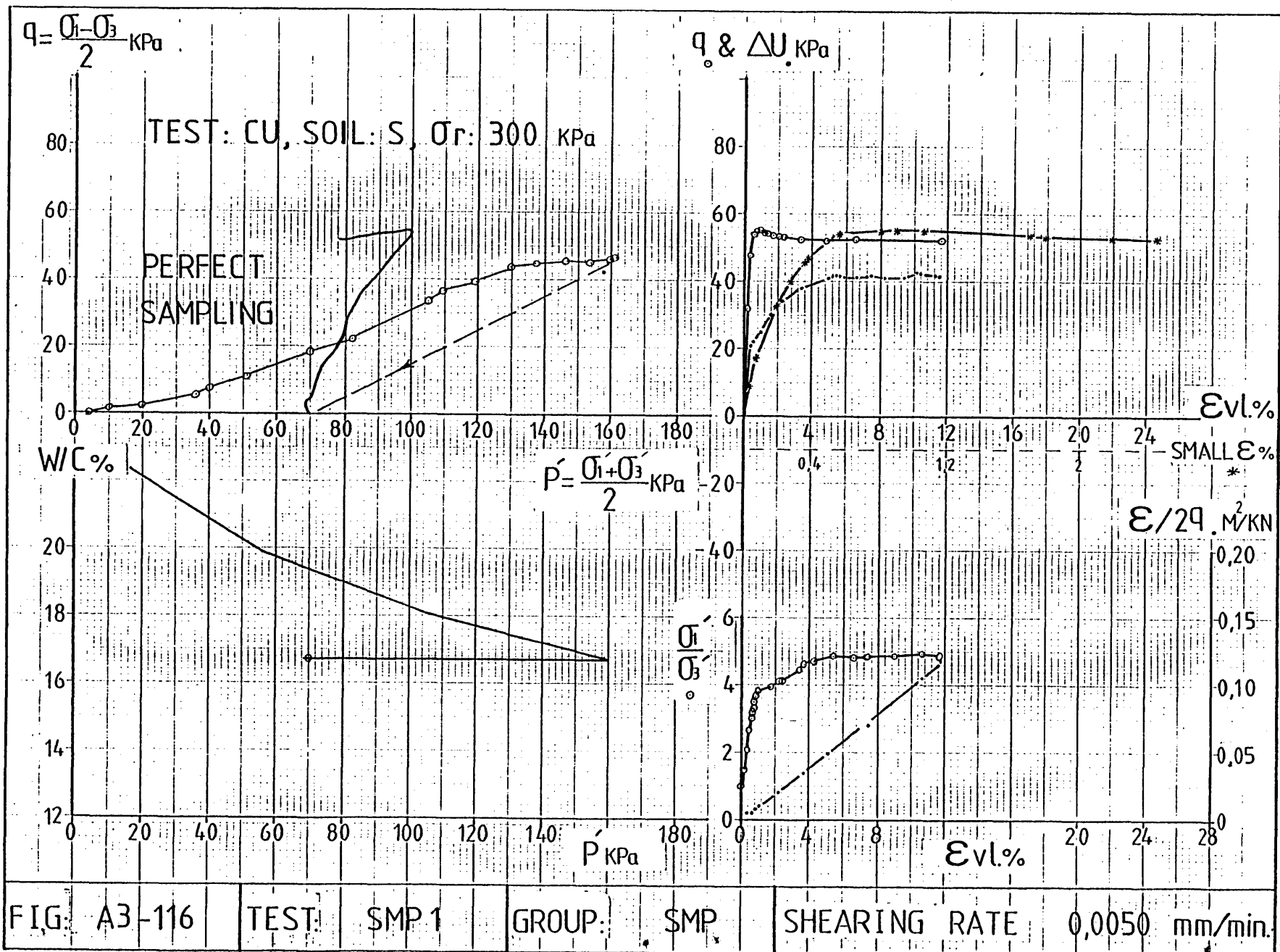
TEST: A04P40 D

GROUP: A0

SHEARING RATE: 0.0050 mm/min

A3-114





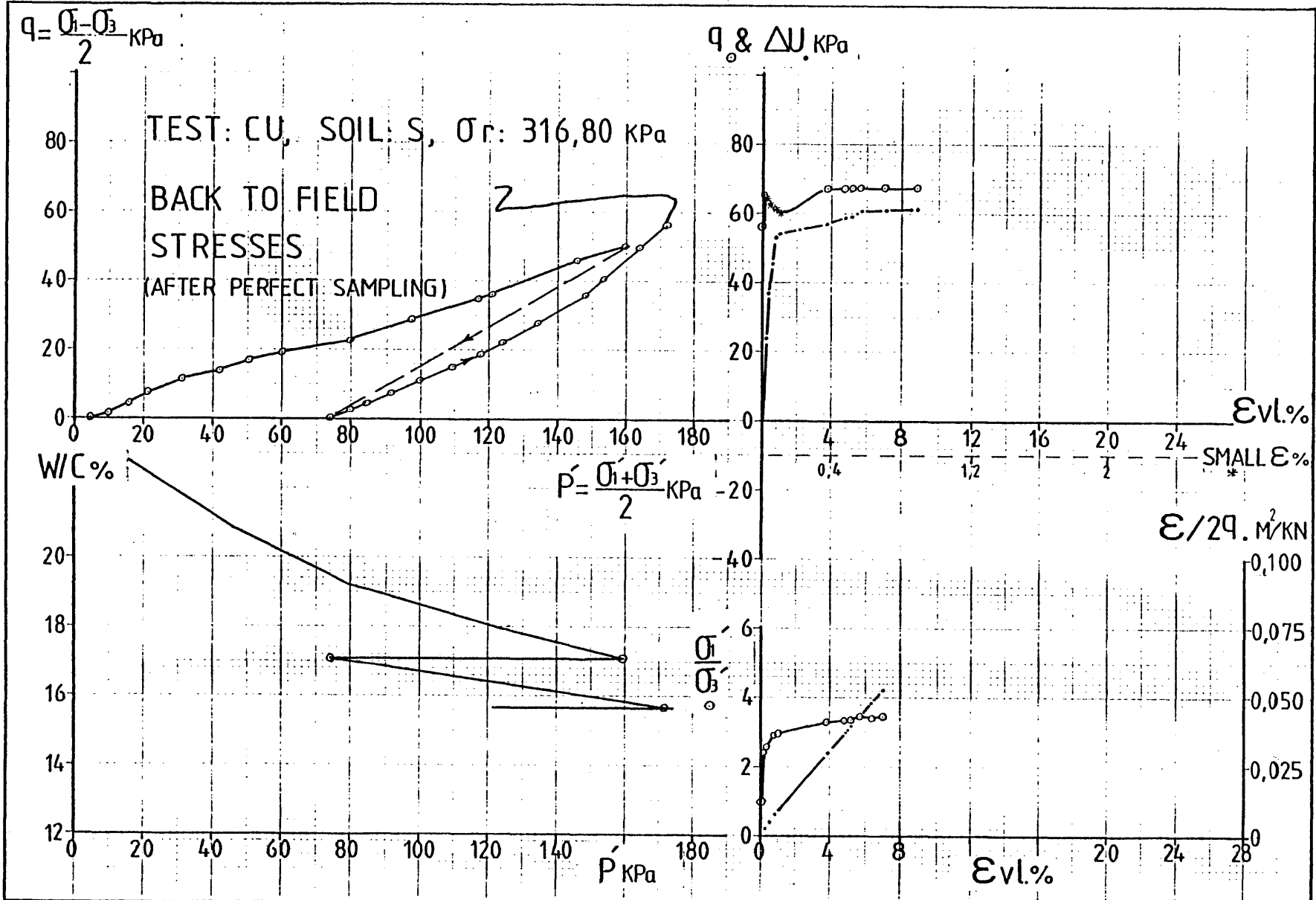
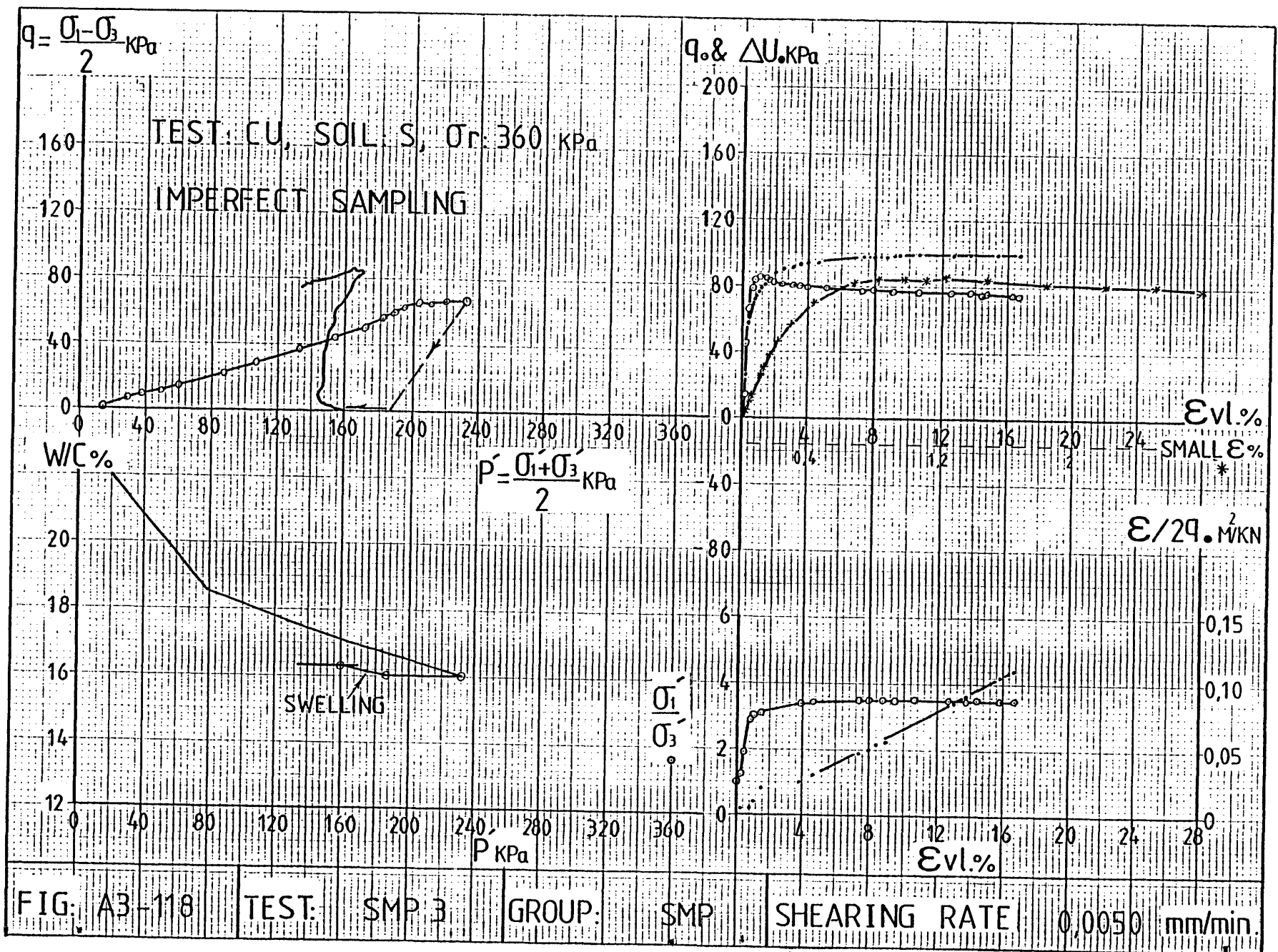
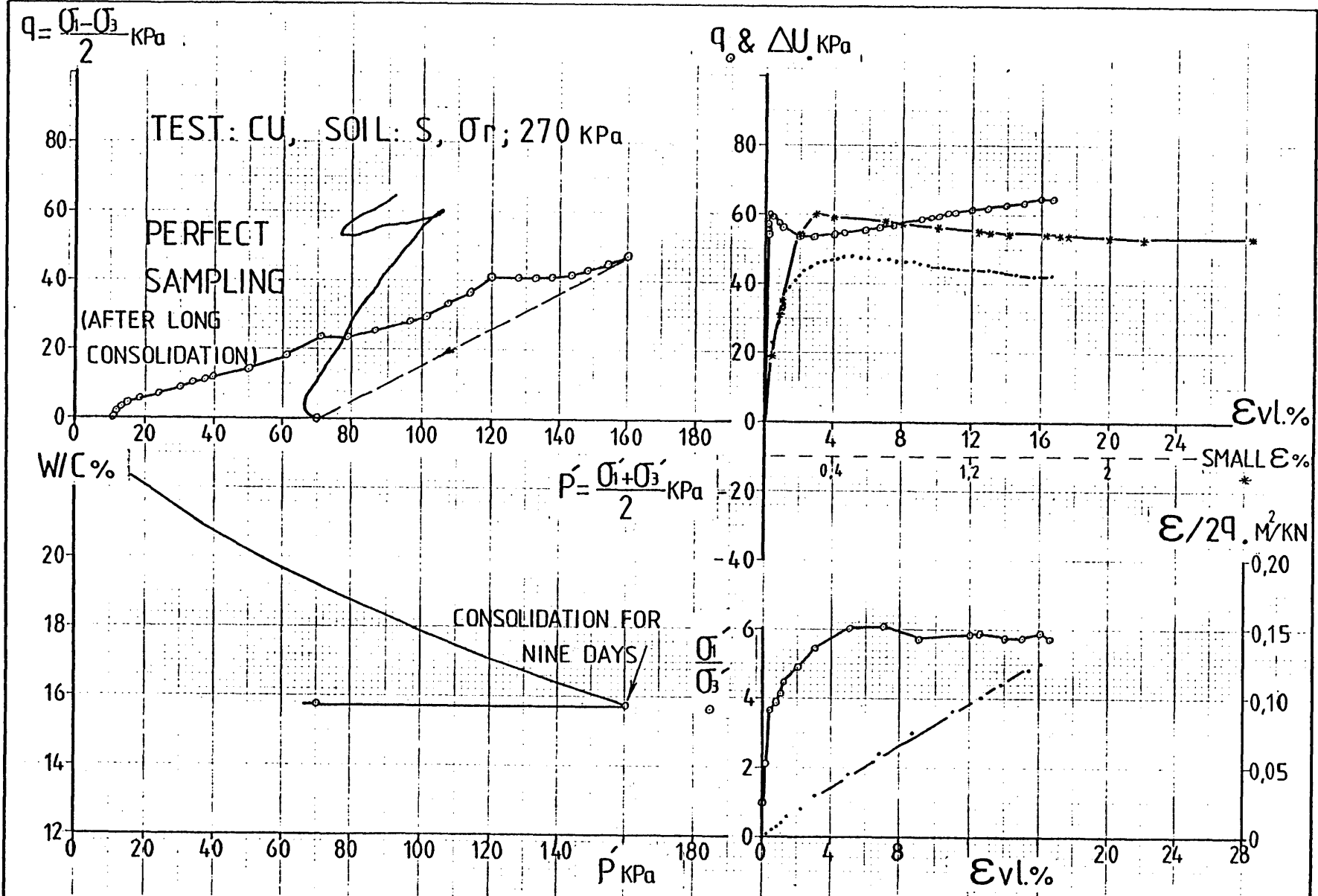
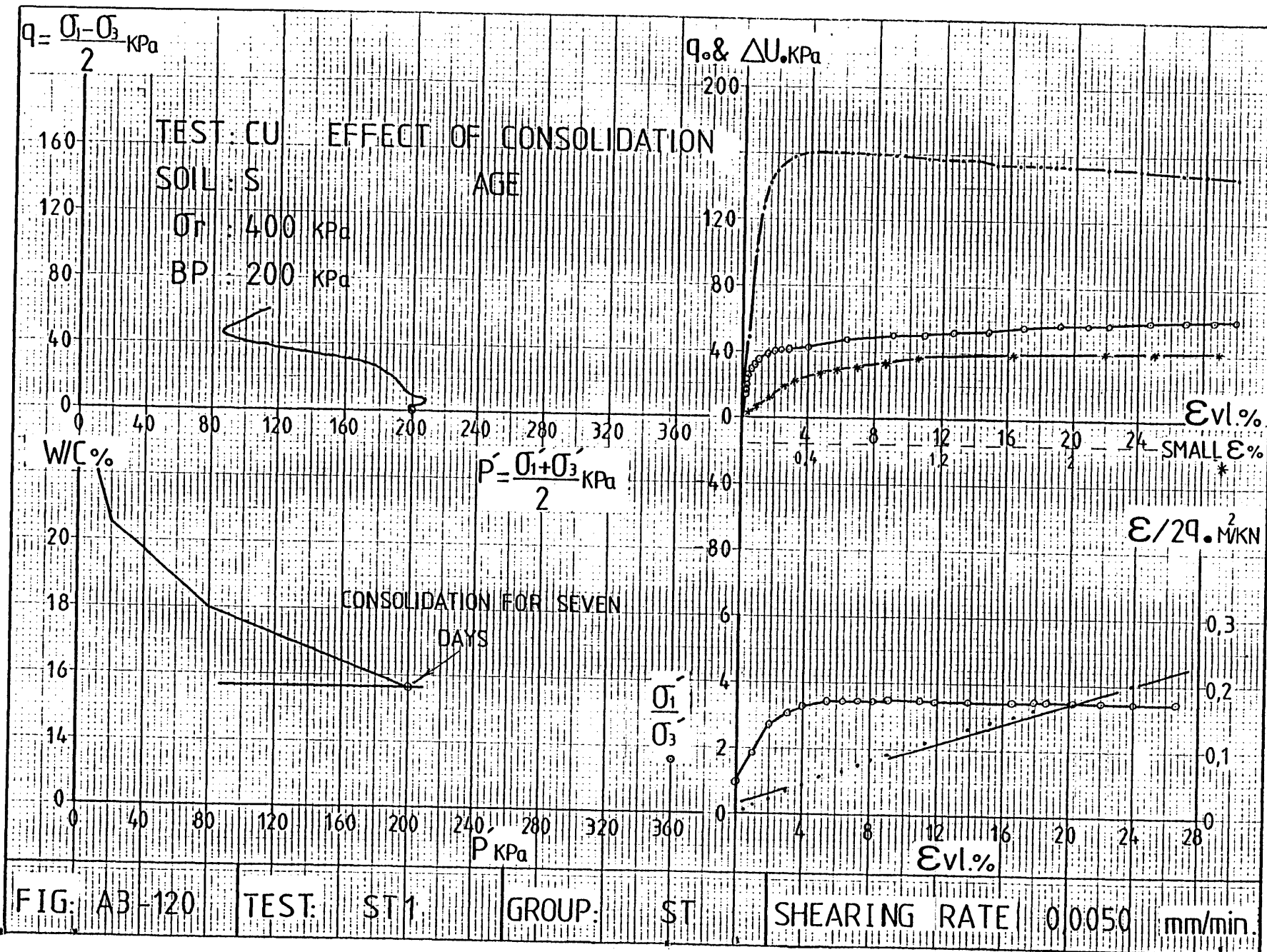
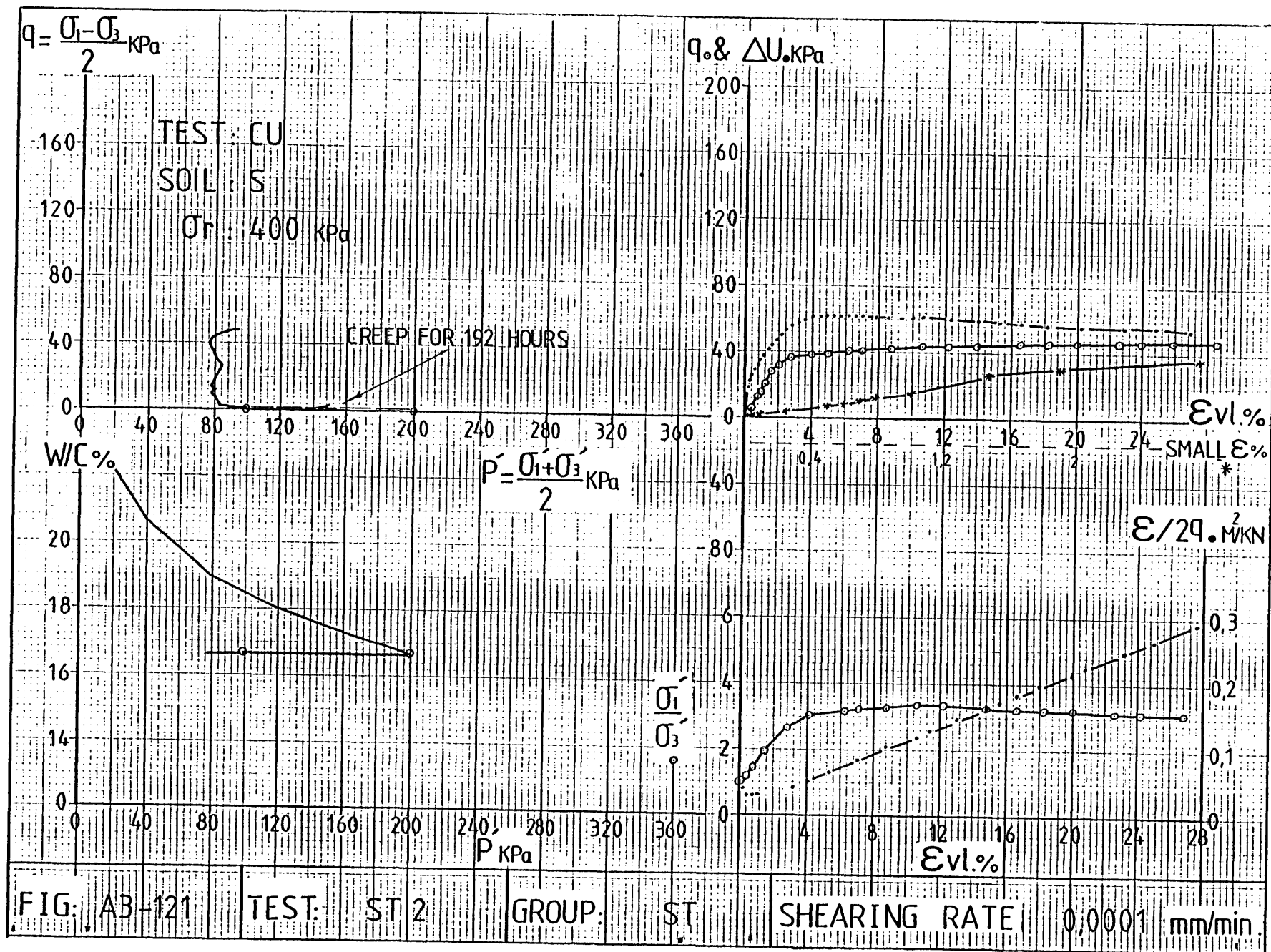


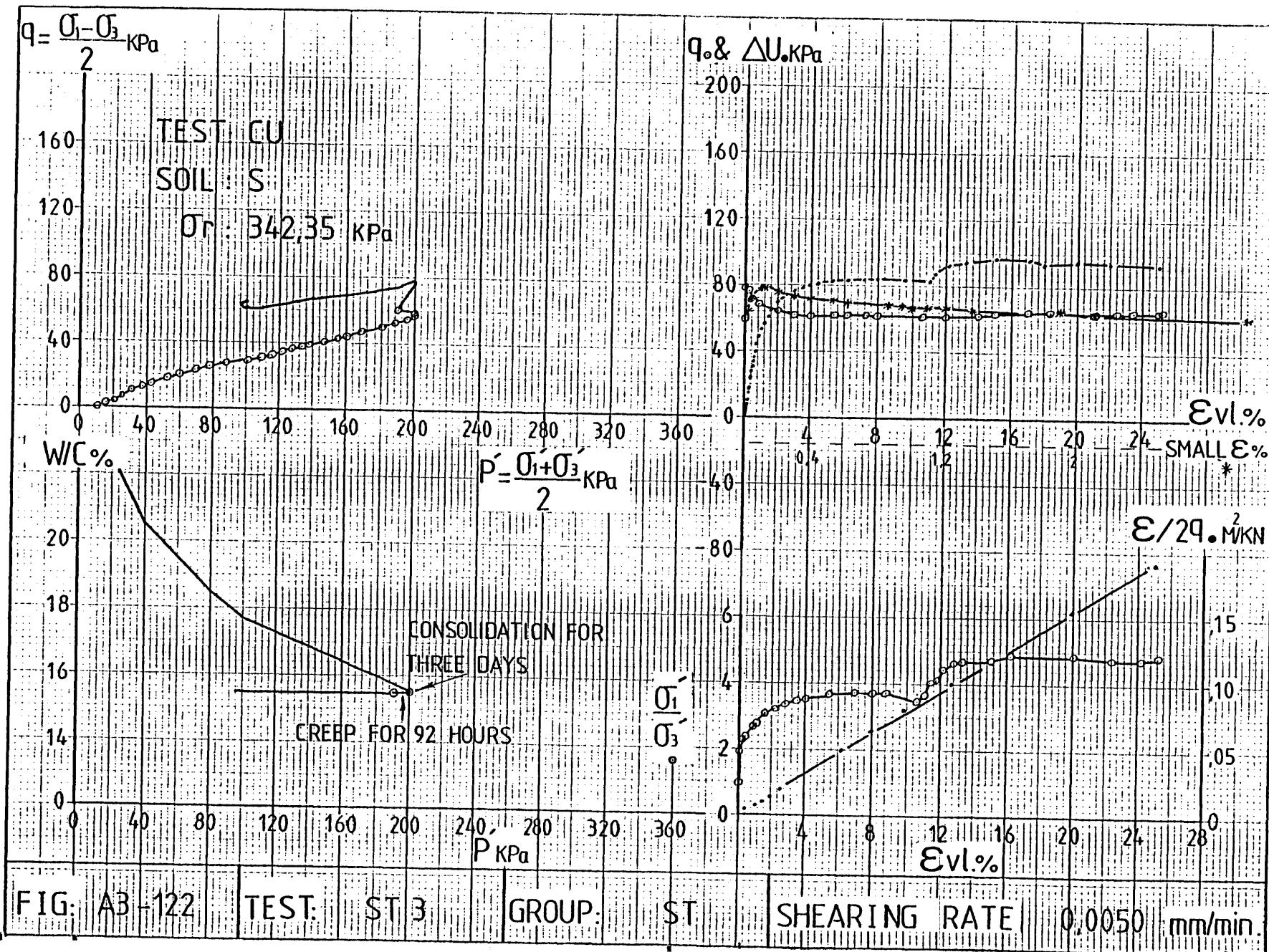
FIG: A3-117	TEST: SMP 2	GROUP: SMP	SHEARING RATE 0,0050 mm/min.
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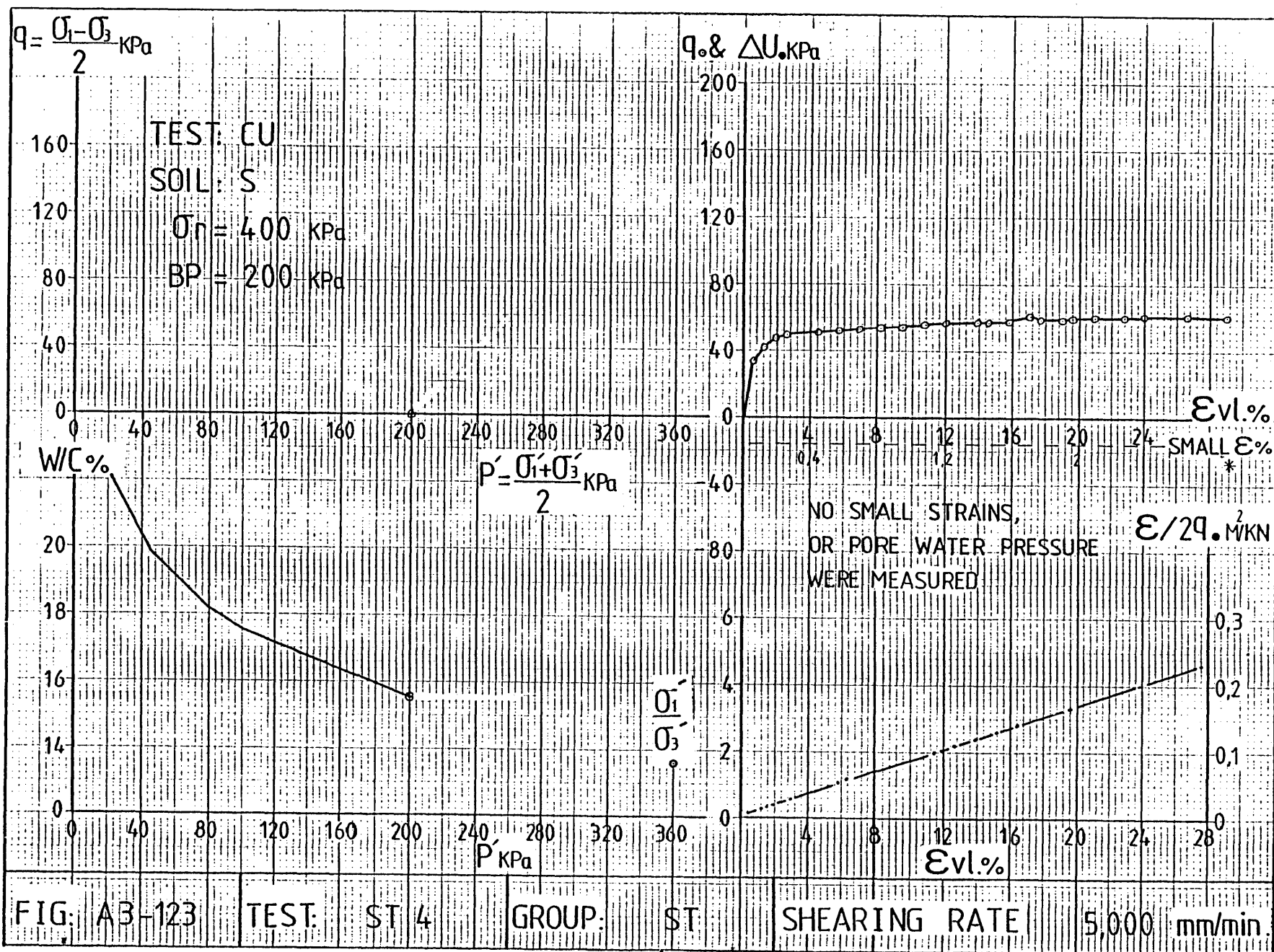


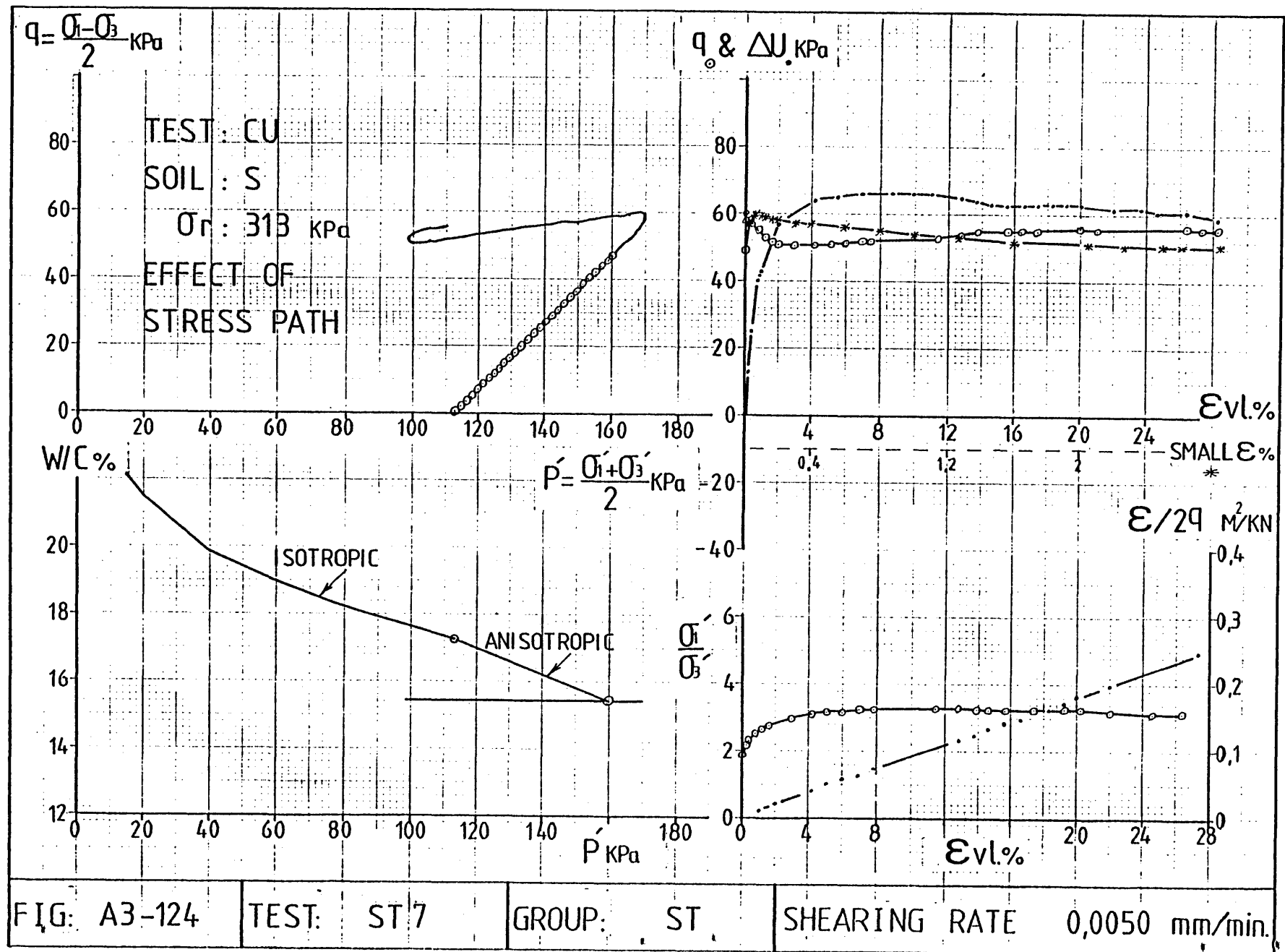












Appendix (4)

Settlements and Contact Pressures Beneath
Off-Shore Gravity Structures

- 1) Results of the analysis of rigid strip foundations,
by Vaughan (1974). See Figs 7.6 and 7.7

$$\rho_{e \max} = C_1 + C_2 \left(\frac{1}{K_2} + \frac{1}{K_1} \right) \quad (A4.1)$$

$$\rho_{e \min} = C_1 - C_2 \left(\frac{3}{K_2} + \frac{1}{K_1} \right) \quad (A4.2)$$

$$\rho_{c \max} = C_1 - C_2 \left(\frac{1}{K_2} + \frac{1}{K_1} \right) \quad (A4.3)$$

$$\rho_r = C_1 - C_2 \left(\frac{1}{K_2} + \frac{1}{K_2} \right) \quad (A4.4)$$

$$q_{e \max} = C_3 + C_2 \left(\frac{K_1}{K_2} + 1 \right) \quad (A4.5)$$

$$q_{e \min} = C_3 + C_2 \left(\frac{K_1}{K_2} - 3 \right) \quad (A4.6)$$

$$q_{c \max} = C_3 - C_2 \left(\frac{K_1}{K_2} + 1 \right) \quad (A4.7)$$

$$q_{cr} = C_3 - C_2 \left(\frac{K_1}{K_2} + 1 \right) \quad (A4.8)$$

$$q_{er} = C_3 + C_2 \left(\frac{K_1}{K_2} - 1 \right) \quad (A4.9)$$

where,

$$C_1 = V/B \cdot K_1 \quad (A4.10)$$

$$C_2 = 3M/B^2 \quad (A4.11)$$

$$C_3 = V/B \quad (A4.12)$$

In the limiting case of $K_2 \rightarrow \infty$, equations (A4.1-A4.9) become

$$\rho_{e \max} = C_1 + C_2 \frac{1}{K_1} \quad (\text{A4.13})$$

$$\rho_{e \min} = C_1 - C_2 \frac{1}{K_1} \quad (\text{A4.13})$$

$$\rho_{c \max} = C_1 - C_2 \frac{1}{K_1} \quad (\text{A4.15})$$

$$\rho_r = C_1 - C_2 \frac{1}{K_1} \quad (\text{A4.16})$$

$$q_{e \max} = C_3 + C_2 \quad (\text{A4.17})$$

$$q_{e \min} = C_3 - 3C_2 \quad (\text{A4.18})$$

$$q_{c \max} = C_3 - C_2 \quad (\text{A4.19})$$

$$q_{cr} = C_3 - C_2 \quad (\text{A4.20})$$

$$q_{er} = C_3 - C_2 \quad (\text{A4.21})$$

2) Analysis of a Rigid Circular Foundation (Waves from All Directions). Figs 7.8 and 7.9

The equation of the cone of maximum depth is,

$$\rho_m = \rho_c + (\rho_{e \max} - \rho_c) \frac{r}{r_o} \quad (\text{A4.22})$$

The equation of the plane of settlement is,

$$\rho = \rho_c + (\rho_{e \max} - \rho_c) \frac{r \cos \theta}{r_o} \quad (\text{A4.23})$$

The general equation of contact pressure during wave action is,

$$q = \rho_m \cdot K_1 - (\rho_m - \rho) K_2 \quad (A4.24)$$

$$\therefore q = \rho_c \cdot K_1 + \frac{r}{r_o} C_4 + \frac{r \cos \theta}{r_o} C_5 \quad (A4.25)$$

where

$$C_4 = (K_1 \cdot \rho_{e \max} - K_1 \cdot \rho_c - K_2 \rho_{e \max} + K_2 \rho_c) \quad (A4.26)$$

$$C_5 = (K_2 \rho_{e \max} - K_2 \rho_c) \quad (A4.27)$$

The general equation of contact pressure "at rest" is,

$$q = \rho_c \cdot K_1 + \frac{r}{r_o} C_4 \quad (A4.28)$$

The vertical load V and the moment M could be expressed as,

$$V = \int_0^{r_o} \int_0^{2\pi} q \cdot r \cdot d\theta dr \quad (A4.29)$$

$$\therefore V = \pi r_o^2 \rho_c K_1 + \frac{2\pi}{3} r_o^2 C_4 \quad (A4.30)$$

and

$$M = \int_0^{r_o} \int_0^{2\pi} q \cdot r^2 \cos \theta d\theta dr \quad (A4.31)$$

$$\therefore M = \frac{\pi r_o^3}{4} (\rho_{e \max} - \rho_c) K_2 \quad (A4.32)$$

$$\therefore \rho_c = \frac{V}{\pi r_o^2} \cdot \frac{1}{K_1} - \frac{8M}{3\pi r_o^3} \left(\frac{1}{K_2} - \frac{1}{K_1} \right) \quad (A4.33)$$

and

$$\rho_{e \max} = \frac{V}{\pi r_o^2} \cdot \frac{1}{K_1} + \frac{4M}{3\pi r_o^3} \left(\frac{1}{K_2} + \frac{2}{K_1} \right) \quad (A4.34)$$

and from linearity, Fig. 7.9b,

$$\rho_{e \min} = \frac{V}{\pi r_o^2} \cdot \frac{1}{K_1} + \frac{4M}{3\pi r_o^3} \left(\frac{2}{K_1} - \frac{5}{K_2} \right) \quad (A4.35)$$

The contact pressure "at rest" is,

$$q_c = \rho_c \cdot K_1 \quad (A4.36)$$

$$\therefore q_c = \frac{V}{\pi r_o^2} - \frac{8M}{3\pi r_o^3} \left(\frac{K_1}{K_2} - 1 \right) \quad (A4.37)$$

$$q_e = \rho_{e \max} K_1 - (\rho_{e \max} - \rho_c) K_2 \quad (A4.38)$$

$$\therefore q_e = \frac{V}{\pi r_o^2} - \frac{4M}{3\pi r_o^3} \left(1 - \frac{K_1}{K_2} \right) \quad (A4.39)$$

The contact pressure during wave action is,

$$q_{e \max} = \rho_{e \max} \cdot K_1 \quad (A4.40)$$

$$\therefore q_{e \max} = \frac{V}{\pi r_o^2} + \frac{4M}{3\pi r_o^3} \left(2 + \frac{K_1}{K_2} \right) \quad (A4.41)$$

and

$$q_{e \min} = \rho_{e \min} \cdot K_1 - (\rho_{e \max} - \rho_{e \min}) K_2 \quad (A4.42)$$

$$\therefore q_{e \min} = \frac{V}{\pi r_o^2} - \frac{4M}{3\pi r_o^3} \left(4 - \frac{K_1}{K_2} \right) \quad (A4.43)$$

In the limiting case of $k_2 \rightarrow \infty$, the above equations become,

$$\rho_c = C_6 \frac{1}{K_1} + 8C_7 \frac{1}{K_1} \quad (A4.44)$$

$$\rho_{e \max} = C_6 \frac{1}{K_1} + 8C_7 \frac{1}{K_1} \quad (A4.45)$$

$$\rho_{e \min} = C_6 \frac{1}{K_1} + 8C_7 \frac{1}{K_1} \quad (A4.46)$$

$$q_c = C_6 + 8C_7 \quad (A4.47)$$

$$q_{e \max} = C_6 + 8C_7 \quad (A4.48)$$

$$q_{e \min} = C_6 - 16C_7 \quad (A4.49)$$

where

$$C_6 = V/\pi r_o^2 \quad (A4.50)$$

$$C_7 = M/3\pi r_o^3 \quad (A4.51)$$

3) Analysis of a Rigid Circular Foundation (Waves from One Direction) Fig. 7.10

The equation of planes of maximum previous settlement is,

$$\rho_m = \rho_c + (\rho_{e \max} - \rho_c) \frac{r \cos \theta}{r_o} \quad (A4.52)$$

The equation of plane of settlement at compression side is,

$$\rho_{\text{comp}} = \rho_c + (\rho_{e \max} - \rho_c) \frac{r \cos \theta}{r_o} \quad (A4.53)$$

The equation of plane of settlement at tension side is,

$$\rho_{\text{ten.}} = \rho_c - (\rho_{e \max} - \rho_c) \frac{r \cos \theta}{r_o} \quad (A4.54)$$

The equations of contact pressure during wave action are;

$$q_{\text{comp}} = \rho_m \cdot K_1 - (\rho_m - \rho_{\text{comp}}) K_2 \quad (\text{A4.55})$$

$$\therefore q_{\text{comp}} = \rho_c \cdot K_1 - \frac{r \cos \theta}{r_o} (\rho_{e \text{ max}} - \rho_c) K_1 \quad (\text{A4.56})$$

and

$$q_{\text{ten.}} = \rho_m \cdot K_1 - (\rho_m - \rho_{\text{ten.}}) K_2 \quad (\text{A4.57})$$

$$\therefore q_{\text{ten.}} = \rho_c \cdot K_1 + \frac{r \cos \theta}{r_o} (\rho_{e \text{ max}} - \rho_c) (K_1 - 2K_2) \quad (\text{A4.58})$$

The vertical load V and the moment M will be the algebraic sum of:

$$V = \int_0^{r_o} \int_{\pi/2}^{3\pi/2} q_{\text{comp.}} \cdot r \cdot dr d\theta + \int_0^{r_o} \int_{3\pi/2}^{\pi/2} q_{\text{tens.}} \cdot r \cdot dr d\theta \quad (\text{A4.59})$$

$$\therefore V = \pi r_o^2 \rho_c K_1 + \frac{4r_o^2}{3} (\rho_{e \text{ max}} - \rho_c) (K_1 - K_2) \quad (\text{A4.60})$$

and

$$M = \int_0^{r_o} \int_{\pi/2}^{3\pi/2} q_{\text{comp.}} \cdot r^2 \cos \theta \cdot dr d\theta + \int_0^{r_o} \int_{3\pi/2}^{\pi/2} q_{\text{ten.}} \cdot r^2 \cos \theta \cdot dr d\theta \quad (\text{A4.61})$$

$$\therefore M = \frac{\pi r_o^3}{4} (\rho_{e \text{ max}} - \rho_c) K_2 \quad (\text{A4.62})$$

$$\therefore \rho_c = \frac{V}{\pi r_o^2 K_1} - \frac{4M}{3\pi r_o^3 K_1} \left(1.28 \frac{K_1}{K_2} - 1.28 \right) \quad (\text{A4.63})$$

and

$$\rho_{e \text{ max}} = \frac{V}{\pi r_o^2 K_1} + \frac{4M}{3\pi r_o^3 K_1} \left(1.73 \frac{K_1}{K_2} + 1.28 \right) \quad (\text{A4.64})$$

From linearity,

$$\rho_{e \min} = \frac{V}{\pi r_o^2 K_1} - \frac{4M}{3\pi r_o^3 K_1} \left(4.29 \frac{K_1}{K_2} - 1.28\right) \quad (A4.65)$$

The contact pressure "at rest" is,

$$q_c = \rho_c \cdot K_1$$

$$\therefore q_c = \frac{V}{\pi r_o^2} - \frac{4M}{3\pi r_o^3} \left(1.28 \frac{K_1}{K_2} - 1.28\right) \quad (A4.66)$$

and

$$q_{er} = \rho_{e \max} K_1 - (\rho_{e \max} - \rho_c) K_2 \quad (A4.67)$$

$$q_{er} = \frac{V}{\pi r_o^2} + \frac{4M}{3\pi r_o^3} \left(1.73 \frac{K_1}{K_2} - 1.73\right) \quad (A4.68)$$

The contact pressures during wave action are,

$$q_{e \max} = \rho_{e \max} K_1 \quad (A4.69)$$

$$\therefore q_{e \max} = \frac{V}{\pi r_o^2} + \frac{4M}{3\pi r_o^3} \left(1.73 \frac{K_1}{K_2} + 1.28\right) \quad (A4.70)$$

and

$$q_{e \min} = \rho_{e \max} \cdot K_1 - (\rho_{e \max} - \rho_{e \min}) K_2 \quad (A4.71)$$

$$\therefore q_{e \min} = \frac{V}{\pi r_o^2} + \frac{4M}{3\pi r_o^3} \left(1.73 \frac{K_1}{K_2} - 4.75\right) \quad (A4.72)$$

In the limiting case of $K_2 \rightarrow \infty$ the above equations become,

$$\rho_c = C_6 \frac{1}{K_1} + C_7 \frac{5.18}{K_1} \quad (A4.73)$$

$$\rho_{e \max} = C_6 \frac{1}{K_1} + C_7 \frac{5.18}{K_1} \quad (A4.74)$$

$$\rho_{e \min} = C_6 \frac{1}{K_1} + C_7 \frac{5.18}{K_1} \quad (A4.75)$$

$$q_c = C_6 + 5.18 C_7 \quad (A4.76)$$

$$q_{er} = C_6 - 6.92 C_7 \quad (A4.77)$$

$$q_{e \max} = C_6 + 5.18 C_7 \quad (A4.78)$$

$$q_{e \min} = C_6 - 19 C_7 \quad (A4.79)$$

REFERENCES

The following abbreviations are used herein.

ASCE	American Society of Civil Engineering.
ASTM	American Society of Testing of Materials.
BOSS	Behaviour Of Off-Shore Structures.
C.G.J.	Canadian Geotechnical Journal.
Engn.	Engineering.
GED	Geotechnical Engineering Division.
Geot.	Geotechnique.
ICSMFE	International Conference of Soil Mechanics and Foundation Engineering.
J1	Journal.
MIT	Massachussets Institute of Technology.
NGI	Norwegian Geotechnical Institute.
Proc.	Proceedings (of)
QJEG	Quarterly Journal of Engineering Geology.
S & F	Soils and Foundations
SMFD	Soil Mechanics and Foundation Division.
SMFE	Soil Mechanics and Foundation Engineering.
STP	Special Technical Publication.

References

- Aas, G. (1965). A study of the effect of vane shape and rate of strain on the measured values of in-situe shear strength of soils. Proc. 6th ICSMFE, Montreal, 1, 141-145.
- Aas, G. (1967). Vane tests for investigation of anisotropy of undrained shear strength of clays. Proc. Geot. Conf. on Shear Strength Properties of Natural Soils and Rocks. Oslo, 1, 3-8.
- Agarwal, K.B. (1967). The influence of size and orientation of sample on the undrained strength of London Clay. Ph.D. thesis, Imperial College, Univ. of London.
- Akroyd, T.N.W. (1964). Laboratory testing in soil engineering. Geotechnical Monograph No. 1. Soil Mech. Ltd.
- Al-Dahir, Z.A. (1967). Correlation between field and laboratory measurements on earth dams. Ph.D. thesis, Imperial College, Univ. of London.
- Apted, J.P. (1977). Effects of weathering on some geotechnical properties of London Clay. Ph.D. thesis, Imperial College, Univ. of London.
- Arthur, J.R.F. and Menzies, B.K. (1972). Inherent anisotropy in sands. Geot., 22, 1, 115-128.
- Atkinson, J.H. (1973). The deformation of undisturbed London Clay. Ph.D. thesis, Imperial College, Univ. of London.
- Atkinson, J.H. (1974). Discussion. The Interpretation of Field Observations. Field Instrumentation in Geotechnical Engineering. 2, 667-679. Butterworths, London.
- Baden-Powell, D.F.W. (1948). The chalky boulder clay of Norfolk and Suffolk. Geol. Mag. 85, 279-296.
- Baden-Powell, D.F.W. and West, R.G. (1960). Summer field meeting in East Anglia. Proc. Geol. Assoc. 71, 61-80.

- Banks, J.A. (1948). Construction of Muirhead Reservoir, Scotland. Proc. 2nd ICSMFE, 2, 24-31.
- Barden, L. and McDermott, R.J.W. (1965). Use of free ends in triaxial testing of clays. JI. MSFD, ASCE, 91, SM6, Nov., 1-23.
- Barden, L. (1969). Time dependent deformation of normally consolidated clays and plates. JI. SMFD, ASCE, 95, SM1, 1-31.
- Barden, L. and Sides, G. (1970). The influence of weathering on the microstructure of Keuper Marl. Technical Note, QJEG, Vol. 3:4, 259-260.
- Bauer, C.E.A., Scott, J.D. and Shields, D.H. (1973). The deformation properties of a clay crust. 8th ICSMFE, 1, 31-68.
- Berre, T. and Bjerrum, L. (1973). Shear strength of normally consolidated clays. 8th ICSMFE, Moscow, 1, p. 39.
- Bishop, A.W. and Eldin, A.K.G. (1953). The effect of stress history on the relation between ϕ and the porosity in sands. Proc. 3rd ICSM, 1, 100-105.
- Bishop, A.W. (1958). Test requirements for measuring the coefficient of earth pressure at rest. Proc. Brussels Conf. Earth Pressure Problems, 1, 2-14.
- Bishop, A.W., Alpan, I., Blight, G.E. and Donald, I.B. (1960). Factors controlling the strength of partly saturated cohesive soils. Proc. ASCE Research Conf. on Shear Strengths of Cohesive Soils, Boulder, Colorado.
- Bishop, A.W. and Henkel, D.J. (1962). The measurement of soil properties in the triaxial test. 2nd Ed. by Edward Arnold. (Pub.) Ltd.
- Bishop, A.W. and Green, G.E. (1965). The influence of end restraint on the compression strength of a cohesionless soil. Geot., 15, 243-266.
- Bishop, A.W. (1967). Progressive Failure - with special reference to the mechanism causing it. Panel Discussion, Geotechnical Conf., Oslo, 2, 142-150.
- Bishop, A.W. and Al-Dhahir, Z.A. (1969). Some comparison between laboratory tests, in-situ tests and full scale

- performance, with special reference to permeability and coefficient of consolidation. Proc. Conf. In-Situ Investigations in Soils and Rocks, 251-264.
- Bishop, A.W. (1971). The influence of progressive failure on the choice of the method of stability analysis. Technical Note. Geot., 21, 168-172.
- Bishop, A.W., Green, G.E., Garga, V.K., Andresen, A. and Brown, J.D. (1971). A new ring shear apparatus and its application to the measurement of residual strength. Geot., 21, 273-328.
- Bishop, A.W. and Wesley, L.D. (1975). A hydraulic triaxial apparatus for stress path testing. Geot., 25, No. 4, 657:670.
- Bishop, A.W. and Hight, D.W. (1977). The value of Poisson's Ratio in saturated soils and rocks stressed under undrained conditions. Geot., 27, 3, 369-384.
- Bishop, A.W. (1978). Private Communication.
- Bishop, A.W. and Wesley, L.D. (1978). The influence of stress path and anisotropy and time on the strength of a soft alluvial clay from the Thames estuary. To be published.
- Bjerrum, L. (1954). Theoretical and experimental investigation on the shear strength of soils. NGI Pub. No. 5.
- Bjerrum, L., Simons, N. and Torblaa, I. (1958). The effect of time on the shear strength of a soft Marine Clay. Brussels Conference on Earth Pressure Problems (also NGI internal report no. 33, Oslo, 1960).
- Bjerrum, L. and Lo, K.Y. (1963). Effect of aging on the shear-strength properties of a normally consolidated clay. Geot., 13, No. 2, 147-157.
- Bjerrum, L. (1972). The effect of rate of loading on the P'_c value observed in consolidation tests on soft clay. NGI Pub. No. 95.

- Bjerrum, L. (1973). Geotechnical problems involved in the foundation of structures in the North Sea. *Geot.*, 23, No. 3, pp. 319:358.
- Blight, G.E. (1965). Shear stress and pore pressure in triaxial testing. *Jl. SMFD., ASCE*, 91, SM6, Nov. 25-39.
- Boswell, P.G.H. (1914). On the occurrence of the North Sea Drift (Lower Glacial) and certain other Brickearths in Suffolk. *Proc. Geol. Assoc.* Vol. 25, 121:152.
- Boswell, P.G.H. (1916). Petrology of the North Sea Drift and Upper Glacial Brickearths in East Anglia. *Proc. Geol. Assoc.*, 27, 79:98.
- Boswell, P.G.H. (1923). The geology of the country around Cromer and Norwich. *Proc. Geol. Assoc.* 34, 207:222.
- Buri, P.B. (1978). Influence of secondary consolidation and overconsolidation on the behaviour of a soft alluvial clay. Ph.D. thesis, Imperial College, Univ. of London.
- Cadling, L. and Odenstad, S. (1950). The vane borer. *Proc. Royal Swedish Geot. Insit.*, Pub. No. 2.
- Campanella, R.G. and Vaid, Y.P. (1974). Triaxial and plane strain creep rupture of an undisturbed clay. *C.G.J.*, 11, No. 1.
- Capper, P.L. and Cassie, W.F. (1969). The mechanics of engineering soils. A Halsted Press Book, John Wiley and Sons Inc., New York.
- Casagrande, A. (1932). Research on the Atterberg limit of soils. *Public Roads*, 13, 121-136.
- Casagrande, A. and Hirschfeld, R.C. (1962). Second progress report on investigation of stress, deformation and strength characteristics of compacted clays. *Harvard Soil Mechanics Series No. 65*.
- Chandler, R.J. (1966). The measurement of residual strength in triaxial compression. *Geot.*, 16, 181-186.

- Clausen, C.J.F. (1973). Stability problems related to off-shore gravity structures. NGI Int. report, 52406.
- Collin, K. and McGown, A. (1974). The form and function of microfabric features in a variety of natural soils. Geot., 24, No. 2.
- Conlon, R.J. and Isaacs, R.M.F. (1970). Effect of sampling and testing technique on the shear strength of a Glacial-Lacustrine Clay from Welland, Ontario. Proc. ASTM Symp. of Soils and Rocks, 1970, ASTM, STP 483, pp. 10-29.
- Crawford, C.B. (1959). The influence of rate of strain on effective stresses in sensitive clays. Symposium on (Time Rates of Loading in Soil Testing), ASTM, STP 254.
- Cummings, A.E., Kirkhoff, G.O. and Peck, R.B. (1950). Effect of driving piles into soft clay. Trans. ASCE, 115, 275.
- Daramola, O. (1978). The influence of stress-history on the deformation of a sand. Ph.D. thesis, Imperial College, Univ. of London.
- Davachi, M.M. (1978) Finite element analysis of footings and embankment dam cores. Ph.D. thesis, Imperial College, Univ. of London.
- Davis, E.H. and Poulos, H.G. (1968). The use of elastic theory in settlement prediction under three-dimensional conditions. Geot., 18, 67-91.
- Davis, E.H. and Booker, J.R. (1973). The effect of increasing strength with depth on the bearing capacity of clays. Geot, 23, No. 4, 551-563.
- Davis, P. (1975). Creep characteristics of three undisturbed clays. Ph.D. thesis, Imperial College, Univ. of London.
- De-Mello, V.F.B. (1977). Reflections on design decisions of practical significance to embankment dams. 17th Rankine Lecture, Geot., 27, 3, 279-356.
- Dhonau, T.J. and Dhonau, N.B. (1963). Glacial structures on the North Norfolk coast. Proc. Geol. Assoc. 74, 433-439.

- Duncan, J.M. and Seed, H.B. (1966). Strength variations along failure surfaces in clay. *JI. SMFD, ASCE*, 92, SM6.
- Duncan, J.M. and Chung, C.Y. (1970). Non-linear analysis of stress and strain in soils. *JI. SMFD, ASCE*, SM5, 96, 1629-1653.
- Duncan, J.M. (1972). Foundation study for the North Sea oil tank. *NGI Publication No. 92*, 13-15.
- Egeli, I. (1978). Ph.D. thesis (in preparation) Imperial College, Univ. of London.
- Eide, D. (1974). Marine soil mechanics, applications to North Sea off-shore structures. *NGI pub. 109*, 1-20.
- El-Ruwayih, A.A. (1975). Stress-strain characteristics of rockfall and of clays under high pure water tension. Ph.D. thesis, Imperial College, Univ. of London.
- Flaate, K. (1966). Factors influencing the results of vane tests. *C.G.J.*, 3, 1, 18:31.
- Frimann Clausen, C.J. (1973). See under "C".
- Frydman, S. and Zeitler, J.G. (1969). Some pseudo-elastic properties of granular media. *Proc. 7th ICSMFE, Mexico*, 1, 135-141.
- Garga, V.A. (1970). Residual strength under large strains and the effect of sample size on the consolidation of fissured clay. Ph.D. thesis, Imperial College, Univ. of London.
- Gens, A. (1978). Ph.D. thesis (in preparation) Imperial College, Univ. of London.
- Gibson, R.E. (1967). Some results concerning displacements and stresses in a non-homogeneous elastic half space. *Geot.*, 17, 58-67.
- Gibson, R.E. (1974). The analytical methods in soil mechanics. 14th Rankine Lecture, *Geot.*, 24, 2, 115-140.
- Green, G.E. (1969). Strength of granular materials under generalised strain conditions. Ph.D. thesis, Imperial College, Univ. of London.

- Green, G.E. (1971). Discussion, Roscoe Memorial Symp., Cambridge, 401:407.
- Hamza, M.M.A.F. (1976). The analysis of embankment dams by non-linear finite element method. Ph.D. thesis, Imperial College, Univ. of London.
- Hansen, J.B. and Gibson, R.E. (1948). Undrained shear strength of anisotropically consolidated clays: Geot., 1, 189-204.
- Hansen, J. Brinch (1961). A general formula for bearing capacity. Danish Geotechnical Institute, Bulletin No. 11.
- Hansen, J. Brinch (1970). A revised and extended formula for bearing capacity. Danish Geotechnical Institute, Bulletin No. 28, 1970 (Reprint from lecture in Japan, 1968).
- Harris, E.W. (1966). Recent developments in the design of soil testing equipment. D.I.C. Dissertation.
- Henkel, D.J. and Gilbert, G.D. (1952). The effect of rubber membrane on the measured triaxial compression strength of clay samples. Geot., 3, 20-29.
- Henkel, D.J. (1960). The relationship between the effective stresses and water content in saturated clays. Geot., 10, pp. 41.
- Henkel, D.J. (1960). The shear strength of saturated remoulded clays. ASCE Spec. Conf. on Shear Strength of Cohesive Soils, Univ. of Colorado, Boulder, Colorado, June 13-17, 533-554.
- Henkel, D.J. and Souva, V.A. (1963). Discussion. Proc. Symp. Lab. Shear Testing of Soils, ASTM, Speciality Pub. 361.
- Henkel, D.J. (1970). The role of waves in causing submarine landslides. Geot., 20, 1, 75-80.
- Herbertson, R.P. (1977). A model test facility for soil-structure interaction. M.Phil thesis, Imperial College, Univ. of London.

- Hight, D.W. (1978). Ph.D. thesis (in preparation) Imperial College, Univ. of London.
- Hogben, N. (1976). Wave loads on structures. Proc. BOSS 1976, Conf. 1, 187-217.
- Hutchinson, J.N. (1976). Coastal landslides in cliffs of pleistocene deposits between Cromer and Overstrand, Norfolk, England. Laurits Bjerrem Memorial Volume, N.G.I., Oslo, Norway.
- Hvorslev, M.J. (1949). Sub-surface exploration and sampling of soils for civil engineering purposes. U.S. Waterways Experiment Station, Vicksburg, Miss.
- Hvorslev, M.J. (1960). Physical components of the shear strength of saturated clays. Proc. Research Conference on Shear Strength of Cohesive Soils. ASCE, Boulder, Colorado, 169-273.
- Jaky, J. (1948). The coefficient of earth pressure at rest. Proc. 2nd ICSMFE, 1, 103-107.
- Janbu, N., Grande, L. and Eggereide, K. (1976). Effective stress stability analysis for gravity structures. Proc. Conference BOSS 1976, 1, 449-467.
- Kazi, A. and Knill, J.L. (1969). The sedimentation and geotechnical properties of the Cromer Till between Happisburgh and Cromer, Norfolk. QJEG, 2, no. 1, 63:87.
- Kelly, W.E. (1978). Correcting shear strength for temperature. Tech. Note, JI. GED, ASCE, 104, GT5, 664-667.
- Kenny, T.C. (1959). Discussion on Crawford's 1959 paper, ASTM Symposium (Time Rates of Loading) STP 254.
- Kiekbusch, M. and Schuppener, B. (1977). Membrane penetration and its effect on pore pressures. JI. GED, ASCE, 103, GT11.
- Kondner, R.L. (1963). Hyperbolic stress-strain response: Cohesive Soils. JI. SMFD. ASCE, SM1 Feb. 63.
- Kondner, R.L. and Zelasko, J.S. (1963). A hyperbolic stress-strain formulation for sands. 2nd Pan. Am. Conf. in S.M.

- Kulhawy, F.H., Duncan, J.M. and Seed, H.M. (1969). Finite element analysis of stresses and movements in embankments during construction. Report No. TE-69-4, Dept. Civil Eng., Instit. of Trans. and Traf. Eng., Univ. of California, Berkeley.
- Ladd, C.C. and Lambe, T.W. (1963). The strength of undisturbed clay determined from undrained tests. Laboratory Shear Testing of Soils, ASTM, SPT No. 361.
- Ladd, C.C. (1964). Stress-strain modulus of clay in undrained shear. JI. SMFD, ASCE, 90, SM5.
- Ladd, C.C. (1965). Stress-strain behaviour of anisotropically consolidated clays during undrained shear. 6th ICSMFE, 1, 282-287.
- Ladd, C. and Foott, R. (1974). New design procedure for stability of soft clays. JI. GED, ASCE, GT7, July 763-785.
- Ladd, C., Foott, R., Ishihara, K., Poulos, H. and Schlosser, F. (1977). Stress-deformation and strength characteristics. State-of-the-Art. Report 9th ICSMFE, Tokyo.
- Lambe, T.W. (1951). Soil testing for engineers. John Wiley and Sons Inc. New York.
- Lambe, T.W. (1953). The structure of inorganic soils. Proc. ASCE, Separate No. 315.
- Lambe, T.W. (1958a). The structure of compacted clay. JI. SMFD, ASCE, 84, SM2, paper 1654.
- Lambe, T.W. (1958b). The engineering behaviour of compacted clay. JI. SMFD, ASCE, 84, SM2, paper 1655.
- Lambe, T.W. (1962). Pore pressure in a foundation clay. JI. SMFD, ASCE, 88, SM2, 19-47.
- Lambe, T.W. and Whitman, R.V. (1969). Soil Mechanics. John Wiley and Son Inc. New York.
- Lang, J.G. (1967). Longitudinal variation of soil disturbance within tube samples. Proc. 5th Australia-New Zealand Conf. SMFE, Melbourne, 1, 39-42.

- Lang, J.G. (1970). Reduced soil strength and stiffness at the top of tube samples. Proc. 5th Australia-New Zealand Conf. SMFE, Melbourne, 1, 232-237.
- Leonards, G.A. and Ramiah, B.K. (1959). Time effects on consolidation of clays. ASTM, STP 254.
- Lewin, P.I. (1970). Stress-deformation characteristics of a saturated soil. M.Sc. thesis, Imperial College, Univ. of London.
- Lo, K.Y., Seychuk, J.L. and Adams, J.I. (1970). A study of the deformation characteristics of a Stiff Fissured Clay. Sampling of Soil and Rock, ASTM, STP 483, 60:76.
- Lo, K.Y. (1965). Stability of slopes in anisotropic soils. JI. SMFD, ASCE, 91, SM4, 85-106.
- Løken, T. (1976). Geology of superficial sediments in the Northern North Sea. BOSS 76 Conference, 1, 501:515 Trondheim, Norway.
- Lopez, F.R. (1976). Ph.D. thesis (in preparation) Imperial College, Univ. of London.
- Lovenbury, H.T. (1969). Creep characteristics of London Clay. Ph.D. thesis, Imperial College, Univ. of London.
- Lowe, J. III, Jonas, E. and Obrician, V. (1969). Controlled gradient consolidation test. JI. SMFD, ASCE, 95, SM1.
- Lupini, F.J. (1978). Ph.D. thesis (in preparation) Imperial College, Univ. of London.
- McGown, A., Saldivar-Sali, A. and Radwan, A.M. (1974). Fissure patterns and slope failures in till at Hurlford, Ayrshire. QJEG, 7, 1, 1-26.
- Maguire, W.M. (1975). The undrained strength and stress-strain behaviour of Brecciated Upper Lias lay, Ph.D. thesis, Imperial College, Univ. of London.
- Marsland, A. (1971). Laboratory and in-situ measurements of the deformation moduli of London Clay. Proc. Symp.

- Interaction of Structures and Foundations, Midland Soil Mechanics and Foundation Engineering Society, Dep. Civil Eng., Univ. of Birmingham, 7-17.
- Martins, M.C.R. (1977). Large model footing tests: a first interpretation. M.Sc. thesis, Imperial College, Univ. of London.
- Meyerhof, G.G. (1953). The bearing capacity of foundations under eccentric and inclined loads. Proc. 3rd ICSMFE, 1, Zurich.
- Meyerhof, G.G. (1963). Some recent research on the bearing capacity of foundations. C.G.J., 1, No. 1, Sept. 16-26.
- Mitchell, J.M. and Gardner, W.S. (1971). Analysis of load bearing fills over soft subsoil. JI. SMFED, ASCE, 97, SM11, 1549-1571.
- Mitchell, J.K. (1976). Fundamentals of soil behaviour. John Wiley and Sons Ltd.
- Oda, M. (1972). Initial fabric and their relations to mechanical properties of granular material. S and F, 12, No. 2, 1-18.
- Olson, R.E. (1963). Shear strength properties of Sodium Illite. JI. SMFD, ASCE, SM1, 183:208.
- Parry, R.H.G. (1960). Triaxial compression and extension tests on remoulded saturated clay. Geot., 10, 166-180.
- Parry, R.H.G. (1971). Undrained shear strengths in clay. Proc. 1st Australian-New Zealand Conference on Geomechanics, 1, 11-15.
- Parry, R.H.G. and Nadarajah, V. (1974). Observations on laboratory prepared lightly overconsolidated specimens of Kaolin, Geot., 24, No. 3, 345-358.
- Perloff, W.H. and Osterberg, J.O. (1963). The effect of strain rate on the undrained shear strength of cohesive soils. 2nd Pan. Am. Conf. on SM, 1, 103:128.
- Pugh, R.S. (1978). The strength and deformation characteristics

- of a soft alluvial clay under full scale loading conditions. Ph.D. thesis, Imperial College, Univ. of London.
- Pusch, R. (1973). Structural variation in boulder clay. Proc. The International Symposium on Soil Structures, Gothenburg, Sweden, 113-121.
- Raymond, G.P., Townsend, D.L. and Lojkasek, M.J. (1971). The effect of sampling on the undrained soil properties of a Leda soil. C.G.J., 8, 546-557.
- Reades, D.W. (1972). Stress-strain characteristics of a sand under three-dimensional loading. Ph.D. thesis, Imperial College, Univ. of London.
- Rendulic, L. (1936a). Porenziffer und porewasserdruck (pore index and pore-water pressure in clay). Bauingenieur, 17, 559.
- Rendulic, L. (1936b). Discussion. Relation between void void ratio and effective principal stresses for a remoulded silty clay. Proc. 1st ICSMFE, 3, 48-51.
- Rendulic, L. (1937). Ein grundgesetz der tonmechnaik und sein experimenteller beweis (the fundamental law of clay mechanics and the experimental proof). Bauingenieur, 18, 459-467.
- Richardson Jr., A.M. and Whitman, R.T. (1963). Effect of strain rate upon undrained shear resistance of a saturated remoulded fat clay. Geot. 13, No. 4, 310:324.
- Rowe, P.W. and Barden, L. (1964). Importance of free ends in triaxial testing. JI. SMFD, ASCE, 90, SM1, 1-27.
- Rowe, P.W. (1970). Representative sampling in location, quality and size. Sampling of Soil and Rock, ASTM, STP 483, 77:106.
- Sandroni, S.S. (1977). The strength of London Clay in total and effective stress terms. Ph.D. thesis, Imperial College, Univ. of London.

- Sano, Y. (1977). A report on the rate effect in Happisburgh Lower Till. Soils Internal Report 2/77 Imperial College, London.
- Seed, H.B. and Reese, L.B. (1955). Action of soft clay along friction piles. Proc., ASCE, 81, No. 842.
- Simons, N.E. (1960). Comprehensive investigation of the shear strength of an undisturbed Drammen Clay. ASCE, Research Conference on Shear Strength of cohesive soils, Boulder, Colorado. 727-745.
- Simons, N.E. (1963). The influence of stress path on tri-axial test results. Proc. Symp. on Laboratory Shear Testing of Soils, Ottawa, ASTM, STP 361, 270-278.
- Simons, N.E. and Som, N.N. (1969). The influence of lateral stresses on the stress-deformation characteristics of London Clay. Proc. 7th ICSMFE, Mexico, 1, 369-377.
- Simons, N.E. (1974). Normally consolidated and lightly overconsolidated cohesive material. Settlement of Structures, British Geot. Soc., Halsted Press, London, 500-530.
- Simons, N.E. and Menzies, B.K. (1975). A short course in foundation engineering. IPC Science and Technology Press.
- Skempton, A.W. and Sowa, V.A. (1963). The behaviour of saturated clays during sampling and testing. Geot., 13, 269-289.
- Skempton, A.W. and La Rochelle, P. (1965). The Bradwell slip: A short-term failure in London Clay. Geot., 15, 3, 221-242.
- Skempton, A.W. (1959). Cast in-situ bored piles in London Clay. Geot., 9, 4, 153-173.
- Skempton, A.W. (1964). Long-term stability of clay slopes. 4th Rankine Lecture, Geot., 14, 77-101.
- Skinner, A.E. (1975). The effect of high pore water pressures on the mechanical behaviour of sediments. Ph.D. thesis,

- Imperial College, Univ. of London.
- Skinner, A.E. (1978). Private Communications.
- Slater, G. (1972). The disturbed glacial deposits in the neighbourhood on LØNSTRUP, near HJØRRING, North Denmark. Trans. Roy. Soc. Edinb, 55, p. 303.
- Smith, R.E. and Wahls, H.E. (1969). Consolidation under constant rate of strain. JI. SMFD, ASCE, 95, SM2.
- Sodha, V.G. (1974). The stability of embankment dam fills of plastic clays. M. Phil. thesis, Imperial College, Univ. of London.
- Soil Mechanics for Road Engineers (1952). (RRL). HMSO, London.
- Solomon, J.D. (1932). The glacial succession in the North Norfolk Coast. Proc. Geot. Assoc., 43, 241:271.
- Som, N.N. (1968). The effect of stress path on the deformation and consolidation of London clay. Ph.D. thesis, Imperial College, Univ. of London.
- Souto, Silveira (1967). Thoughts concerning the applicability of the theory of elasticity to soils. Proc. 3rd Pan. Am. Conf. on Soil Mechanics, Caracas, 1, 3-27.
- Souva, V.A. (1963). A comparison of the effects of isotropic and anisotropic consolidation on the shear behaviour of a clay. Ph.D. thesis, Imperial College, Univ. of London.
- Sparks, B.W. and West, R.G. (1972). The Ice Age in Britain. Methuen and Co. Ltd. (1st Ed.).
- Takahashi, M. (1978). Ph.D. thesis (in preparation) Imperial College, Univ. of London.
- Taylor, D.W. (1948). Fundamentals of soil mechanics. John Wiley and Sons. Inc., New York.
- Tchalenko, J.S. (1967). The influence of shear and consolidation on the microscopic structure of some clays. Ph.D. thesis, Imperial College, Univ. of London.

- Terzaghi, K. (1931). Underground erosion and the Corpus Christi dam failure. Eng. News. Record, 107, 90-92.
- Terzaghi, K. and Peck, R.B. (1967). Soil mechanics in engineering practice. John Wiley and Sons Ltd.
- Vaughan, P.R. (1971). Undrained failure of clay embankments. Roscoe Memorial Symposium, Cambridge University.
- Vaughan, P.R. (1974). Halcrow-Costain North Sea platform, draft notes on soil mechanics aspects for combined report. Private Imperial College Internal Report.
- Vaughan, P.R. and Walbancke, H.J. (1975). The stability of cut and full slopes in boulder clay. Proc. The Symposium "The Engineering Behaviour of Glacial Material", Birmingham, 209-219.
- Vaughan, P.R., Lovenburg, H.T. and Horswill, P. (1975). The design, construction and performance of Cow Green embankment dam, Geot., 25, No. 3, 555-580.
- Vaughan, P.R. (1976). Draft figures for papers and lectures. Private Communication.
- Vaughan, P.R., Davachi, M.M., El-Ghamrawy, M.K., Hamza, M.M. and Hight, D.W. (1976). Stability analysis of large gravity structures. Proc. BOSS '76 Conference, 1, 467-488 Trondheim, Norway.
- Vaughan, P.R. and Hamza, M.M. (1977). Clay embankments and foundations. Monitoring stability by measuring deformations. Proc. 9th. ICSMFE, Tokyo, 37-48 (Proc. Speciality Session No. 8).
- Vaughan, P.R., Hight, D.W., Sodha, V.G. and Walbancke, H.J. (1978). Factors controlling the stability of clay fills in Britain. To be published.
- Walbancke, H.J. (1976). Pore pressures in clay embankments and cuttings. Ph.D. thesis, Imperial College, Univ. of London.
- Walker, L.K. (1969). Secondary compression in the shear of clays. JI. SMFD, ASCE, 95, SM1, 167-188.

- Webb, D.L. (1966). The mechanical properties of undisturbed samples of London Clay and Pierre Shale. Ph.D. thesis, Imperial College, Univ. of London.
- Werneck, M.L.G. (1974). Field observations of the upper lias clay at the Epingham Dam site. Ph.D. thesis, Imperial College, Univ. of London.
- Wesley, L.D. (1975). Influence of stress-path and anisotropy of a soft alluvial clay. Ph.D. thesis, Imperial College, Univ. of London.
- West, R.G. and Donner, J.J. (1956). The glaciation of East Anglia and the East Midlands. Q. JI. Geot. Soc. London 112, 69-91.
- Wiesel, C.E. (1973). Some factors influencing in-situ vane test results. 8th ICSMFE, Moscow, 1, p. 475.
- Wissa, A.E.Z., Christian, J.T., Davis, E.H. and Heiberg, S. (1971). Consolidation at constant rate of strain. JI. SMFD, ASCE, 97, SM10.
- Whitman, R.V.C. (1960). Some consideration and data regarding the shear strength of clays. Proc. ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado.
- Wroth, C.P. (1971). Some aspects of the elastic behaviour of overconsolidated clay. Roscoe Memorial Symposium, Cambridge.
- Wroth, C.P. (1975). In-situ measurements of initial stresses and deformation characteristics. Reprinted from the Proceedings of the Speciality Conference on In-Situ Measurement of Soil Properties. ASCE/Releigh, N.C./ June 1-4, 1975.
- Yong, R.N. and Warkentin, B.P. (1975). Soil properties and behaviour. Developments in Geotechnical Engineering 5 Elsevier Scientific Publishing Co. New York.

Errata

page	line*	word	corrected	page	line*	word	corrected
iv	6	Lupin	Lupini	190	17	sensitive	soft
xii	34	Ra	Re	196	5	16.05%	16.50%
xii	35	Rb	Ra	211	15	"ailure"	"failure"
xii	36	Rc	Rb	248	12	A040P40D	A010P40D
xiii	1	Rd	Rc	249	12	6.9.2	6.8.2
xiii	2	Re	Rd	256	25	very	every
xiii	15	(horizontal)	(horizontal)	279	20	6.10	6.8
19	8	shows	shows	280	14	NAP40	ANP40
21	24	HIGHT 7	HIGHT 1978	292	21	5.18	5.8
45	17	0.0008	0.0008 mm/min	292	26	5.18	5.8
45	19	:0.0008	:0.0008 mm/min	295	8	5.1	5.4
52	13	(1972)	(1971)	296	20	ig.	Fig.
58	9	Walbaneke	Walbancke	311	4	7.22	7.2.2
58	21	1970	1969	328	3	colcite	calcite
67	5	tan 4	tan ψ	463	3	sitee	situ
68	5	Wissa	Wissa et al.	464	4	MSFD	SMFD
69	18	Wissa	Wissa et al.	466	4	pare	pore
95	6	Wissa	Wissa et al.	468	13	rockfall	rockfill
102	18	residimented	resedimented	468	13	pure	pore
106	4	WRC	WCR	469	28	Souva	Sowa
136	24	(A3.66)	(A3.65)	469	29	Speciality	Spec. Tech.
151	23	Wissa	Wissa et al.	472	29	lay	Clay
155	15	(1972)	(1962)	476	21	Souva	Sowa
155	17	lower till	Lower Till	477	11	full	fill
180	29	postglacial	and postglacial	477	14	Lovenburg	Lovenbury
137	11	froming	forming	471	34	Melbourne	Auckland
57	19	operning	opening	183	7	farter	faster
59	17	quickley	quickly	226	12	1970	1971
65	10	adopeted	adopted				
70	1	95%	~ 90%				

* lines counted from top of page including titles and subtitles

Amendments

- p16 "A recent morphological Appendix (1)" should have been deleted
- p22-24 the years of reference should have been given in full (e.g. 1914,1973...)
- Fig. 6.32, p206 the lower arrow of OCR4 (right hand side of figure) should point to the middle point and not to the one at the bottom
- p214, line 7, the word "pressure" should have been deleted
- Fig. 6.42, p239 the notations a) and b) have been plotted twice. The ones inside the frames should have been deleted
- Fig. 6.48, p265, the expression (E at $\epsilon_1 = 1\%$) refers to isotropically consolidated samples
- p300, line 4, the number of the equation is (7.1)
- Fig. 7.1c, p316, the expression "from Fig. 5.1" should have been "from fig. 5.4"
- Figs 7.1h and 7.1k, p316, at the origin of axes the letter O should have been plotted
- missing in references, p 464, Bishop A.W. (1965). Discussion. Proc. 6th ICSMFE, 3, 306-310
- missing in references, p465, Bjerrum, L. (1967). Engineering geology of Norwegian normally consolidated marine clays as related to settlements of buildings. 7th Rankine lecture, Geot., 17, 81-118
- instead of Terzaghi, K. (1931) mentioned in references p477, the following should have been typed, Terzaghi, K. (1931). The static rigidity of plastic clays. Jl. of Rheology, 2,2
- missing in references, p478, Wu, T.H. (1966). Soil Mechanics. Allyn and Bacon, Inc., Boston, U.S.A.
- p472 the first reference should have been written Lang, J.G. (1971). Reduced soil strength and stiffness at the top of tube samples. Proc. 1st Australia-New Zealand Conf. on Geomechanics, Melbourne, 1, 232 -237.
- p287, in Fig. 6.58 in order to have better identification of test curves, refer Figs. A3.101 (p430) and A3.124 (p453).