

~~THE EFFECT OF FAST SHEARING~~
RATE OF
ON THE RESIDUAL
STRENGTH OF SOIL

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ABSTRACT

This thesis is composed of two parts.

The first part describes a laboratory investigation into the influence of fast rates of displacement on the residual strength of soils. This influence is studied in a displacement controlled ring shear apparatus. A shear surface is formed by slow shear and then tested at fast and slow rates of displacement alternately. The results from tests on a wide range of natural soils are presented. Three types of displacement rate effects are identified, neutral, negative and positive. All soils show an increase of strength when resheared at fast rates which then tends to decrease with displacement at the fast rate to a constant value (the fast residual strength). In some soils the fast residual strength is higher than the slow residual. This is termed positive rate effect. However, some soils show a fast residual strength which is less than the slow residual. This is termed negative rate effect. The relation between the types of displacement rate effects identified and the basic modes of residual strength at slow drained rate of displacements is discussed. Special tests were performed to demonstrate that the negative rate effect is an intrinsic soil property and that it does not depend on heat or pore water pressure generation.

The second part presents results of ring shear tests involving soil shearing against a rigid interface. The influence of normal stress, rate of displacement and interface roughness is investigated and the implications of the results in the context of pile foundations are discussed.

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NOTATIONS

Those symbols which do not appear here are defined where they appear in the text.

A/C	: alternating current
CF or C	: clay fraction (% <2 μ m)
c'	: effective cohesion intercept
$\bar{c}_R$	: effective residual cohesion intercept for a linear Mohr-Coulomb envelope
D/C	: direct current
D ₅₀	: particle diameter for which 50% of particles are finer
D ₁₀	: particle diameter for which 10% of particles are finer
G _s	: specific gravity of the soil
G _{sc}	: specific gravity of the clay particles
G _{sg}	: specific gravity of the granular phase
LI	: liquidity index of the soil
LL	: liquid limit (also w _L)
m _v	: volume compressibility
n	: porosity of the soil
PL	: plastic limit (also w _p)
PI or I _p	: plasticity index
S	: sliding shear mode
T	: room temperature
T _c or t _c	: pause allowed after fast shearing of the soil
TU	: Turbulent shear mode
TR	: Transitional shear mode
TR/TU	: Transitional shear mode tendency towards the turbulent mode
TR/S	: Transitional shear mode tendency towards the sliding mode
tan ϕ	: coefficient of friction, equal to τ/σ_n
tan ϕ'_R or tan ϕ'_r	: drained residual coefficient of friction, equal to τ/σ'_n (for c' = 0)
tan $\bar{\phi}'_R$	: drained residual coefficient of friction for a linear Mohr-Coulomb envelope
WC (w.c.)	: water content of the soil

w_f	: water content at failure
ΔU	: excess pore water pressure
$\Delta \theta$	: temperature increase
σ	: nominal normal stress (total)
σ_n	: normal stress (net)
σ'_n	: normal effective stress
τ	: shear stress
τ_r	: drained residual shear stress (or strength)
τ/σ_n	: stress ratio
τ/σ'_n	: drained stress ratio
τ_r/σ'_n	: drained residual stress ratio, drained residual coefficient of friction
ϕ_μ	: interparticle friction angle
ϕ'_{cv}	: effective friction angle for constant volume strength
ϕ'_r or ϕ'_R	: drained residual, friction angle
$\bar{\phi}'_R$	: drained residual friction angle for a linear Mohr-Coulomb envelope
ϕ'_p	: peak effective friction angle of shearing resistance

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND AND AIMS OF THE RESEARCH

It has been long established in Geotechnical engineering that the drained residual strength of soils is a parameter of great fundamental value. This is so because of its independence of original structure, stress history, and other factors which dominate the stress path dependent properties of soils.

The magnitude of drained residual strength is required for the assessment of stability when dealing with soils, subjected to previous shear movements, or with stiff intact soils when progressive failure is involved.

Early investigations of the effect of rate of shearing on the residual strength of soils, which have been carried out during the last two decades, used rates of displacement up to 10 mm/min approximately. They have found that the residual strength was not noticeably rate-dependent. While such rates of displacement are relevant to slow reactivated landslides, they are considerably slower than the rates of movements observed during rapid loading of pre-existing shear zones or surfaces in slopes. The existence of significant rate effects on the residual strength of soils at rates faster than 10 mm/min approximately was first identified with the works of Lupini (1981) and Martins (1983). Lemos (1986) investigated the effects of rapid rates of displacements on pre-existing shear zones or surfaces in soils from the Kalabagh dam site in connection with the earthquake design of the Kalabagh dam project. He showed that if a shear zone or surface is formed by slow drained shearing, and then subjected to more rapid rates of displacement, the initially observed increase of strength is followed by a drop to a constant minimum value. The minimum value is either similar to or higher than or lower than the slow drained residual strength of soils.

The aims of this thesis is to examine whether the rate effects, observed in the soils tested previously, were unique to them or of more general applicability, and to investigate the mechanism involved in the

drop of strength below the slow drained residual strength at fast rates of displacement.

The research has been carried-out using the ring shear apparatus, developed by Bishop et al (1971).

1.2 LAY-OUT OF THE THESIS

The thesis consists of seven chapters and six appendices.

Chapter 2 presents a review of previous investigations on the effect of rate of shearing on the residual strength. The hypotheses advanced to account for the low strengths at high shearing rates, observed both in the field and in the laboratory are also reviewed.

Chapter 3 describes the equipment and the testing techniques used, and the procedures followed in the classification tests and the study of the microstructure of the soils. The equipment used include the ring shear apparatus, the shear box and the cone-and-plate viscometer.

Chapter 4 summarizes the ring shear tests, carried out by the author and previous investigators at Imperial College at fast rates of displacement, and presents the analysis of the results.

Chapter 5 discusses the difficulties, associated with testing in the ring shear apparatus, and the typical strengths of soils (slow residual, threshold, fast peak, fast residual and slow peak strengths). Factors that influence the typical strengths are reported. The types of variation of the residual strength with an increasing rate of displacement are identified. The correlations between these types with the drained residual shear modes and the index properties are discussed. Soils showing the same type of behaviour are summarized.

Chapter 6 presents the investigation into the causes of the drop of strength below the slow drained residual value at fast rates of displacement.

Chapter 7 contains the main conclusions and the practical implications of the research.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

This chapter describes previous investigations on the effects of rate of shearing on the residual strength. The hypotheses advanced to explain the low strengths at high shearing rates, observed both in the field and in the laboratory are also reviewed.

The review of the effects of rate of shearing on the residual strength of soils is divided into two parts, referring to cohesionless and cohesive soils respectively. The contributions of various researchers is presented in an approximate chronological sequence.

2.2 THE EFFECT OF RATE OF SHEARING ON THE RESIDUAL STRENGTH OF COHESIONLESS SOILS

The behaviour of cohesionless soils or granular materials in general at high rates of shearing is relevant to a large number of engineering and physical processes. From an engineering point of view interest is centered around problems of materials handling and transportation, and mineral and powder processing. Examples of granular flows in the context of soil and rock mechanics involve several types of slope movements, such as rockfalls, avalanches, debris flows, small scale sand and talus flows and subaqueous quick sand flows.

Experimental studies of the behaviour of granular materials at high rates of shearing have been mainly carried out by means of flume and ring shear apparatus experiments. The following review is confined to investigations carried out by means of ring shear apparatus, as in the present research work.

Novosad (1964) studied the effect of high shearing speeds and small normal stresses on the internal friction angle of granular materials using a modified Hvorslev ring shear apparatus. He found that within a range between 0.5 and 50cm/sec the speed of rotation had no significant influence on the internal friction angle of the materials and that granular material 3mm above and below the shearing plane was in motion.

Scarlett and Todd (1969) investigated the behaviour of flowing sand in an annular shear cell. They reported measurements of dilation and critical porosity by the gamma ray attenuation technique. The thickness of the shear zone was found to increase with both particle size and normal stress. The number of particles (five to ten) in the shear zone decreased with increasing particle size. They found that the critical porosity of a bed of particles, which exists once the material is flowing, is related to the shape and size of particles and decreases with increasing normal stress. Rates of shearing between 1.3 to 5.1 rad/min did not affect the friction angle, which showed a tendency to increase with increasing normal stress. This tendency, however, was not conclusive.

Savage (1982) presented theoretical and experimental work on rapid granular flows. The experimental work was carried out by means of an annular shear cell. Tests were performed on materials with single size spherical particles, binary mixtures of materials with particles of different sizes and materials with angular particles. He found that both the normal and shear stresses increase with increasing concentration (defined as the ratio of the volume of the grains to the total volume). At lower concentrations the shear and the normal stresses vary with the square of the shearing rate. The dynamic friction angle is not significantly different to the quasi-static internal friction angle and is weakly dependent upon shearing rate and concentration. Savage (1982) concluded that his data at lower concentration were consistent with the behaviour predicted by Bagnold (discussed later). For a particular concentration both the normal and shear stresses can be separated into two parts, for example

$$\tau_{\text{total}} = \underbrace{\tau(\nu)_{\text{Coulomb}}}_{\text{rate-independent dry friction part}} + \underbrace{\mu(\nu) \cdot \rho \cdot D^2 \cdot \left(\frac{du}{dx}\right)^2}_{\text{rate-dependent dynamic part}} \quad (2.1)$$

where the terms $\tau(\nu)_{\text{Coulomb}}$ and $\mu(\nu)$ are monotonically increasing functions of solids concentration, and the terms ρ , D and (du/dx) represent the mass density, the particle diameter and the mean shear rate respectively. The rate-independent part results from dry Coulomb friction and particle overriding during enduring contacts between particles, and it is dominant at high concentrations and low shearing rates. The rate-dependent part is considered to be due to momentum

transfer during nearly instantaneous collisions between particles and it prevails at low concentrations and high shearing rates. Though Savage (1982) stated that the shear stress, τ_{total} , consisted of two parts, the rate-independent and the rate-dependent one, it is difficult to perceive how the two parts can coexist.

Hungr and Morgenstern (1984a) conducted a series of laboratory flume experiments to investigate the flow behaviour of dry sand at high velocities. They showed that sand retains its frictional character under very high rates of shearing up to 500cm/sec. To check whether this is the case when the normal stress is high, Hungr and Morgenstern (1984b) conducted high velocity ring shear tests on sand materials. Perfect frictional behaviour was observed in all their tests, uninfluenced by either velocity varying from 0.1 to 98cm/sec or normal stress varying from 20 to 200kpa.

Sassa (1984, 1985) reported ring shear tests on glass beads under constant volume and constant normal stress conditions within a range of speeds from 5×10^{-4} to 90.76cm/sec. He was concerned with the mechanism of debris flow. According to Sassa (1984), the constant volume ring shear tests on loose glass beads in a damp state (water content 1~3%) produce liquefaction and a steady state of low shearing resistance at a velocity of 90.76cm/sec similar to real debris flows. The constant normal stress ring shear tests on medium density dry glass beads showed that the dynamic angle of internal friction was almost the same as the static one.

Finally, Lemos (1986) reported a ring shear test on a coarse to medium sand. The rate of displacement varied from 0.0038 to 133mm/min. He found that the ultimate shear resistance was slightly affected by the displacement rates used.

A summary of the investigations and their findings is given in Table 2.1.

2.3 THE EFFECT OF RATE OF SHEARING ON THE RESIDUAL STRENGTH OF COHESIVE SOILS

Early investigations on the effect of rate of shearing on the residual strength of cohesive soils were confined in studying the effect of

relatively small rates of displacement. They were mainly concerned with the reduction of testing time by using the highest possible rates of displacement.

Petley (1966) was the first to report a systematic investigation into the effect of rate of displacement on the residual strength using the shear-box-technique with reversals. He found that for pre-cut samples of brown London clay the increase of residual strength with increasing rate of displacement from 4×10^{-5} to 0.10 mm/min was of the order of 4%. He was concerned with the discrepancy between laboratory and field rates of loading.

Kenney (1967) performed direct-shear tests on a wide range of materials. He concluded that there was little influence of rate of displacement on the residual strength within the range of 1×10^{-6} to 0.17 mm/min.

De Beer (1967) reported ring torsion tests on Boom clay using remoulded and pre-cut samples at 0.0066 and 0.035 mm/min. Tests performed at 0.035 mm/min gave lower residual friction angles than those at 0.0066 mm/min. According to De Beer (1967), when the rate of deformation becomes sufficiently small some recovery of strength in the shear zone occurs due to reformation of bonds. The reported displacements, however, are rather small and it is questionable whether real residual conditions had been established.

Ramiah et al (1967) tested a silty clay using the direct shear box. They found that the rate of displacement does not have significant influence on the residual strength of soil for rates of displacement between 0.023 and 10.2 mm/min.

Garga (1970) studied rate effects on the residual strength of blue London clay by means of a strain-controlled ring shear apparatus. He concluded that the residual strength of blue London clay is relatively insensitive to the rate of displacement, varying between 0.0038 and 0.0076 mm/min.

La-Gatta (1970) investigated the effect of rate of displacement on the residual strength of three soils, Pepper shale, Cucaracha shale and blue London clay by means of a displacement-controlled rotation shear

machine. The rate of displacement varied between 0.0048 and 0.48 mm/min. No significant change in the residual strength was measured for any of the three materials.

La-Gatta (1971) presented additional results from ring shear tests on Bearpaw shale and blue London clay. Rate effects were investigated for a range between 0.0048 mm/min and 2.4 mm/min. He found that for Bearpaw shale the variation of residual strength with the rate of displacement was insignificant and with no consistent trend. For blue London clay, an increase in the residual strength with increasing rate of displacement was observed.

Cullen and Donald (1971) determined the residual strength of a silurian clay using the direct shear box technique with reversals. The rate of displacement varied from 0.017 to 0.025 mm/min. They found that fast rates altered the residual condition reached at a slow drained rate of displacement. When the rate was increased rapidly, the shear strength at zero relative displacement was found to be 10-15% higher than the slow residual value. When a slow rate was reapplied, after steady values had been reached at faster rates, the residual strength was 0-15% lower than the steady state values at fast rates.

Gostelow (1974) performed a ring shear test on Penwortham Upper Till. Rate effects were examined by increasing the rate of shearing by a factor of a hundred from 0.0145 to 1.35 mm/min. A slight decrease in the residual strength was observed. It was noted also that if the rate was increased rapidly a slight peak occurred.

Bucher (1975), as reported by Lupini (1981), studied the effect of rate of displacement on the residual strength of a medium plasticity clay using ring shear devices. The residual strength was found to increase with increasing rate of displacement within a range between 0.0145 to 14.6 mm/min.

Lupini (1981) studied the influence of fast rates of displacement on the residual strength of four natural soils. Fast rates up to 177 mm/min were used. He discussed their behaviour during fast shearing in relation with their basic shear mode at drained rates of displacement. Lupini (1981) and Lupini et al (1981) identified three modes of drained residual shear.

- (i) a *turbulent mode* in soils with a high proportion of rotund particles, or, possibly in soils dominated by platy particles when the coefficient of interparticle friction between these particles is high. The residual friction ^{angle} is high (in excess of 25°), no preferred particle orientation occurs and brittleness is due to dilatant behaviour only. The residual friction angle depends primarily on the shape and packing of the rotund particles and not on the coefficient of interparticle friction. A shear zone, once formed, is a zone of different porosity only and it is considerably modified by subsequent stress history.
- (ii) a *sliding mode* in soils with platy, low-friction particles. A low strength shear surface of strongly orientated platy particles then develops. The residual friction angle depends primarily on mineralogy, pore water chemistry and on the coefficient of interparticle friction. A shear surface, once formed, is not significantly affected by subsequent stress history. Brittleness during first shearing is due primarily to preferred particle orientation.
- (iii) a *transitional mode* in soils with no dominant particle shape involving turbulent and sliding behaviour in different parts of a shear zone. In this mode the residual friction angle is sensitive to small changes in grading of the soil.

A soil, which is sheared in turbulent mode, may be sheared at least partly in sliding, if it is sheared against a smooth hard interface (Skinner, 1969).

The type of drained residual shear mode and, to a much lesser degree, the magnitude of the residual strength, relate best with the granular void ratio, defined as the ratio of the volume of platy particles plus voids to the volume of rotund particles (Fig.2.1). The shape of clay mineral particles present is of great importance. The correlations of change in drained residual shear mode with granular void ratio depend on the relative size of the platy to the rotund particles and on the grading of the rotund particles (Fig.2.2).

The picture that arised from Lupini's tests on the rate effects suggested that soils with sliding mode of drained residual shear show,

in general, an increase of residual strength with increasing rate of displacement. Soils with turbulent mode of drained residual shear show a tendency towards a decrease in residual strength with increasing rate of displacement.

For the soils with the sliding shear mode (Fiji clay, Kaolin, blue London clay), Lupini (1981) concluded that the model of sliding residual shear with parallel face to face arrangement of the platy particles explained their behaviour with respect to rate effects. According to this model, if drained residual shear occurs in the sliding mode ($\phi'_R = f(\phi'_p)$), the use of fast rates will increase the coefficient of interparticle friction, ϕ'_p , by virtue of *viscous effects*. If the rates are very fast and the viscous effects are significant, the interparticle friction may reach the value of constant volume strength, ϕ'_{CV} . This is equivalent to saying that the platy particles, which are sheared originally at slow drained rates in the sliding mode, switch to the turbulent mode with random particle orientation and their alignment in the shear surface is consequently destroyed. The likelihood of this happening is primarily dependent on two factors. Firstly, on the magnitude of the slow drained residual strength when compared to the constant volume strength, and secondly on the magnitude of the rate effects which bring about the increase in the coefficient of the interparticle friction, for example:

	ϕ'_R	$\phi'_P = \phi'_{CV}$	Shearing mode at fast rates
Fiji clay	17°	32°	turbulent
Kaolin	12°	19°	sliding
London clay	7°	24°	sliding predominant with some turbulence

Renewed brittleness, at a slow drained rate following a fast one, was observed with the soils which contained massive minerals [Fiji clay, London clay] and which during the fast shearing switch their behaviour from the pure sliding to the turbulent or to the sliding with some turbulence mode.

For the soils with a turbulent mode (Lower Cromer Till), the reduction in the residual strength with an increasing rate of displacement may be a consequence of an increase in the granular void ratio in the shear zone. This increase reduces the interlocking between the massive

minerals. This explanation is conjectural, however, as volume changes in the shear zone of the sample cannot be measured directly and accurately in a ring shear apparatus.

Lupini (1981) performed also a ring shear test on a silty London clay from South Ockendon. He noted that, because of the high sand and silt content of the soil and its high value of drained residual friction angle ($19-20^\circ$), the soil showed a hybrid behaviour in terms of the two postulated basic shearing modes, sliding and turbulent. The soil showed slight reverse rate effects at residual conditions and its residual condition was not affected by the application of a rate of 3.7 mm/min.

Martins (1983) performed a series of ring shear tests on weathered and mechanically degraded Tuff from Panama. The objective was to study the response of pre-existing shear surfaces to rapid rates of displacement and specifically the immediate (or threshold) increase of the shear strength required to initiate slip. Fast rates of displacement up to 100 mm/min were used. The test results showed that there are four rate dependent strength effects (Fig.2.3):

- (i) a *threshold effect* that applies prior to slippage.
- (ii) an increase in the angle of shearing resistance with rapid displacement, presumably due to restructuring of the shear surface.
- (iii) a pore pressure effect which reduces the effective stresses and hence the shearing resistance after slippage has occurred.
- (iv) A *non-transient increase* in strength, in excess of the original residual strength, when the sample was slowly re-sheared to residual conditions. The non-transient increase in strength increased with increasing rate of displacement during fast shearing. According to Martins (1983), this behaviour suggested that there is perhaps a critical rate of displacement below which the shear surface is not disrupted by the shearing.

Osipov et al (1984) studied microstructural changes associated with thixotropic phenomena in clay soils. Monomineral clays (Kaolinite and

Montmorillonite) and polymineral silty clay and loess were used. The experiments were performed on reconstituted paste samples prepared at moisture contents ranging from 0.8 to 2.2 times the liquid limit. Shear resistance was measured in static conditions and with applied vibrations by employing a rotary viscometer. After the sample had been sheared at 15 mm/min and a constant shear resistance had been achieved, it was vibrated for 5 min and then sheared again in the absence of external vibrations. Vibration caused a drastic reduction in the shear resistance of the soils depending on the vibration parameters which remained up until the vibration ceased.

Microstructure was examined by freezing the sample with liquid nitrogen as rapidly as possible while the test was in progress and then observing the structure with the aid of a scanning electron microscope. They observed that under static shear, deformation occurs over a limited volume in the shear zone and is attended by marked changes in the soil microstructure of this zone. For the system composed of small-sized particles the deformation in the shear zone is volumetric in character, and does not lead to disruption of structural continuity (shear planes). In coarser dispersed clayey soils both volumetric deformation and local disruption of the structural framework occur along developing shear planes.

The joint action of shear force and vibration resulted not in rupture, but, on the contrary, in the development of a more homogeneous microstructure over the entire volume. A dynamic state is induced in the soil by the impact of a vibration field as a result of the forced oscillations of the particles and their microaggregates. The structural elements are displaced, structural cohesion weakens and a partial disturbance of microaggregates and an increase in soil dispersion takes place. Cessation of the vibration leads to rapid restoration of the soil structure. A drastic increase in soil strength well above the residual strength (referred to as renewed brittleness by Lupini, 1981 and as non-transient increase in strength by Martins, 1983) is caused by the mutual fixation of the particles and the creation of coagulation contacts between them. As a result, the clay microstructure after thixotropic soil restoration shows a similarity with its initial state before the shear tests. A subsequent increase in thixotropic soil strength with time is related to the gradual establishment of thermodynamic equilibrium and complete restoration of

the initial microstructure.

Pyles (1984) performed field and laboratory vane tests to study the undrained residual strength of San Francisco mud. Rate effects were examined by performing the tests at rates of 3°/min and 17.5°/min (diameter of vane: 1 in). No significant rate effect were found.

Bernander (1985) reported direct shear box tests on soft swedish clays. The residual strength was found to decrease with both an increasing rate of displacement and an increasing overconsolidation ratio. According to Bernander (1985), the dependency of residual strength on the rate of displacement is due to the interaction between pore water pressure build-up in the slip surface and the rate of dissipation of these excess pore water pressures. The magnitude of these high localized pore water pressures cannot be studied in conventional apparatus for undrained testing, as they only measure the mean pore water pressure rise. The ratio of residual strength to the critical state shear strength evaluated from standard tests (field vane, cone, unconfined compression etc.) may attain values as low as 0.30 for soft clays at strain rates which are relevant to landslides. Brittleness ratios of this order readily explain the large planar landslides in Sweden which remain unexplained if an analysis based on the slow drained residual strength is used. The reported displacements by Bernander (1984), however, are rather small and it is doubtful whether real residual conditions had been achieved.

Skempton (1985) discussed the residual strength of clays in the field and laboratory as well as the effects of rate of shearing on the residual strength of soils with different clay fractions.

Finally, Lemos (1986) presented an extensive study on the effects of the rate of shearing on the residual strength of natural soils by means of a displacement-controlled ring shear apparatus. Fast rates of displacement up to 6000mm/min were used. Fig 2.4 shows the typical behaviours of pre-existing shear zones or surfaces during fast shearing, as identified by Lemos et al (1985) and Lemos (1986). The following features were observed:

- (i) Soils with very low clay fractions and with a turbulent shear mode at drained rates of displacement, show negligible rate

effects (Fig 2.4a).

- (ii) A sudden change in behaviour occurs with increasing clay fraction, and rate effects became significant. Soils with turbulent or transitional shear mode at drained rates of displacement show a reversed rate effect. The fast residual strength is less than the slow residual strength (Fig 2.4b).
- (iii) This behaviour changes progressively with increasing clay fraction. Soils with sliding shear mode at drained rates of displacement show conventional rate effects. The fast residual strength is higher than the slow residual strength (Fig 2.4c).
- (iv) The disordering effect of fast shearing, indicated by the slow peak strength after the fast shearing, increases with increasing clay fraction. The two strengths are the same in soils showing turbulent shear in which residual strength does not depend on particle orientation.
- (v) The increase of the threshold strength above the slow residual strength occurs mostly as an intercept to the failure envelope rather than as an increase of the angle of internal friction.
- (vi) The increase of the fast peak strength above the threshold strength approximates to the increase of the slow peak strength above the slow residual strength. Thus this increase may well be due to the disordering effect postulated to occur for the slow peak strength.

Table 2.2 presents a summary of the previous investigations into the effects of the rate of shearing on the residual strength of cohesive soils, except those reported by Lupini (1981) for the residual soil from Fiji and Kaolin, Martins (1983) and Lemos (1986). The tests carried out on the residual soil from Fiji and Kaolin by Lupini (1981) and those performed by Martins (1983) and Lemos (1986) are presented in Appendix 2.

2.4 HYPOTHESES ADVANCED TO EXPLAIN THE LOW STRENGTHS AT HIGH RATES OF SHEARING OBSERVED IN THE FIELD AND IN THE LABORATORY

The large, rapidly moving masses of fast landslides exhibit a degree of mobility which can not be explained by reasonably assumed friction angles. They are composed of materials such as disintegrated rock and soil both in dry and wet conditions. Laboratory investigations have also been reported, showing a decrease in shear strength of certain soils at increasing rates of shearing.

Hypotheses advanced to account for these low strengths at high rates of shearing, observed in the field and laboratory, include *mud-lubrication*, *air-fluidization*, *acoustic-fluidization*, *entrapped-air cushion*, pore water pressure generation due to *frictional heating*, *dissociation due to melting*, *liquefaction*, and hypotheses based on the *rheology of the materials involved*. All of the hypotheses are based on limited available evidence. In particular, the hypotheses of *mud-lubrication*, (Heim, 1932), *air-fluidization* (Kent, 1966), *vapour-fluidization* (Habib, 1967, 1975 and Goguel, 1978), and *entrapped-air cushion* (Shreve, 1968), used to explain rock avalanches, are subject to criticism, given the facts that large avalanches are now known to exist on the moon despite the lack of lubricating or cushioning fluid (Howard, 1973) and that terrestrial avalanches are very mobile in the absence of much water (Heim, 1932, Sharp and Nobles, 1963, and Kojan and Hutchinson, 1978). Since the hypotheses just mentioned above seem to be irrelevant to the loss of strength during fast shearing, observed in this research, reference is now made to the remaining hypotheses of *mechanical* and *acoustic-fluidization*, pore water pressure generation due to *frictional heating*, *dissociation due to melting*, *liquefaction*, and on the *rheology of the materials involved*.

The concept of *mechanical fluidization* has been based on the dispersive-grain flow of cohesionless materials described by Bagnold (1954), and it has been called as such by McSaveney (1978). The hypothesis assumes that the nature of shearing in granular materials changes at extremely high rates of shearing, because the type of particle contacts is different than that of the slow quasi-static movement. When shearing is rapid, continuous particle friction no longer takes place as the mass dilates. As dilation occurs during flow,

particle contacts become brief impacts, the momentum of which transfers both the shear and normal stress within the dispersion.

Bagnold (1954) performed experiments on gravity-free dispersions of large solid spheres in Newtonian fluids by means of a torsional viscometer. Bagnold (1954, 1956) reasoned that a static granular mass can not flow without some degree of dilation. In the case of grain-flow, the dispersion must be upward against the bedward body force. He assumed that the overall shear resistance can be written as

$$\tau = T + \tau' \quad (2.2)$$

where T and τ' are the resisting shear forces due respectively to the encounters between the grains and to the distortion of the intergranular fluid.

A substantial dispersive pressure, P , due to encounters between the grains, was found to be exerted between them. The dispersive pressure, P , is related to the shear, T , by $T/P = \tan \alpha$, where the angle α is associated with the mean encounter conditions, and $\tan \alpha$ is therefore the dynamic analogue of the static friction coefficient. Irrespectively of whether the shearing of the intergranular fluid is inertial or otherwise, the shearing of the grain dispersion within it, regarded as a separate fluid, may also be inertial or otherwise, the inertia being that of the grains. For concentrations (defined as the ratio of the volume of the grains to the total volume) smaller than 60%, both stresses T and P may vary as the square or the first power of rate of shearing depending on whether the grain inertia (large rate of shearing) or the fluid viscosity (small rate of shearing) dominate. The shear stress, T , for example is given as:

$$T \propto \sigma \cdot (\lambda \cdot D)^2 \cdot \left(\frac{du}{dy}\right)^2 \quad \text{for the inertia region} \quad (2.3)$$

$$T \propto \lambda^{3/2} \cdot n \cdot \left(\frac{du}{dy}\right) \quad \text{for the viscous region} \quad (2.4)$$

where σ is the density of grains

D is the diameter of grains

$\lambda = D/s$ is the linear concentration

s is the free distance between two grains

n is the fluid viscosity, and

$\left(\frac{du}{dy}\right)$ is the rate of shearing

When the grain shearing is inertial and the encounters become collisions, the grain stresses T and P arise from the diffusion of tangential and normal components of grain momentum created at the collisions. Hence the normal stress P may increase by a factor of two as between elastic and inelastic collisions, with a subsequent reduction in the value of stress ratio, $\tan \alpha$, for elastic grains. The ratio T/P was constant at 0.30 approximately in the inertia region and 0.75 approximately in the viscous region. Bagnold (1954) applied his experimental results to the case of dry sand flow under gravity and flowing gravel.

Howard (1973) used the concept of mechanical fluidization in explaining the low friction coefficient of friction observed at lunar avalanches.

Hsü (1975) supported the view that according to Heim's (1937) descriptions for the catastrophic Elm rockfall in Switzerland, 1881, it can be explained as a grain-flow phenomenon, referred to by Bagnold (1954). Heim (1932) described the Elm rockfall. It did not slide, it 'crashed', and its dispersed debris flowed down the flat valley bottom and up the slope on the other side. According to Hsü (1975) and Sassa (1985), flowing grain masses can move down gentle inclines, because of the presence of a dense interstitial fluid which reduces the effective normal pressure of grains and subsequently reduces the frictional resistance. As the interstitial fluid involved, Hsü (1975) suggested water-saturated mud and compressed air. Dispersion of fine particles between colliding grains was invoked to account for the fluidity of lunar avalanches.

Körner (1977) used the mechanical fluidization concept in his rolling model of avalanche movements. According to Körner (1977), loosening goes beyond the critical density at flow shearing rates, so that parts can move more easily past each other.

McSaveney (1978) stated that the role of the fine component within the flowing mass is not only to increase the number of particles and thus the frequency of collisions but also to increase the dispersion of larger clasts so that they may move readily past each other. As with molecular solids, debris of different packing should represent

different solid phases with different stability fields. More open states of packing should exist at lesser thicknesses and greater energies and they should have correspondingly lower melting or sublimation points than more close packed debris.

Hungr and Morgenstern (1984a, b), on the basis of their experimental results, referred to previously, concluded that the hypothesis of mechanical fluidization, postulating a breakdown of the Coulomb relationship, is not correct. It is not certain, however, that the requirement of the concept, that the dilation of the flowing mass is such that the grains can pass easily past each other, was achieved at the shearing rates used in their experiments. It is possible that higher shearing rates would indicate a loss of strength for the materials tested.

The *acoustic-fluidization* hypothesis, proposed by Melosh (1979) and applied in a number of geological processes, assumes that sufficiently strong acoustic waves in rock debris can momentarily relieve the static overburden pressure in limited regions of the rock mass, thus allowing sliding to occur in the expanding fluidized region. If acoustic-fluidization is to occur in the rupture zone, energy must be supplied to the rock material to fluidize it. In the case of landslides, the initial fall of the slide down a steep slope is required to produce a large enough acoustic energy to fluidize it. Once this critical level is reached, the slide flows and spreads out as a fluid.

The hypothesis of pore water pressure generation due to *frictional heating* is based on Habib's (1967, 1975) suggestion of a thermodynamic process to explain the high speed of large landslides. When a slide takes place, the work done by the gravity forces is dissipated on the slip surface and heat appears. If the rate of shearing becomes high enough, the heat liberated can increase the pore water pressure inside the sliding mass due to different thermal expansion of water and soil grains, or even convert the water into vapour, creating a vapour cushion which sustains the slide mass and lubricates the motion (referred to as vapour-fluidization previously).

Romero and Molina (1978) and Anderson (1985) tried to explain the kinematic aspect of the Vajont landslide in Italy, 1963, considering

the effect of water heating and the subsequent pore pressure increase in the friction zone.

Anderson (1980) presented an earthquake induced heat mechanism to explain the loss of strength of large rock and earth slides.

Voight and Faust (1982) suggested that high pore water pressures may result, even if vaporization does not take place. They developed an analytical and numerical model to account for the strength loss in some rapid landslides.

Finally, Erismann (1979) proposed the hypothesis of *dissociation due to rock melting*, supported by Voight and Faust (1982) later. The concentrated high energy near the friction surface is transformed in high temperature which causes the self-lubrication of the landslide.

The main criticism of all the investigations, based on the hypotheses of frictional heating, is that their results are very sensitive to the values of the parameters used in them, such as, friction angle, normal stress, and thickness, porosity, and compressibility of the shear zone. All the investigations consisted of theoretical predictions without any field or experimental evidence. Besides, none of the investigators has taken into account the rate dependence of the residual strength of the materials involved. In the case of a material showing a loss of strength at increasing rates of shearing, this would mean that the energy absorbed by friction would have been reduced significantly, so that frictional heating would not have increased the pore water pressure, as much as predicted by their analysis.

Atakol and Larew (1970) measured the temperature changes during dynamic simple shear tests on dry sand by means of thermocouples with 0.3°C sensitivity. No measurable change in temperature took place in the soil during the shearing event even for the highest levels of normal stress (570.kPa) and rate of displacement (1000 cm/sec).

Lemos (1985) measured the temperature change in the shear zone of a clayey siltstone during fast shearing by means of a ring shear test at a normal stress of 500 kPa and rates of displacement of 800 and 6200 mm/min. The maximum temperature rise was only 2.5°C, far too small to account for the increase of pore water pressure needed to explain the

drop of the residual strength of this soil at increasing shearing rates.

The hypothesis of *liquefaction* in accounting for the mobility of flow slides was introduced by Terzaghi (1956). He used the term *spontaneous liquefaction* to indicate the sudden collapse of a metastable structure formed by loose cohesionless or slightly cohesive soils, resulting in a considerable tendency to contract with the generation of high pore water pressure. The resulting flow slide travels until the average degree of consolidation is close to unity and the excess pore water pressures are dissipated.

This idea was further developed by Hutchinson (1986) who presented a sliding consolidation model for flow slides. The model predicts the run-out of a flow slide as a function of the degree of consolidation of a specified thickness of liquefied debris. The results, however, are very sensitive to the thickness of the liquefied debris, which remains unknown in many field cases. The concept of *liquefaction*, as the triggering event of a flow slide, has been also used by other investigators, such as, Casagrande (1975), Seed (1967), Hutchinson and Bhandari (1971), Bishop (1973), and Sassa (1984).

Seed (1967) stated that a slide, caused by soil liquefaction, may not result in such large deformations that it would be called a flow slide, since liquefaction persists, as long as high pore water pressures persist in a soil. If drainage can occur rapidly then liquefaction may persist for such a short period of time that large displacements are unable to develop. From the description of field flow slides, he concluded that the types of materials involved are loose to medium dense, saturated sandy soils and rain-soaked loess.

Hutchinson and Bhandari (1971) studied the ability of lower parts of a mud flow or other mass movements to advance by shearing on slopes considerably flatter than those corresponding to limiting equilibrium, for residual strength on the sliding surface, and ground water coincident with and flowing parallel to the slope surface. They concluded that sliding is brought about by the essentially undrained loading of the headward parts of the landslides by debris discharged from steeper slopes to the rear.

Casagrande (1975) stated that when sand is liquefied and is actually flowing, its effective friction angle changes, as its structure is different from that when the soil is static. During flow each grain is constantly rotating in relation to all surrounding grains so as to offer a minimum of friction resistance. Such a flow structure spreads by a chain reaction and exists only during flow. The moment that flow stops, the grains rearrange themselves and revert to a static structure, which, after the excess water has drained, will be slightly denser than the static structure before the liquefaction occurred.

Sassa (1984, 1985) used also the idea of undrained loading of loose torrent deposits by debris discharged from steeper slopes to the rear, which results in their liquefaction. The flow of the deposit causes a high specific gravity of the pore fluid resulting in a low shearing resistance during flow (referred to previously in the concept of mechanical fluidization).

Finally, Sassa (1987a and b) interpreted the high mobility of the Ontake landslide, Japan, caused by the Nagano-Seibu earthquake, 1983, by considering the similarity of the undrained loading of saturated deposits overlying the bed-rock by the non-saturated moving mass with the hydro-planing phenomenon in a wet speed way. When a car moves on a wet speed way, water inserts below the tyre from the front, and it is drained to the slides. The inserting speed of water increases with the car speed while the drain speed to the sides is almost the same. Then, at a certain speed, a water film is formed below the tyre supporting a part of the car weight. Accordingly, the friction between road and tyre decreases.

The hypotheses that can be advanced from the rheological perspective to explain the observed low strengths at high rates of shearing consider the change in the rheological condition of a flowing mass as a result of some process, like thixotropy, remoulding, formation of a flow structure, e.t.c..

Freudlich (1935a) used the term *thixotropy* to describe the isothermal reversible sol-gel transformation. Since the process of softening and stiffening is completely reversible in a thixotropic material, the phenomenon in soils excludes any changes in water content. According to Freudlich (1935a), thixotropic behaviour depends on the attracting

and repelling forces acting between the particles of the sol or gel.

Freudlich (1935b) concluded that the thixotropic behaviour of a quicksand from Knott end, Fleetwood, was ascribed to the presence of 2.1% of a fine plastic clay, the ordinary sand containing only 0.3%. When the clay together with all particles having a diameter below $10\mu\text{m}$ was separated from the coarser grains, the residue was no longer thixotropic. On re-mixing the clay with the residue, thixotropic behaviour reappeared.

Freudlich and Jones (1936) studied the sedimentation volume, dilatancy, and thixotropic and plastic properties of concentrated suspensions. The materials used, most of which were minerals, had particles with sizes between $1\mu\text{m}$ to $10\mu\text{m}$. They concluded that the degree of independence of the particles is the best criterion for judging the probable behaviour of the suspensions. Non-adhering particles were considered to be independent. The tendency of dilatant systems to form closely packed sediments with a small volume after sedimentation is due to the independent behaviour of the particles. Particles which are not independent always give materials which are to a certain extent thixotropic, but they need not be plastic and their volume of sedimentation may even be fairly small. Plastic pastes are always thixotropic and associated with large volumes of sedimentation. Markedly anisometric especially plate like particles always give thixotropic behaviour. Particles of very hard substances are always strongly independent, and therefore give systems with close packed particles. Suspensions of quartz particles convert from dilatant to thixotropic, when the liquid in the suspension is changed from a polar (i.e water, ethyl, alcohol, glycerol) to a non-polar one (i.e benzene). Powdered quartz mixed with an aqueous solution of an electrolyte exhibits thixotropic behaviour, but only within a very narrow range of concentrations, due to the adhesion change between the particles owing to the action of an electrolyte. Above a certain size (more than $10\mu\text{m}$) adhesion is not able to overcome gravity and is rather small. Particles with diameters 2 to $3\mu\text{m}$ adhere practically 100%, while smaller particles have smaller adhesion because thermal agitation overcomes adhesion.

Seed and Chan (1957) and Mitchell (1960) studied the thixotropic characteristics of a compacted silty clay (LL = 38, PL = 24 %). They

concluded that in compacted soils the influence of thixotropy appears to be significant at high degrees of saturation, but very small to samples compacted on the dry side of optimum. Mitchell (1960) concluded that thixotropic behaviour is the natural response of a soil structure to a change in ambient conditions. When a soil is remoulded or compacted a structure is induced which is compatible with the externally applied shearing stresses. When shearing stops the soil is left with an excess of internal energy which is dissipated by means of small particle movements and water redistribution until a structure in equilibrium is created with the forces at rest.

Day (1955), as reported by Mitchell (1960), measured water tension changes of saturated clay samples at water content near the liquid limit by means of a simple tensiometer before and after stirring. Tension was always found to drop to a minimum after stirring and then gradually return to nearly the initial value. Such pore water pressure changes following remoulding of London clay at constant water content have been also reported by Bishop et al (1960). According to Mitchell (1960), these tension measurements reflected the changes in the amount of double layer interaction between particles and the decrease of free energy of the system with time after stirring.

Osipov et al (1984), referred to previously, studied microstructural changes associated with thixotropic phenomena in clay soils.

Kerr and Drew (1967) investigated the thixotropic behaviour of soft, saturated samples of clay known to be sensitive in the field. On stirring or shaking the strength was reduced to near zero. On standing for about 40 min the clay samples regained their initial water-saturated strength. They attributed the loss of strength to the collapse of the original cardhouse structure changing the clay mass to a flowing fluid.

Ward and Whitmore (1950) studied the viscosity and sedimentation of stable, unflocculated suspensions of methyl-methacrylate polymer spheres in an aqueous solution of lead nitrate and glycerol of viscosity of about 6 centipoise. They concluded that the relative viscosity of a suspension of hard, smooth, non-interacting spheres at a given concentration (defined as the ratio of the grain volume to the total volume) is practically independent of the rate of shearing up to

volume concentrations of 30%, independent of the absolute size of the spheres, independent of the viscosity of the suspending liquid and dependent upon the size distribution of the spheres, decreasing with increasing size range ratio (defined as the ratio of the largest to the smallest particle diameter). When the suspensions consist of dielectric particles in a non-polar liquid, electrostatic charging of the particles may affect the measured viscosity.

Chong, Christansen and Baer (1971) studied the viscosity of highly concentrated monodispersed and bidispersed suspensions. For monodispersed suspensions, a rather rapid increase in viscosity at about 50% was observed. At a concentration of about 60% the relative viscosity tended to infinity and the suspension showed peculiar hysteretic behaviour caused by volumetric dilatancy. For the bidispersed systems the particle size ratios (ratio of the smallest to the largest diameter) was chosen to permit packing of small spheres into the voids between large spheres. According to Chong et al (1971), the fact that the relative viscosities of bimodal suspensions decreased significantly as the number of small spheres increased indicated that the fine spheres act like ball bearings between large spheres. However, if the particle size ratio is less than 1/10 this action appeared to cease gradually and the small spheres seem to behave essentially like a fluid.

In addition to the previous hypotheses, developed in explaining the low strengths at high rates of shearing, there are also those advanced by Hight (1983) for the residual strength and by Casagrande and Wilson (1951), Bishop et al (1960), Richardson (1961), and Blight (1965), for the peak strength.

Hight (1983) studied the effect of rates of shearing on the undrained behaviour of lower Crouer Till. The residual strength of soil was found to decrease with increasing rate of shearing irrespective of the soil's initial state. He concluded that at fast rates of shearing the particular microfabric gives rise to pore-pressure inhomogeneities in the soil, which contribute to the rate effect and lead to difficulties in interpreting the behaviour in terms of the effective stress.

Casagrande and Wilson (1951) presented results from unconfined

compression tests at constant water content on an undisturbed sandy clay (LL=32, PL=21%) and two compacted soils (clayey sand, LL=18, PL=16% and silty clay, LL=37, PL=23%). Tests with time of loading greater than 1 day gave higher strengths than the ones with time of loading 10 min, whereas tests with time of loading between 10 min and 1 day gave lower strengths than the ones with time of loading 10 min. According to Casagrande and Wilson (1951), the soils were not fully saturated and it is probable that the increase of strength was accompanied by a decrease of void ratio at constant water content. This means that internal consolidation had occurred, the gas in the voids being compressed and absorbed by the water.

Bishop et al (1960) presented undrained triaxial test results on a compacted shale at different strain-rates, showing decreasing shear strength with decreasing time to failure. The pore pressure parameter A was found to increase with decreasing time to failure. According to Blight (1965), the pore pressure parameter A for heavily overconsolidated clays increases as the time to failure in undrained shear decreases because of the tendency to slow dilation under load.

Finally, Richardson (1961) carried out static (time to failure 1000 sec), slow (time to failure 11 sec), and rapid (time to failure 10 msec) triaxial compression tests on saturated loess (PL=26, LL=37, CF=14%). The increase in strength over its static value was higher for the slow than the rapid tests. Richardson (1961) attributed this type of behaviour either to the change of the overall pore water pressure parameter A, due to the effect of pore water migration to or away from the zone in which failure is taking place, or to the scatter of the experimental results.

Table 2.1 Summary of previous investigations on the effects of rate of shearing on the residual strength of granular materials

No	AUTHOR	MATERIAL TESTED	D ₅₀ (mm)	D ₆₀ / D ₁₀	NORMAL STRESS (kPa)	RATES USED (cm/s)	ANGLE OF FRICTION (°)	RATE EFFECT
1	NOVOSAD (1964)	Dry coarse sand	1.1	1.0	0.5~2.0	0.5	33	NONE
		Dry fine-medium sand	0.17	1.90	>>	50.0	33	
		Sugar	0.57	1.79	>>	0.5	31	
						50.0	32	
		Dry glass spheres	2.5	1.0	>>	0.5	34	
						50.0	33	SLIGHT
						50.0	26	
2	SCARLET AND TODD (1969)	Dry coarse sand	1.01	1.0	2.2	-	33	NONE
					4.3		33	
					6.5		34	
		Dry medium sand	0.36	1.0	2.2		33	
					4.3		32	
					6.5		30	
		Dry medium-fine sand	0.21	-	2.2		26	
					4.3		26	
					6.5		31	
3	SAVAGE (1982)	Dry single-sized glass ballotini	0.5~2.0	1.0	-	30~500		NONE
		Dry single-sized polystyrene	0.5~2.0	1.0		>>		
		Binary mixture of glass ballotini				>>		
		Binary mixture of polystyrene				>>		
		Crushed walnut shells				>>		
4	SASSA (1984,1985)	Dry glass beads	1.0	1.0	8-20	0.01	19	NONE
						0.23	19	
						4.54	19	
						90.76	19	
5	HUNGR & MORGENSTERN (1985)	Coarse uniform rounded quartz sand (dry and wet)	1.5~2.0	1.0	dry:20~200	0.1	29	NONE
						16.0	29	
						98.0	29	
					wet:50~100	0.1	26	
						16.0	26	
						98.0	26	
		Coarse uniform rounded quartz sand (dry)	2.0-3.0	1.0	45-60	0.1	30	
						16.0	30	
		Polystyrene beads	1.5	1.0	40-80	0.1	21	
						16.0	21	
		4:1 sand-flour mixture*			20-100	0.1	31	SLIGHT
						16.0	31	
						98.0	29	NONE
		4:2 sand-flour mixture*			20-80	0.1	31	
						16.0	31	
						98.0	31	
6	LEMOs (1986)	Saturated coarse to medium sand	0.63	1.06	100	6.3×10 ⁻⁶	30.5	SLIGHT
						4.2×10 ⁻⁶	31.0	
						0.0059	31.0	
						0.2217	27.0	

* Mixture of the coarse sand with D₅₀ = 1.5 ~ 2.0 mm and crushed pure quartz with D₅₀ = 0.044 mm.

Table 2.2 Summary of previous investigations on the effects of rates of shearing on the residual strength of cohesive soils

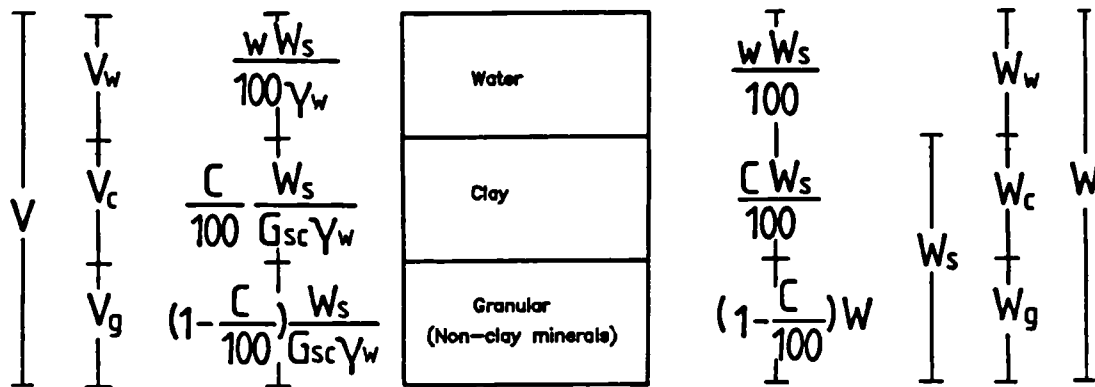
No	AUTHOR	TYPE OF APPARATUS	TYPE OF SOIL	LL %	PI %	CF %	PI CF	NORMAL STRESS (kPa)	RATES USED (mm/min)	$(\tau/\sigma_n)_o$ ¹	$\frac{(\tau/\sigma_n)}{(\tau/\sigma_n)_o}$ ²
1	PETLEY (1966)	Shear box	Brown London clay	82	49	60	0.82	40~140	4×10^{-8} 0.1	0.252	1.00 1.04
2a	KENNEY (1967)	Shear box	Muscovite < 6 μ					98	1×10^{-6} 0.02	0.370	1.00 1.08
b			Na-Hydrous mica, 0.5M. NaCl					784	1×10^{-6} 0.02	0.360	1.00 1.06
c			Na-montmorillonite OM. < 2 μ					98	1×10^{-6} 0.0027	0.080	1.00 1.00
d			Na-montmorillonite OM. < 0.1 μ					98	3×10^{-4} 0.16	0.060	1.00 1.00
e			Ca-montmorillonite OM. < 2 μ					98	7×10^{-4} 0.02	0.170	1.00 1.05
f			Saines quick clay	23	5	42	0.12	98	1×10^{-6} 0.0033	0.580	1.00 1.03
g			Kaolinite	59	22	72	0.30	98	7×10^{-4} 0.17	0.260	1.00 1.04
h			Vajoint clay < 60 μ	62	44	52	0.85	98	1×10^{-6} 0.02	0.147	1.00 1.02
3	DE BEER (1967)	Ring torsion apparatus	Boom clay	81	52	49	1.06	50-250	0.0066 0.035	0.448	1.00 0.77
								50-250 ³	0.0066 0.035	0.347	1.00 0.77
4											
4	RAMIAH et al (1967)	Shear box	Silty clay	45	17	8	2.12	45	0.023 10.2	0.613	1.00 1.05
								68	0.023 10.2	0.628	1.00 1.04
								135	0.023 10.2	0.541	1.00 1.02
5	GARGA (1970)	Ring shear	Blue London Clay	72	43	57	0.75	185 200	0.0038 ⁴ 0.0076 ⁵	0.165	1.00 1.00
6a	LA-GATTA (1970)	Ring shear	Pepper shale	70	49			100 ⁶	0.0048 0.048	0.152	1.00 0.93
b			Crushed Cucaracha shale	49	21	42	0.5	800 ⁷	0.0048 0.048 0.48	0.122	1.00 1.00 1.04
c			Blue London Clay	71	49	57	0.86	400	0.0048 0.048 0.48	0.135	1.00 1.04 1.07
			>>	>>					2.40		1.18
d			Bearpaw shale	104 ⁶	79	53	1.49	400	0.0048 0.048 0.48 2.40	0.066	1.00 0.98~1.18 0.94~1.04 0.92~0.98

Notes: 1: $(\tau/\sigma_n)_o$, residual stress ratio at a slow drained rate of displacement
 2: (τ/σ_n) , residual stress ratio at the current rate of displacement
 3: pre-cut sample
 4: remoulded
 5: undisturbed.
 6: air-dried
 7: slaked four times

Table 2.2 (continued)

No	AUTHOR	TYPE OF APPARATUS	TYPE OF SOIL	LL %	PI %	CF %	PI CF	NORMAL STRESS (kPa)	RATES USED (mm/min)	$(\tau/\sigma_n)_o^1$	$\frac{(\tau/\sigma_n)^2}{(\tau/\sigma_n)_o}$
7	CULLEN & DONALD (1971)	Shear box	Silurian clay		35				0.017 0.250		1.00 1.00~1.15
8	GOSTELOW (1971)	Ring shear	Penwortham Upper Till	42	23	14	1.64	46	0.0145 1.350		1.00 <1.00
9	BUCHER (1975)	Ring shear	Medium plasticity clay	43	27			-	0.0145 0.10 14.6		1.00 1.03 1.24
10a	LUPINI (1981)	Ring shear	Lower Cromer Till	22	10	14	0.71	220	0.0001 0.014 3.6 1.33 0.014	0.600	1.00 0.92 0.83-0.95 0.59 1.00
b			Blue London clay	75	51	48	1.06	220	133 0.001 17.74	0.555 0.122	0.54 1.00 1.31
c			>>	57	36	36	1.00	352	177.3 0.0005 0.0013 0.0050	0.380	1.64 1.00 0.98 0.98
11a	OSIPOV et al (1984)	Rotary viscometer	Kaolinite	56		47			15 V ⁸		1.00 <1.00
b			Montmorillonite	200		97			15 V		1.00 <1.00
c			Silty clay	40		19			15 V		1.00 <1.00
d			Loess	25		11			15 V		1.00 <1.00
12	PYLES (1984)	Vane test	San Francisco Mud						0.66 3.88		1.00 1.00
13a	BERNANDER (1985)	Shear box	Lilla Edet	60					0.3 3 30		1.00 <1.00 <1.00
b			Linnarhult	40-65					0.3 3 30		1.00 <1.00 <1.00
c			Rollsbo	70					0.3 3 30		1.00 <1.00 <1.00

Notes: 1: $(\tau/\sigma_n)_o$, residual stress ratio at a slow drained rate of displacement
 2: (τ/σ_n) , residual stress ratio at the current rate of displacement
 8: sample vibrated for 5 min after it had been sheared at 15 mm/min
 (frequency 20 Hz, displacement amplitude 0.6 mm, acceleration 10m/s²)



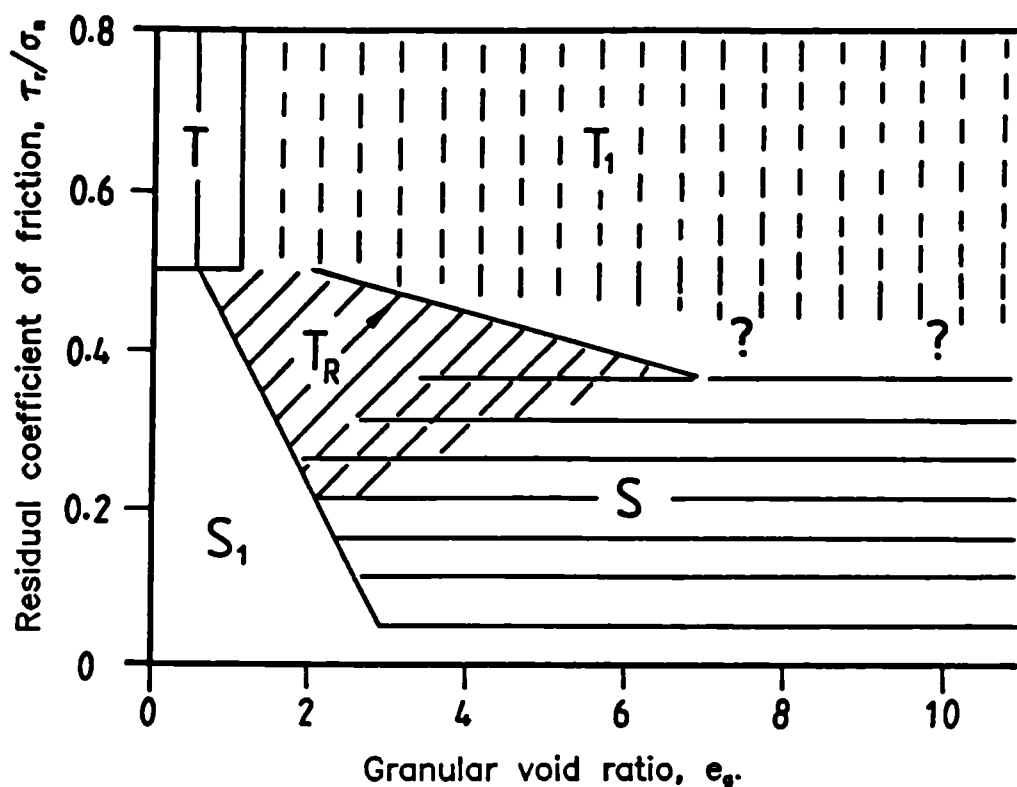
W : water content
 C : Clay fraction
 γ_w : unit weight of water
 G_{sc} : specific gravity of clay particles
 G_{sg} : specific gravity of granular phase
 e_g : void ratio of the granular phase
 e_c : void ratio of clay+water system

$$e_g = \frac{V_w + V_c}{V_g} = \frac{C + wG_{sc}}{(100 - C) \frac{G_{sc}}{G_{sg}}}$$

$$e_c = \frac{V_w}{V_c} = \frac{wG_{sc}}{C}$$

$$C = \frac{W_c}{W_s} \times 100 = \frac{100 e_g \frac{G_{sc}}{G_{sg}} - wG_{sc}}{1 + e_g \frac{G_{sc}}{G_{sg}}}$$

Fig. 2.1 Weight-volume relationships for saturated clay-granular soil mixture



- T Turbulent shear.
- T_1 Turbulent shear for platey particles if $\phi_p > \phi_m$. Also for soils containing non-platey clay particles if these are excluded from the volume of rotund particles when calculating e_g .
- S Sliding shear.
- S_1 Possible sliding shear when soil is failed against a smooth interface.
- T_R Transitional shear. Part turbulent and part sliding. Location of a particular soil within this zone depends on residual friction coefficient of rotund (massive) and platey particles as tested separately and on soil grading. Platey clay particles mixed with well graded sand-silt massive particles fall to the left of the zone. Gap graded fine clay and sand mixtures fall further to the right in the direction of the arrow. Mixtures of platey and rotund particles of comparable size fall to the right part of the zone.

Fig. 2.2 Residual mechanisms as a function of particle packing
summary of conclusions (Lupini, 1981)

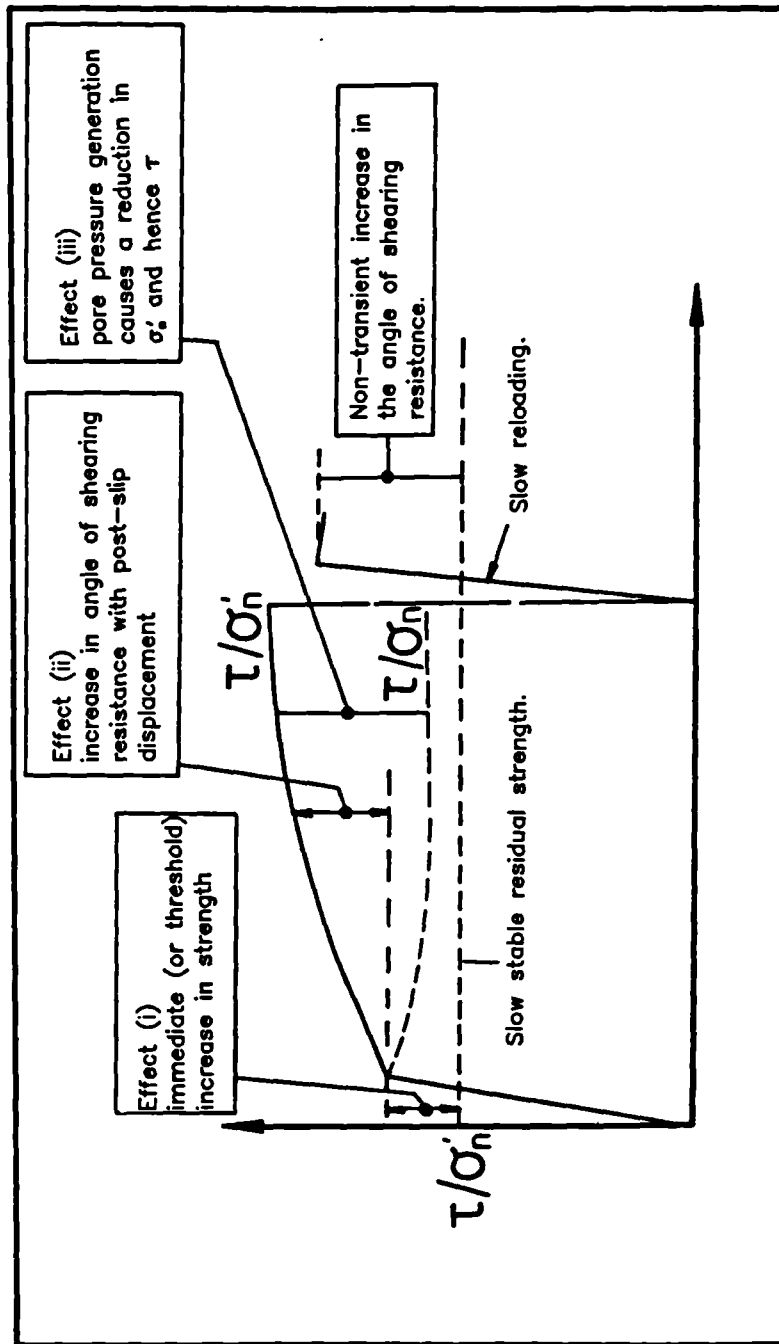


Fig. 2.3 Summary of the rate-dependent phenomena observed
(Martins, 1983)

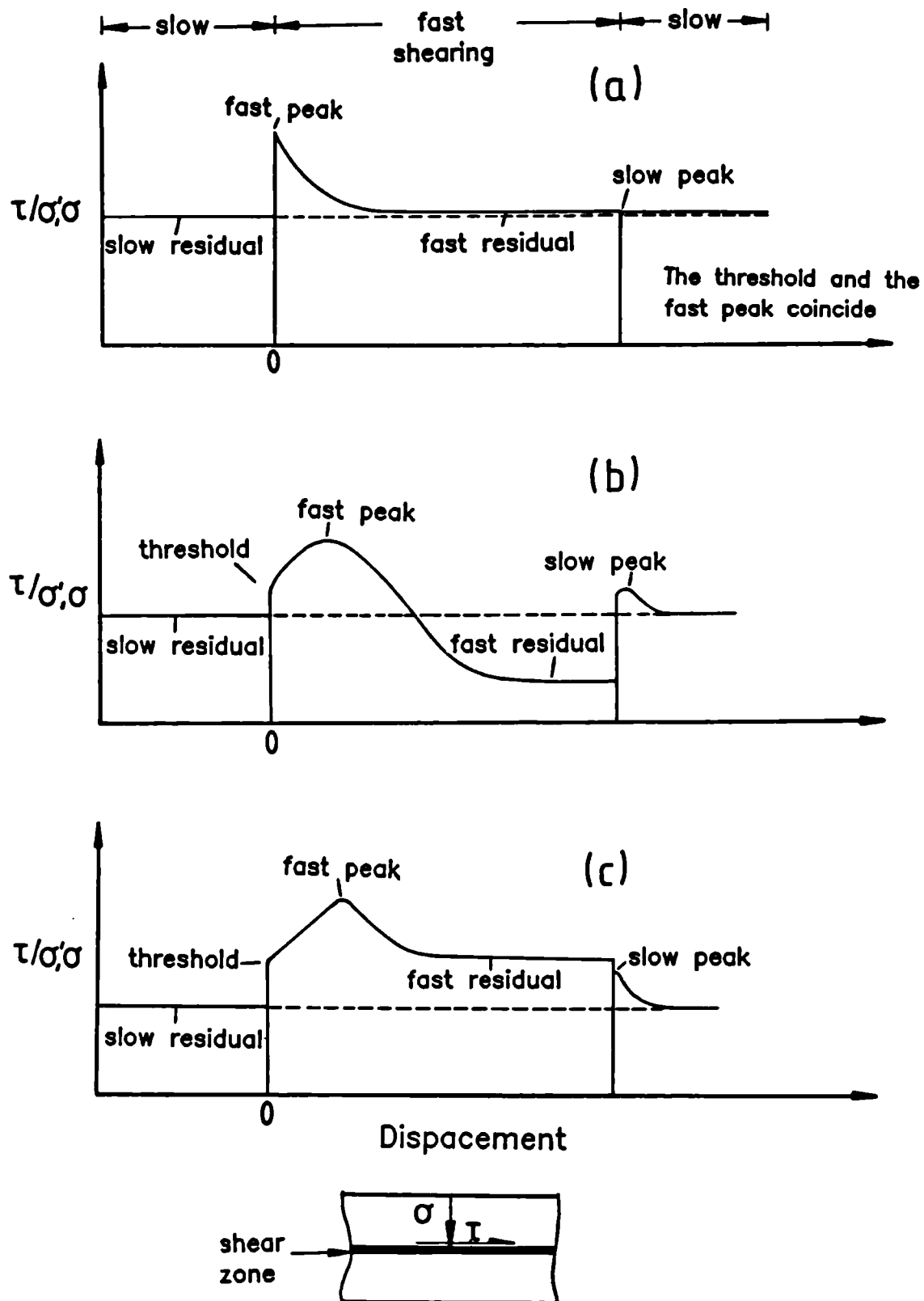


Fig. 2.4 Typical behaviours of pre-existing shear zones/surfaces at fast rates of displacements (Lemos et al, 1985 and Lemos, 1986)

CHAPTER 3

APPARATI AND TECHNIQUES USED

3.1 INTRODUCTION

The pieces of equipment used in the research include the ring shear apparatus, the direct shear box and the cone-and-plate viscometer. In the following, a brief description is given of the equipment, the testing techniques, and the procedures followed in the classification tests and the study of the microstructure of the soils tested.

3.2 THE RING SHEAR APPARATUS

3.2.1 Brief description

The design and principles of operation of the ring shear apparatus used in the present research has been described by Garga (1970) and Bishop et al (1971) (Fig 3.1 and 3.2). An annular ring-shaped specimen is ruptured on a plane of relative rotary motion while being confined laterally and subjected to a constant normal stress. The sample, confined between pairs of upper and lower confining rings, is loaded normally through annular loading platens by a dead load lever system (lever arm ratio 10:1). The lower half of the sample is carried on a rotating table driven by a worm gear. The upper half of the sample reacts through a torque arm against a pair of fixed proving rings or load cells that measure the tangential (shear) load. The gap between the upper and lower confining rings can be controlled and the vertical side friction can be measured by means of a guided linking yoke and a proving ring connected by a screw to the rigid crosshead.

Hence the gap between the rings is controlled relative to a fixed datum, and provided that the proving ring for measuring side friction is stiff enough, this gap will require little or no adjustment as the side friction changes owing to the tendency of the sample to dilate or contract as the test proceeds. The gross normal load is maintained constant during shearing and the net normal load is obtained at any stage of the test by subtracting (or adding) the measured side-friction force.

The ring shear apparatus, briefly described above, is a controlled rate of displacement device and allows the measurement of peak and residual strength of a shear zone or surface, formed either at mid-height of the annular specimen, or between the specimen and a rigid boundary referred to as an interface. The dimensions of the specimen are: outer diameter 152.4mm, inner diameter 101.6mm, and initial thickness 19 or 12.5mm depending on whether shearing takes place within the specimen or between the specimen and an interface. Two ring shear apparatus were used in the present research, identified as ring shear 1 and ring shear 2 (as with Lupini, 1981 and Lemos, 1986).

3.2.2 Interfaces

In the tests involving soil shearing against an interface, modifications similar to those described by Lemos (1986), were followed. The bottom confining rings and porous stone were substituted by the interface holder and the interface. The interface is attached to the interface holder, which in turn is attached to the base plate of the ring shear apparatus, standing on the rotating table. Two types of materials were used for the interfaces, ordinary glass and stainless steel. Stainless steel was chosen in order to avoid roughness changes caused by rusting and possible subsequent abrasion during testing. The glass-interface was glued by means of a heavy duty araldite epoxy resin on a stainless steel ring, which was attached to the interface holder.

The surfaces of the glass-interfaces were prepared by sand-blasting with sand of mesh size 0.150 - 0.425mm. Grit-blasting was used for the stainless steel-interfaces with iron grit (Grit 17) of mesh size 0.420-0.850mm. Care was taken to ensure that blasting produced the same roughness over the whole interface surface.

The micro-texture of the interface surfaces was described quantitatively by roughness measurements. Specifically, the *centre-line-average* (C.L.A) roughness parameter was used (Fig 3.3a). The centre line of a profile is defined as a line parallel to the geometrical profile, such that the area of the solid above it is equal to the area of void below it (Fig 3.3a). The departure of the profile from this centre line is identified by the C.L.A parameter which is defined as:

$$C.L.A = \frac{1}{N} \sum_{i=1}^{i=N} |Z_i| \quad (3.1)$$

where Z_i is the height of the asperities with respect to the centre line and N is the number of the asperities (Thomas, 1981). Measurements of roughness were made by means of a profilometer, a Talysurf instrument (Rank Taylor Hobson, Fig.3.3c). Roughness measurements of the interface surfaces were made along the shearing direction during testing over a sampling length (cut-off) of 2.5mm. This means that geometrical characteristics of the interface surface, such as waviness or macro-texture, longer than 2.5mm were ignored (Fig 3.3b). At least six profiles were taken from each interface surface. An average value of the roughness measurements was finally used to characterize the particular interface surface. Compressed air and cleaning with acetone were used to rid the surface from the wear debris and lubricants, before roughness measurements and testing in the ring shear apparatus.

3.2.3 Instrumentation and Measuring Systems

The instrumentation of the ring shear apparatus included load cells, proving rings, displacement transducers, pressure transducers, dial gauges, a rotational potentiometer and a thermocouple.

Shear loads were measured by means of load cells initially and proving rings afterwards for ring shear 1, and load cells for ring shear 2. Imperial College load cells with a capacity of 180 kgf (1765.8N) were employed. The principles of their design have been described by El-Ruwayih (1975). The deflection of the proving rings was measured by means of inductive displacement transducers with 2mm travel. They were provided with linear variable differential transformers (L.V.D.T), connected to modulator-demodulator cards with a D/C output with an output range of $\pm 5V$.

Side-friction on the confining rings was measured by means of flexible proving rings provided with resistive displacement transducers initially, and stiff proving rings with bonded strain gauges afterwards. Flexible proving rings had an increased sensitivity and performed reasonably well during slow shearing, as it was possible to adjust the gap size within acceptable limits. However, a sudden

increase in the gap size which could not be corrected quickly resulted in significant loss of soil during fast shearing. For this reason, the technique used by Lemos (1986) was followed initially (Tests 16, 17 and 48).

This technique consisted in controlling the size of the gap during fast shearing by keeping the side friction relatively constant. Clamps were used to stiffen the proving ring, and thus to prevent the deflection of its sides. After slow shearing of the sample to residual conditions, the sample was unloaded in shear and the clamps were introduced onto the proving ring. This resulted in a significant change in the reading of the transducer that measured the side friction, as the compliance of the system was changed. The gap was then opened to the same size as previously and slow shearing resumed until the readings for the shear loads attained a stable value again. The side friction was then back-calculated assuming slow residual conditions. This value of side friction, which was assumed to remain constant during fast shearing, was used in the calculation of the results. It was found however, that side friction did change especially during the initial stage of fast shearing (the sides of the proving ring were seen to deflect), and it was decided that the use of stiff proving rings with continuous recording of the side friction during fast shearing was preferable for the accuracy of the results. When, however, the accuracy of the measurement of the side friction by means of stiff proving rings was not considered to be satisfactory for the calculation of the results (i.e tests run at relatively low normal stresses), flexible proving rings, without clamps and continuous recording of side friction during fast shearing were used.

The vertical deformation of the sample during testing was measured by means of a pair of dial gauges for ring shear 1, and by means of a pair of dial gauges initially and resistive displacement transducers afterwards, with 25mm travel, for ring shear 2. The use of displacement transducers enabled continuous measurement of the vertical deformation of the sample. These volume changes, however, have been considered only qualitatively, since they are inaccurate, due to the loss of soil through the gap.

The size of the gap between the upper and lower confining rings was measured by means of a pair of dial gauges for both ring shear

apparati.

The relative rotational motion of the sample halves was measured by means of a vernier scale. Towards the end of the research a 10 turn rotational potentiometer with a resistance of $500\Omega \pm 5\%$ and a linearity of $\pm 25\%$ was also used for the accurate measurement of displacement during the initial stage of fast shearing.

The measurement of pore water pressure during fast shearing is discussed in chapter 6.

Temperature was measured by means of a base thermocouple of type K (Model 3750 according to B.S. 4937 DIN 43710). The hot junction of the thermocouple was connected to a unit consisting of a high-gain high-stability D/C amplifier with an output of $1\text{mV}/^\circ\text{C}$ and an indicator, by means of an extension lead. The indicator scale was calibrated in $^\circ\text{C}$.

3.2.4 Data recording systems

All the signals from the electrical transducers were recorded by a Solatron data transfer unit (DTU) initially and by an Orion data logger afterwards. The Orion data logger was linked to a mini-computer (PRIME 250) where the data could be accessed and processed (Toll and Ackerley, 1988). The shortest time interval at which a set of readings could be taken was 1 sec with the Solatron data transfer unit (DTU) and 0.1 sec per channel with the Orion data logger. For recording at shorter time intervals a light-beam recorder (U/V recorder) was used. Its specifications are given in the Appendix 1. The calibrations of the instrumentation, used for the ring shear apparatus, are given in Appendix 1.

3.2.5 Gear boxes and motor drive units

A description of the gear boxes and motor drive units has been given by Lupini (1981).

Ring shear 1 is driven by a constant speed A/C motor (M1), which is connected to a kopp variator through a belt (fig 3.4a). The kopp variator unit in turn is connected by means of a shaft to a secondary

gear box (not shown in fig 3.4a), this in turn drives the main gear box. The main gear box incorporates six gears. The reduction factor between successive gears is five. The input-output ratio of the main gear box can also be changed by altering the configuration of the two gear wheels located outside the housing of the main gear box. A chain connects the output drive of the main gear box to the worm drive of the rotating table.

Ring shear 2 is driven by a variable speed D/C motor (M2) (fig 3.4b). The motor speed is varied by means of a semi-automatic control unit on a scale from 0 to 999. The motor is connected to a gear box (engine gear box) which drives the lower axle to which two further gear boxes are connected. By connecting one of the three clutches located in the upper axle three alternative reductions of the gear boxes can be obtained. The upper axle is connected to the worm drive of the rotating table by means of a chain.

The largest rates of displacement achieved at the mid-radius of the sample with the gear boxes and motor drive units, described above, are 3.50 mm/min for ring shear 1 and 885.6 mm/min for ring shear 2 (fig A1.2 and fig A1.3). For testing at faster rates of displacement three different independent motors (M3, M4, M5), connected directly to the shaft of the worm drive of the rotating table, were used. The specifications and calibrations of the motor drive units are given in Appendix 1.

3.2.6 Sample Preparation

Most of the ring shear tests at fast rates of displacement had been performed on natural soils, received in various states. The procedures therefore followed for the sample preparation prior to testing varied considerably. They are given for each individual soil in Appendix 2. After the soil had been prepared following a particular method, it was remoulded with distilled water and brought to a plastic or a relatively dense state and left to hydrate. It was then placed in the confining rings assembly according to the procedure, described by Garga (1970) and Bishop et al (1971). The samples were placed in the ring shear apparatus in a state such that the water content during shear was close to the plastic limit in order to have dilatant behaviour during shear.

3.2.7 Testing techniques

The initial water content and the height of the test specimen after placement were determined to permit estimates of the changes in volume and water content during the consolidation stage. Consolidation (or swelling) of the test specimen usually followed a loading schedule similar to that of standard consolidation tests (each load increment was approximately equal to the existing load). However, smaller load increments were employed, whenever it was thought that a large load increase would cause a condition of plastic flow through the clearance between the confining rings. Consolidation under a load increment was completed before the next load increment was applied. From the consolidation stage, a drained rate of displacement ensuring 95% of pore water pressure dissipation at a certain displacement (1 to 2mm) was deduced according to Gibson and Henkel (1954). Raising the upper confining rings before the shearing stage caused a slight decrease in the vertical load on the sample. Shearing was started after swelling under this reduced vertical load was completed.

Slow shearing was performed at a rate equal to or lower than that deduced above. Friction developed in the gap between the confining rings and was checked periodically by closing and enlarging the gap immediately. For soils of very high plasticity (i.e blue London clay, Tests 38 and 39), because of the large displacement required to establish residual conditions, the speed-up/slow-down process suggested by Bishop et al (1971) was used. After the peak strength had been mobilized, faster rates of displacement were used with the confining ring gap closed (to minimize sample loss by squeezing). When the desired large displacement had taken place the gap was enlarged and the shearing rate was reduced to its initial value.

When a stage of fast shearing was to be performed, the torque arm was set up so that after the engagement of the gear box or the motor shaft to the worm drive of the rotating table, a small travel of the torque arm was only required for the shear load to be transmitted to the sample. This minimized the transient effects of the apparatus.

Whenever the normal stress during testing was changed, the sample was first relieved from the shear stress and then allowed to consolidate (or swell) to the new normal stress. Bishop et al (1971) observed that

although at each stage of testing a residual state had been established corresponding to a particular normal stress, a further displacement was required to re-establish the residual state for the next loading stage, if it was associated with a reduction in effective normal stress. They did not relieve the sample from shear stress, whenever the normal stress was reduced. Chandler (1977) stated that if the shear stress is left on the sample during swelling, subsequent relaxation of the torque measuring system results in a small reverse movement along the shear plane. Renewed shearing in the forward direction under the reduced stress results in a small peak in the initial part of the stress-displacement curve, representing the work required to erase the disturbance to the fabric of the shear plane produced by the reverse movement. An outline of the procedures followed during the shearing stage of the tests is given in section 4.3.2.

3.2.8 Calculation and presentation of the results

The calculation of the results was based on the assumption that both the normal and shear stress are uniformly distributed across the plane of relative rotary motion (Bishop et al, 1971). The average net normal stress, the average shear stress, and the stress ratio are given by the equations:

$$\sigma_n = \frac{W_n}{\pi(r_2^2 - r_1^2)} \quad (3.2)$$

$$\tau = \frac{3M}{2\pi(r_2^3 - r_1^3)} \quad (3.3)$$

$$\tan\phi = \frac{\tau}{\sigma_n} = \frac{3M(r_1 + r_2)}{2W_n(r_1^2 + r_1r_2 + r_2^2)} \quad (3.4)$$

where W_n is the net normal load on the sample

M is the torsional moment

r_2 is the external radius, equal to 76.2mm

r_1 is the internal radius, equal to 50.8mm

$$\text{But } W_n = 10L + P \pm W \quad (3.5)$$

$$\text{and } M = \frac{(F_1 + F_2) \cdot l}{2} \quad (3.6)$$

where L is the load in the hanger of the lever arm

P is the weight of the upper loading platen

ΔW is the side friction at the upper confining rings

F_1 and F_2 are the readings of the torque measuring instruments (proving rings or load cells),

and l is the distance between the two torque measuring instruments, equal to 480 ± 0.2 mm

Taking into account equations (3.5) and (3.6), equation (3.4) becomes:

$$\tan \phi = \frac{\tau}{\sigma_n} = 3.7298 \frac{F_1 + F_2}{10L + P \pm \Delta W} \quad (3.7)$$

The average displacement, calculated at the mean radius of the sample, is:

$$\delta = \frac{(r_1 + r_2)}{2} \cdot \theta \quad (3.8)$$

or

$$\delta = 1.1083 \theta^0 \text{ (mm)} \quad (3.9)$$

where θ is the angular displacement in radians, and

θ^0 is the angular displacement in degrees

The main types of graphs that have been used for the presentation of the results are graphs of the stress ratio, given by equation 3.7, against linear cumulative displacement (displacement run from the beginning of the test until a particular stage), and of the stress ratio against linear or logarithmic displacement (displacement run only during a particular stage).

3.2.9 Testing errors

The errors originating from the non-uniform distribution of stresses, the friction in the gap between the confining rings, and the inaccuracy of measurements are discussed in Appendix 1.

3.3 THE DIRECT SHEAR BOX APPARATUS

A standard 60mm displacement-controlled direct shear box was used. The shear force was measured by means of a load cell and recorded by a Solatron data transfer unit (DTU). Vertical and horizontal displacements were measured by means of dial gauges.

The samples were placed at relatively high water contents (liquidity index about one), since the objective of these tests was to determine the normally consolidated peak strength of soils. From the consolidation stage, a rate of displacement, ensuring 95% dissipation of pore water pressure in a displacement of 2 mm, was calculated (Gibson and Henkel, 1954). A nominal rate of displacement slower than the above calculated was used during shearing. A set of shear box tests at normal stresses of 100, 200 and 500 kPa was typically performed for each tested soil.

3.4 THE CONE-AND-PLATE VISCOMETER

A Weissenberg Rheogoniometer was used for the measurement of viscosity. The device is a plate-and-cone viscometer, highly instrumented (Van Wazer et al, 1966). It contains a stationary flat plate (upper part), and a rotating conical disc (lower part). The material under test is held between these two plates. The upper plate is held by a clamped torsion bar. An inductive transducer detects the tangential shear stress by measuring the torsional displacement of the bar. The entire upper system is strongly supported so as to prevent sideways deflections. The thrust on the lower plate, due to the normal force established in the material, is detected by a transducer. To maintain a constant gap between plates under a variable normal force, a null-point servo-system is available. The sample is placed on the lower platen, and the gap between the plates is set and controlled accurately by a transducer mounted vertically on the torsion head. Two gear boxes, driven by two synchronous motors, are coupled to the lower part of the rheogoniometer through an electromagnetic clutch, which can be energized or released at two different current settings. The cone-and-plate arrangement produces a uniform rate of shearing throughout the gap.

The tests were performed under constant temperature conditions (ambient

room temperature). Samples of the tested soil were prepared at different liquidity indexes, ranging from 0 to 1, at least 24 hr before testing. Difficulties were experienced in testing relatively stiff soils (liquidity index near 0), because of the loss of adhesion between the soil and the edges of the plate, resulting in a change of the apparent contact area and in inaccurate torque-measurements.

3.5 SOIL CLASSIFICATION TESTS

3.5.1 Index properties

Index properties were determined according to BS 1377:1975. The liquid limit was determined using the cone penetrometer method (Test 2(A), BS 1377:1975).

3.5.2 Specific gravity

The specific gravity was determined according to the method suggested by BS 1377:1975 for fine grained soils (Test 6(B), Density bottle method). An average value of three measurements is reported for each soil.

3.5.3 Particle size distribution

Particle size distributions were determined by wet sieving (Test 7(A), BS 1377:1975) and by the hydrometer method (Test 7(D), BS 1377:1975). Pretreatment of the soils for organic and calcareous matter, with hydrogen peroxide and hydrochloric acid respectively, was occasionally performed. After pretreatment the soil was filtered through a Whatman No 50 filter paper on a Buchner funnel, set up on a vacuum filtration flask, and washed thoroughly with water (note 2 on Test 7(D)). Dispersion of the soil by a sodium hexametaphosphate solution (33 gr of sodium hexametaphosphate and 7gr of sodium carbonate dissolved in 1 litre of distilled water) and mechanical shaking (usually 8 to 12 hr.) was always used.

3.6 THIN SECTIONS

Two different techniques were used for the preparation of thin sections. Specimens of sands (sandstone) and sandy silts (residual

soils from Mauritius and Japan) were impregnated with an epoxy resin (Araldite AY18 and hardener HZ18). The thin sections were prepared at the Royal School of Mines. Impregnation with Carbowax was found to cause disintegration of the specimens. The specimens were air-dried before impregnation. Specimens of clayey soils were impregnated with Carbowax 6000 in the manner described by Tchalenko (1967). After dismantling the ring shear apparatus the specimens were submerged under fresh Carbowax 6000 at 65°C for two weeks. The thin sections were prepared at the Queen Mary College, London.

The thin sections were examined by means of a standard polarizing microscope (described by Tchalenko, 1967, and supplied by Vicker instruments L.t.d), and a camera set-up with polarizer and analyzer.

3.7 SCANNING ELECTRON-MICROSCOPE

The procedure adopted in preparing the soil specimens observed under a scanning electron-microscope had been based on the techniques described and used by Gillot (1969, 1976) and Smart and Tovey (1982).

The specimens were air-dried and subsequently fractured in the required direction, apart from the views of slickenside surfaces. Because of the volumetric shrinkage, which causes considerable densification and fabric changes during air-drying, the technique of substituting the pore water with a low surface tension liquid (acetone) before air-drying was used to reduce drying shrinkage. Loose particles were removed by a jet of compressed air. Approximately 1 cm cube specimens were cemented on stubs made of brass (non magnetic and conductive) by means of a heavy duty araldite resin and covered with gold palladium to make them electrically conductive and prevent surface charging. A silver paint was then added to the sides of the sample, where these met the stub surface to improve reflectivity. The scanning electron-microscope used for the study of the microstructure of the soils was a low voltage JEOL JSM T200 scanning electron-microscope.

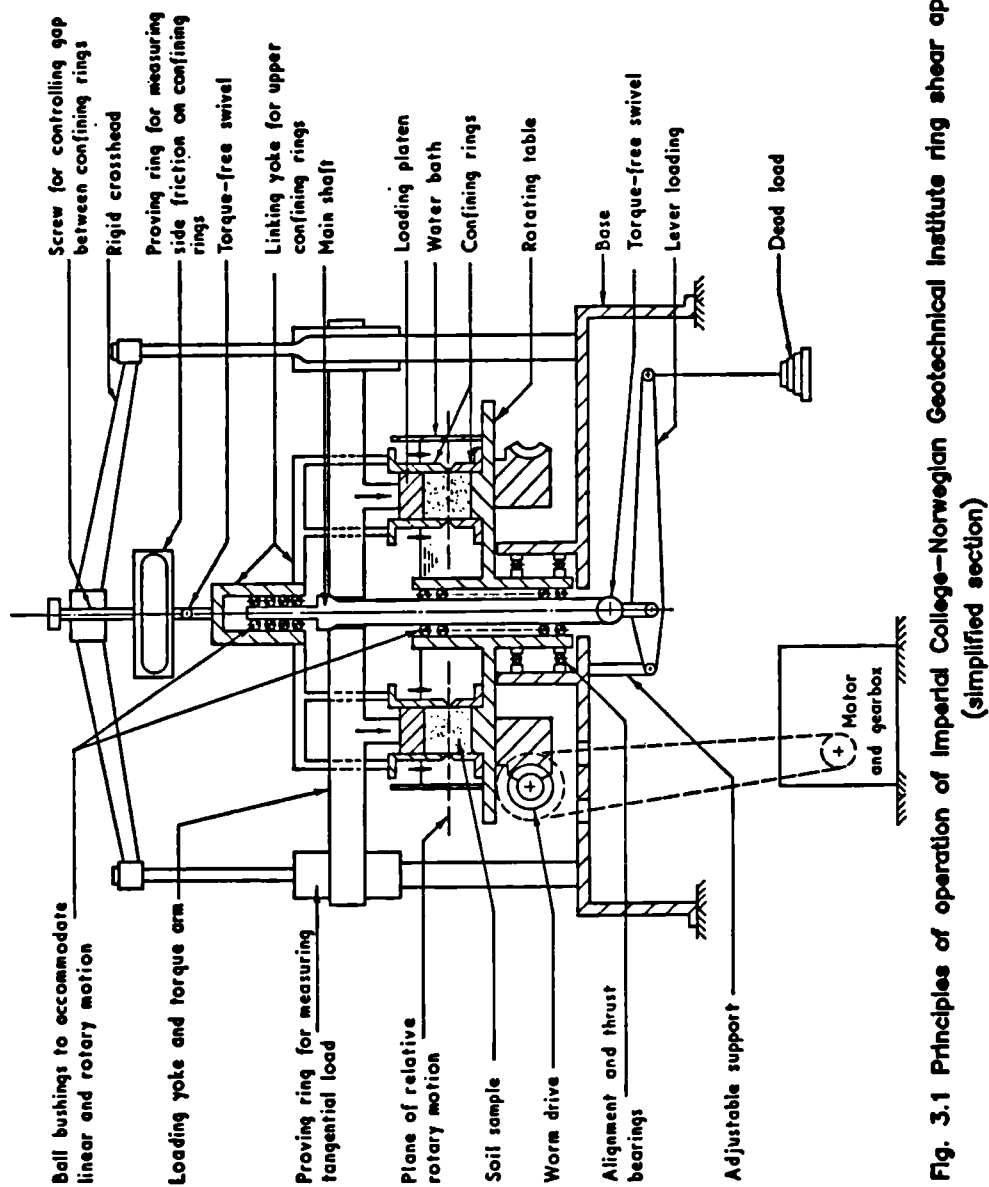


Fig. 3.1 Principles of operation of Imperial College-Norwegian Geotechnical Institute ring shear apparatus (simplified section)

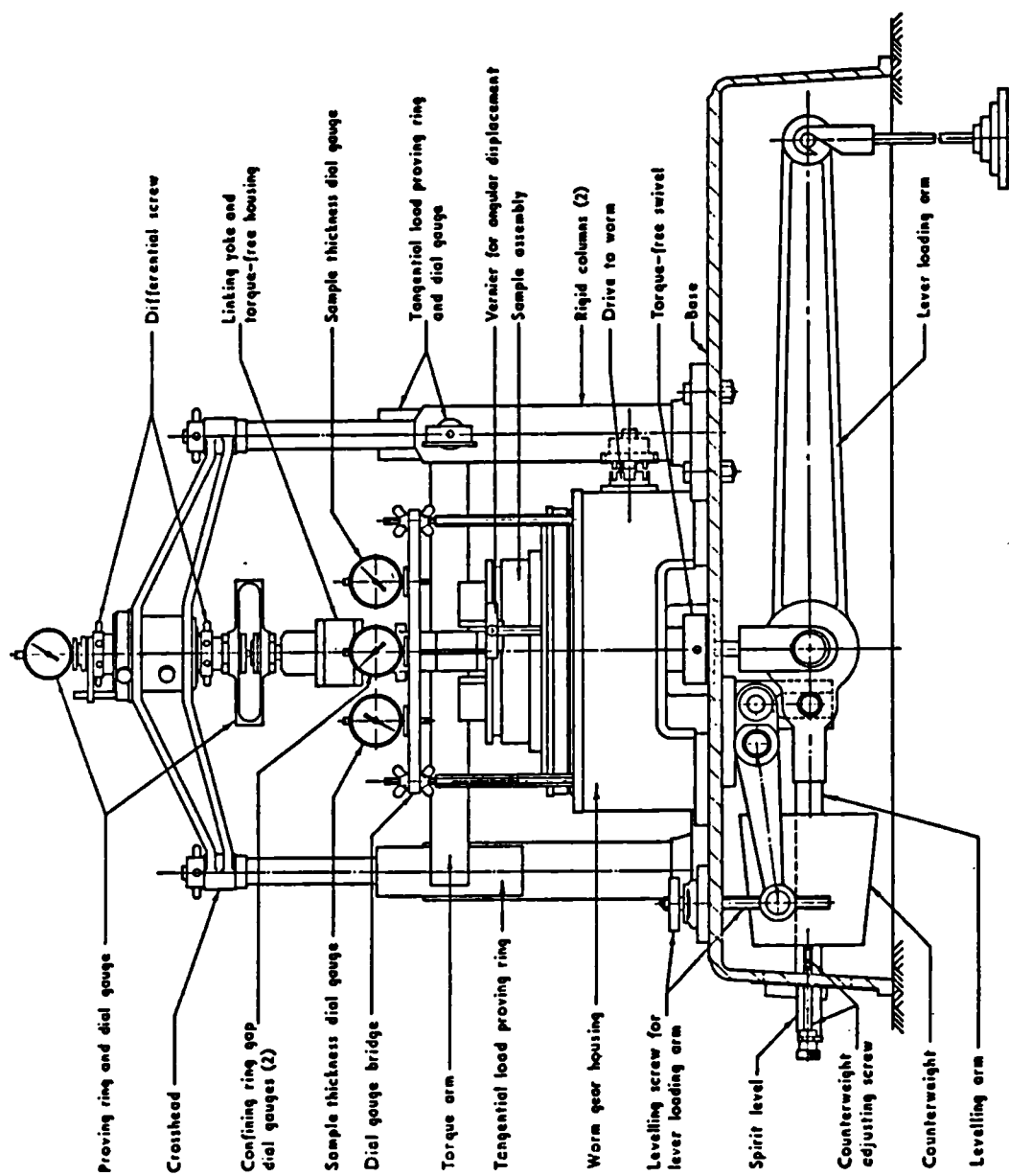


Fig. 3.2 General layout of ring shear apparatus

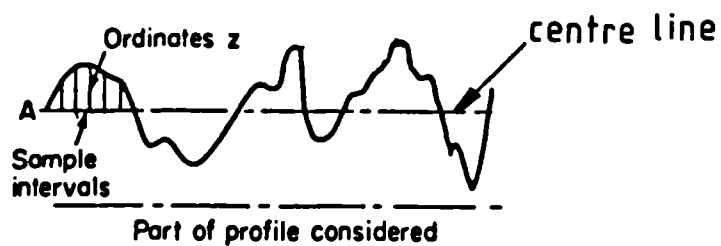


Fig. 3.3a Definition of centre-line-average (C.L.A) roughness parameter

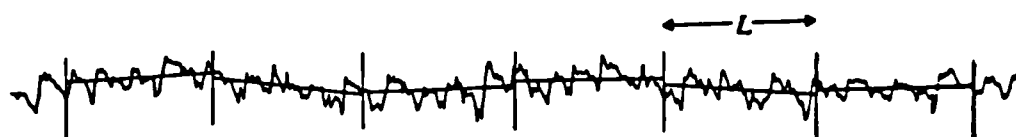


Fig. 3.3b A surface profile split into sampling lengths L

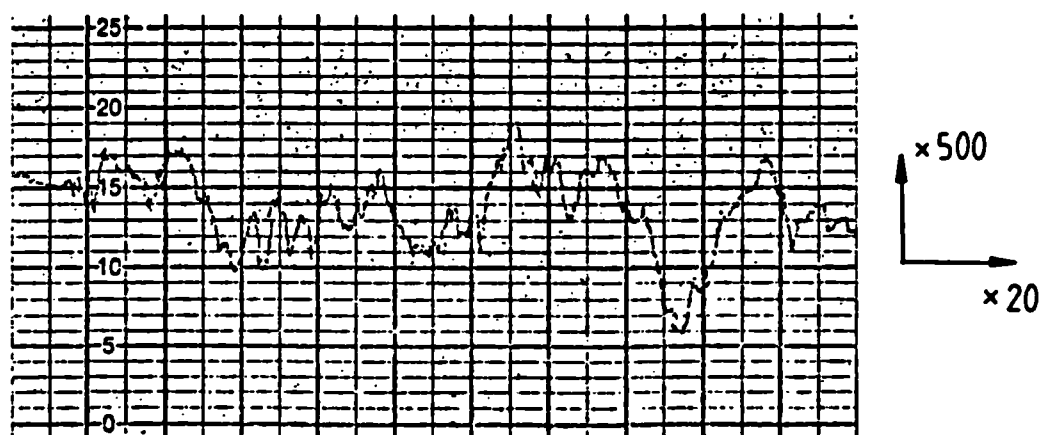


Fig. 3.3c A typical surface profile, as measured by Talysurf instrument.

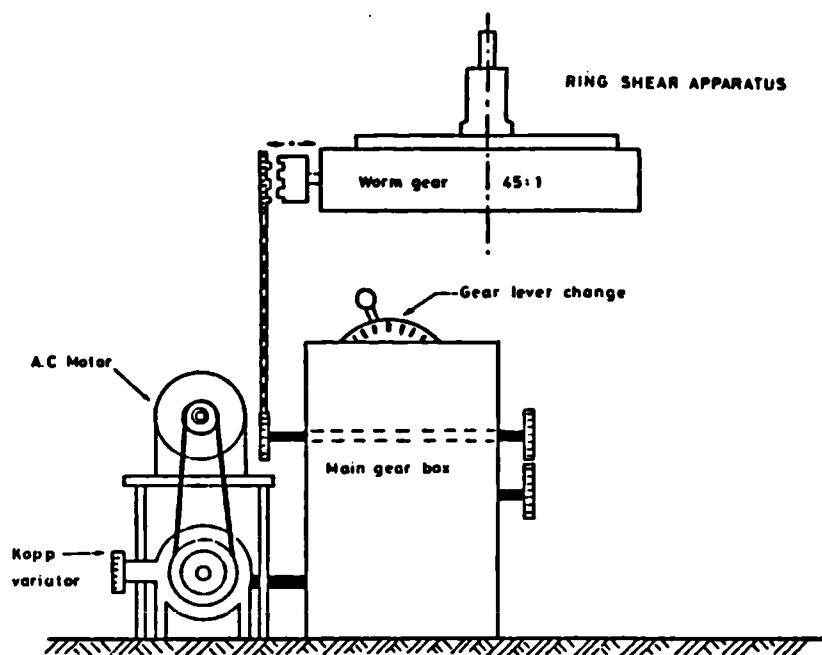


Fig. 3.4a Ring shear apparatus 1, gear box arrangement (motor 1)

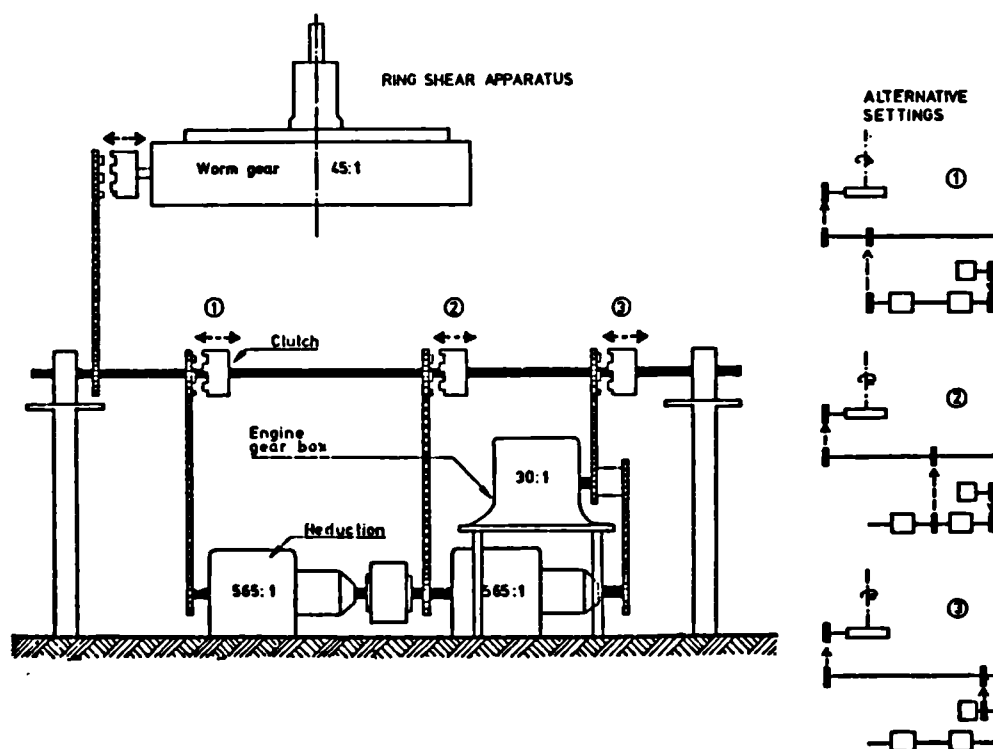


Fig. 3.4b Ring shear apparatus 2, gear box arrangement (motor 2)

CHAPTER 4

SUMMARY OF RING SHEAR TESTS AND ANALYSIS OF TEST RESULTS

4.1 INTRODUCTION

It was mentioned in chapter 1 that previous investigations at Imperial College have corroborated the existence of rate effects on the residual strength of soils. In the course of this chapter, reference is made to all the ring shear tests involving fast shearing of soils, carried-out by the author, Lemos (1986), Martins (1983) and Lupini (1981).

4.2 SUMMARY OF RING SHEAR TESTS

The tested soils are divided into eight main groups and are summarized in Table 4.1. Each test is described individually in Appendix A2, and identified by a number.

Group 1 : Soils from Kalabagh dam site (Tests 1 to 22).

Group 2 : Tillis (Tests 23 to 25).

Group 3 : Residual soils (Tests 26 to 30).

Group 4 : Other natural soils (Tests 31 to 34).

Group 5 : Artificial soils (Tests 35 and 36).

Group 6 : Soils of very high plasticity (Tests 37 to 42).

Group 7 : Sands (Tests 43 to 47).

Group 8 : Tests carried-out to investigate the loss of strength during fast shearing (Tests 48 to 52).

At the start of the present research, most of the available data on the effect of rate of shearing on the residual strength of soils were from tests on soils from the Kalabagh dam site (Tests 1 to 15). They had been performed by Lemos (1986) in connection with the seismic design of the Kalabagh Dam project, allowing for the presence of pre-existing shear surfaces in the foundations of the dam. Three more soils from the same site were tested by the author, a sandstone (Test 16), a clayey siltstone (Tests 17 to 21), and a claystone (Test 22). To examine whether the effects of fast shearing on the residual strength of soils from the Kalabagh dam site were unique, or of a more general character, tests on other natural soils were carried out thereafter.

The development of the North Sea oil industry prompted the investigation of the engineering behaviour of tills present in the foundation of offshore structures. A major research programme had been carried-out at Imperial College during the period of 1975-1983, using low and medium plasticity terrestrial tills of similar properties to the North sea deposits. A series of ring shear tests on Lower Cromer Till, carried out to determine the mode of drained residual shear has been reported by Hight et al (1979) and Lupini (1981). Lupini (1981) reported an additional ring shear test on Lower Cromer Till as a special investigation into rate effects. The rate effect investigation showed a clear trend towards a decrease of the residual strength with an increasing rate of displacement. An additional test on Lower Cromer Till (Test 23) was performed by the author in order to verify the drop of strength below the slow residual value during fast shearing, observed by Lupini (1981), and establish the normal stress dependence of the rate effects on the residual strength of the soil. The effects of the rate of shearing on the residual strength of two other glacial tills have been investigated by Lemos (1986). Lemos (1986) conducted Tests 24 and 25 on the Magnus Till and the weathered Cowden Till respectively.

Lupini (1981) reported Test 26 on a weathered volcanic soil from Fiji, during which fast shearing of the pre-formed shear zone was performed. The test was carried out as a part of an investigation for the construction of an embankment dam in Fiji. The soil, in situ, contained sub-vertical slickensided shear surfaces. Because Fiji is a region of considerable seismicity, the study of the behaviour of this material under different shearing rates became of interest. An additional test (Test 27) on the weathered volcanic soil from Fiji was conducted by the author in order to verify and extend Lupini's results.

Test 28 on a weathered volcanic soil from Mauritius was then performed. The soil was of intermediate plasticity and its behaviour became of interest, as existing data on soils of the same plasticity suggested either a negligible rate effect or a drop of strength to less than the slow residual value during fast shearing.

The results from the tests, presented so far, on soils showing a drop of strength during fast shearing to less than the slow residual value are unrelated to field observations of earthquake induced landslides or

other fast movements. In an attempt to relate the experimental data with field observations, Test 29 was carried out on a weathered volcanic soil from Japan. The soil was taken from the exposed shear surface of the huge Mt. Ontake landslide, Japan, triggered by the Naganoken-Seibu earthquake of 1984. The results from Test 29, indicating a negligible rate effect, did not corroborate the anticipated loss of strength at increasing rates of displacement, required to interpret the high mobility of the landslide. This apparent discrepancy suggested either a difference in the material properties, or an additional agency in the field. With this in view and the fact that the results previously obtained suggested that a small increase of the clay fraction of the soil might change its behaviour during fast shearing from a rate-independent to a rate-dependent one, the clay fraction of the soil was increased by removing fines from part of the sample and adding them to the remainder. A further test was then performed (Test 30).

The series of tests on natural soils continues with the tests on soils from Group 4. Martins (1983) performed Tests 31 and 32, on weathered and mechanically degraded Tuff respectively, in order to determine the dynamic behaviour of the materials, sheared along pre-existing shear surfaces, and in particular to determine the immediate increase of strength prior to slippage. Lemos (1986) carried out Tests 33 and 34 on soils from the Senerchia and Buoninvetre slides, South Italy, re-activated immediately after the Irpinia earthquake of 1980.

Having obtained the results from the previously presented soils, it seemed that identification of the soils showing a drop of strength during fast shearing below the slow residual value through their clay fraction or their mode of slow shear was uncertain. While these soils had generally low to intermediate clay fractions and a turbulent or transitional shear mode, this was not sufficient to identify the type of the shearing behaviour. To investigate the effect of grading, two tests on artificially prepared soils were conducted. Test 35 was performed on a mixture of an artificial silt and spheswhite kaolin. The purpose was to simulate the grading of the clayey siltstones from the Kalabagh dam site. Test 36 was performed on a sand-kaolin mixture.

A series of tests was then conducted on clay and sand in order to assess the rate effects at the two ends of the plasticity spectrum.

Lupini (1981) reported Test 37 on kaolin, in which the effect of fast shearing on the residual strength was investigated to a limited extent. Tests 38 and 39 were carried out by the author on blue London clay. Tests 40,41 and 42 involved shearing of blue London clay against a stainless steel interface. The objective was to compare the test results to those of a current research programme on the behaviour of model piles jacked in overconsolidated clays in-situ (Bond, 1989).

In order to investigate the low end of the plasticity spectrum, Test 44 was performed on a medium sand. Because of the strong stick-slip behaviour of the sand particles during shearing and the difficulty in controlling the size of the gap (once the grains entered the gap between the confining rings, the gap kept opening), a fine sand was then tested (Tests 44 to 47).

Finally, a series of special tests, presented as Group 8, were carried out to investigate the causes of the drop of strength during fast shearing below the slow residual value. The results of these tests are discussed in chapter 6.

4.3 TESTING PROCEDURES

Testing procedures for individual tests are given in Appendix 2.

In the *standard testing procedure*, a sample was consolidated (section 3.2.7) and then sheared slowly to establish a shear zone and to measure the slow drained residual strength. Slow drained rates of displacement from 0.003 to 0.102 mm/min were used to ensure dissipation of any excess pore water pressure in a shearing displacement of 1 to 2mm.

After the slow drained residual strength had been established, the shear stress was reduced to zero and fast shearing was performed for a displacement of about 200 mm. Immediately after fast shearing, the shear load was either reduced or removed completely to prevent any displacement by creep, except in the tests on the sandy soils (Tests 16,23,28 to 30 and 36) and the sands (Tests 43 to 47), in which shear unloading is associated with a decrease of the volume of the sample. This decrease of volume can affect significantly the value of the slow peak strength, measured during slow reshearing of the sample. The sample was allowed to consolidate fully prior to further shearing.

After the pause, shearing to the residual state was commenced again at the slow standard rate of displacement. Fast shearing at a different rate of displacement was performed again, and so on. Once shearing had been completed at the first normal stress, the sample was unloaded in shear and then subjected to a new normal stress, so that the influence of normal stress on the rate effects could be assessed. In most of the tests, shearing was performed at normal stresses between 200 and 500 kPa (usually at about 200 and 500 kPa). In some tests, even lower normal stresses were used. Table 4.2 summarizes the normal stresses and the slow rates of displacement used in the tests.

In order to investigate further the nature of the rate effects, four testing procedures A, B, C and D additional to the standard one were used.

Procedure A consisted in testing a sample either without a preformed shear zone or with one partly formed at a slow drained rate of displacement. The objective was to investigate the effect of the previously mobilized slow strength (or starting strength) on the threshold strength (Fig.2.4). Table 4.3 summarizes the tests in which this procedure was implemented.

Procedure B consisted in reshearing a sample at a fast rate of displacement either immediately after a fast stage or after allowing the sample to partially or fully consolidate. Table 4.4 summarizes the tests in which this procedure was implemented.

Procedure C consisted in reshearing a sample slowly (not necessarily at a drained rate of displacement) immediately after a fast stage, and monitoring the strength so that pore pressure effects could be investigated indirectly. Table 4.5 summarizes the tests in which this testing procedure was used.

Procedure D involved prolonged fast shearing of a sample in order to investigate the possible role of pore water pressure dissipation at fast rates of displacement. Table 4.6 summarizes the tests in which this procedure was used.

Procedures B,C and D were used in tests on soils showing a drop of strength to less than the slow residual strength and they are discussed

in chapter 6.

4.4 ANALYSIS OF THE TEST RESULTS

In this section, the drained residual shear modes of the tested soils are identified and the test results for the variation of the residual strength with the rate of displacement are presented. The results from each group are presented in order of increasing plasticity of soils.

The identification of the drained residual shear modes (turbulent, transitional and sliding) is made by means of the method illustrated in Fig. 4.1, and by examining the structure of the shear zones after testing. The full procedure consists of a minimum of two stages. The first one involves slow drained shearing to the residual state of a normally consolidated sample (stage 1). The second one involves slow reshearing of the sample at a lower normal stress (stage 2). The types of the results to be expected, depending on the orientability of the soil, are indicated in Fig. 4.1. In most of the tests, the full procedure was not carried out and the shear mode is determined from the value of the drained residual coefficient of friction, and the structural and microstructural observations made after testing.

The changes of the residual strength during fast shearing have been assessed with respect to the slow residual strength, measured at the slow standard rate of displacement. In addition to the test results concerning the variation of the residual strength with the rate of displacement, results for the other typical strengths (threshold, fast peak and slow peak strength after fast shearing, Fig. 2.4) are also presented.

Results from shear box tests are presented in Appendix 3. The results from the ring shear tests are presented in the form of drained residual coefficient of friction versus normal effective stress, and ratios of typical strengths to slow residual strength versus log rate of displacement.

4.4.1 Soils from Kalabagh dam site

The sandstone (Fig. 4.2 and 4.3) showed the turbulent shear mode. Its residual and normally-consolidated peak strengths coincided (Fig. 4.3).

No significant change of the residual strength at increasing rates of displacement was observed (Fig. 4.3).

The siltstone (Fig. 4.2 and 4.4) showed the transitional shear mode tending towards the turbulent mode. It had identical residual and normally-consolidated peak strengths at the normal stress of 200 kPa, but a slightly lower residual strength than the normally-consolidated peak value at the normal stress of 500 kPa (Fig. 4.4). The residual strength was not affected significantly by changes of the rate of displacement (Fig. 4.4). A drop of strength below the slow residual value of the order of 10% was observed at the normal stress of 200 kPa at 400 mm/min (Fig. 4.4b).

The clayey siltstone C (Fig. 4.5 and 4.6) showed the transitional shear mode tending towards the turbulent mode. It exhibited slight brittleness due to clay particles orientation (Fig. 4.6). The clayey siltstones A, B and D (Fig. 4.7 to 4.12) showed transitional shear modes. They exhibited considerable brittlenesses due to clay particles orientation (Fig. 4.8, 4.10 and 4.12). Brittle behaviour was observed for all the clayey siltstones, A, B, C, and D, when they were sheared in an overconsolidated state, after the slow residual strengths had been established (Fig. 4.5, 4.7, 4.9 and 4.11). Observations of thin sections of the soils after the ring shear tests showed shear zones bounded by discontinuous shear surfaces (Plate 2).

Fig. 4.6, 4.8, 4.10 and 4.12 illustrate results from the tests on the clayey siltstones C, A, B and D respectively. Fig. 4.12c shows results from Test 49 on the clayey siltstone D with its clay fraction reduced and Fig. 4.12d shows results from tests involving shearing of the soil against an interface. All the clayey siltstones showed an increase of the residual strength at increasing rates of displacement up to 10 to 12 mm/min, followed by a drop below the slow residual strength at higher shearing rates.

The claystone D (Fig. 4.13 and 4.14) showed the transitional shear mode tending strongly towards the sliding mode. It exhibited considerable brittleness due to clay particles orientation (Fig. 4.14). Slight brittleness was observed when the soil was resheared at a lower normal stress, after the slow residual strength had been established. The soil showed a drop of the residual strength to less than the slow

residual value at 9.5 mm/min at all the normal stresses, and at 95 mm/min at the normal stresses of 235 and 500 kPa (Fig.4.14). The gain in residual strength, observed at 360 mm/min at all the normal stresses, was followed by a drop below the slow residual value at 2600 mm/min. The displacements involved in some fast stages are rather short and it is questionable whether a steady state had been always established. The results from Test 52 which involved soil shearing against a glass interface and in which a fast steady state had been established showed a drop of strength to less than the slow residual value at 400 mm/min and at the normal stress of 200 kPa (Fig. 4.14b).

The claystones B and C (Fig. 4.15 to 4.17) showed transitional shear modes tending strongly towards the sliding mode. They showed considerable brittlenesses due to clay particles orientation (Fig. 4.16 and 4.17). The soils were not sheared in an overconsolidated state, after the slow residual strengths had been established. A thin section from the sample of the claystone B after the ring shear test indicated a shear zone with discontinuous shear surfaces at its boundaries. Visual observations of the sample of the claystone C after the ring shear test showed that more than one shear surfaces were present within the sample. The sample separated either through an unstriated but highly polished shear surface or through a shear surface which was striated but un-polished. The claystone C showed a drop of the residual strength below the slow residual value during fast shearing, except at 370 mm/min and at a normal stress of 200 kPa (Fig. 4.16). The claystone B showed a drop of the residual strength below the slow residual value at 9.5 mm/min, followed by an increase at higher shearing rates (Fig. 4.17). It is not certain, however, whether a steady state was established during some of the fast stages.

The claystone A (Fig. 4.18 and 4.19) showed the sliding shear mode. It showed considerable brittleness due to clay particles orientation (Fig. 4.19). No brittleness was observed when the soil was resheared at a lower normal stress, after the slow residual strength had been established (Fig. 4.18). Visual observations of the sample after the ring shear test showed a polished and striated circumferentially shear surface. A drop of the residual strength below the slow residual value was observed at 9.5 mm/min, followed by an increase at higher shearing rates (Fig. 4.19). The displacements involved in some of the fast stages were rather small and it is questionable whether the fast

residual state had been always established.

4.4.2 Tills

The Lower Cromer Till (Fig. 4.20 and 4.21) showed the turbulent shear mode. Despite the sample being overconsolidated when it was first sheared in the ring shear apparatus, it showed a non-brittle behaviour. It exhibited brittleness whenever the normal stress was reduced after the slow residual state had been established (Fig. 4.20). Test results are plotted in Fig. 4.21 together with results from a test, carried-out by Lupini (1981). The soil showed a significant drop of the residual strength to less than the slow residual value, at rates equal or faster than 133 mm/min, the results corroborating those reported by Lupini (1981).

The Magnus Till showed probably the transitional shear mode strongly tending towards the turbulent mode. It had a rather high drained residual friction angle (about 27° , Fig. 4.22) and exhibited a slight brittleness when it was sheared through the peak to the residual strength from a remoulded state. No thin sections or studies of brittleness were made. The soil showed an increase of the residual strength above the slow residual value when sheared at 133 mm/min and at a normal stress of approximately 500 kPa (Fig. A2.26).

The weathered Cowden Till (Fig. 4.22 and 4.23) showed the transitional shear mode tending towards the turbulent mode. The soil showed brittleness when it was sheared in an overconsolidated state, after the slow residual strength had been established (Fig. 4.22). A thin section of the sample after the ring shear test showed a shear zone with discontinuous shear surfaces at its boundaries. The soil showed a slight decrease of the residual strength at increasing rates of displacement up to 0.06, 0.02 and 0.015 mm/min at the normal stresses of 50 kPa, 100 kPa, and 150 kPa and 200 kPa respectively, followed by an increase of the residual strength at higher rates (Fig. 4.23).

4.4.3 Residual soils

The residual soil from Mauritius (Fig. 4.24 and 4.25) showed the turbulent shear mode. It had a high drained residual friction angle (33°) and showed considerable brittleness when it was sheared in an

overconsolidated state, after the slow residual state had been established (Fig.4.24). Slight increases and decreases of the residual strength at increasing rates of displacement were initially observed at the normal stress of 450 kPa and 220 kPa respectively. A transition, however, in the type of soil behaviour from a rate-independent to a rate dependent one was observed, due to the degradation of the particles within the shear zone (Plate 5, photographs 33 to 35). A significant drop of strength below the slow residual value was then observed at both the normal stresses. The degradation did not seem to have affected the drained residual coefficient of friction which indicates that a change in the shear mode had not occurred.

The residual soils from Japan (Fig. 4.26 to 4.29) showed turbulent shear modes. Both had high drained residual friction angles. They showed considerable brittleness when sheared in an overconsolidated state after the slow residual strengths had been established (Fig. 4.26 and 4.28). Observations of a thin section from the soil used in Test 29 showed a well-defined shear zone, but there was no evidence of preferred particle orientation or of any shear surfaces at the boundaries of the shear zone (Plate 5, photographs 36 to 38). A quite neutral behaviour with respect to the rate of shearing was observed in Test 29 (Fig. 4.27). The increase of the clay fraction of the soil used in Test 30, however, caused a change of the type of soil behaviour from a neutral to a rate-dependent one (Fig.4.29). A drop of strength below the slow residual value at rates of displacement, above 100 mm/min, was observed at the normal stresses of 260 kPa and 130 kPa.

Both residual soils from Fiji (Fig. 4.30 and 4.31) showed sliding shear modes. They showed a non-brittle behaviour when they were sheared in an overconsolidated state, after the slow residual strengths had been established (Fig. 4.30). Observations of the structure of the soils after testing indicated a shear zone of variable thickness, bounded by two shear surfaces. The soil used in Test 26 showed an increase of the residual strength at rates of displacement up to 17.72 mm/min and at a normal stress of 176.7 kPa, followed by a drop of strength below the slow residual value when the shearing rate was increased to 177.2 mm/min (Fig. 4.31b). The soil used in Test 27 showed a drop of strength below the slow residual value too (Fig. 4.31). The drop of strength was of the same order of magnitude to that showed by the soil used in Test 26, despite the soils having different drained residual

coefficients of friction (Fig. 4.31b).

4.4.4 Other natural soils

The mechanically degraded and the weathered Tuff showed the transitional and the sliding shear mode respectively, as indicated by the values of the drained residual friction angles, 17.5° and 9° respectively at a normal stress of 300 kPa (Fig. 4.32). Visual observations of the shear surfaces showed a striated and slightly polished surface for the mechanically degraded Tuff and a highly polished surface for the weathered Tuff. Fig. 4.33 illustrates results from Test 31 on the weathered Tuff.

The soils from Buoninvetre and Senerchia slides (Fig. 4.34 to 4.37) exhibited sliding shear modes. They both showed considerable brittlenesses due to clay particles orientation (Fig. 4.35 and 4.37). Slight brittlenesses were observed when the soils were sheared in an overconsolidated state after the slow residual strengths had been established (Fig. 4.34 and Fig. 4.36). Post failure observations of the samples showed well-defined, highly polished shear surfaces for both soils. A slight decrease of the residual strength below the slow residual value was observed at rates of displacement between 0.1 and 1 mm/min at the normal stress of 490 kPa for the soil from the Buoninvetre slide, and at 3.69 mm/min at the normal stress of 920 kPa for the soil from the Senerchia slide. The slight decrease of the residual strength was followed by an increase at higher shearing rates up to 100 mm/min. Some indication exists for a decrease again of the residual strength to less than the slow residual value at rates faster than 100 mm/min for the soil from Buoninvetre slide (Fig. 4.35a). It is questionable whether a steady state had been reached during some of the fast stages.

4.4.5 Artificial soils

Both the clayey silt and the sandy clay (Fig. 4.38, 4.39 and 4.40) showed turbulent shear modes. They had high drained residual friction angles and they showed brittle behaviours when they were sheared in an overconsolidated state after slow the residual strengths had been established (Fig. 4.38). No separation of the test specimens on shear zones was observed.

The rate of shearing had a negligible effect on the residual strength of the clayey silt at a normal stress of 450 kPa. At a lower normal stress of approximately 60 kPa, the initially observed increase of the residual strength, up to a rate of 360 mm/min, was followed by a drop to less than the slow residual value at higher shearing rates (Fig. 4.39).

The sandy clay showed generally an increase of the residual strength at fast rates of displacement (Fig. 4.40). A slight decrease of the residual strength below the slow residual value was observed at 133 mm/min at the normal stress of 200 kPa.

4.4.6 Soils of very high plasticity

Both kaolin and blue London clay (Fig. 4.41, 4.42 and 4.43) showed sliding shear modes. Kaolin exhibited considerable brittleness when it was first sheared from a normally-consolidated state (Fig. A2.43b). Post-failure observations of its microstructure showed that the clay particles were strongly orientated in the direction of shear. The blue London clay showed well defined, highly polished shear surfaces with light striations and a non-brittle behaviour when it was sheared in an overconsolidated state, after the slow residual strength had been established (Fig. 4.41).

Both soils showed slight to moderate increases of the residual strength during fast shearing (Fig. 4.42 and 4.43). The increase of the residual strength of the blue London clay was lower than that observed for the more plastic clays, tested by La-Gatta (1970) and Lupini (1981). Fig. 4.43c to 4.43d show results from Tests 40, 41 and 42 on the blue London clay, sheared against a stainless steel interface.

4.4.7 Sands

The medium and fine sands showed turbulent shear modes (Fig. 4.44). Their drained residual friction angles remained constant irrespectively of the rate of displacement (Fig. 4.45). A decrease of the residual strength of the order of 10% was observed at 6000 mm/min and at a normal stress of 110 kPa for the fine sand. It is not certain whether this loss is due to the variation of the experimental results or due to an on-set of a rate-dependent behaviour.

Table 4.1 Ring shear tests involving fast shearing of soils

Test No	Site and soil type	Liquid limit (%)	Plasticity Index (%)	Clay Fraction % < 2 μ m	Reference
<u>Soil from Kalabagh dam site</u>					
1,2	claystone A	62	36	52	Lemos(1986)
3-5	clayey siltstone A	39	18	20	>>
6-8	clayey siltstone A	49	27	27	>>
9	claystone B	56	31	40	>>
10	claystone C	55	28	39	>>
11	siltstone	40	13	3	>>
12-14	clayey siltstone B	44	21	8	>>
15	clayey siltstone C	39	17	10	>>
16	sandstone	-	NP	1	This study
17-21	clayey siltstone D	44	24	25	>>
22	claystone D	53	26	37	>>
<u>Tills</u>					
23	Lower Cromer Till	26	14	22	This study
24	Magnus Till	32	15	37	Lemos (1986)
25	weathered Cowden Till	39	21	31	>>
<u>Residual soils</u>					
26	Fiji	90	37	40	Lupini (1986)
27	Fiji	85	22	55	This study
28	Mauritius	49	6	11	>>
29	Japan	63	9	10	>>
30	Japan	63	13	14	>>
<u>Other natural soils</u>					
31	weathered Tuff	58	17	-	Martins (1981)
32	mechanically degraded Tuff	50	14	-	>>
33	Senerchia	52	28	37	Lemos (1986)
34	Buoninvetre	53	26	39	>>
<u>Artificial Soils</u>					
35	clayey silt	25	9	18	This study
36	sandy clay	22	10	23	>>
<u>Soils of very high plasticity</u>					
37	kaolin	72	36	82	Lupini (1981)
38-42	blue London clay	71	45	53	This study
<u>Sands</u>					
43	medium sand		NP		This study
44-47	fine sand		NP		>>
<u>Tests to investigate the loss of strength during fast shearing</u>					
48	clayey siltstone D	44	24	22	This study
49	>> D	41	21	13	>>
50,51	clayey siltstone D	44	24	22	>>
52	claystone D	53	26	44	>>

Note: Tests 8, 14, 40 to 42, 48 and 50 to 52 involved soil shearing against an interface.

Table 4.2 Summary of normal stress levels and slow rates of displacements used in the tests

Test No	Site and Soil Type	Maximum consolidation normal stress (kPa)	Operating nominal normal stress (kPa)						slow rate of displacement (mm/min)
			1st	2nd	3rd	4th	5th	6th	
1	claystone A	900.0	525.0	206.0					0.01
2	>>	988.0	241.0	593.0					0.01,0.014
3	clayey siltstone A1	922.0	593.0	242.0					0.01
4	>>	878.0	557.0						0.01
5	>>	878.0	241.0						0.0022
6	clayey siltstone A2	988.0	593.0						0.01
7	>>	1010.0	593.0						0.004,0.007,0.01
8	>>	590.0	590.0						0.006
9	claystone B	987.0	214.0	535.0					0.01
10	claystone C	992.0	248.0	544.0					0.01
11	siltstone	992.0	248.0	493.0					0.01
12	clayey siltstone B	1037.0	609.0	200.0					0.01
13	>>	992.0	608.0	200.0					0.01
14	>>	594.0	594.0						0.03
15	clayey siltstone C	1037.0	688.0	200.0					0.01,0.0129,0.0135, 0.014,0.015, 0.016
16	sandstone	1000.0	500.7	242.6					0.01
17	clayey siltstone D	1000.0	527.9	264.6					0.01
18	>>	966.8	264.6						0.0053
19	>>	966.8	264.6						0.014
20	>>	966.8	264.6						0.01
21	>>	264.6	264.6						0.01
22	claystone D	966.8	527.9	264.9	132.9	89.0	264.6	527.9	0.01
23	Lower Cromer Till	966.8	571.8	484.0	264.6	132.9	67.1		0.025
24	Magnus Till	530.0	530.0						0.0169,0.017,0.058, 0.086
25	weathered Cowden Till	50.0	50.0	100.0	150.0	200.0			0.0001,0.005,0.0035 0.01,0.015,0.05,0.1 0.37,0.5
26	Fiji	352.0	352.0	177.0					0.000025,0.00025, 0.0023,0.157
27	Fiji	922.9	484.0	220.7					0.0153
28	Mauritius	264.6	264.6	527.9	264.1				0.01
29	Japan	264.6	264.6	527.9	440.1				0.01
30	Japan	527.9	527.9	264.6	132.9				0.0925
31	weathered Tuff	1000.0	300.0	100.0					0.006
32	mechanically degraded Tuff	1000.0	100.0	300.0					0.006
33	Senerchia	920.0	920.0	485.0					0.003
34	Buoninvetre	960.0							
35	clayey silt	966.8	527.9	89.0					0.0102
36	sandy clay	264.6	264.6						0.102
37	kaolin	352.0	352.0	519.0					0.0001,0.001,0.015
38	blue London Clay	922.9	527.9	440.1					0.00625
39		922.9	264.6						0.00625
40		967.0	484.0						0.0038, 0.0053
41		967.0	484.0						0.00484
42		964.0	484.0						0.00487, 0.00628
43	medium Sand	264.6	264.6						0.0142
44	fine Sand	484.0	484.0						0.0917
45	>>	264.6	264.6						0.0940
46	>>	132.9	132.9						0.0926
47	>>	440.1	440.1	132.9					0.0935
48	clayey siltstone D	1000.0	527.9	264.6					0.01
49	>>	966.8	264.6						0.024
50	>>	966.8	264.6	527.9					0.0157
51	>>	922.9	154.8	264.6	527.9				0.0150
52	claystone D	922.9	264.6	527.9					0.0141

Table 4.3 Summary of tests in which procedure A was implemented

Test No	Site and soil type	Stage	Nominal normal stress (kPa)	Rate of displacement (mm/min)	Notes
A. Unsheared soil					
2	claystone A	1	241.0	400.0	OCR = 4.10 ⁽¹⁾
5	clayey siltstone A	1	241.0	400.0	OCR = 3.64
8	clayey siltstone A	1	590.0	400.0	OCR = 1.00
9	claystone B	1	214.0	400.0	OCR = 4.61
10	claystone C	1	248.0	400.0	OCR = 4.00
11	siltstone	1	248.0	400.0	OCR = 4.00
18	clayey siltstone D	1	264.6	370.0	OCR = 3.65
19	>>	1	264.6	12.0	OCR = 3.65
20	>>	1	264.6	100.0	OCR = 3.65
21	>>	1	264.6	400.0	OCR = 1.00
B. Presheared soil					
1	claystone A	13	206.0	400.0	0.01 ⁽²⁾
5	>>	10	241.0	133.0	0.0022
7	clayey siltstone A	2	593.0	400.0	0.004
32	mechanically degraded Tuff		100.0	10.0	
			100.0	100.0	
34	Buoninvete	16	490.0	400.0	0.05

(1), OCR : ratio of the maximum consolidation normal stress to the nominal normal stress during fast shearing

(2), preceding rate of displacement (mm/min)

Table 4.4 Summary of tests in which procedure B was implemented

Test No	Site and soil type	Stage	Nominal normal stress (kPa)	Rate of displacement (mm/min)		pause
				current	previous	
5	clayey siltstone A	2	241.0	400.0	400.0	(1)
5	>>	3	241.0	400.0	400.0	>>
5	>>	4	241.0	400.0	400.0	>>
5	>>	6	241.0	400.0	400.0	>>
5	>>	7	241.0	133.0	400.0	>>
5	>>	8	241.0	133.0	133.0	>>
14	clayey siltstone B	5	594.0	6200.0	6200.0	>>
17	clayey siltstone D	16	264.6	2000.0	2000.0	>>
17	>>	17	264.6	2000.0	2000.0	>>
22	claystone D	20	264.6	2600.0	2600.0	>>
23	Lower Cromer Till	18	132.9	1200.0	1200.0	>>
23	>>	21	67.1	133.0	133.0	>>
23	>>	22	67.1	133.0	133.0	>>
27	Fiji	5	484.0	133.0	133.0	>>
30	Japan	18	264.0	6000.0	6000.0	>>
5	clayey siltstone A	5	241.0	400.0	400.0	5 min
5	>>	7	241.0	133.0	133.0	few sec
5	>>	8	241.0	133.0	133.0	few sec
7	>>	5,8	593.0	400.0	400.0	few sec
30	Japan	22	264.6	6000.0	6000.0	1.3 min
34	Buoninvete	11	490.0	100.0	100.0	few sec
49	clayey siltstone D	24-37	264.6	1200-6000	900-5400	2.5 min
	>>	38-54	264.6	1200-5400	1400-3700	2.5 min

(1) The sample had been allowed to consolidate fully prior to fast shearing involved in this procedure.

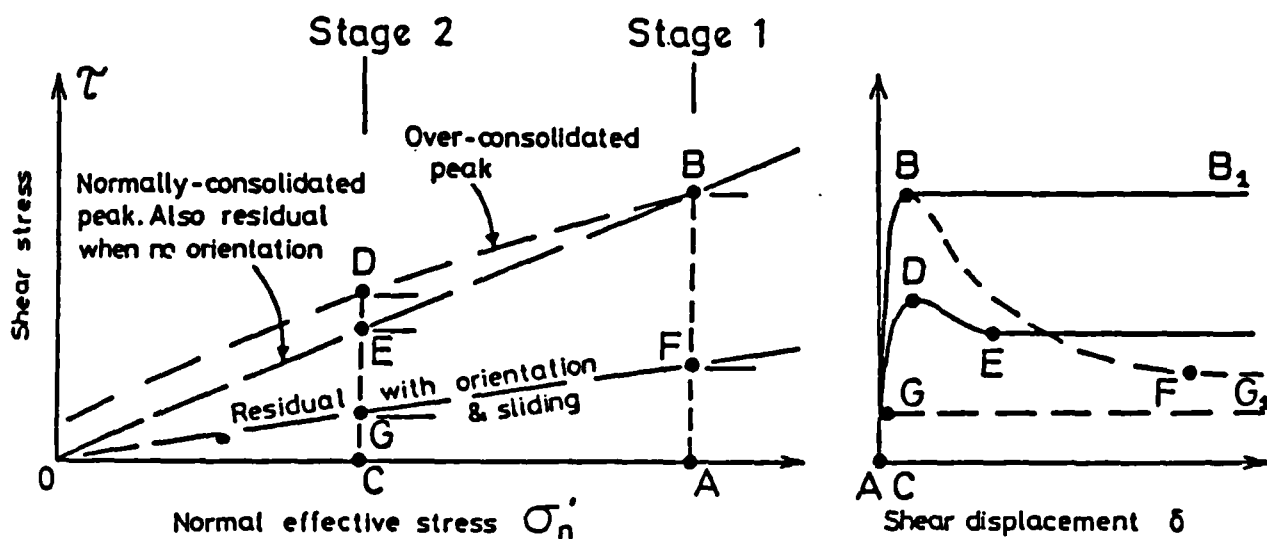
Table 4.5 Summary of tests in which procedure C was implemented

Test No	Site and soil type	Stage	Nominal normal stress (kPa)	Rate of displacement (mm/min)		pause
				current	previous	
7	clayey siltstone A	3,6	593.0	0.36	400.0	0 sec
7	>>	9	593.0	1.55	400.0	0 sec
12	clayey siltstone B	3	609.0	0.35	400.0	0 sec
12	>>	6	609.0	0.35	100.0	0 sec
12	>>	7	609.0	0.32	100.0	0 sec
12	>>	9	609	0.36	11.5	0 sec
12	>>	13	200.0	0.35	464.0	0 sec
12	>>	18	200.0	0.317	11.9	0 sec
13	>>	16-18	200.0	0.327	870.0	2.5 min
15	clayey siltstone C	3	608.0	0.319	400.0	0 sec
15	>>	5	608.0	0.393	100.0	0 sec
19	clayey siltstone D	4	264.6	0.343	400.0	1.75min
23	Lower Cromer Till	10	264.6	0.133	133.0	10.2min
25	weathered Cowden Till	28-38	200.0	0.37	10.0	0 sec
26	Fiji	3d	177.0	17.72	177.2	0 sec
30	Japan	19	264.6	0.0925	6000.0	12 min
30	>>	23	264.6	1.89	6000.0	14 min
51	clayey siltstone D	5	264.6	0.6123	370.0	7.5 min
52	claystone D	7	264.6	1.5379	400.0	6 min

Table 4.6 Summary of the tests in which procedure D was implemented

Test No	Site and soil type	Stage	Nominal normal stress (kPa)	Rate of displacement (mm/min)	Notes
48	clayey siltstone D	5	264.6	300	(1)
49	clayey siltstone D	24-37 38-54	264.6 264.6	1200-6000 1200-5400	
52	claystone D	9	527.9	160	(1)

(1), the tests involved soil shearing against a glass-interface



Stage	Shear Behaviour	
	No Orientation	Orientation
1. Normally-consolidated	Turbulent shear. Non-brittle. High residual friction angle.	Sliding shear. Brittle. Low residual friction angle.
2. Over-consolidated	Brittle and dilatant. Residual friction angle same as stage 1.	Non-brittle, residual friction angle same as stage 1.

Fig. 4.1 Idealised behaviour in the ring shear test according to loading sequence and mode of residual behaviour (Lupini, 1981).

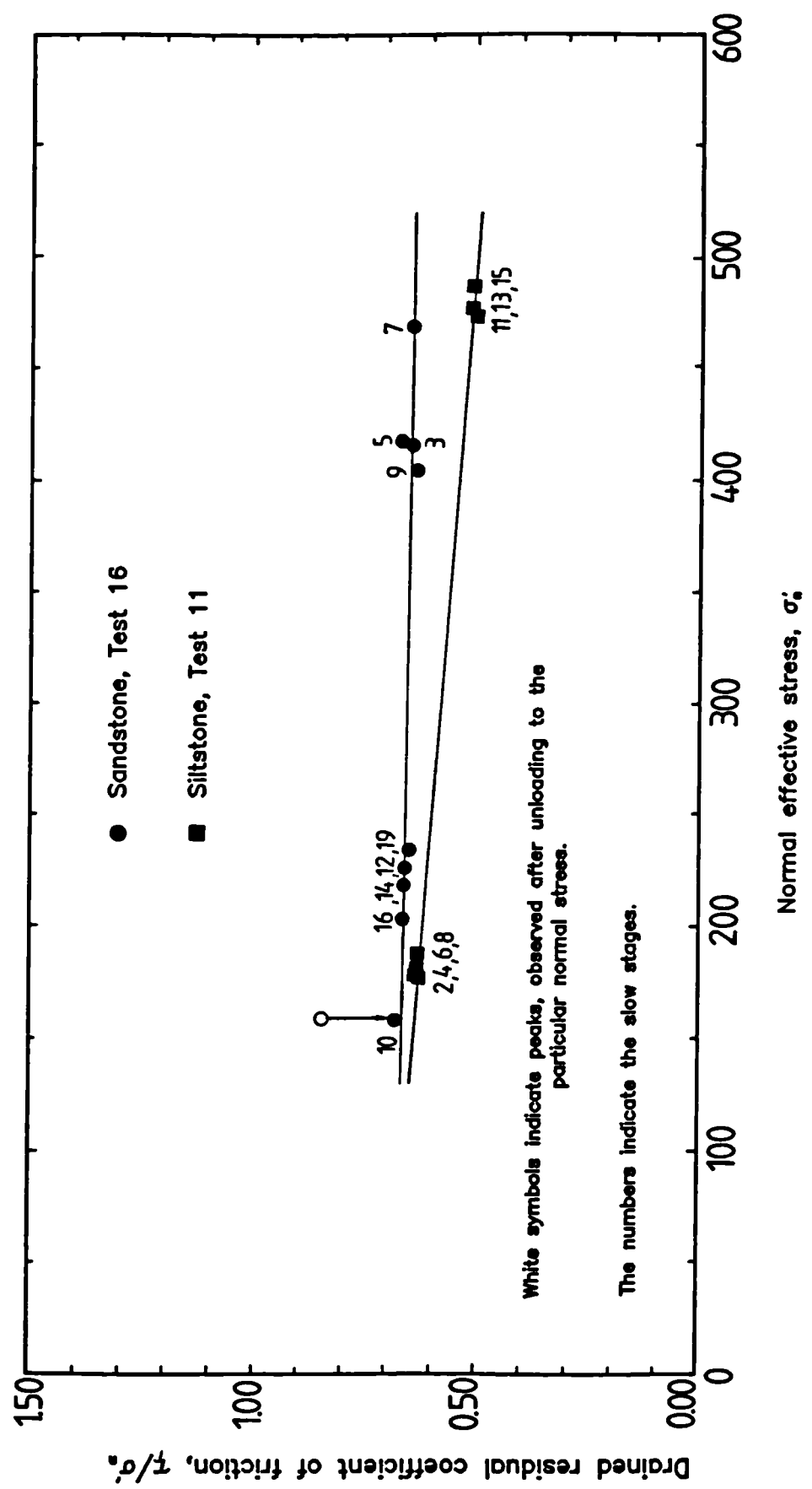


Fig. 4.2

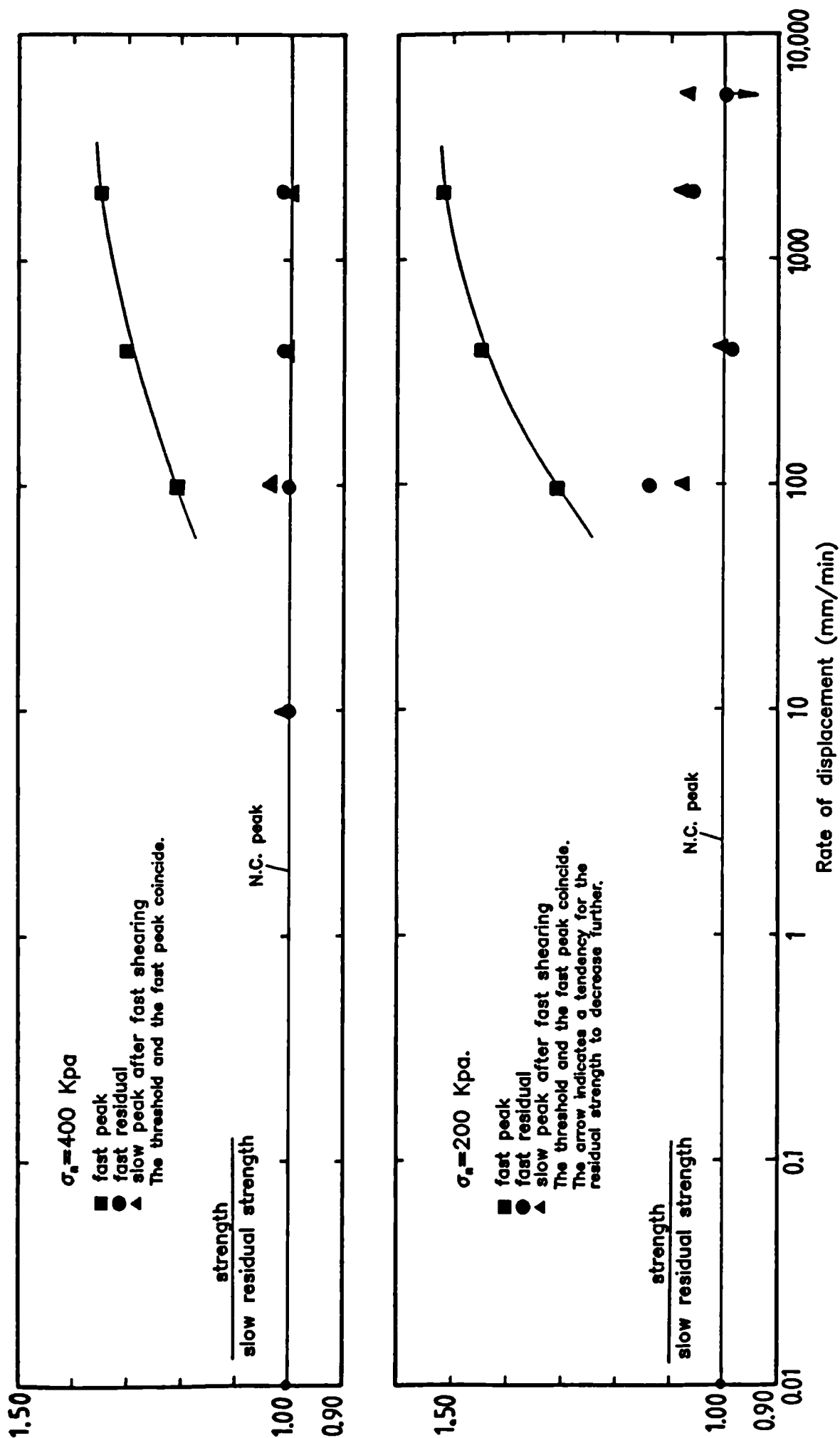


Fig. 4.3 Results from Test 16 on sandstone.

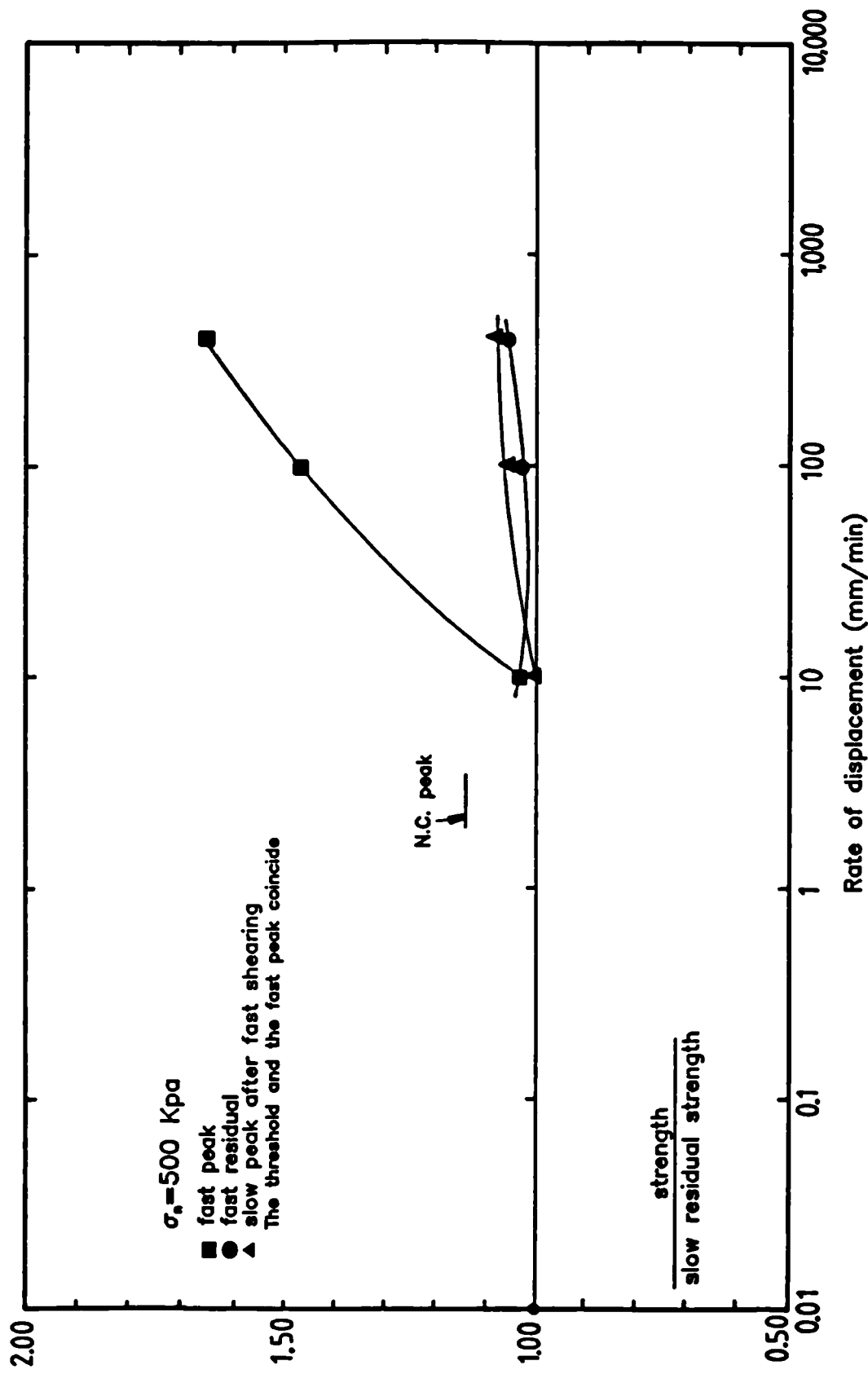


Fig. 4.4a Results from Test 11 on siltstone.

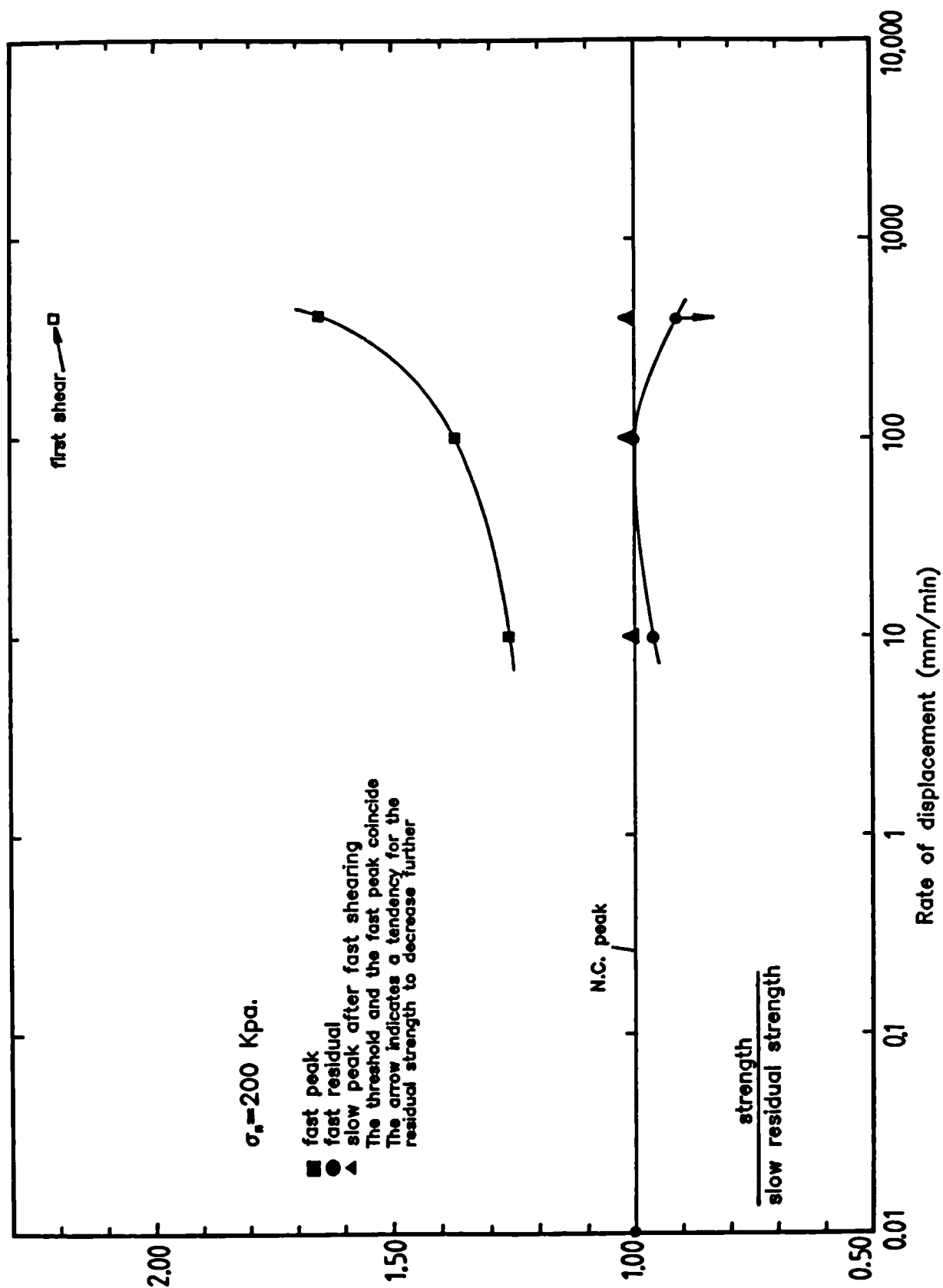


Fig. 4.4b Results from Test 11 on siltstone.

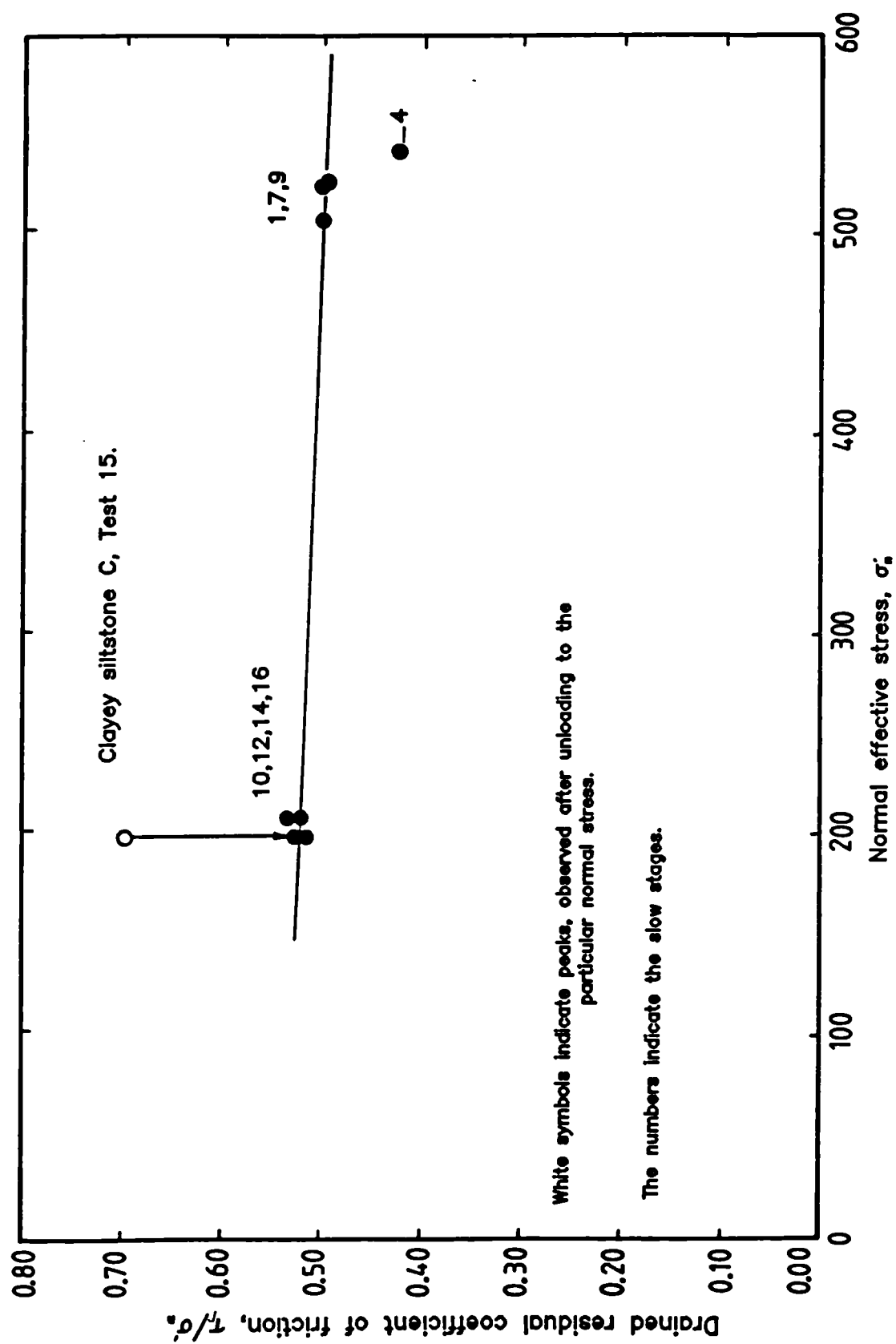


Fig. 4.5

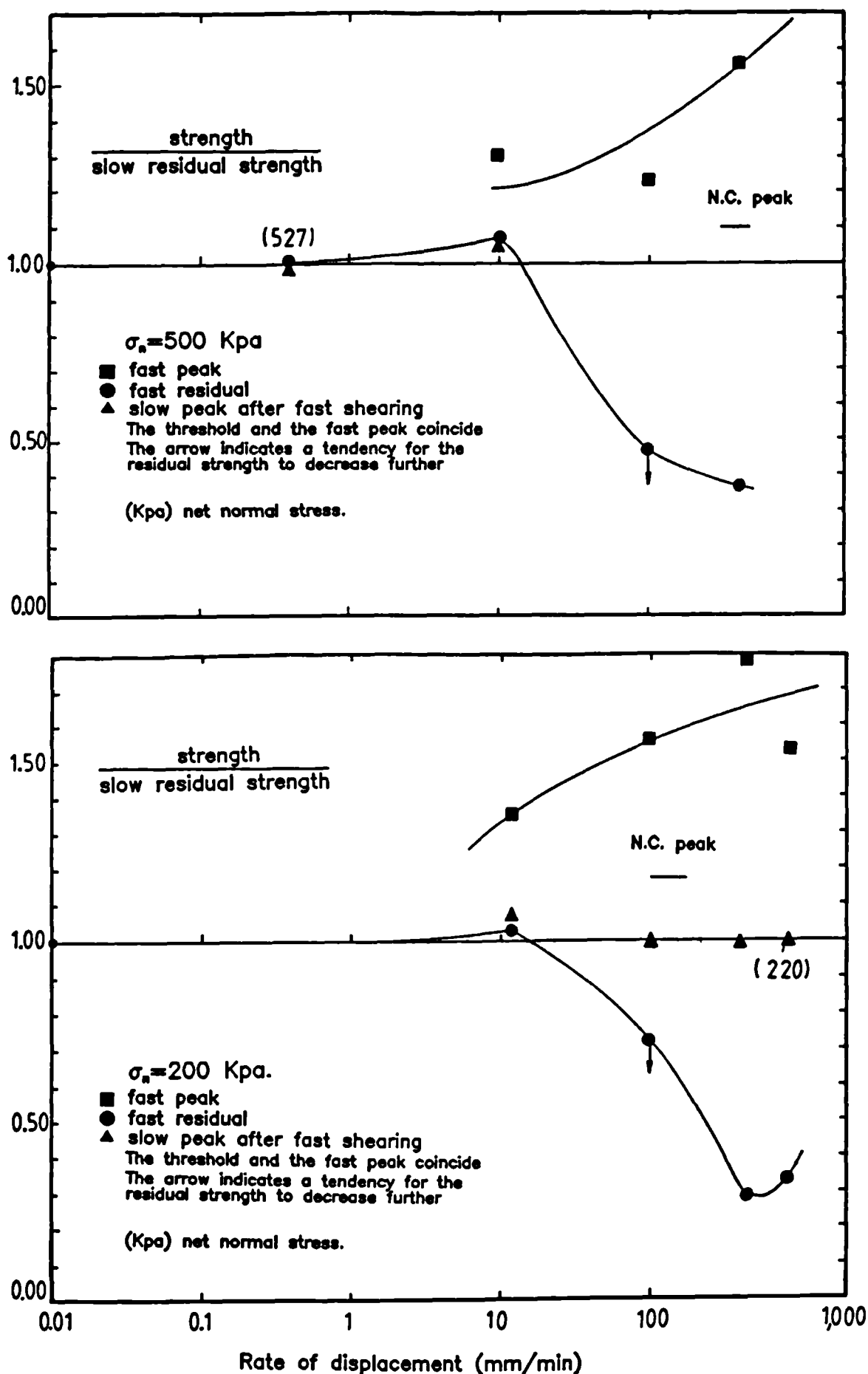


Fig. 4.6 Results from Test 15 on clayey siltstone C.

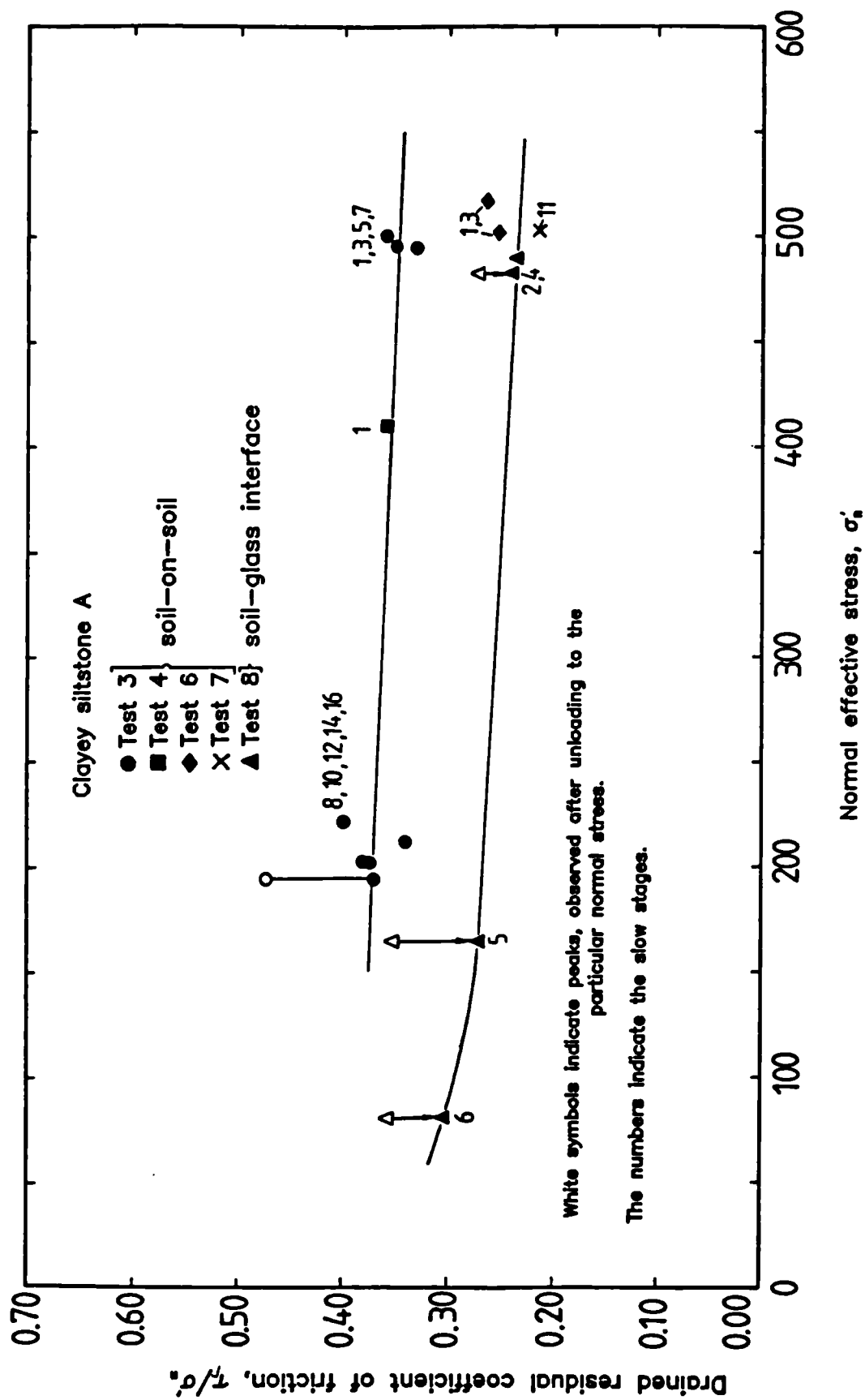


Fig. 4.7

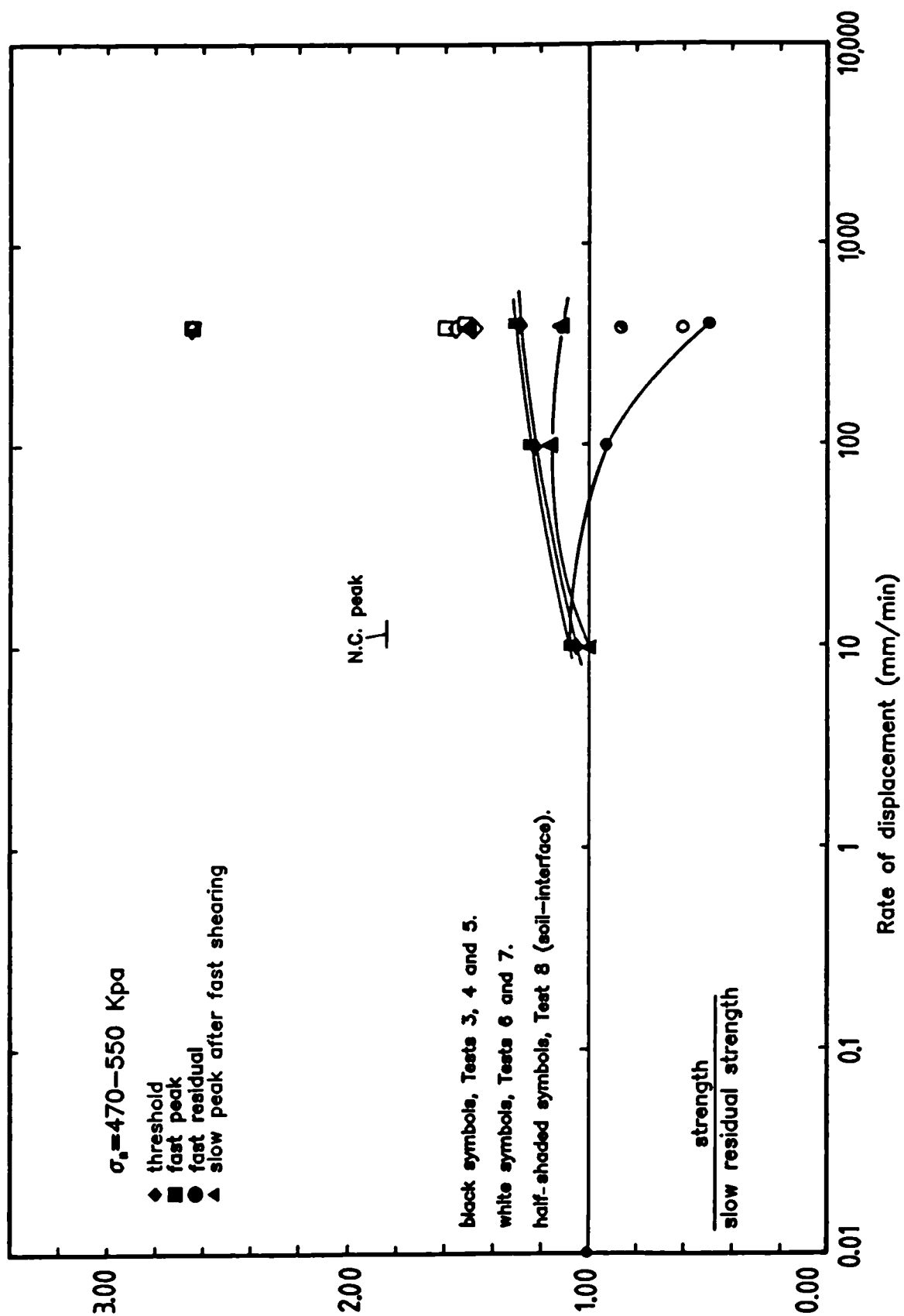


Fig. 4.8a Results from Tests 3 to 8 on clayey siltstone A.

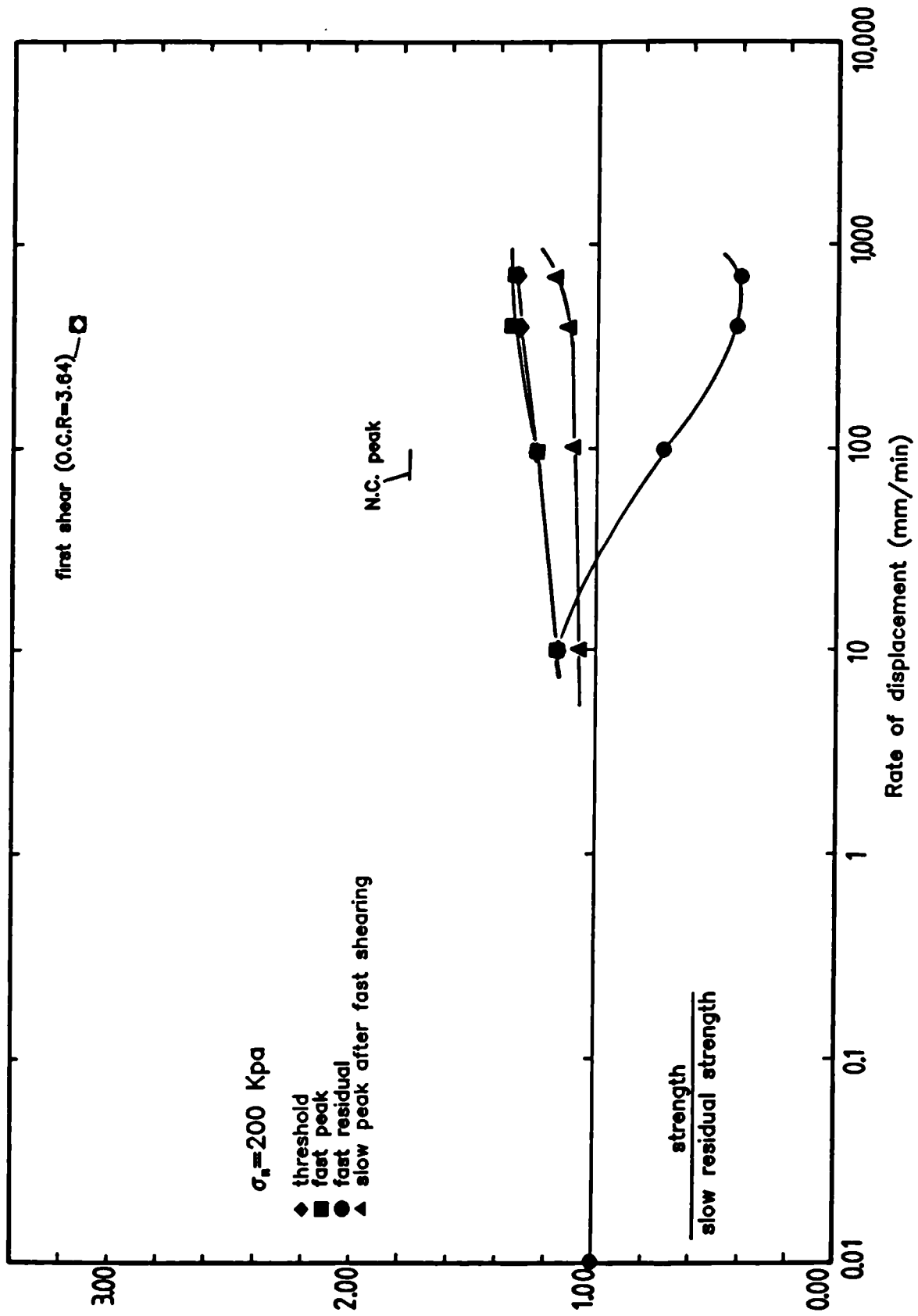


Fig. 4.8b Results from Tests 3 to 5 on clayey siltstone A.

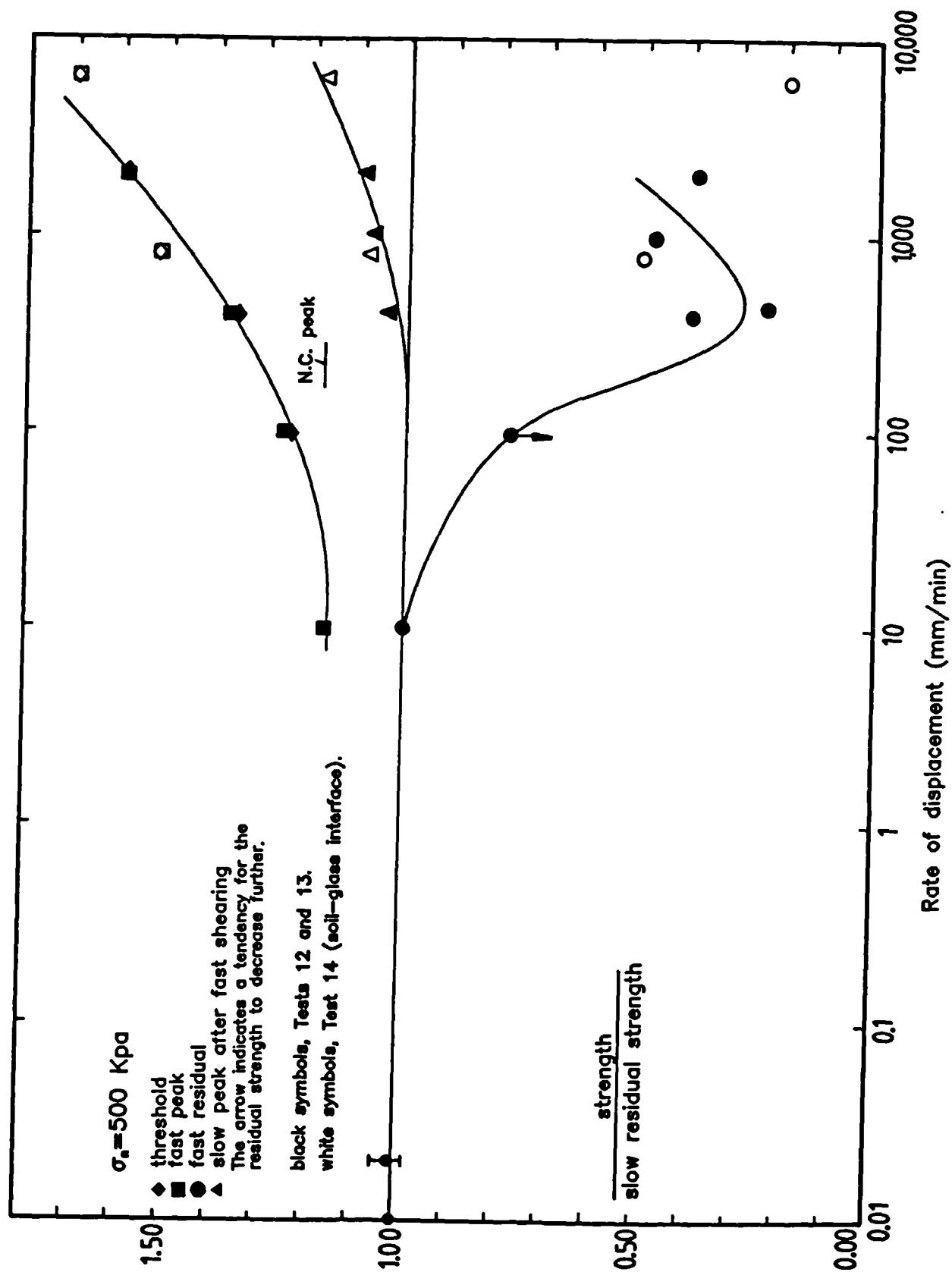


Fig. 4.10a Results from Tests 12, 13 and 14 on clayey siltstone B.

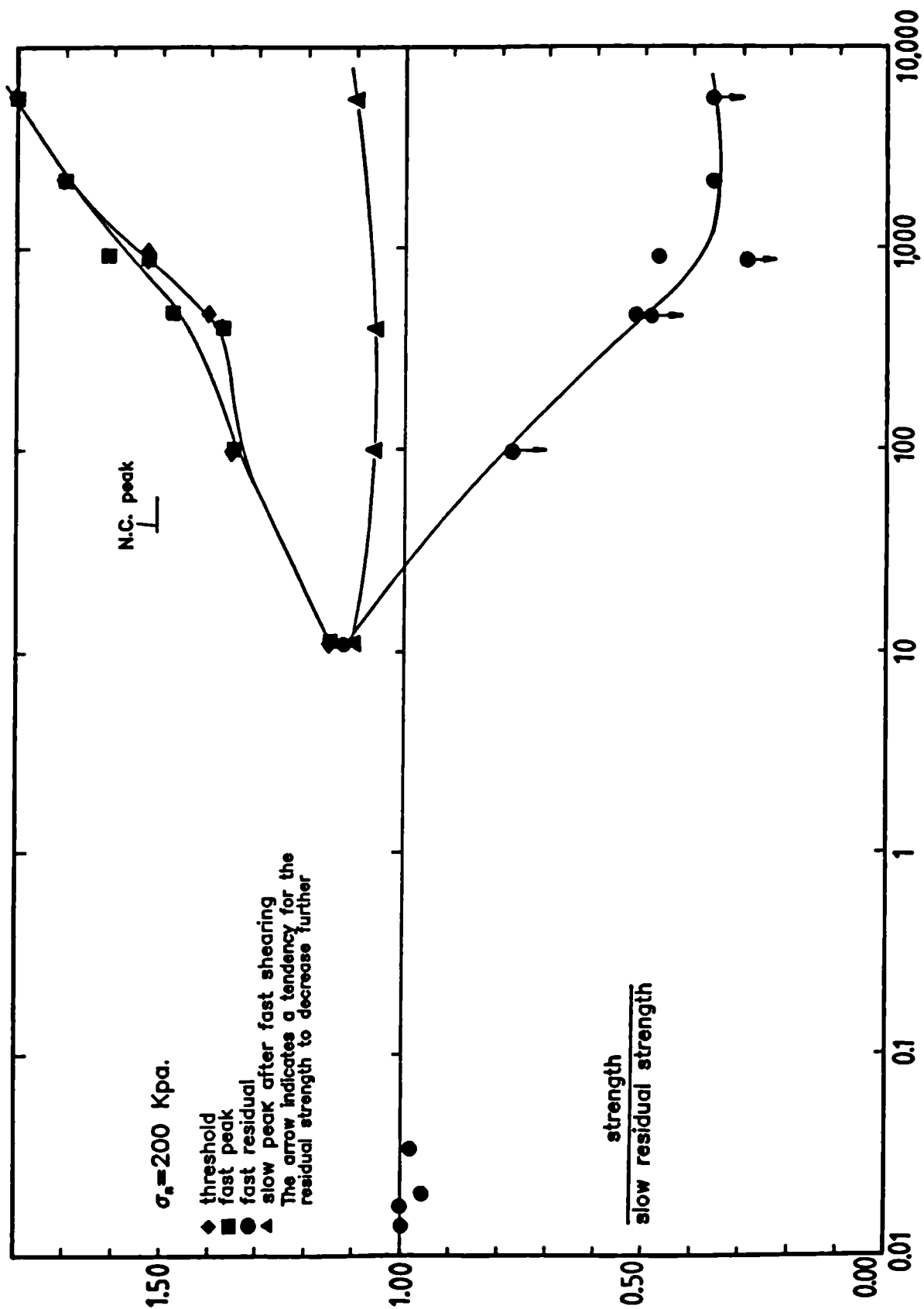


Fig. 4.10b Results from Tests 12 and 13 on clayey siltstone B.

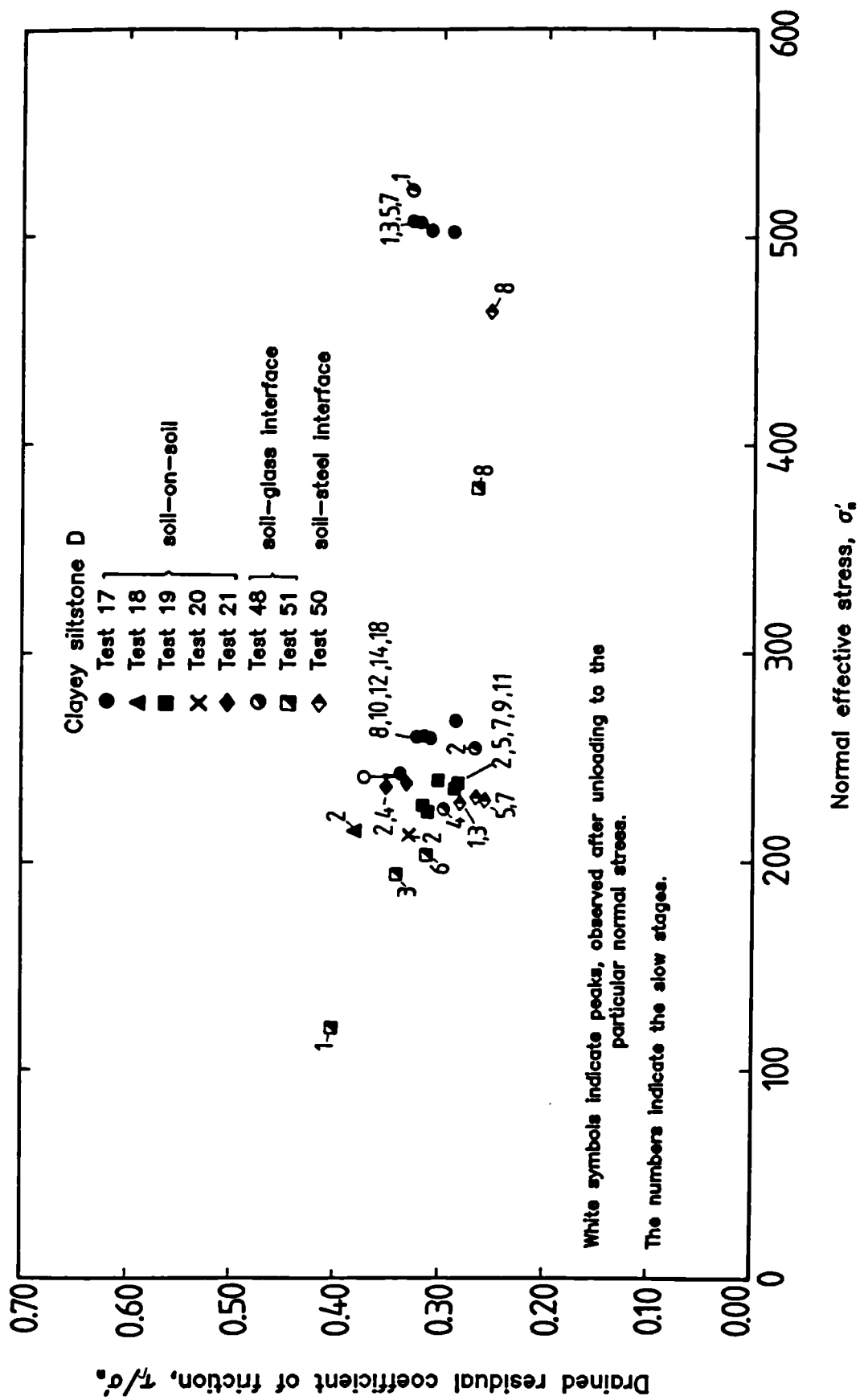


Fig. 4.11

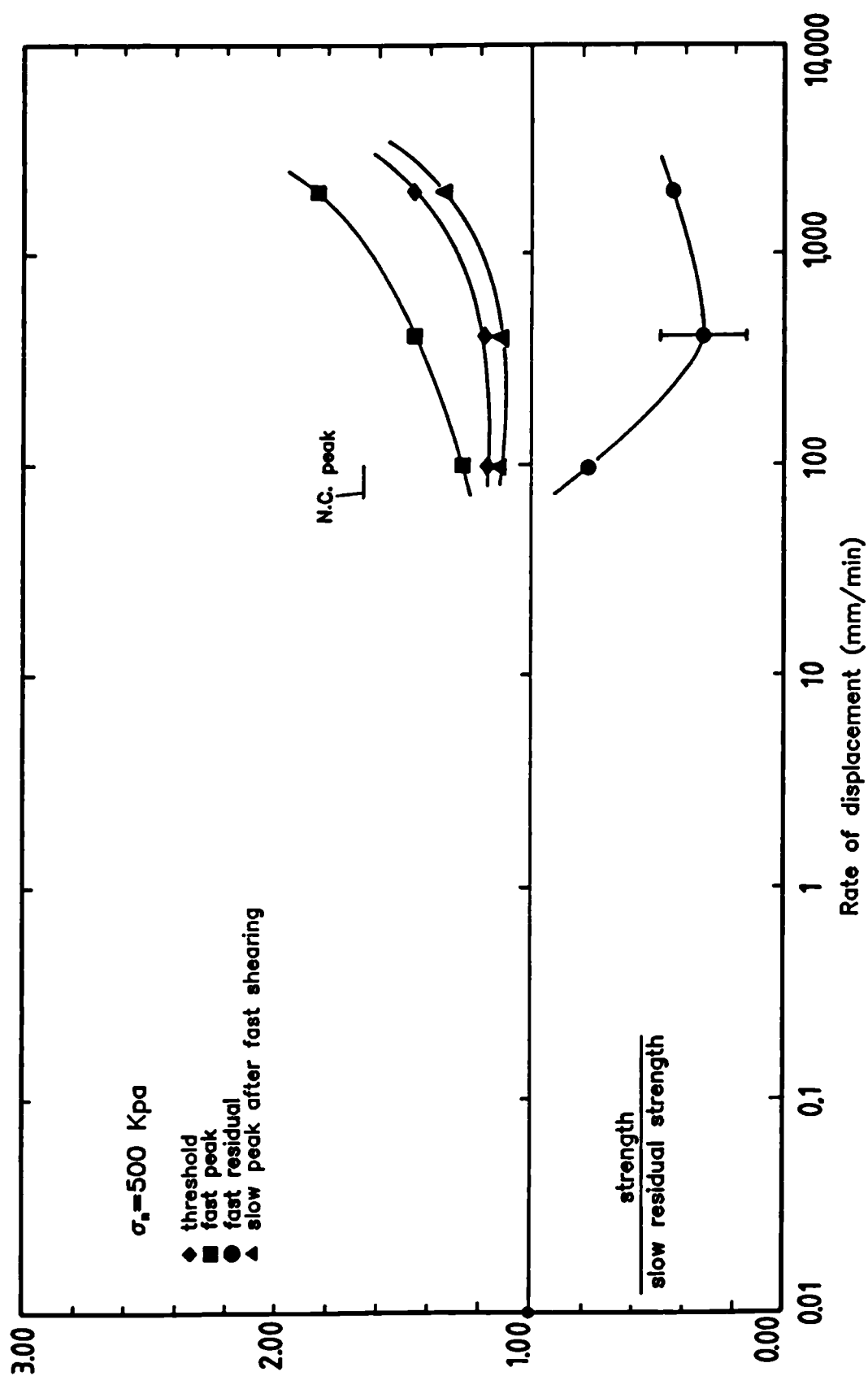


Fig. 4.12a Results from Test 17 on clayey siltstone D

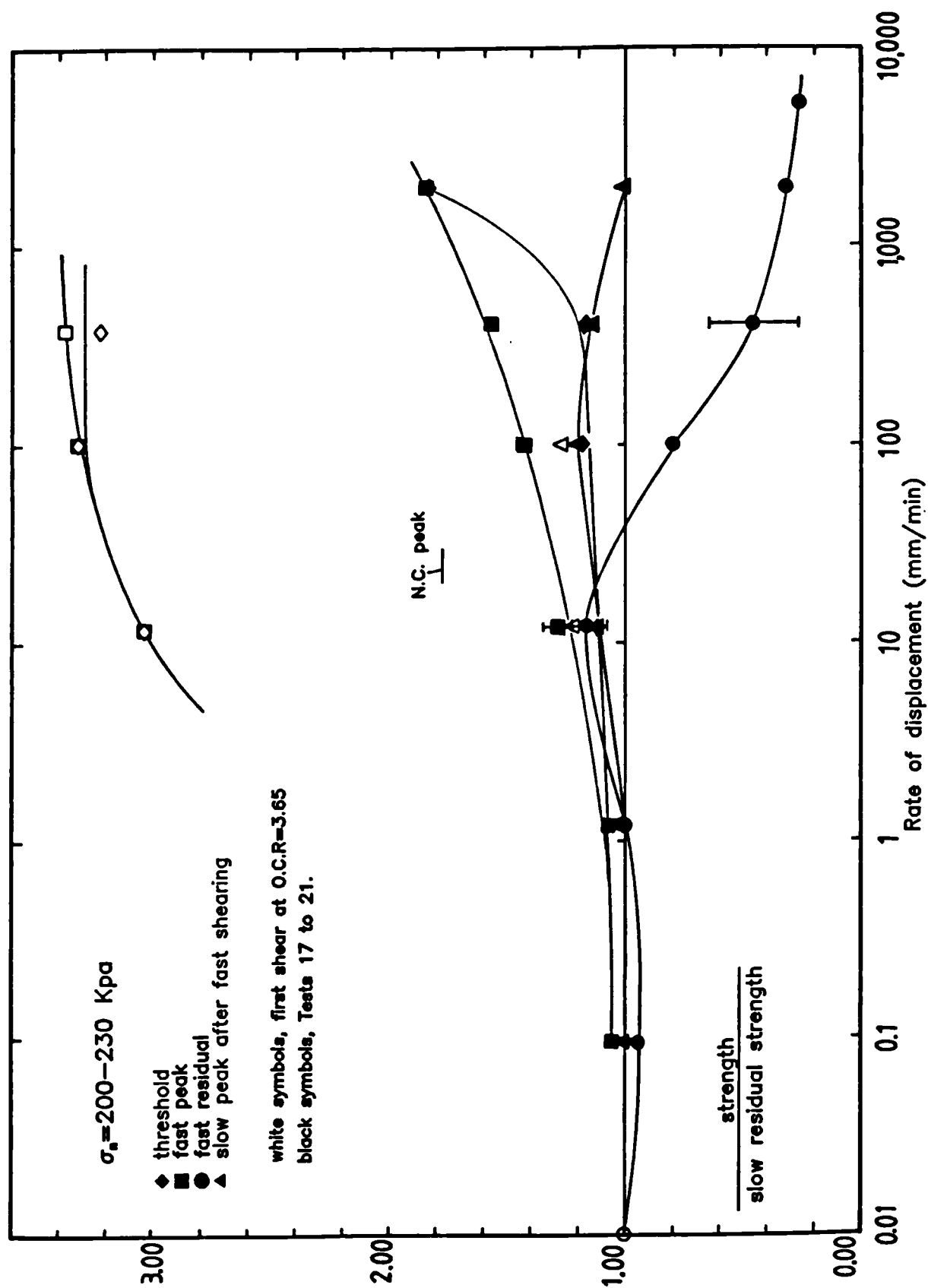


Fig. 4.12b Results from Test 17 to 21 on clayey siltstone D

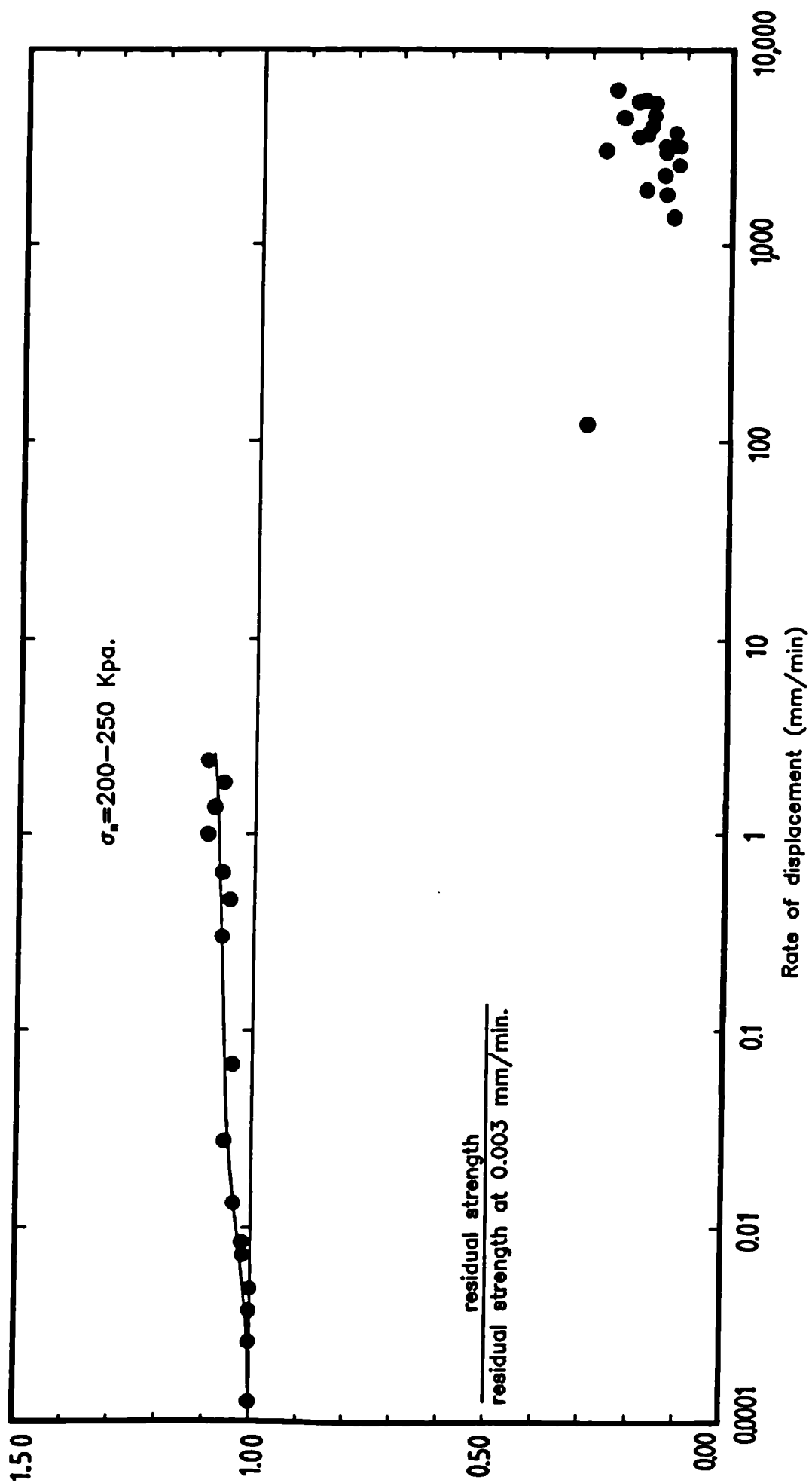


Fig. 4.12c Results from Test 49 on clayey siltstone D.

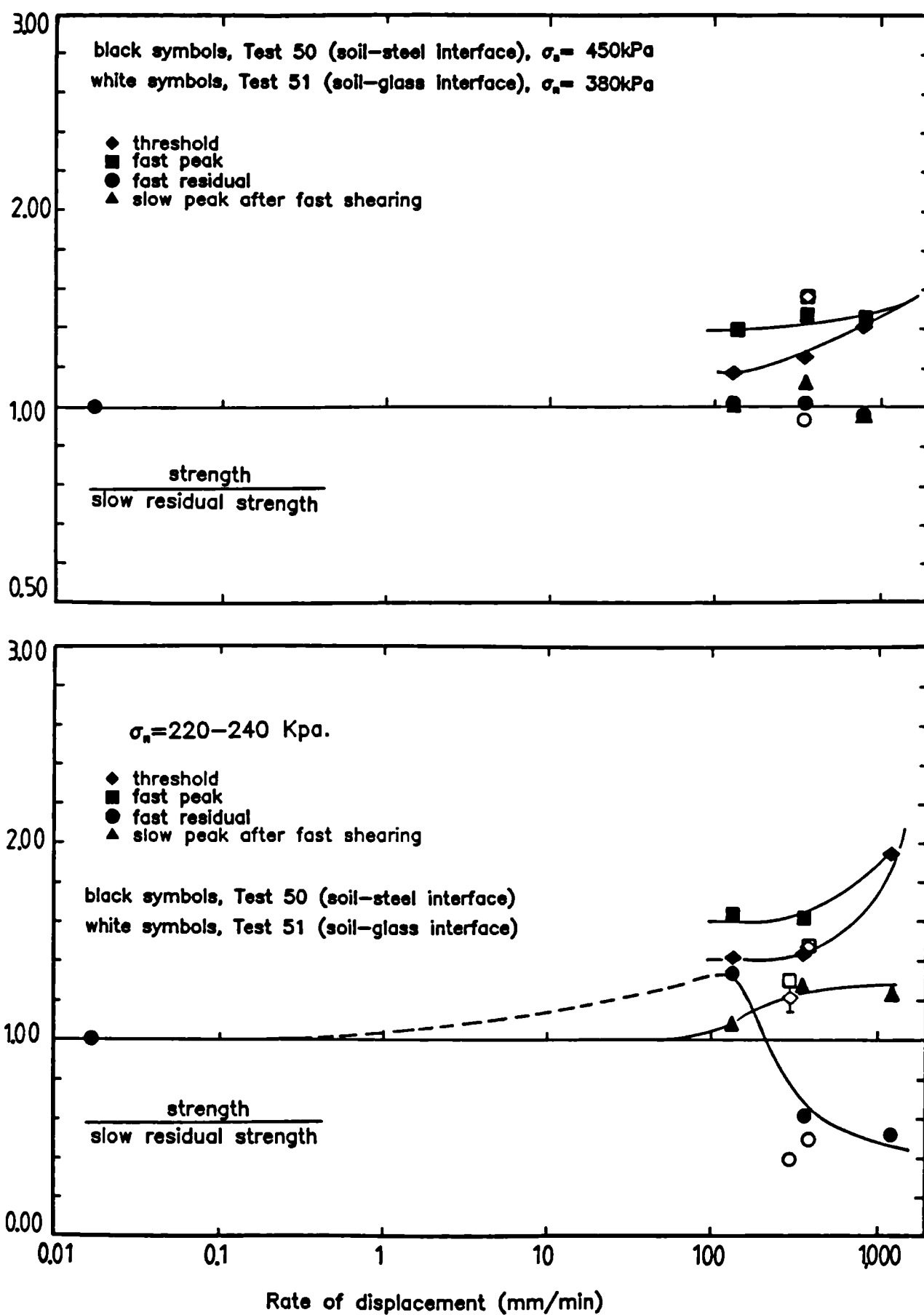


Fig. 4.12d Results from Tests 50 and 51 on clayey siltstone D
 (soil-interface)

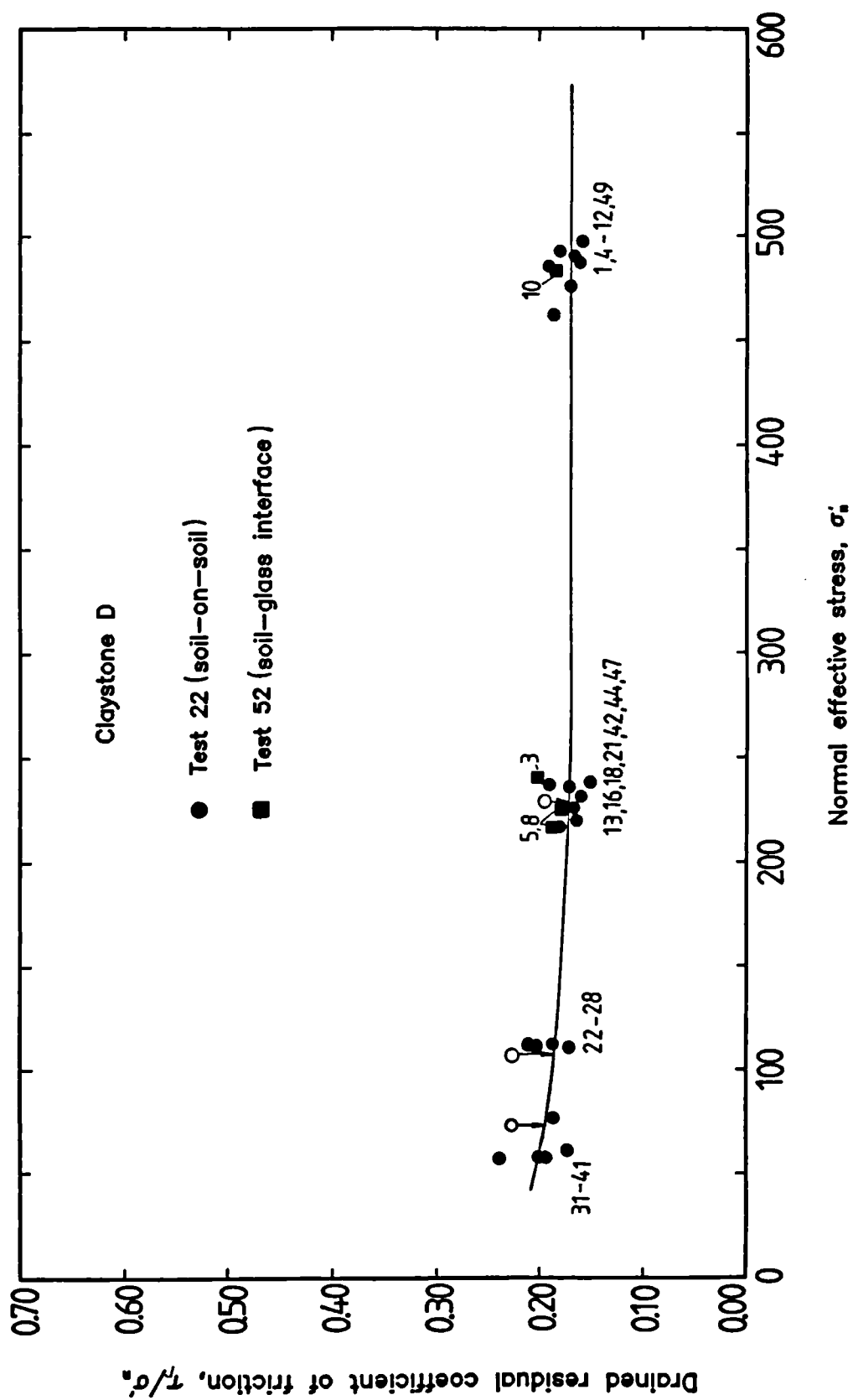


Fig. 4.13

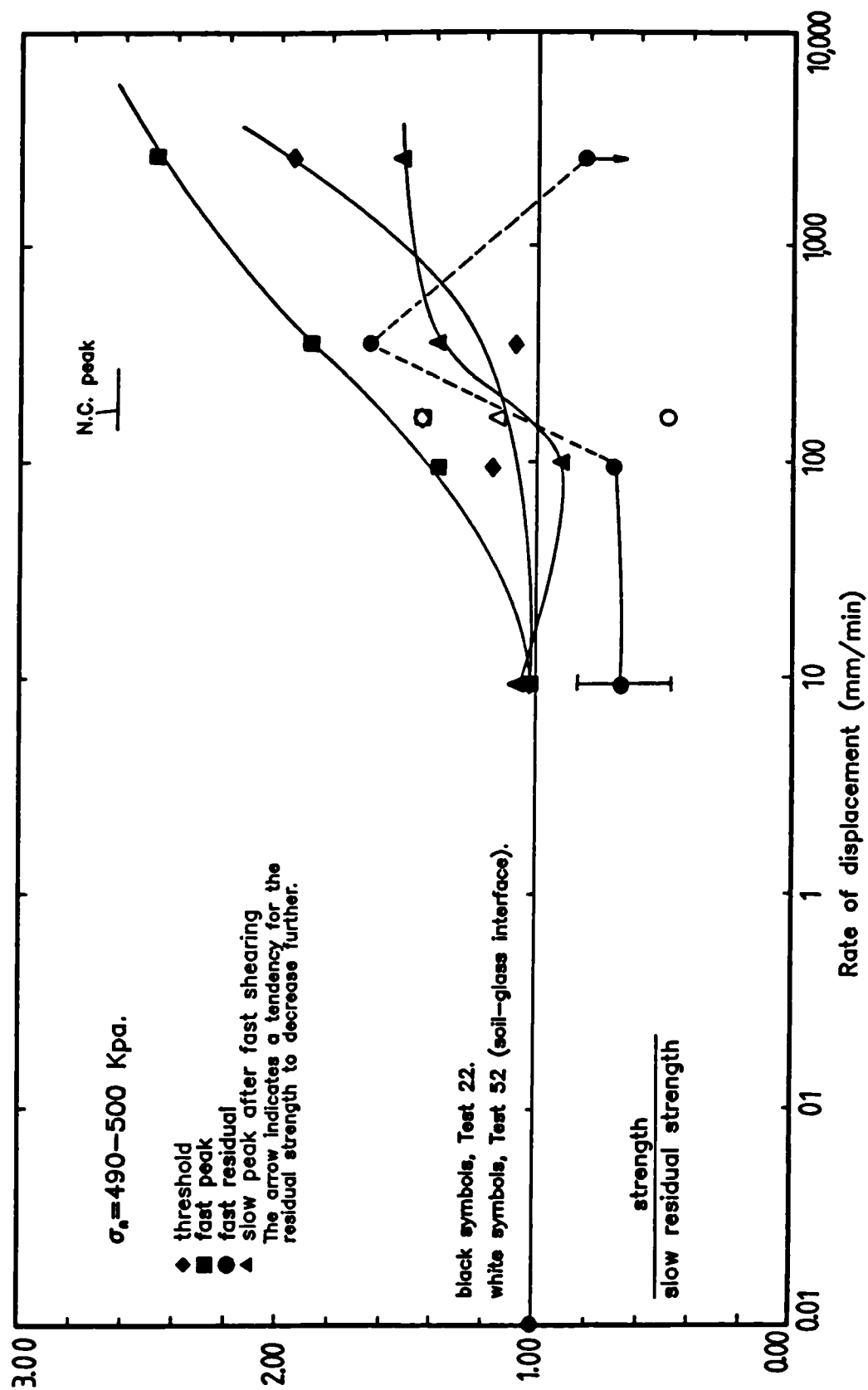


Fig. 4.14a Results from Tests 22 and 52 on claystone D.

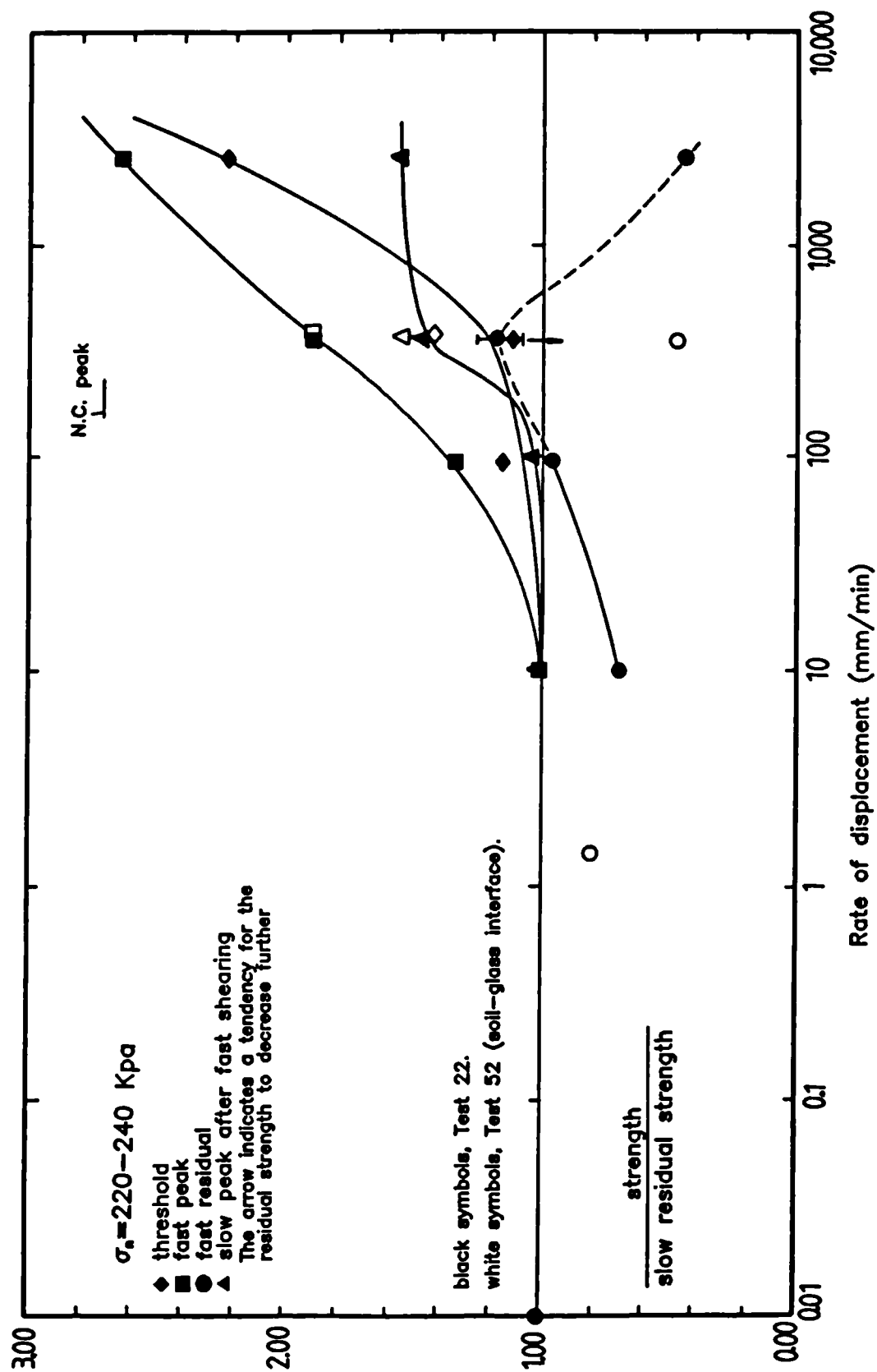


Fig. 4.14b Results from Tests 22 and 52 on claystone D.

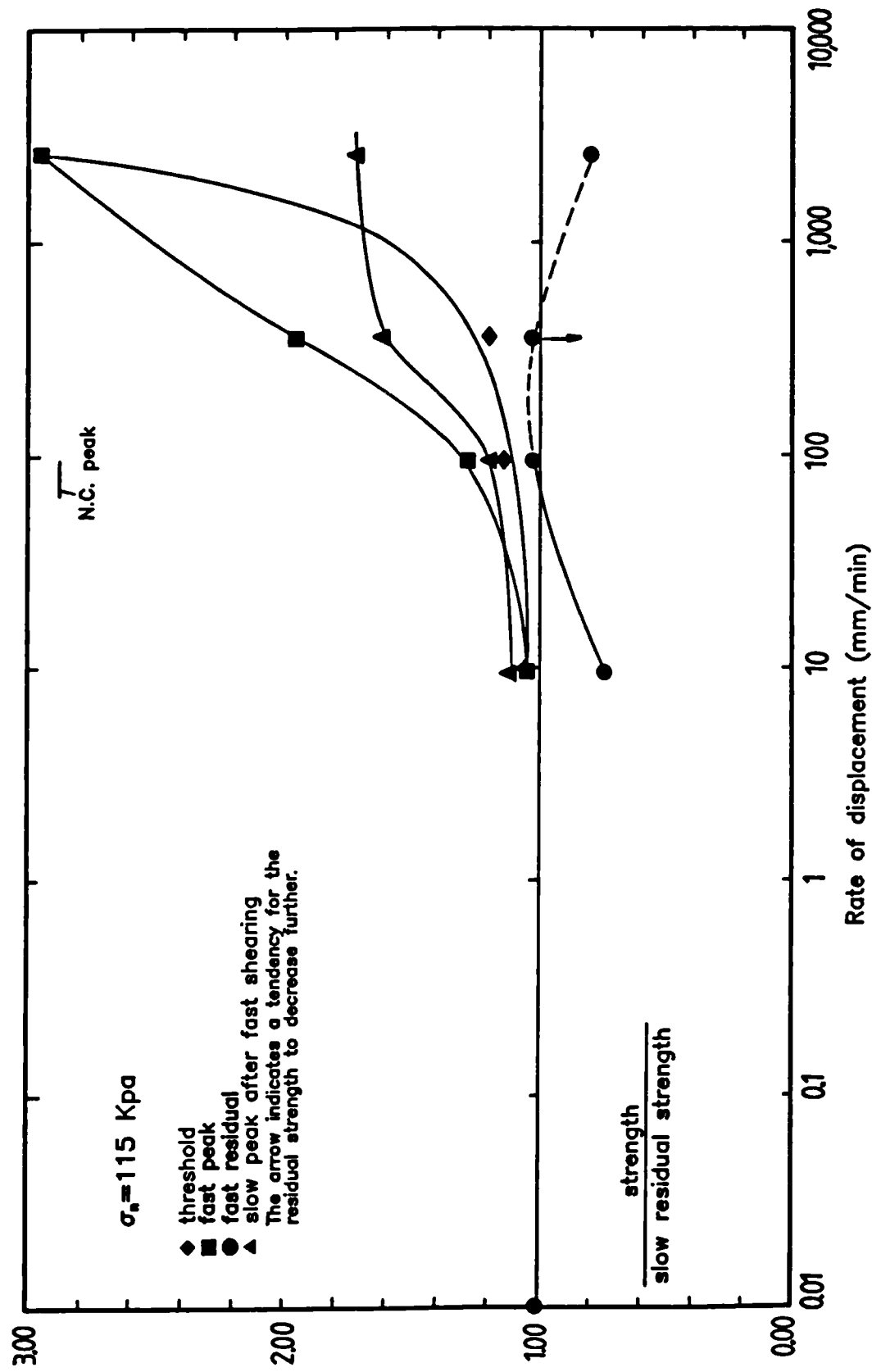


Fig. 4.14c Results from Test 22 on claystone D.

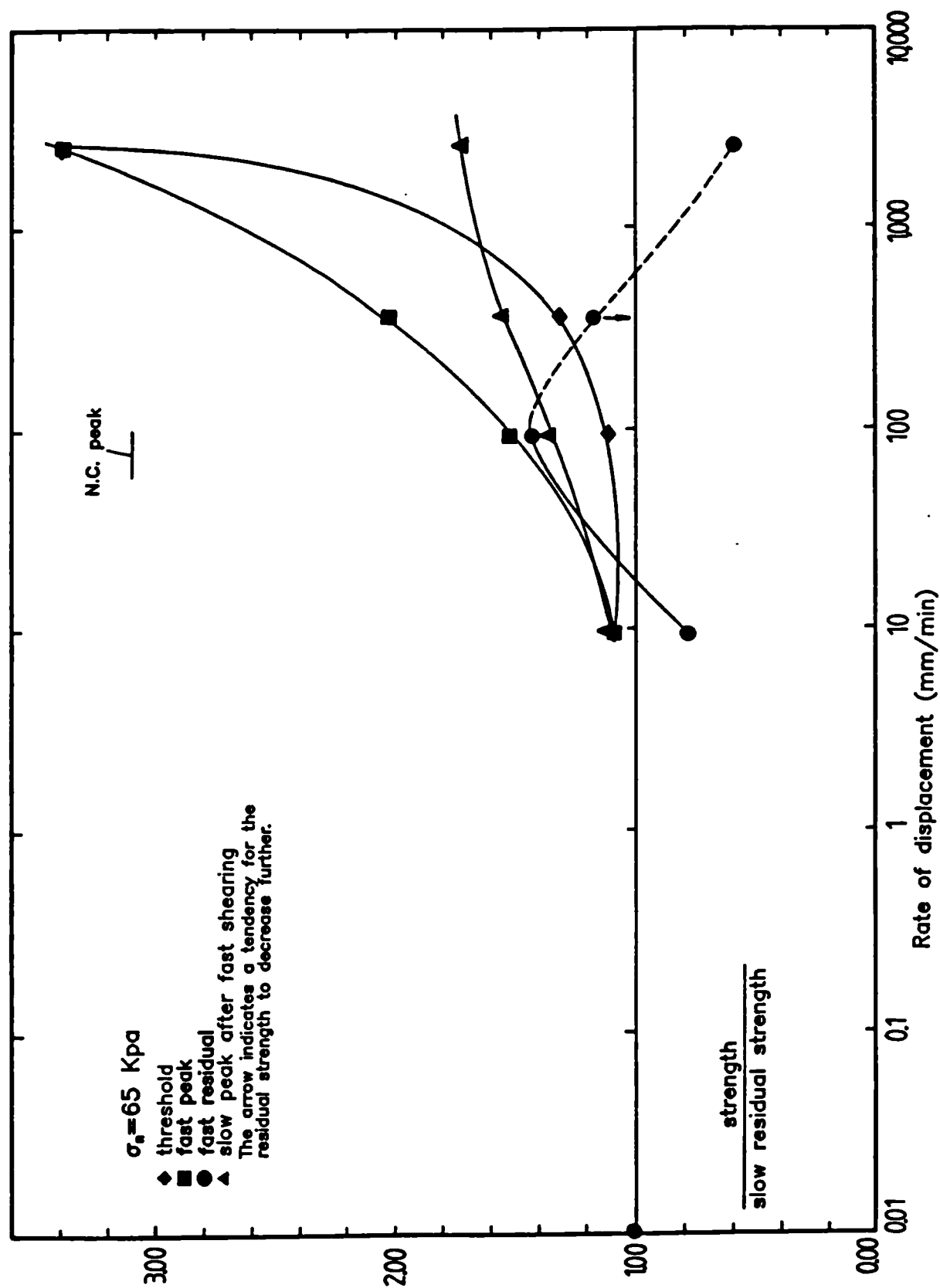


Fig. 4.14d Results from Test 22 on claystone D.

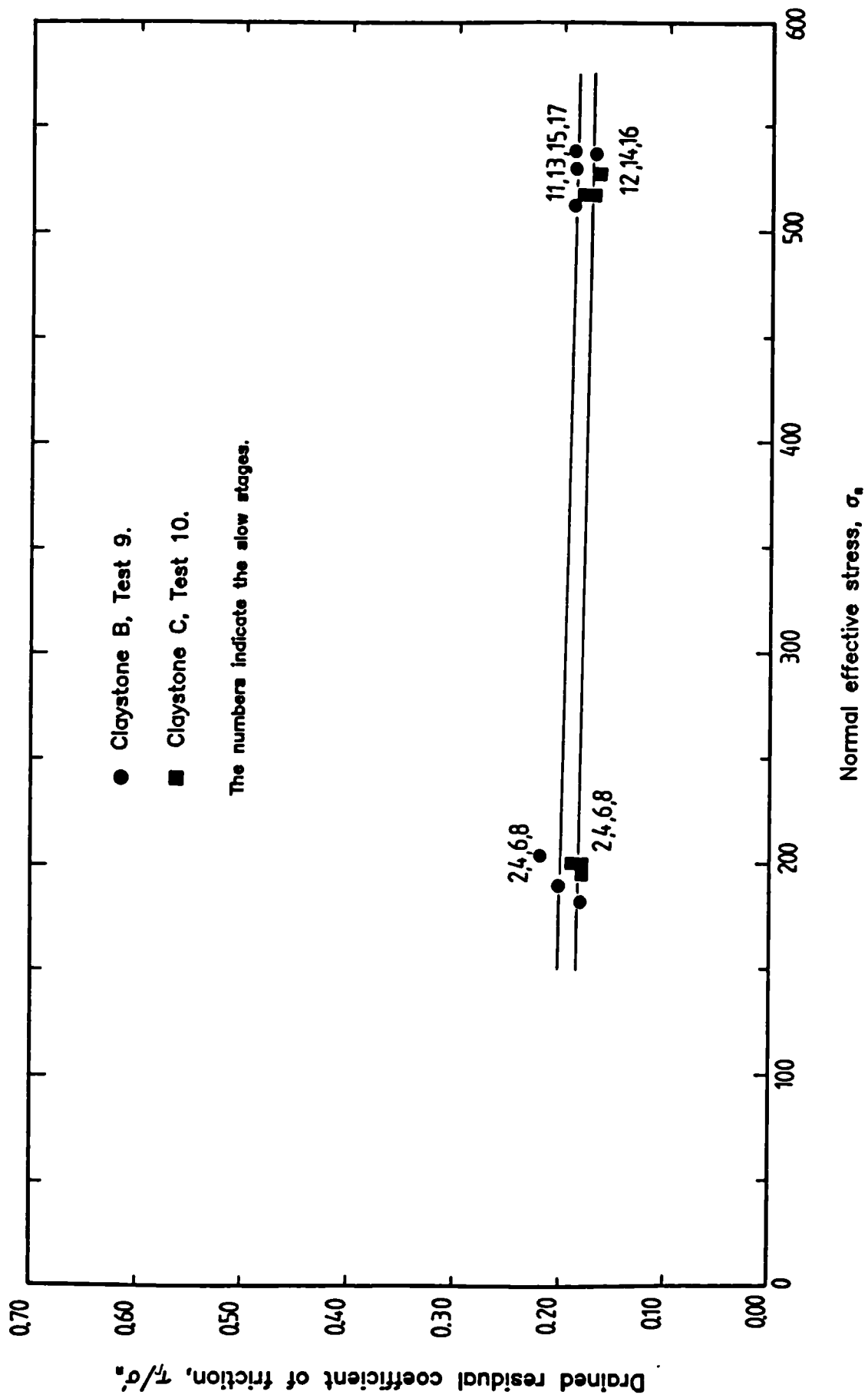


Fig. 4.15

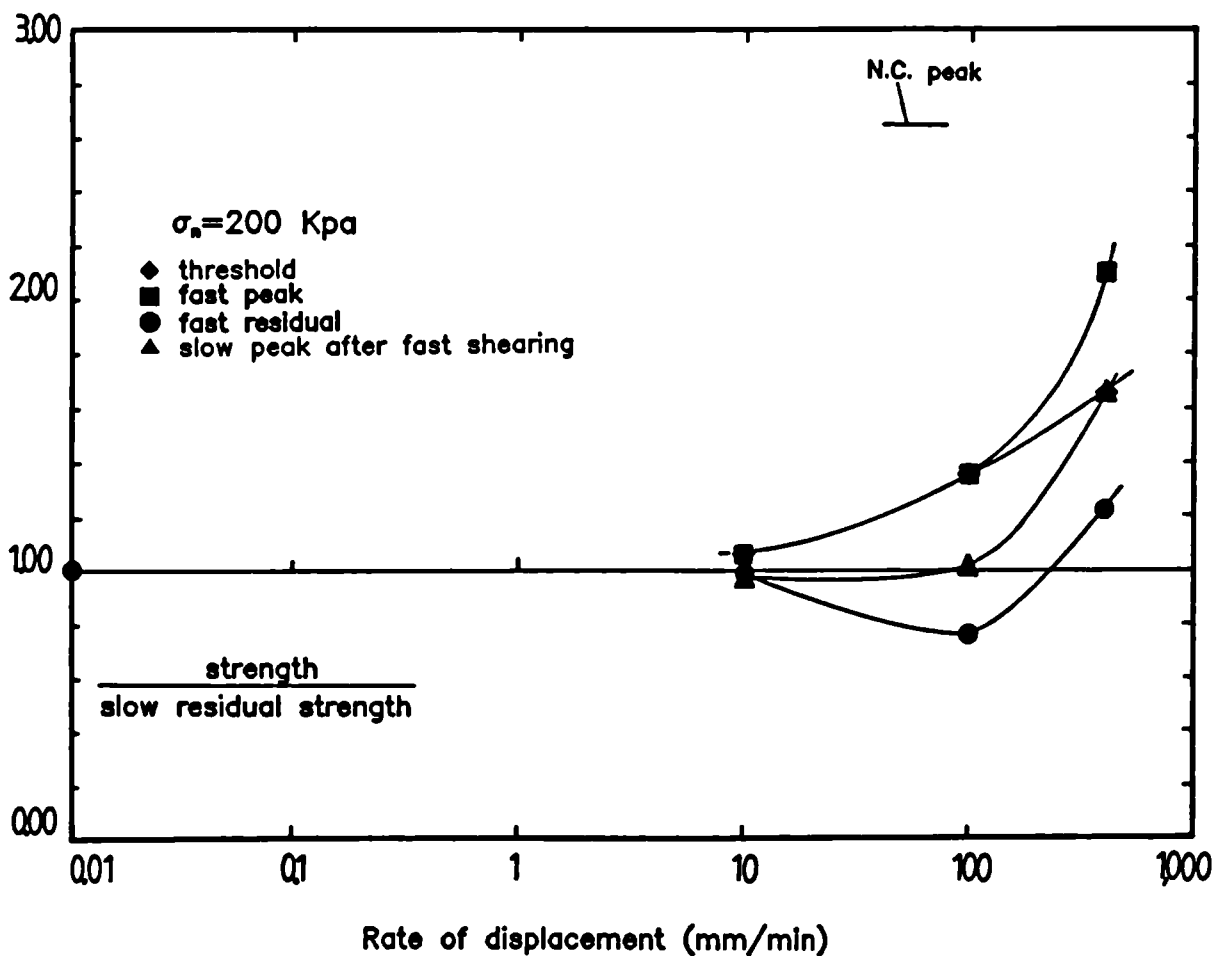
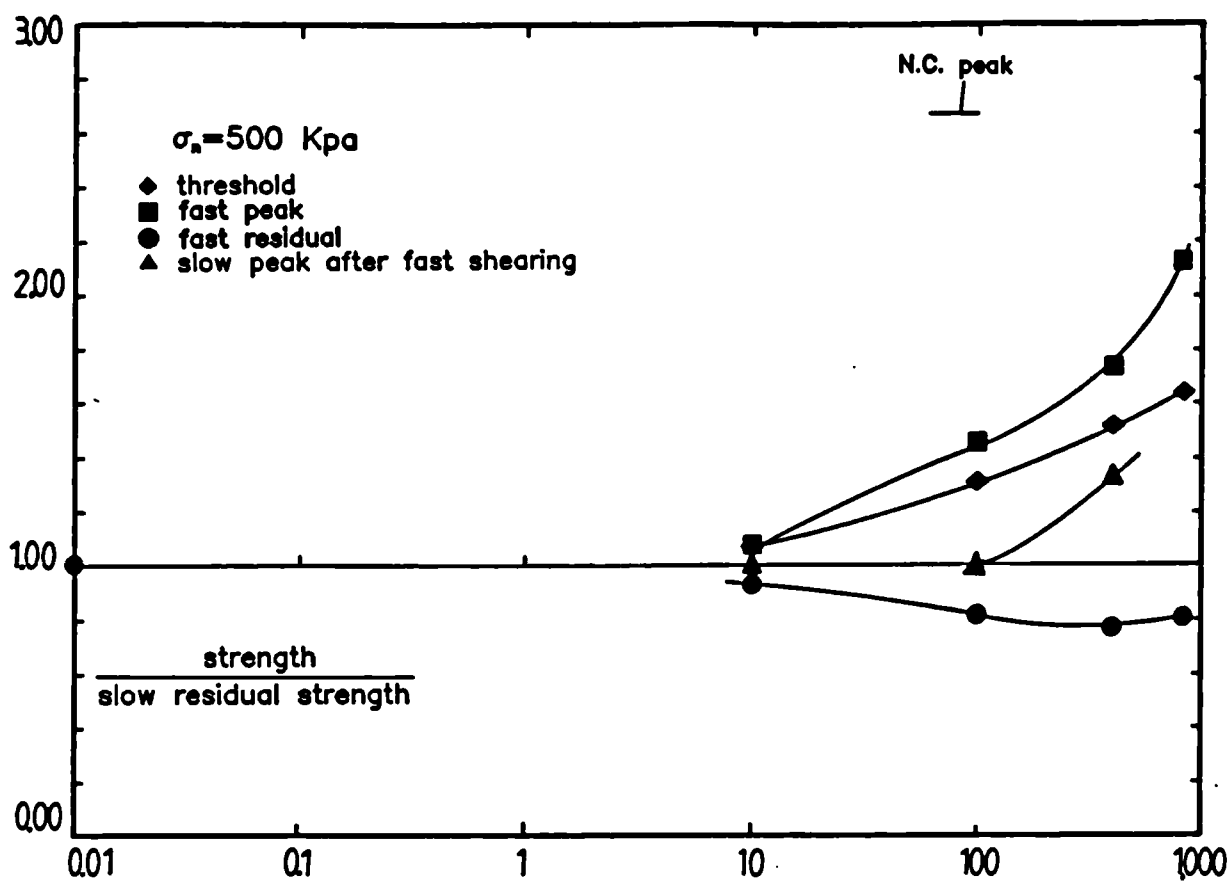


Fig. 4.16 Results from Test 10 on claystone C.

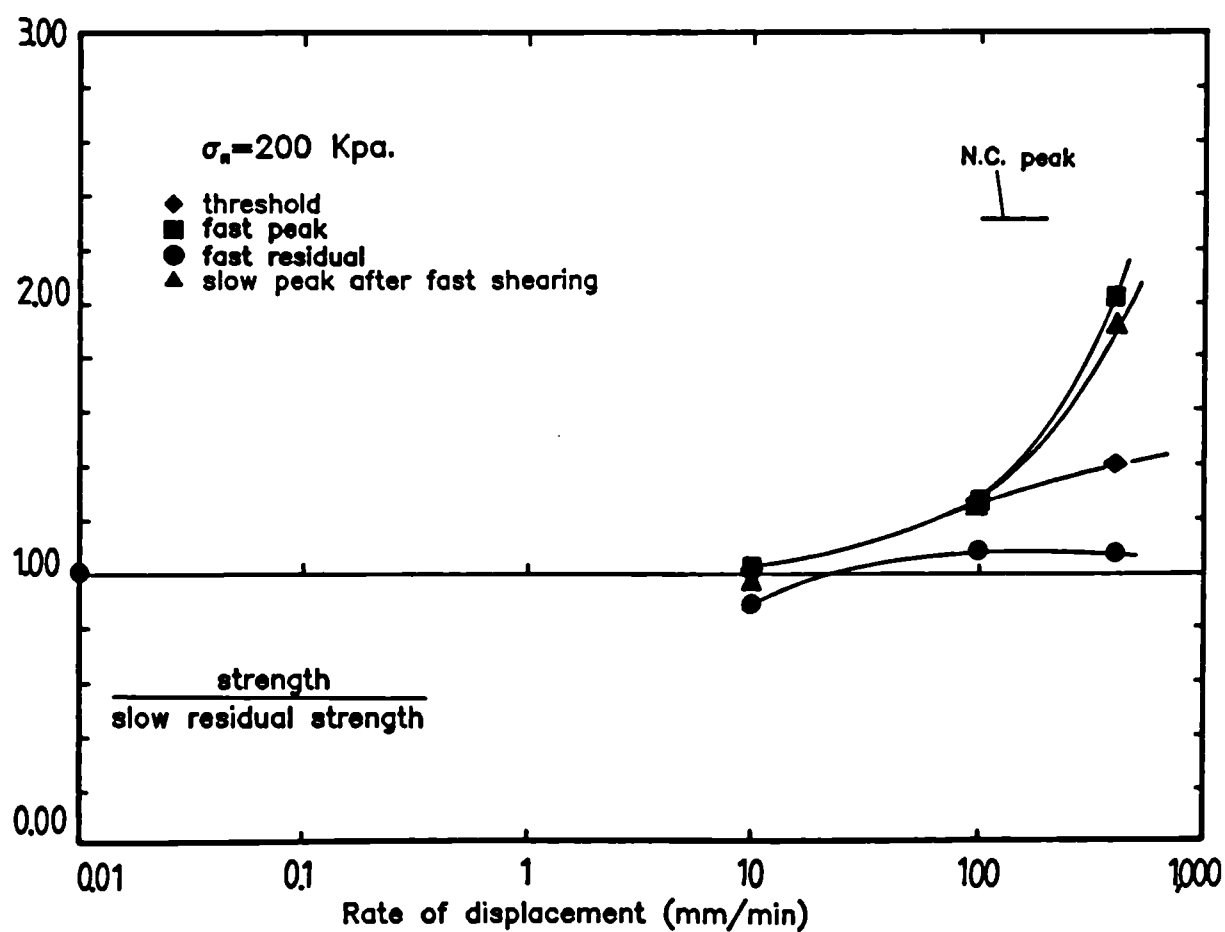
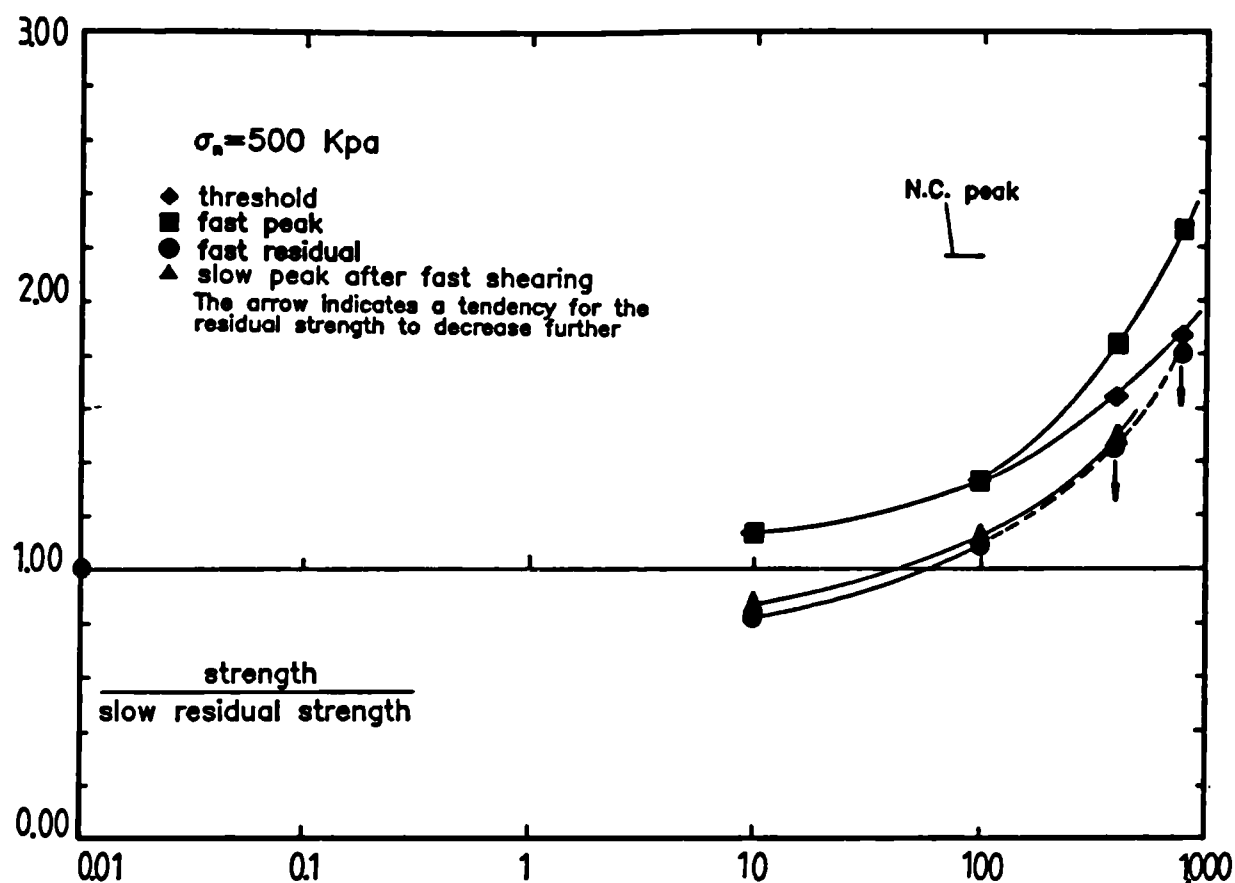


Fig. 4.17 Results from Test 9 on claystone B.

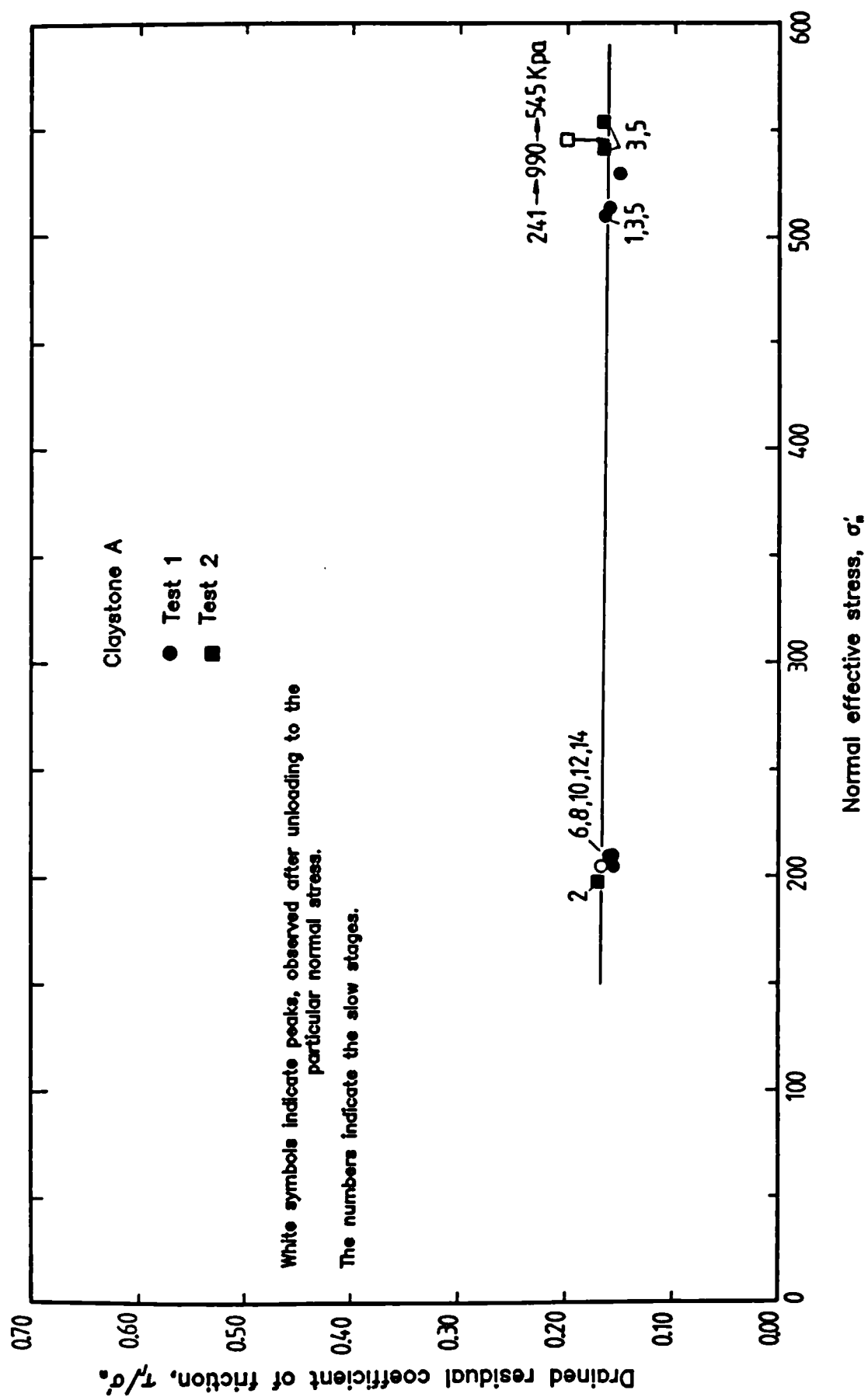


Fig. 4.18

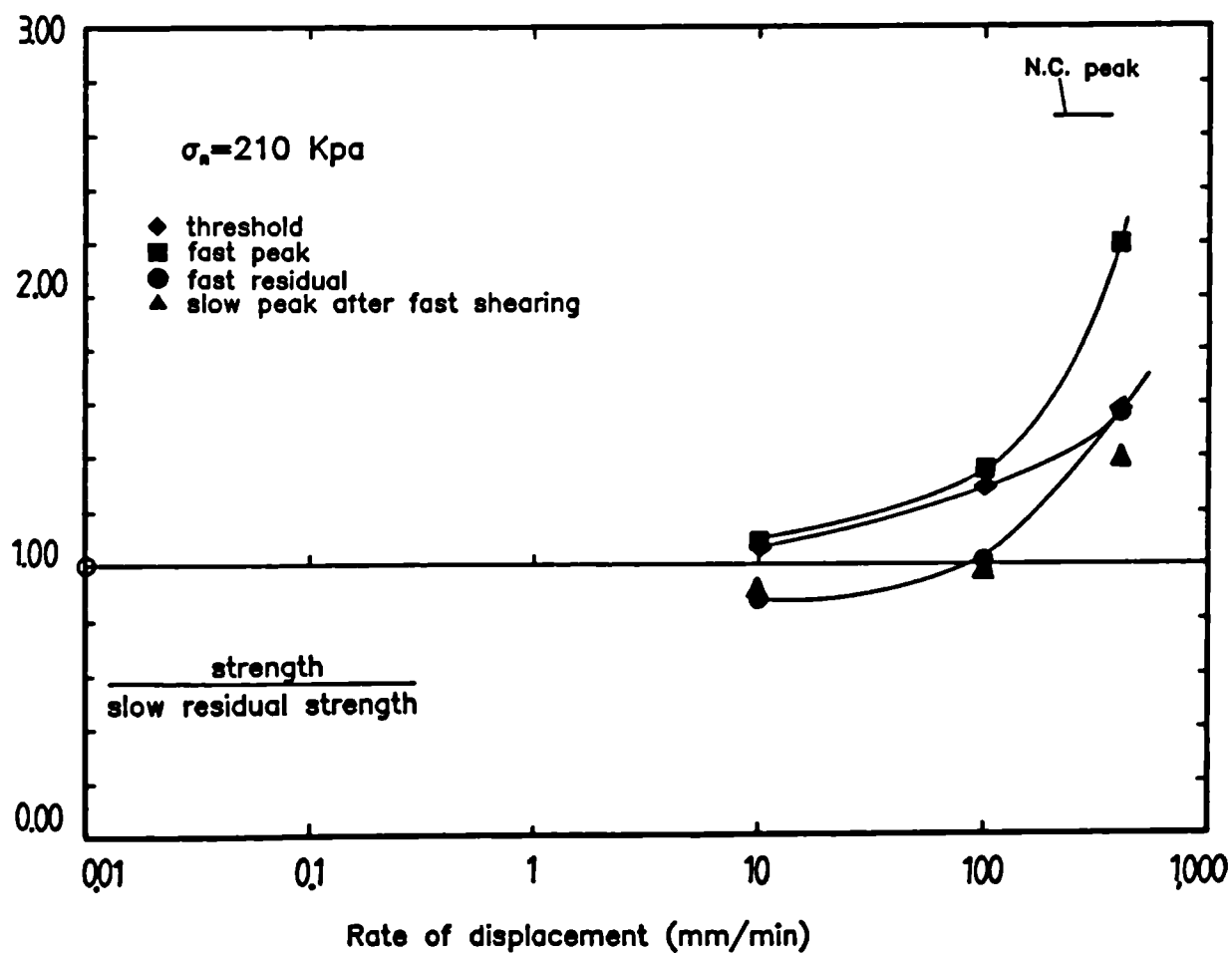
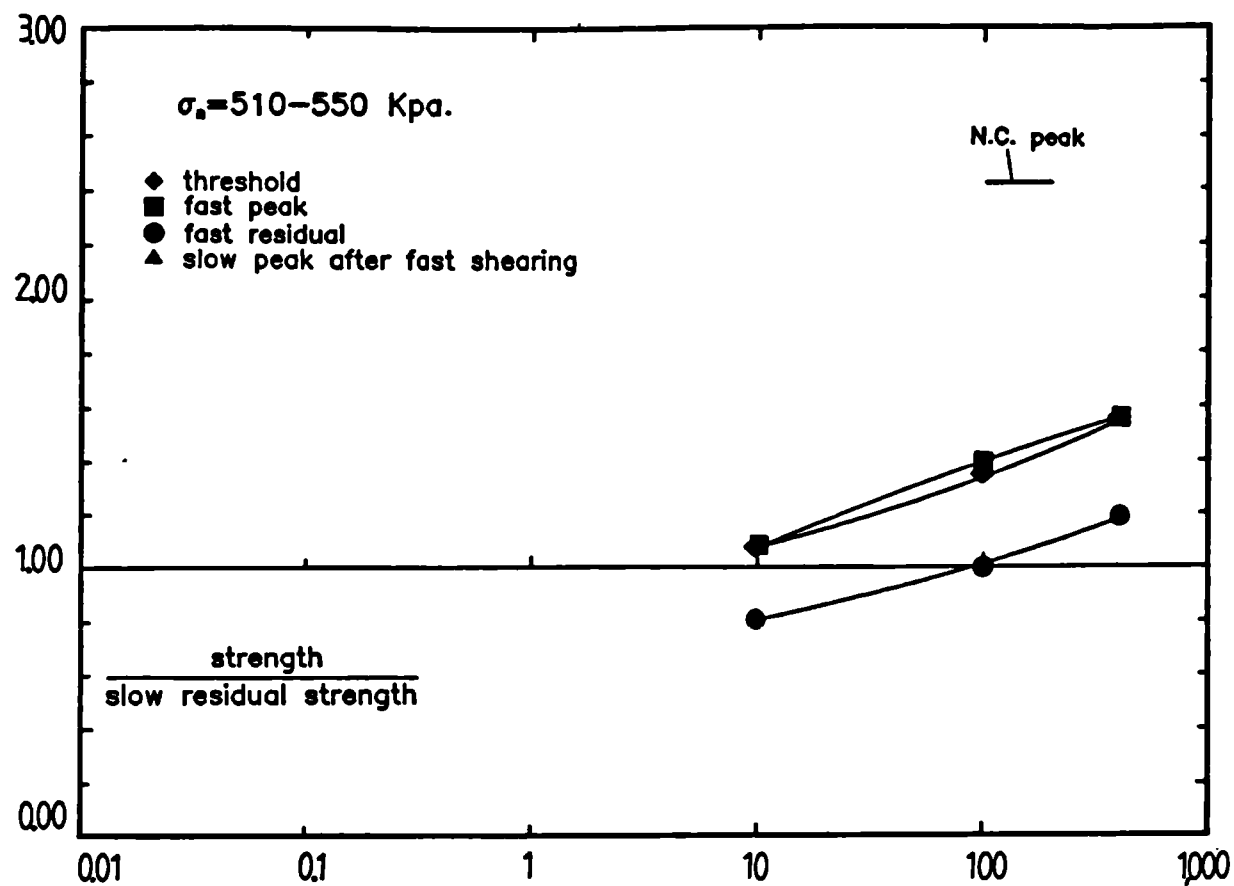


Fig. 4.19 Results from Tests 1 and 2 on claystone A.

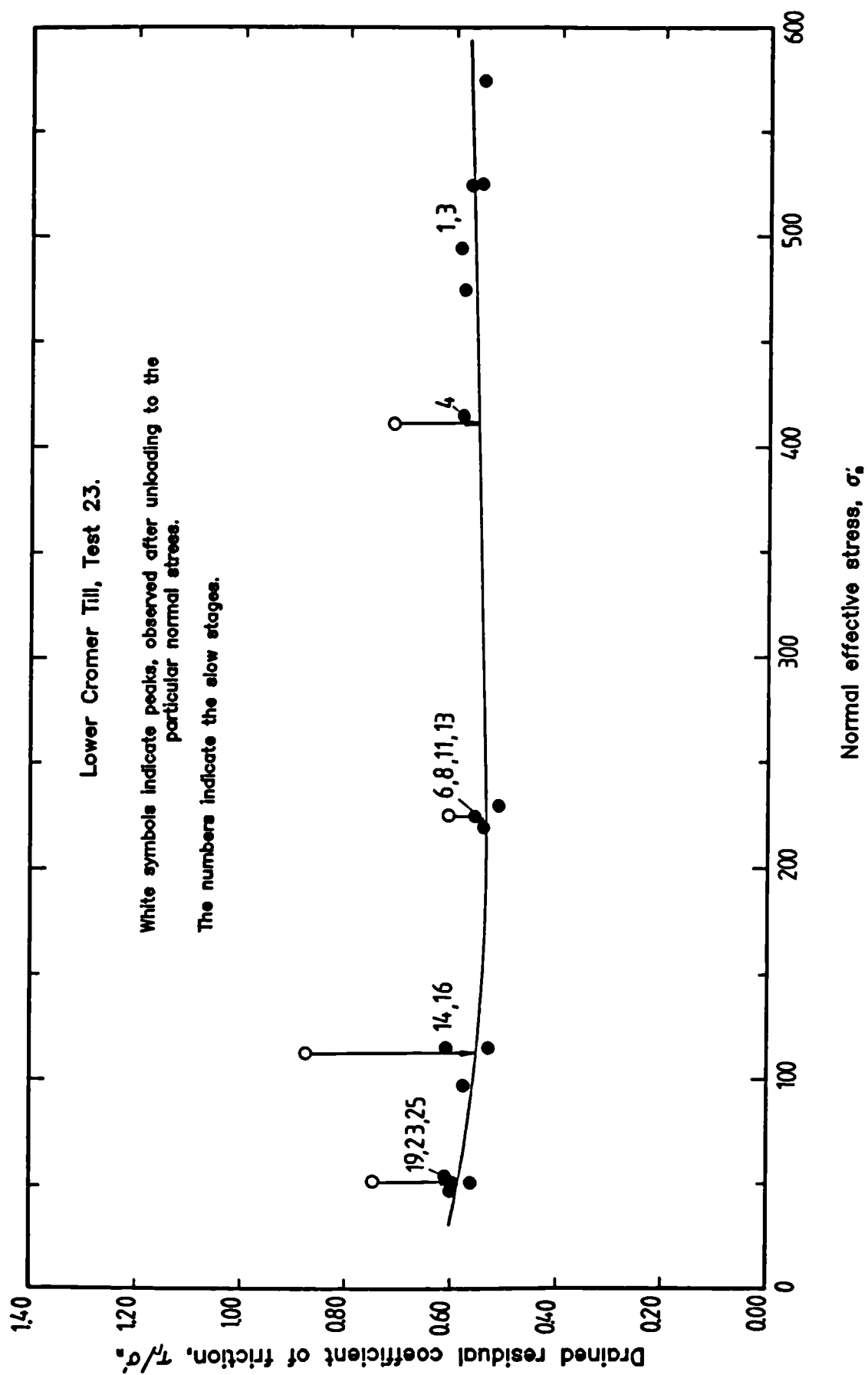


Fig. 4.20

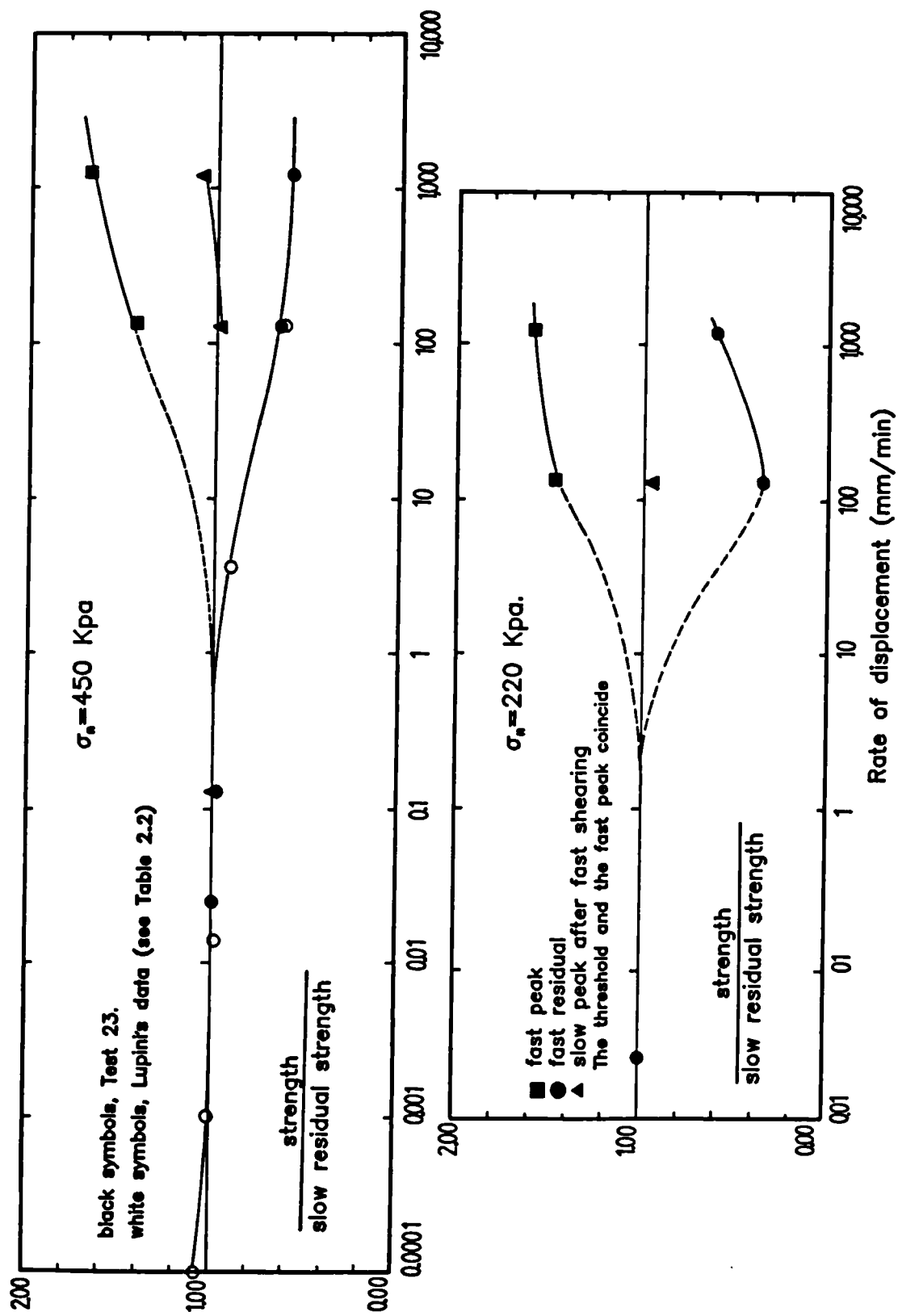


Fig. 4.21a Results from Test 23 on Lower Cromer Till.

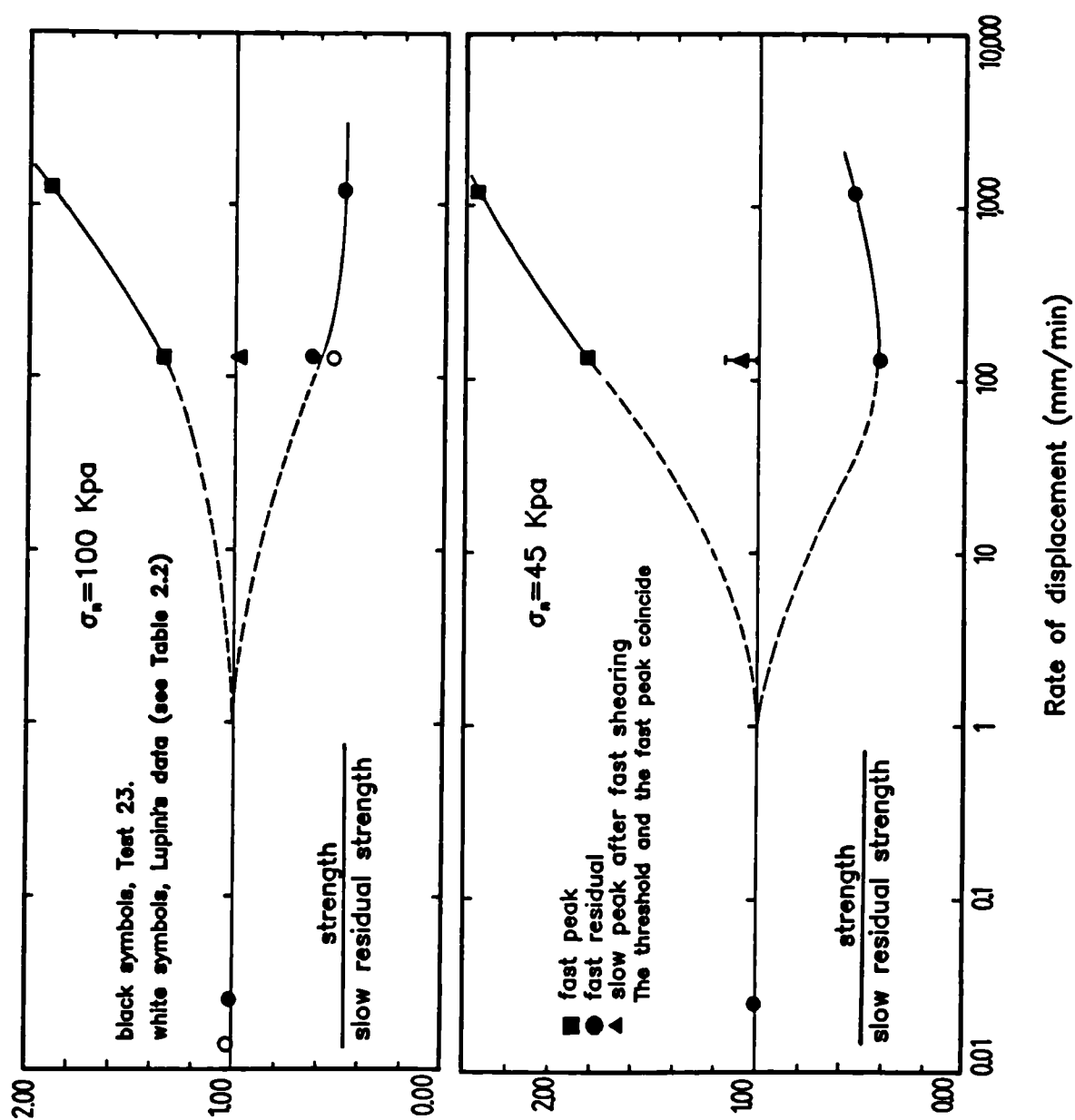


Fig 4.21b Results from Test 23 on Lower Cromer Till.

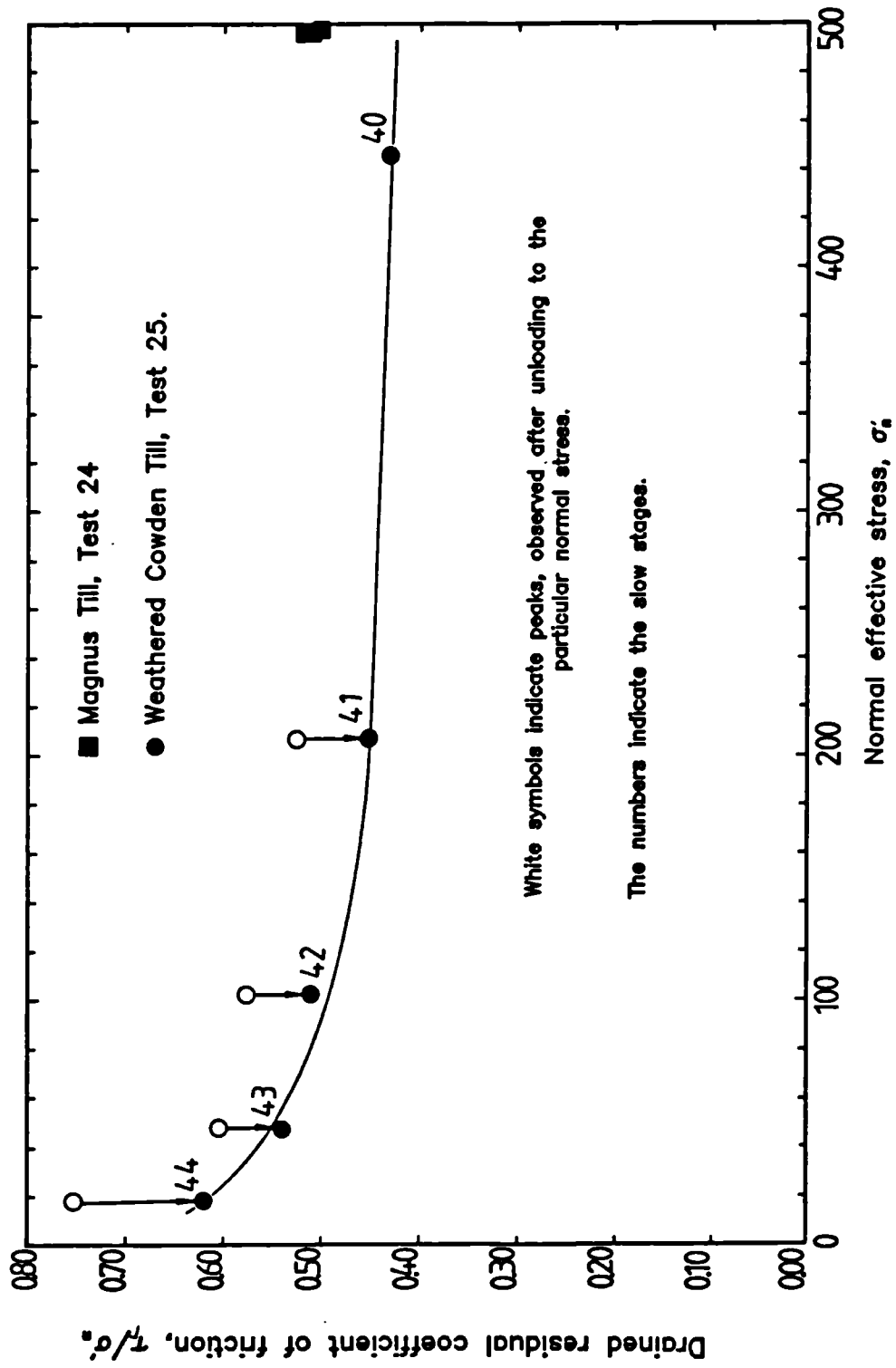


Fig. 4.22

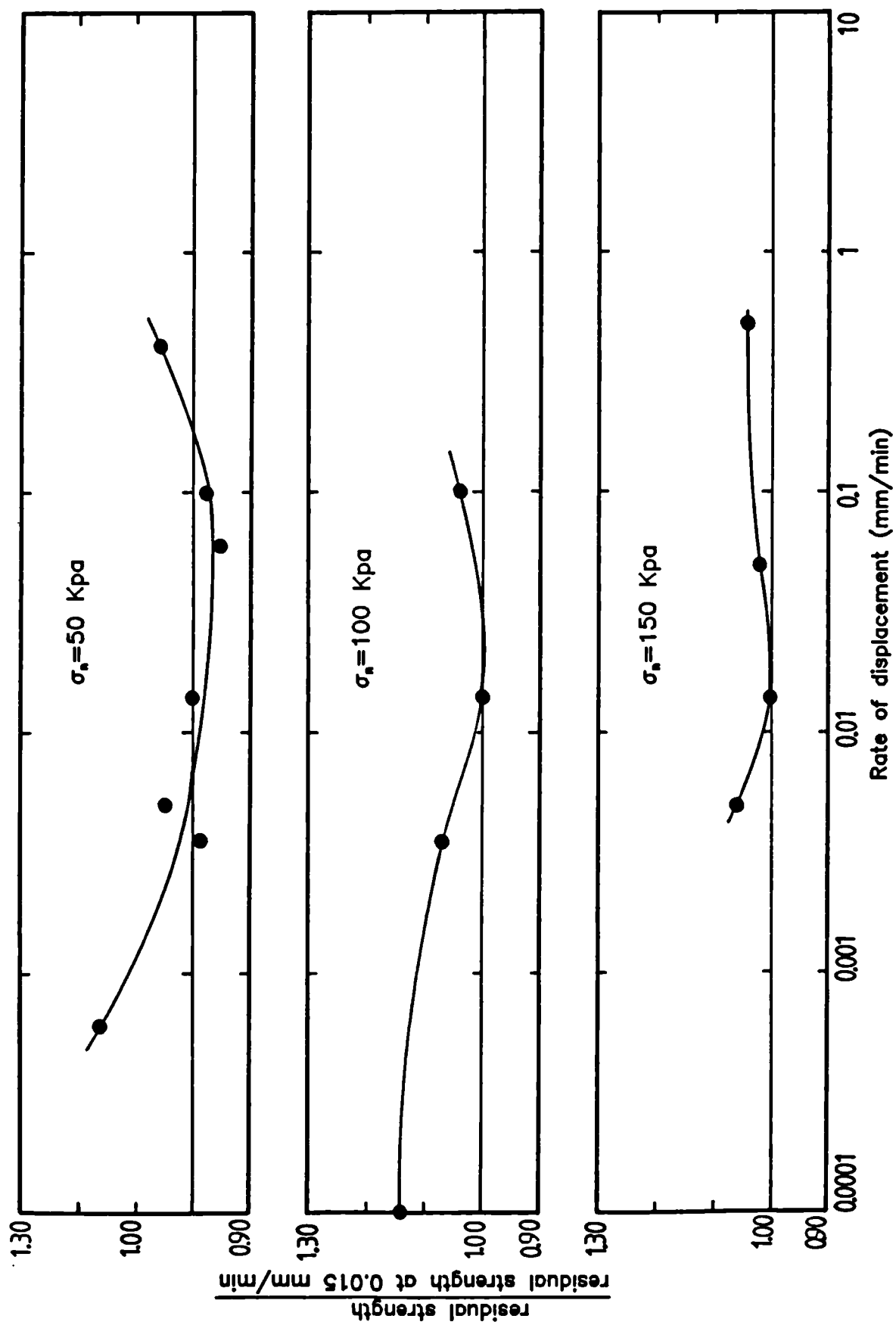


Fig. 4.23a Results from Test 25 on weathered Cowden Till.

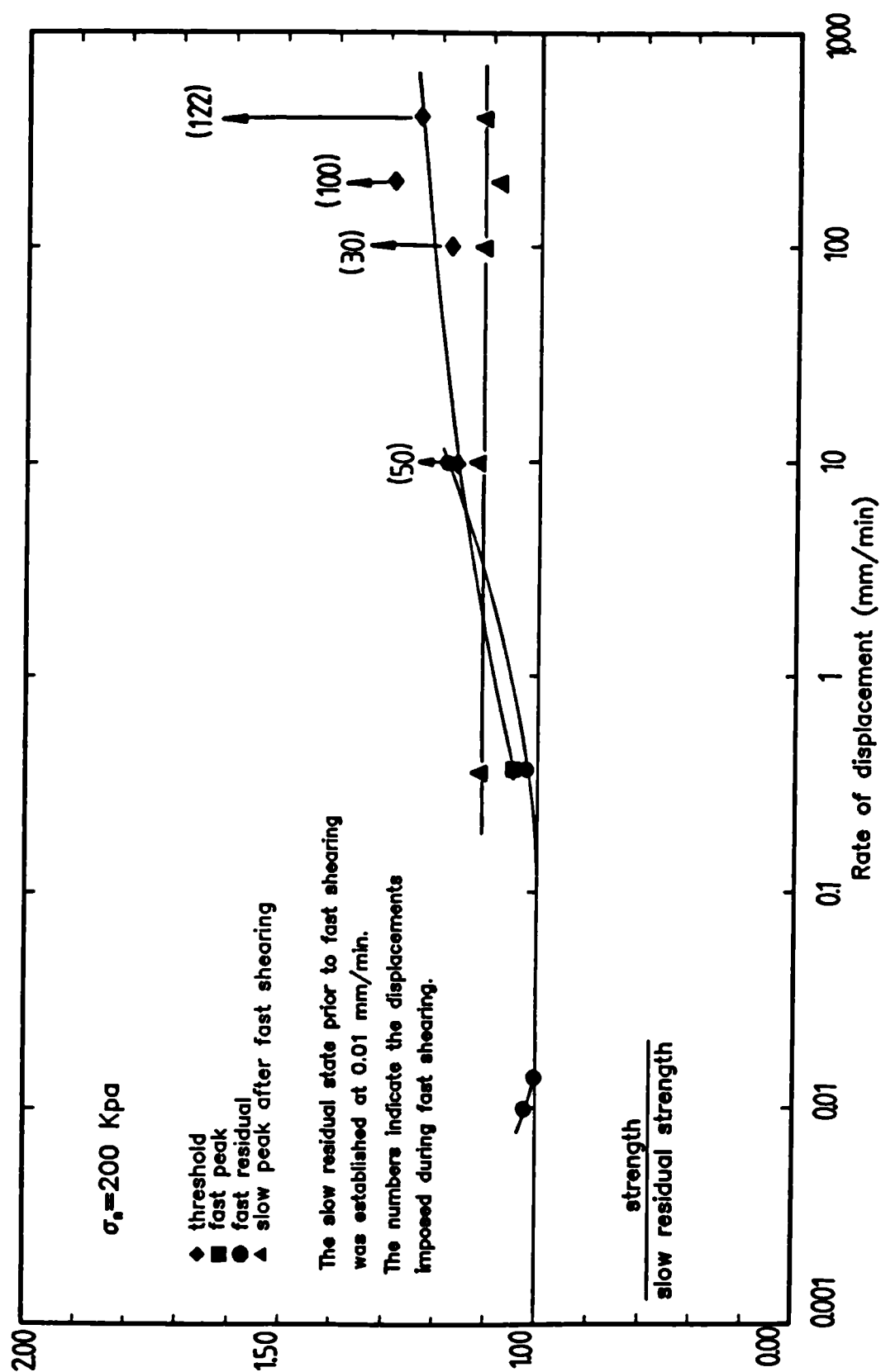


Fig. 4.23b Results from Test 25 on weathered Cowden Till.

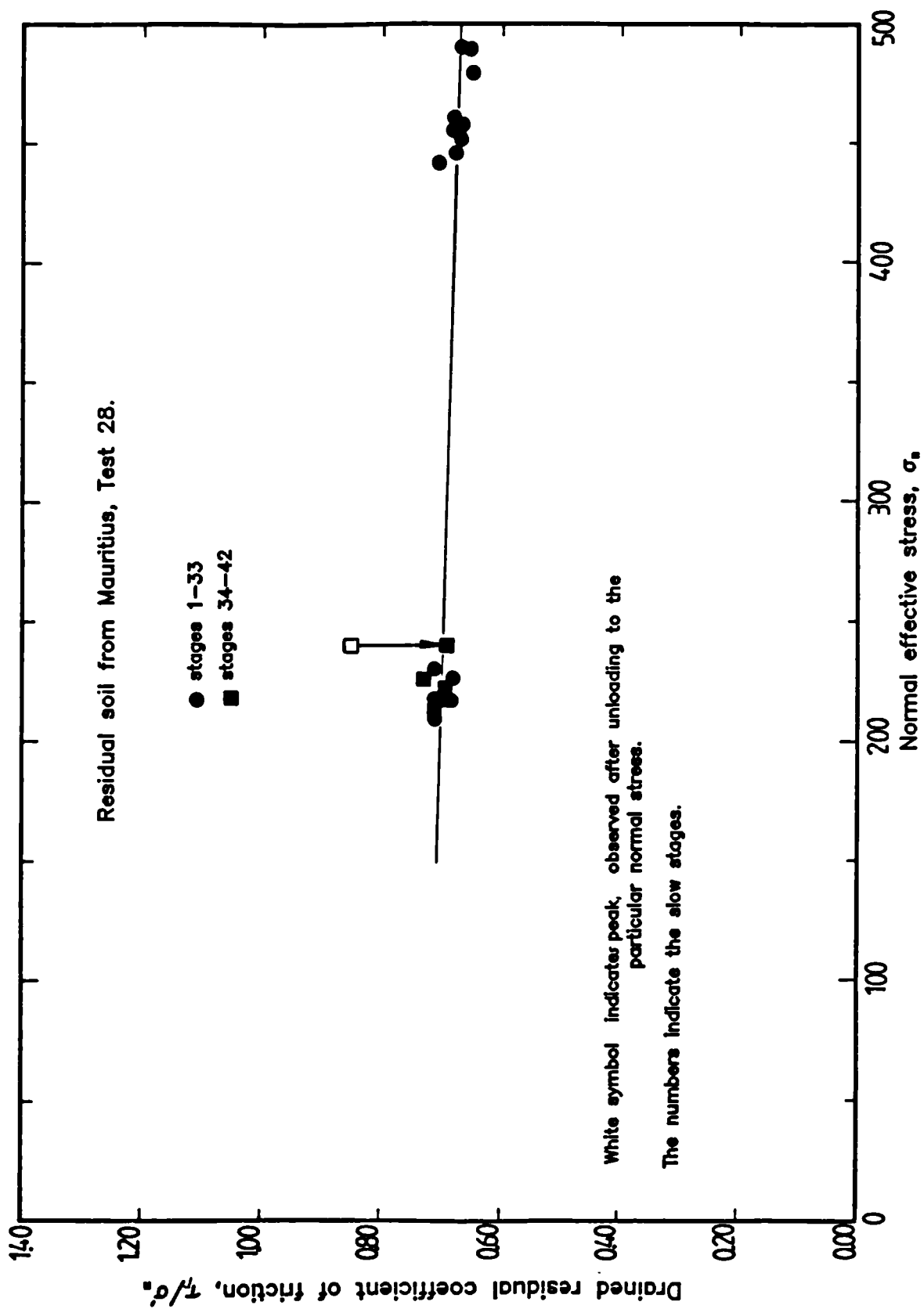


Fig. 4.24

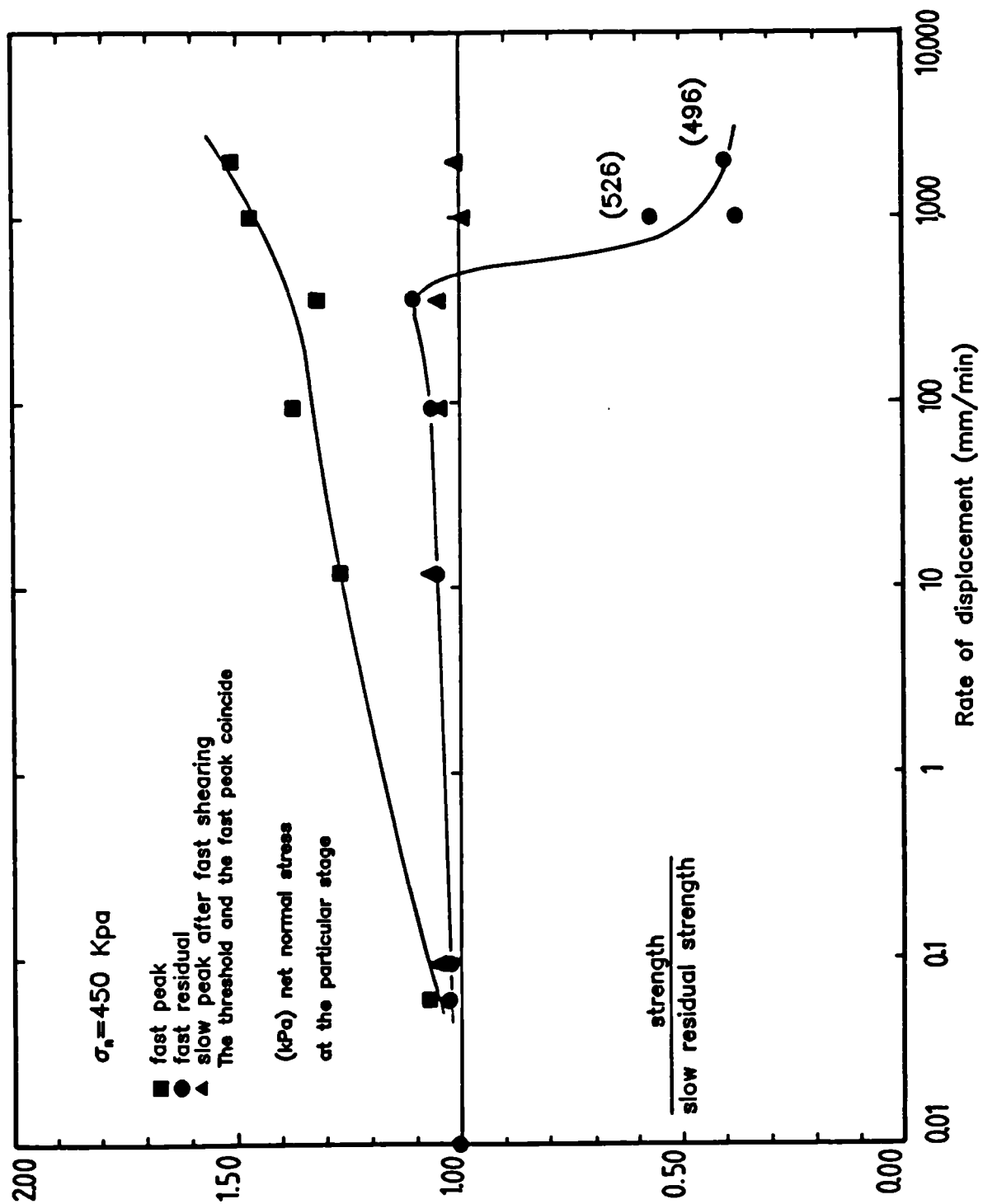


Fig. 4.25a Results from Test 28 on residual soil from Mauritius.

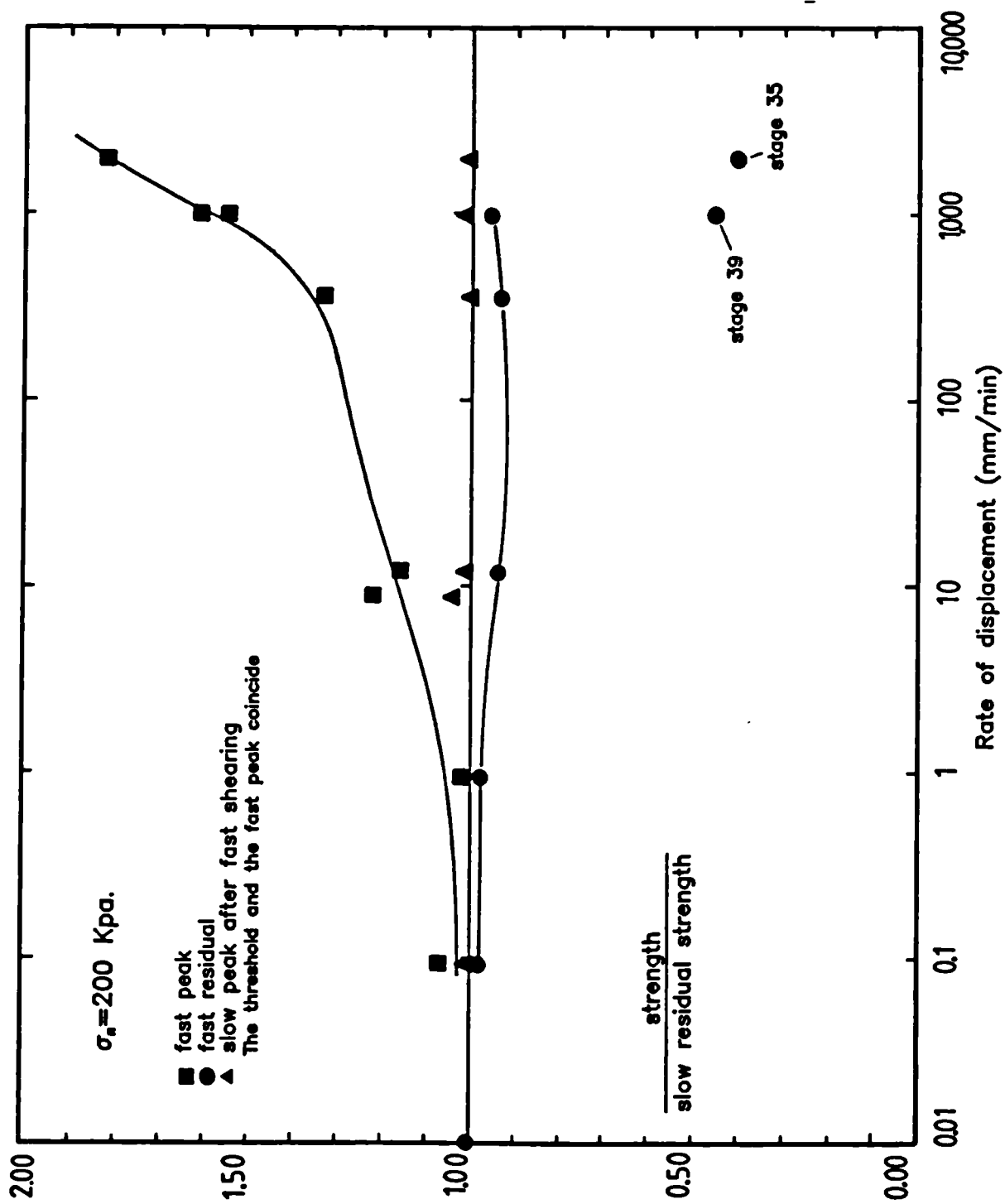


Fig. 4.25b Results from Test 28 on residual soil from Mauritius.

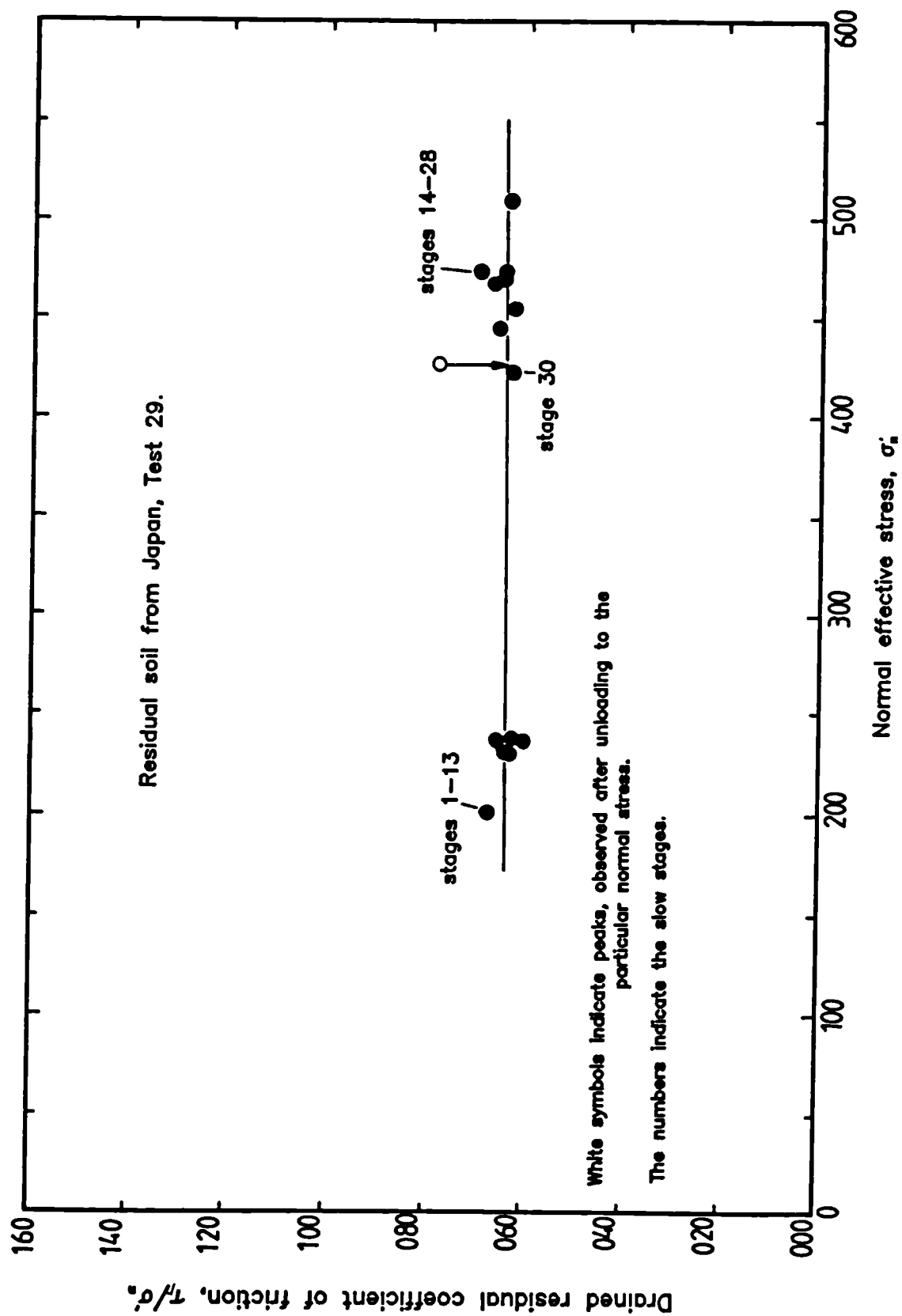


Fig. 4.26

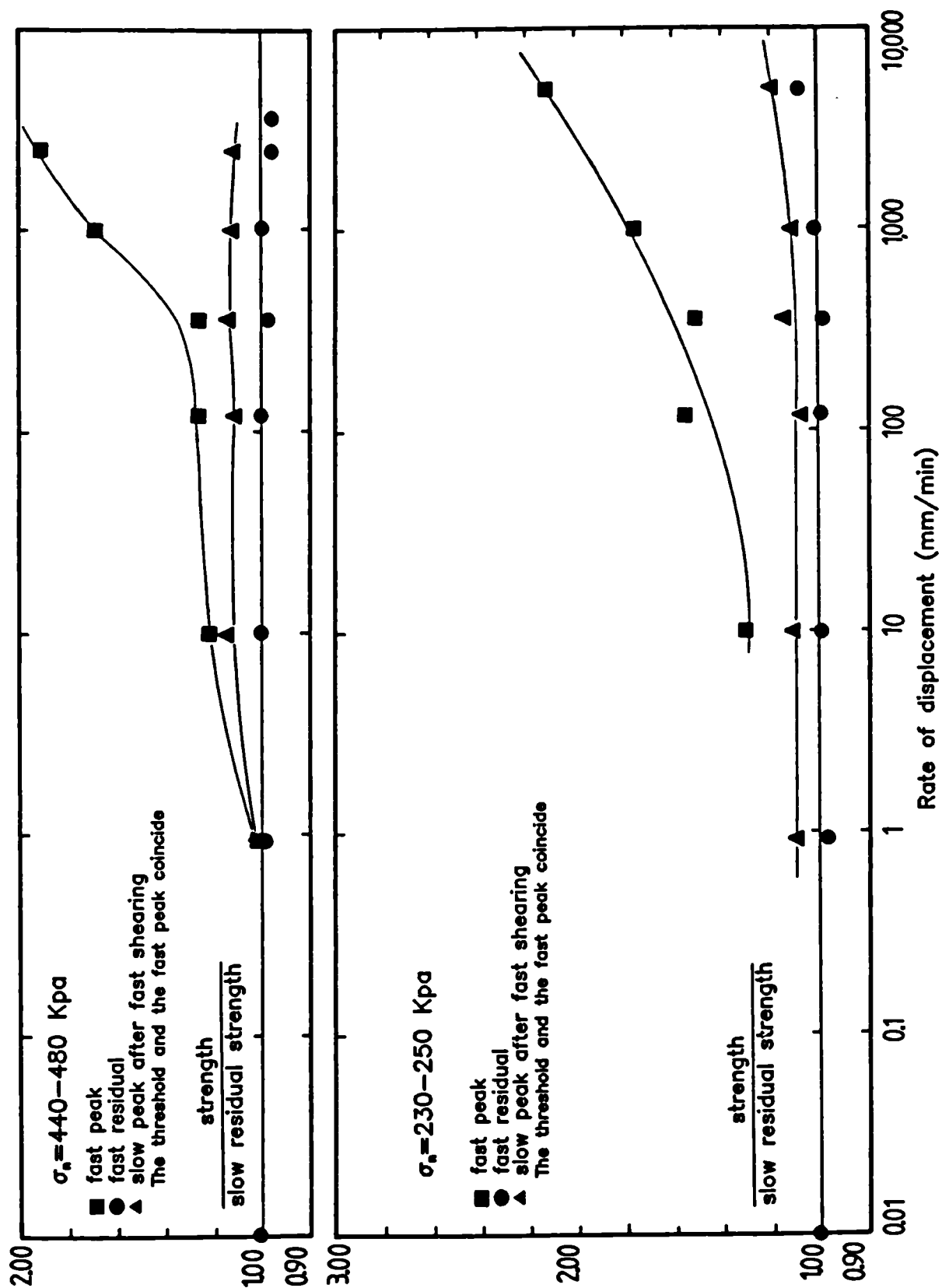


Fig. 4.27 Results from Test 29 on residual soil from Japan.

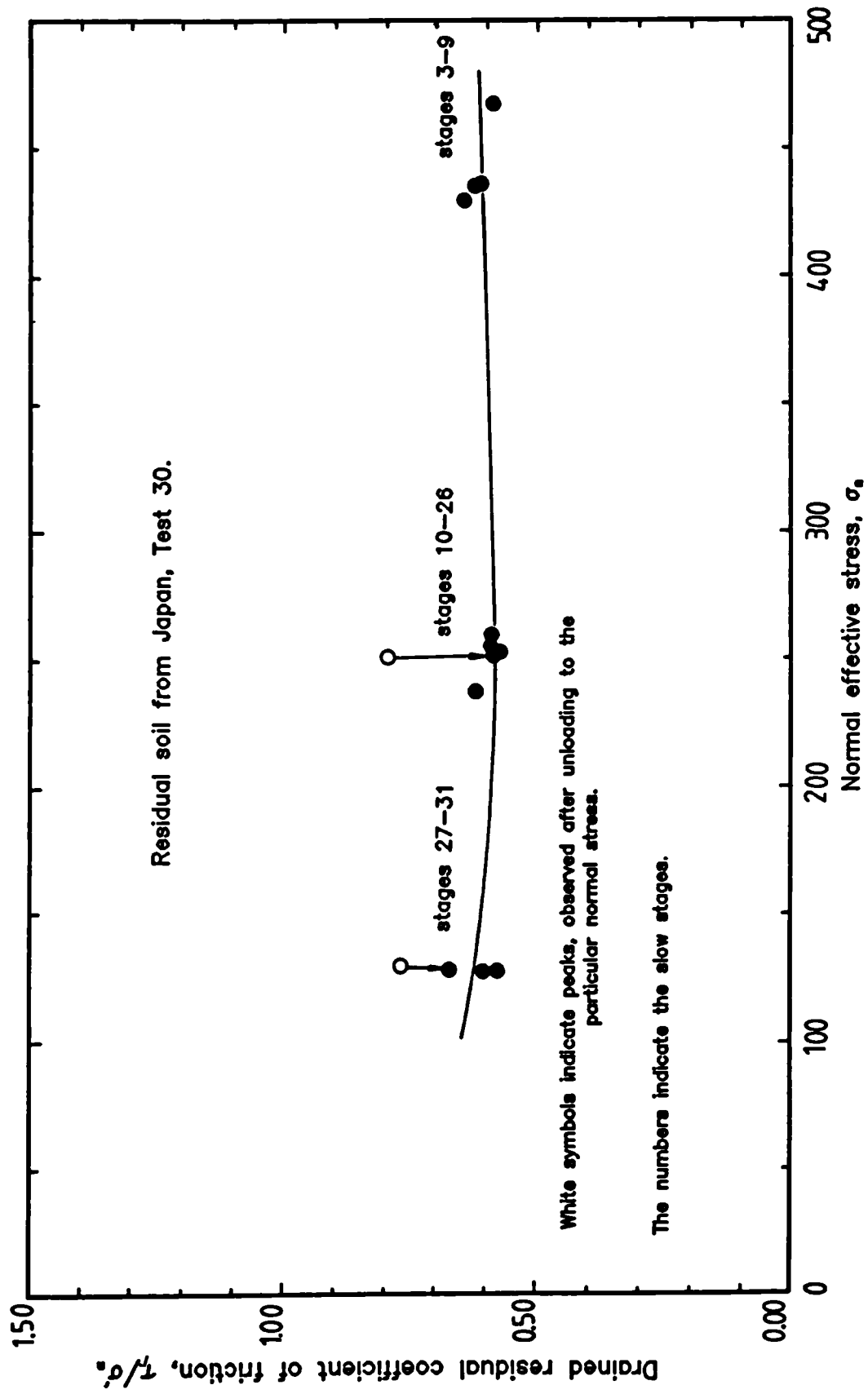


Fig. 4.28

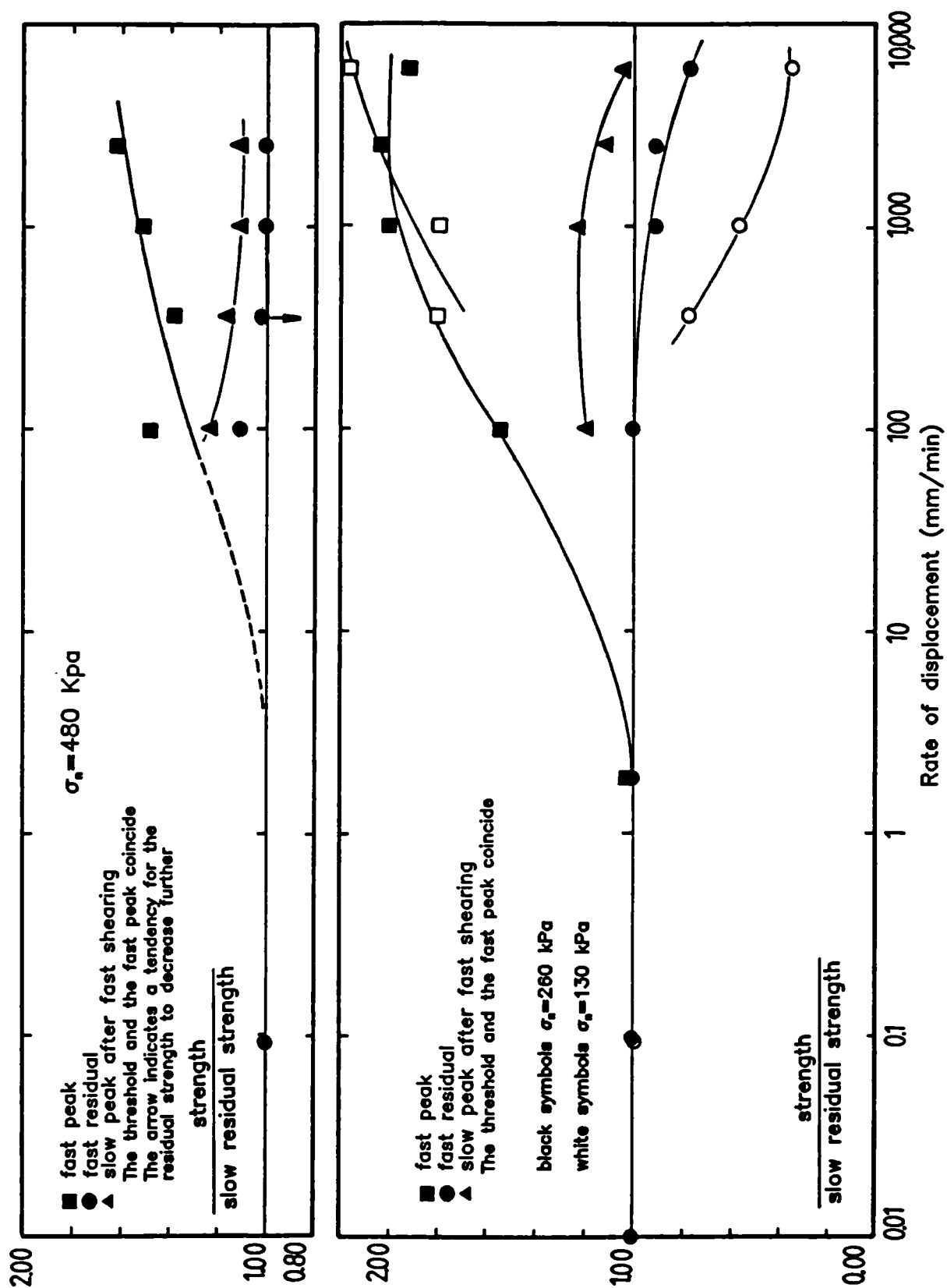


Fig. 4.29 Results from Test 30 on residual soil from Japan.

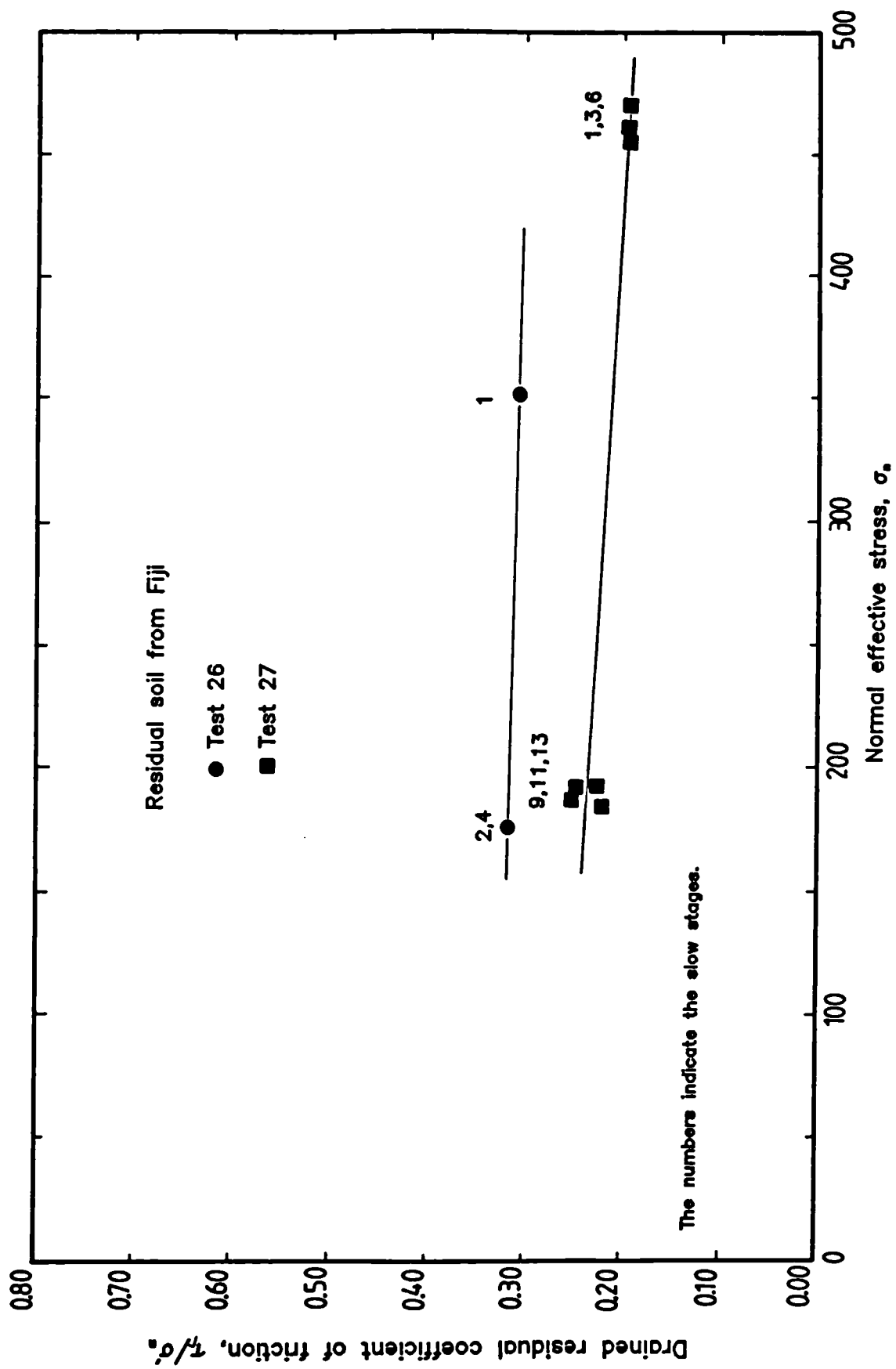


Fig. 4.30

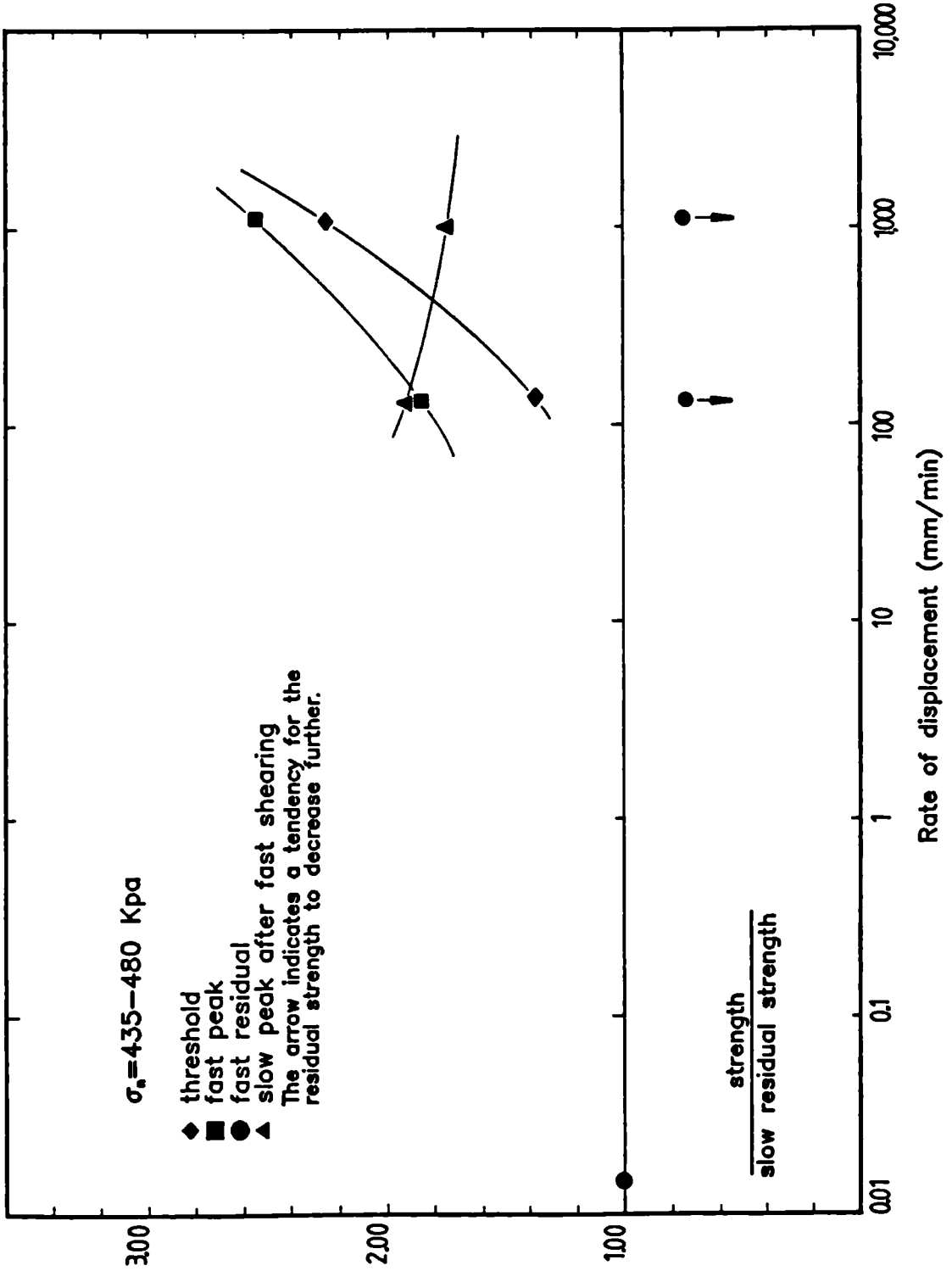


Fig. 4.31a Results from Test 27 on residual soil from Fiji.

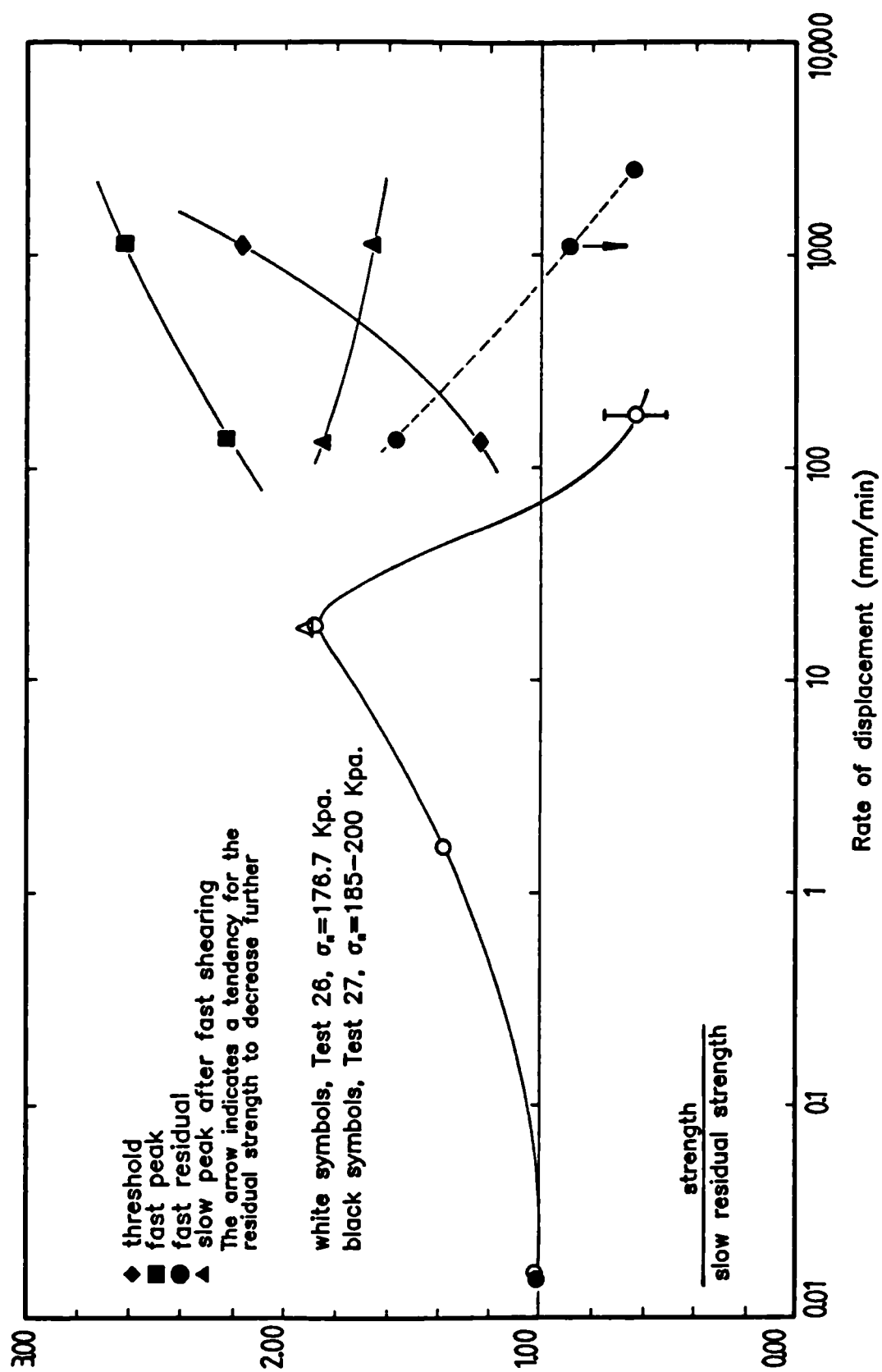


Fig. 4.31b Results from Tests 26 and 27 on residual soil from Fiji.

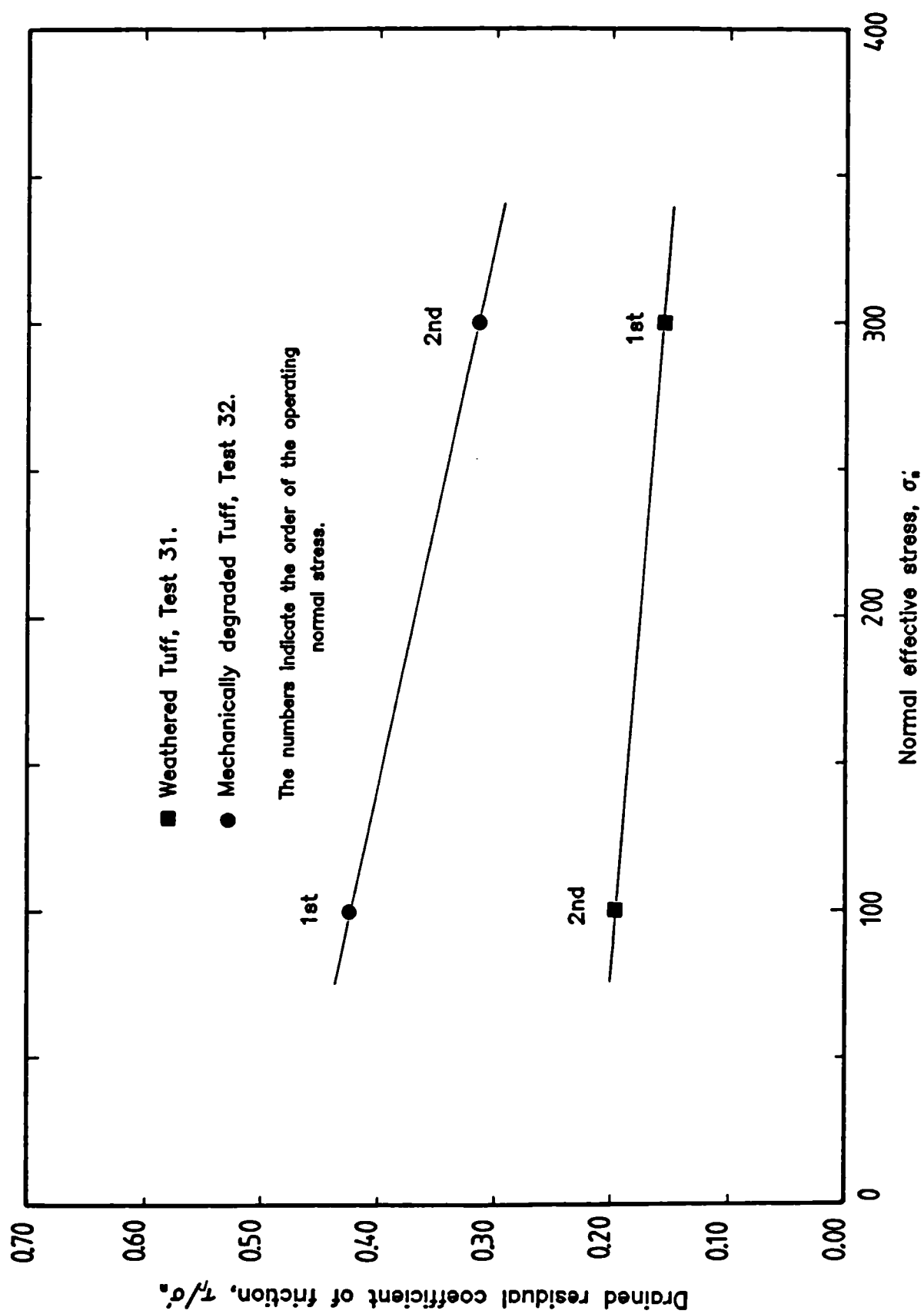


Fig. 4.32

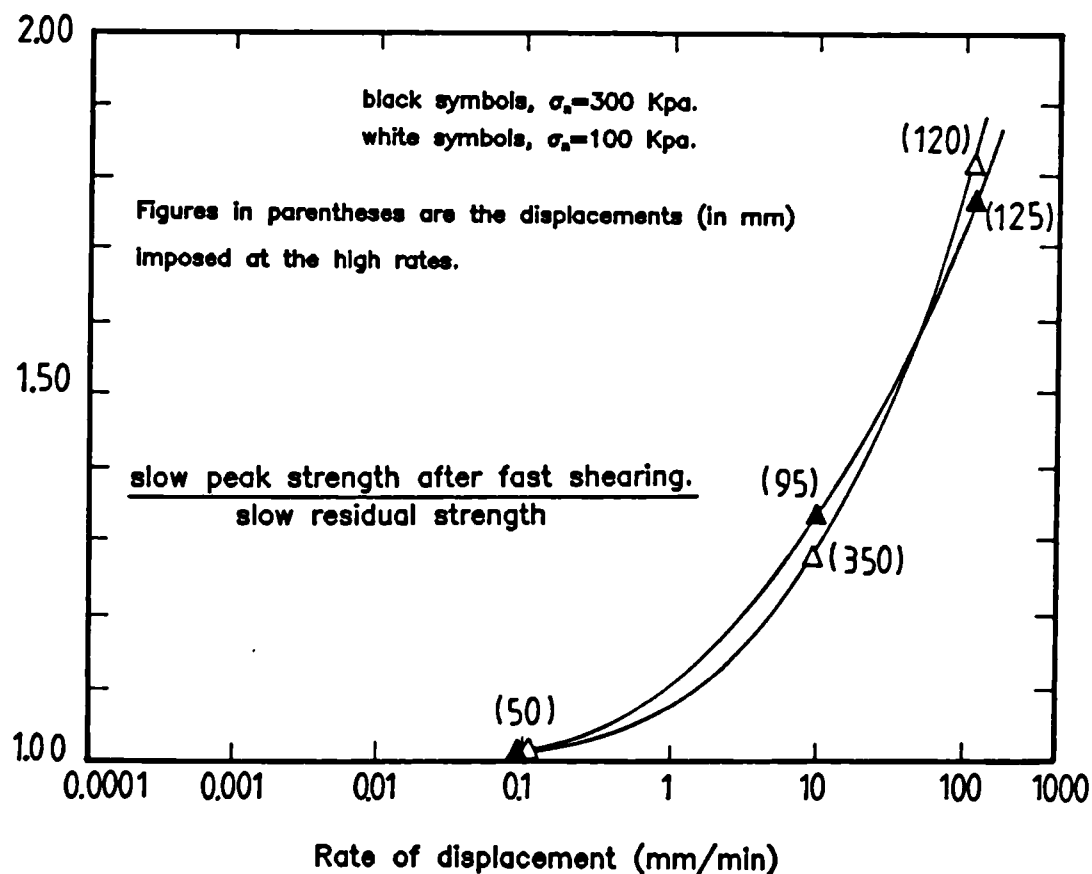
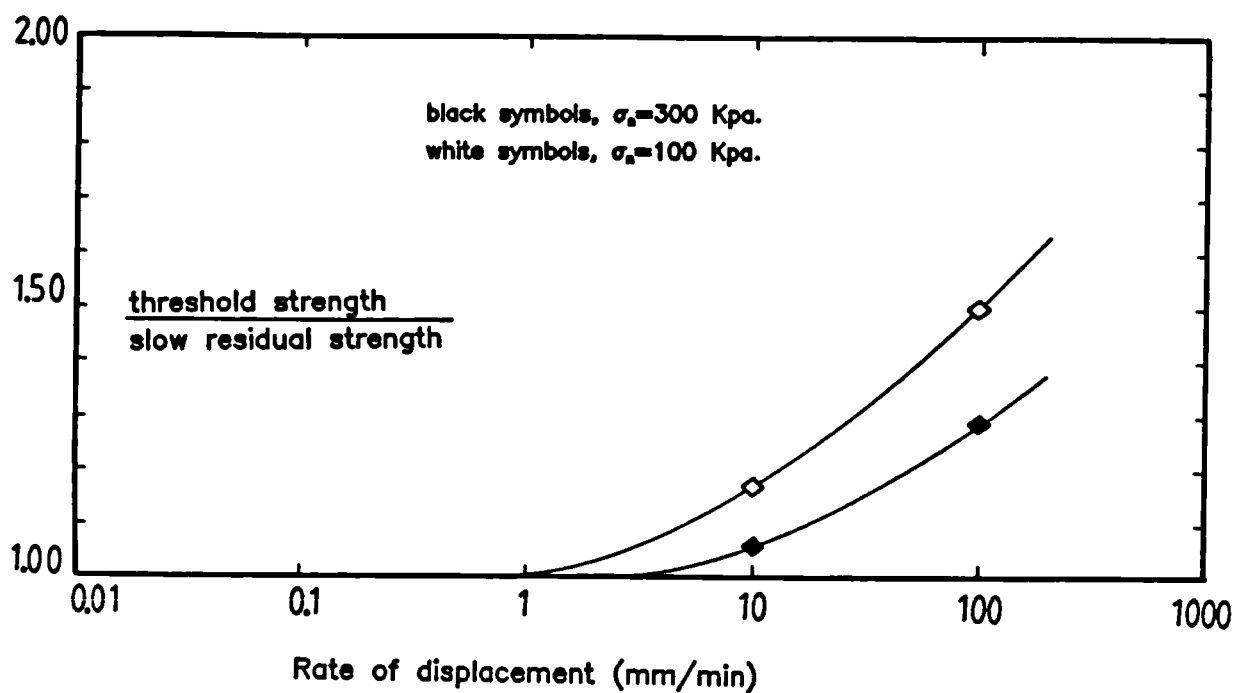


Fig. 4.33 Results from Test 31 on weathered Tuff.

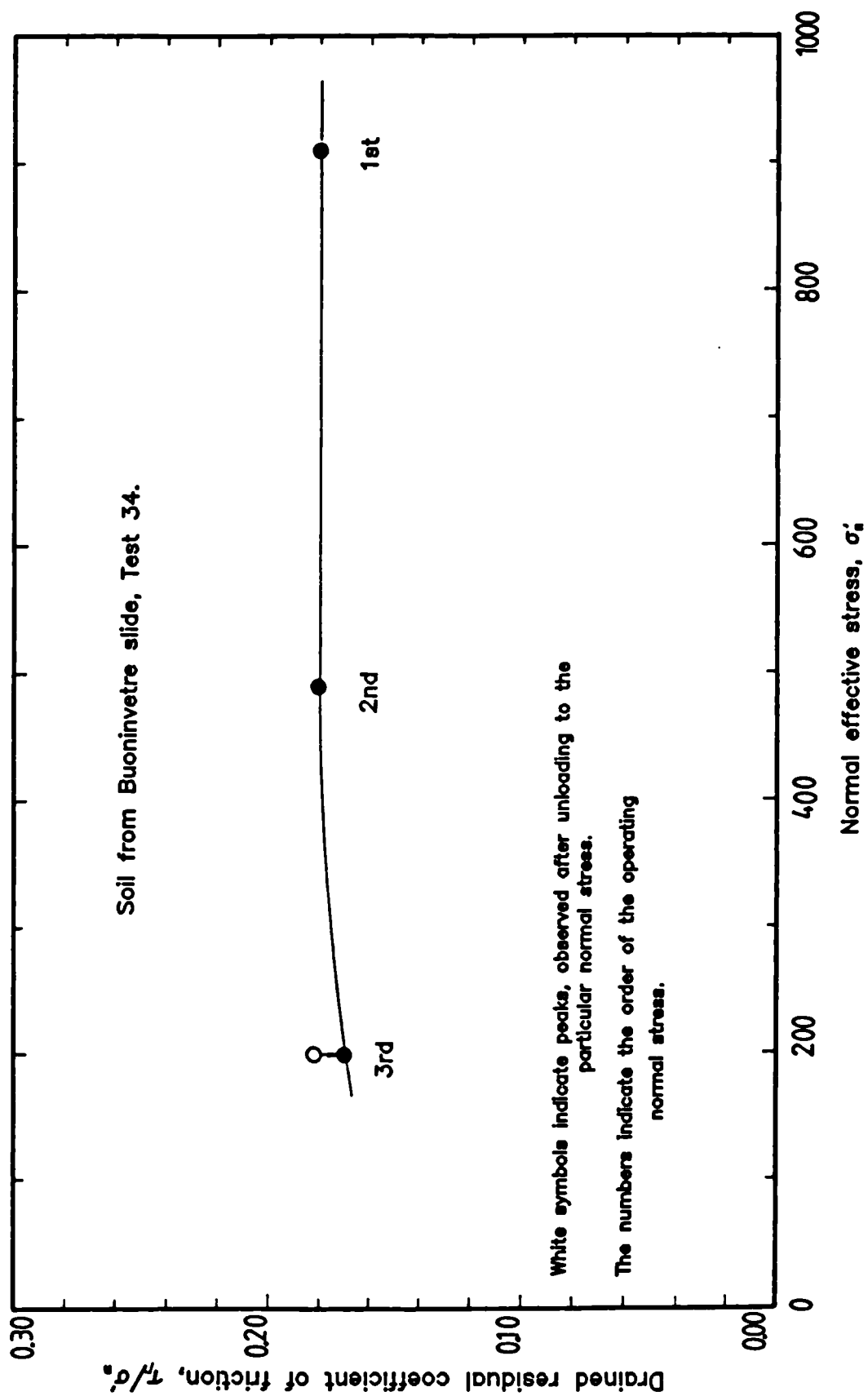


Fig. 4.34

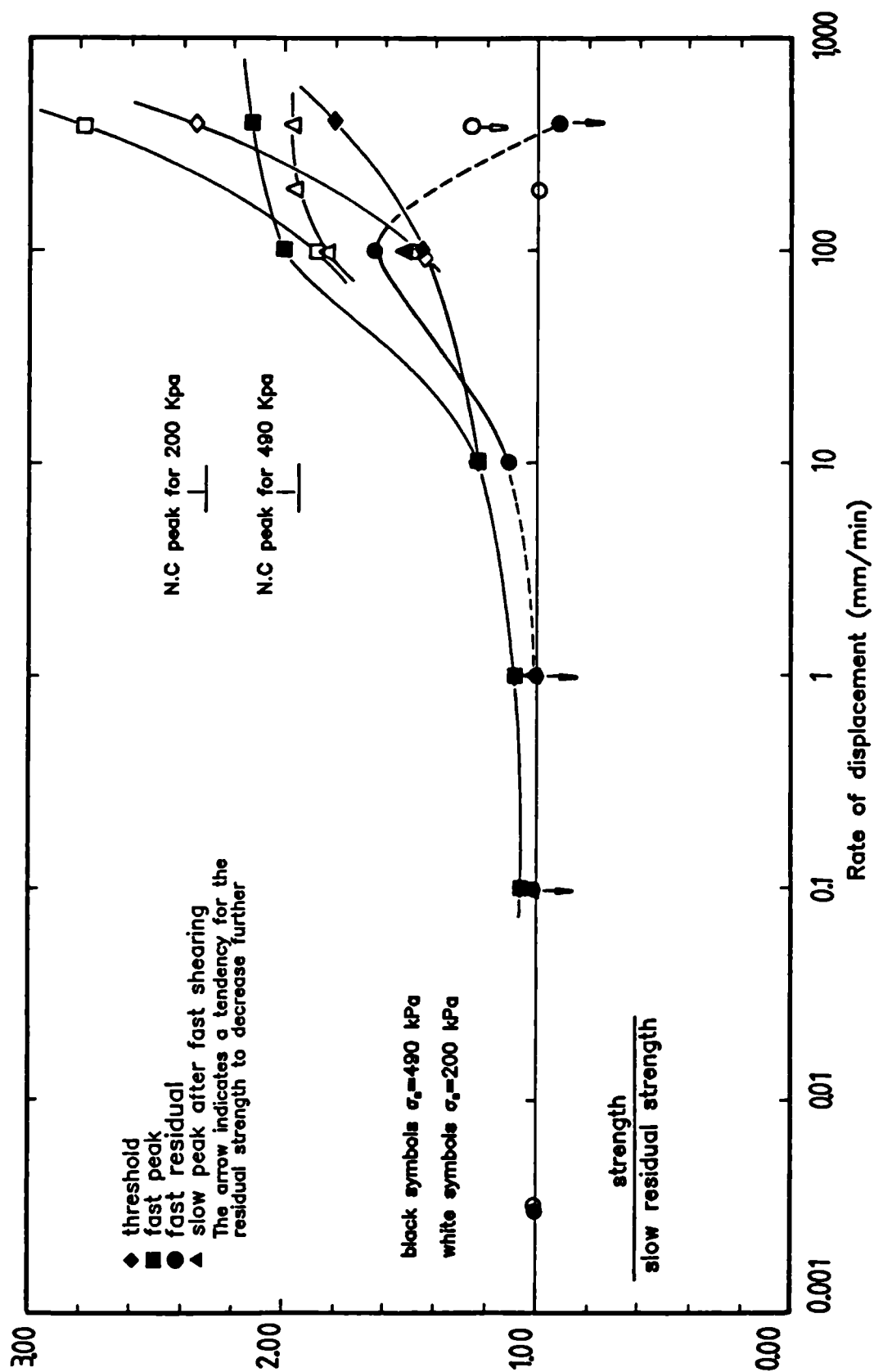


Fig. 4.35 Results from Test 34 on soil from Buoninvetre slide.

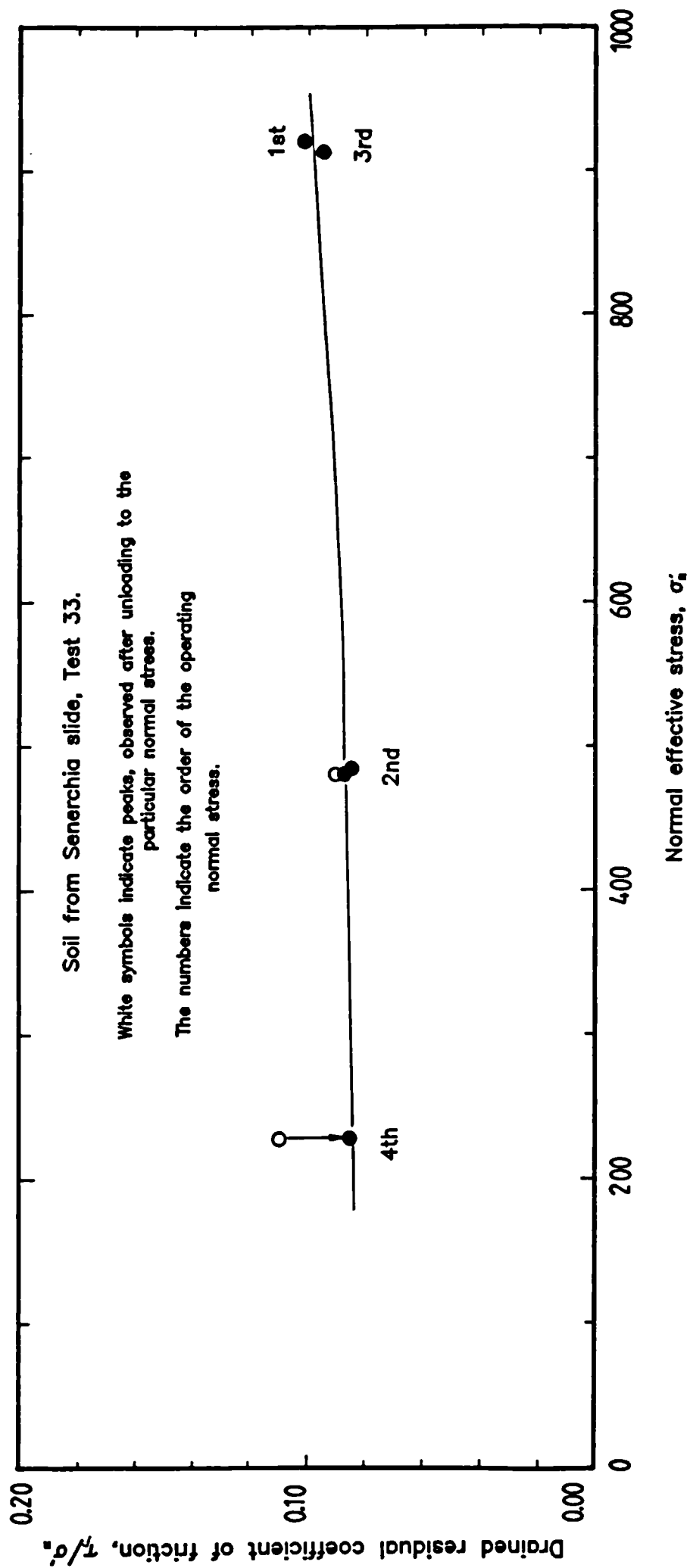


Fig. 4.36

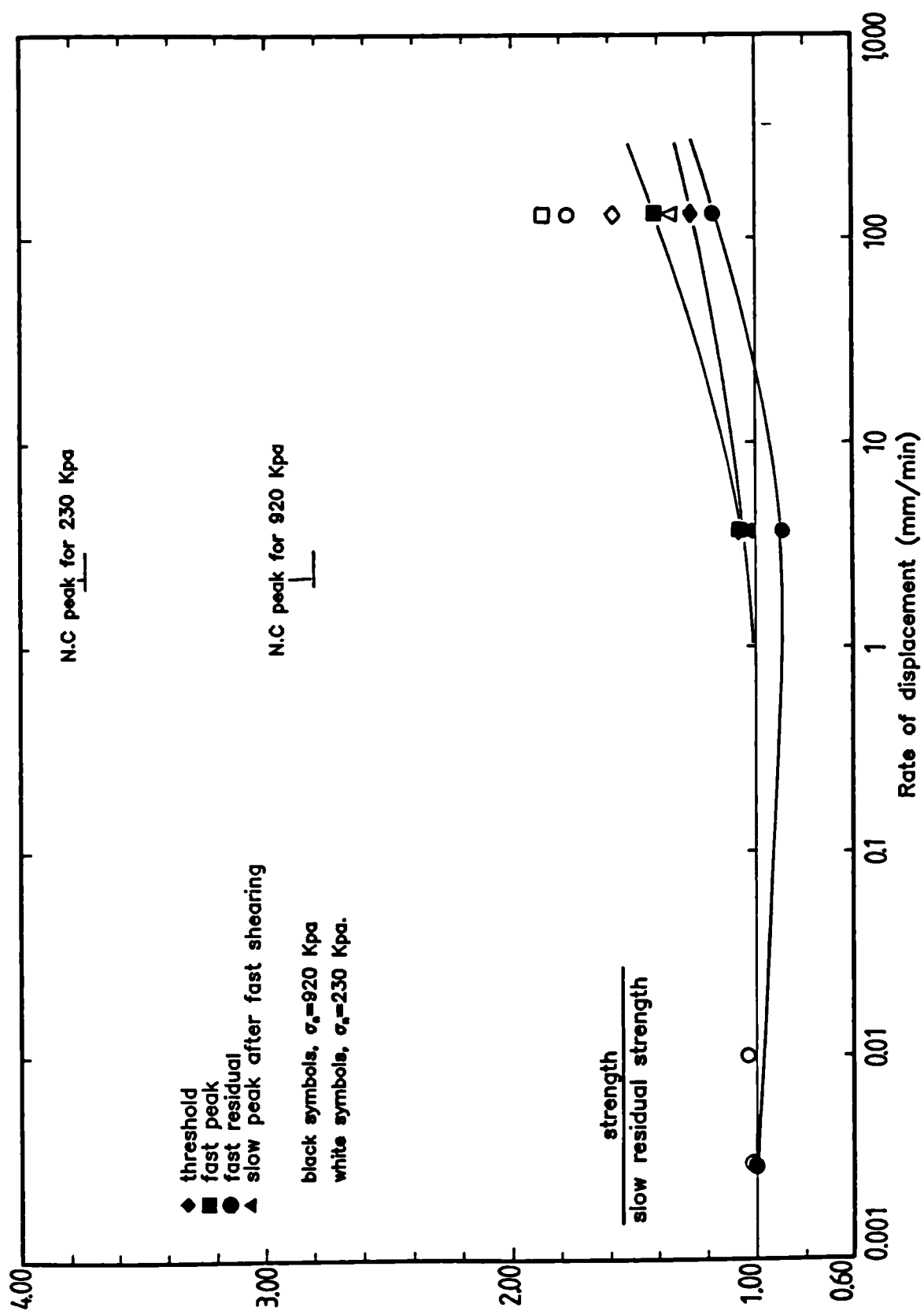


Fig. 4.37 Results from test 33 on soil from Senenrchia slide.

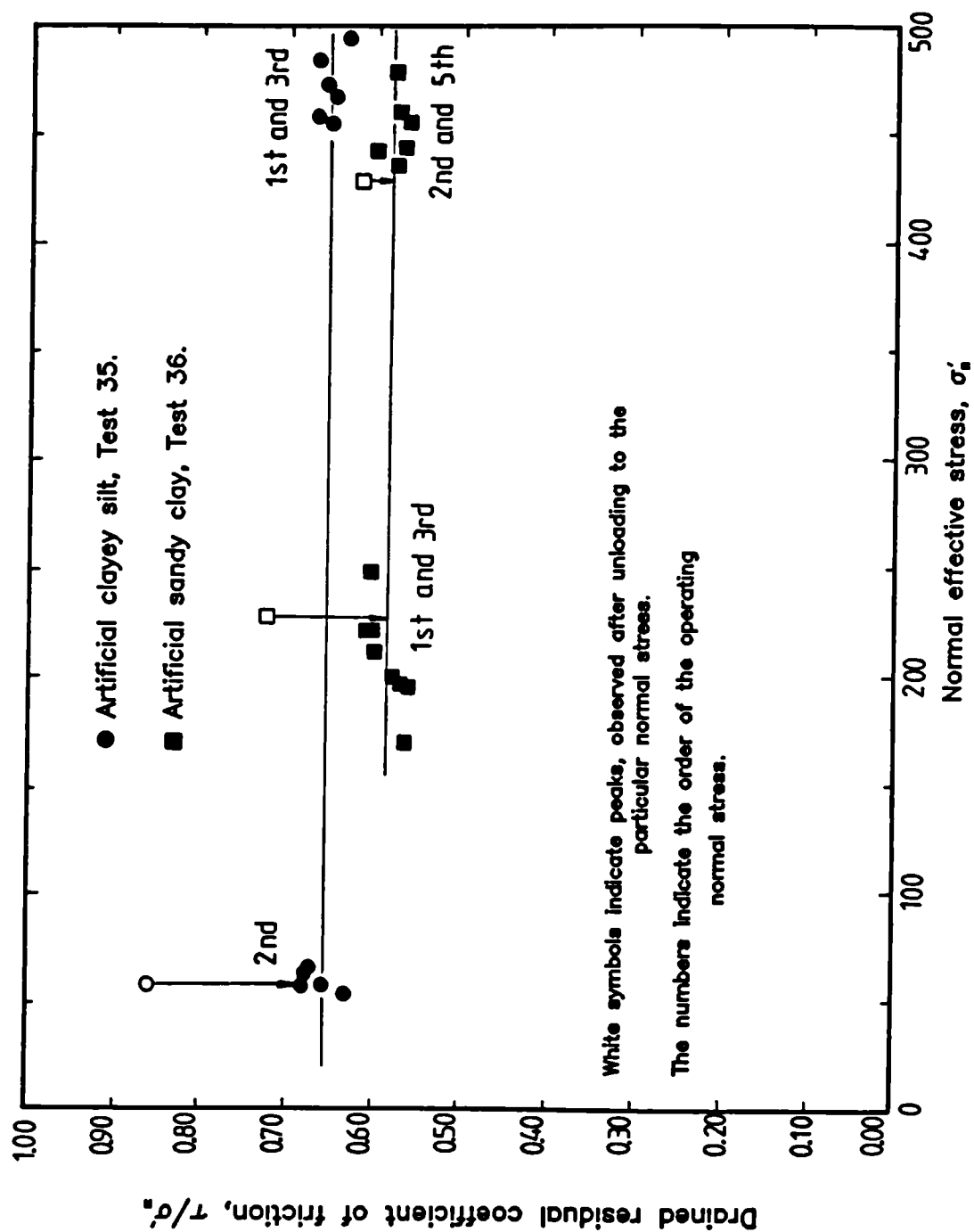


Fig. 4.38

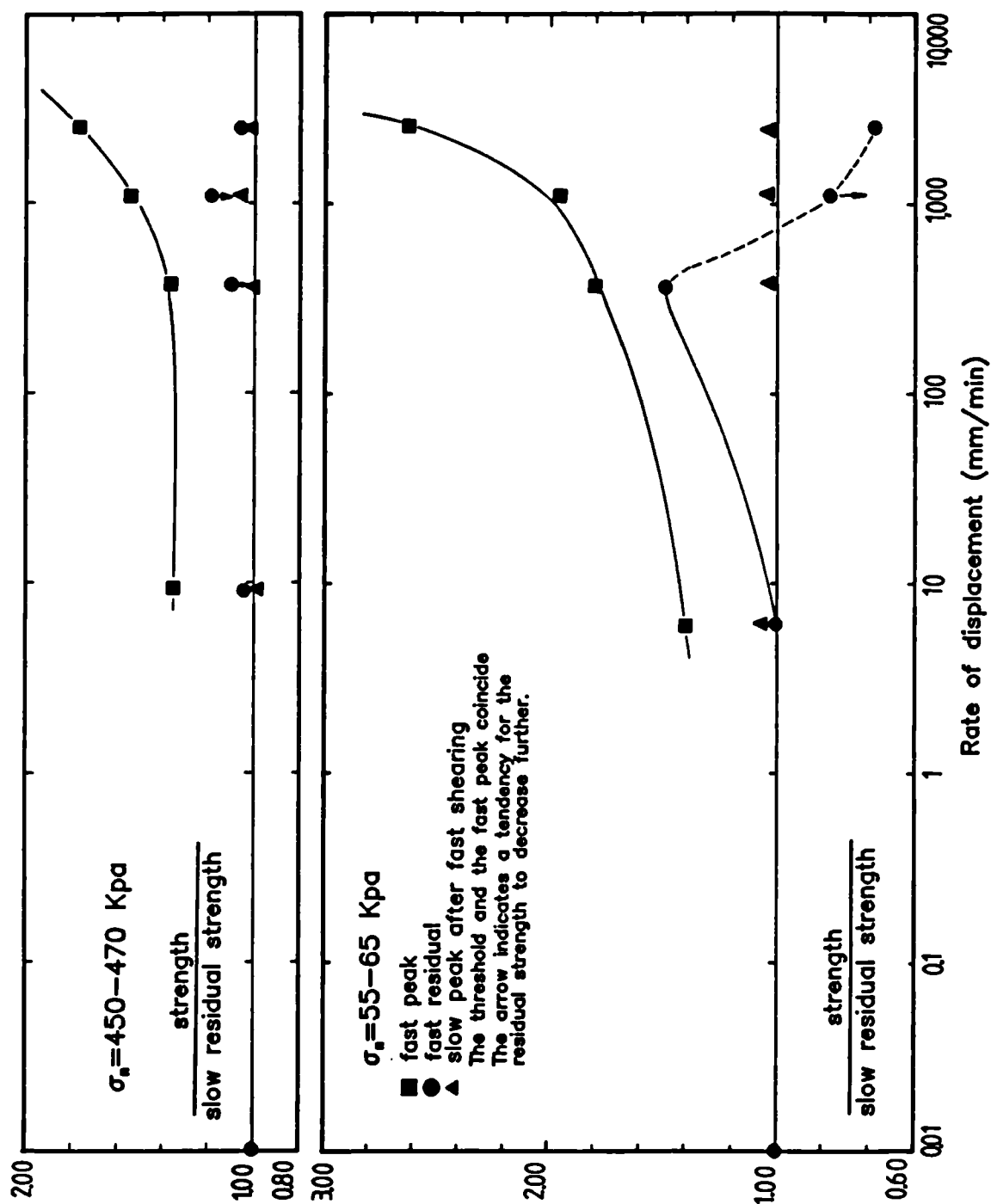


Fig. 4.39 Results from Test 35 on artificial clayey silt

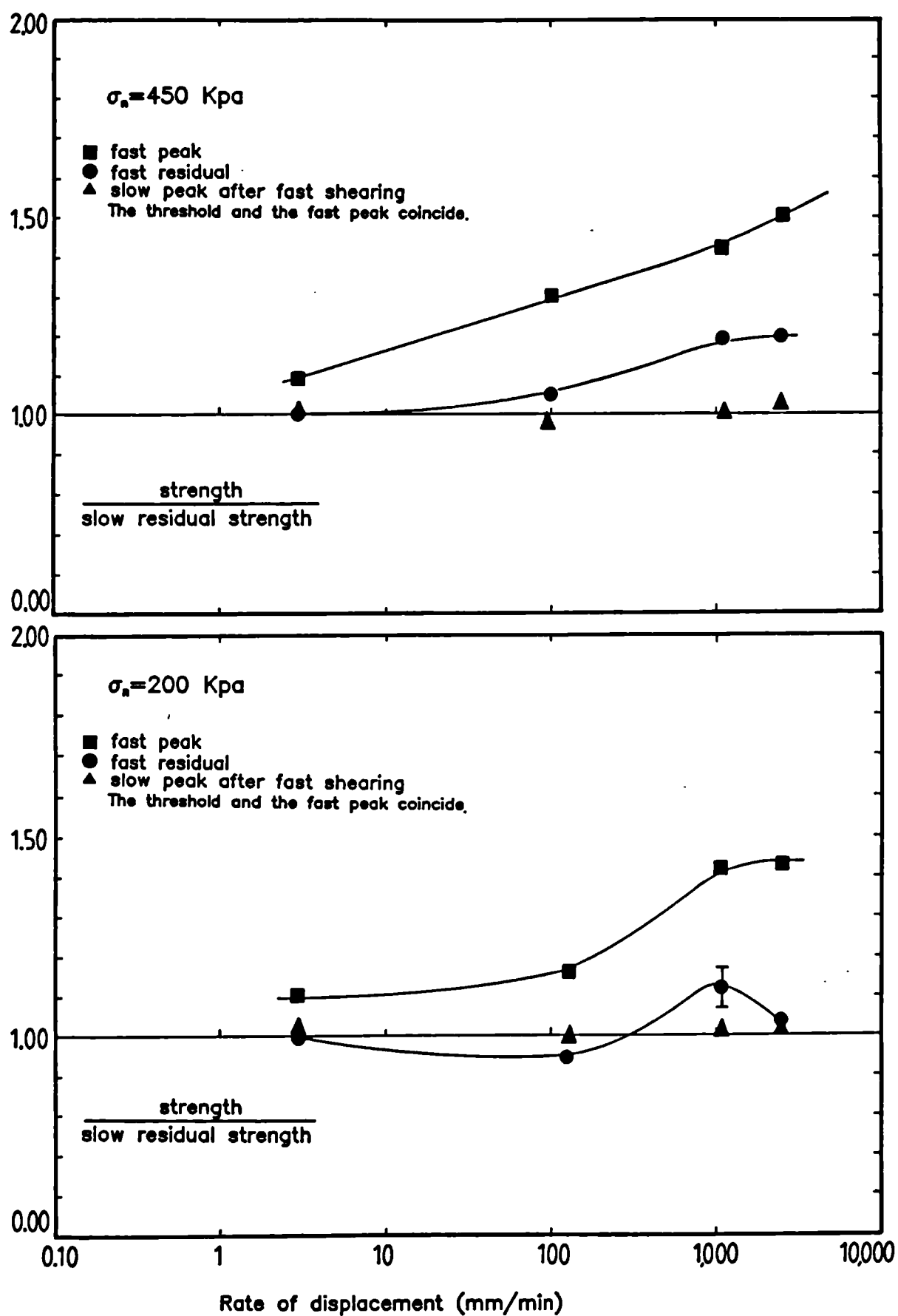


Fig. 4.40 Results from Test 36 on artificial sandy clay.

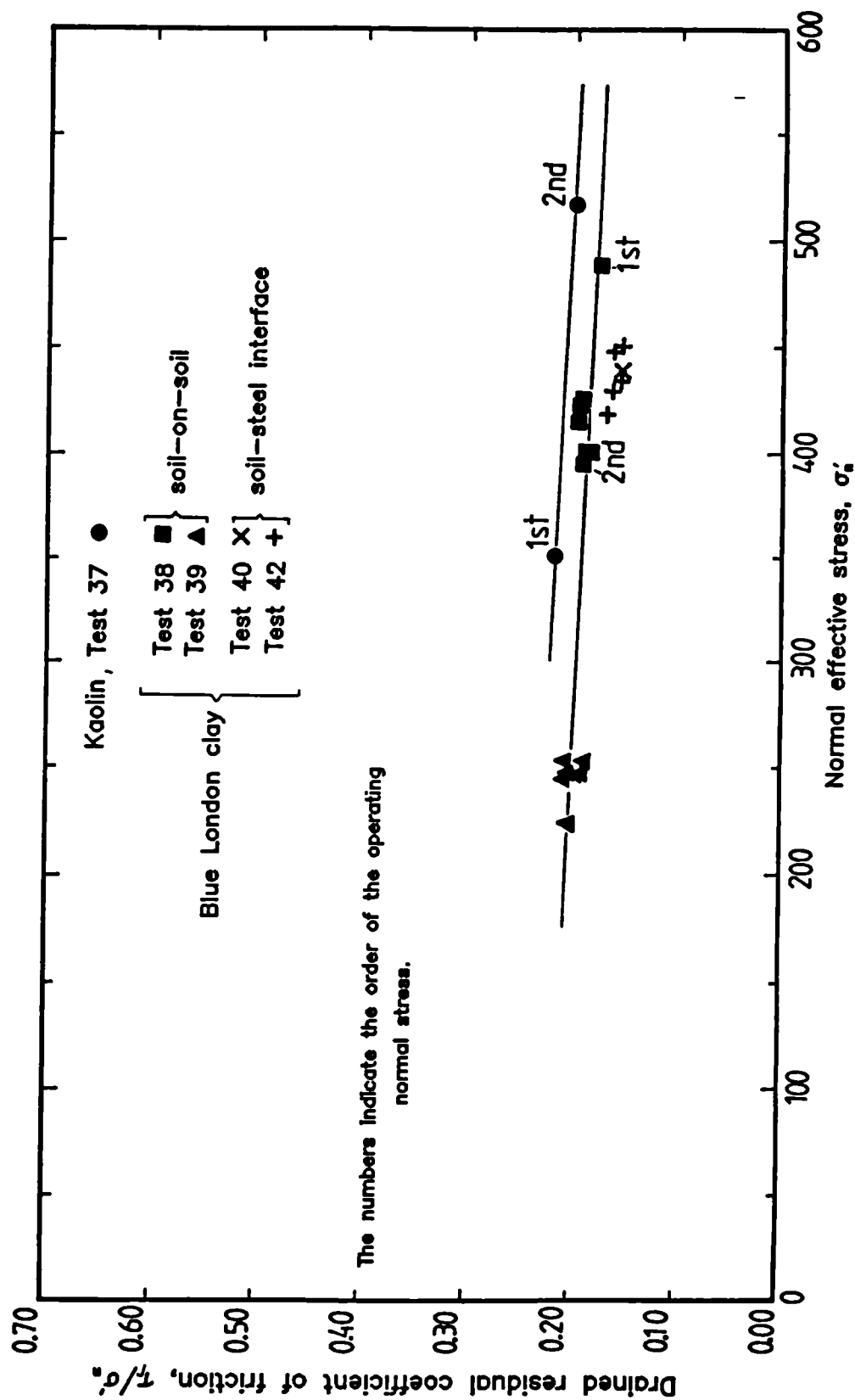


Fig. 4.41

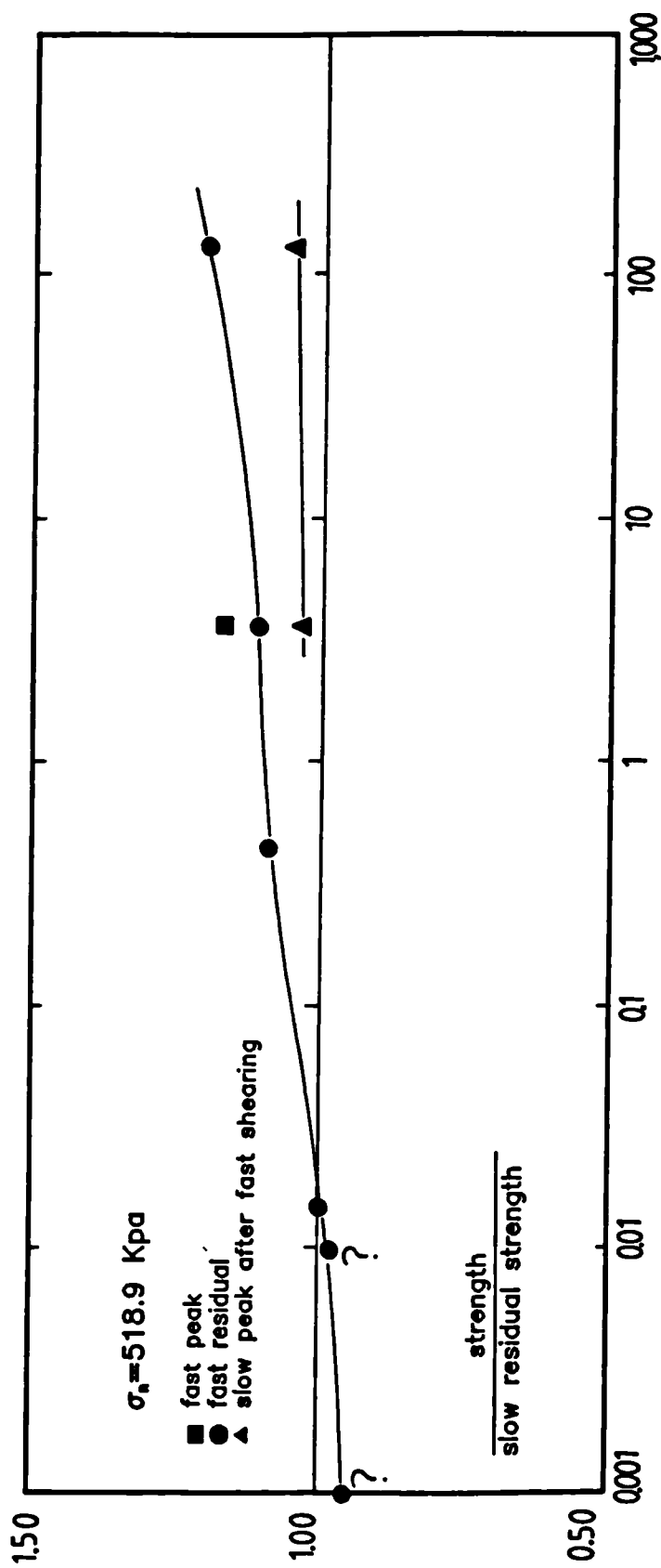


Fig. 4.42 Results from Test 37 on kaolin.

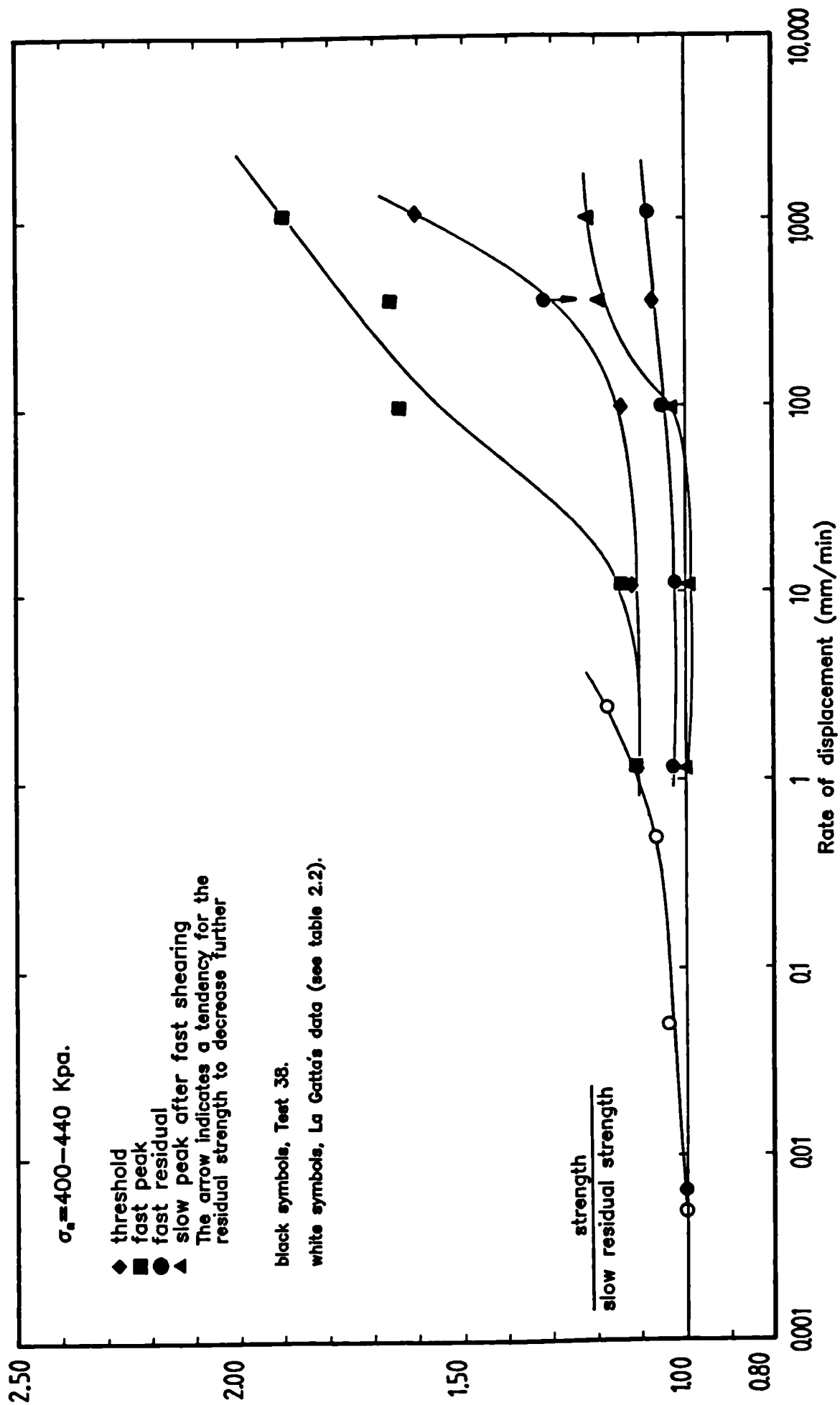


Fig. 4.43a Results from Test 38 on Blue London clay.

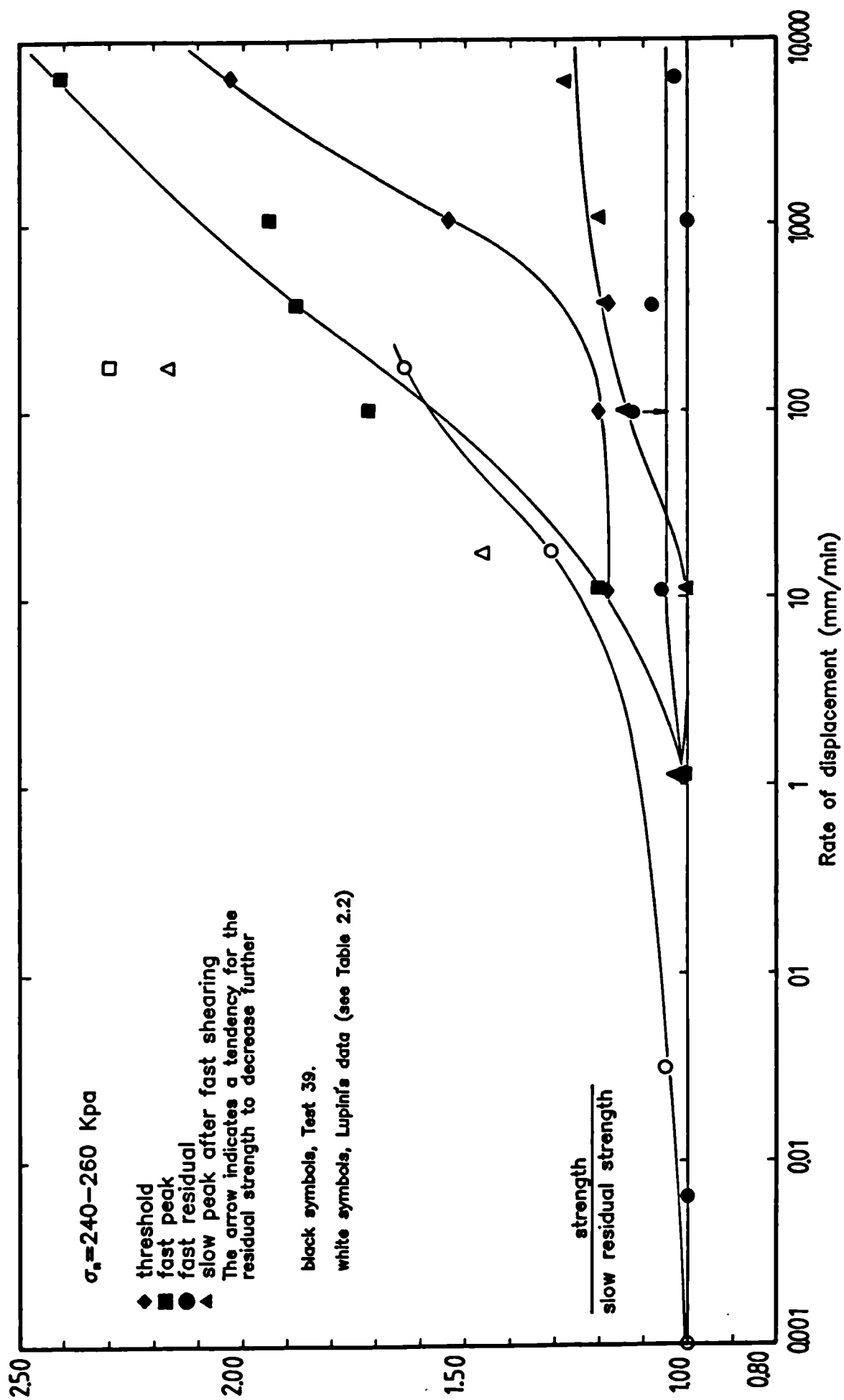


Fig. 4.43b Results from Test 39 on Blue London clay.

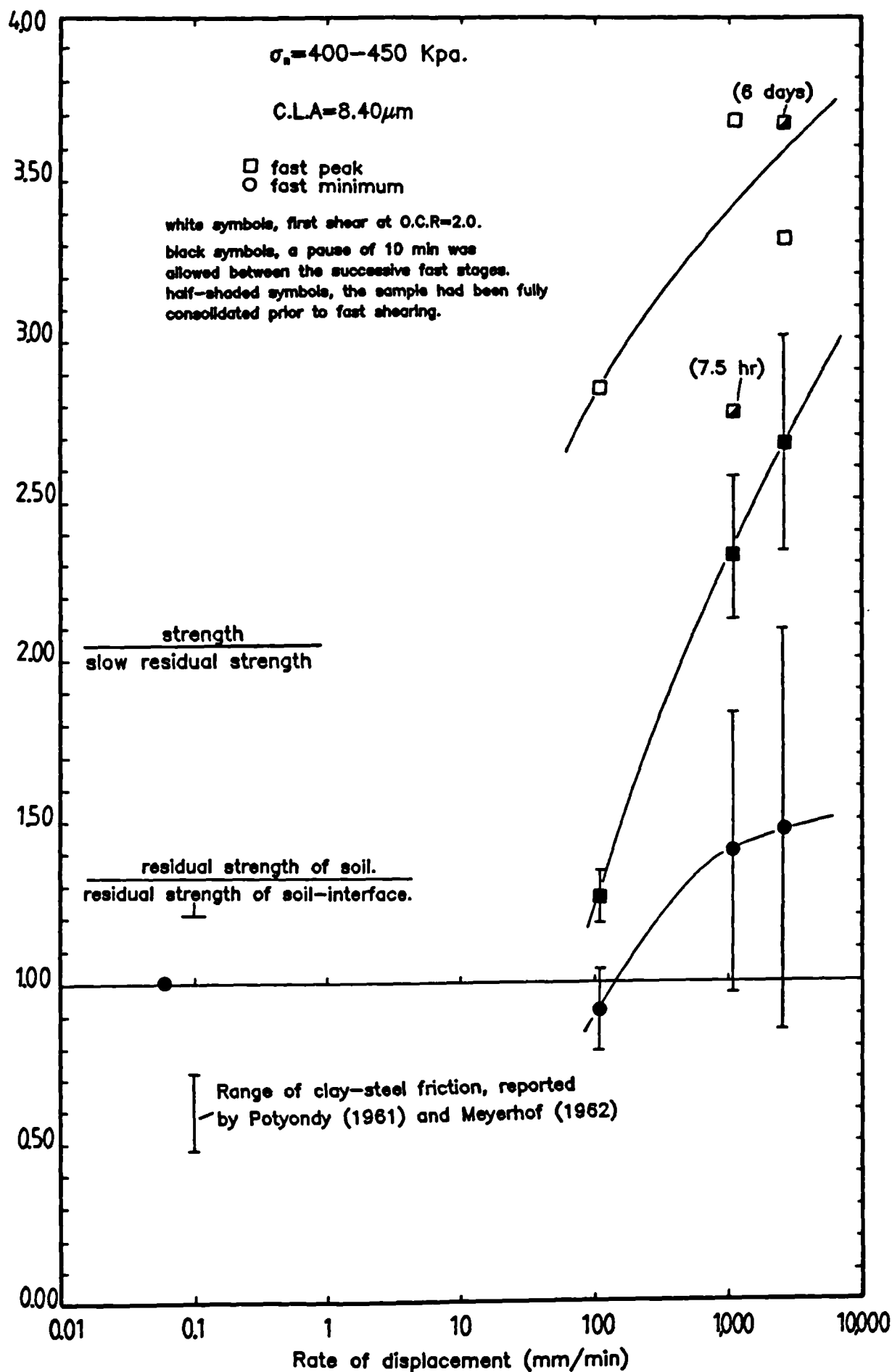


Fig. 4.43c Results from Tests 40, 41 and 42 on Blue London clay.
(soil-steel interface).

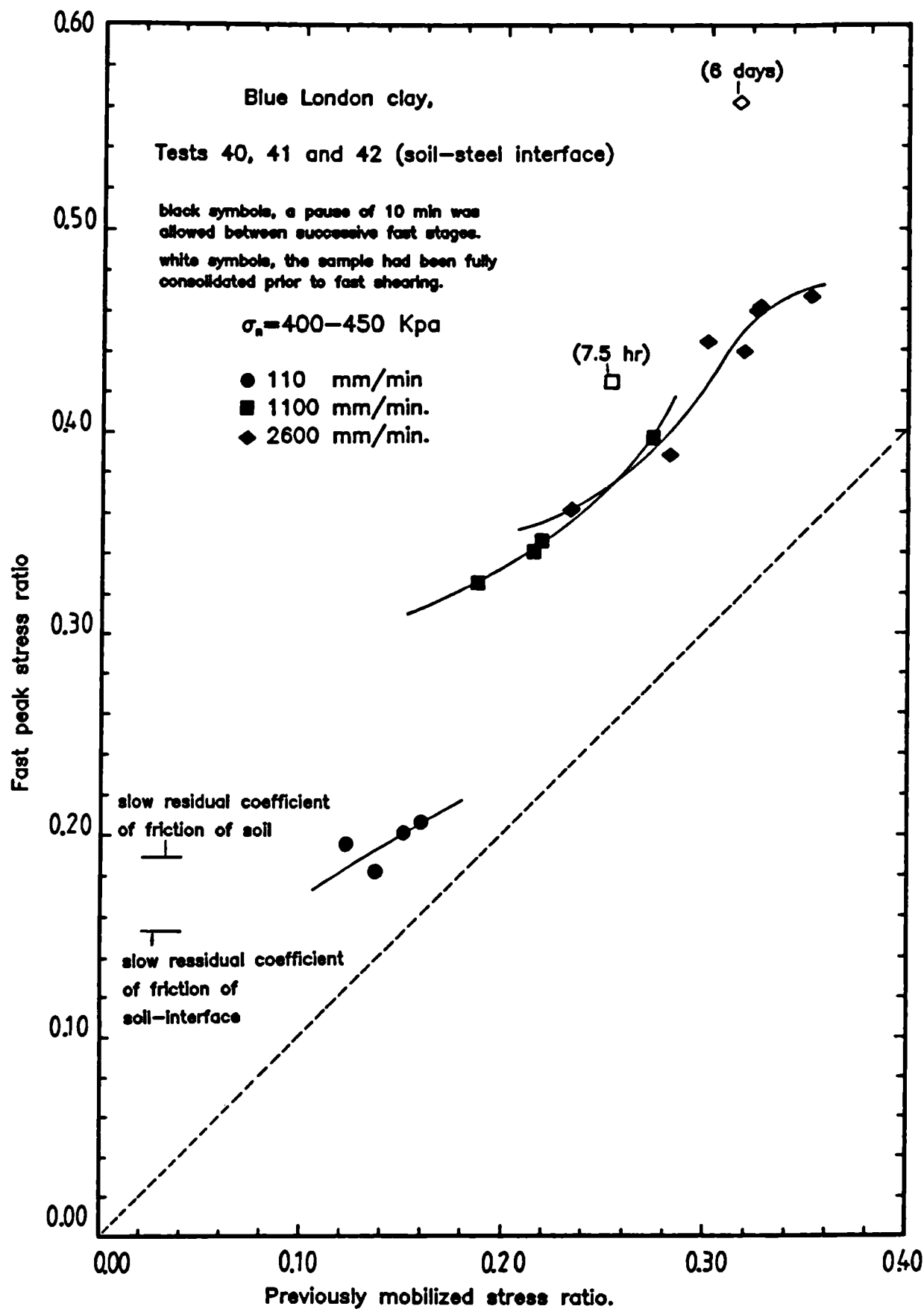


Fig. 4.43d

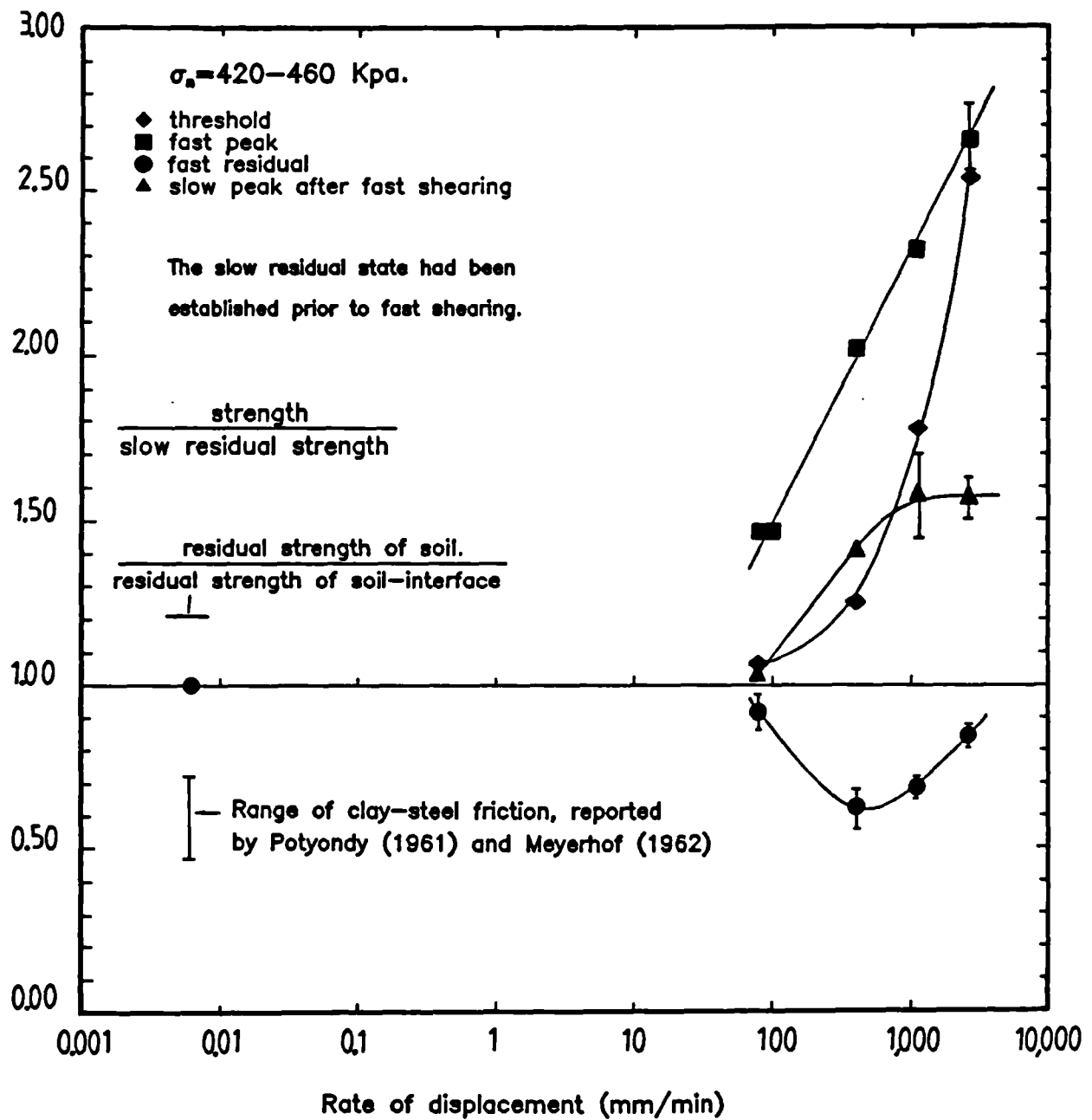


Fig. 4.43e Results from Test 42 on Blue London clay (soil-steel interface)

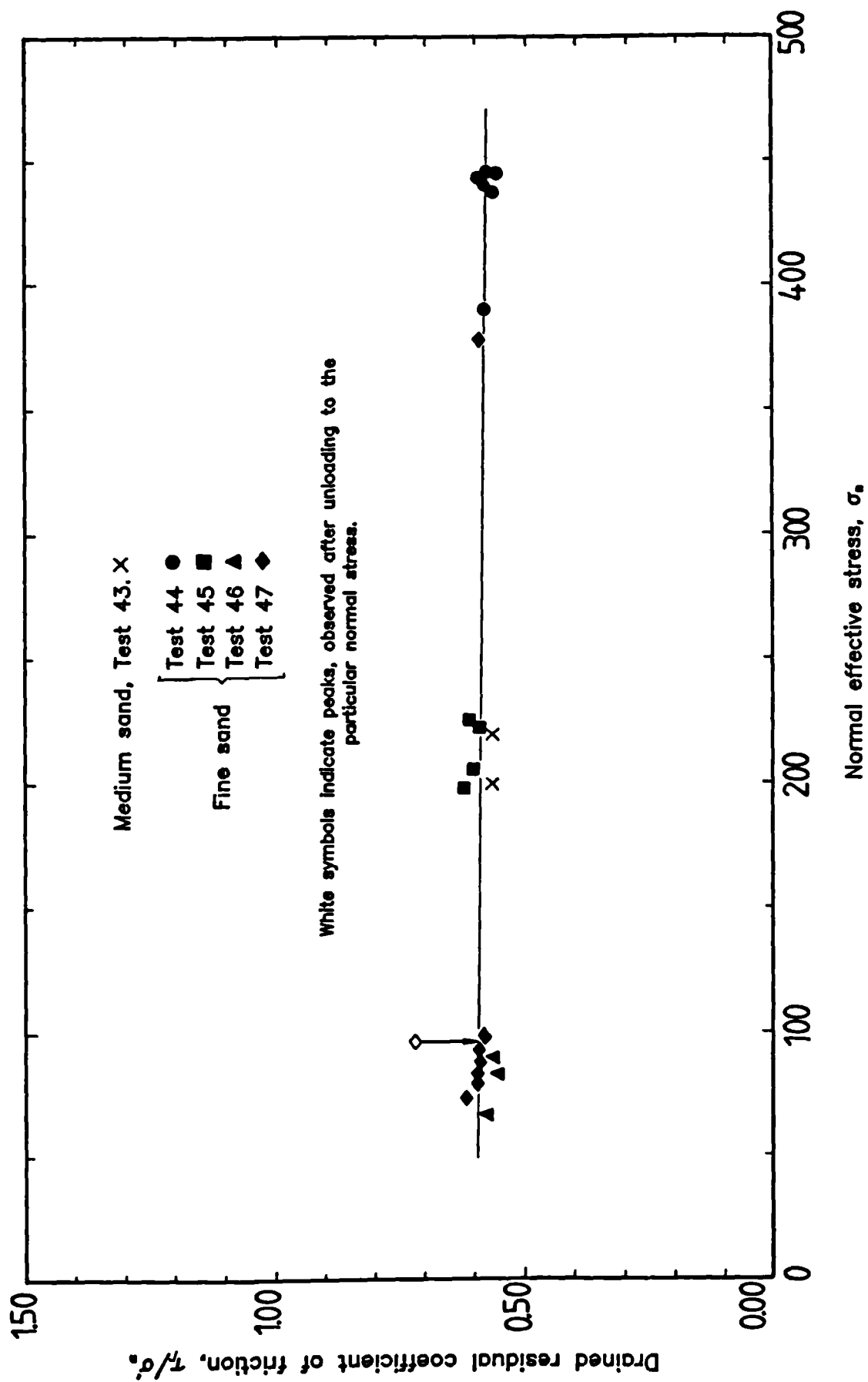


Fig. 4.44

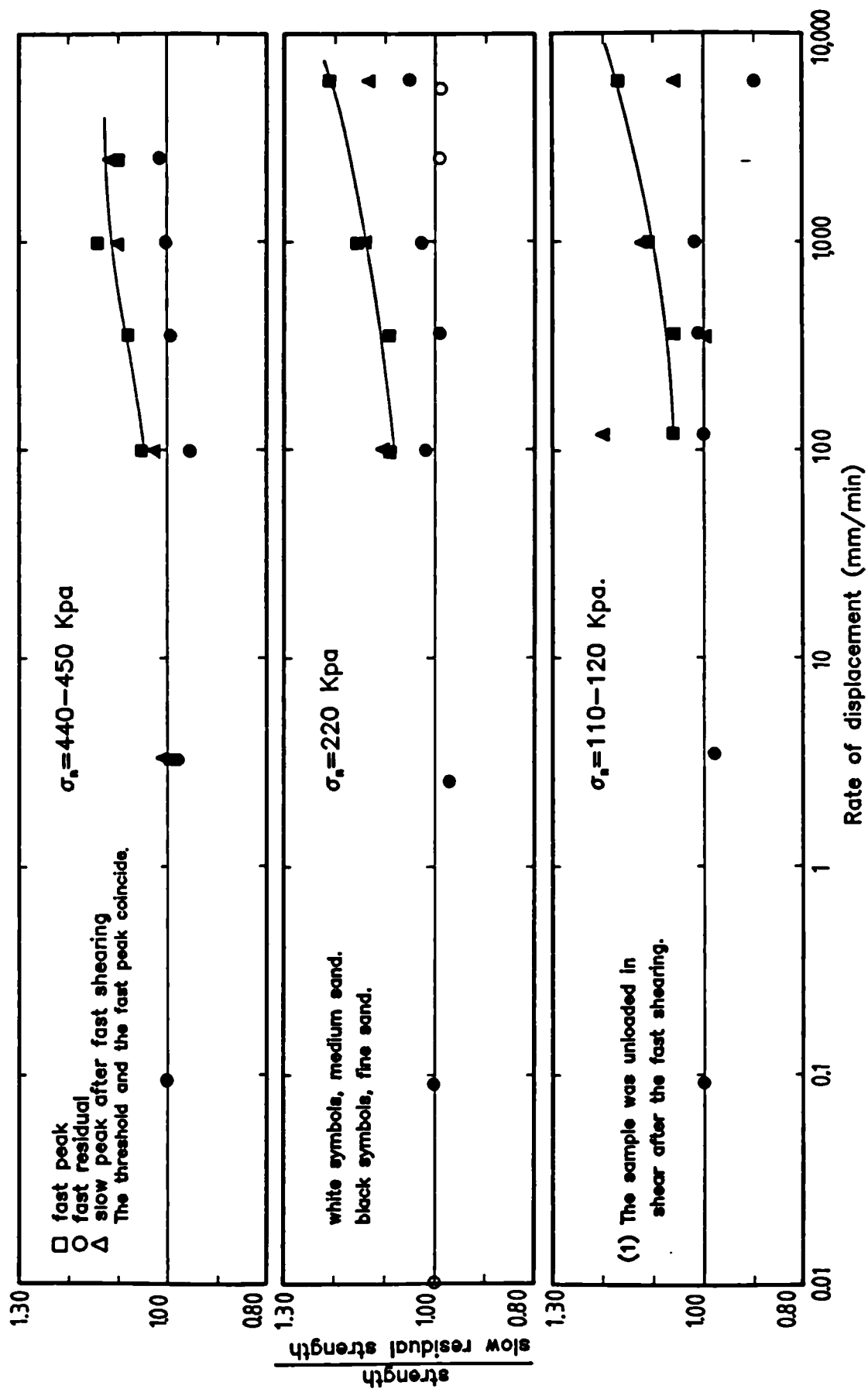


Fig. 4.45 Results from Tests 43 to 47 on medium and fine sand.

CHAPTER 5

DISCUSSION OF THE TEST RESULTS

5.1 INTRODUCTION

In this chapter the difficulties associated with testing in the ring shear apparatus and the typical strengths (slow residual, threshold, fast peak, fast residual, and slow peak after fast shearing, Fig. 2.4) are discussed according to the test results.

5.2 TEST DIFFICULTIES

Loss of soil through the gap between the two halves of the ring shear apparatus makes the measurement of volume changes inaccurate during shearing. Moreover, it influences the results due to friction, developed in the gap, and to non-uniform distribution of the normal stress. An estimation of the error involved due to friction, developed in the gap, has been given in Appendix 1.

The loss of soil was found to increase with both the normal stress and the size of the gap. For the typical displacement of 200 mm during fast shearing, the largest loss of soil occurred with rates in the range of 1 mm/min to 100 mm/min, while it reduced at higher rates of displacement.

Changes in sample thickness during fast shearing are shown in Fig.5.1 and A2.18. These changes reflect both the change in void ratio and the loss of soil. When a steady residual state is reached, shearing should be at a constant volume. Instead, there is a steady apparent compression due to loss of soil. If the loss of soil per unit of displacement is constant, at a given rate of displacement, then the void ratio increases during fast shearing. However, this cannot be fully proved.

Whereas the loss of soil makes the measurement of the volume changes inaccurate, it does not seem to have a significant effect on the behaviour of soils during fast shearing. This was demonstrated as follows:

During stages 17 and 6 of Tests 3 and 17 respectively, the gap was completely closed. Hence the loss of soil was slight. The soil behaviour during fast shearing did not change. As was noted by Lemos (1986) for Test 3 and as was observed for Test 17, the loss of strength during fast shearing persisted irrespectively of the gap size.

It has been observed by Lemos (1986) and the author that the loss of soil in the tests involving soil shearing against an interface is less than that observed in the soil-on-soil tests. This may be due to the different normal stress distributions in the two types of tests. In a soil-interface test, the presence of the rigid interface boundary may result in a more uniform distribution of the normal stress than that in a soil-on-soil test. In the latter type of test, the normal stress, at the location where the soil squeezing occurs, may be lower than that in a soil-interface test. The observations, made on the shear zones of the tested soils, support the above argument. In the soil-on-soil tests most of the test specimens had a shear zone which had variable thickness. The thickness reduced from the mid-radius of the sample towards its sides, from the maximum to the minimum normal stress. In the soil-interface tests a shear zone with a relatively constant thickness was observed.

Despite the different losses of soil, observed in the two types of tests, the results from the soil-glass interface tests, performed on the clayey siltstones A (Fig.4.8a), B(Fig.4.10a) D (Fig.4.12a,4.12b and 4.12d) and claystone D (Fig.4.14a and 4.14b) corroborated those obtained from the soil-on-soil tests. This indicates that loss of soil through the gap does not affect significantly the shearing behaviour of soils.

5.3 DISCUSSION OF THE TYPICAL STRENGTHS

5.3.1 The slow residual strength (drained)

An extensive study of the drained residual strength of cohesive soils, as measured in the ring shear apparatus, has been reported by Lupini (1981) and Lupini et al (1981). In the following, the drained residual strength of the tested soils and the factors that influence it are briefly discussed.

While the drained residual coefficient of friction of soils is independent of the placement conditions, loading sequence and previous history, it depends on other factors, such as, normal effective stress, pore water chemistry, mineralogy, rate of shearing, and particle grading and shape.

It has been long established that the drained residual coefficient of friction of soils is normal effective stress dependent. This dependence is small for soils with a turbulent shear mode, becoming greater for soils with a sliding shear mode.

A small cohesion intercept was observed for the soils with a turbulent shear mode which was positive in the case of sandstone, clayey silt, sandy clay and fine sand, and negative in the case of Lower Cromer Till and the residual soils from Japan (Table 5.1).

Lupini (1981) attributed the slight negative curvature of the Mohr-Coulomb envelope for the soils with turbulent shear mode to the separation of the massive minerals at the contacts by clay minerals. As the normal effective stress is increased, it is possible that a reduction in the granular void ratio could bring an increase of the interlocking between the massive grain domains.

All the soils with a sliding shear mode showed a positive cohesion intercept, except the claystone A and the soils the from Buoninvetre and Senerchia slides (Table 5.1). This dependence of the drained residual coefficient of friction on the normal effective stress is consistent with the results reported in the literature by various investigators. The claystone A showed a slight negative intercept which can be considered as being within the accuracy of the experimental results. It is difficult, however, to explain the negative cohesion intercept, observed for the soils from the Senerchia and Buoninvetre slides.

Various hypotheses have been advanced to explain the dependence of the drained residual coefficient of friction of soils with a sliding shear mode on the normal effective stress, especially at normal stresses below 200 kPa.

Herrmann and Wolfkill (1966) attributed it to the change of the

frictional characteristics of the shear surfaces at different normal stresses.

Kenney (1967) attributed it to the existence of a net attraction between the particles.

Chattopadhyay (1972) used elastic junction theory. The interparticle friction angle varies according to the expression:

$$\tan\phi_{\mu} = \tau_i K (\sigma'_n)^{-1/3} \quad (5.1)$$

where τ_i is the shear strength of contact, K is a function of geometry, and σ'_n is the normal effective stress across the plane. The drained residual coefficient of friction is expected to vary accordingly.

Chandler (1977) showed that a small increase in the shear displacement is required to establish the new residual strength on a surface presheared to the residual state, if the normal effective stress acting on it has been changed. The stress-displacement curve becomes brittle when the effective stress has been increased. This small displacement suggests that the structure of the shear surface requires modification for the new residual strength to be established, and that the effect of normal effective stress on the drained residual friction angle is due to changes in structure and interference between particles, rather than to changes in intrinsic friction between the particles.

This argument is supported by Cotecchia and Federico (1980) who, as reported by Lupini (1981), presented electron micrographs of shear surfaces of a clay, formed in a reversal shear box. These showed that the clay became more ordered when sheared at higher normal stresses.

All the soils with a transitional shear mode showed a positive cohesion intercept (Table 5.1).

Lupini (1981) stated that in the transitional shear mode the effect of an increasing normal effective stress will be to reduce the drained residual friction angle in sliding and at the same time to cause a denser packing of the rotund particles and more interference between them. The total effect of an increasing normal effective stress on the residual friction angle may thus be a product of two opposing

mechanisms.

Lemos (1986) studied the effect of reversing the shear direction on the drained residual coefficient of friction in both soil-on-soil and soil-interface tests. The peak coefficient of friction which was observed when the shear direction was reversed increased with increasing soil plasticity. The drained residual coefficient of friction in the reverse direction was slightly lower than the previously measured in the forward direction. This decrease was higher for the more plastic soils.

Kenney (1967, 1977) concluded that the mineralogy was the most important factor affecting the drained residual coefficient of friction of soils. The importance of mineralogy becomes greater for soils with transitional and sliding shear modes, in which the drained residual coefficient of friction depends on the interparticle friction angle, ϕ_p , of the clay minerals.

Kenney (1967) found that the drained residual coefficient of friction of natural soils and of the clay minerals hydrous mica and montmorillonite are influenced by system chemistry. Increases of the drained residual coefficient of friction are caused by increased ion concentration in the pore fluid and by cations of larger valence and greater polarizability.

Ramiah et al (1970) found that a change in pore water chemistry from flocculating to dispersive conditions changed the drained residual friction angle of a silty clay (LL=45, PI=17, CF=18%) from 33° to 28° . However, they do not give post-shear water contents and, as Lupini (1981) commented, the difference in the drained residual friction angle might have been due to the difference in granular void ratio of the rotund particles, rather than to a change of the shearing resistance of the clay between them.

The rate of shearing has a significant effect on the drained residual coefficient of friction of soils. This effect is discussed in section 5.3.5.

The importance of the relative proportions of clay minerals to massive minerals and of the particle shape has been realized by Skempton

(1964), Borowicka (1975), Petley (1966), Vaughan and Walbancke (1975), Kenney (1977) and Chattopadyay (1972). Kenney (1977) and Lupini (1981) quantified the proportions of clay to massive minerals present in soil. Lupini (1981), as mentioned in section 2.3, quantified his results in terms of the void ratio of the granular phase (Fig.2.1).

Table 5.1 summarizes the granular void ratios of the tested soils. The drained residual coefficient of friction of the soils is plotted against the granular void ratio in Fig.5.2, along with the results for the mixtures tested by Lupini (1981). Where the final water contents of the soils were unknown (Magnus Till and residual soil from Fiji, used in Test 26), the following expression, suggested by Lupini et al (1981), was used:

$$w_f = w_l - I_p \cdot \frac{\log(p'_m) - \log(p'_{ml})}{\log(p'_{mp}) - \log(p'_{ml})} \quad (5.2)$$

where p'_{ml} is the mean effective stress at the liquid limit (w_l), taken to be 5.5 kPa,

p'_{mp} is the mean effective stress at the plastic limit (w_p), taken to be 550 kPa, and

p'_m is the mean effective stress in the test, taken to be the vertical applied stress.

The soils from the Kalabagh dam site, and the Buoninvetre and Senerchia slides showed lower values of granular void ratios, at a given drained residual coefficient of friction, than those of the Happisburgh-London clay mixtures which have been suggested by Lupini (1981), as being typical of natural soils. The lower values of granular void ratio, observed for the soils from Kalabagh dam site, are thought to be associated with the shape of the massive grains, the use of the clay fraction as an indicator of the proportion of the platy minerals and the well-graded granular fraction of the soils.

As observed for clayey siltstone D and claystone D, most of the massive particles, retained on the 63 μ m sieve, when viewed under a microscope showed platy and elongated shapes. Shearing to the residual state should involve therefore some orientation of these platy massive particles. A shear surface could form with smaller massive particle separation and smaller values of granular void ratio than if the massive particles were rotund.

It has been recognized by various investigators (Heley and MacIver, 1971 and Townsend and Gilbert, 1974) that the classification indexes of clay shales are greatly influenced by the test preparation procedures. The preparation procedures, suggested by Heley and MacIver, were used for most of the soils from the Kalabagh dam site for the determination of clay fraction. Blenderization was found to cause only a small increase of the clay fraction of the soils. Comparison of the clay fraction and the clay mineral content, as determined from the mineralogical analysis, shows a good correlation between clay fraction and the clay mineral content for the claystones A and B, but a substantially smaller clay fraction than the clay minerals content for the clayey siltstone A. This indicates that the clay particles of the clayey siltstone A may be as large as 5 to 6 μm .

soil	clay minerals	clay fraction	
	content	% < 2 μm	% < 5 μm
claystone A	50	52	40
claystone B	38	40	57
clayey siltstone A	40	20	32

The implicit assumption is usually made that the clay fraction (% < 2 μm) is identical to the clay minerals content. This assumption holds in most cases, as the clay minerals are usually less than 2 μm in size. It does not hold in the case of platy clay minerals with diameters larger than 2 μm . Considering that the main clay minerals for the soils from the Kalabagh dam site were illite and kaolinite, it can be concluded that the true clay fraction of the clayey siltstone A is larger than that determined from the sedimentation analysis.

The correlation between the drained residual coefficient of friction and the granular void ratio for the soils from the Kalabagh dam site, using the percentage of particles with a diameter less than 5 μm , as the proportion of platy minerals, is shown in Fig. 5.2. The granular void ratios of the clayey siltstones are still lower than those of the Happisburgh-London clay mixtures, whereas those of the claystones are higher. Thus the results remain inconsistent even when a revised definition of clay content is used.

All soils from the Kalabagh dam site had a graded granular fraction.

Lupini (1981) observed that a well-graded granular fraction is likely to give lower values of granular void ratio for the transitions in behaviour than a uniform one. If the platy particles are much smaller than the rotund ones, then a shear surface could form with smaller rotund particle separations and smaller values of granular void ratio than if the platy and rotund particles were of similar size.

The residual soils showed considerably higher granular void ratios than those of the Habbisburgh-London clay mixtures. This may be due partly to the fact that the true values of the specific gravities of the clay and the massive grains were not considered in the determination of the granular void ratio. Instead the values of 2.75 and 2.67 for the clay and massive grains respectively were used. The influence of the specific gravities of clay and massive grains increases in soils with high failure water contents, as in the residual soils

The granular void ratio of the residual soil from Mauritius was based on the clay fraction, determined from the whole test specimen prior to testing. Because of the generation of platy particles by degradation during shearing, this value of the granular void ratio is ambiguous and does not represent the true granular void ratio at the shear zone of the sample.

Several correlations between the drained residual coefficient of friction and the index properties have been reported in the literature. However, the view that the drained residual coefficient of friction is correlated with the index properties of the soils is not shared by all the investigators (i.e Kenney, 1967). Lupini (1981) pointed out that, in a cohesive soil deposit in which pore water chemistry is constant, the drained residual coefficient of friction in the sliding mode is likely to correlate with clay fraction and activity. If the clay mineralogy is constant, the drained residual coefficient of friction should correlate with clay fraction, and also with plasticity index, since this will be proportional to clay fraction.

The correlation of drained residual coefficient of friction with clay fraction and plasticity index is plotted in Fig.5.3 and 5.4. For the soils from the Kalabagh dam site, characterized by both constant pore water chemistry and mineralogy, a good correlation between the drained residual coefficient of friction and the index properties was

established.

5.3.2 The threshold strength

The threshold strength has been defined as the strength mobilized on a shear zone or surface during fast shearing prior to slippage.

The existence of threshold strength was typically identified for rates of displacement equal to or larger than 10mm/min. The threshold strength coincided with the fast peak strength, for soils with a turbulent shear mode or a transitional shear mode tending towards the turbulent mode.

Ratios of threshold strength to slow residual strength, plotted against rate of displacement, have been presented in section 4.4. All tested soils showed increasing ratios of threshold strength to slow residual strength at increasing rates of displacement.

Fast shearing at a constant average velocity can involve slightly different loading times to threshold and fast peak strengths. Loading times to reach the threshold strength are summarized in Table 5.2. Ratios of threshold strength to slow residual strength versus loading time are plotted in Fig.5.5 to 5.7. All tested soils showed increasing ratios of threshold strength to slow residual strength at decreasing loading times.

Typical envelopes for the threshold strength are shown in Fig.5.8 to 5.10.

The variation of threshold stress ratio with index properties for the soils from the Kalabagh dam site is illustrated in Fig.5.11 and 5.12.

According to the limited data available (Procedure A), the increase of the threshold strength above the starting slow strength (or previously mobilized strength), at a given rate of displacement and normal stress, is not significantly dependent on the starting slow strength, except for the mechanically degraded Tuff at 100 kPa and 100 mm/min (Fig.5.13). It mainly occurs as a constant increase to the starting slow strength.

All tested soils, except the soil from Fiji, showed higher ratios of threshold strength to slow residual strength at lower normal stresses, at a given rate of displacement or loading time.

In all the tests the plot of stress ratio versus time was assumed equivalent to that of stress ratio versus displacement. The rate of displacement was assumed to be constant. This resulted, sometimes, in an overestimation of the displacement at which the threshold strength operated, because of the time required for the system to acquire constant velocity. The accurate measurement of displacement during fast shearing in the tests on blue London clay (Fig.A2.45d) and fine sand (Fig.A2.51b) indicated a hardly noticeable movement on the shear zones of these soils, when the threshold strength operated. Thus, the results from these soils suggest that the assumption for the mobilization of threshold strength on the shear zones of the soils, prior to slippage, is a reasonable one.

5.3.3 The fast peak strength

The fast peak strength generally exceeds the threshold strength in soils showing transitional and sliding shear modes. As noted previously, it coincides with the threshold strength in soils with a turbulent shear mode or a transitional shear mode tending towards the turbulent mode.

The increase of the fast peak strength above the threshold strength with fast displacement is due to the disordering of the structure of the shear zone of the soils. Plate 3 illustrates the microstructure of the shear zone of the clayey siltstone D when the fast peak strength operates. Observations of the microstructure indicate that the clay particles and the shear surfaces within the shear zone are less preferred orientated with respect to the direction of shear than when the slow residual strength operates.

Ratios of fast peak strength to slow residual strength, plotted against rate of displacement, have been presented in section 4.4. All tested soils showed increasing ratios of fast peak strength to slow residual strength at increasing rates of displacement.

Loading times to reach fast peak strength are summarized in Table 5.3

for soils showing a turbulent shear mode or transitional shear mode tending towards the turbulent mode, and in Table 5.2 for soils showing a sliding shear mode. Ratios of fast peak strength to slow residual strength, plotted against loading time, are shown in Fig. 5.5 to 5.7.

Typical envelopes for the fast peak strength are shown in Fig. 5.8 to 5.10.

The variation of fast peak stress ratio with index properties for the soils from the Kalabagh site is illustrated in Fig. 5.11 and 5.12.

All tested soils showed higher ratios of fast peak strength to slow residual strength at lower normal stresses, at a given rate of displacement.

5.3.4 The slow peak strength after fast shearing

It is convenient to discuss this strength prior to the fast residual strength, considered in the following section.

The slow peak strength after fast shearing has been defined as the peak strength, observed during slow shearing after a fast stage and a pause for consolidation.

Ratios of slow peak strength to slow residual strength, plotted against rate of displacement, have been presented in section 4.4.

Slight increases of strength above the slow residual value during slow shearing, after a fast stage were observed for some of the soils showing a turbulent shear mode (fine sand, sandstone, residual soils from Mauritius and Japan). Since in these soils, clay particles orientation at the slow residual state does not take place, these increases are probably due to the relaxation of the shear load acting on the sample rather than to the disordering of the soil structure in the shear zones during fast shearing.

For the soils showing a transitional or a sliding shear mode, a slow peak strength was observed when the preceding rate of displacement was fast enough, so that a disturbance of the originally ordered structure could be produced. The test results suggest that the critical rate of

displacement below which very little or no disturbance of the shear zone occurs increases with both soil plasticity and normal stress. For normal stresses between 200 and 500 kPa, it is practically of the order of 1 mm/min for soils of intermediate plasticity, and 10 to 100 mm/min for soils of higher plasticity.

For the soils showing a transitional or a transitional with some turbulence shear mode, the ratio of slow peak strength to slow residual strength did not correlate systematically with the normal stress, at a given preceding rate of displacement, or with the preceding rate of displacement, at a given normal stress.

Soils with a sliding shear mode or a transitional shear mode tending towards the sliding mode, showed increasing ratios of slow peak strength to slow residual strength at decreasing normal stresses, at a given preceding rate of displacement, and at increasing preceding rates of displacement at a given normal stress, except for the soil from Fiji in which the opposite trend was observed.

The results for the latter soils suggest that the disordering of the shear zone is less when the presence of the massive grains is reduced (Fig. 5.14). The kaolin and the blue London clay showed smaller increases of the slow peak strength above the slow residual strength than the claystones A,B,C and D, and the soils from the Buoninvetre and Senerchia slides which had a larger amount of massive grains. The maximum increase of strength was observed for the claystone B and the soils from the Buoninvetre slide and Fiji. In these soils the slow peak strength reached values as high as 80% to 90% of the normally consolidated peak strength. This gain in strength indicates that the structure of the shear zone corresponding to the slow residual state was almost completely destroyed during fast shearing of these soils.

There are indications that, for a given type of soil, the ratio of slow peak strength to slow residual strength reaches a maximum (Fig.4.14, 4.35 and 4.43).

Typical envelopes for the slow peak strength are shown in Fig.5.9 and 5.10.

The variation of slow peak stress ratio with index properties for the

soils from the Kalabagh dam site is illustrated in Fig.5.11 and 5.12.

5.3.5 The fast residual strength

Ratios of fast residual strength to slow residual strength, plotted against rate of displacement, have been presented in section 4.4.

The following types of variation of the residual strength with an increasing rate of displacement were recognized (Fig.5.15).

- (i) Type I showing a constant residual strength irrespective of the rate of displacement.
- (ii) Type II showing either a negligible to moderate increase (IIb, IIc) or a slight decrease (IIa) of residual strength at increasing rates of displacement, followed by a significant drop of strength to less than the slow residual value at higher rates of displacement.
- (iii) Type III showing a decrease of residual strength at increasing rates of displacement, followed by an increase and then a significant drop of strength to less than the slow residual value at higher rates of displacement.
- (iv) Type IV showing a decrease of residual strength at increasing rates of displacement, followed by an increase above the slow residual strength at higher rates of displacement.
- (v) Type V showing a slight to moderate increase of residual strength at increasing rates of displacement.

Table 5.4 summarizes the soils showing the same type of behaviour at increasing rates of displacement and Fig.5.16 shows the gradings of the soils from each type.

Outlined below is a summary of the behaviour of soils showing turbulent, transitional and sliding shear modes.

Soils with a turbulent shear mode or a transitional shear mode tending towards the turbulent mode exhibited behaviour of types I, II, IV and V

. The normal stress had a strong influence on the behaviour of some of the soils. Some soils altered the type of their behaviour when the normal stress on the sample was changed (i.e siltstone, Cowden Till, residual soil from Japan, used in Test 30, clayey silt and sandy clay).

Soils with a transitional shear mode showed behaviour of types IIc and III. The significant drop of strength to less than the slow residual value was independent of the normal stress on the sample.

Soils with a transitional shear mode tending towards the sliding shear mode showed behaviour of types III and IVa, except the claystones C and D at the normal stress of 500 kPa. The claystones C and D showed a drop of strength to less than the slow residual value within the range of tested rates. The significant drop of strength to less than the slow residual value, observed with type III of behaviour, increased with increasing normal stress.

Soils with a sliding shear mode exhibited behaviour of types IIc, III, IV and V.

The increase of the residual strength above the slow residual value, observed with types II, III, IV and V of behaviour, may be due to different causes for soils with different shearing modes. For soils showing a turbulent shear mode (residual soil from Mauritius, clayey silt and sandy clay), it may be due to viscous effects. For soils with a transitional shear mode or a transitional shear mode tending either towards the turbulent or the sliding mode, it is due to structural changes taking place in the shear zones. That structural changes take place is indicated by the slow peak strength after fast shearing. Lupini (1981) attributed the increase of strength above the slow residual value, observed for the soil from Fiji, to the change of the behaviour of soil during fast shearing from the pure sliding to the turbulent or to the sliding with some turbulence mode, as indicated by the slow peak strength after fast shearing.

The slight decrease of strength to less than the slow residual value, observed initially with types IIa, III and IV of behaviour, has been also observed and discussed by Hight (1983). He postulated that the soil fabric consisted of a granular framework, the matrix clay and the clay bridges. When boundary stresses are applied to the fabric, the

stress levels in the two clay phases differ significantly resulting in pore water pressure inhomogeneities. During fast shearing there is a continuous unloading of clay bridges, accompanied by formation and loading of new ones with no overall volume change. When a clay bridge is unloaded, there is tendency for a local volume increase and development of negative excess pore water pressure. When a bridge is formed and loaded, there is tendency for volume decrease and development of local positive excess pore water pressure. These locally existing pore water pressures may be responsible for the observed reduction in strength.

The possible causes of the significant drop of strength to values less than the slow residual at fast rates of displacement, observed in types II and III of behaviour, are discussed in chapter 6. There are indications that the loss of strength reaches a limiting value (Fig.5.17).

Types IVa and IVb may be tentative due to lack of data for the behaviour of soils. The soil from the Senerchia slide and the claystones B and C might have shown behaviour of type III, had they been tested at faster rates of displacement, similar to the soil from the Buoninvetre slide and the claystone D. The available data for the claystones C and D at the normal stress of 500 kPa, the Magnus Till and ^{the} residual soil from Fiji were not sufficient for the identification of their behaviour.

The slow standard rates of displacement, used in the tests presented here, varied from 0.003 mm/min to 0.10 mm/min. A change of the slow standard rate of displacement may result in a change of the type of the soil behaviour. The weathered Cowden Till shows type V of behaviour at normal stresses between 100 and 200 kPa and type IVa at a normal stress of 50 kPa, at the slow standard rate of 0.015mm/min, and type IVa at all the normal stresses at a lower rate.

Having identified the types of variation of the residual strength with an increasing rate of displacement (Table 5.4, Fig. 5.15), it seems that the identification of the type of behaviour, for a given soil, through its slow shear mode or its index properties is uncertain. Soils that had identical slow shear modes or index properties behaved differently (Fig. 5.18 and 5. 19).

The grading of the granular phase is certainly a factor controlling the type of behaviour of soils. The two artificial soils, the clayey silt and the sandy clay, had the same mineralogy and shear mode and similar clay fractions, but different gradings of the granular phase (Fig. 5.16). Different types of behaviour were observed for the soils. The clayey silt was a mixture of a silt and spheswhite kaolin in proportions such that its grading was similar to those of clayey siltstones A,B,C and D. It showed the same type of behaviour (IIc) with the clayey siltstones A,B,C and D (Test 49) at the lower normal stress at which it was tested, despite the soils having different shear modes.

The use of the granular void ratio to distinguish the type of behaviour is thus uncertain, because it fails to account for the grading of the granular phase, the shape of the granular particles, and the relative sizes of the rotund and platy particles.

Athough, it is not possible at this stage to assess exactly or quantify the relationship between the grading of the granular phase and the type of behaviour, it can be noticed, however, that (Fig.5.18 and 5.19):

- (i) The soils that showed behaviour of type I had a turbulent shear mode. These were sands and sandy silts with a plasticity index less than 9%, a sand content larger than 20%, and a clay content less than 10%.
- (ii) The soils that showed behaviour of type II, except the residual soil from Fiji, had a turbulent or a transitional shear mode, and a plasticity index between 9 and 21%. These were silts, with a silt content higher than 60% and each of the sand and silt contents less than 20%, and a gap-graded clayey sand (Lower Cromer Till). Some of the soils with a plasticity index between 9 and 13% showed behaviour of type I, at high normal stresses ranging from 450 to 500 Kpa.
- (iii) The soils that showed behaviour of type III were silty clays with a transitional or a sliding shear mode and a plasticity index between 24 and 26%.
- (iv) The soils that showed behaviour of type IV were low plasticity

clays and silty clays. The low plasticity clays showed a turbulent shear mode or a transitional shear mode tending towards the turbulent mode, had a plasticity index between 10 and 21% and were, characterized by either a very well graded or a uniform granular phase. The silty clays showed a transitional shear mode tending towards the sliding mode or a sliding shear mode, and had a plasticity index between 26 and 36%.

- (v) The soils that showed behaviour of type V were clays which had a sliding shear mode, a plasticity index between 36 and 51%, and a clay content higher than 48%.

The residual soil from Fiji contained poorly crystallized kaolinite. It showed pure sliding behaviour in slow residual shear, and behaviour of type II at increasing rates of displacement. When the soil was subjected to slow shearing, after a fast stage, in which a significant drop of strength below the slow residual was observed, the slow peak strength was similar to the normally consolidated strength. This indicates that in this soil slow shearing involves pure sliding, while fast shearing involves purely turbulent behaviour. The residual soil from Mauritius showed behaviour of type I initially. However, a transition in the behaviour from type I to type II was observed due to the degradation of the particles within the shear zone during shearing.

The index properties can be useful in the study of the effect of rate of shearing on the residual strength of soil deposits, provided they reflect changes in the grading. The variation of fast residual stress ratio with index properties for the soils from the Kalabagh dam site is shown in Fig.5.11 and 5.12.

Fig.5.8 to 5.10 illustrate typical envelopes for the fast residual strength.

The effect of a change of the normal effective stress during fast shearing on the behaviour of soils was investigated in Tests 21 and 36 on clayey siltstone D and sandy clay respectively. In Test 36, after having established the slow residual state, the normal stress was increased (stages 6 and 23) or decreased (stage 25) and fast shearing was then commenced immediately. In Test 21, the normal stress on the

sample was increased after the fast residual state at 100 mm/min had been established. Fast shearing was restarted at the same rate of displacement immediately afterwards. A short pause was allowed necessarily after the change of the normal stress in both tests, as it is practically impossible to change the normal load and start fast shearing simultaneously.

The results of the tests are shown in Fig.5.21 and 5.22. The measured strengths during fast shearing compared well with those predicted considering the normal effective stress and the corresponding strength envelope (threshold strength envelope for Test 36 and fast residual strength envelope for Test 21). The normal effective stress during fast shearing was estimated from the normal effective stress prior to fast shearing, allowing for the consolidation or swelling due to the change of normal stress which occurred during the short pause and the fast stages.

The results suggest that, if during fast shearing of pre-existing shear surfaces changes of the normal load occur, these changes should be accounted for when fast shearing is accompanied by drainage, and can be neglected when no drainage takes place during fast shearing.

5.4 A NOTE ON THE EFFECT OF RATE OF SHEARING ON THE SOIL-INTERFACE RESISTANCE

Many practical problems in soil mechanics require a knowledge of the shearing resistance of soils against various structural materials. There has been a number of investigations, examining the factors affecting the soil-interface shearing behaviour, such as, type of soil, interface material, roughness of interface, soil placement conditions, normal stress and rate of shearing.

The soil-interface tests of the present research were carried-out on the clayey siltstone D, the claystone D, and the blue London clay. Lemos (1986) had performed soil-interface tests on the clayey siltstones A and B.

The tests on the clayey siltstones A, B and D and the claystone D, which had been performed in order to investigate the role of pore water pressure during fast shearing, involved soil shearing against glass and

steel interfaces. In the tests on the blue London clay, carried-out to investigate the effects of rate of shearing on the soil-interface resistance, steel interfaces were used.

Results from the tests are shown in Fig. 4.7 and 4.8a for the clayey siltstone A, Fig. 4.9 and 4.10a for the clayey siltstone B, Fig. 4.11 and 4.12d for the clayey siltstone D, Fig. 4.13, 4.14a and 4.14b for the claystone D, and Fig. 4.43c to 4.43e for the blue London clay.

The experimental results indicated that, for the tested soils (medium to very high plasticity), the range of the interface roughnesses used (7.3 to 8.5 μm), and a normal stress between 200 and 500 kpa, the soil-interface drained residual coefficient of friction is within 85 and 107% of the drained residual coefficient of friction of soil for a glass interface, and 85% approximately for a steel interface (Table 5.5).

The values of the soil-steel drained residual coefficient of friction are higher than those, reported by Potyondy (1961), Meyerhof (1962) and Clark and Meyerhof (1972), but of the same order, as those reported by Littleton (1976) and Lemos (1986) for clays shearing on smooth steel interfaces (Table 5.5). It is possible that the interfaces, characterized as rough by the first authors, were smoother than those used in the present work.

For the claystone D, the drained residual coefficient of friction, observed in the soil-glass interface test was higher than that observed in the soil-on-soil test. This increase may be either considered as being within the accuracy of experimental results (Fig. 4.13) or due to a slight increase of the interlocking component of the residual strength, due to the interaction of massive particles with the asperities of the interface.

A hardly noticeable change in the interface roughness during shearing was observed in all the tests. The shear surface was formed always within the soil, away from the soil-interface boundary (Plate 6).

The behaviour of the clayey siltstones A, B and D and the claystone D when sheared against a rough glass interface at fast rates of displacement was qualitatively similar to that observed in the

soil-on-soil tests. The same typical strengths, as in the soil-on-soil tests, were identified when a fast rate of displacement was imposed to the sample after the soil-interface slow residual strength had been established. Some indications exist for the clayey siltstone D, that the drop of strength to less than the slow residual during fast shearing decreases with increasing normal stress. This decrease was more pronounced in the soil-steel test in which at a normal stress of 500 kPa approximately, the loss of strength was almost muted (Fig.4.12d).

The investigation of the influence of the rate of shearing on the resistance of the blue London clay, sheared against steel, indicated the existence of rate-effects which became significant for rates of shearing faster than 100 mm/min. In a series of successive fast stages, carried out at a constant rate of displacement, the London clay-steel resistance increased with increasing rate of displacement (Fig.4.43c). This behaviour is in agreement with that observed during installation of jacked piles in London clay at different penetration rates (Jardine, 1985 and Bond, 1989), and the results reported by Murff and Coyle (1973), Dayal and Allen (1975), Heerema (1979), Litkouhi and Poskitt (1980), and Poskitt and Leonards (1982).

During a series of successive fast stages, carried-out at a constant rate of displacement with a short pause between them (representing a degree of consolidation of 35-50%), the increase of peak strength, observed in each pulse, to above the previously mobilized fast strength was almost constant, similar to the behaviour observed in the soil-on-soil tests (Fig.4.43d).

The same typical strengths, as in the soil-on-soil tests, were identified when a fast rate of displacement was imposed to the sample, after the soil-steel slow residual strength had been established. The influence of the rate of shearing became significant for rates higher than 80 mm/min, as it was indicated by the slow peak strength after the fast shearing (Fig.4.43e). For the rates of displacement used, the soil-steel fast residual strength was within the range between the soil-steel slow residual strength measured and the corresponding values reported by Potyondy (1961), Meyerhof (1962) and Clark and Meyerhof (1972).

The experimental results on the effect of rate of shearing on soil-interface resistance have important implications for the behaviour of displacement piles. The strength of the shear surface (indicated by the slow peak strength after fast shearing in the ring shear tests), formed parallel to the pile surface, will depend on the normal effective stress, the interface roughness, and the rate of driving.

For the blue London clay, the slow peak strength is within one and one and a half times the soil-steel slow residual strength when a rate of displacement of 80 mm/min and 2600 mm/min respectively had been used during the pile driving. Despite the increase of the slow peak strength with increasing preceding rate of displacement, the presence of the pre-existing shear surface will cause the soil-interface slow residual strength to be established after a short displacement. For a preceding rate of 2600 mm/min, the slow residual conditions will be established after a displacement of about 25mm (Fig.A2.48c).

Table 5.1 Summary of granular void ratios, slow shear modes, and residual strength envelopes of soils

Test No	Site and soil type	LL %	PI %	<2 μ m %	G _s	w _f ¹ %	e _g	shear mode	$\bar{c}'_R$	$\tan\bar{\phi}'_R$
<u>Soils from Kalabagh dam site</u>										
16	sandstone	-	NP	1	2.76	17.4	0.479	TU	3.4	0.644
11	siltstone	40	13	3	2.67	23.9	0.688	TR/TU	34.6	0.440
15	clayey siltstone C	39	17	10	-	20.9	0.729	TR/TU	16.1	0.449
3	clayey siltstone A	39	18	20	2.66	20.0	0.910	TR	11.2	0.322
4	>>					18.7	0.867			
5	>>					20.3	0.920			
12	clayey siltstone B ²	44	21	8	-	21.5	0.960	TR	7.7	0.315
13	>>					21.0	0.943			
17	clayey siltstone D	44	24	25	2.66	(17.0)	0.929	TR	2.8	0.302
18	>>					(22.2)	1.114			
19	>>					(21.8)	1.100			
20	>>					(20.9)	1.189			
21	>>					(24.3)	1.080			
6	clayey siltstone A	49	27	27	2.66	19.4	1.069	TR	-	-
7	>>					19.1	1.058			
22	claystone D	53	26	37	2.71	20.3	1.430	TR/S	1.9	0.168
10	claystone C	55	28	39	2.74	24.0	1.734	TR/S	3.2	0.165
9	claystone B	56	31	40	-	22.4	1.644	TR/S	3.9	0.178
1	claystone A	62	36	52	2.72	27.6	2.587	S	-0.5	0.161
2	>>					25.4	2.465			
<u>Tills</u>										
23	Lower Cromer Till	26	14	22	2.64	13.3	0.729	TU	-1.1	0.569
24	Magnus Till	32	15	37	-	17.3 ³	1.303	TR/TU	-	-
25	weathered Cowden Till	39	21	31	-	21.6	1.272	TR/TU	6.5	0.417
<u>Residual Soils</u>										
28	Mauritius	49	6	11	2.69	31.6	1.068	TU	9.6	0.657
29	Japan	63	9	10	2.36	(54.0)	1.710	TU	-7.6	0.673
30	Japan	63	13	14	2.36	(50.5)	1.726	TU	-6.7	0.625
27	Fiji	85	22	55	2.12	58.2	4.640	S	12.9	0.167
26	Fiji	90	37	40	-	55.5-61.1 ³	3.117-3.366	S	3.6	0.299
<u>Other Natural Soils</u>										
32	mechanically degraded Tuff	50	14	-	-	-	-	TR	16.3	0.260
31	weathered Tuff	58	17	-	-	-	-	S	6.0	0.138
33	Senerchia	52	28	39	-	(26.2)	1.767	S	-6.7	0.106
34	Buoninvetre	53	36	39	-	25.5	1.737	S	-2.8	0.185
<u>Artificial Soils</u>										
35	clayey silt	25	9	18	2.64	(16.3)	0.744	TU	0.6	0.655
36	sandy clay	22	10	23	2.64	(14.5)	0.793	TU	2.8	0.572
<u>Soils of very high plasticity</u>										
37	kaolin	72	36	82	2.66	(40.8)	10.470	S	10.7	0.188
38	blue London clay	71	45	53	2.70	(29.8)	2.788	S	6.4	0.172
39	>>					(33.4)	2.992	S		
<u>Sands</u>										
43	medium sand	-	NP	-	2.69	-	-	TU		
44-47	fine sand	-	NP	-	2.65	-	-	TU	1.9	0.574

1: w_f is the final water content at the shear zone. The numbers in parenthesis indicate the final water content of the whole sample.

2: The % < 2 μ m of soil is rather low for its plasticity index. A % < 2 μ m = 20 was used for the calculation of e_g

3: Water content calculated from expression (5.2).

Table 5.2 Loading times to threshold strength (sec)

Site and soil type	Rate of displacement (mm/min)					
	9.5-12	100-133	360-400	1000-1200	2000-2600	5400-6000
$\sigma_n = 400 - 500$ kPa						
sandstone		0.89	0.14		0.04	
clayey siltstone D		0.35	0.30		0.01	
claystone D	0.90-1.65	0.20	0.17		0.05	
Lower Cromer Till		0.40		0.02		
Fiji (Test 27)		0.10		0.04		
Mauritius	4.10	1-1.3	1.3-1.4	0.03-0.04	0.01-0.03	
Japan (Test 29)	4.53	1.2	1.05	0.07	0.03	
Japan (Test 30)		1.5	1.30	0.09	0.025	
clayey silt			1.12	0.06	0.025	
sandy clay		0.56		0.04	0.020	
blue London clay	3.50-10.5	0.28	0.40	0.007		
fine sand		1.124	0.96-1.94	0.17	0.020	
$\sigma_n = 300$ kPa						
weathered Tuff	0.38	0.056				
mechanically degraded Tuff	0.50-0.72	0.052				
$\sigma_n = 200 - 250$ kPa						
sandstone		0.43	0.10		0.011	0.01
clayey siltstone D	1.80-5.8	0.15-0.40	0.20-0.30		≤ 0.01	≤ 0.01
claystone D	0.47-0.57	0.16	0.08-0.11		< 0.01	
Lower Cromer Till		0.12		0.015		
Fiji (Test 27)		0.05		0.03	0.01	
Mauritius	1.90		0.74-0.89	0.02	≤ 0.01	
Japan (Test 29)	4.23	1.01	0.63	0.06		0.03
Japan (Test 30)		0.68-0.70		0.04	0.02	0.01-0.03
sandy clay		0.20		0.04-0.10	0.02	
blue London clay	132-272	0.12	0.20	0.01	< 0.01	
fine Sand		1.29	0.80	0.03		0.03
$\sigma_n = 100 - 130$ kPa						
claystone D	0.40	0.06	0.08		< 0.01	
Lower Cromer Till		0.10		0.005		
Japan (Test 30)		0.42	0.85	0.03	0.015	0.03
weathered Tuff	0.35					
mechanically degraded Tuff	0.45	0.06				
fine Sand		0.9-1	0.7-0.95	0.03		0.04
$\sigma_n = 50 - 60$ kPa						
claystone D	0.77	0.07	0.07		< 0.01	
Lower Cromer Till		0.07-0.09		{0.003 - 0.004		
clayey silt			0.46	0.005	0.005	

Table 5.3 Loading times to fast peak strength (sec)

Site and soil type	Rate of displacement (mm/min)					
	9.5-12	100-133	360-400	1000-1200	2000-2600	5400-6000
<u>$\sigma_n = 400 - 500$ kPa</u>						
clayey siltstone D		5.60	2.60		0.05	
claystone D	0.90-1.65	1.30	3.80		0.08	
Fiji (Test 27)		1.60-5.80		0.13-0.19		
blue London clay	3.5-10.5	4.3-10.7	6.50	0.18		
<u>$\sigma_n = 200 - 250$ kPa</u>						
clayey siltstone D	56-584	4.1-4.5	2.1-2.7		≤ 0.01	≤ 0.01
claystone D	0.47-0.57	1.33	2.4-2.8		0.06	
Fiji (Test 27)		7.0		0.31	0.01	
blue London clay	132-272	8.4-14.9	3.59	0.93		0.21
<u>$\sigma_n = 100 - 130$ kPa</u>						
claystone D	0.40	5.59	2.73		< 0.01	
<u>$\sigma_n = 50 - 60$ kPa</u>						
claystone D	0.77	14.08	2.60		0.08	

Table 5.4 Summary of types of soil behaviour during shearing at increasing rates of displacement

Site and soil type	LL (%)	PI (%)	<2 μ m (%)	shear mode	Net normal stress σ_n (kPa)	
Type I						
medium sand		NP	-	TU	200	
fine sand		NP	-	TU	110-450	
sandstone		NP	-	TU	200-400	
Mauritius (prior to degradation)	49	6	11	TU	220	
Japan	63	9	10	TU	230-480	} Soils showing type II behaviour at lower normal stresses
Japan	63	13	14	TU	480	
clayey silt	25	9	18	TU	450-470	
siltstone	40	13	3	TU/TR	500	
Type II						
Lower Cromer Till	26	14	22	TU	45-450	} Type IIa
Japan	63	13	14	TU	130-260	} Type IIb
siltstone	40	13	3	TU/TR	200	
clayey silt	25	9	18	TU	55-65	} Type IIc
clayey siltstone C	39	17	10	TU/TR	200-500	
clayey siltstone A	39	18	20	TR	200-500	
clayey siltstone B	44	21	8	TR	200-500	
clayey siltstone D	41	21	13	TR	200-500	
Fiji	90	37	40	S	177	
Type III						
clayey siltstone D	44	24	25	TR	200-500	
claystone D	53	26	37	TR/S	65-500	
Buoninvetre	53	26	39	S	490	
Type IV						
sandy clay	22	10	23	TU	200	} Type IVa
Buoninvetre	53	26	39	S	200	
Cowden Till	39	21	31	TR/TU	50	} Type IVb
Senerchia	52	28	37	S	920	
claystone C	55	28	39	TR/S	200	
claystone B	56	31	40	TR/S	200-500	
claystone A	62	36	52	S	200-500	
Type V						
sandy clay	22	10	23	TU	450	
Cowden Till	39	21	31	TR/TU	100-200	
kaolin	72	36	82	S	510	
blue London clay	71	45	53	S	240-440	
blue London clay ¹	75	51	48	S	220	

Note:

1, soil tested by Lupini (Table 2.2)

Table 5.5 Summary of results from tests involving soil shearing against an interface

Site and soil type	LL (%)	PI (%)	<2 μ m (%)	Interface	Roughness C.L.A. (μ m) Initial	Final	Cut-off (mm)	Net Normal stress σ_n (kPa)	$\tan\phi'_{EI}$	$\tan\phi'_E$	a	Reference
clayey siltstone A	49	27	27	glass	8.50	-	2.50	500	0.237	0.260	0.91	Lemos (1986)
clayey siltstone B	44	21	8	>>	8.50 ¹	-	>>	500	0.280	0.330	0.85	>>
clayey siltstone D	44	24	25	>>	7.30	7.00	>>	200	0.295-0.320	0.316	0.93-1.01	This study
claystone D	53	26	37	>>	7.40	7.00	>>	500	0.262	0.309	0.85	>>
clayey siltstone D	44	24	25	stainless steel	8.30	8.10	>>	220	0.180	0.176	1.02	>>
blue London clay	71	45	53	>>	8.40	8.30	>>	480	0.185	0.172	1.07	>>
clay (w.c = 22.8%)	37	15	38	mild steel	rough ²	-	-	200	0.273	0.316	0.86	>>
>> (w.c = 26.1%)	>>	>>	>>	>>	>>	-	-	500	0.262	0.308	0.85	>>
clay	-	-	-	steel	rough ³	-	-	450	0.155	0.186	0.83	>>
clay	43	20	34	>>	>>	-	-				0.61	Potyondy (1961)
kaolinite	85	55	-	steel	0.18	-	-				0.56	>>
illite	53	23	-	>>	0.18	-	-	50-260	0.200	0.255	0.78	Meyerhof (1962)
kaolin	76	36	82	mild steel	0.185-0.215	-	-	200	0.175-0.185	0.241	0.57	Clark and Meyerhof (1972)
					2.90-3.10	-	-	200	0.200-0.220	0.241	0.73-0.77	Littleton (1976)
											0.80	>>
											0.83-0.91	Lemos (1986)
												>>

$\tan\phi'_{EI}$ is the drained residual coefficient of friction of soil sheared against an interface

$\tan\phi'_E$ is the drained residual coefficient of friction of soil and

$$a = \tan\phi'_{EI}/\tan\phi'_E$$

1: assumed as identical to that used in Test 8 on clayey siltstone A, since the same forming process of the surface finish was used (sand blasting).

2: the interface roughness was not reported. The surface finish was prepared by artificially rusting and removing then the loose debris.

3: the interface roughness was not reported.

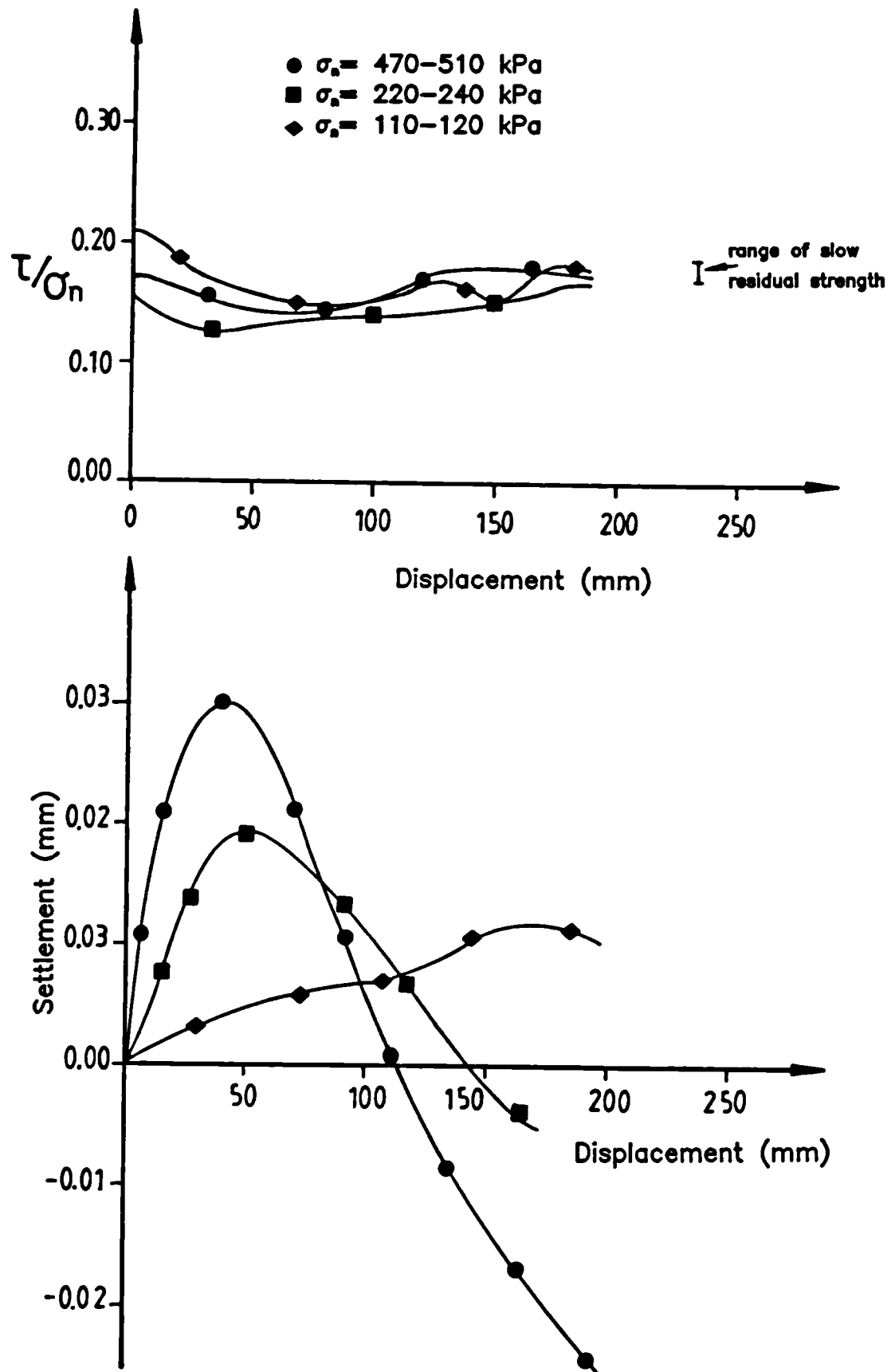


Fig. 5.1 Volume changes of claystone D during fast shearing at 9.5 mm/min

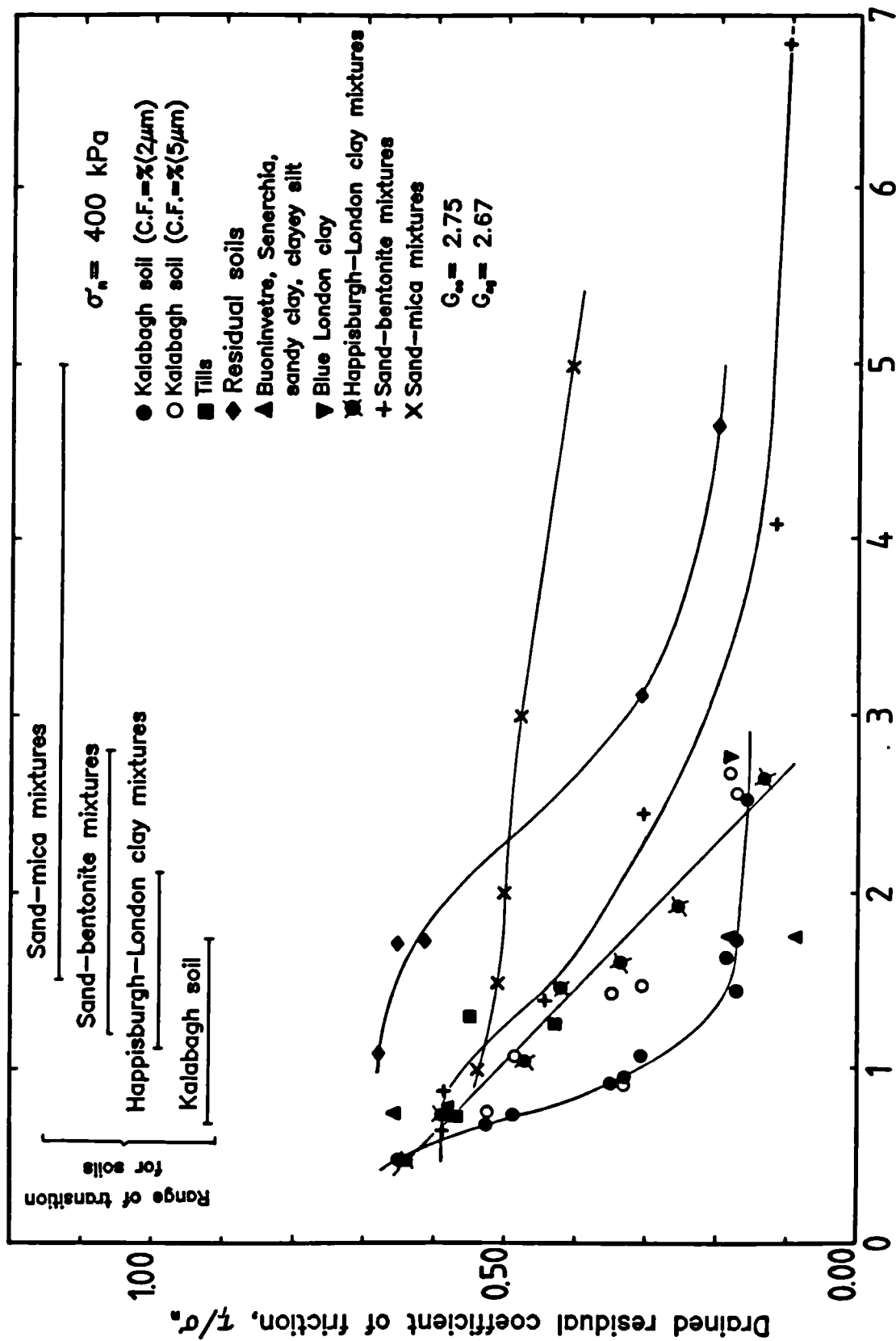


Fig. 5.2a Drained residual coefficient of friction of tested soils against granular void ratio

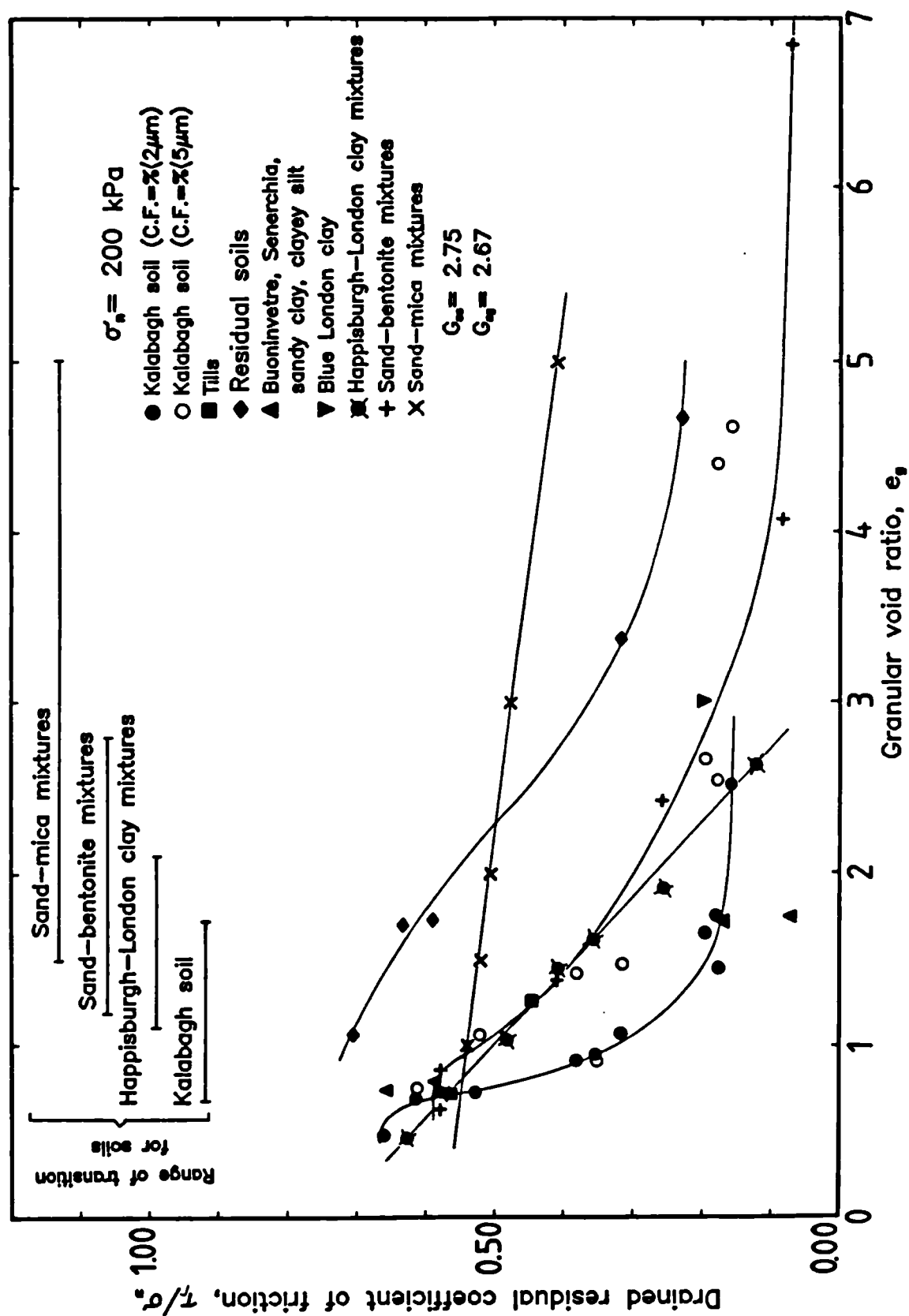


Fig. 5.2b Drained residual coefficient of friction of tested soils against granular void ratio

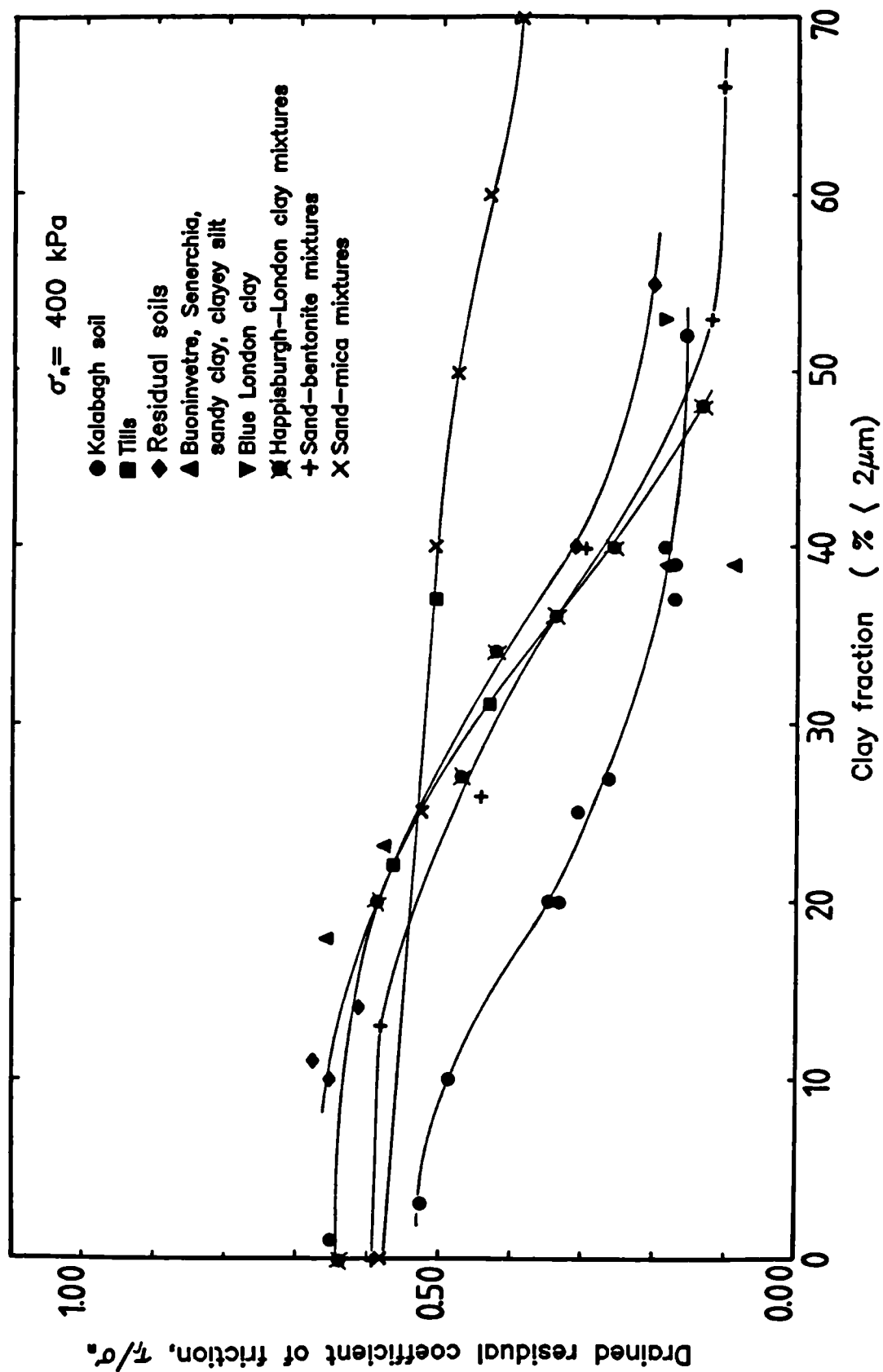


Fig. 5.3a Drained residual coefficient of friction of tested soils against clay fraction

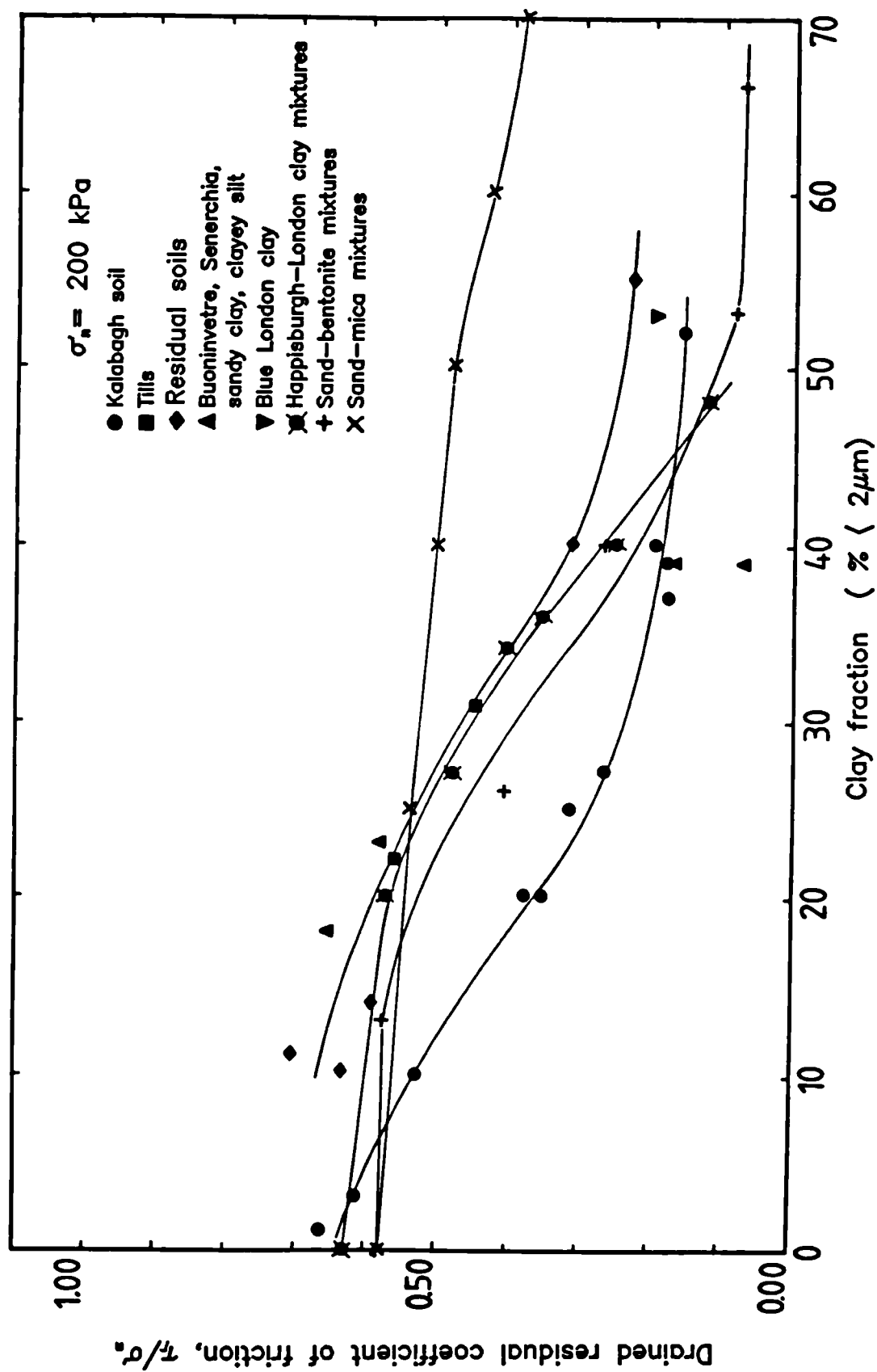


Fig. 5.3b Drained residual coefficient of friction of tested soils against clay fraction

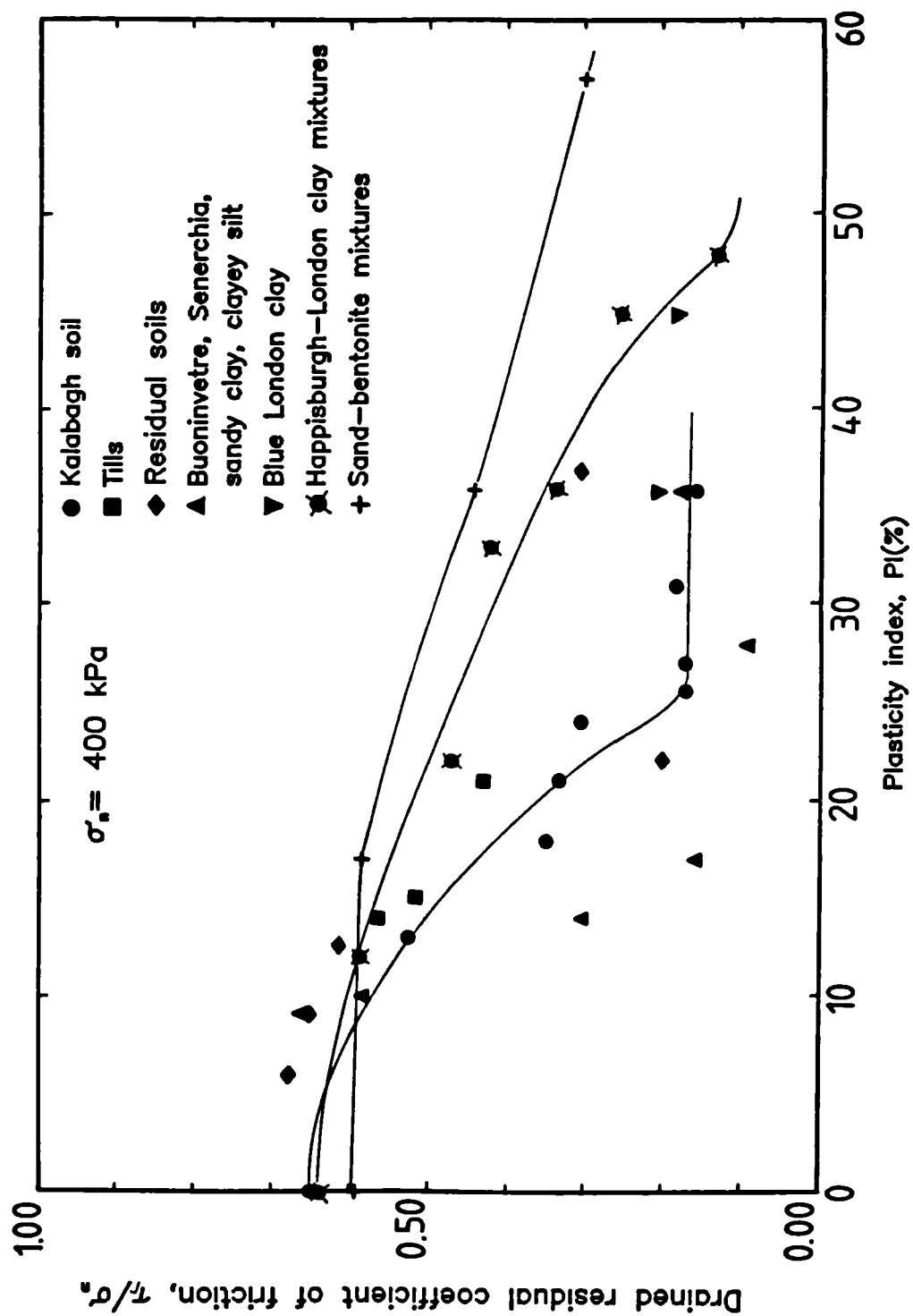


Fig. 5.4a Drained residual coefficient of friction of tested soils against plasticity index

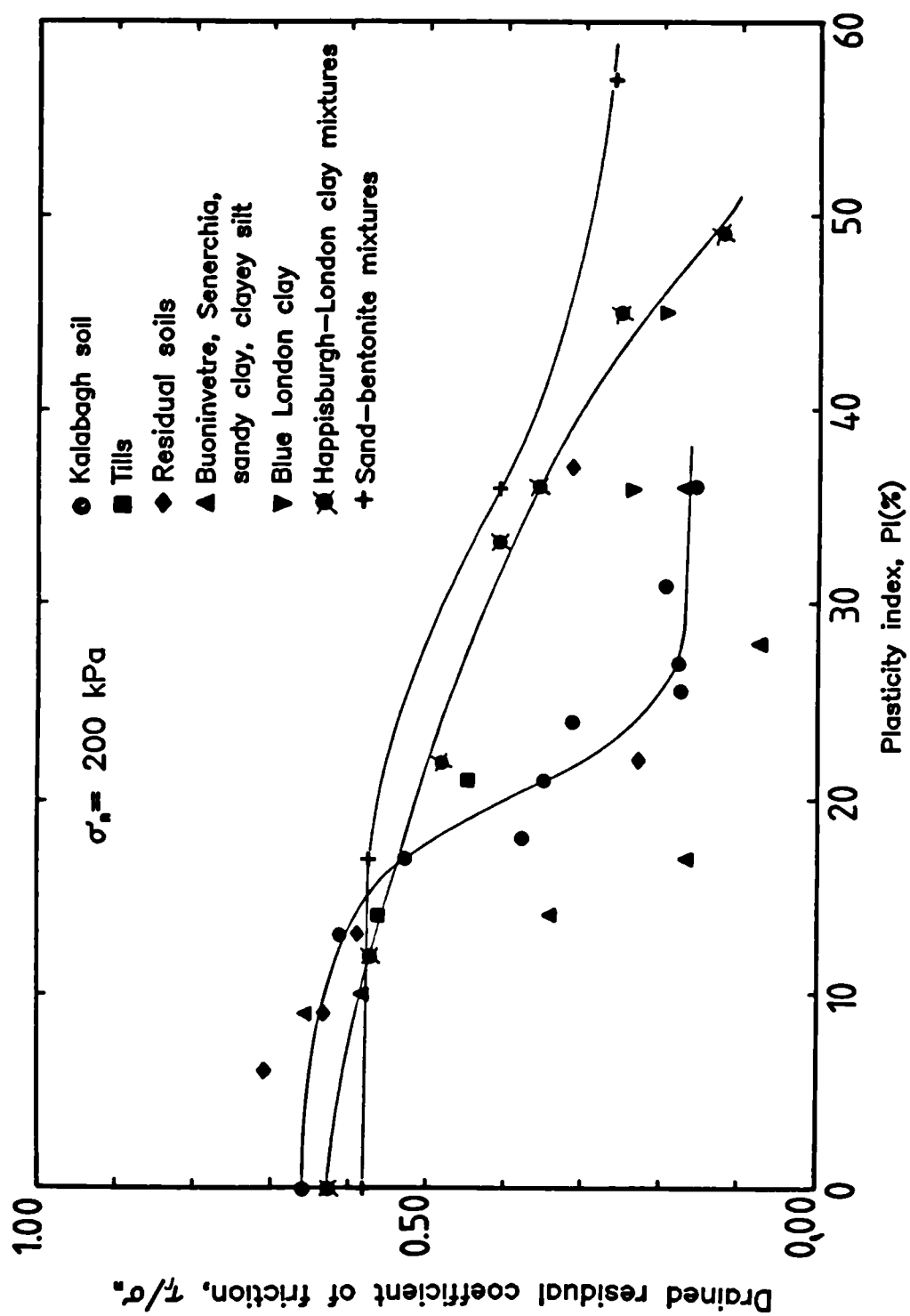


Fig. 5.4b Drained residual coefficient of friction of tested soils against plasticity index

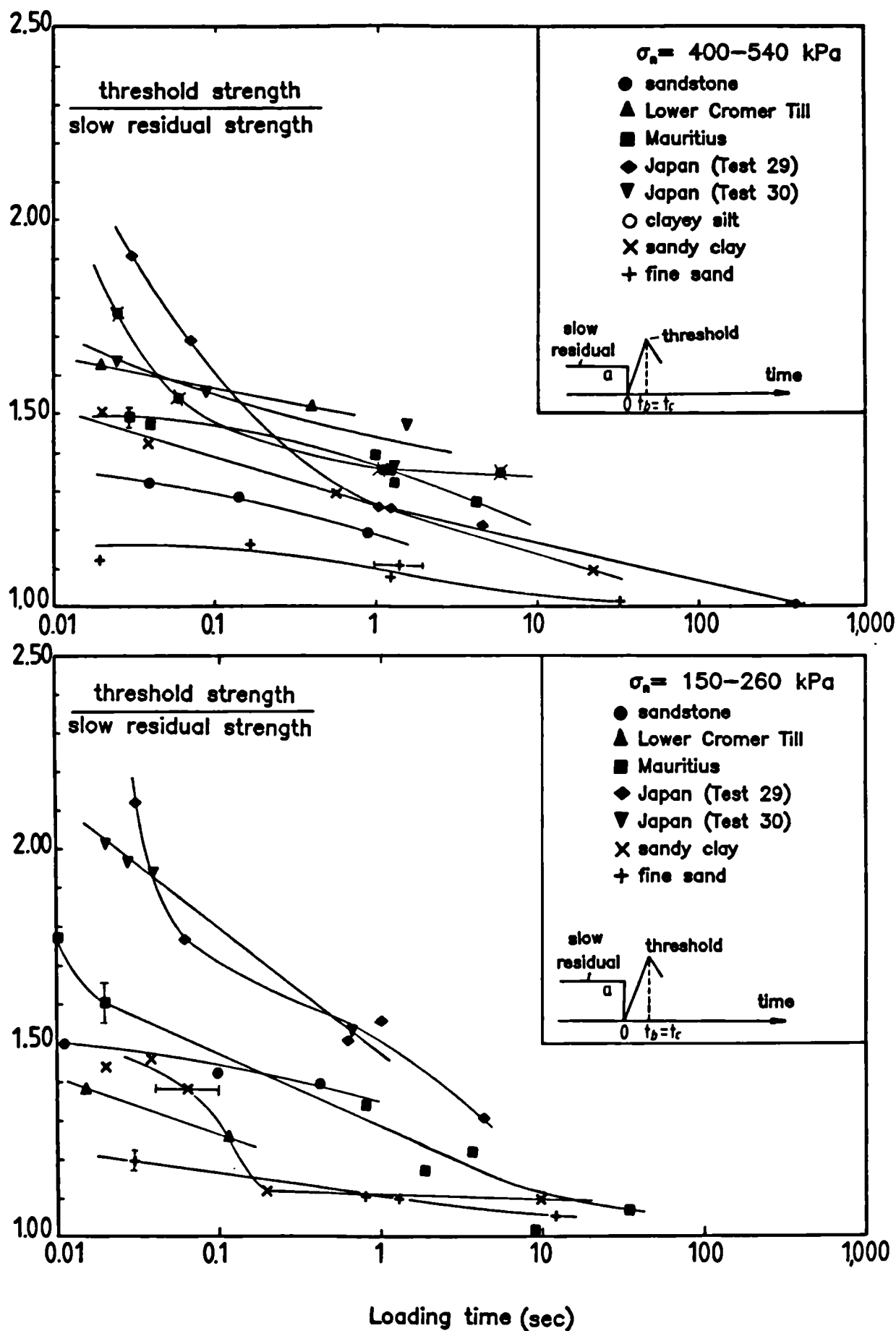


Fig. 5.5 Increase of threshold strength above the slow residual value against loading time (turbulent and transitional-turbulent shear modes)

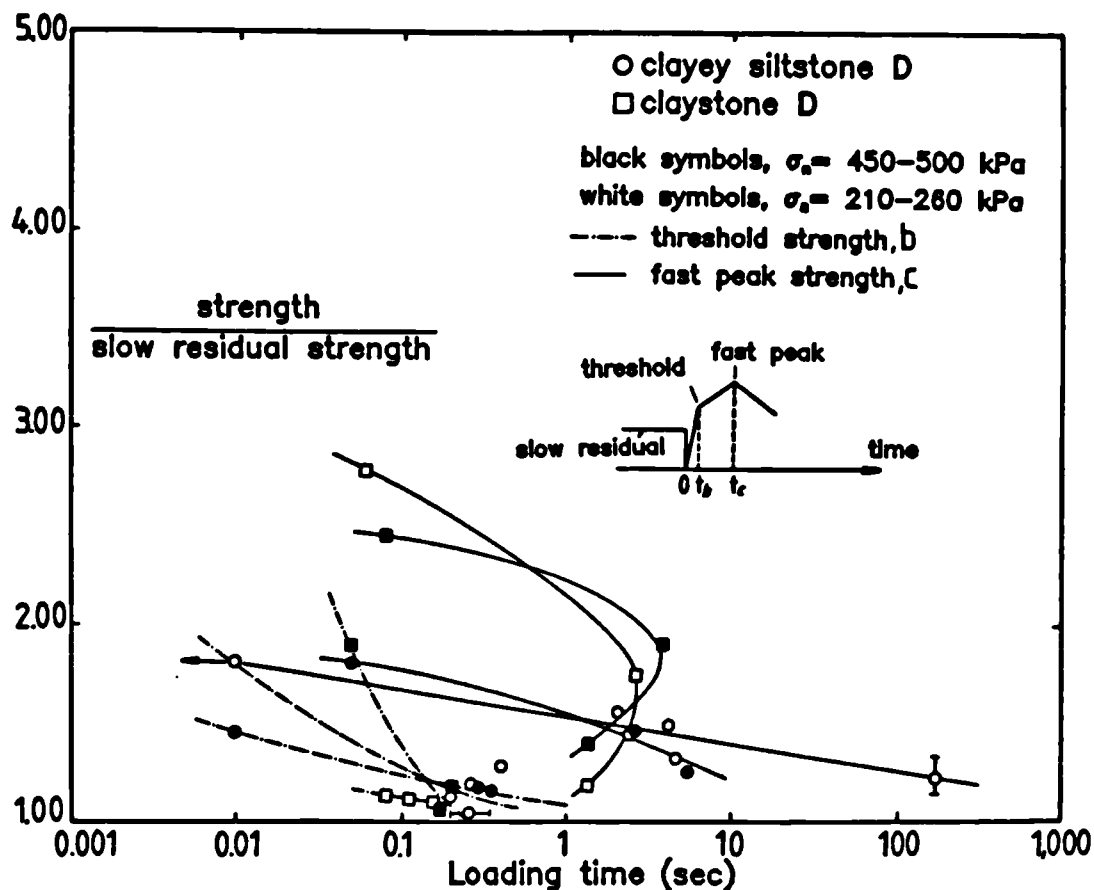


Fig. 5.6 Increase of threshold and fast peak strengths above the slow residual value against loading time (transitional and transitional-sliding shear modes)

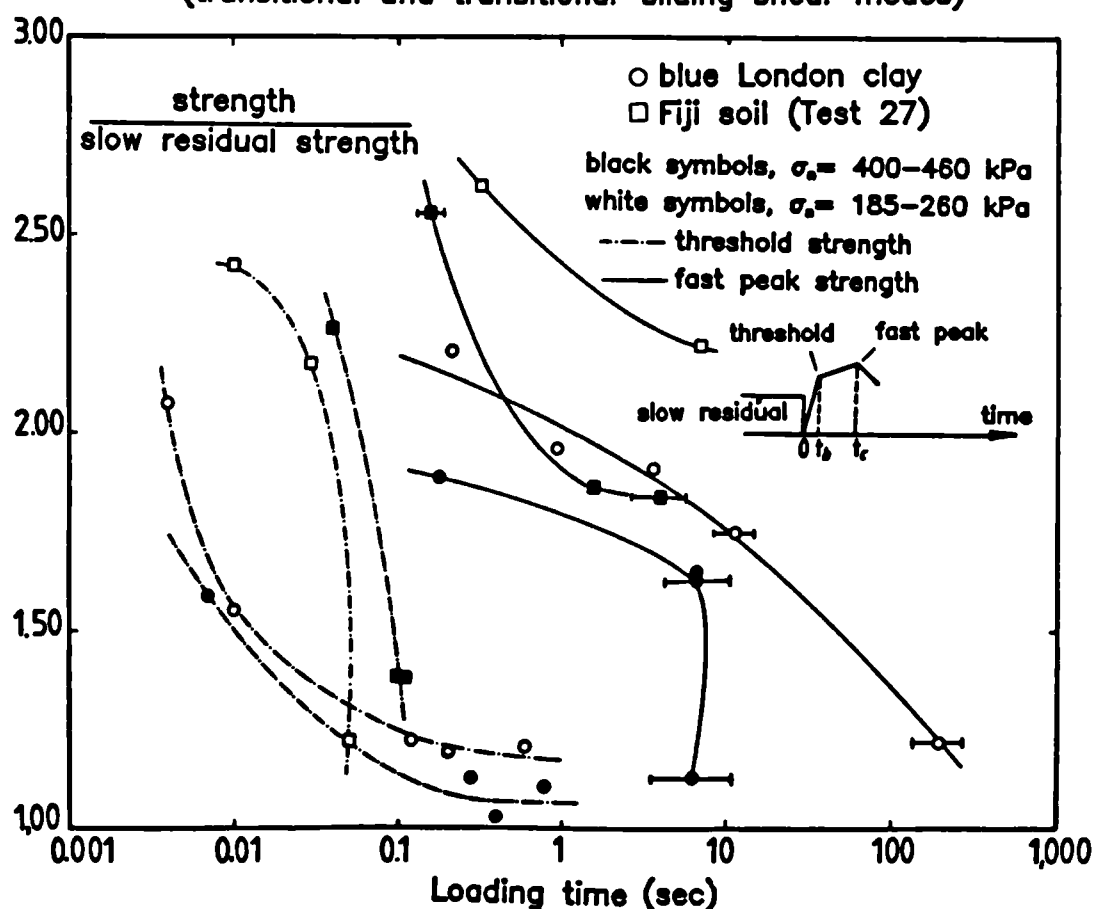


Fig. 5.7 Increase of threshold and fast peak strengths above the slow residual value against loading time (sliding shear mode)

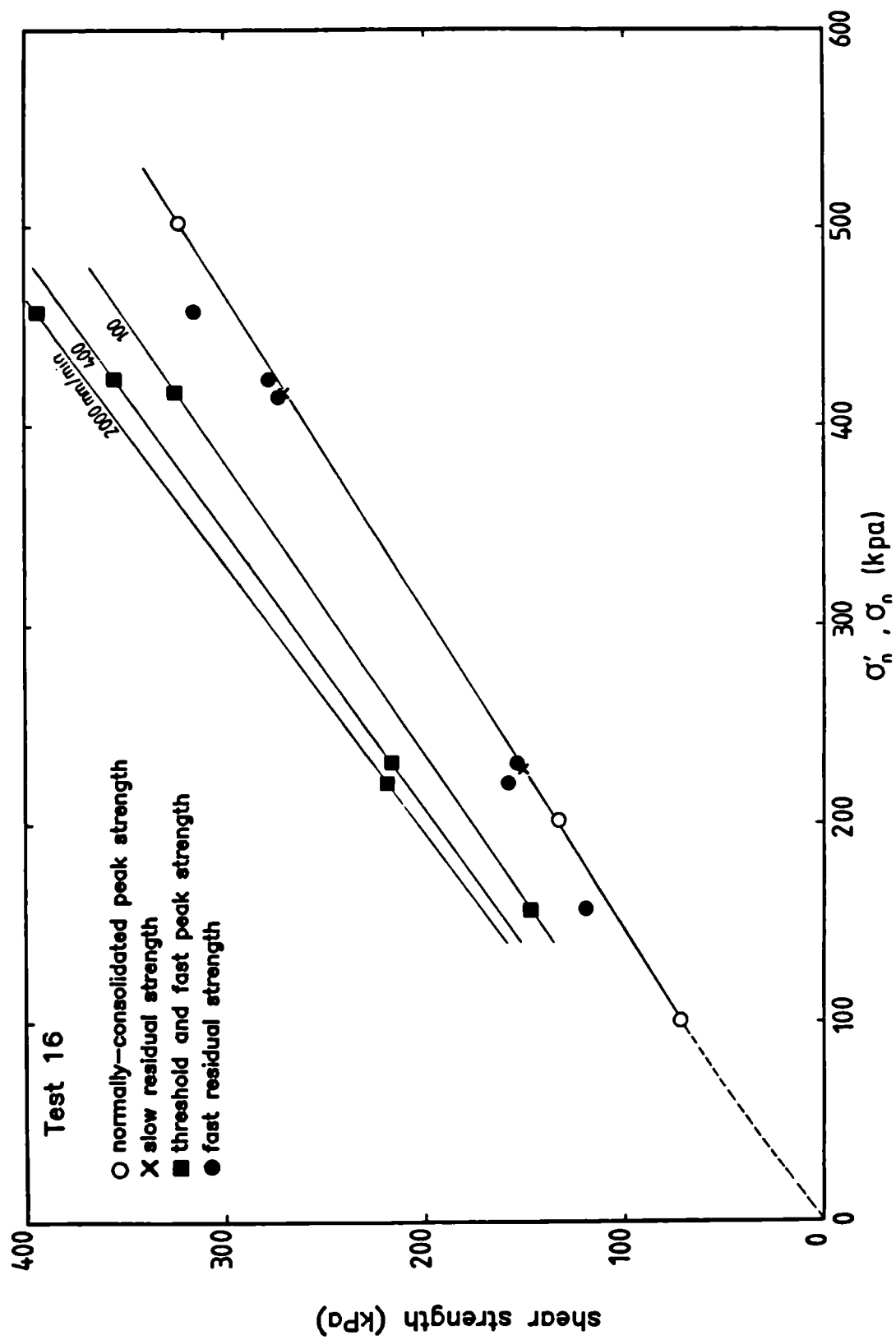


Fig. 5.8 Shear strength envelopes of sandstone

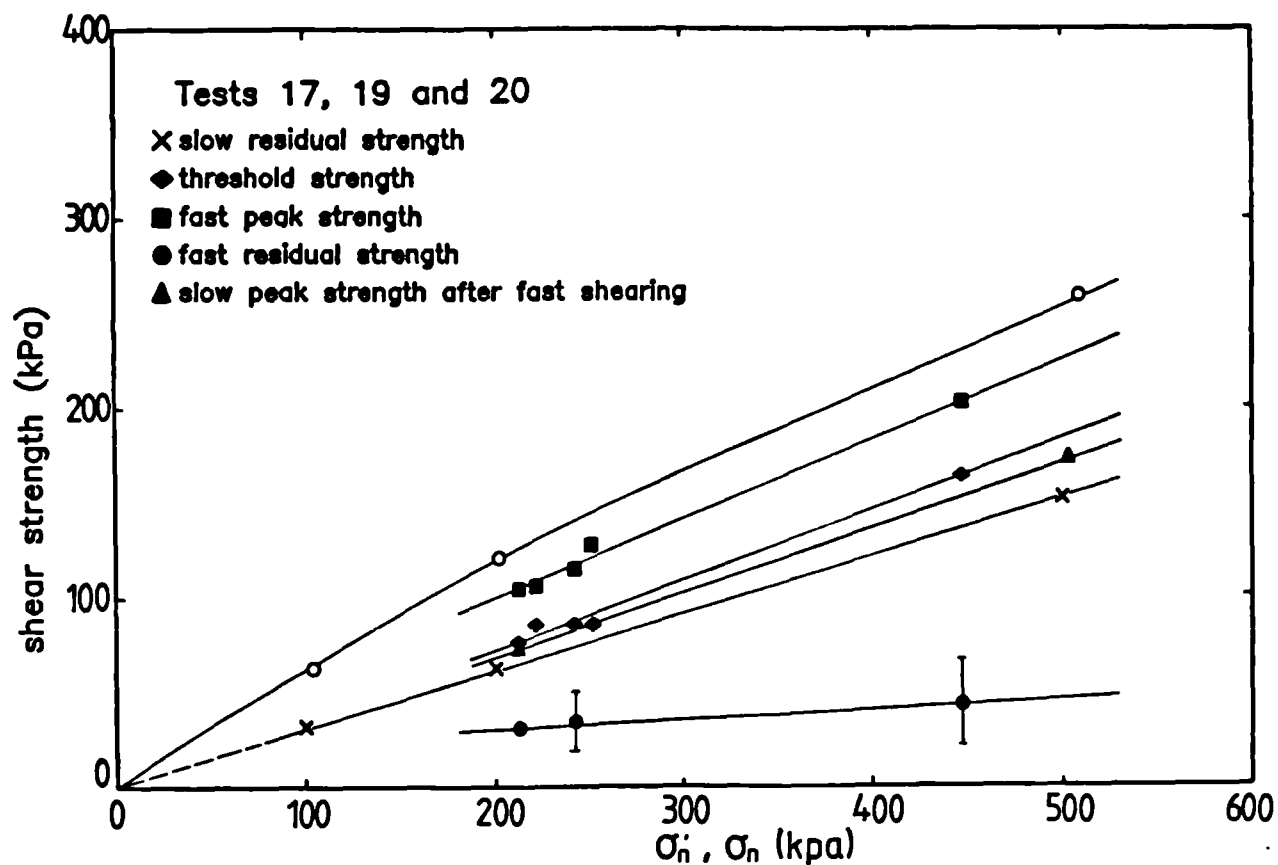


Fig. 5.9 Shear strength envelopes of clayey siltstone at 400mm/min

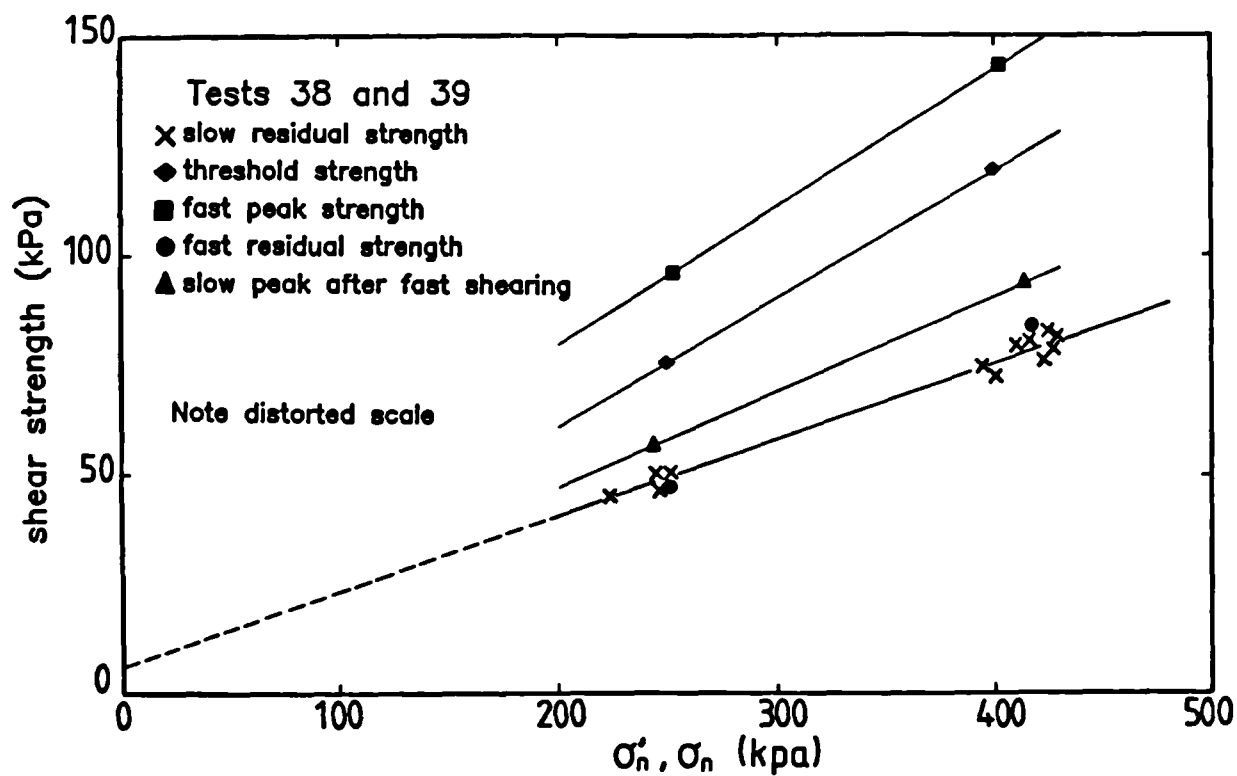


Fig. 5.10 Shear strength envelopes of blue London clay at 1100mm/min

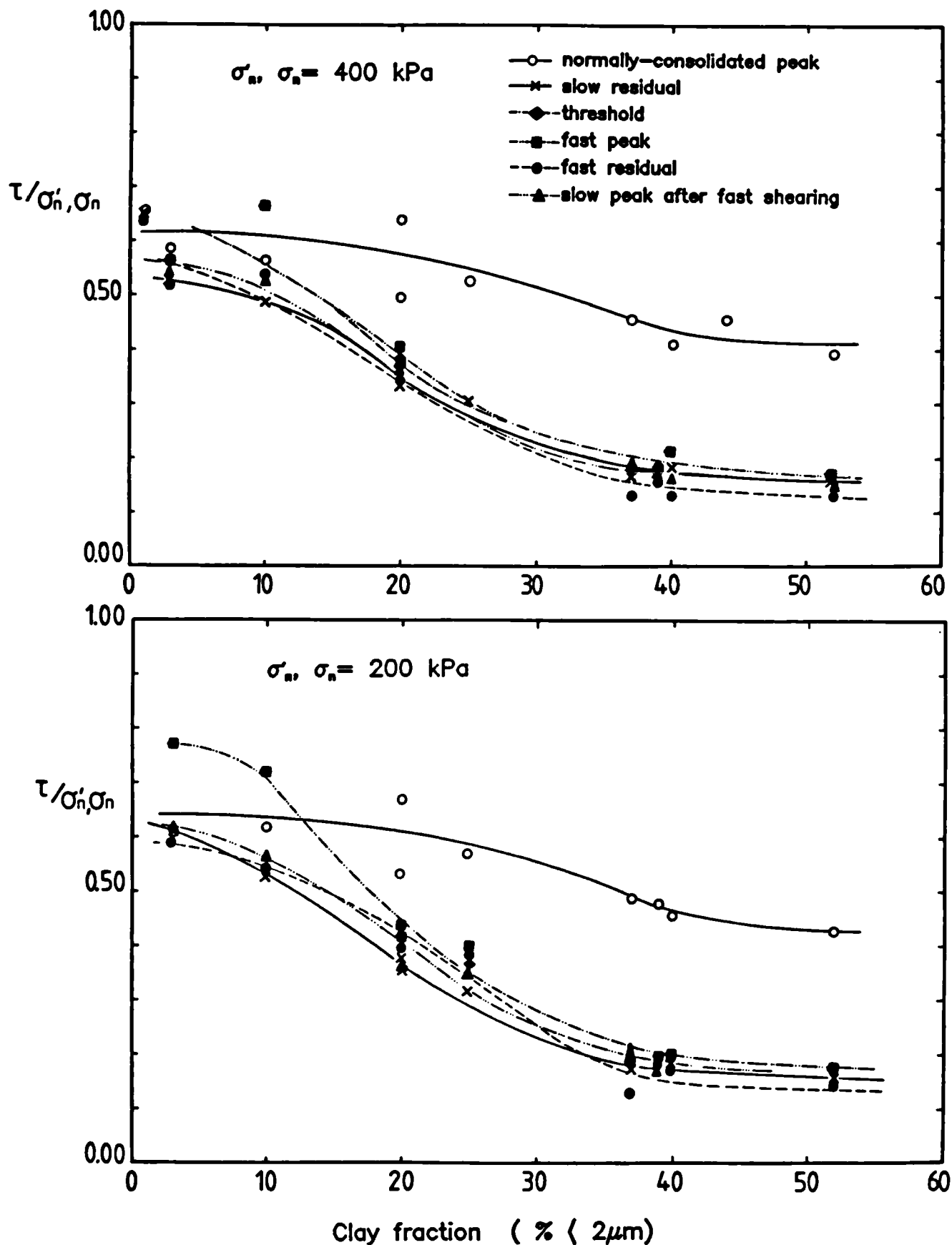


Fig. 5.11a Typical stress ratios of soils from Kalabagh dam site against clay fraction at 9.5 to 12mm/min

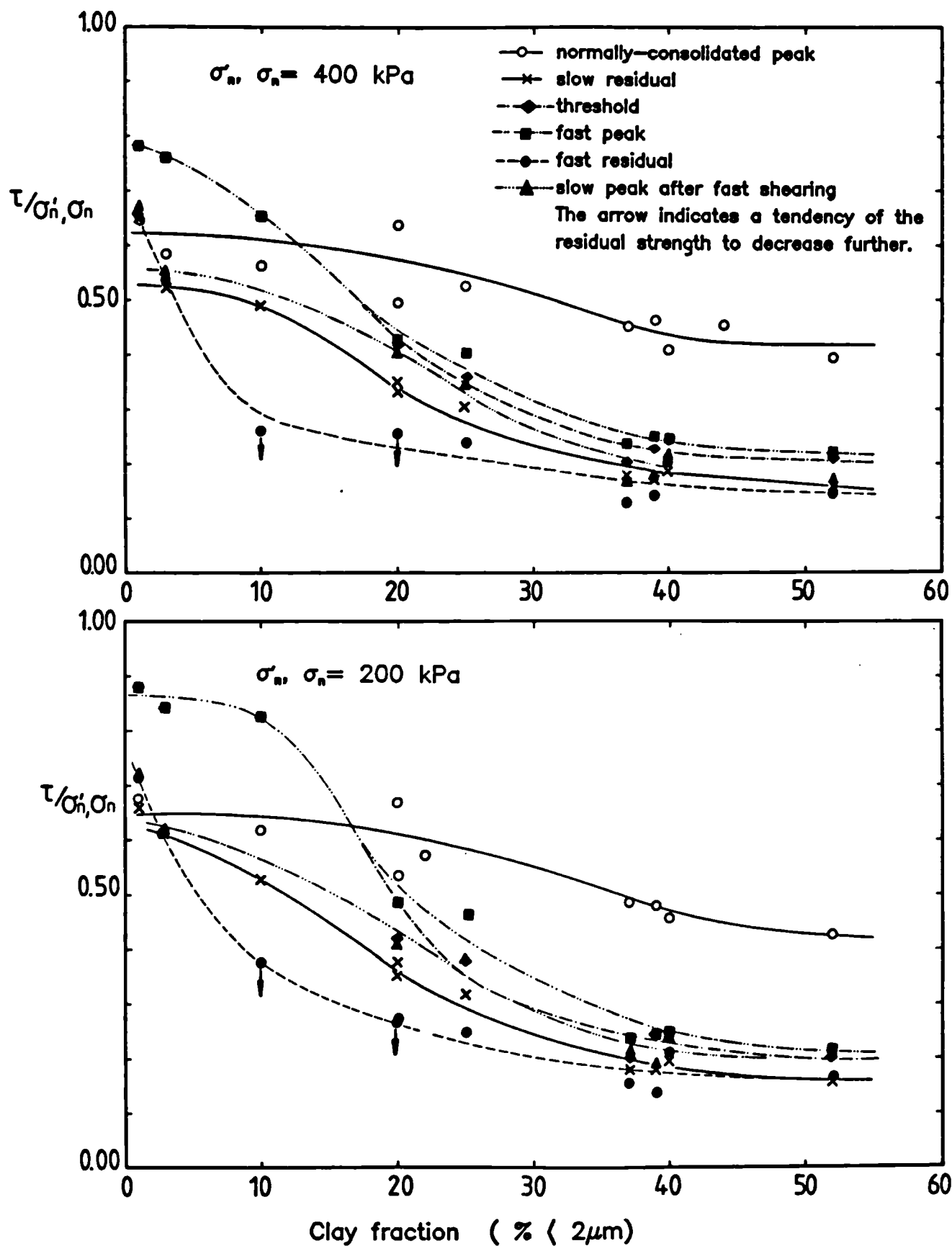


Fig. 5.11b Typical stress ratios of soils from Kalabagh dam site against clay fraction at 95 to 100mm/min

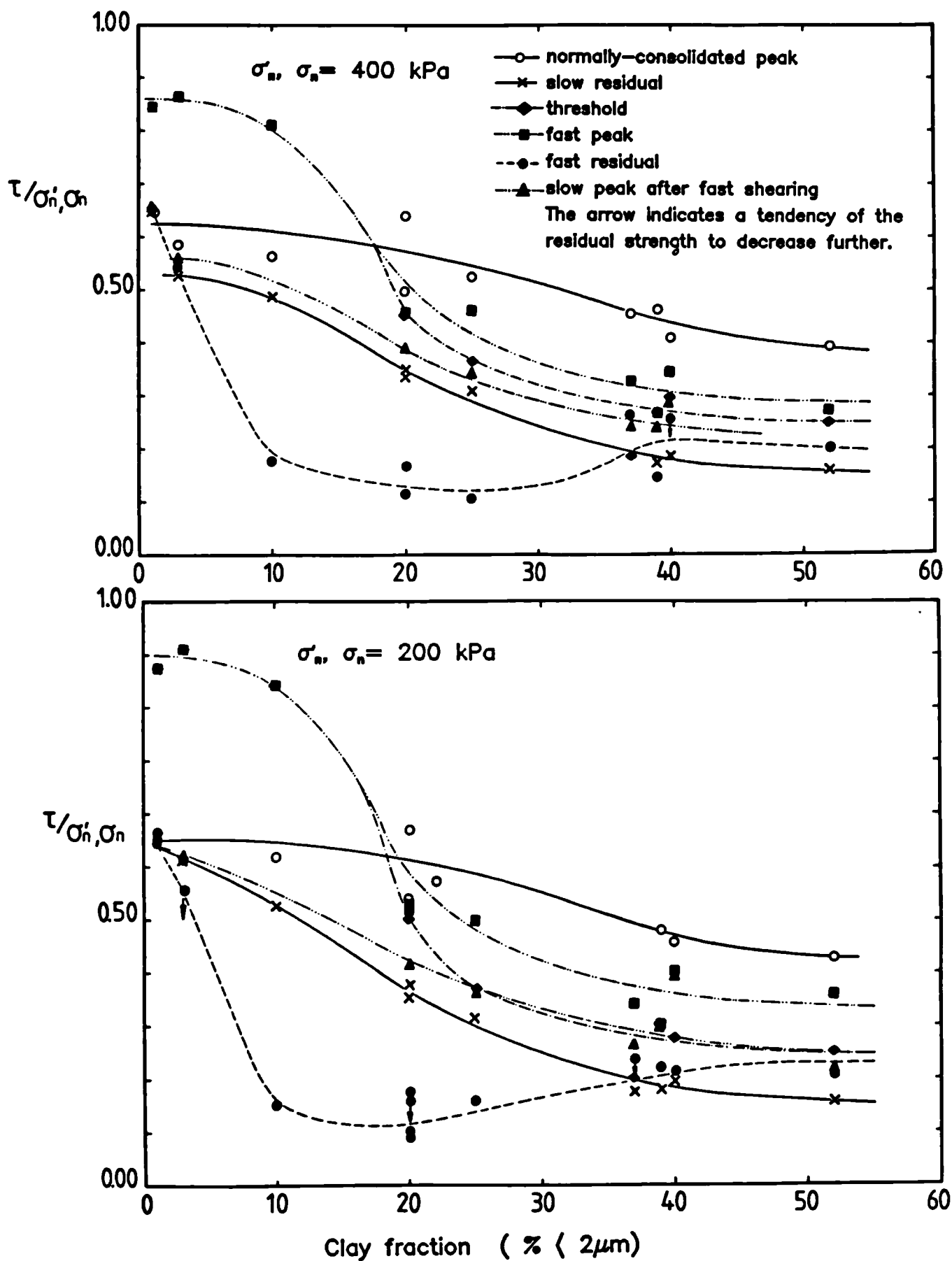


Fig. 5.11c Typical stress ratios of soils from Kalabagh dam site against clay fraction at 360 to 400mm/min

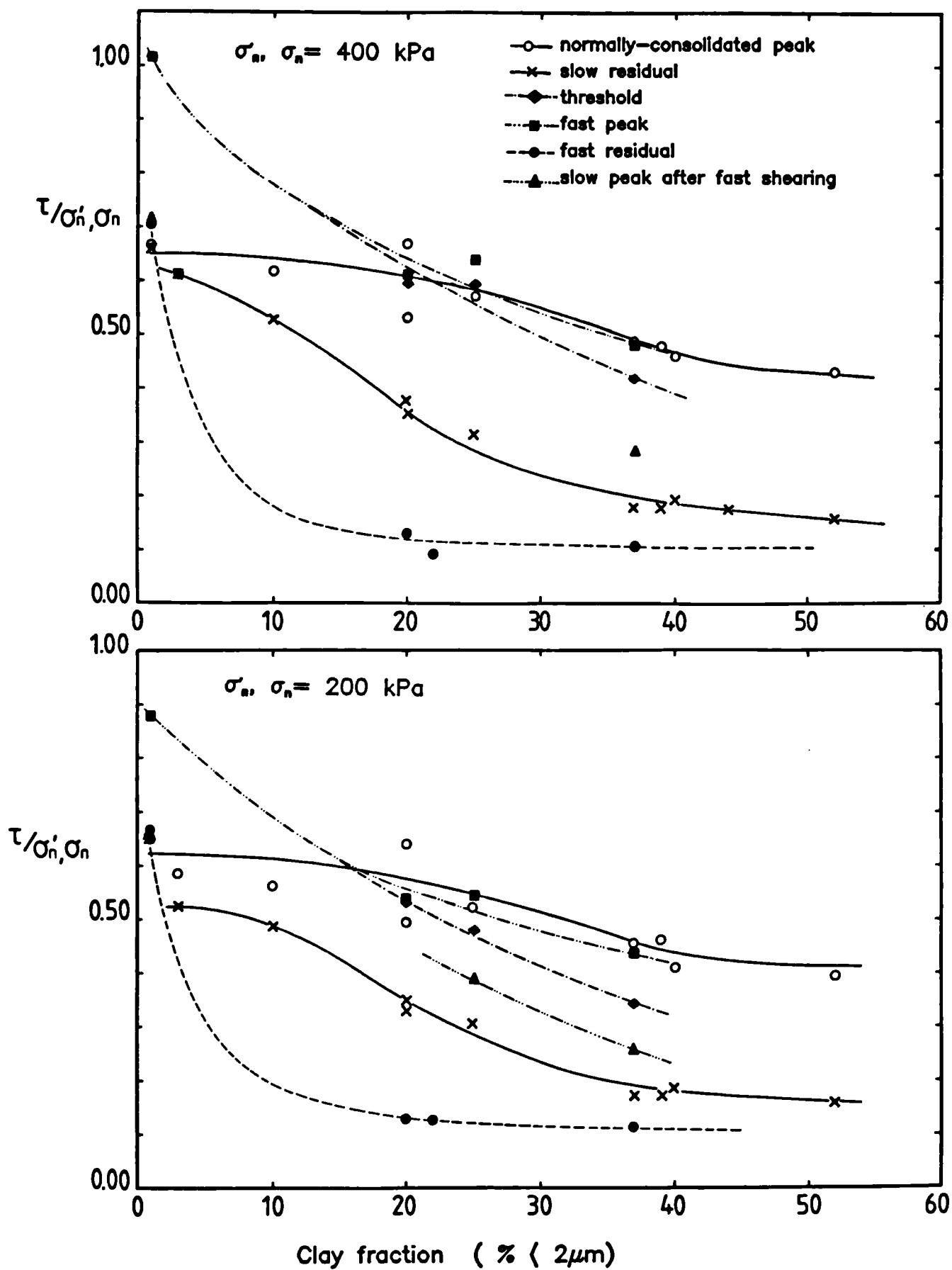


Fig. 5.11d Typical stress ratios of soils from Kalabagh dam site against clay fraction at 2000 to 2600mm/min

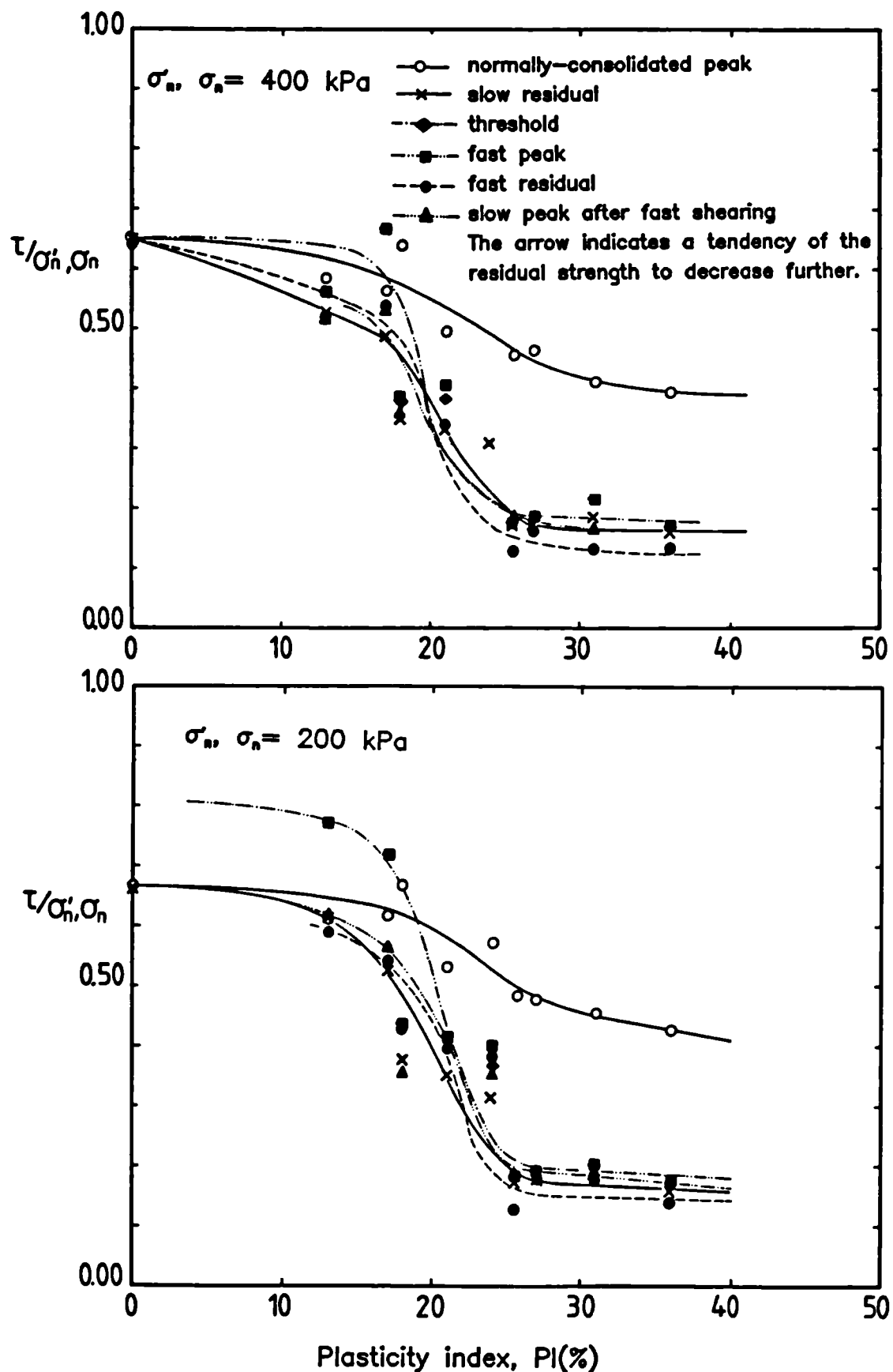


Fig. 5.12a Typical stress ratios of soils from Kalabagh dam site against plasticity index at 9.5 to 12mm/min

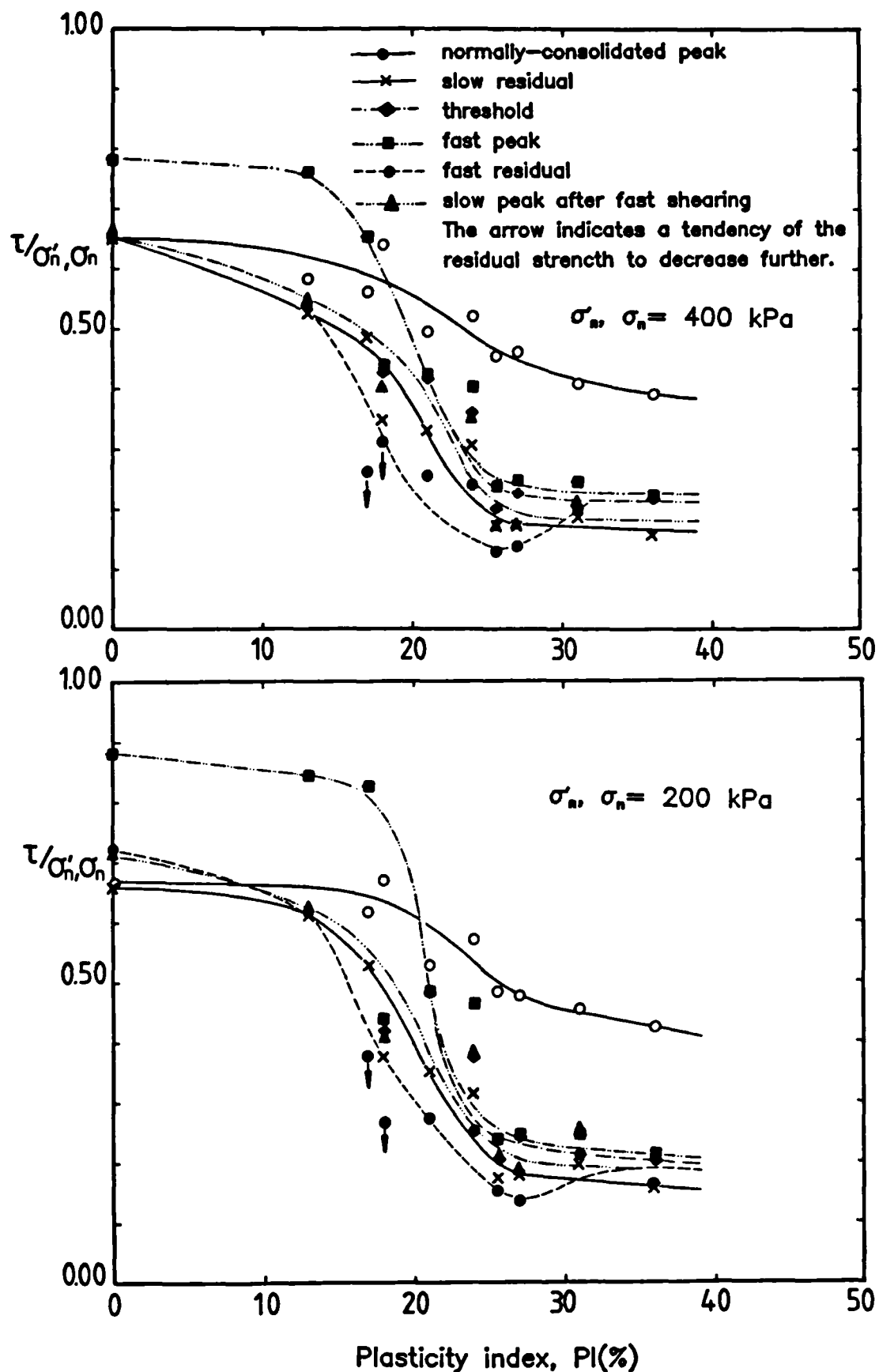


Fig. 5.12b Typical stress ratios of soils from Kalabagh dam site against plasticity index at 95 to 100mm/min

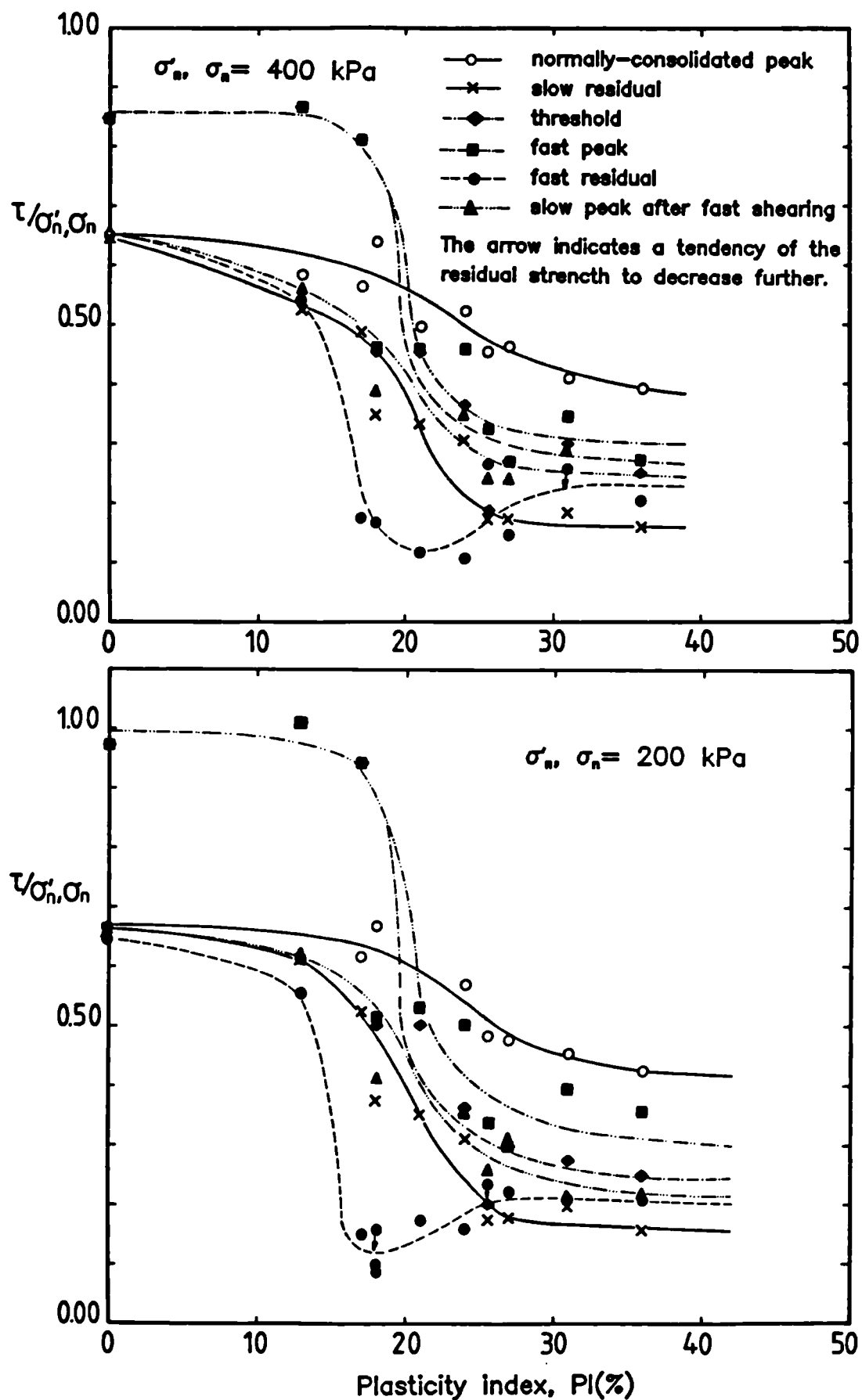


Fig. 5.12c Typical stress ratios of soils from Kalabagh dam site against plasticity index at 360 to 400mm/min

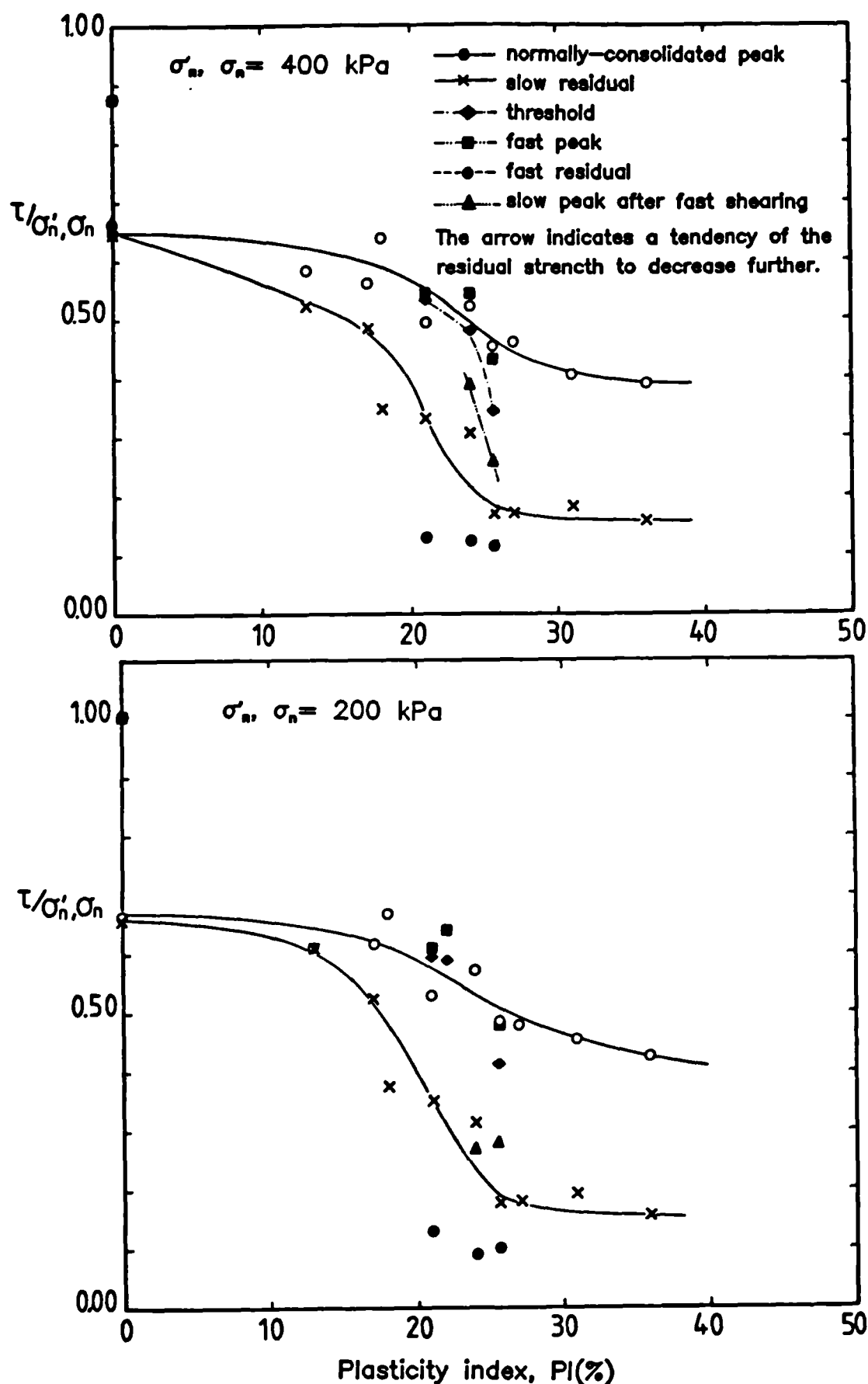


Fig. 5.12d Typical stress ratios of soils from Kalabagh dam site against plasticity index at 2000 to 2600mm/min

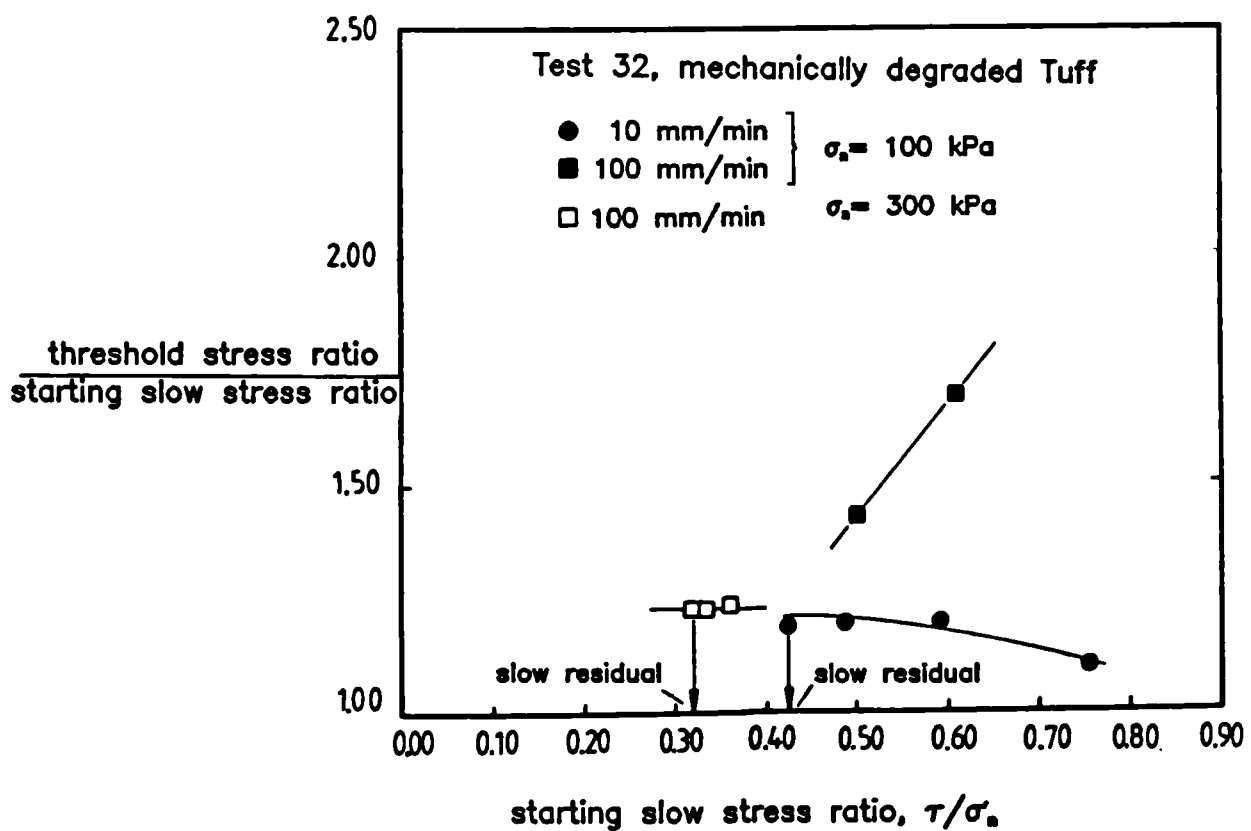
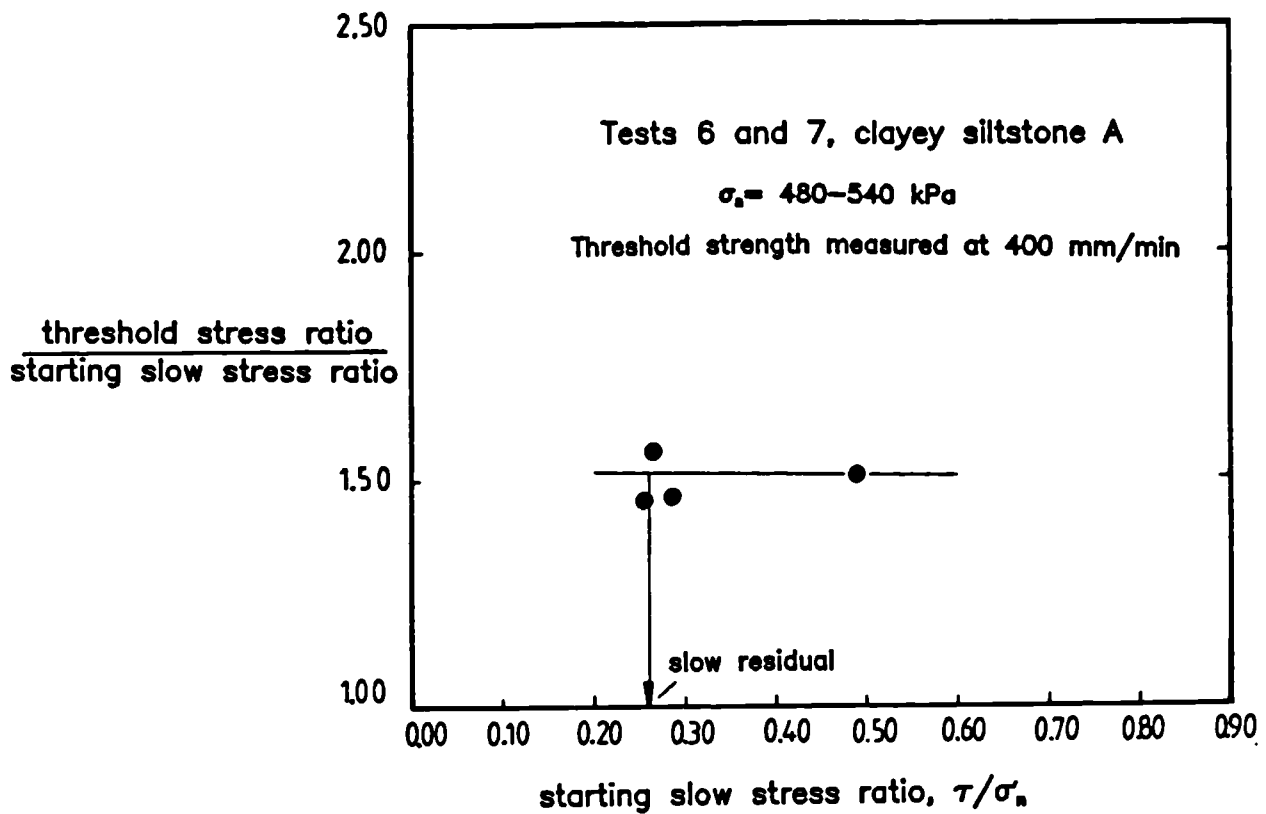


Fig. 5.13 Effect of starting strength on threshold strength

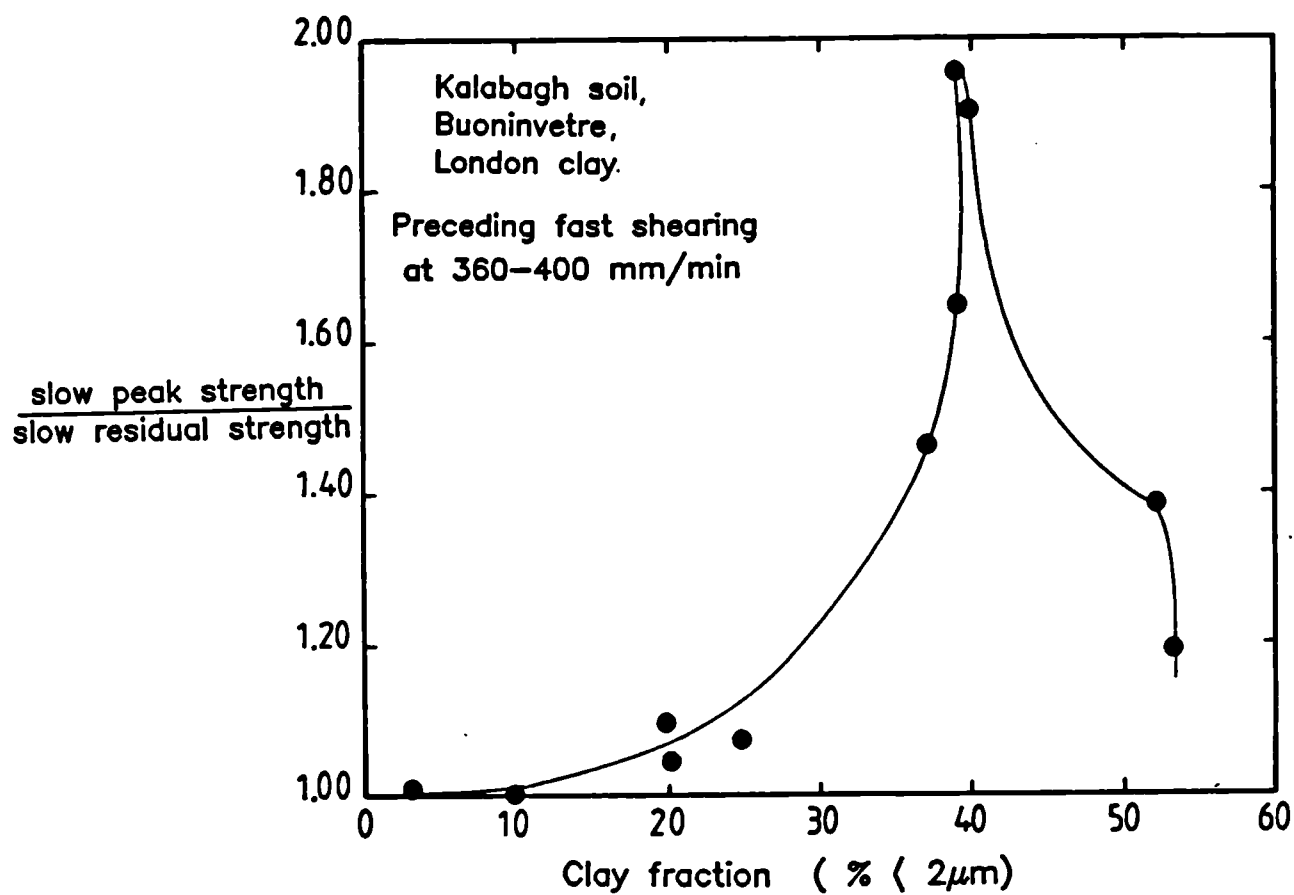


Fig. 5.14 Effect of clay fraction on the slow peak strength after fast shearing

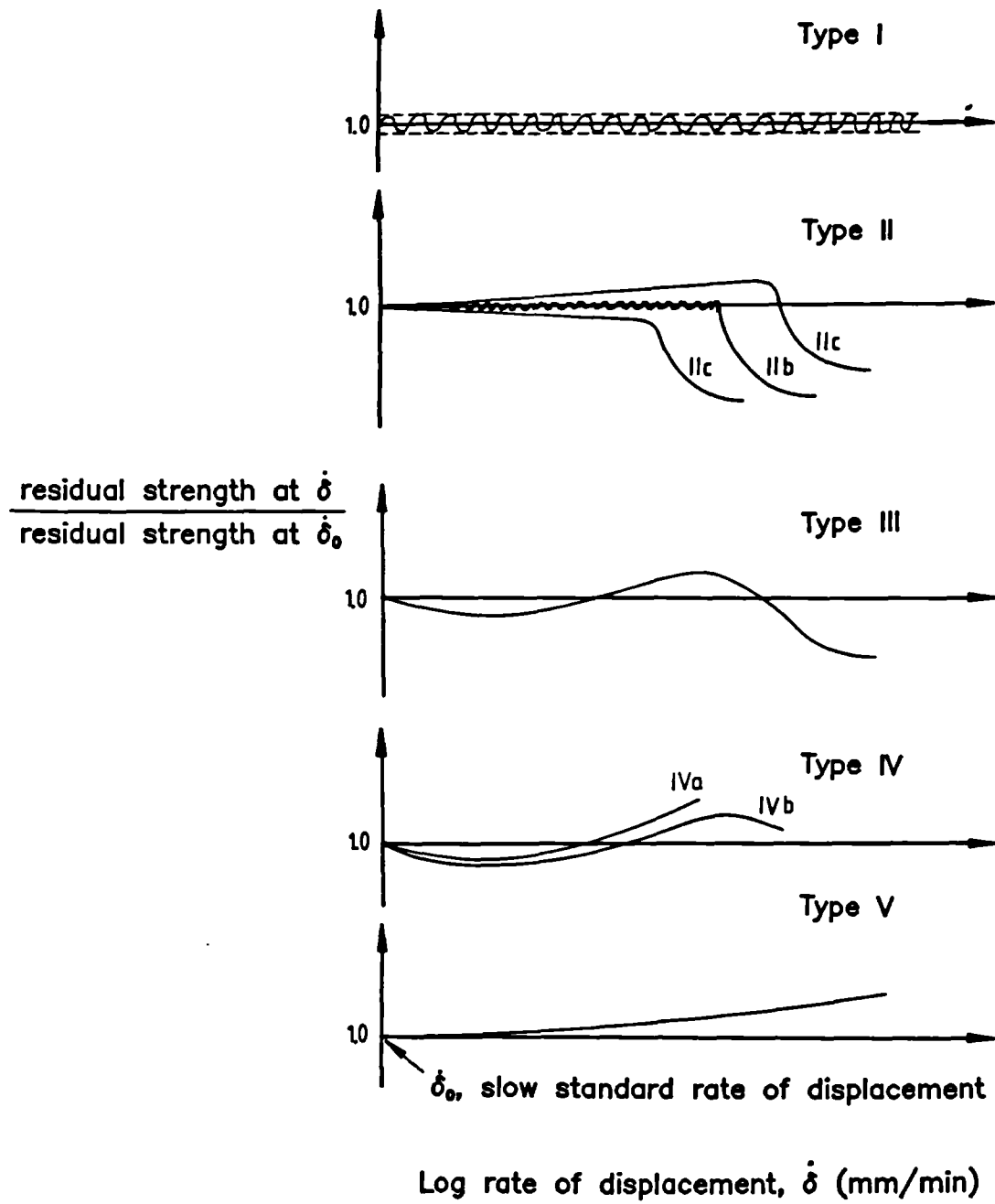


Fig. 5.15 Variation of fast residual strength with an increasing rate of displacement

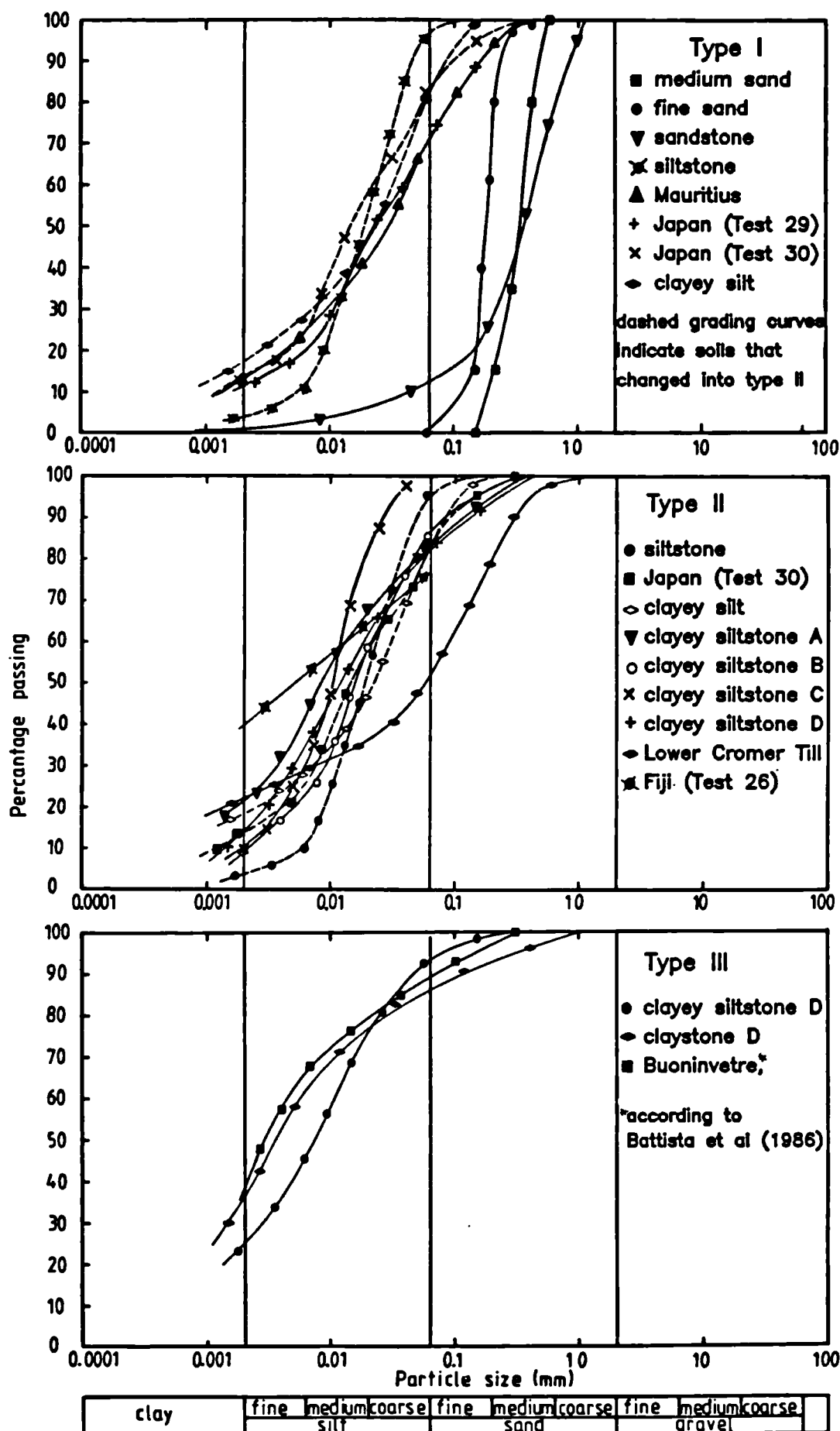


Fig. 5.16a Grading curves of soils showing types I, II and III of behaviour

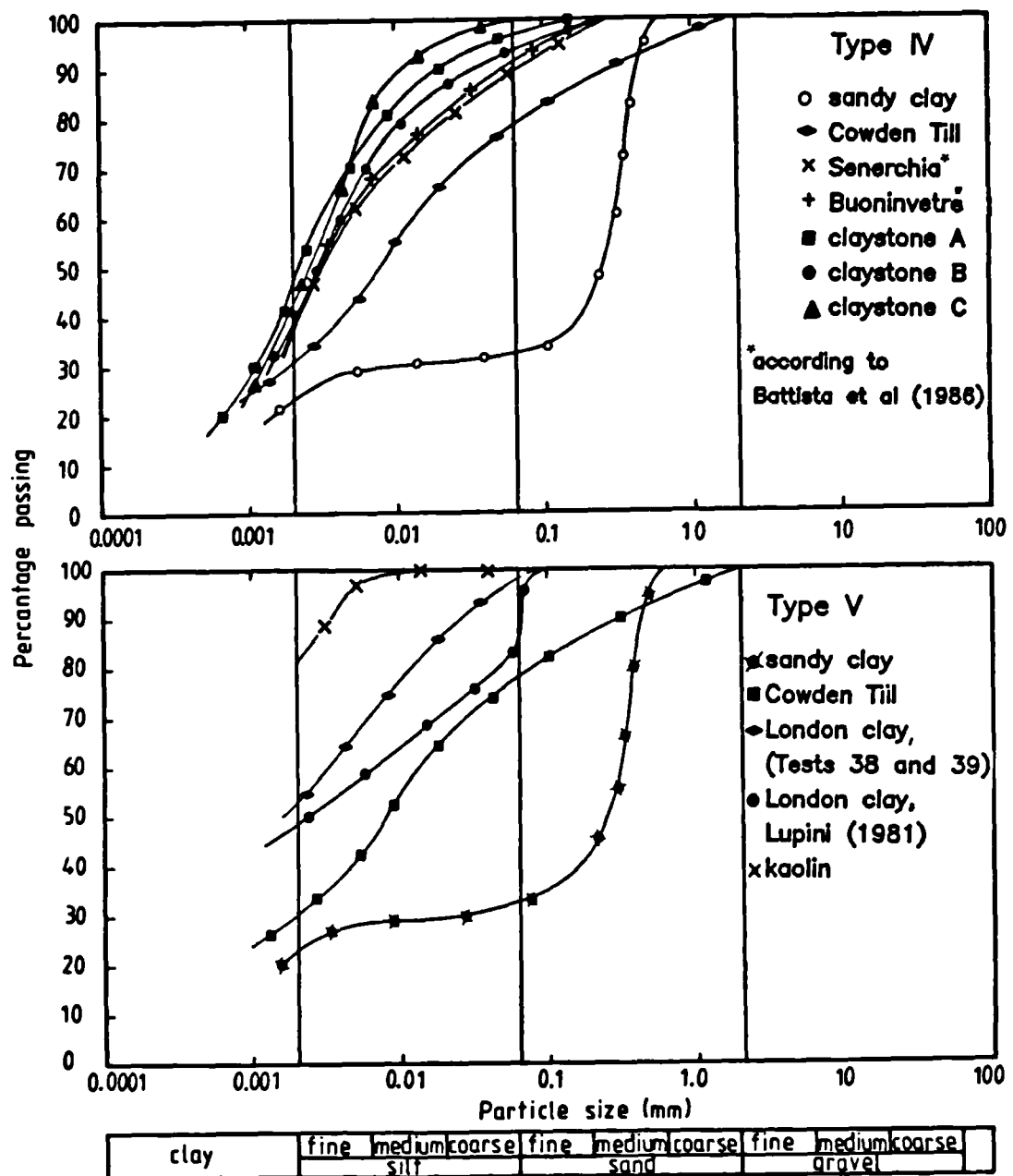


Fig. 5.16b Grading curves of soils showing types IV and V of behaviour

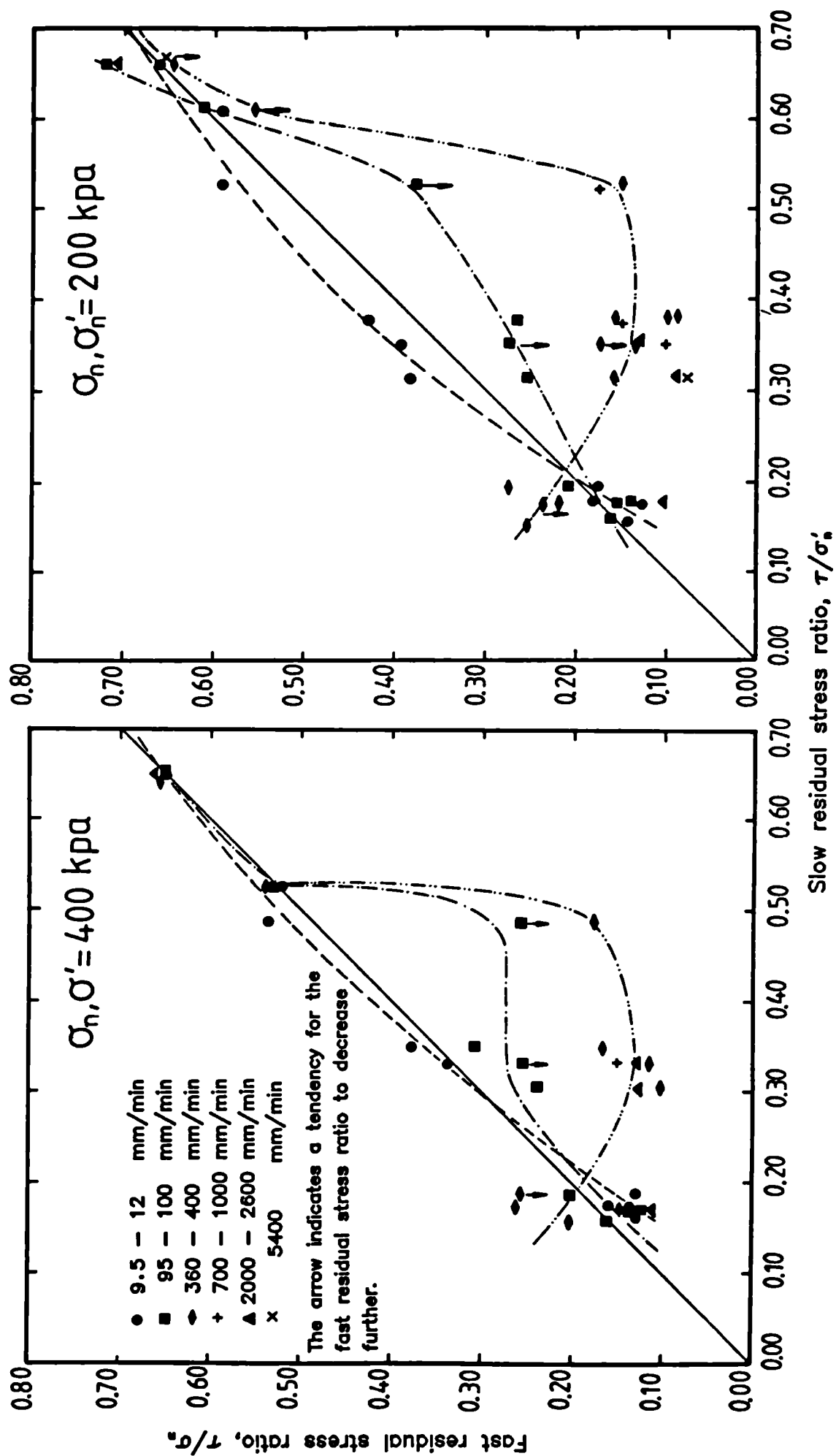


Fig. 5.17 Fast residual stress ratio of soils from Kalabagh dam site against slow residual stress ratio

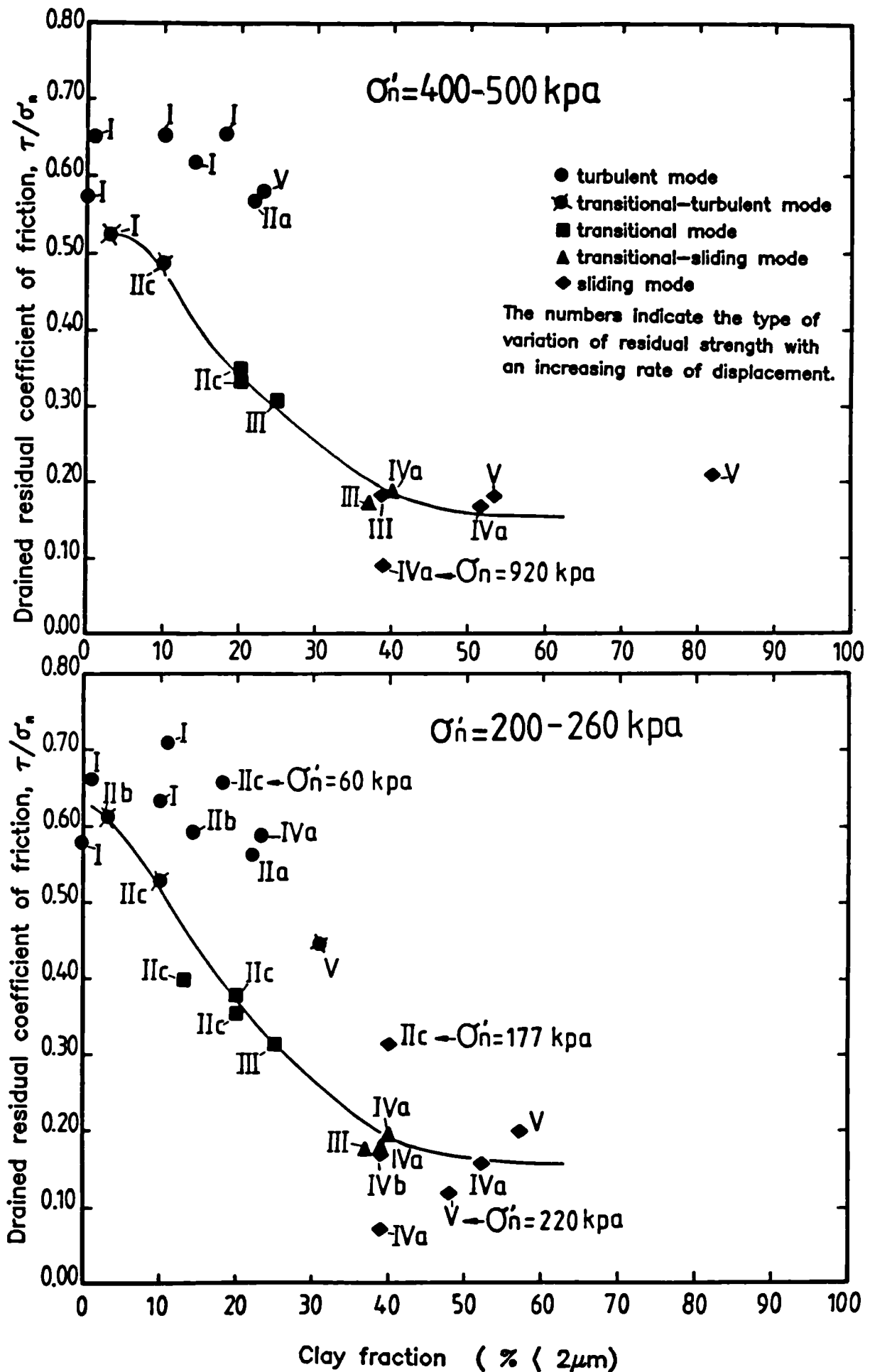


Fig. 5.18 Types of soil behaviour against clay fraction

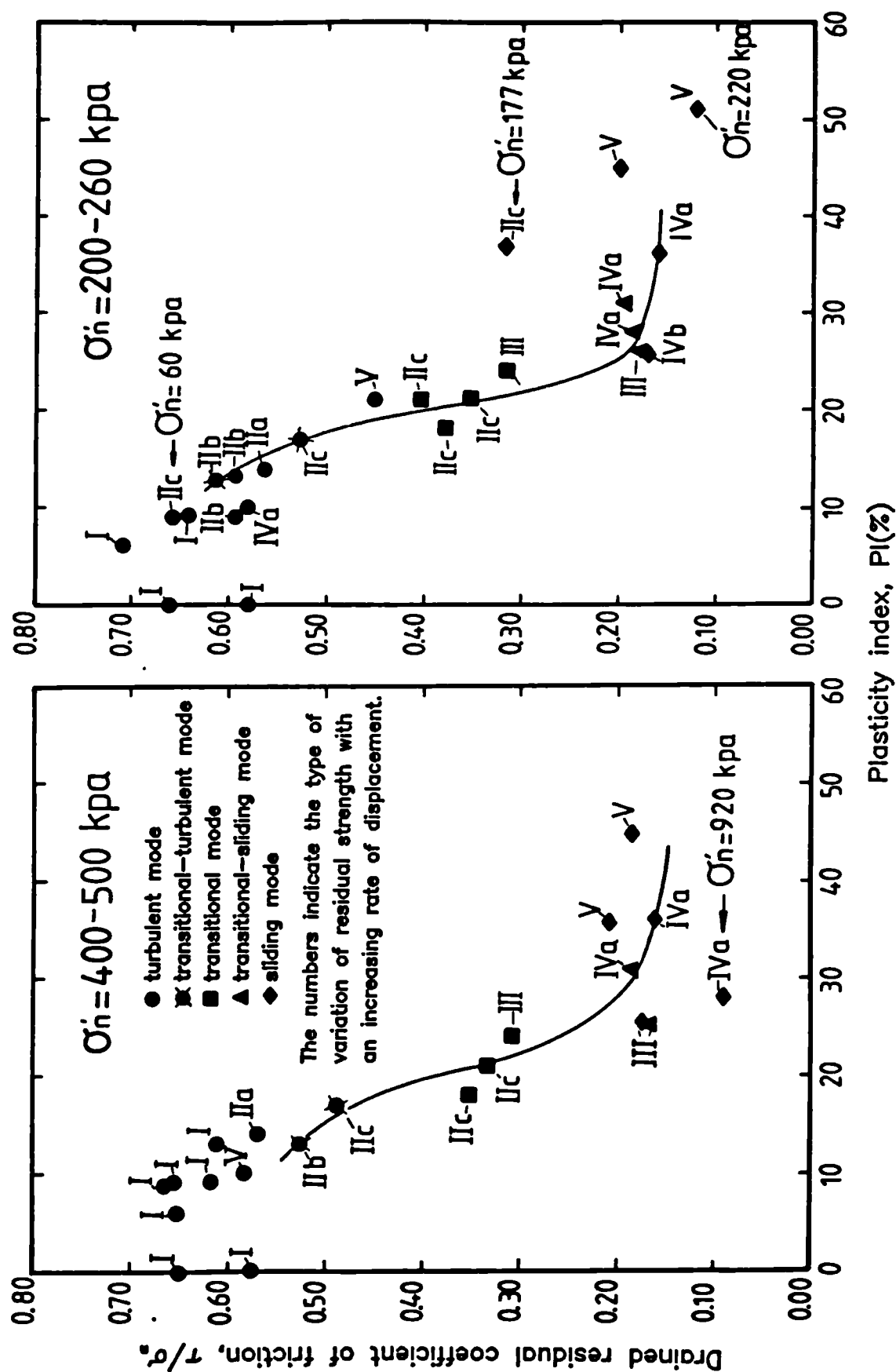


Fig. 5.19 Types of soil behaviour against plasticity index

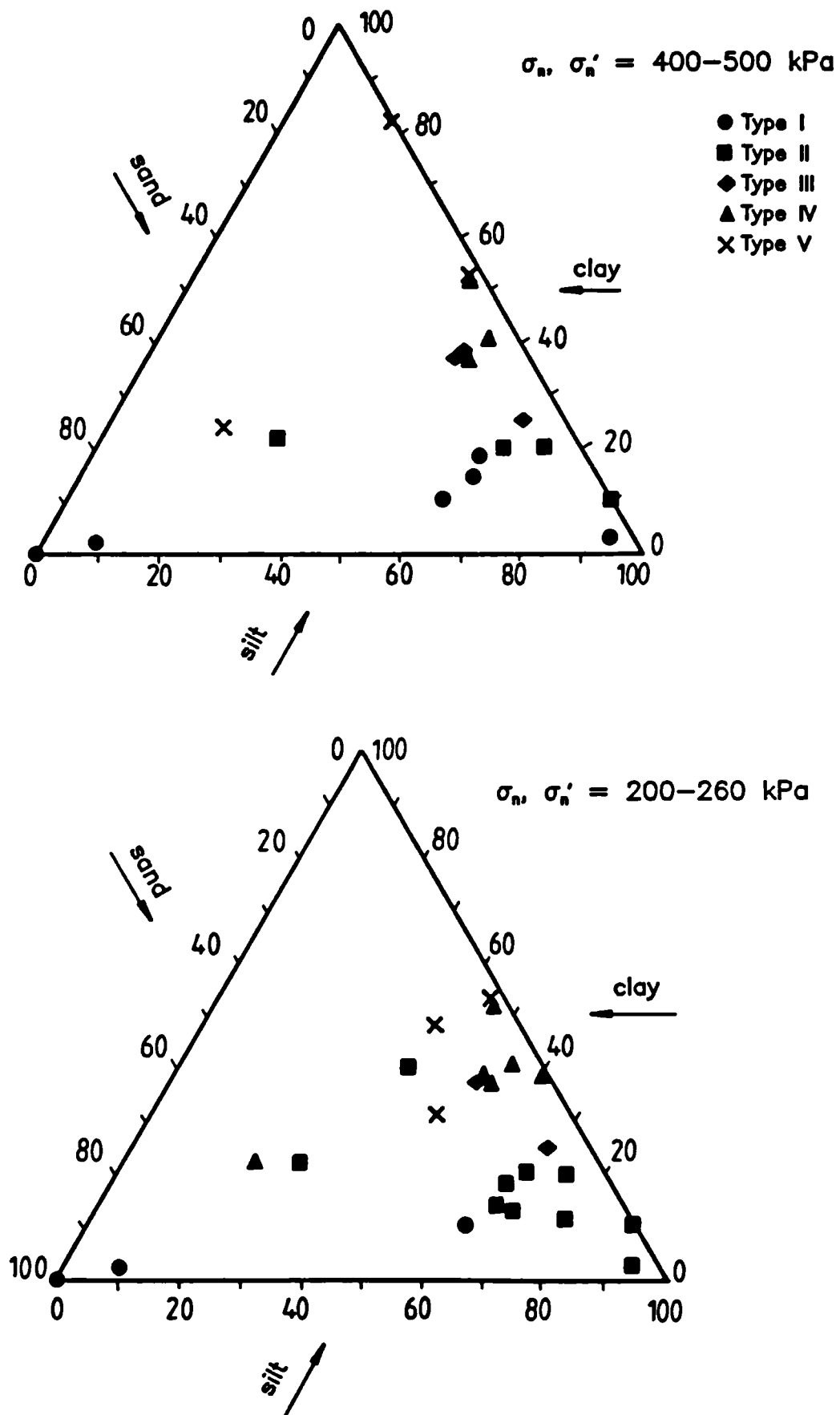


Fig. 5.20 Triangular classification chart of tested soils

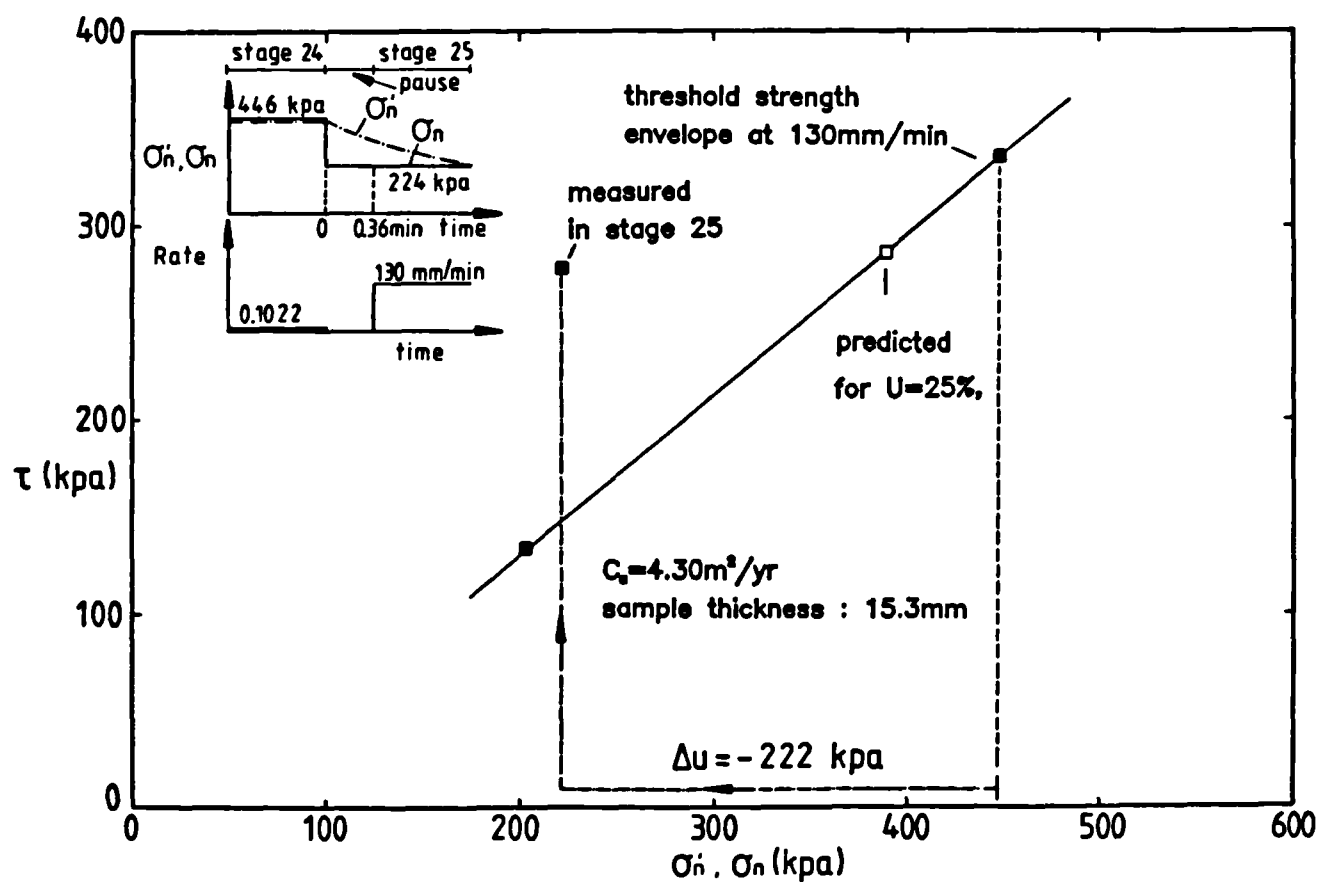
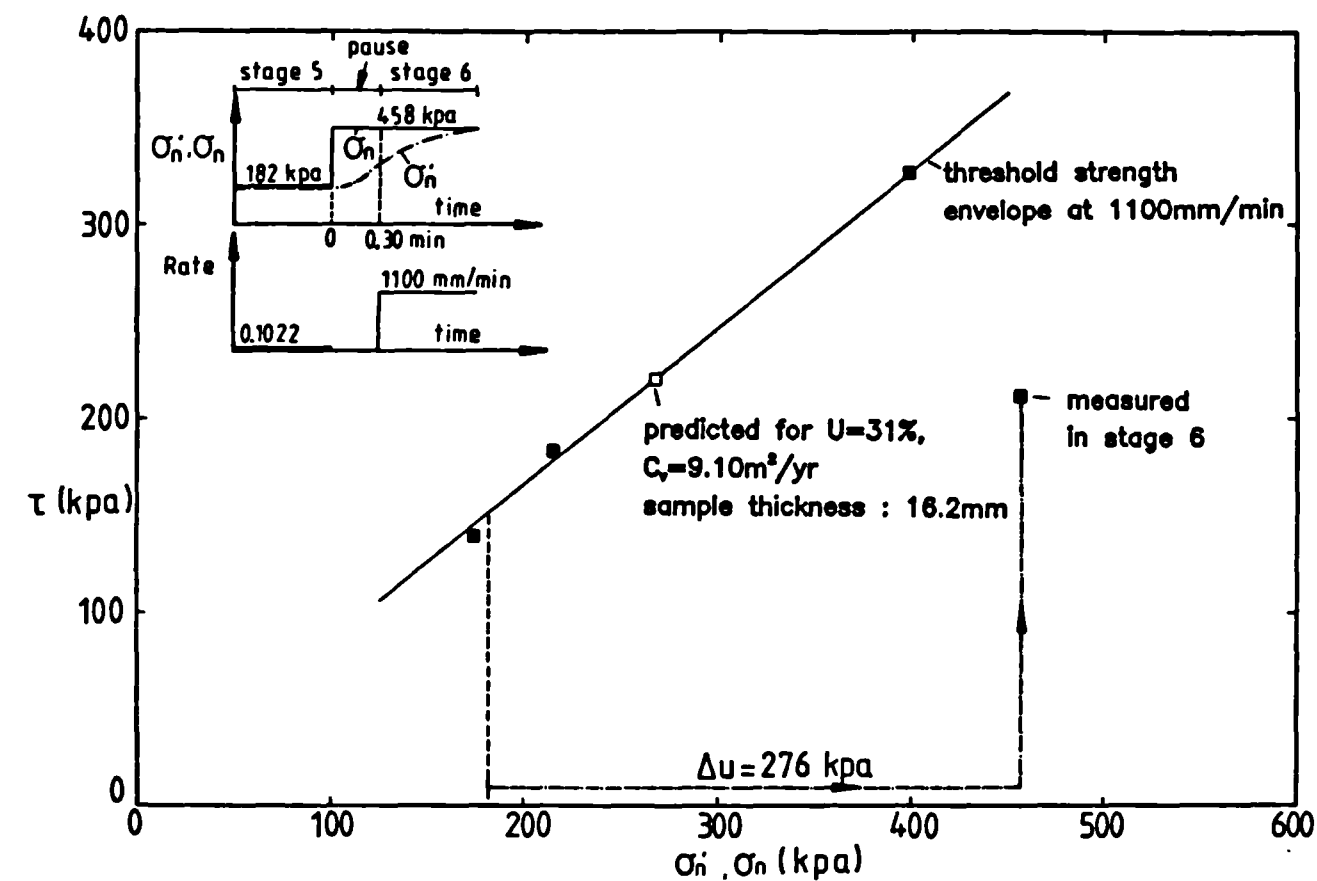


Fig. 5.21. Effect of changing normal effective stress on the threshold strength of sandy clay (Test 36)

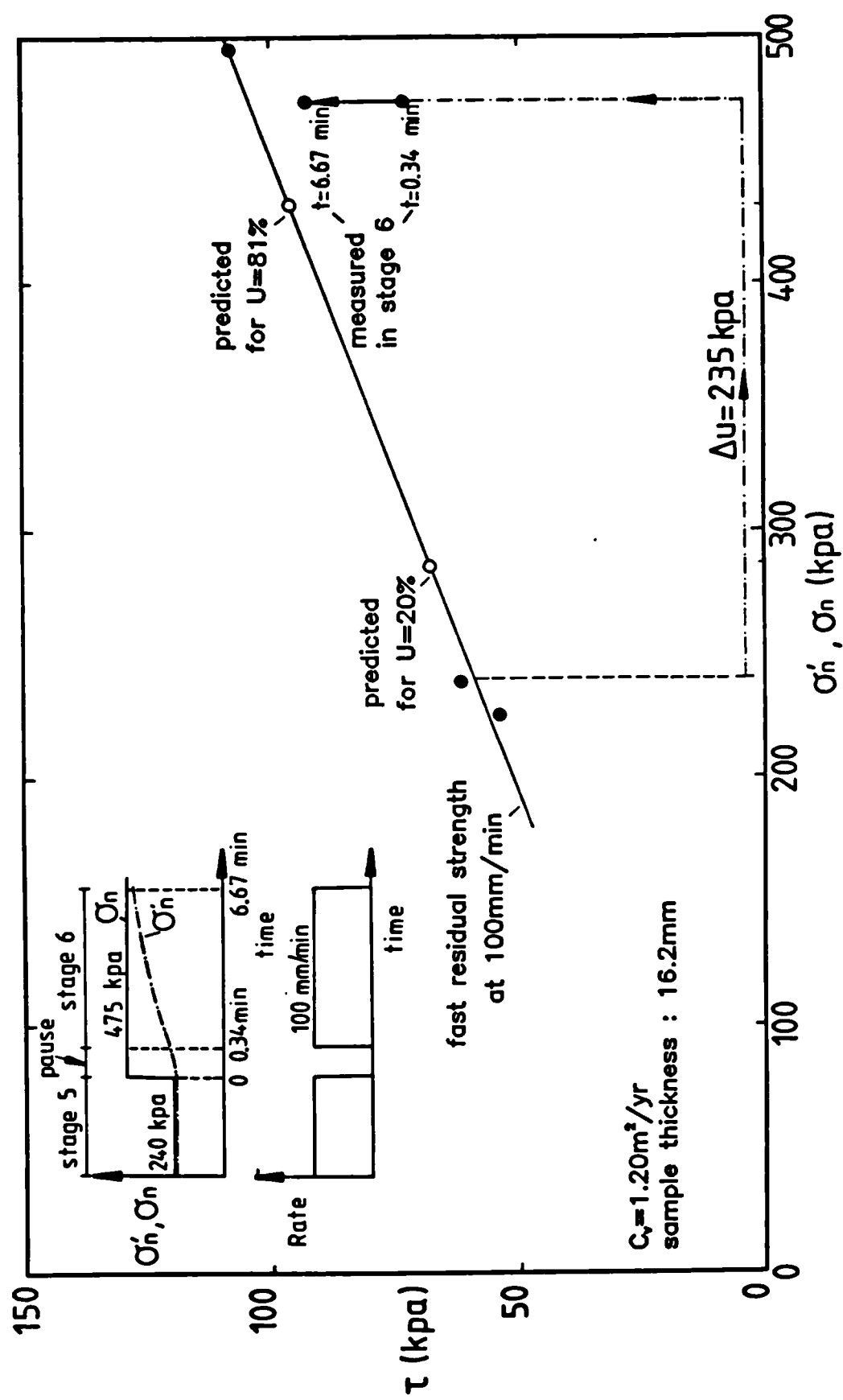


Fig. 5.22 Effect of changing normal effective stress on the fast residual strength of clayey siltstone D (Test 21)

CHAPTER 6

TESTS TO INVESTIGATE THE LOSS OF STRENGTH AT FAST RATES OF DISPLACEMENT

6.1 INTRODUCTION

As mentioned in chapter 2, a number of hypotheses have been advanced to account for the loss of strength at fast rates of displacement, observed in the field and in the laboratory. In the course of this chapter, the results from tests, carried-out to investigate the factors involved, are discussed. These factors include pore water pressure, generated due either to shearing or to frictional heating in the shear zones, and rheology effects.

6.2 PORE WATER PRESSURE GENERATED DUE TO SHEARING

Most of the soils which showed a loss of strength during fast shearing were placed at low water contents, so that they should have been dilatant during drained shear and significant excess pore water pressures could not have been generated. To investigate the role of pore water pressure during fast shearing both direct and indirect procedures were used.

6.2.1 Indirect procedures

These involved testing procedures B, C and D, referred to in section 4.3.

In testing procedure B, when a fast stage was followed by a second fast stage, after a delay sufficient for partial or full consolidation, the second stage showed behaviour similar to the first, with a peak strength followed by a drop to the same fast residual strength (Fig.6.1). Thus the low strength is lost when consolidation without shearing is allowed, although it is recovered with further fast shearing. However, the peak strength in the second stage is lower, probably indicating the difference in the structure of the shear zones, produced by fast and slow shearing. When fast shearing was stopped and restarted immediately, the low strength was recovered immediately (Fig.6.2).

In testing procedure C during which shearing at a slow rate of displacement was commenced immediately after a fast stage, the strength, mobilized initially, was similar to the low strength operating at the end of the fast stage. It recovered then with time to the residual strength corresponding to the particular slow rate of displacement. The time of recovery was similar to the consolidation time of the soil, measured independently from the settlement observations (Fig.6.3). This behaviour indicates that the low strength, created by the fast shearing, is not dependent on it, and that the recovery in strength occurs as if it was due to dissipation of excess pore water pressure.

In testing procedure D, the sample was sheared at a fast rate of displacement for sufficient time for consolidation to occur. Prolonged shearing was performed preferably in tests involving shearing against a rough glass interface in order to minimize the loss of soil through the gap. As stated previously in section 5.4, the behaviour of the clayey siltstones A, B and D, when fast sheared against a rough glass-interface, was similar to that observed in the soil-on-soil tests. The low strength persisted throughout the prolonged fast stages (Fig.A2.55 and 6.9b). The strength did not increase, as it should have done, had the loss of strength been due to an excess pore water pressure. During a series of successive fast stages with a short pause for partial consolidation (40-50%) allowed between them and a total duration comparable to the consolidation time of the soil, the drop of strength to less than the slow residual persisted throughout the series of the fast stages (Fig.A2.56c and A2.56d).

6.2.2 Direct procedures

These involved the measurement of the pore water pressure in the shear zones, in tests involving soil shearing against steel or glass interfaces. The tests were performed on the clayey siltstone D and the claystone D.

Because of the difficulties, associated with the cutting and handling of glass, the first tests involved shearing of the clayey siltstone D against a steel-interface. To assess the soil behaviour, when sheared against a rough steel-interface, Test 50 was carried out prior to pore water pressure measurements. The results from Test 50 which are

presented in Fig.4.12d showed that the behaviour of the soil was qualitatively similar to that observed in the soil-on-soil tests.

Installation details of the pore water pressure measuring system, used in these first measurements, are given in Fig.6.4. Miniature pore water pressure transducers (Druck) were used for the measurements. The application of this type of instrument in triaxial tests on saturated soils has been discussed by Hight (1982, 1983), (Table A1.3). Because of the operating difficulties, experienced by previous investigators (Hight, 1983, De-Campos, 1982 and Lemos, 1986), due to the interference between the back of the porous stone of the probe and its silicon diaphragm, the porous stones fitted to the probes were removed. The probes were seated into screws underneath sintered porous stones (Table 6.1).

De-airing of the porous stones was accomplished by pressurizing both sides at 500 kPa, flushing water through them under a pressure gradient of 250 kPa and applying a pressure gradient across the porous element, so that the air could be removed to one side and then flushed away, for a period of about ten days. The piezometer probes were kept submerged in water inside a vacuum dessiccator during this period. The instrumented interface, the piezometer probes, the rotating plate of the ring shear apparatus and the confining rings were assembled together under water. The whole assembly was then transferred on to the ring shear apparatus where the sample was set up and then consolidated.

During this process, however, the suction in the sample caused desaturation of the porous stones and cavitation in the cavity behind the porous stones. Diffusion might have also induced transference of air to the cavity at the back of the porous stone, as a result of the relatively long duration of the tests and the low pore water pressures involved. This increased greatly the compliance of the measuring system and destroyed the accuracy of the pore water pressure measurements. Slight negative pore water pressures were measured during the fast shearing of clayey siltstone D at a normal stress of 200 to 250 kPa, even when a drop of strength to less than the slow residual occurred. The time these negative pore water pressures needed to attain equilibrium was in excess of 1 day, indicating a significant time lag in the response of the measuring system and the

non-reliability of the measurements.

Because of the problems encountered, it was then decided to equip the pore water pressure measuring system with a water flushing facility (Fig.6.5). The ability to flush water past the back of the porous stones could ensure that there was no air present in the system, so that the measurement of pore water pressure during fast shearing could be regarded as reliable.

In the Tests 51 and 52 performed on clayey siltstone D and claystone D respectively, the piezometer probes were installed in glass-interfaces in order to speed up the test duration and measure the temperature changes in the shear zones during fast shearing. The results, obtained from the previous tests in which pore water pressure measurements had been made, indicated that the displacements required to reach the residual state in the soil-steel tests were larger than those required in the soil-on-soil and soil-glass tests, and that faster rates of displacement had to be applied for the loss of strength to occur.

De-airing of the system was accomplished by using the layout, shown in Fig.6.6. The whole system was pressurized under a pressure of 500 kPa for a period of at least one week by using the clamps shown in Fig. 6.6. A small pressure gradient (of the order of 60 kPa) was allowed from time to time across the porous elements, so that the air could be removed to one side and then flushed away. After the de-airing of the system, the clamps were removed and the whole assembly was transferred onto the ring shear apparatus. During this process, water was flushed continuously through the porous stones. The porous stones were covered with polythene pads prior to and during setting-up the sample and consolidation in order to prevent desaturation during the consolidation stage. The polythene pads were sheared away during the shearing stage of the tests.

The results from the pore water pressure measurements during fast shearing are given in figures 6.7 and 6.8 for clayey siltstone D and in Fig.6.9 and 6.10 for claystone D.

The hydraulic response of the system was checked prior to performing a fast stage by increasing the pore water pressure through the air-water interface. A response time of less than 1 sec was observed prior to

all fast stages. The response time of the pore water pressure measuring system with regard to the tested soils was estimated to be of the order of 1.2 to 7.9 sec, assuming that the soils were fully saturated and no air was present in the system (Appendix 4). The response of the system to an increase of the normal stress on the sample was 25 min and 55 min in Tests 51 and 52 respectively. The mean sample thickness was 9.80 mm in Test 51 and 12.50 mm in Test 52. Assuming that the shear zone was formed 1 mm away from the interface, the response of the system during shearing was estimated to be 1 min and 1.40 min in Tests 51 and 52 respectively. Visual observations of the samples in Tests 48 and 50, carried out on clayey siltstone D, and involving soil shearing against glass and steel interfaces respectively, had showed that the shear zone was formed at a distance of less than 0.5 mm away from the interface. The response of the system was checked by flushing de-aired water through it either during fast shearing (Fig. 6.9a) or immediately after the fast shearing (Fig. 6.8a to 6.8c and 6.10b to 6.10c) and monitoring its performance immediately afterwards. Similar readings with those prior to water flushing were established. The fast stages in Tests 51 and 52 lasted from 1.655 to 5.402 min and 2.906 to 25.085 min respectively and thus the pore water pressure measurements can be considered reliable.

Although a steady state had been reached during the fast stages in Test 51 and the fast stages 2, 4 and 6 in Test 52, the measured pore water pressure showed a tendency to increase further (Fig. 6.7 and 6.9a). Thus the results cannot be considered conclusive with regard to the role of pore water pressure during fast shearing. The results from fast stage 9 in Test 51 (Fig. 6.9b and 6.9c), however, which was prolonged for a period of 25 min approximately and during which the measured pore water pressures had reached a stable state, indicated that the excess pore pressures were small, yet the strength dropped to half the slow residual strength. An estimation of the positive pore water pressure, required to explain the loss of strength during fast shearing, can be made on the basis of the conventional relationship between strength and normal effective stress:

$$\Delta U = \sigma_n \left(1 - \frac{\tan \phi}{\tan \phi_r} \right) \quad (6.1)$$

Where ΔU is the pore water pressure generated during the fast shearing,

σ_n is the net normal stress acting on the sample,
 $\tan\phi$ is the apparent coefficient of friction operating during the fast shearing, and
 $\tan\phi'_r$ is the drained residual coefficient of friction.

The pore water pressures, predicted by using equation 6.1, are summarized in Table 6.2. The data, presented in Table 6.2, indicate that the measured pore water pressures during fast shearing accounted for 28% of the observed loss in strength for claystone D.

When fast shearing was stopped, an increase in pore water pressure was always observed (Fig.6.8 and 6.10). These excess pore water pressures lagged considerably behind the recorded settlements. This time lag in the measurement of pore water pressures after fast shearing may be associated with the increased thickness of the shear zone during fast shearing, suggested from the observations of the microstructure of the clayey siltstone^D (Plate 4). The measured pore water pressure, measured after fast shearing, if it developed during it, could have accounted for the 7 to 23% of the loss of strength for the clayey siltstone D and 30 to 57% for the claystone D. Lemos (1986) had found that the excess pore water pressure, generated after fast shearing, could have accounted for up to the 66% of the loss of strength, observed for clayey siltstone B (Table 6.2).

The finding that the pore water pressures, measured during fast shearing, cannot account for the observed loss of strength, does not contradict the principle of the effective stress, used in conventional soil mechanics for rate-independent phenomena, as this principle might not be applicable in the case of the rate-dependent behaviour, observed here.

6.3 PORE WATER PRESSURE GENERATED DUE TO FRICTIONAL HEATING

Frictional heating, generated during fast shearing along a shear surface within a soil mass, can induce an increase of pore water pressure, and consequent decrease of shear strength. The main causes of a pore water pressure increase, resulted by a corresponding temperature rise, are the difference between the coefficients of thermal expansion of water and soil grains, and the decrease of the shear strength of the individual particle contacts.

Considering the relationship between volume change under drained conditions and a pore water pressure change under undrained conditions of constant total stress and with varying temperature, the following expression for the pore water pressure rise can be derived (Appendix 5):

$$\Delta U = \frac{-n \cdot \Delta\theta (a_w - a_s) + a_{st} \cdot \Delta\theta}{m_v + n \cdot m_w} \quad (6.2)$$

where n is the porosity of soil,

$\Delta\theta$ ($^{\circ}\text{C}$) is the temperature increase,

a_w ($/^{\circ}\text{C}$) and a_s ($/^{\circ}\text{C}$) are the thermal coefficients of cubical expansion of water and soil grains respectively which are both temperature and confining pressure dependent,

a_{st} ($/^{\circ}\text{C}$) is a physical-chemical coefficient of structural volume change, caused by a change in temperature,

m_v ($/\text{kPa}$) and m_w ($/\text{kPa}$) are the coefficients of volume compressibility of the soil and the pore water respectively.

The above equation, in its simplest form, suggests that the main factors controlling the pore water pressure increase are the change in pore water volume and the soil compressibility.

To investigate the effect of frictional heating on the loss of strength during fast shearing, the changes in temperature in the shear zones of clayey siltstone D and claystone D were monitored by means of a thermocouple installed in a glass interface (Fig.6.5). Glass was chosen because of its small thermal conductivity [i.e compare the thermal conductivities 10, 10 and 150 Joule/m,sec, $^{\circ}\text{C}$ of soil, glass and stainless steel respectively].

The results from Tests 50 and 51 on clayey siltstone D and claystone D respectively in which the temperature changes during fast shearing were monitored are shown in Fig.6.7 and 6.9. Lemos (1986) had measured the temperature changes during fast shearing in the shear zone of the clayey siltstone B.

Table 6.3 summarizes all the temperature measurements and the pore water pressure rises due to the observed temperature increases, predicted by using equation 6.2 and assuming $a_w = 2.5 \times 10^{-4}/^{\circ}\text{C}$ (Clark,

1966), $\alpha_s = 0.38 \times 10^{-4}/^{\circ}\text{C}$ (for quartz particles, Lange, 1966) and $m_w = 4.7 \times 10^{-7}/\text{kPa}$ (Lange, 1966). The physical-chemical coefficient of structural volume change, α_{st} , was neglected, as being small, of the order of $-0.5 \times 10^{-4}/^{\circ}\text{C}$ according to Campanella and Mitchell (1968).

The results, shown in Table 6.3, indicate that the reduction of shear strength due to frictional heating cannot account for the significant loss of strength, observed for these soils during fast shearing. The above predictions are likely to have overestimated the actual pore water pressure, as they had been based on the maximum observed temperature rise, and as the conduction of heat away from the shear surfaces and the temperature induced structural volume changes were not considered.

6.4 RHEOLOGY EFFECTS

Having found that pore water pressure changes due to either shearing or temperature rises could not account for the loss of strength at fast rates of displacements, it was decided to look into the rheological behaviour of the soils involved.

Constitutive relationships for the rheology of flowing materials include the Newtonian, the visco-plastic (Bingham), the pseudo-plastic and the dilatant behaviours. Materials with Newtonian behaviour show a linearly increasing shear stress with increasing shear rate; their viscosity is constant and independent of shear rate. Visco-plastic (Bingham) materials show a Newtonian behaviour after they have reached a yield shear stress. For the pseudo-plastic and dilatant materials there is no constant of proportionality between shear stress and shear rate. Pseudo-plastic and dilatant materials show a decrease and an increase of the apparent viscosity with increasing shear rate respectively.

Fig.6.11 and 6.12 illustrate results from tests on clayey siltstone D and kaolin respectively by means of a cone-and-plate viscometer. The results, indicating a pseudo-plastic type of behaviour for clayey siltstone D and a visco-plastic (Bingham) type of behaviour for kaolin, agree qualitatively with those obtained from the ring shear tests.

All tested soils that showed a loss of strength at fast rates of

displacements had significant granular fractions and low to intermediate clay contents. Consideration of the behaviour of these soils as a flow phenomenon would require a reference to the flow of cohesionless grains in fluids with the fine particles and the water being the fluid medium in this case.

To investigate the effect of fluid viscosity on the shear strength of a mixture of cohesionless grains and fluid during fast shearing, a series of tests on sand mixed with viscous fluids, was carried out. Glycerol and silicon oil with specific gravities of 1.260 and 0.975 respectively were used. The variation of the viscosity of glycerol (non-Newtonian fluid) with shear rate is given in Fig.6.13. Silicon oil (Newtonian fluid) had a viscosity of 12,187 cp. A comparison of the viscosities of the fluids used with those of soils can be made by referring to Table 6.4 which summarizes the reported values of viscosities of soils in the literature. The tests were carried-out on medium and fine sand, identical to those used in Tests 43 and 44 to 47 respectively. The results from the tests, illustrated in Fig 6.14, showed the following:

- (i) the loss of strength during fast shearing, observed for the mixture of fine sand and glycerol at granular void ratios ranging from 0.935 to 1.059, increased with decreasing normal stress level in accordance with the behaviour observed for the soils.
- (ii) an increase in the viscosity of the fluid medium (from glycerol to silicon oil) resulted in a change of the rate effect on the behaviour of the mixture from negative to positive due probably to an increase in the viscous effects (Fig.6.14).
- (iii) The water content of kaolin in the artificial sandy clay, used in Test 37, was estimated to be 63% approximately. Hence its viscosity was very close to that of silicon oil (Table 6.4). Similar behaviours were observed at similar rates of displacement in the tests performed on the sandy clay and the mixture of medium sand and silicon oil, at a granular void ratio of 0.793 and 0.805 respectively (Fig. 4.40 and 6.14).
- (iv) In the tests performed on the mixtures of medium or fine sand with silicon oil and on the sandy clay, a significant relaxation

of the shear strength was observed, as soon as fast shearing stopped, caused probably by the removal of the viscous component of the strength, operating during fast shearing. The remaining part of the shear strength was close to the fast residual strength, observed for the mixture of medium or fine sand with glycerol in which a loss of strength during the fast stages and no relaxation of the shear strength immediately after the fast shearing were observed. A significant relaxation of the shear strength immediately after the fast shearing was observed also in the tests on the residual soils from Mauritius (prior to degradation) and Japan (Test 29) and the clayey silt (at a normal stress of 450 to 470 kPa).

6.5 DISCUSSION OF THE LOSS OF STRENGTH AT FAST RATES OF DISPLACEMENT

Consideration of all the observations on the behaviour of soils, showing a loss of strength at fast rates of displacement, suggests that the following mechanism may apply.

The soils which showed a significant loss of strength during fast shearing exhibited behaviour of types II and III. Types II and III indicated that the loss of strength occurred when the rate of displacement was increased beyond a critical level.

Fast shearing of these soils involves dilation of massive grains to permit them to pass over one another. This results in an increase of the porosity (or granular void ratio) in the shear zone.

The low strength, reached during fast shearing at a rate of displacement above the critical level, is a steady state without significant excess average pore water pressure due to shearing or frictional heating. It is caused by a reduction in the viscosity of the soils and thus an increase in the ability to flow. Whenever the fast shearing is stopped or slowed down, the shear zone attempts to contract and excess pore water pressure is generated. The low strength in the shear zone, while it results from fast shearing, is not significantly rate dependent. It persists even during slow shearing following a fast stage until the excess pore water pressure dissipates.

Evidence from the experimental results for an increase in granular void

ratio at increasing rates of displacement is limited, owing to the difficulty in measuring accurate volume changes in the ring shear apparatus (Fig.5.1 and A2.18). Observations of the microstructure of the shear zone of clayey siltstone D, formed by fast shearing, indicated that the shear zone had a higher void ratio and was considerably thicker than that formed by slow shearing (Plate 4). Such an increase in void ratio during fast shearing of soils has been reported by Whitman and Healy (1962), Richardson (1961) and Scarlett and Todd (1969).

The transformation of the initial stable structure of these soils, showing types II and III of behaviour, into a loose state and the consequent ability to flow more easily, when the rate of displacement is increased beyond some critical level, may be analogous to the change in phases, observed in non-particulate substances (solids, liquids and vapours).

Soils with different gradings have different granular void ratios and thus represent different initial phases with different critical levels of rate of displacement. On the evidence of the test results reported here, the loss of strength is associated with a significant silt content, above 30% approximately, and a clay content between 3 and 40%. It does not happen in the absence of clay, or when the clay content is higher than 48% and ^{the} soil shows the sliding shear mode.

Table 6.1 Characteristics of the sintered porous stones

	Bronze	Stainless steel
Grade	A	A
Maximum particle diameter to pass (μm)	2.5	2.5
Diameter (in)	0.5	0.5
Thickness (in)	0.125	0.125
Permeability (cm/sec)	1.76×10^{-3}	1.76×10^{-3}
Air entry value* (kPa)	117	117
Porosity (%)	30	27

* Calculated by considering the equilibrium of a spherical air-bubble ($2t/r$) and assuming $t = 7,288 \times 10^{-2} \text{ N/m}$ (20°C) for the surface tension of water

Table 6.2 Summary of pore water pressure measurements during fast shearing

Soil type	Test	Stage	Rate mm/min	σ_n kPa	$\tan\phi'_r$	$\tan\phi$	ΔU_m kPa	ΔU_r kPa	$\frac{\Delta U_m}{\Delta U_r}$ (%)	ΔU_{ps} kPa	$\frac{\Delta U_{ps}}{\Delta U_r}$ (%)
clayey siltstone B	14	2	800	492	0.274	0.143	-	235	-	155	66
		4	6200	500	>>	0.091	-	334	-	119	36
		5	6200	512	>>	0.055	-	409	-	172	42
clayey siltstone D	51	2	380	135	0.383	0.140	-2.4	86	-3	5.7	7
		4	370	195	0.326	0.165	25.4	96	26	-	-
		7	0.6295	192	0.328	0.290	2.6	22	12	-	-
		9	370	385	0.262	0.247	-	22	-	5.0	23
claystone D	52	2	400	225	0.189	0.100	- 8	106	-7	32	30
		4	370	223	0.189	0.092	-14	114	-12	51.5	45
		6	400	232	0.189	0.082	-10	131	-8	74.5	57
		9	160	490	0.185	0.083	75	270	28	48.0	19

Notes:

 ΔU_m : maximum pore water pressure measured ΔU_r : pore water pressure predicted by using equation (6.1) ΔU_{ps} : pore water pressure measured after the fast shearing

Table 6.3 Summary of temperature measurements during fast shearing

Soil type	Test	Stage	Rate mm/min	σ_n kPa	n (%)	m_v (/kPa) $\times 10^{-5}$	$\Delta\theta^0$	$\Delta U_{\Delta\theta}$ kPa	ΔU_m kPa	$\frac{\Delta U_{\Delta\theta}}{\Delta U_m}$
clayey siltstone B	14	2	800	492	33	1.5 ⁽¹⁾	2.50	11.5	-	-
		5	6200	512		1.5	2.40	11.1	-	-
clayey siltstone D	51	2	380	135	33.5	2.5 ⁽²⁾	0.71	2.0	-2.4	-
		4	370	195		2.1	0.84	2.8	25.4	11.0
		9	370	385		1.6	2.42	10.6	-	-
claystone D		2	400	232	36.2	2.5 ⁽³⁾	0.45	1.4~1.8	-8	-
		4	370	225		>>	0.58	>>	-14	-
		6	400	232		>>	0.59	>>	-10	-
		9	160	499		1.8	0.64	2.7	75	3.6

Notes: $\Delta U_{\Delta\theta}$: pore water pressure predicted by using equation 6.2. ΔU_m : maximum pore water pressure measured.

(1) : assumed as being similar to the volume compressibility of clayey siltstone A for which results from an oedometer test exist (Lemos, 1986).

(2),(3): estimated from the consolidation stage of the ring shear tests.

Table 6.4 Viscosities of soils reported in literature (back-calculated)

Soil type	Viscosity (poise)	Velocity - shear rate	Notes	References
Matrix of flowing debris	1,000		W.C = 23%	Iida (1938)
Matrix of flowing debris	2,000-6,000	1.2-3.0 m/s	unit weight: 2.4 gr/cm ³ W.C : 25-30% solids concentration: 79-85%	Sharp and Nobles (1965)
Matrix of flowing debris	30,000	2.5 m/s	unit weight: 2.53 gr/cm ³ W.C : 10% silt and clay content: 11%	Curry (1966)
Mud >>	23-4,900* 3-890*	6 rpm 60 rpm		Shih (1967)
Quick clay >>	2,000* 500*	0.4 rpm 4 rpm		Kerr and Drew (1968)
Mud-debris	30,000	60 m/s		Daido (1971)
Submarine slides Subaerial quick clays	7,000-14,000 2,000-4,000			Edgers and Karlsrud (1982)
Phosphate tailings	0.0816-8.16	1.65-16.5m/s	unit weight: 1.01-1.26gr/cm ³	Jeypalan, Duncan and Seed (1983)
Other tailings	816-40,800	1.65-6.6 m/s	unit weight: 1.14-1.39gr/cm ³	>>
Kaolin >> >>	1,050* 140* 40*	0-30 sec ⁻¹ 0-47 sec ⁻¹ 0-47 sec ⁻¹	W.C = 47.5%, LI = 0.32 W.C = 58-69%, LI = 0.61-0.92 W.C = 92%, LI = 1.54	This study

* measured viscosity

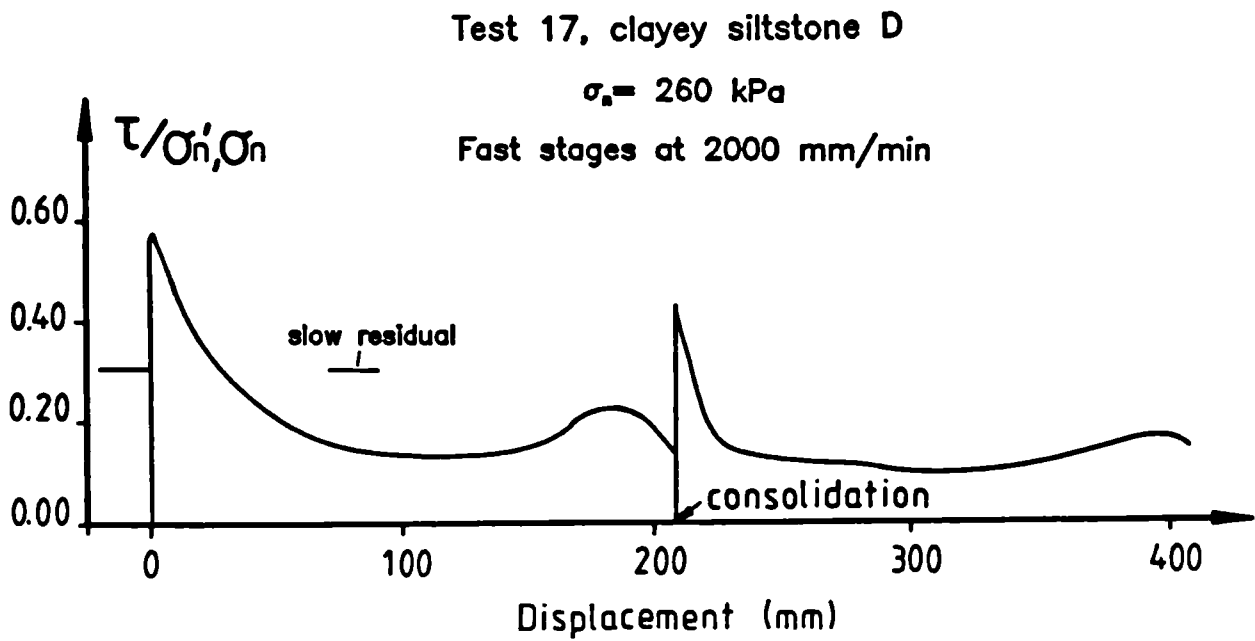


Fig. 6.1 Fast shearing – stopping and restarting after a pause

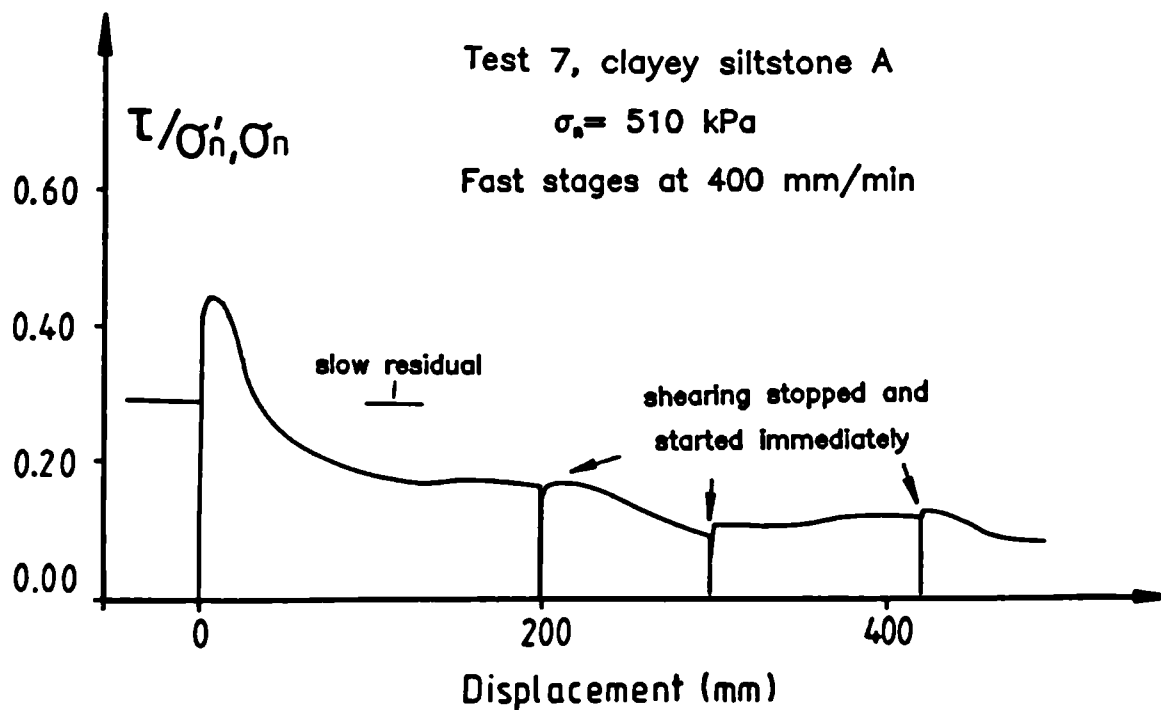


Fig. 6.2 Fast shearing – stopping and restarting immediately

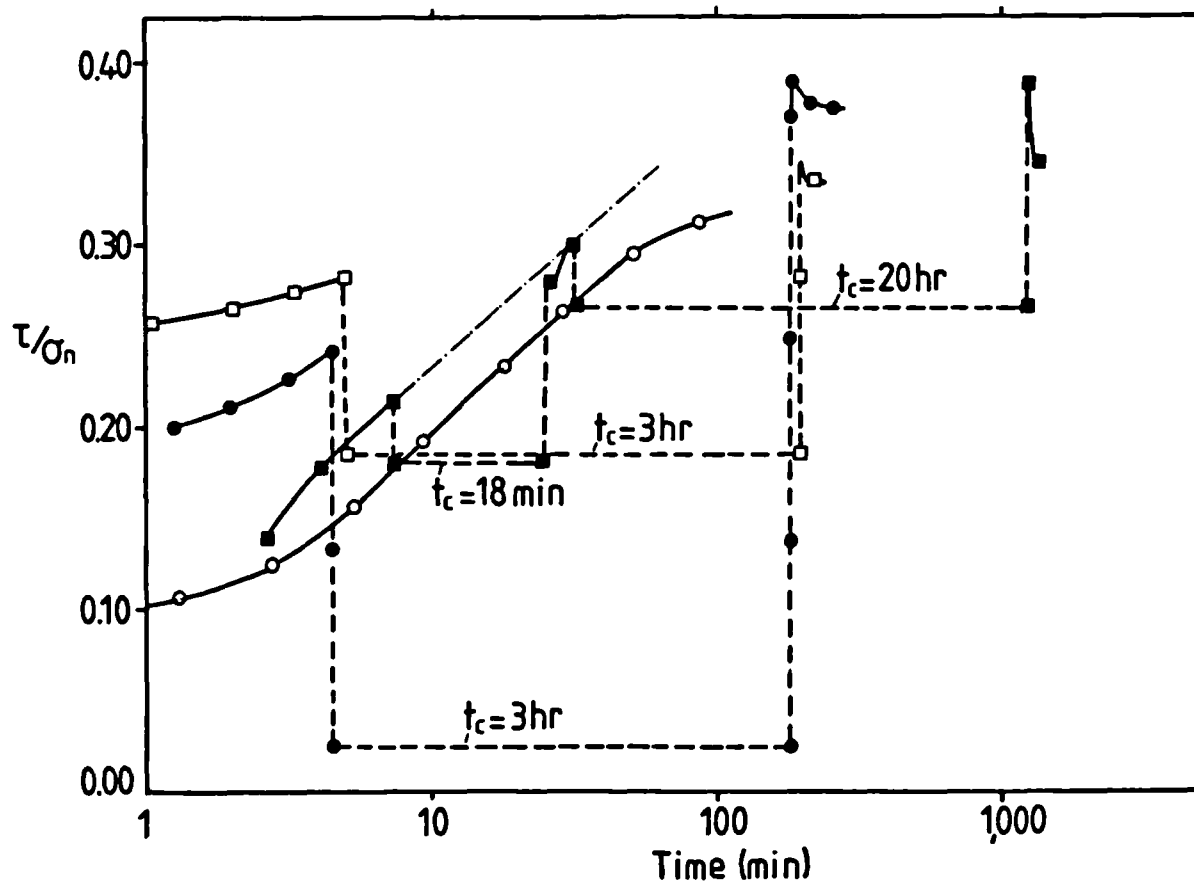
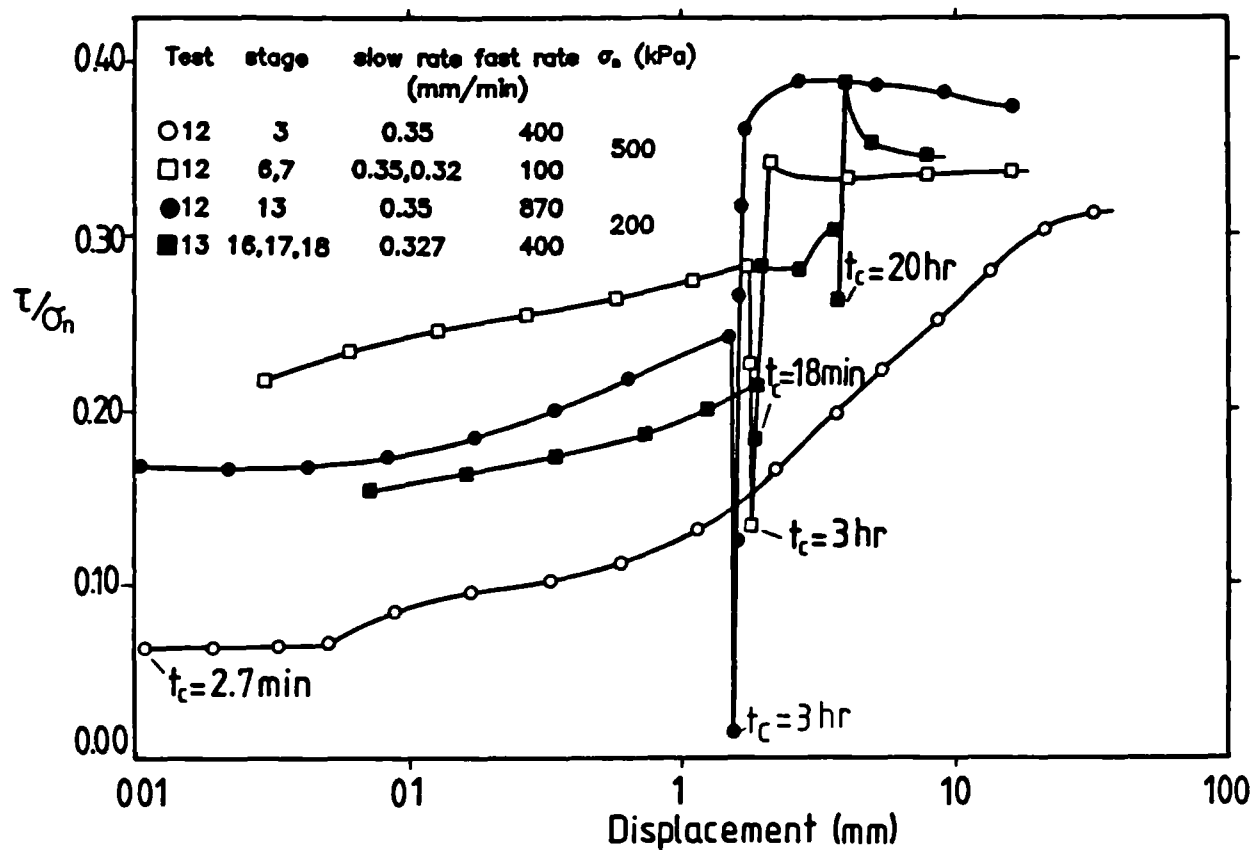


Fig. 6.3a Slow shearing following fast shearing of clayey siltstone B without a pause

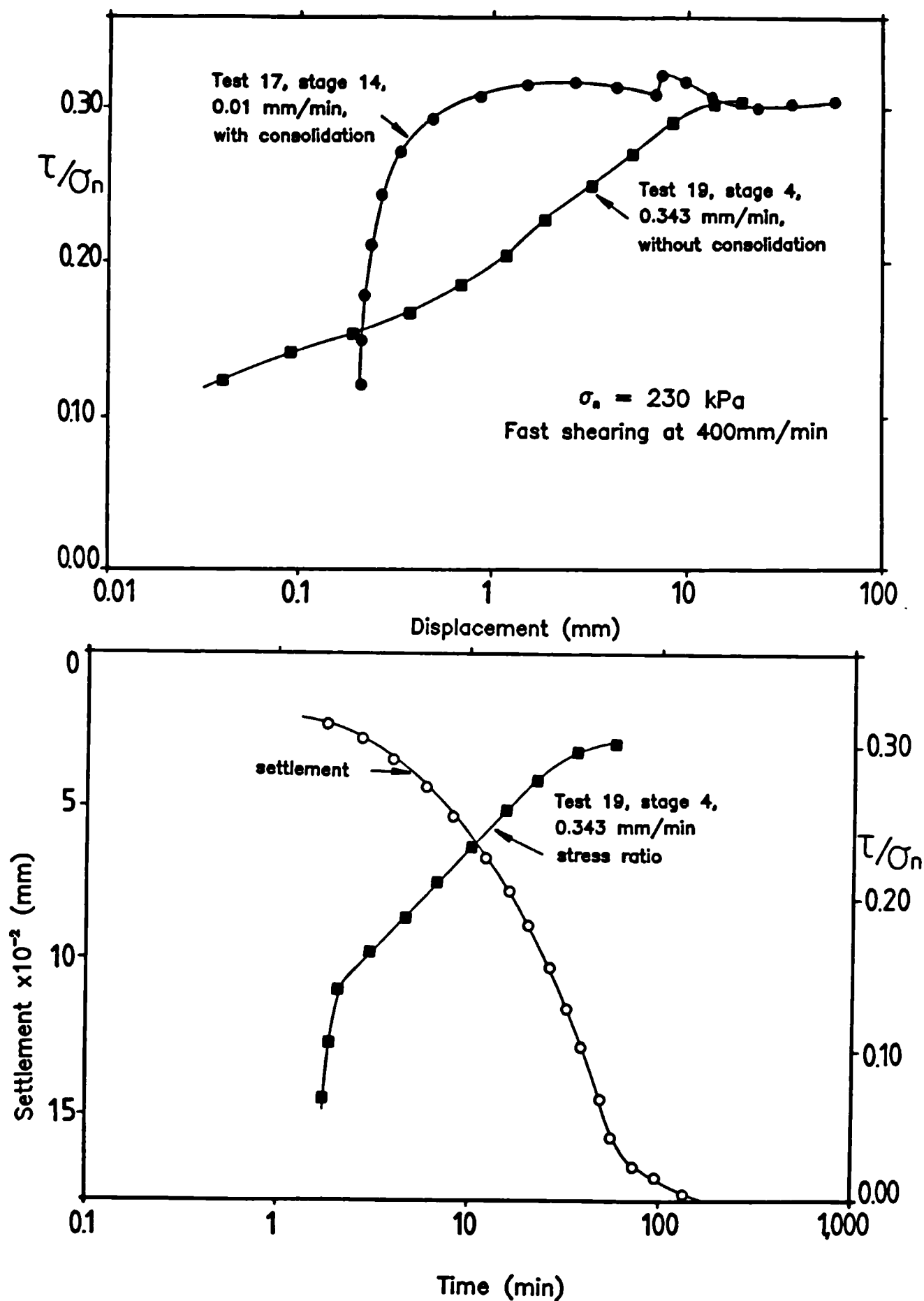


Fig. 6.3b Slow shearing following fast shearing of clayey siltstone D with and without a pause

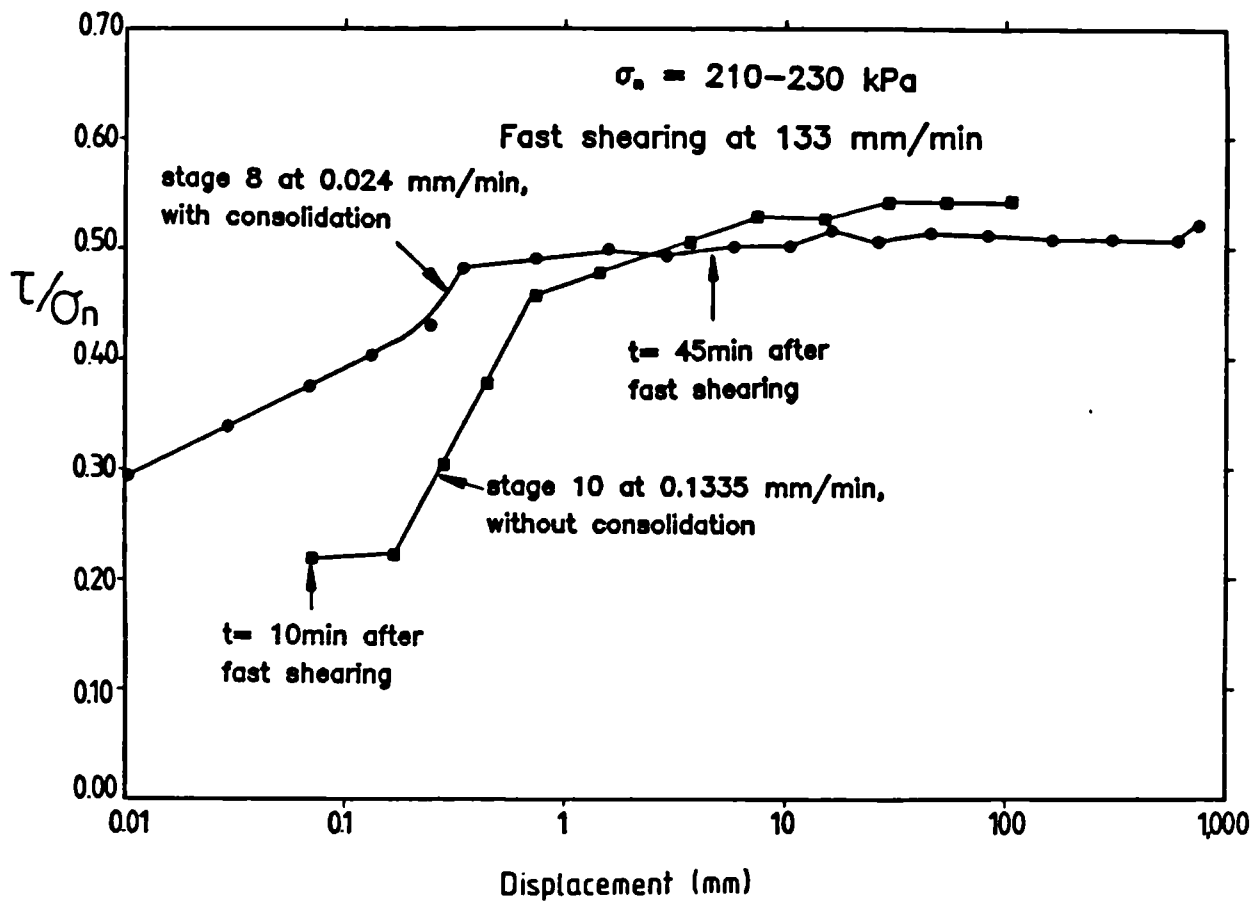


Fig. 6.3c Slow shearing following fast shearing of Lower Cromer Till with and without a pause

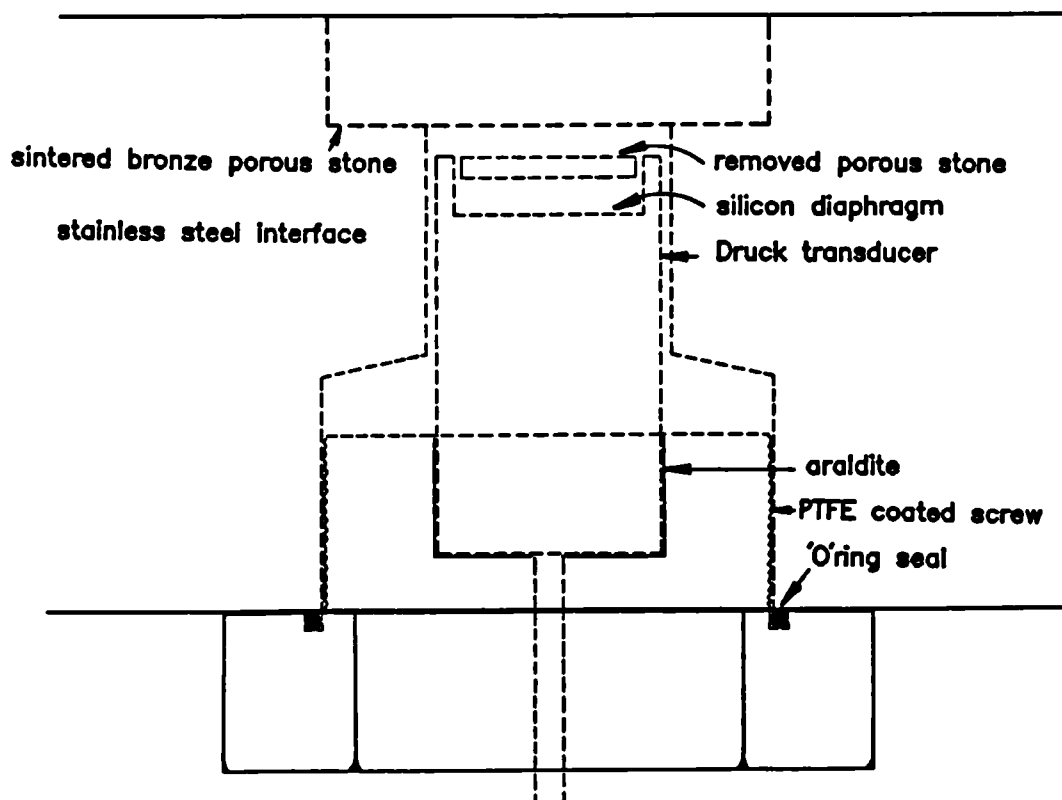


Fig. 6.4 Layout of the pore water pressure measuring system, used initially

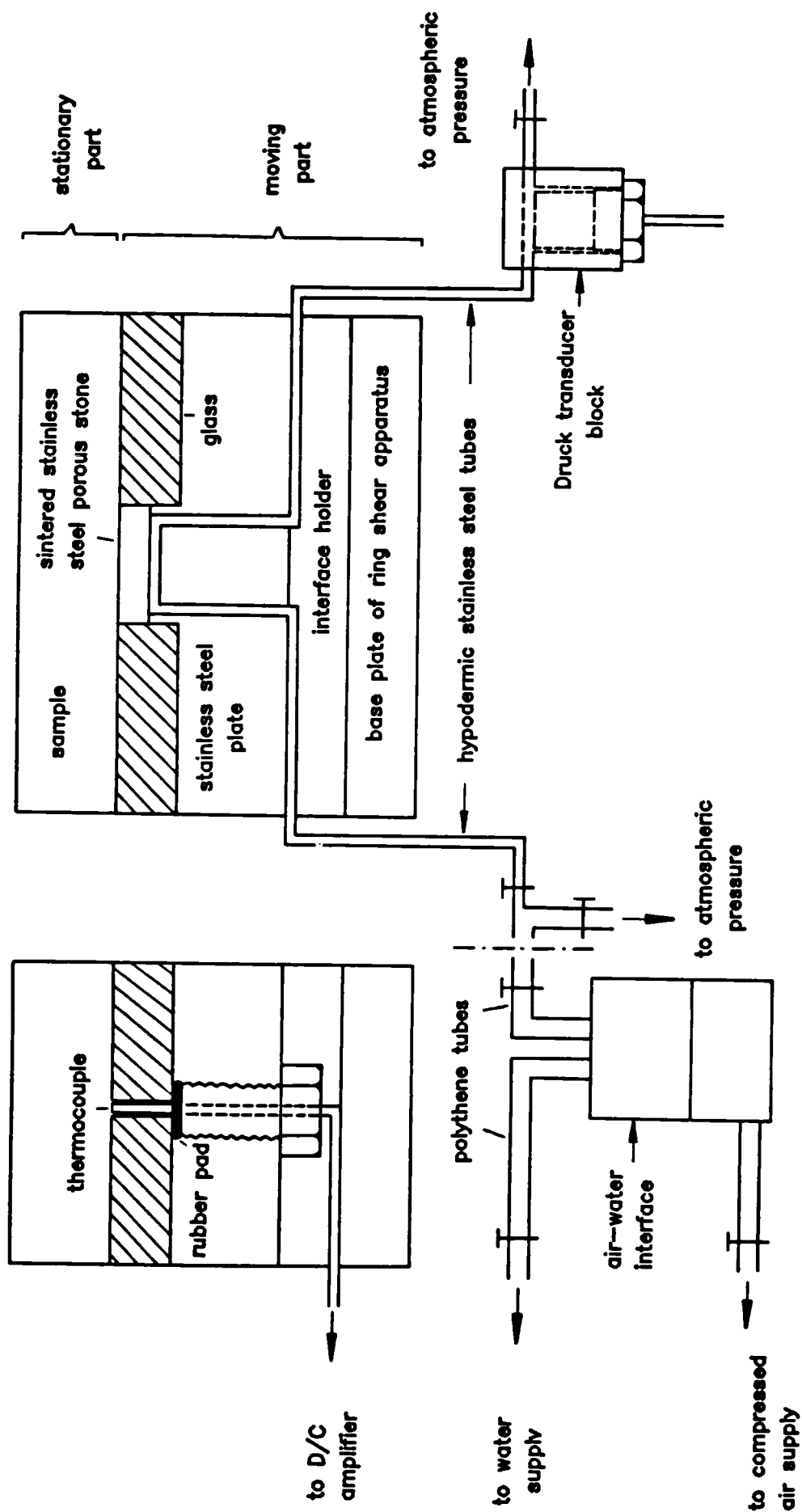


Fig. 6.5 Layout of the pore water pressure and temperature measuring systems

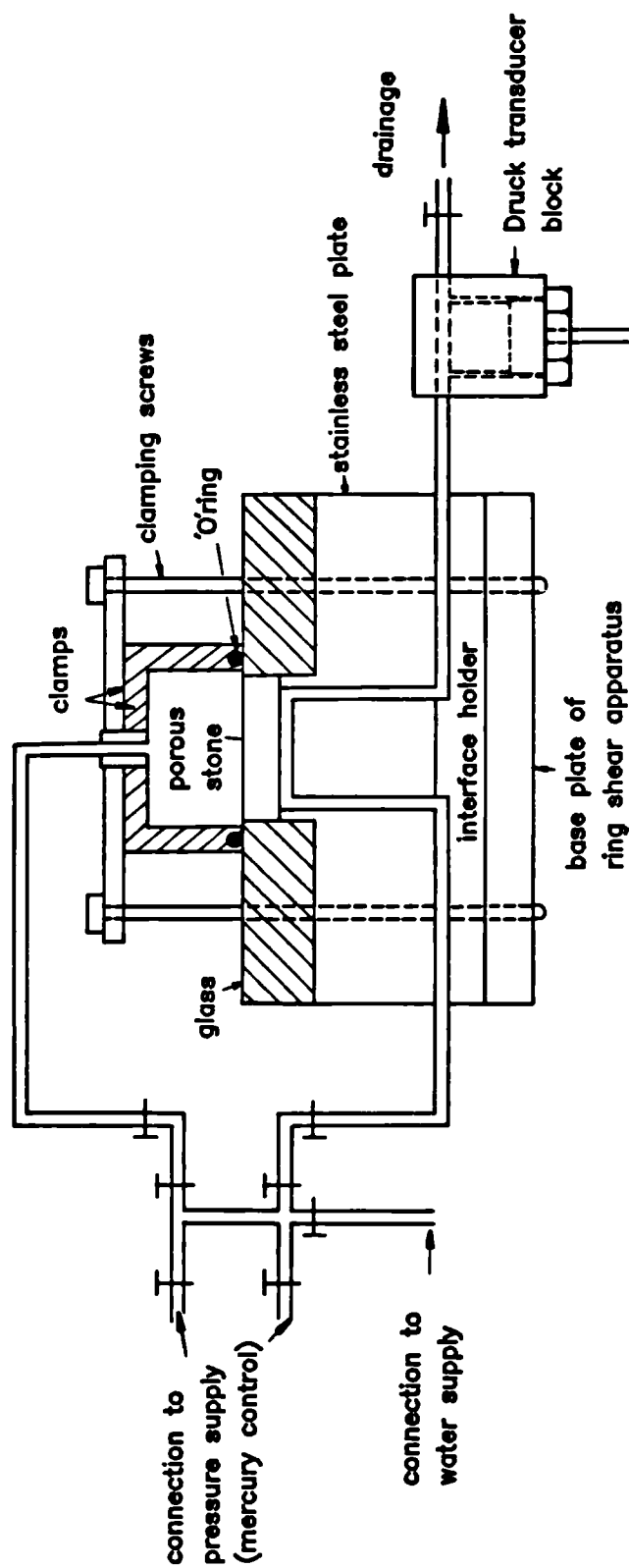


Fig. 6.6 Layout of the de-airing system

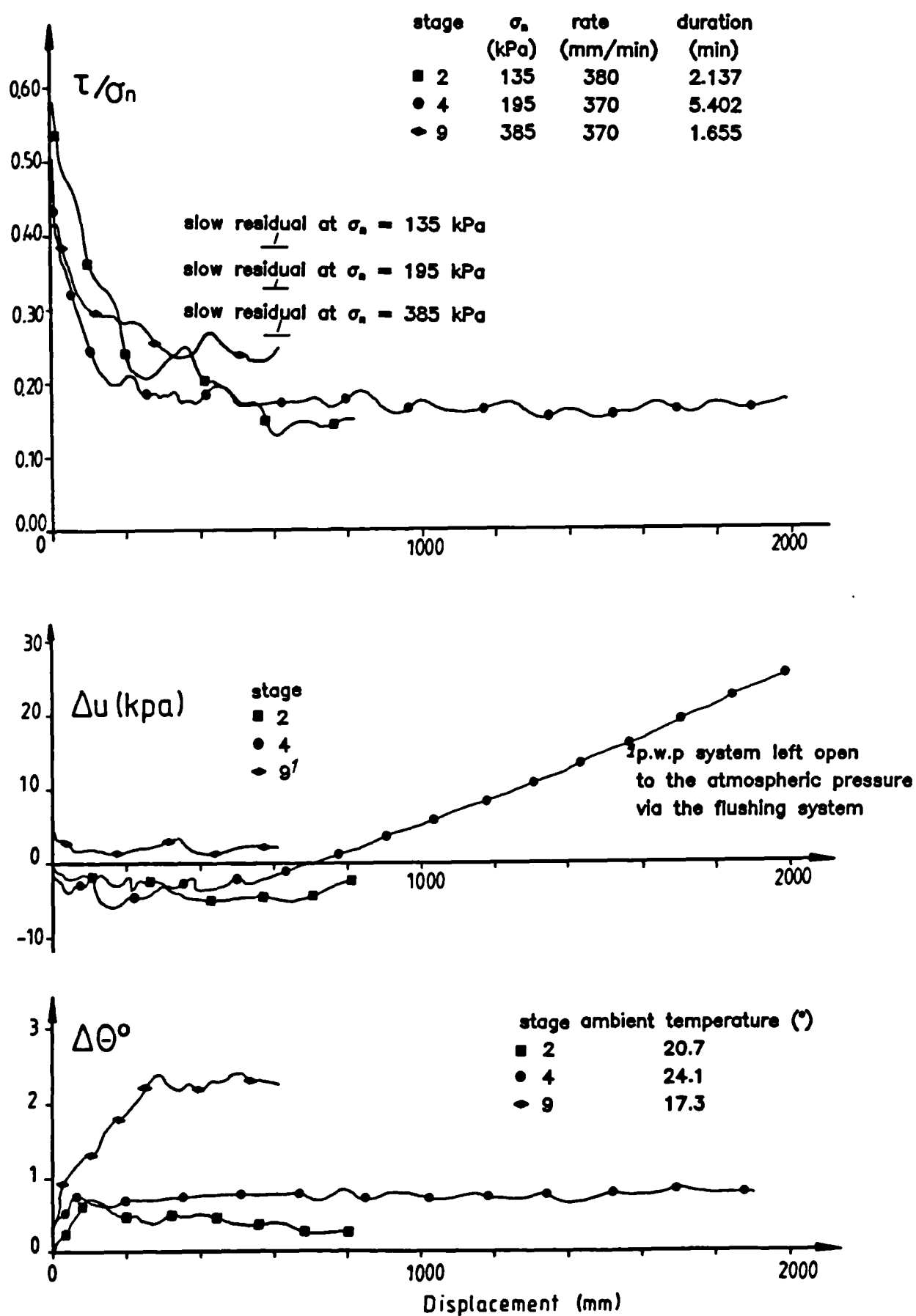


Fig. 6.7a Fast stages of Test 51 on clayey siltstone D

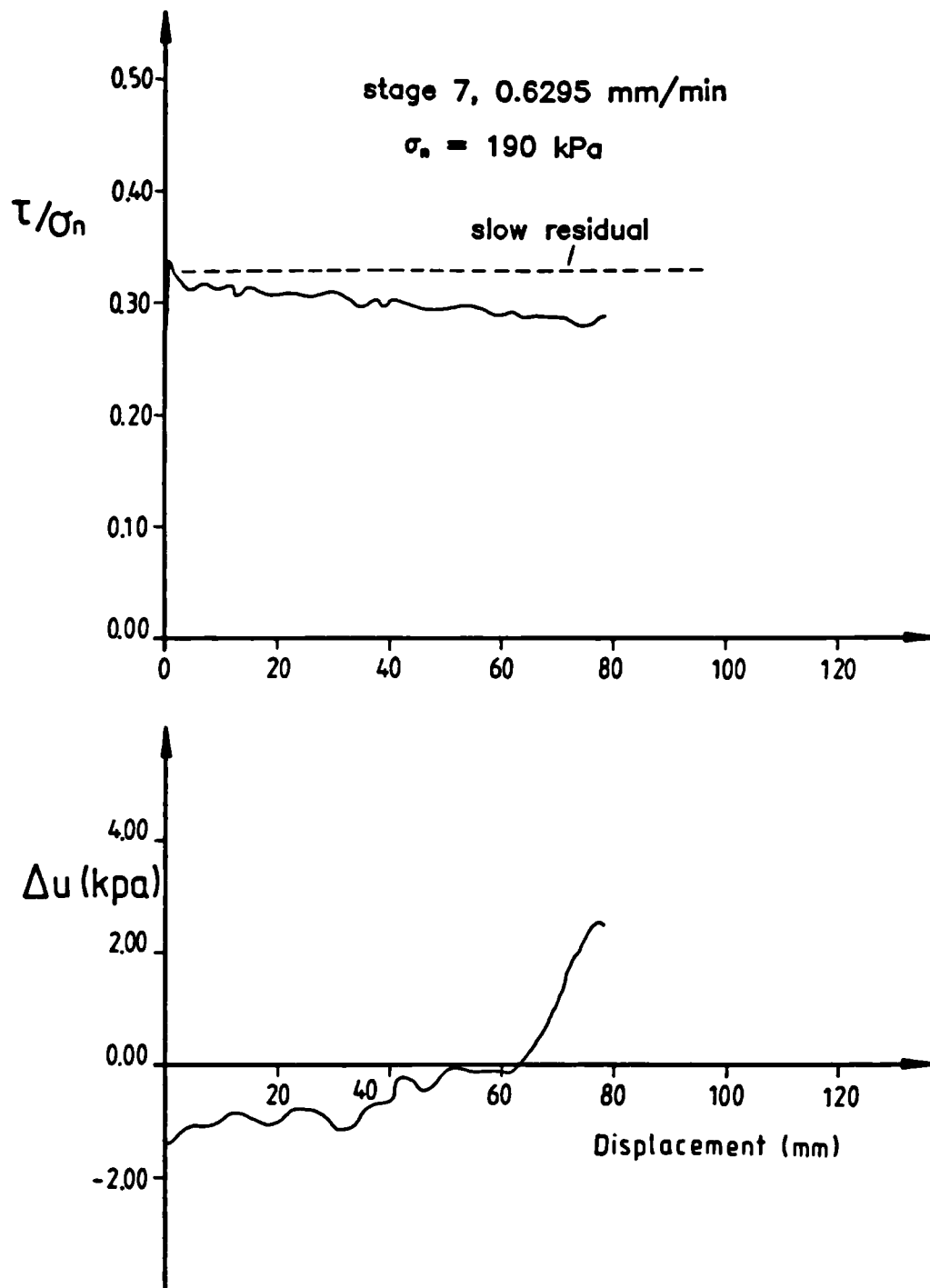


Fig. 6.7b Fast stages of Test 51 on clayey siltstone D

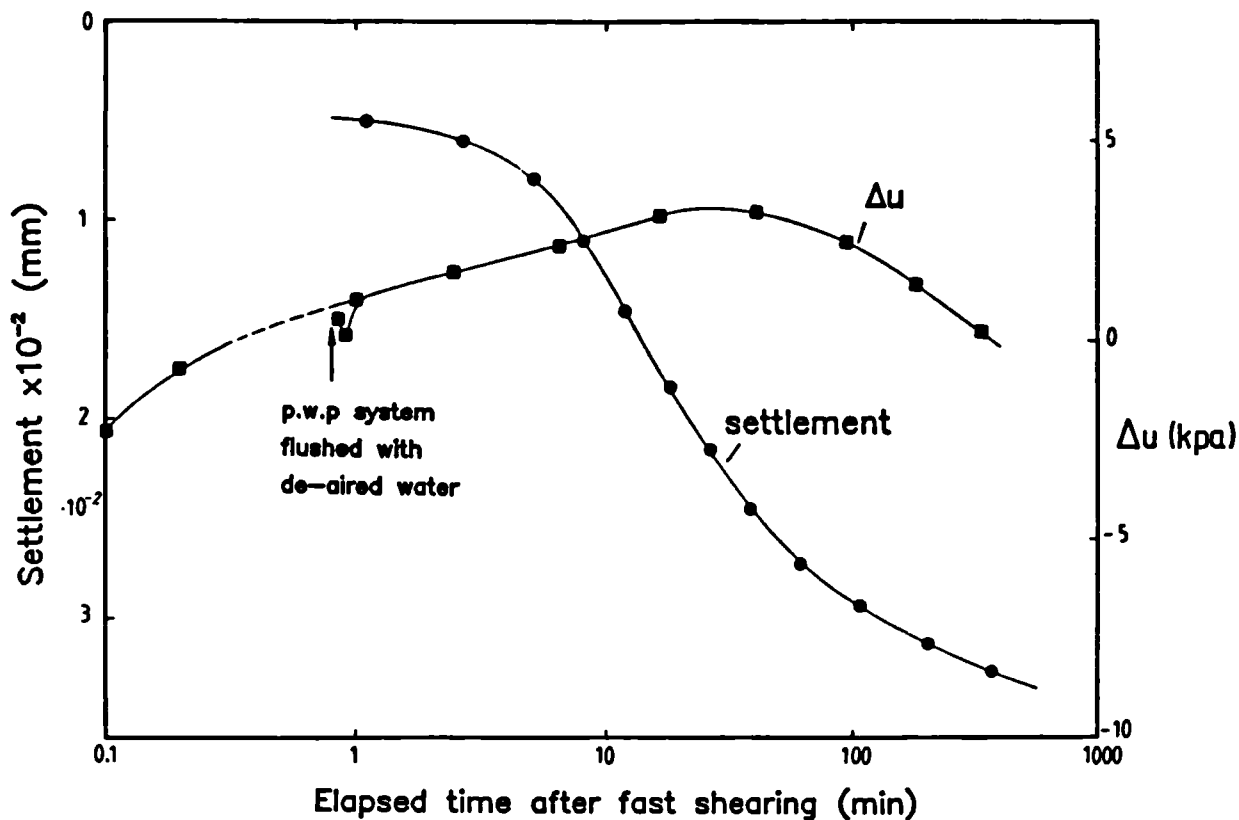


Fig. 6.8a Settlement and pore water pressure immediately after fast stage 2 of Test 51

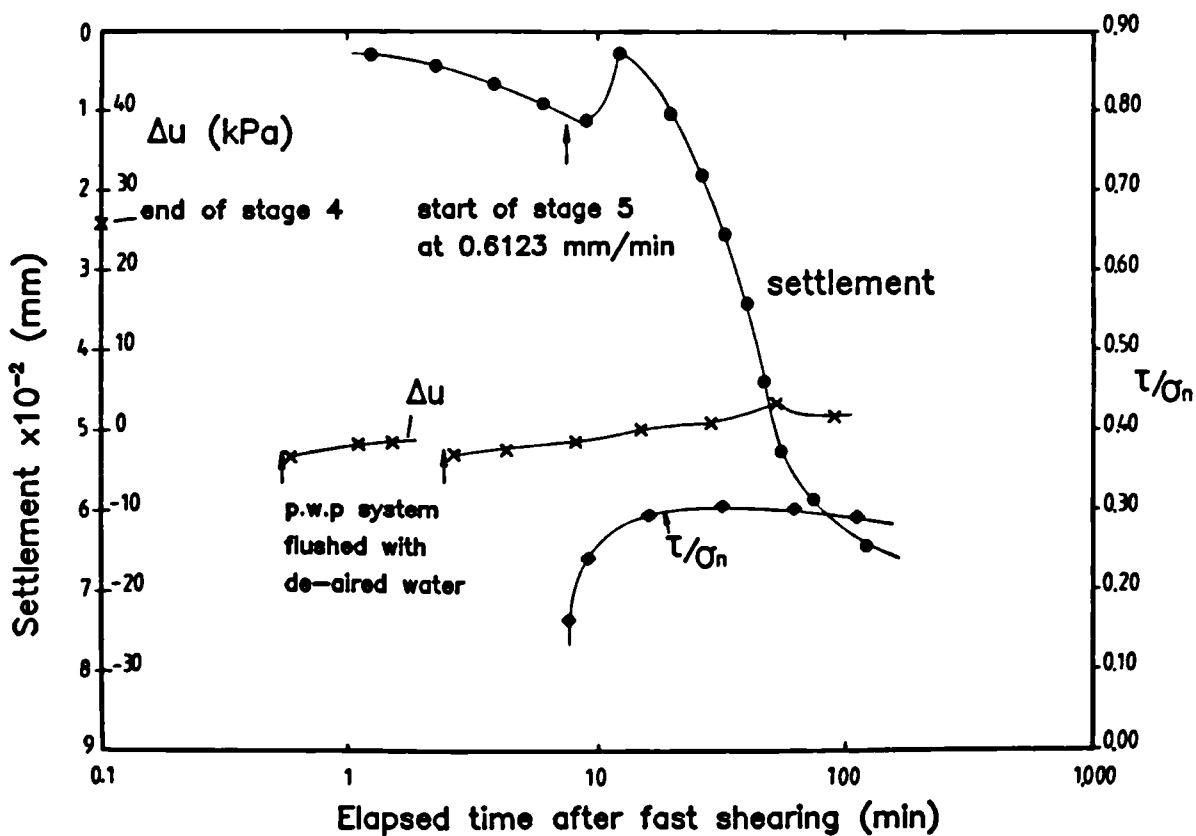


Fig. 6.8b Settlement and pore water pressure after fast stage 4 and during slow stage 5 of Test 51

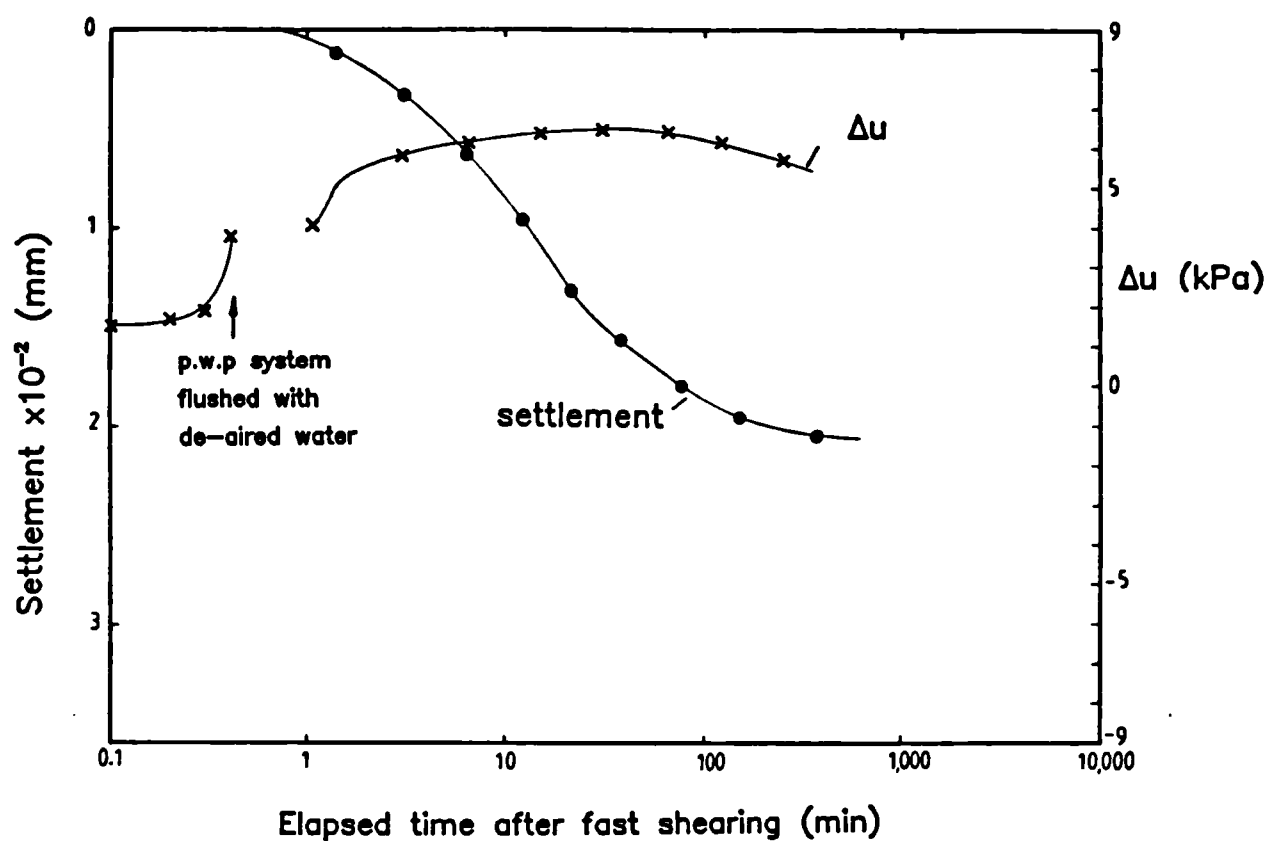


Fig. 6.8c Settlement and pore water pressure immediately after fast stage 9 of Test 51

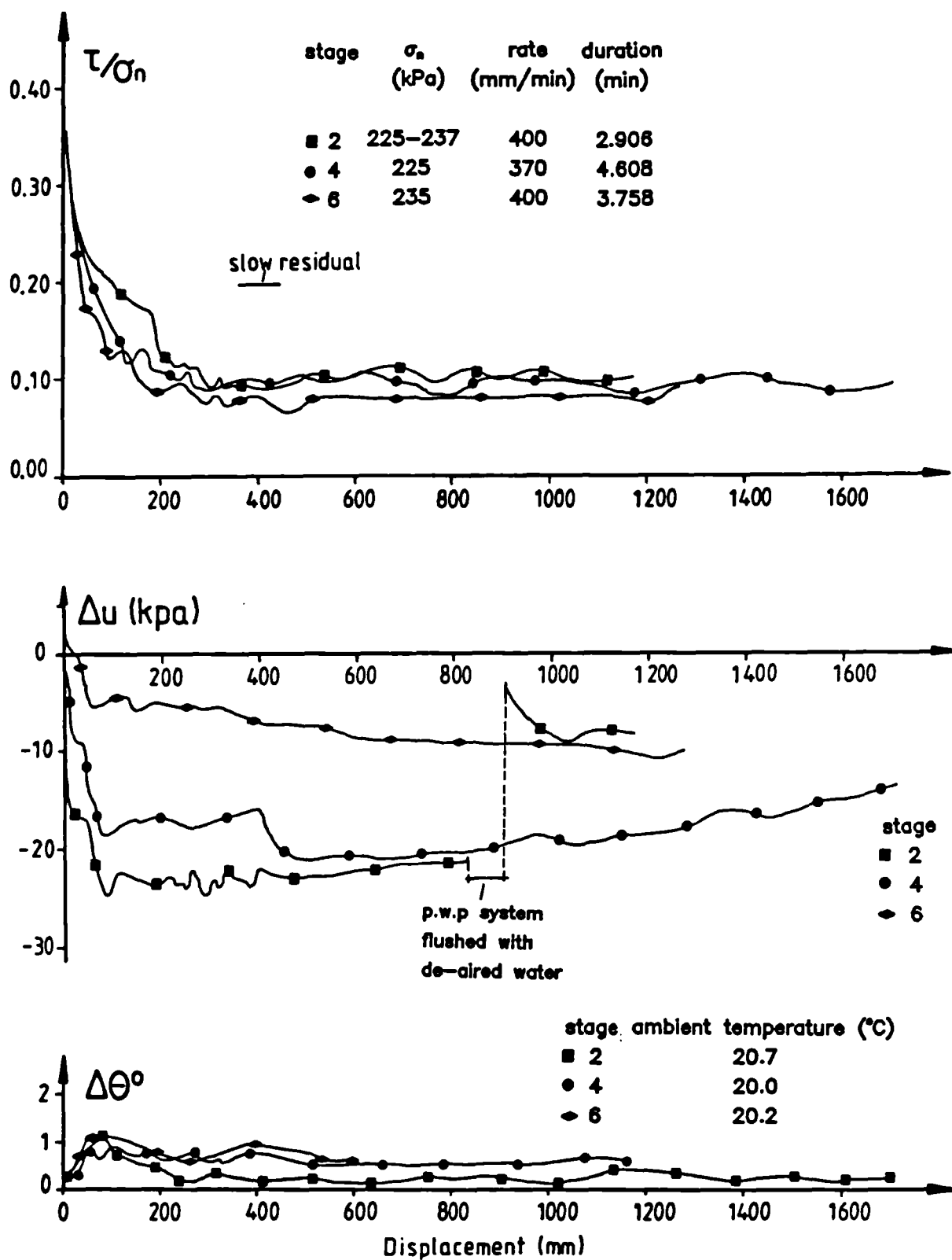


Fig. 6.9a Fast stages of Test 52 on claystone D

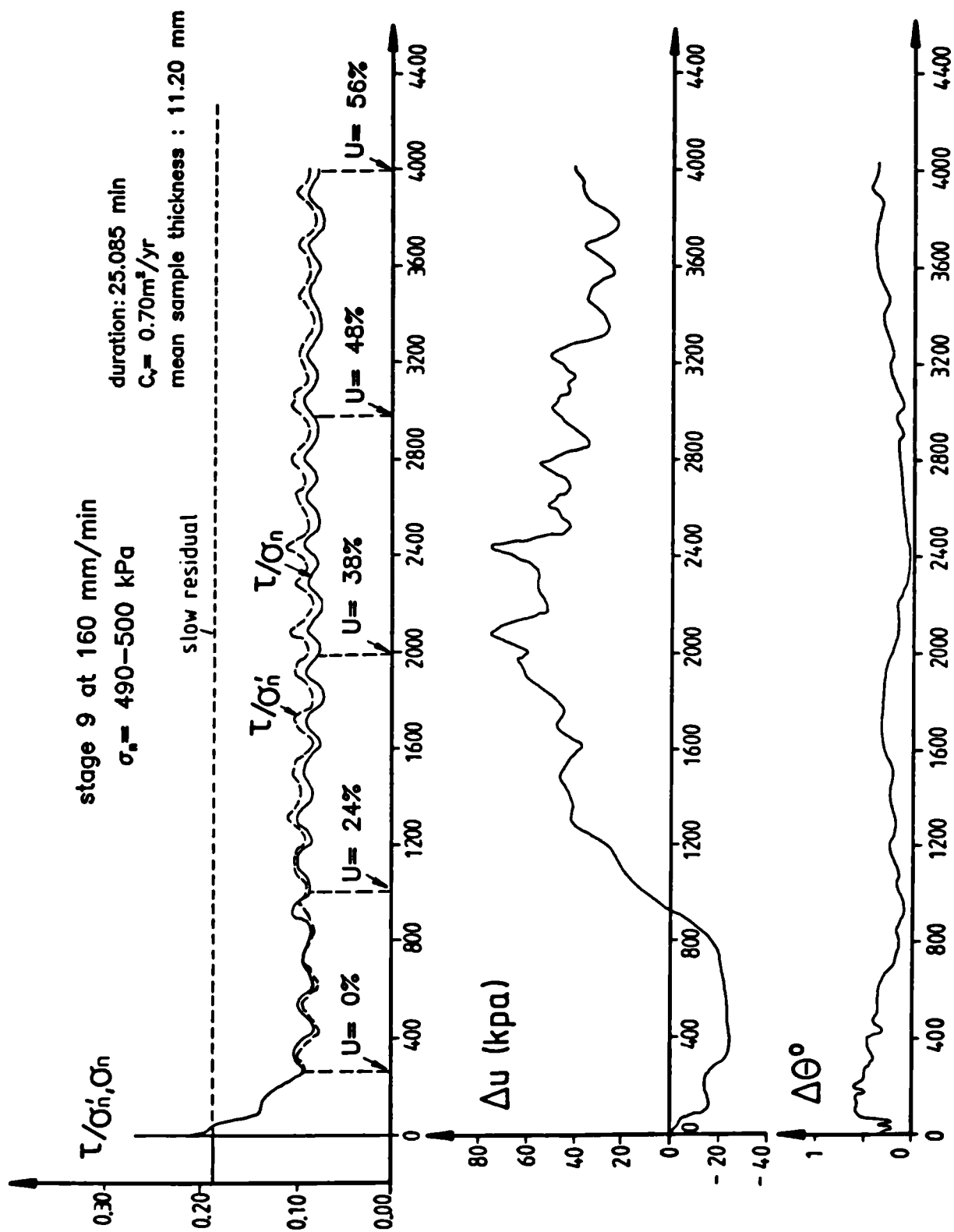


Fig. 6.9b Fast stages in Test 52 on claystone D

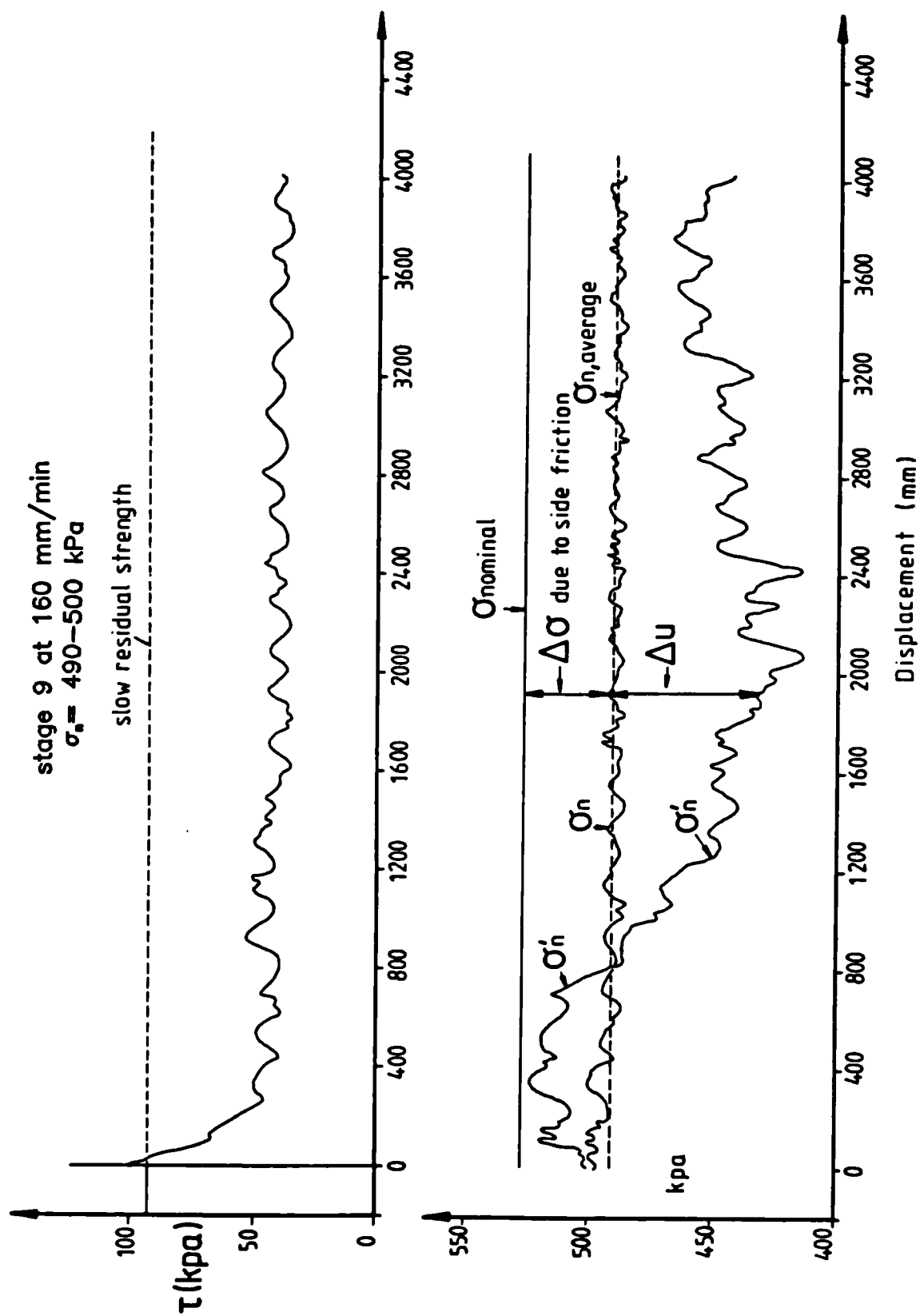


Fig. 6.9c Fast stages of Test 52 on claystone D

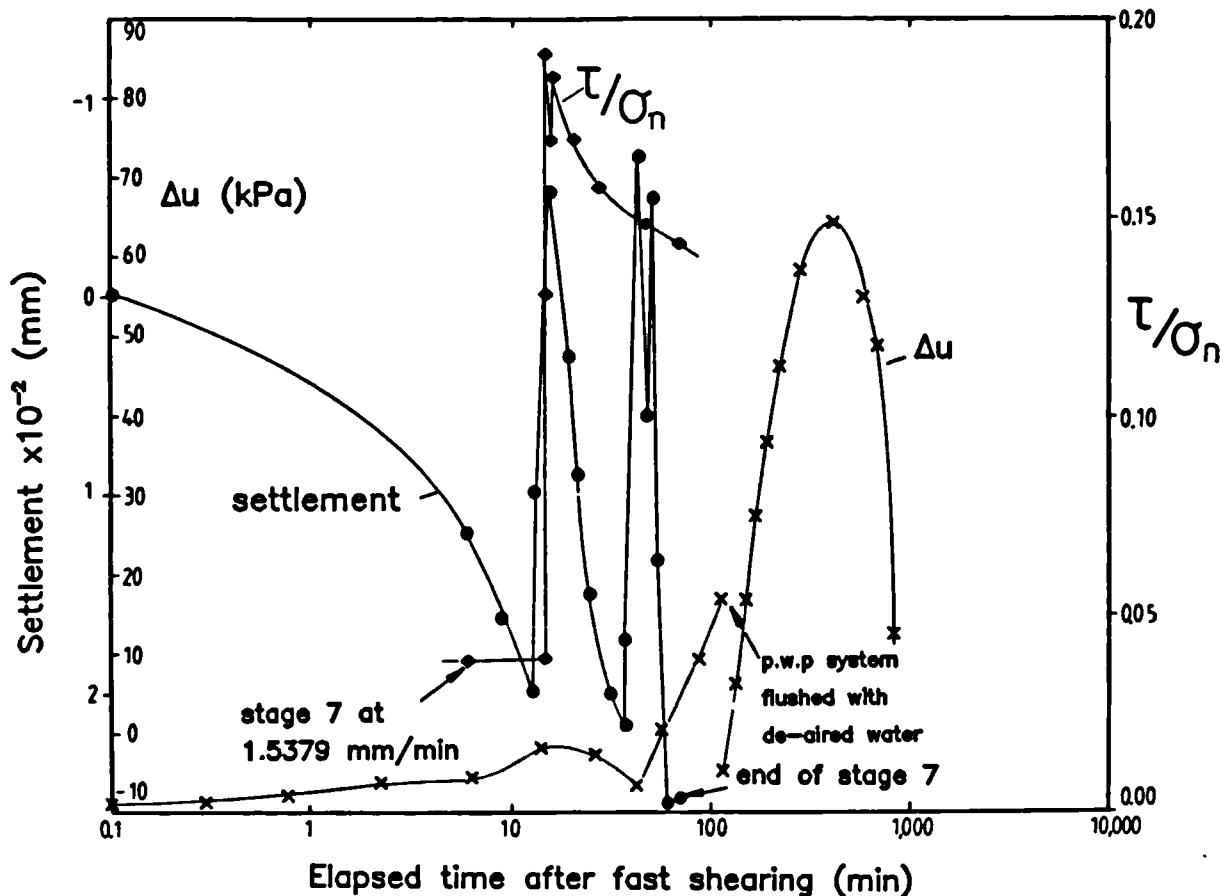


Fig. 6.10d Settlement and pore water pressure immediately after fast stage 9 of Test 52

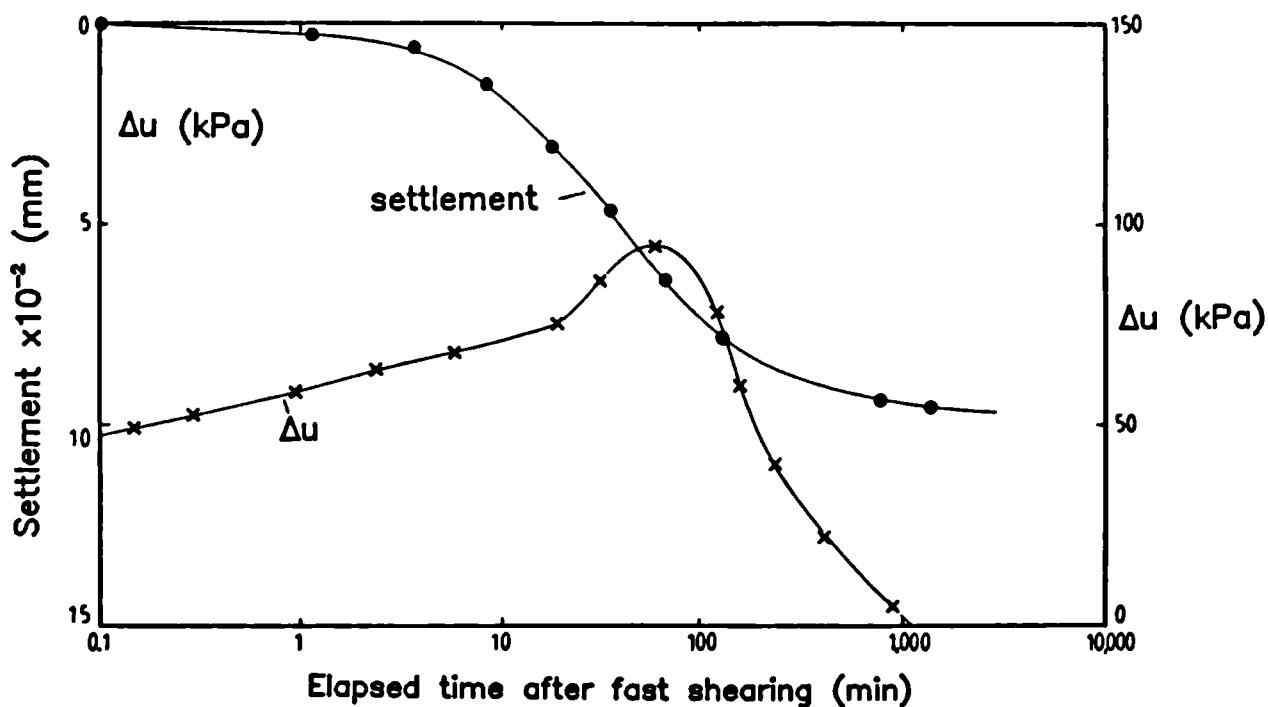


Fig. 6.10c Settlement and pore water pressure immediately after fast stage 6 and during stage 7 of Test 52

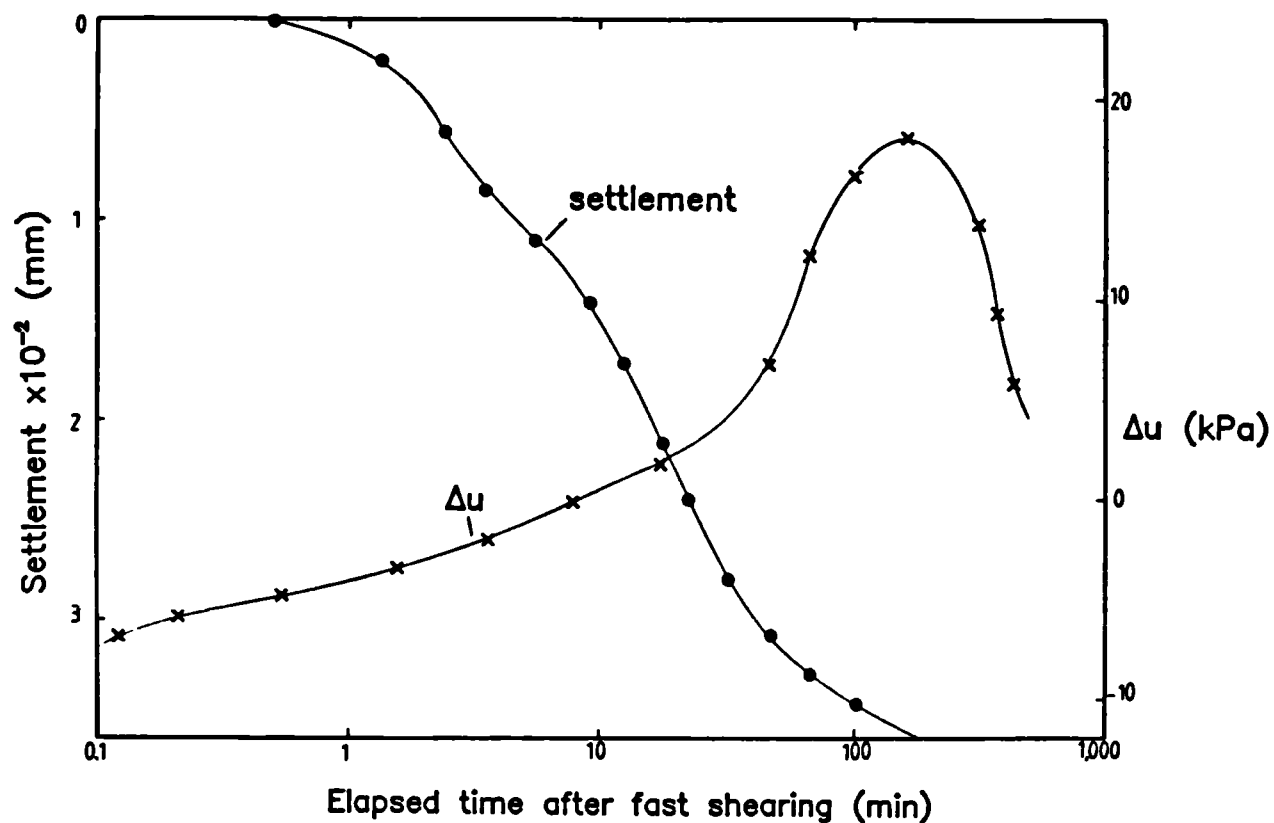


Fig. 6.10a Settlement and pore water pressure immediately after fast stage 2 of Test 52

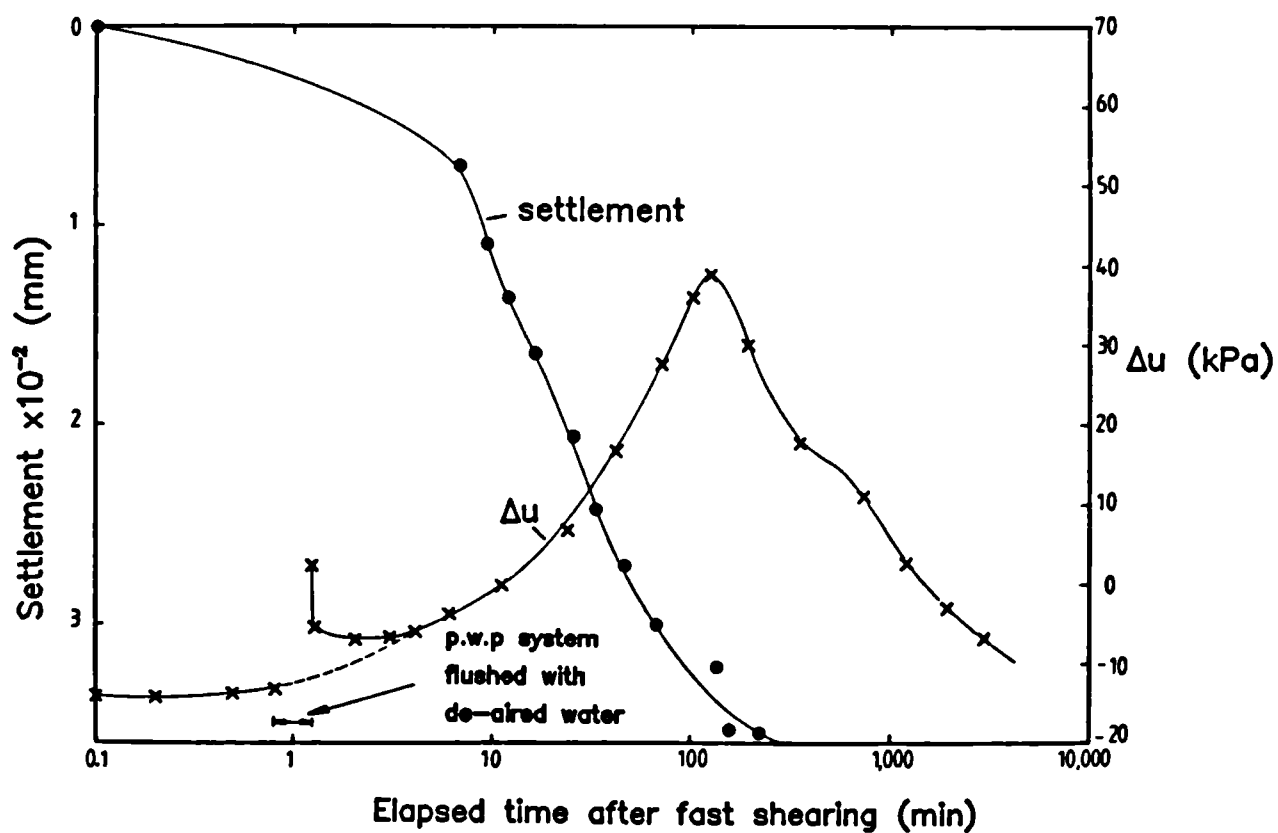


Fig. 6.10b Settlement and pore water pressure immediately after fast stage 4 of Test 52

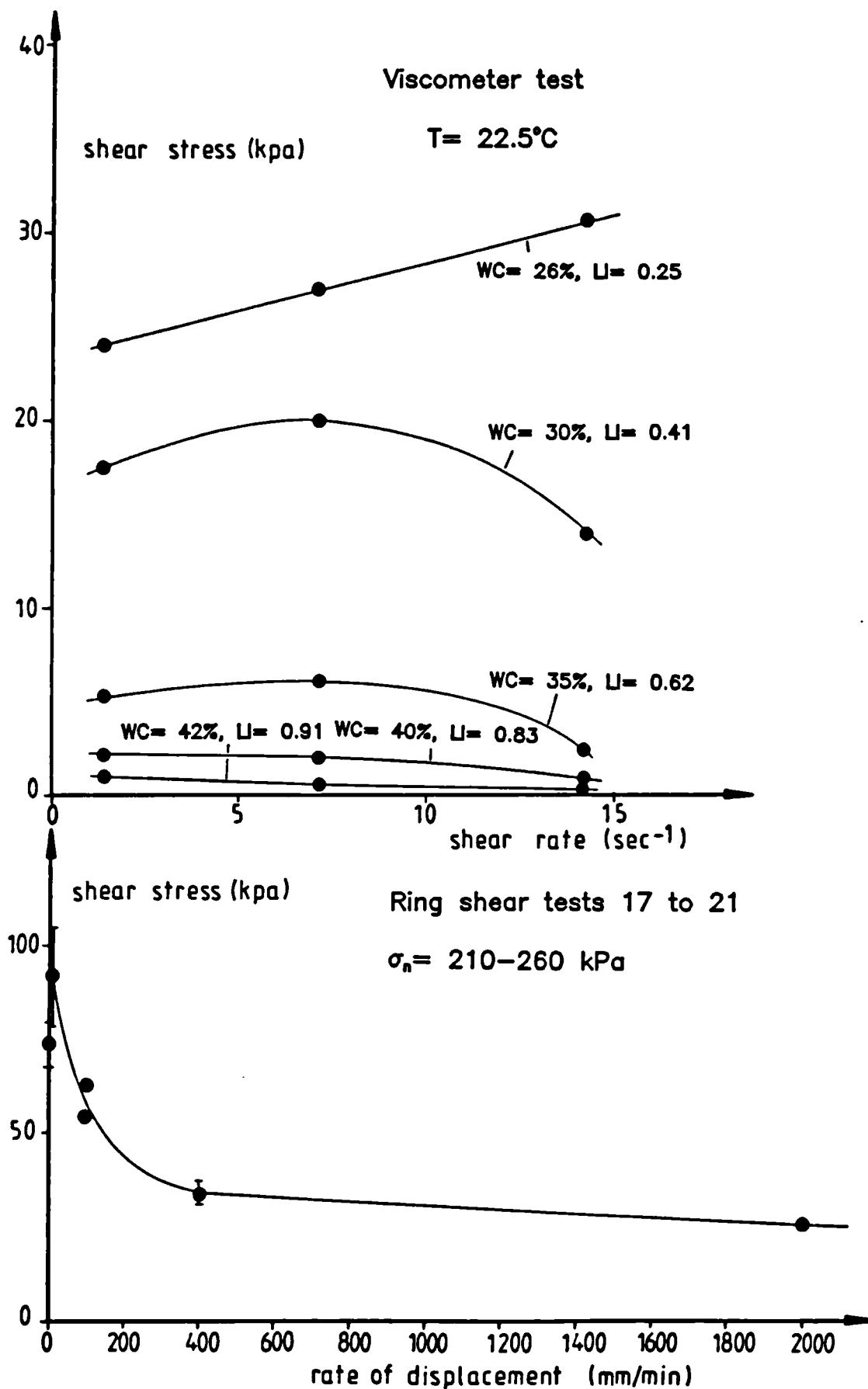


Fig. 6.11 Comparison of results from a viscometer test on clayey siltstone D with those from ring shear tests

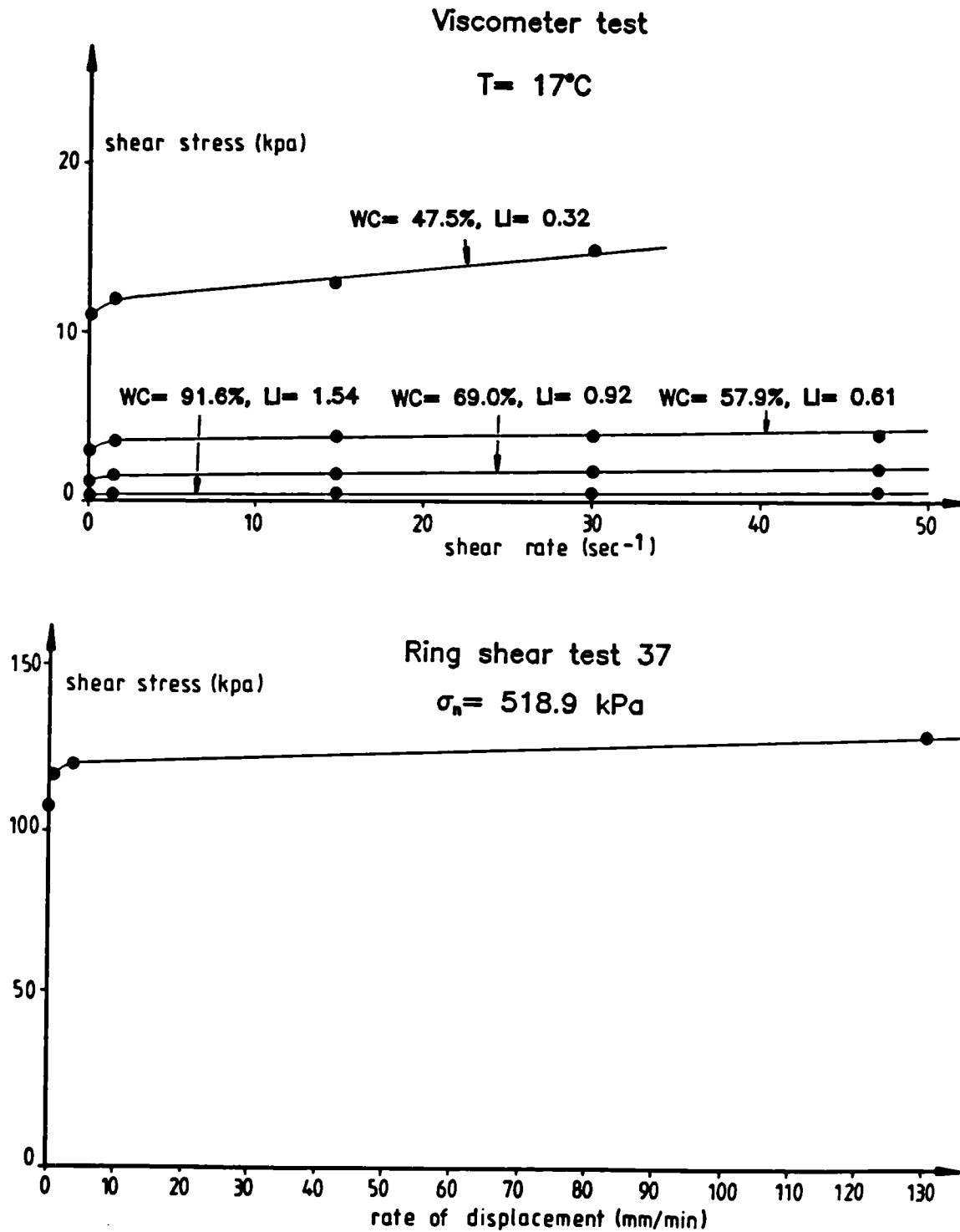


Fig. 6.12 Comparison of results from a viscometer test on kaolin with those from ring shear test 37

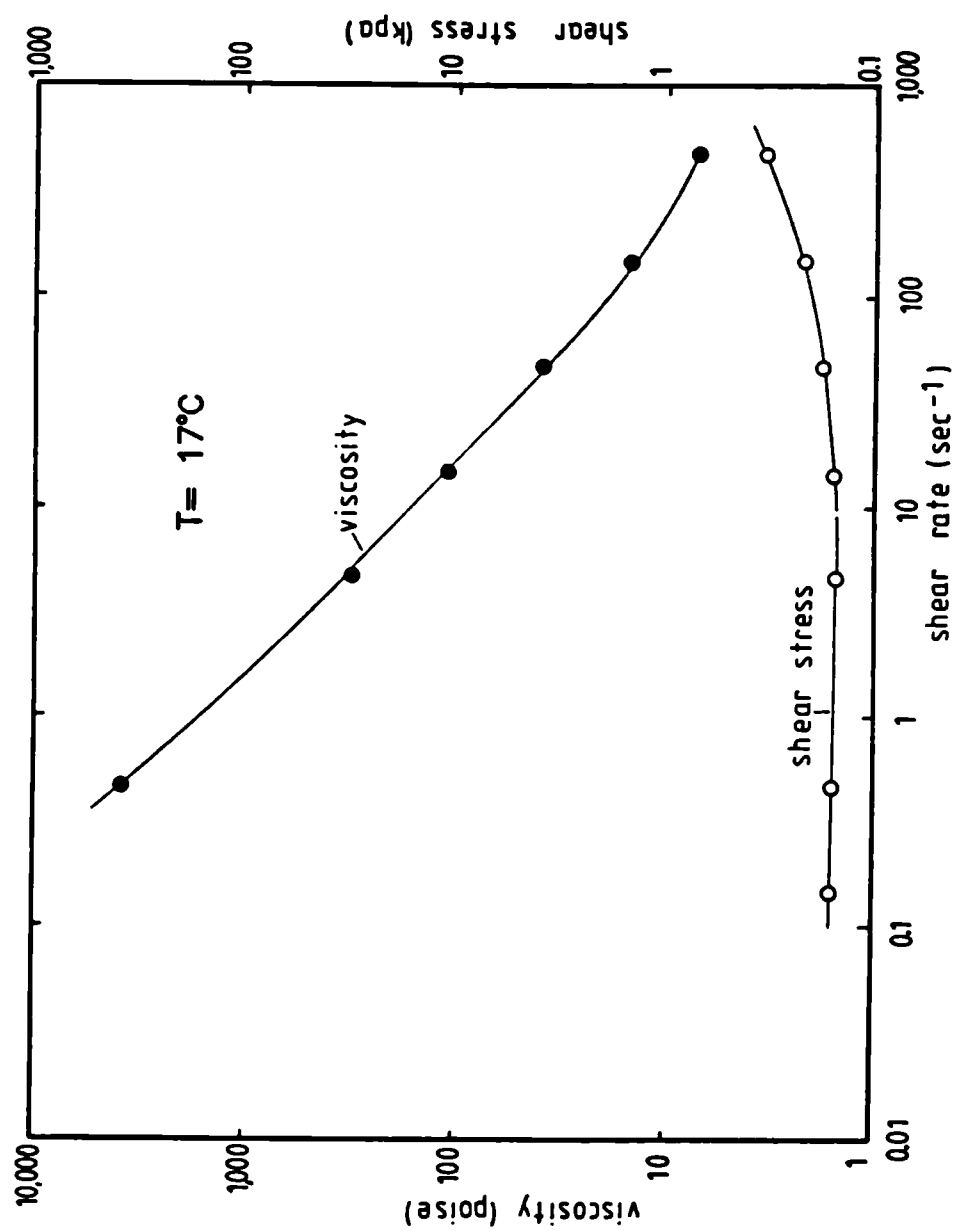


Fig. 6.13 Results from a viscometer test on glycerol.

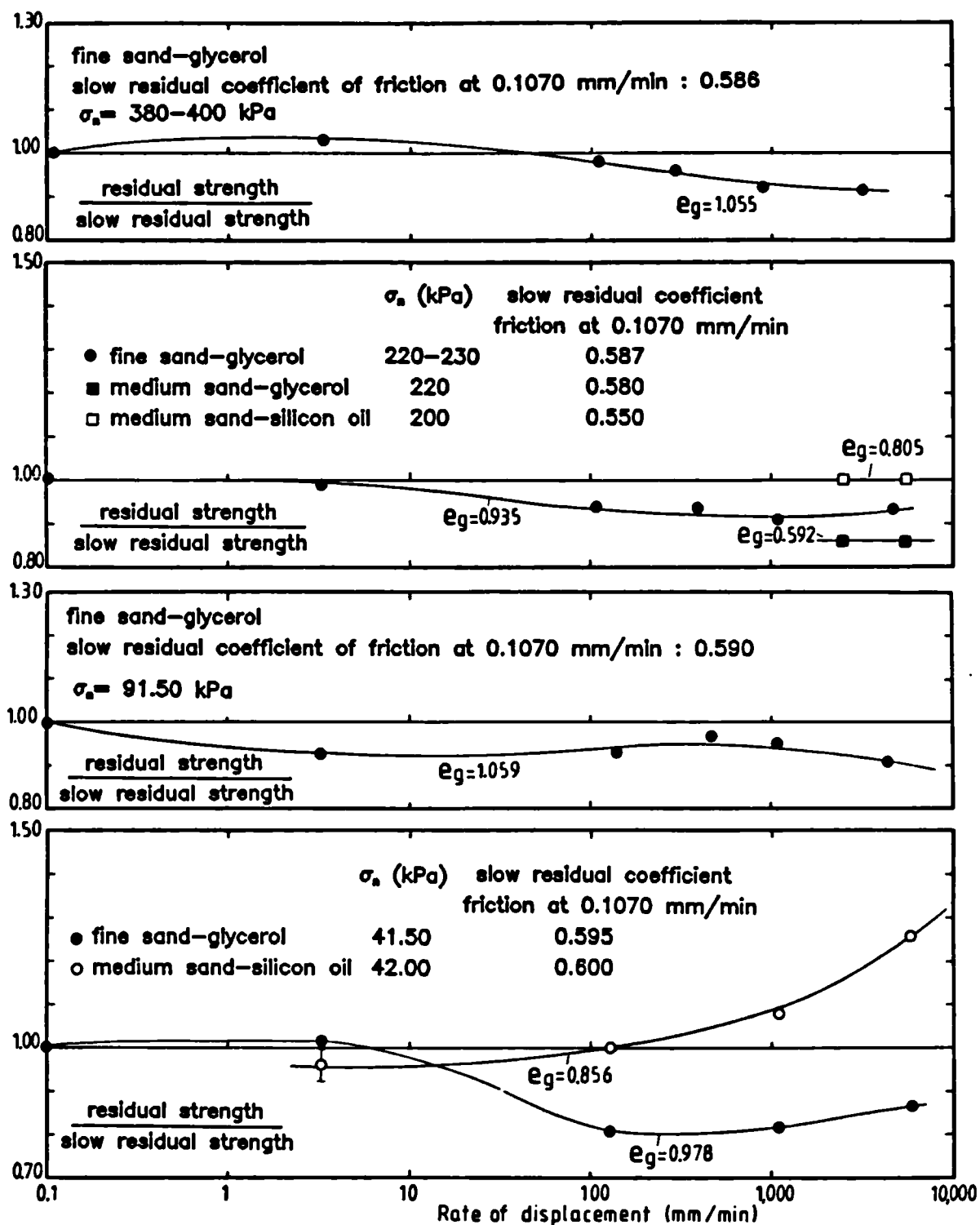


Fig. 6.14 Results from ring shear tests on mixtures of sand and viscous fluids

CHAPTER 7

7.1 CONCLUSIONS

When a shear zone or surface is formed at residual strength by slow drained shearing, and then subjected to more rapid rates of displacement, the following features are typically observed (Fig. 2.4).

There is an initial threshold strength on the shear zone, mobilized without further displacement, for rates of displacement equal to or faster than 10 mm/min typically. The ratio of threshold strength to slow residual strength increases with an increasing rate of displacement. All tested soils, apart from the soil from Fiji, showed higher ratios of threshold to slow residual strength at lower normal stresses, at a given rate of displacement.

For soils showing a transitional or a sliding shear mode, there is a further increase in strength with fast displacement on the shear zone up to a maximum value, the fast peak strength. The increase of fast peak strength above the slow residual strength is due to the disordering of the structure of the shear zone. The ratio of fast peak strength to slow residual strength increases with an increasing rate of displacement. All tested soils showed higher ratios of fast peak strength to slow residual strength at lower normal stresses, at a given rate of displacement.

The strength is then likely to drop with further fast displacement to a constant value, the fast residual strength. The fast residual strength is either similar to or higher than or lower than the slow residual.

If after fast shearing of soils with a transitional or a sliding shear mode, and a pause for consolidation, the shear zone is tested slowly, a peak strength greater than the slow residual strength is observed. This indicates that fast shearing causes disordering of the structure of the shear zone. There is a critical rate of displacement below which very little or no disturbance of the shear zone occurs. The critical rate of displacement increases with both soil plasticity and normal stress. For soils with a transitional shear mode tending towards the sliding mode or a sliding shear mode, apart from the soil from Fiji, the ratio of fast peak strength to slow residual strength increases with an increasing rate of displacement, at a given normal

stress, and with a decreasing normal stress, at a given rate of displacement.

Five types of variation of the residual strength with an increasing rate of displacement were identified (Fig.5.15).

- (i) Type I showing a constant residual strength irrespective of the rate of displacement.
- (ii) Type II showing either a negligible to moderate increase or a slight decrease of residual strength with an increasing rate of displacement, followed by a significant drop of strength to less than the slow residual value at higher rates of displacement.
- (iii) Type III showing a decrease of residual strength with an increasing rate of displacement, followed by an increase and then a significant drop of strength to less than the slow residual value at higher rates of displacement.
- (iv) Type IV showing a decrease of residual strength with an increasing rate of displacement, followed by an increase above the slow residual value at higher rates of displacement.
- (v) Type V showing an increase of residual strength with an increasing rate of displacement.

Degradable soils, in which platy particles may be generated by massive grains breakdown during shearing, can change the type of their behaviour.

The above described types of soil behaviour were defined with respect to the slow residual strength, measured at the slow standard rate of displacement which, for the tests reported here, varied from 0.003 to 0.10 mm/min, with 0.01 mm/min taken as the typical one.

The type of behaviour does not relate to the type of drained residual shear mode or the index properties of soils, but rather to the grading of their granular phase. Soils that exhibited the same type of behaviour can be summarized as follows (Fig. 5.18 to 5.20):

- (i) The soils that exhibited behaviour of type I had a turbulent shear mode. These were sands and sandy silts with a plasticity index less than 9%, a sand content larger than 20%, and clay content less than 10%.
- (ii) The soils that exhibited behaviour of type II, except the residual soil from Fiji, showed a turbulent or a transitional shear mode, and had a plasticity index between 9 and 21%. These were silts with a silt content greater than 60% and each of the sand and silt contents less than 20% and a gap-graded clayey sand. Some of the soils with a plasticity index between 9 and 13% showed behaviour of type I, at high normal stresses ranging from 450 to 500 kPa.
- (iii) The soils that exhibited behaviour of type III were silty clays with a transitional or sliding shear mode and a plasticity index between 24 and 26%.
- (iv) The soils that exhibited behaviour of type IV were low plasticity clays and silty clays. The low plasticity clays showed a turbulent or a transitional with some turbulence shear mode, had a plasticity index between 10 and 21%, and were characterized by either a very well graded or a uniform granular phase. The silty clays showed either a transitional shear mode tending towards the sliding mode or a sliding shear mode, and had a plasticity index between 26 and 36%.
- (v) The soils that exhibited behaviour of type V were clays which had a sliding shear mode, a plasticity index between 36 and 51%, and a clay content higher than 48%.

The residual soil from Fiji contained poorly crystallized kaolinite. It showed pure sliding behaviour in slow residual shear, and behaviour of type II at increasing rates of displacement. When the soil was subjected to slow shearing, after a fast stage in which a significant loss of strength was observed, the slow peak strength was similar to the normally consolidated peak strength. This indicates that in this soil slow shearing involves pure sliding, while fast shearing involves turbulent behaviour.

The increase of the residual strength above the slow residual value, observed with types II, III, IV and V of behaviour, may be due to viscous effects for soils showing a turbulent or a transitional with some turbulence shear mode, and to structural changes taking place in the shear zone, for the soils showing a transitional or a sliding shear mode.

The slight decrease of the residual strength with an increasing rate of displacement, observed initially in types IIa, III and IV of behaviour, may be due to pore water pressure inhomogeneities risen by the particular microfabric of the soils, as postulated by Hight (1983).

The following mechanism may be tentatively advanced to explain the significant drop of strength below the slow residual value, observed for soils showing behaviour of types II and III.

Shearing of these soils at a rate of displacement above a critical level involves an increase of the porosity in the shear zone to permit massive grains to pass over one another. The low strength, reached during shearing at a rate faster than the critical, is a steady state without significant excess average pore water pressure due to shearing or frictional heating. It is caused by a reduction in the viscosity of the soils and thus an increase in the ability to flow. Whenever the fast shearing is stopped or slowed down, the shear zone attempts to contract and excess pore water pressure is generated. The low strength in the shear zone, while it results from fast shearing, is not significantly rate dependent. It persists even at slow rates of displacement until the excess pore water pressure dissipates.

The transformation of the initial stable structure of these soils into a loose state and the consequent ability to flow more easily, when the rate of displacement is increased beyond some critical level, may be analogous to the change in phases observed in non-particulate substances (solids, liquids and vapours).

Soils with different gradings have different granular void ratios and thus represent different initial phases with different critical levels of rate of displacement. On the evidence of the test results reported here, the loss of strength is associated with a significant silt content, above 30% approximately, and a clay content between 3 and 40%.

It does not happen in the absence of clay, or when the clay content is higher than 48% and the soil shows a sliding shear mode.

7.2 PRACTICAL IMPLICATIONS

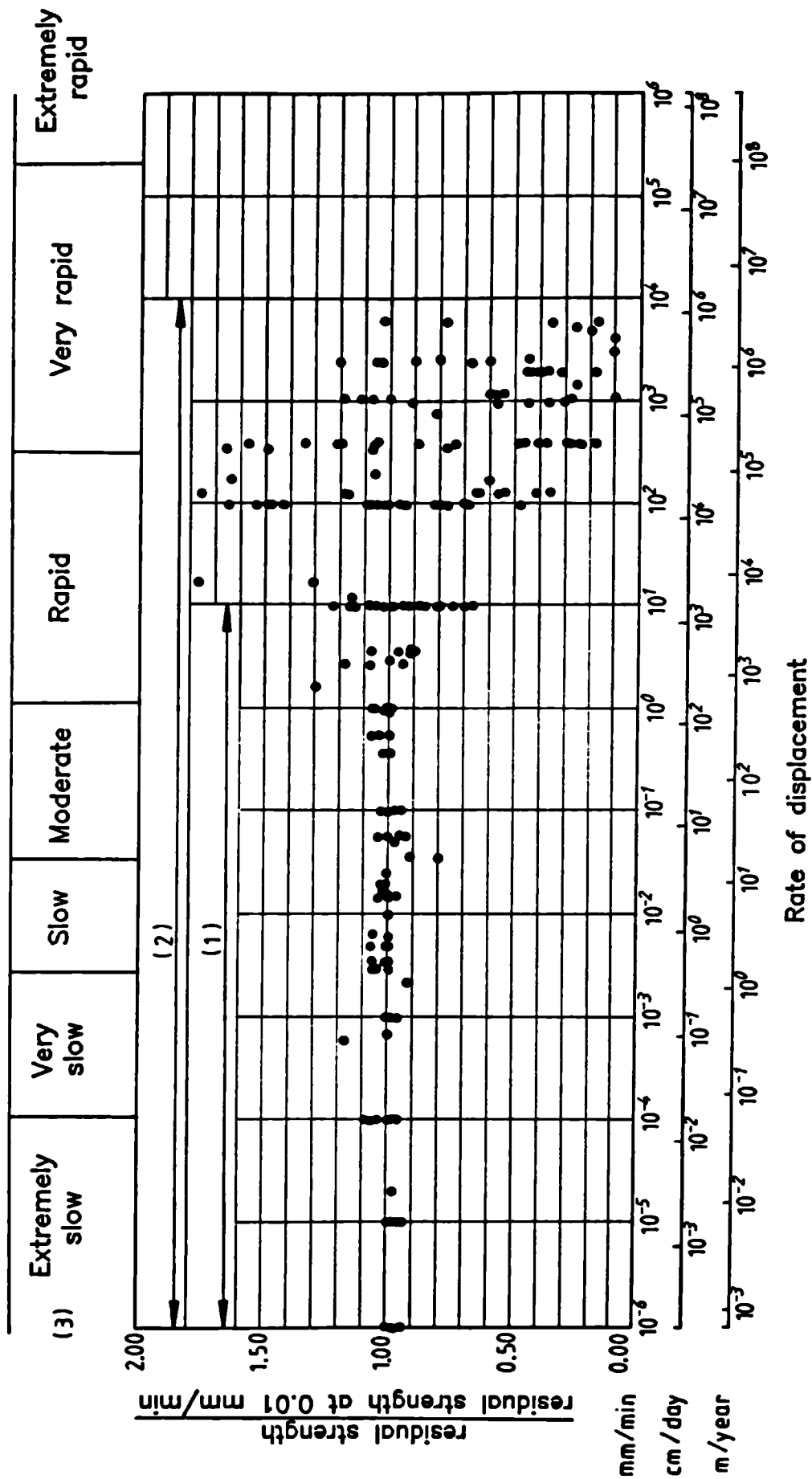
Rates of displacement on pre-existing shear zones or surfaces in field can vary by many orders of magnitude from exceedingly slow movements to extremely fast one (see for example, Terzaghi, 1950, Skempton and Hutchinson, 1963, Hamel, 1973, as reported by Morgenstern, 1977).

Fig 7.1 presents a rate of movement scale suggested by Varnes (1978) and the range of rates of displacement used in this research. It also plots the results from the tests on the soils showing types II to V of behaviour, and those of the reviewed investigations for the cohesive soils. The results have been normalized by the residual strength at 0.01 mm/min.

The picture emerging from the above figure is that, whereas the effect of rate of shearing on the residual strength of cohesionless soils is minimal, the residual strength of cohesive soils depends to an appreciable extent on this factor. Cohesionless soils are considered to be sands and sandy soils with a plasticity index less than 9%, a sand content larger than 20% and a clay content less than 10%. In cohesive soils, for rates of displacement lower than 1 mm/min, that is for extremely slow to moderate movements, the variation of residual strength is within a range of $\pm 10\%$, if a rate of 0.01 mm/min is taken as a standard one. For rates of displacement, however, greater than 1 mm/min, that is for rapid to extremely rapid movements, the variation of residual strength with the rate of displacement becomes significant. A knowledge of the effect of rate of shearing on the residual strength of cohesive soils is therefore necessary when studying the kinematics of a potential slide mass.

The results from the tests presented here allow the formulation of a soil model linking the strength on a pre-existing shear zone or surface with the displacement and rate of displacement. This model can be used in the study of stability and prediction of instability of slopes containing pre-existing shear zones or surfaces at or close to the residual strength under various disturbing events, such as earthquakes (Ambraseys and Menu, 1988), fluctuations of ground water table and

others, or other phenomena, such as, the strength of shear surfaces at the sides of piles, which are formed by pile driving.



- (1) Range between the field lowest and field highest rate of movement for reactivated landslides and mud-flows (Skempton, 1985).
 (2) Range of rates of displacement used in this research work.
 (3) Range of movement scale (Varnes, 1978).

Fig. 7.1 Variation of residual strength of cohesive soils with rate of displacement

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APPENDIX 1

CALIBRATIONS OF RING SHEAR APPARATUS AND TESTING ERRORS

A1.1 CALIBRATIONS OF THE INSTRUMENTATION

The instrumentation used for the ring shear apparatus was calibrated following standard calibration procedures used in the laboratory (at least 20 cycles of loading-unloading up to 80% of the instrument capacity prior to calibration and three cycles of loading-unloading during the calibration were performed). The calibrations were checked regularly throughout the experimental period.

Calibration constants were obtained from relationships between secant calibration factors and changes in output reading from a given reference zero, or from the relationship between the physical parameter and the output reading, depending on the type of the instrumentation used.

The load cells were calibrated in compression using a Budenberg dead weight calibrator. As the minimum load that the calibrator can apply is 108.5N, the main calibration was supplemented by applying dead weights directly onto the load cell itself. The proving rings, used for the torque measuring system of ring shear 1, were calibrated in compression by applying dead loads onto the proving ring itself in the manner described by Garga (1971).

The proving rings, used for the measurement of the side friction, were calibrated both in extension and compression.

The displacement transducers, used for the measurement of the vertical deformation of the sample, were calibrated using a vernier calibrator with a resolution of 0.01 mm/div.

The pore water pressure transducers were calibrated using a Budenberg dead weight calibrator. As the minimum pressure that the calibrator can apply is 68.9 kPa(10psi), the main calibration was supplemented by another one at low pressure values by using a U-tube mercury manometer. The transducers were also calibrated at subatmospheric pressures using

the U-tube mercury manometer.

The torque measuring instruments were zeroed at the start of the tests and whenever the specimen was relieved from the shear load. Zero drift was found to be extremely small (of the order of 2N at the maximum over a period of one week). The proving rings, used for the measurement of side friction, were zeroed at the start and at the end of the tests. The dial gauges and the displacement transducers, used for the measurement of settlement, were zeroed at the start of the shearing stage of the tests. The dial gauges, used for the measurement of the size of the gap, were zeroed at the end of the consolidation stage and after taking out the fastening screws. The calibrations of the instrumentation are given in Table A1.1. The specifications of the U/V recorder and pressure transducers are given in Tables A1.2 and A1.3 respectively.

A1.2 VERTICAL DEFORMATION OF RING SHEAR APPARATUS

This was checked for both ring shear apparatus by placing a metallic annulus which had the dimensions of a typical sample between the upper and lower platens of the apparatus (Fig.A1.1)

A1.3 CALIBRATIONS OF THE MOTOR DRIVE UNITS

The specifications of the fast motors are given in Table A1.4 and the calibrations of all the motor drive units in Fig. A1.2.

A1.4 DISCUSSION OF TESTING ERRORS

A1.4.1 Error caused by the non-uniform distribution of stresses

As mentioned in section 3.2.7, the calculation of the results was based on the assumption that both the normal and shear stresses are uniformly distributed over the sample area. This would be correct, if two hypotheses were true.

The first hypothesis is that the strength is independent of both rate of displacement and displacement during shearing. The displacement and the rate of displacement at the external radius are 50% greater than

those at the internal radius. The shear stress therefore is non-uniform throughout the sample.

The second hypothesis is that there is no loss of soil through the gap between the confining rings. Loss of soil causes redistribution of normal stress on the failure plane. According to La-Gatta (1970), it causes the vertical stress to decrease at the outer specimen periphery. Since the vertical load is constant, the vertical stress increases at the inner periphery of the specimen.

In addition to the above, sample non-uniformity and the fins attached to the porous stones complicate the picture of stress-distribution even further. The assumption of uniform distribution of the normal and shear stresses, however, was considered more reasonable for the computation of the results as it gives only small differences from extreme non-uniform distributions of the stresses (Bishop et al 1971), and as the exact pattern of stress distribution is not known.

A1.4.2 Error caused by the friction in the gap

Loss of soil through the gap between the confining rings causes friction between the soil and the rings which changes the torque resistance.

Hvorslev and Kaufman (1952), Garga (1970), and Lupini (1981) estimated the possible error by considering that squeezing of soil takes place along a lip of the same width as the edges of the confining rings. The maximum possible error is given by the ratio of the apparent area (because of soil squeezing) to the true area of test specimen. The estimation of the error was based on the assumption that the vertical load on the squeezed-out soil is the same as the vertical load on the test specimen, and that the coefficient of friction between the soil and the confining rings is similar to that of the soil itself.

La Gatta (1970) estimated the error in the measured torque, caused by friction in the gap, assuming that the vertical load on the squeezed soil is the load transmitted to the upper confining ring due to side friction, and considering the coefficient of friction between the soil and the confining rings.

An estimation of the error due to the friction in the gap can be made as follows:

Considering equation (3.4) for the stress ratio, we have:

$$\frac{\tau}{\sigma_n} = \frac{3M}{2W_n} \frac{r_1 + r_2}{r_1^2 + r_1 r_2 + r_2^2} \quad (\text{A1.1})$$

or

$$\frac{\tau}{\sigma_n} = F(r) \cdot \frac{M}{W_n} \quad (\text{A1.2})$$

where M is the torsional moment on the sample, and
 W_n is the net normal load on it.

Assuming that the normal load on the squeezed soil is the load transmitted to the upper confining ring due to side friction, ΔW , we obtain for the apparent stress ratio:

$$(\tau/\sigma_n)_A = F(r) \cdot \frac{M + \Delta M}{W_n} \quad (\text{A1.3})$$

for the true stress ratio:

$$(\tau/\sigma_n) = F(r) \cdot \frac{M}{W_n} \quad (\text{A1.4})$$

and for the error:

$$\epsilon = \frac{(\tau/\sigma_n)_A - (\tau/\sigma_n)}{(\tau/\sigma_n)} \quad (\text{A1.5})$$

or

$$\epsilon = \frac{\Delta M}{M} \quad (\text{A1.6})$$

where ΔM is the torsional moment due to friction in the gap, which is given as:

$$\Delta M = \int_{r_1 - \Delta r}^{r_1} \tau_G \cdot r (2 \cdot \pi \cdot r) dr + \int_{r_2}^{r_2 + \Delta r} \tau_G \cdot r (2 \cdot \pi \cdot r) dr \quad (\text{A1.7})$$

or

$$\Delta M = \frac{2\pi}{3} [r_1^3 - (r_1 - \Delta r)^3] \tau_g + \frac{2\pi}{3} [(r_2 + \Delta r)^3 - r_2^3] \tau_g \quad (A1.8)$$

where r_1 is the internal radius, equal to 50.8mm,

r_2 is the external radius, equal to 76.2mm,

Δr is the width of the confining rings, which is equal to 1.20mm and 0.90mm for ring shear 1 and 2 respectively, and

τ_g is the shear stress acting between the soil and the confining rings metal (stainless steel)

$$\text{But } \tau_g = \mu \cdot \sigma_g \quad (A1.9)$$

$$\text{or } \tau_g = \mu \cdot \frac{\Delta W}{\pi[(r_2 + \Delta r)^2 - r_2^2] + \pi[(r_1^2 - (r_1 - \Delta r)^2]} \quad (A1.10)$$

where σ_g is the normal stress on the squeezed soil, and

μ is the coefficient of friction between the soil and the confining rings metal.

Substituting the values of r_1 , r_2 and Δr into equations (A1.8) and (A1.10) we obtain:

$$\Delta M = 6.628 \times 10^{-2} \cdot \mu \cdot \Delta W \text{ (KN.m)} \quad \text{for ring shear 1} \quad (A1.11)$$

and

$$\Delta M = 6.622 \times 10^{-2} \cdot \mu \cdot \Delta W \text{ (KN.m)} \quad \text{for ring shear 2} \quad (A1.12)$$

The torsional moment M transmitted to the shear zone of the specimen is:

$$M = 2 W_n \cdot \frac{(\tau/\sigma_n)}{3} \cdot \frac{r_1^2 + r_1 r_2 + r_2^2}{r_1 + r_2} \quad (A1.13)$$

or

$$M = 6.435 \times 10^{-2} (\tau/\sigma_n) \cdot W_n \quad (A1.14)$$

Equations (A1.10), (A1.11) and (A1.13) give:

$$\frac{\Delta M}{M} = 1.030 \frac{\mu}{(\tau/\sigma_n)} \cdot \frac{\Delta W}{W_n} \quad \text{for ring shear 1} \quad (\text{A1.15})$$

and

$$\frac{\Delta M}{M} = 1.029 \frac{\mu}{(\tau/\sigma_n)} \cdot \frac{\Delta W}{W_n} \quad \text{for ring shear 2} \quad (\text{A1.16})$$

or for both apparatus:

$$\epsilon = \frac{\Delta M}{M} = 1.030 \frac{\mu}{(\tau/\sigma_n)} \cdot \frac{\frac{\Delta W}{W_t}}{1 - \frac{\Delta W}{W_t}} \quad (\text{A1.17})$$

where $\frac{\Delta W}{W_t}$ is the ratio of the side friction to the total normal load.

Equation (A1.17) for the error has been plotted in Fig.A1.3 for various values of (τ/σ_n) , $\frac{\Delta W}{W_t}$ and μ .

A1.4.3 Accuracy of the measurements

A1.4.3.1 Torque-measurement

Considering equation 3.7 for the stress ratio, we have:

$$(\tau/\sigma_n) = 3.7298 \frac{(F_1 + F_2)}{W_n} \quad (\text{A1.18})$$

Assuming $F = F_1 = F_2$, we obtain for the apparent stress ratio:

$$(\tau/\sigma_n)_A = 7.4596 \cdot \frac{F_A}{W_n} \quad (\text{A1.19})$$

for the true stress ratio:

$$(\tau/\sigma_n) = 7.4596 \frac{F}{W_n} \quad (\text{A1.20})$$

and for the error:

$$\epsilon = \frac{(\tau/\sigma_n)_A - (\tau/\sigma_n)}{(\tau/\sigma_n)} = \frac{F_A - F}{F} \quad (\text{A1.21})$$

where F_A is the apparent measurement of the particular torque measuring instrument.

Equation (A1.21) shows that the error involved in the computation of stress ratio due to the inaccuracy of the torque-measuring system is the same as the error involved in the measurement of the particular torque measuring instrument. This error is given in Fig A1.4 and A1.5 for ring shear 1 and 2 respectively.

A1.4.3.2 Side-friction measurement

Considering equation A1.2 for the true stress ratio we have:

$$(\tau/\sigma_n) = F(r) \frac{M}{W_n} \quad (A1.22)$$

or

$$(\tau/\sigma_n) = F(r) \frac{M}{W_t - \Delta W} \quad (A1.23)$$

similarly for the apparent stress ratio:

$$(\tau/\sigma_n)_A = F(r) \frac{M}{W_t - \Delta W_A} \quad (A1.24)$$

where ΔW_A is the apparent measurement at the side-friction measuring instrument,

and for the error:

$$\epsilon = \frac{(\tau/\sigma_n)_A - (\tau/\sigma_n)}{(\tau/\sigma_n)} = \frac{\frac{\Delta W_A}{W_t} - \frac{\Delta W}{W_t}}{1 - \frac{\Delta W_A}{W_t}} \quad (A1.25)$$

$$\text{But } \Delta W = \frac{1}{(1 \pm \epsilon_0)} \Delta W_A \quad (A1.26)$$

where ϵ_0 is the error in the side-friction measurement.

Combining equations (A1.25) and (A1.26), we obtain:

$$\epsilon = \frac{\frac{\Delta W_A}{W_t}}{1 - \frac{\Delta W_A}{W_t}} \left(\frac{\pm \epsilon_o}{1 \pm \epsilon_c} \right) \quad (A1.27)$$

Equation (A1.27) expresses the error involved in the computation of stress ratio as a function of the ratio of the side-friction at the upper confining ring to the total load on the sample and the error involved at the particular side-friction measurement. This error is plotted in Fig. A1.6 and A1.7 for the ring shear 1 and 2 respectively.

A1.4.3.3 Pore-water pressure measurement

The maximum error involved in the pore-water pressure measurement is shown in Fig. A1.8.

A1.4.3.4 Displacement measurement

During fast shearing the relative rotational motion of the sample halves, at a given time, was estimated considering the average velocity. This procedure, however, tended to overestimate the relative displacement during the initial stage of fast shearing, because of the time required for the system to acquire constant velocity. An indication of the error involved is shown in Fig. A1.45d and A1.51b for the blue London clay and the fine sand respectively.

Table A1.1 Calibrations of ring shear apparatus

RING SHEAR	MEASURING SYSTEM	INSTRUMENTATION (1)	DATA RECORDING SYSTEM	TYPE RANGE (2)	TYPE RANGE (3)	SENSITIVITY					RANGE OF ΔV ($\approx 10^{-6}$) μV	RESOLUTION
						a_0	a_1	a_2	a_3	a_4		
1	TORQUE [N/V]	PR1(117)	ORION	2	I	-4.8039×10^2	1.5786×10^2	7.1827×10^2			$0 > \Delta V > -150,000$	-4.8×10^{-4} N
				>>	II	-4.8174×10^2	2.3839	2.4373			$-150,000 > \Delta V > -1,000,000$	
				>>	III	-4.8811×10^2	-1.1827	-3.3695			$-1,000,000 > \Delta V > -2,000,000$	
				>>	IV	-4.7420×10^2					$-2,000,000 > \Delta V > -2,900,000$	
	(N/V)	PR2(118)	ORION	2	I	-3.0481×10^2					$0 > \Delta V > -300,000$	3.05×10^{-4} N
					II	-3.0198×10^2	7.8251	2.7322	3.9685×10^{-1}		$-300,000 > \Delta V > -4,000,000$	
	(N/cm)	PR1(117)	U/V	2	SPAN8	1.3388×10^2	2.1033	-1.2773×10^{-1}				2.68 N
				2	SPAN7	6.5345×10^1	6.6485×10^{-1}	-3.6767×10^{-2}				1.31 N
				2	SPAN6	3.3803×10^1						0.68 N
				2	SPAN5	-6.4559×10^{-1}	1.7253×10^1	1.1218	-1.1898×10^{-1}			0.26 N
	(N/cm)	PR2(118)	U/V	2	SPAN4	8.6267						0.43 N
				2	SPAN8	1.0391×10^2	1.1238×10^1	-1.8329	1.1105×10^{-1}			2.08 N
				2	SPAN7	5.5370×10^1	1.8618	-9.2987×10^{-2}				1.11 N
				2	SPAN6	2.9303×10^1	-2.2391×10^{-1}	1.1222×10^{-1}	6.6367×10^{-3}			0.59 N
	SIDE FRICTION [N/V]	PR3 (110)	ORION	2	SPAN5	1.1996×10^1	8.0103×10^{-1}	-3.9181×10^{-2}				0.24 N
				2	SPAN4	7.7943						0.39 N
				2	I-E	3.7690×10^5	-2.3134×10^7	1.7433×10^{10}			$0 < \Delta V < 1,000$	0.36 N
				>>	II-E	3.7425×10^5	-1.4670×10^6	3.4703×10^8			$1,000 < \Delta V < 4,000$	
		PR4 (116)	ORION	2	III-C	3.7324×10^5	5.7715×10^6	3.1318×10^9	5.3891×10^{11}		$0 > \Delta V > -4,000$	-0.37 N
					E	2.3397×10^5						0.23 N

Table A1.1 (continued)

RING SHEAR	MEASURING SYSTEM	INSTRUMENT- TATION (1)	DATA RECORDING SYSTEM	TYPE RANGE (2) (3)	SENSITIVITY					RANGE OF ΔV ($\approx 10^{-6}$) μV	RESOLUTION
					a0	a1	a2	a3	a4		
1	PORE PRESSURE (kPa/V)	PP1(108)	ORION	2 I-E	-1.7550 $\times 10^4$					0< ΔV <5,000	-0.02 kPa
				>> II-C	-1.7392 $\times 10^4$						0.02 kPa
		PP2(109)	ORION	2 I-E	-1.3635 $\times 10^4$	-5.8668 $\times 10^6$	2.9635 $\times 10^9$	-6.4669 $\times 10^{11}$	5.0836 $\times 10^{13}$	0< ΔV <5,000	-0.01 kPa
				>> II-C	-1.4759 $\times 10^4$	8.9358 $\times 10^5$	7.5148 $\times 10^7$			0< ΔV <5,000	0.01 kPa
				>> III-C	-1.8403 $\times 10^4$					0> ΔV >-7,000	0.01 kPa
	VERTICAL DEFORMATION [mm/div.]	DG		2	0.002					-20,000 ΔV >-7,000	-
				2	0.00254						-
											-
											-
											-
2	THERMOCOUPLE [$^{\circ}C/cm$]		U/V	2 SPAN3	1.29						0.03 $^{\circ}C$
				SPAN4	2.52						0.05 $^{\circ}C$
				2 I	0.6200	0.0559					0.68 N
				2 I	2.2100	0.0545					2.26 N
				2 SPAN2	-0.9100	19.3140					0.06 N
	TORQUE (N/ μV)	LC1 (1) LC2 (2)	SOLATRON (4) >>	2 SPAN2	-1.9940	40.6900					0.04 N
				2 SPAN2	-2.4000	23.3870					-1.23 N
				2 SPAN2	-5.0000	49.5400					-2.52 N
				2 I	-1.3500	-3.5161					2.17 N
				2 I	-1.4667	0.2667					-1.20 N
	SIDE FRICTION (N/ μV)	PR3 (3) PR4 (6)	SOLATRON >>	2 I	-1.3500	-3.5161					2.17 N
				2 I	-1.4667	0.2667					-1.20 N

Table A1.1 (continued)

RING SHEAR	MEASURING SYSTEM	INSTRUMENTATION	DATA RECORDING SYSTEM	TYPE	RANGE	SENSITIVITY					RANGE OF ΔV ($\approx 10^{-6}$) μV	RESOLUTION
						a ₀	a ₁	a ₂	a ₃	a ₄		
2	TORQUE (N/V)	LC1a(101)	ORION	2	I	3.2876×10^4	1.1222×10^5	-2.8039×10^6			$0 < \Delta V < 7,758$ $7,758 < \Delta V < 41,699$	0.03 N
				>>	II	3.2439×10^3	8.8567×10^3					
		LC1b(101)	ORION	2	I	2.8600×10^4	2.5337×10^6	-5.5779×10^8	4.1162×10^{10}		$0 < \Delta V < 6,000$ $6,000 < \Delta V < 21,000$	0.03 N
				>>	II	3.3356×10^4	1.3957×10^5	-1.8234×10^7	5.6005×10^8			
	(N/cm)	LC2 (102)	ORION	2	I	3.1960×10^4	5.1734×10^5	-1.1385×10^8	9.2263×10^9		$0 < \Delta V < 7,000$ $7,000 < \Delta V < 23,000$	0.03 N
				>>	II	3.1869×10^4	1.1293×10^5	-2.9926×10^6				
		LC1a(101)	U/V	2	SPAN1	1.0873×10^1	1.9726×10^{-1}	-8.5542×10^{-3}				0.14 N
				>>	SPAN2	2.1760×10^1	3.3051×10^{-1}	-1.7321×10^{-2}				
	(N/cm)	LC1b(101)	U/V	>>	SPAN3	4.5697×10^1	5.4828×10^{-1}	-2.6320×10^{-2}				0.43 N
				>>	SPAN4	9.0163×10^1	7.4846×10^{-1}	-2.0926×10^{-2}				
		LC1b(101)	U/V	2	SPAN1	1.2036×10^1	-1.8027×10^{-2}	2.8643×10^{-3}				0.60 N
				>>	SPAN2	2.1783×10^1	3.4714×10^{-1}	-1.7473×10^{-2}				
2	(N/cm)	LC2 (102)	U/V	>>	SPAN3	4.1596×10^1	3.0326×10^0	-4.6222×10^{-1}				1.1 N
				>>	SPAN4	9.5029×10^1	1.2924					
		LC2 (102)	U/V	2	SPAN1	1.2750×10^1	-3.7804×10^{-2}	1.0359×10^{-2}				0.25 N
				>>	SPAN2	2.2737×10^1	5.1125×10^{-1}	-2.4324×10^{-2}				
	SIDE FRICTION (N/V)	PR6a(103)	ORION	>>	SPAN3	4.8221×10^1	9.8602×10^{-1}	-5.5751×10^{-2}			$0 < \Delta V < 4,000$ $-4,000 < \Delta V < 0$	0.45 N
				>>	SPAN4	9.5029×10^1	1.2924	-5.5549×10^{-2}				
		PR6b(103) (7)	ORION	2	I-E	-3.8766×10^5	3.3596×10^7	-1.3291×10^{13}			$0 < \Delta V < 4,000$ $-4,000 < \Delta V < 0$	-0.39 N
				>>	I-C	-3.7394×10^5	6.0160×10^5					
		PR6b(103) (7)	ORION	2	I-E	-3.4723×10^5	-3.7975×10^6	-1.5348×10^{10}	-6.6499×10^{12}		$0 < \Delta V < 4,000$ $-4,000 < \Delta V < 0$	0.36 N
				>>	I-C	-3.5667×10^5	8.3487×10^6	-4.7761×10^9	5.8503×10^{11}			

Table A1.1 (continued)

RING SHEAR	MEASURING SYSTEM	INSTRUMENTATION	DATA RECORDING SYSTEM	TYPE RANGE	SENSITIVITY				RANGE OF ΔV ($\approx 10^{-6}$) μV	RESOLUTION
					a_0	a_1	a_2	a_3	a_4	
	VERTICAL DEFORMATION (mm/V)	DT1 (104) DT2 (105)	ORION >>	2 2	1.1216×10^3 -7.4825×10^2	-8.2549×10^2 1.1426×10^2				1.1×10^{-3} mm 7.5×10^{-4} mm
	(mm/div.)	DG		2	0.00254					-
	GAP (mm/div.)	DG		2	0.01					-
	POTENTIOMETER (mm/cm)	DT3	U/V	2 SPAN2 SPAN1	1.6505×10^1 1.6678					0.33 mm 0.03 mm

NOTES:

(1) : PR: Proving ring, LC: Load cell, DT: Displacement transducer,
DG: Dial gauge, PP: Pore water pressure transducer

The numbers in parenthesis indicate the channels

(2) : Type 1 $L = a_0 + a_1 R + a_2 R^2 + a_3 R^3 + a_4 R^4$

Type 2 $L = [a_0 + a_1(\Delta R) + a_2(\Delta R)^2 + a_3(\Delta R)^3 + a_4(\Delta R)^4] [(\Delta R)]$

$$\Delta R = R - R_0$$

where R is the recording system reading

R_0 is the recording system reading corresponding to the zero value of the measured physical parameter

L is the measured physical parameter corresponding to the reading R of the recording system

(3) : E stands for extension and C stands for compression

(4) : The solatron data logger was used in Tests 16 and 17. The calibrations of the instrumentation to the solatron data logger are those performed and used by Lemos (1986)

(5) : Calibrations performed and used by Lemos (1986)

(6) : Proving ring used in Test 17 after the accident

(7) : Proving ring used in Test 46

Table A1.2 Specifications of U/V recorder

Number of channels used	: 1, 3, 5, 7, 9
Writing speed	: greater than 30,000 cm/sec
Maximum spot deflection	: 120 mm
Paper drive speeds	: 1, 3, 10, 30, 100, 300, 1000 mm/sec
Paper drive accuracy	: $\pm 5\%$
Paper drive acceleration	: 300 ms maximum
Recording paper	: Direct print 120 mm width × 30.5 mm length Kodak 1895/2022
Timing speeds	: 0.01, 0.1, 1.0, 10.0 seconds
Timing accuracy	: $\pm 1\%$
Light source	: 6 volt 10 watt Quartz Halogen lamp

Table A1.3 Characteristics of miniature silicon diaphragm pressure transducers

Overall length	: 11.4 mm
Overall diameter	: 6.3 mm
Weight (excluding cable)	: 2 gr
Transduction principle	: 4 arm silicon strain gauge
Output voltage (FSD)	: 200 mV / 10 V energization
Pressure range	: 0 - 1380 kPa

Table A1.4 Specifications of fast motors

No.	Type of motor	Speed	Shaft speed (RPM)	Nominal rate of displacement at sample (mm/min)
M3	single phase A/C	constant	14.25	133
M4 ⁽¹⁾	three phase A/C	variable	10-100	93.33-933.33
M5	three phase A/C	variable	120-600	1120-5600

NOTES:

(1) : Motor M4 is controlled by a single to three phase speed controller

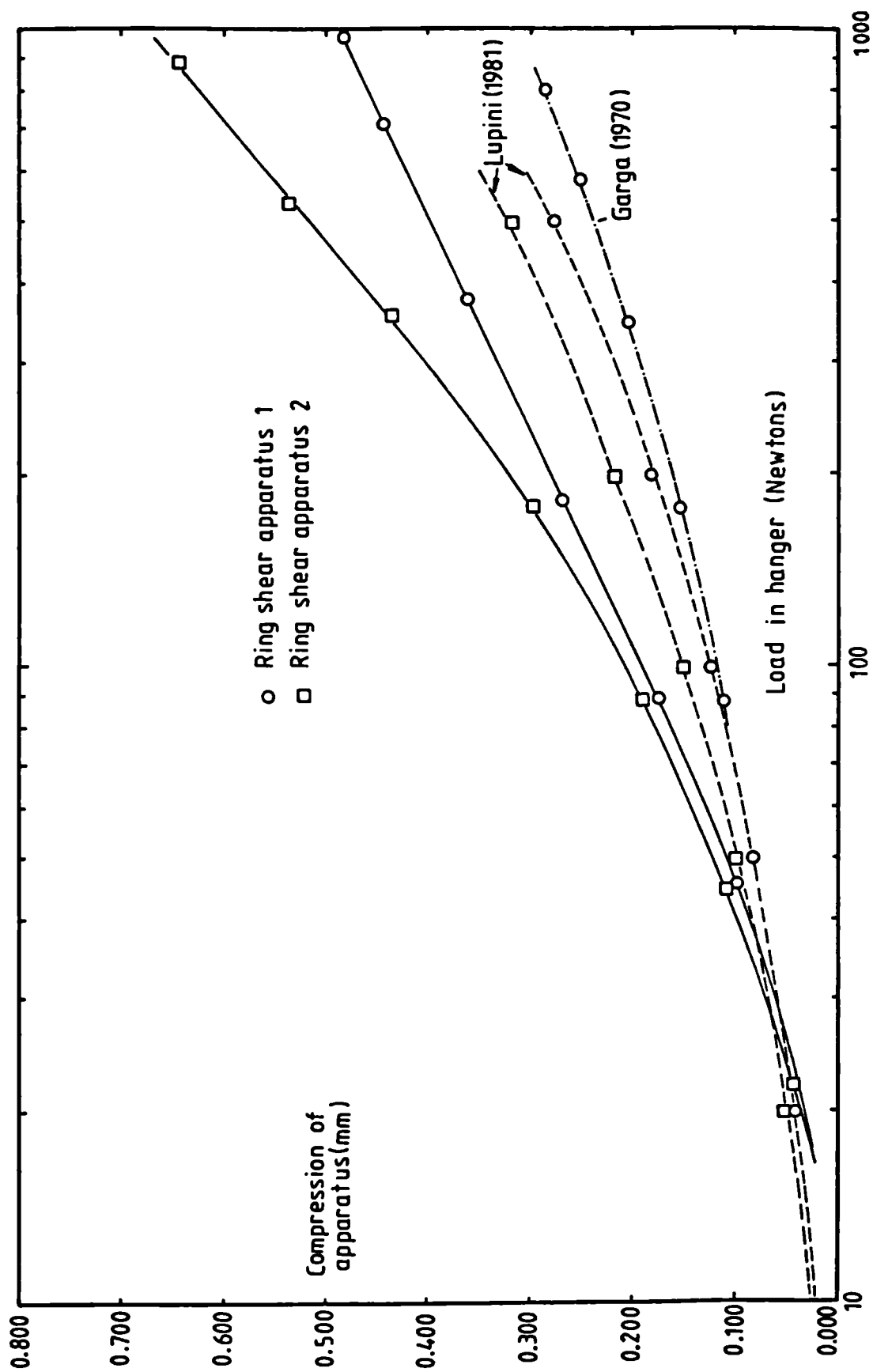


Fig. A1.1 Vertical compressions of the ring shear apparatus

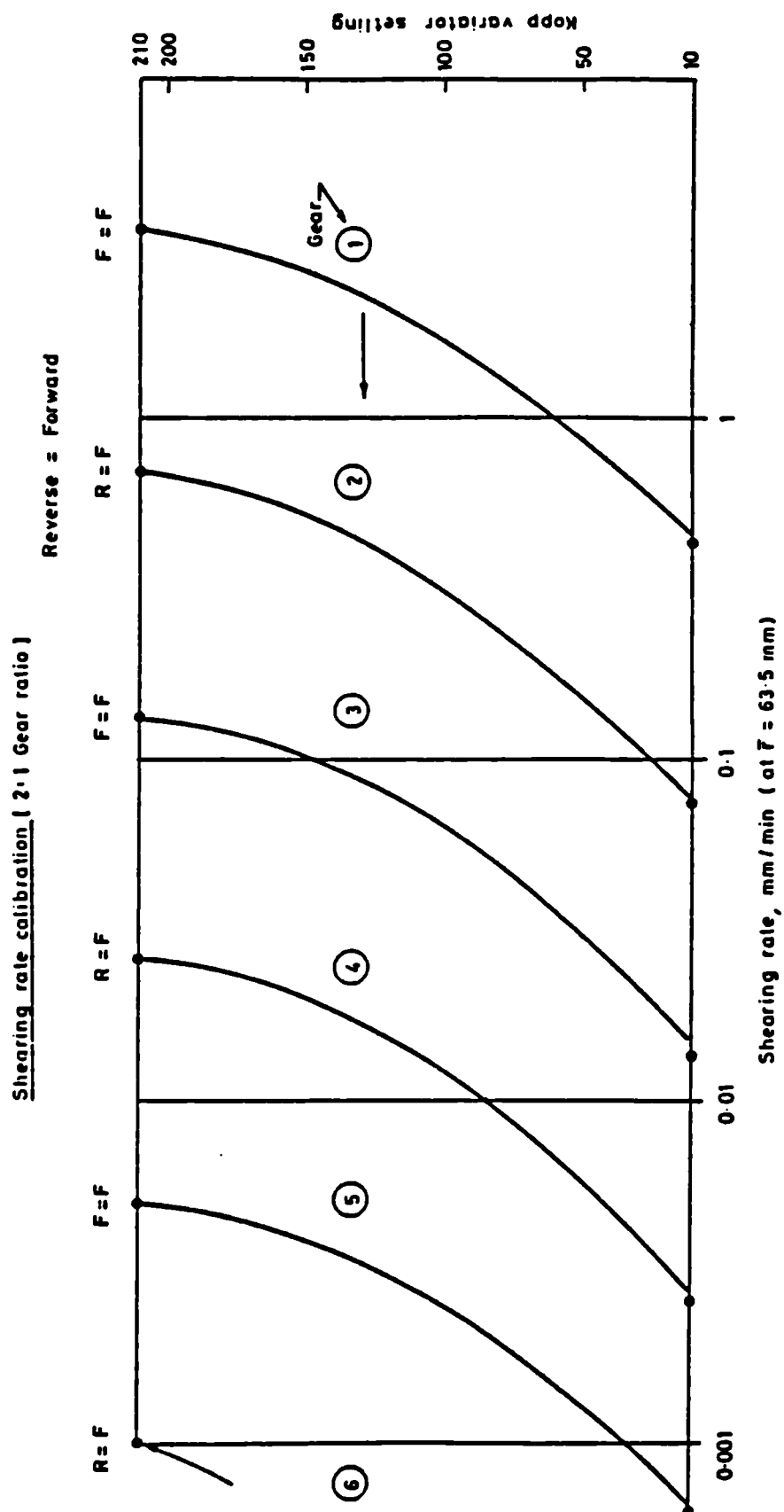


Fig. A1.2a Calibration of motor M1 (ring shear 1, Lupini 1981)

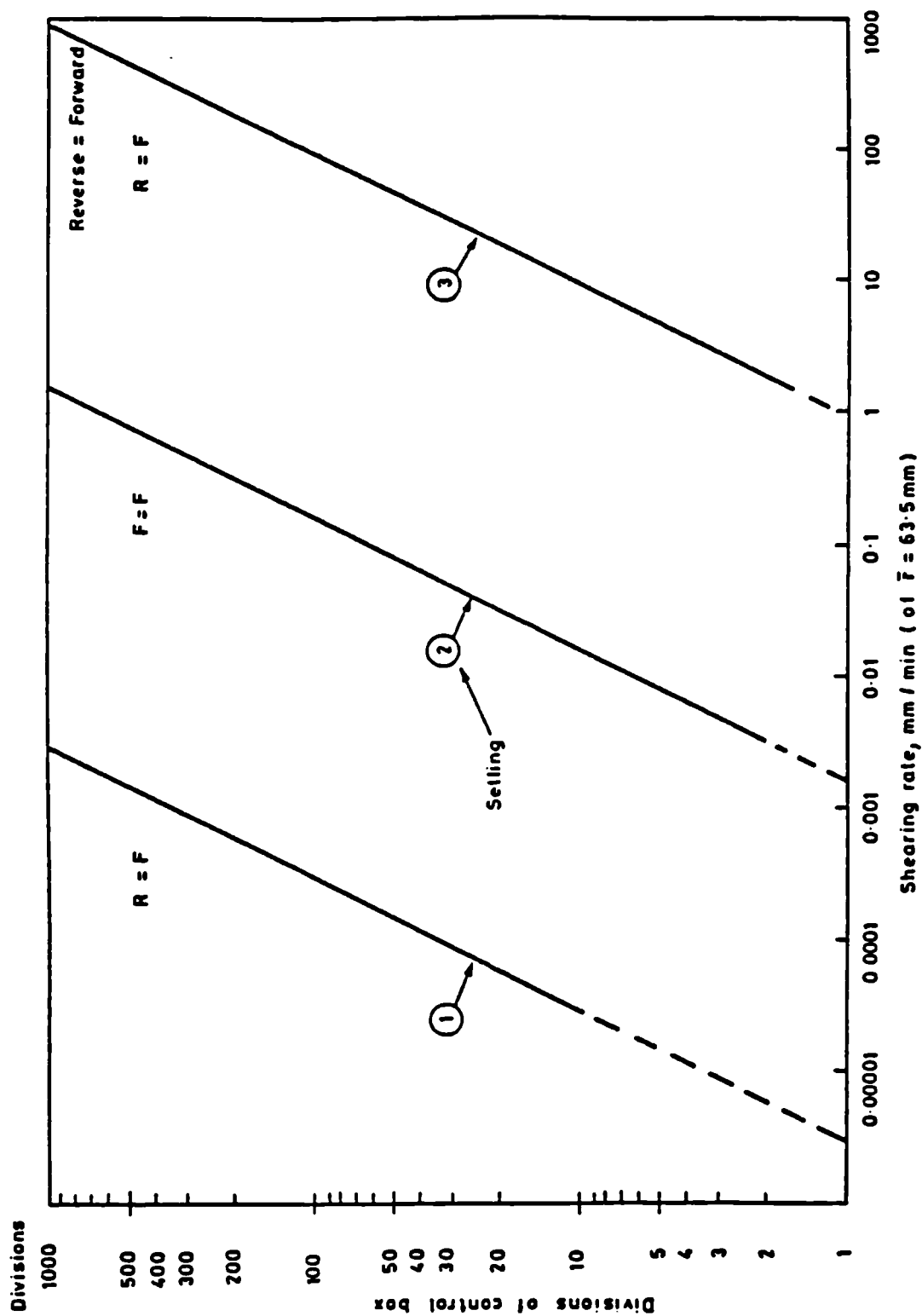


Fig. A1.2b Calibration of motor M2 (ring shear 2, Lupini 1981)

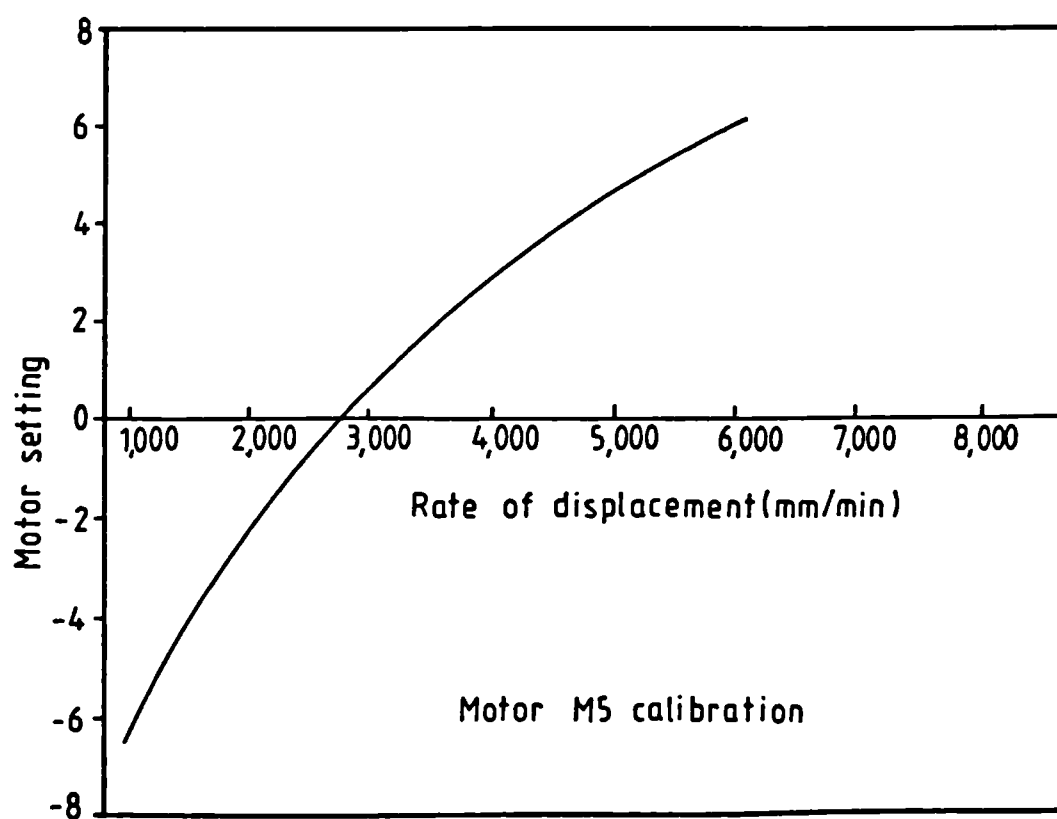
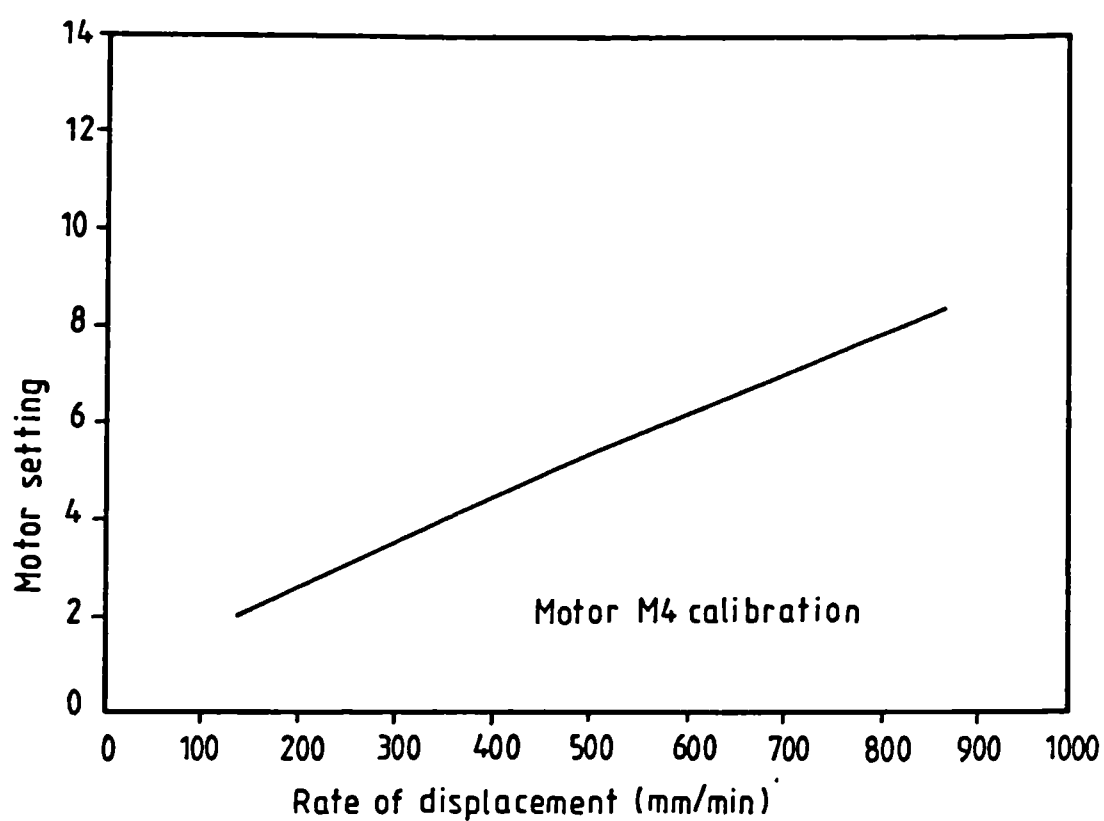


Fig. A1.2c Calibration of fast motors M4 and M5

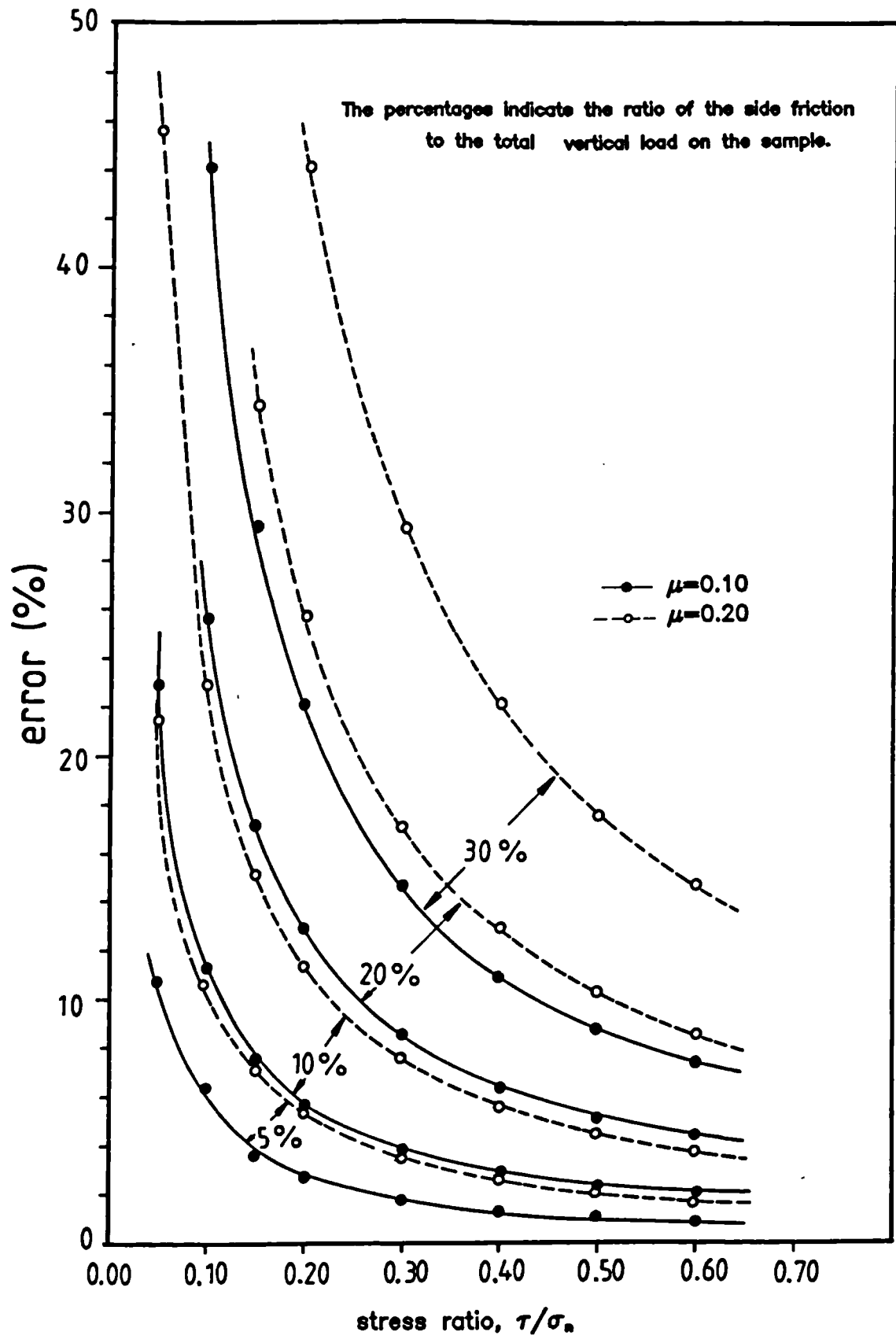


Fig. A1.3 Error due to friction in the gap

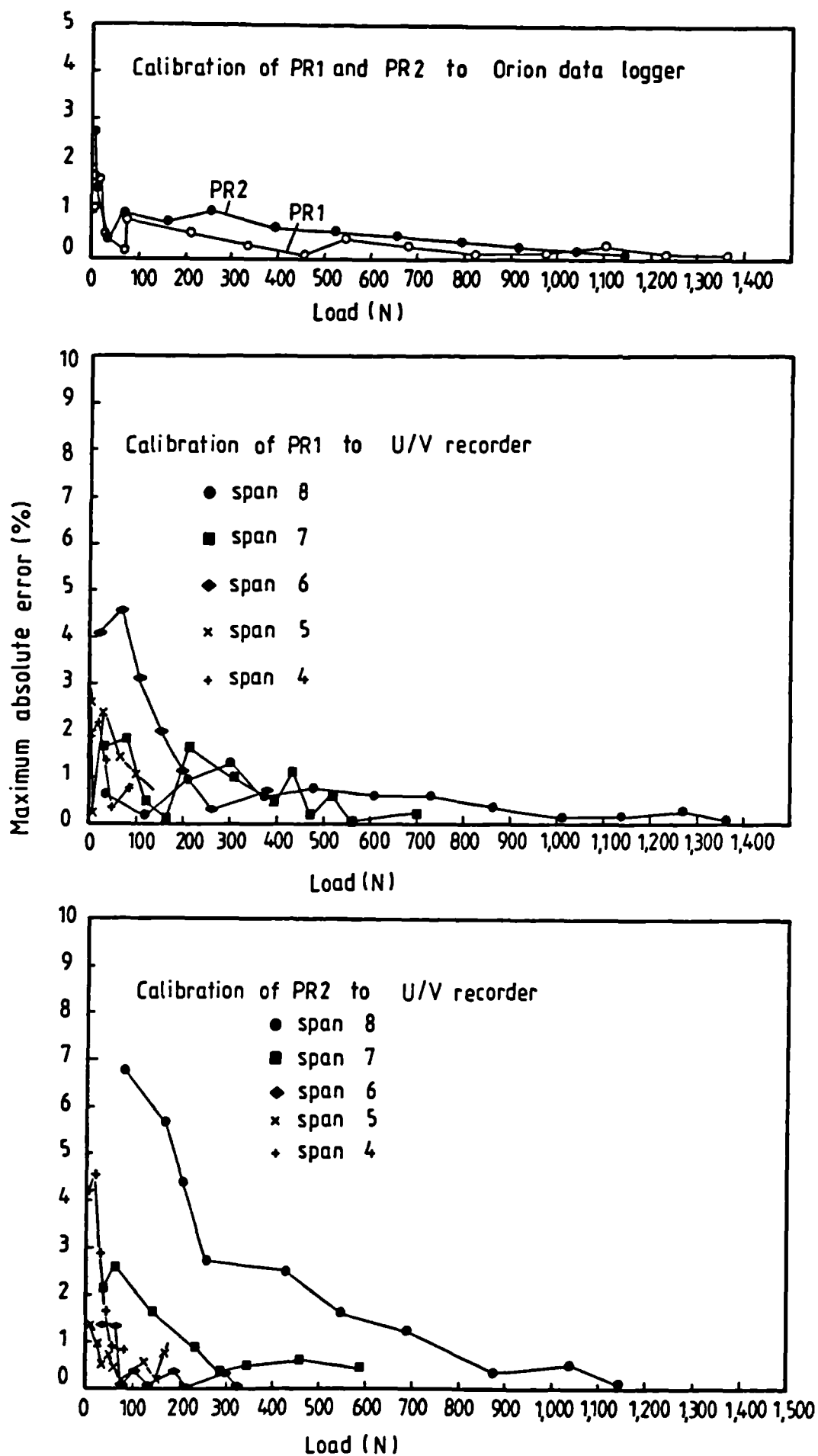


Fig. A1. 4 Maximum absolute error due to torque-measurement (ring shear 1)

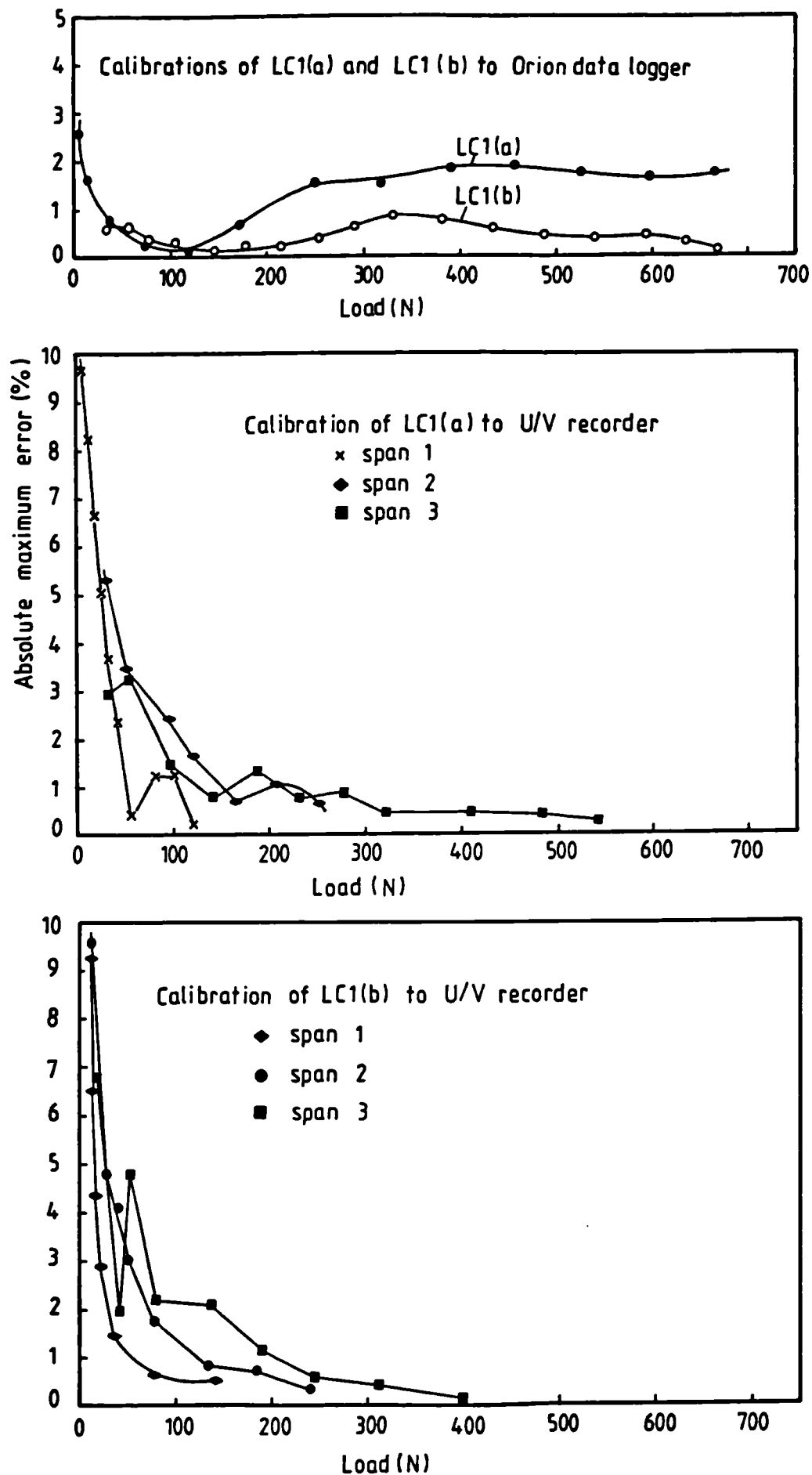


Fig. A1.5a Maximum absolute error due to torque measurement
(ring shear 2)

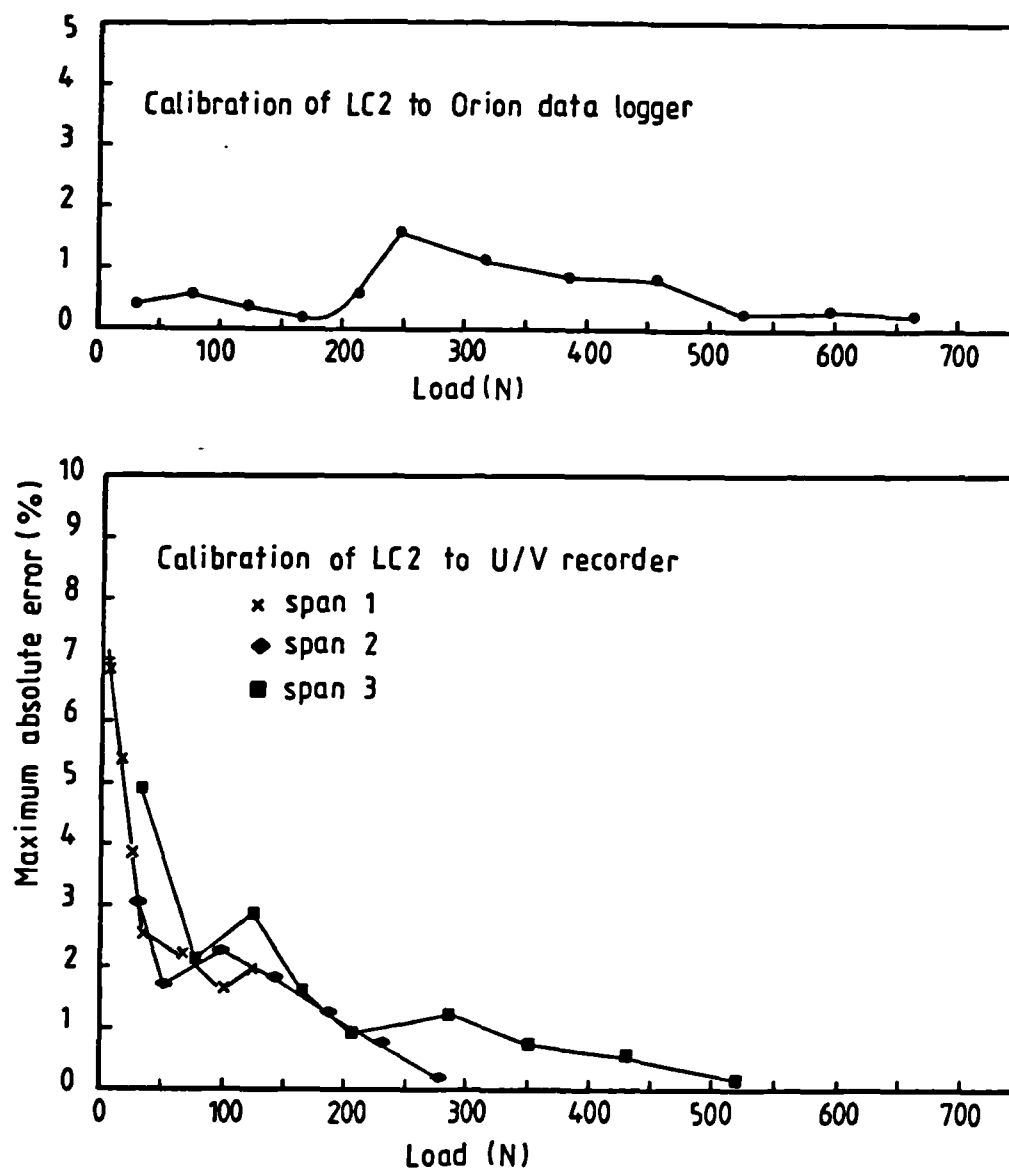


Fig. A1.5b Maximum absolute error due to torque-measurement (ring shear 2)

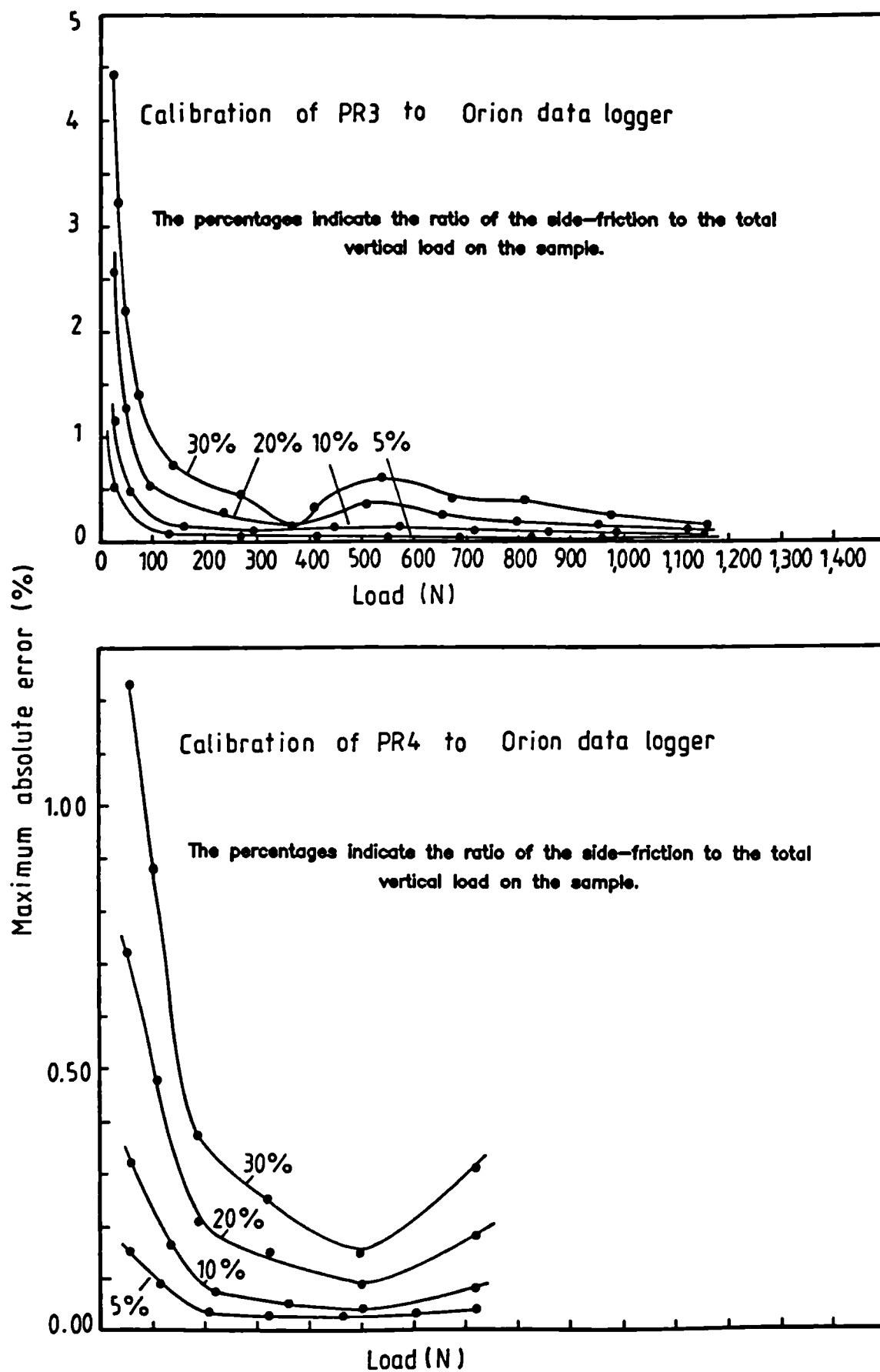


Fig. A1.6 Maximum absolute error due to the side-friction measurement (ring shear 1)

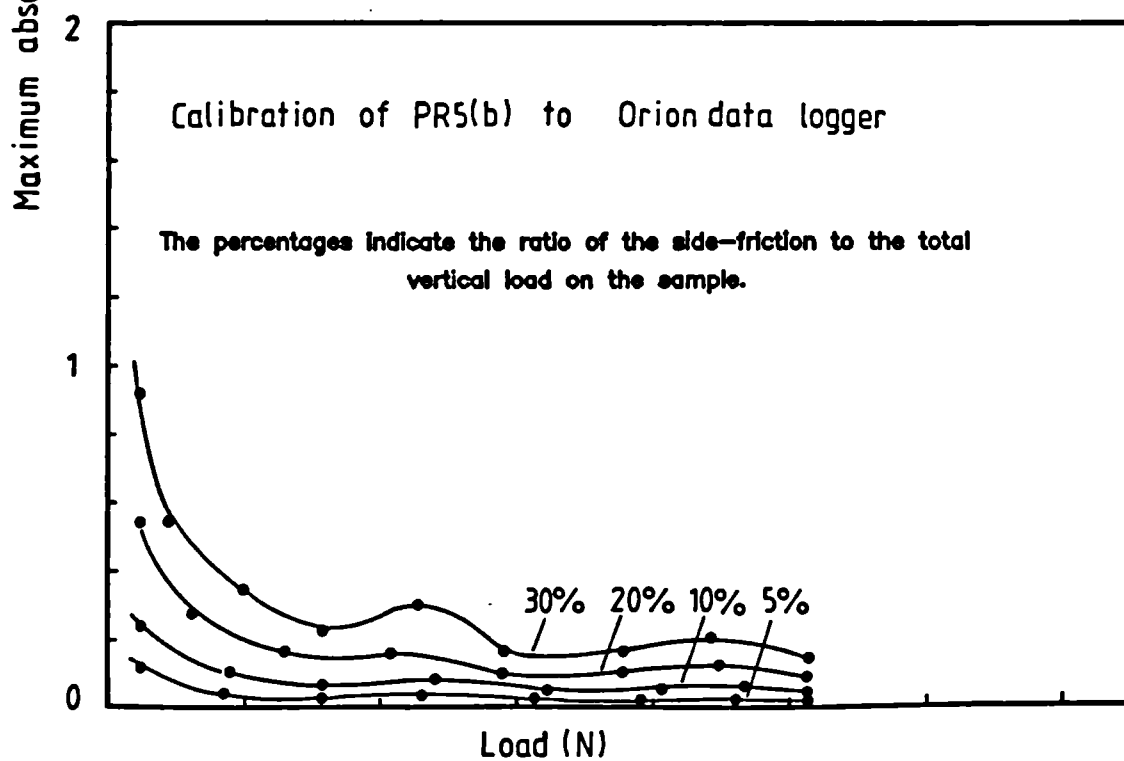
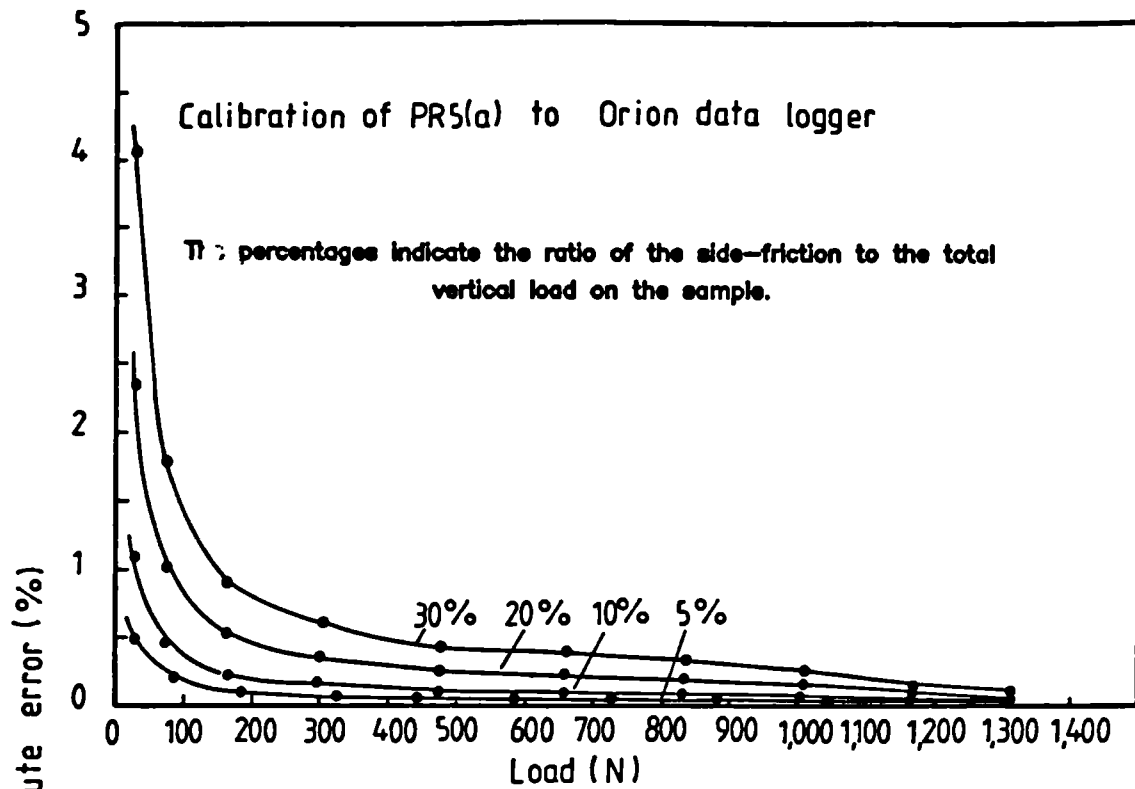


Fig. A1.7 Maximum absolute error due to the side-friction measurement (ring shear 2)

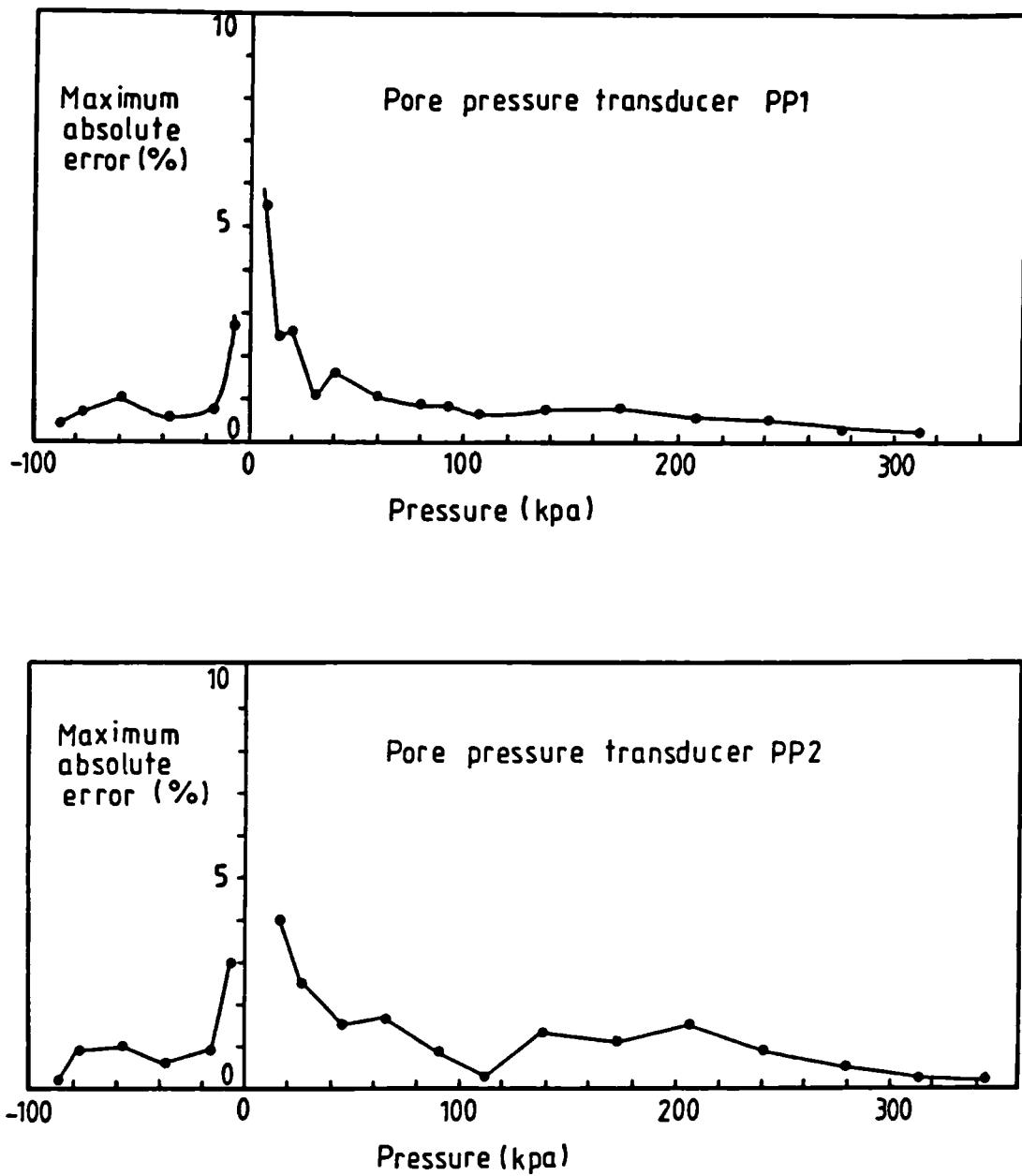


Fig. A1.8 Maximum absolute error in pore-water pressure measurement

APPENDIX 2

PRESENTATION OF RING SHEAR TESTS

A2.1 . SOIL FROM KALABAGH DAM SITE

Site Geology

The foundations at the Kalabagh dam site consisted of Middle Siwaliks (late Pliocene) formations. They included thick beds of sandstone alternating with finer-grained beds of claystone and siltstone. The strata are heavily overconsolidated freshwater deposits and, owing to tectonic folding, most of the claystones contain bedding shears while thrust joints (many of them sheared) characterise the siltstones (Skempton, 1985).

Tests 1,2 , claystone A

Operator : Lemos, L.J.L

Reference : Lemos (1986)

The results of two ring shear tests on a claystone from the Kalabagh dam site are considered here.

Soil description

The soil received was light brown in colour. It had an initial dry unit weight of 19.87 kN/m³ which corresponds to a water content of 9.5% for 100% saturation.

Mineralogy and chemical analysis

Analysis of the mineralogy of the claystone gave the following results (Skempton and Lemos, 1984):

clay minerals	50% [predominantly illite + Kaolinite]
Quartz	26%
Calcite	4%
Muscovite, biotite	9%
Chlorite	4%
Haemonite, limonite	3%
Feldspar	2%

Chemical analysis showed about 6% of calcium carbonate[CaCO₃].

Index properties

The index properties were determined on samples, prepared in three different ways:

1. The sample was shredded and soaked in distilled water for twenty four hours. It was then dried in order to remove the excess water. The sample was then worked on a glass plate with a steel spatula until no further reduction in the size of the soil grains could be observed.
2. The sample was shredded and soaked in distilled water for forty eight hours. The slurry formed was placed in a high speed liquifier for ten minutes. The sample was then dried to remove the excess water and worked on a glass plate with a steel spatula.
3. The soil came from the sample tested in the ring shear apparatus.

		(%)			
Initial	{	Method 1	: LL=60,	PL=27,	PI=33
		Method 2	: LL=71,	PL=28,	PI=43
Final	{	Method 3	: LL=62,	PL=26,	PI=36

The results from the initial and final index tests are similar, indicating that no significant change in the properties of the samples occurred over the twenty four days duration of the test (as can happen in chemically active soils due to leaching by the distilled water bath).

Gradings

Particle size distributions were determined following each of the sample preparation procedures listed below.

1. As method 1 above.
2. As method 2 above.
3. From soil that was air dried, and mechanically ground in a soil mixer.

4. From soil lost through the gap during the ring shear test.
5. From soil from the shear zone of the sample at the end of the ring shear test.
6. From soil, prepared initially as in procedure 1, pretreated with H_2O_2 and HCl, dispersed, and stirred for 15 min.

Method	sand %	silt %	<2 μ m %	<5 μ m %	pretreatment losses %
1	-	60	40	65	-
2	5	48	47	73	-
3	3	50	47	75	-
4	-	47	53	77	-
5	-	49	51	80	-
6	-	60	40	74	10
>>	-	58	42	74	11.4
>>	6	59	35	64	9.9
>>	7	58	35	67	-

The grading curve which was determined by using the third method is shown in Fig. A2.1. The clay fraction showed some tendency to decrease when the samples were pretreated with H_2O_2 and HCl (normal solution). The loss in weight on pretreatment is largely due to calcium carbonate being dissolved by the acid (Skempton and Lemos, 1984). This indicates that the calcium carbonate is concentrated within the silt and clay grades, and that the dispersing agent was fully efficient with or without pretreatment.

Sample preparation

The soil had been cut from a block sample and was in an air-dry condition. It was shredded, soaked in distilled water, and then subjected to three or more cycles of wetting and drying. On the last wetting cycle it was left for two days soaked in distilled water, and then it was thoroughly worked using a spatula on a glass plate. This work with the spatula was repeated two or three times, each lasting about ten to fifteen minutes. Any gravel size nodules, which

consistuted only a small proportion of the whole sample, were removed. The water was evaporated to give a water content just above the plastic limit, with the sample being mixed from time to time in order to obtain a uniform condition.

Test 1

Test results

The soil was placed in the apparatus at a water content of 41%. It was then consolidated to a normal stress of 900 kPa and swelled to 525 kPa. The compression during the consolidation stage was equivalent to a decrease in the water content of about 10%. From the consolidation stage, it was estimated that a rate of displacement of 0.017 mm/min would ensure 95% dissipation of pore water pressure in a displacement of 2mm. The slow standard rate of displacement, used throughout the test, was 0.01 mm/min.

Results from the shearing stages are summarized in Table A2.1 and the stress ratio versus cumulative displacement is plotted in Fig. A2.2a and A2.2b.

Stage 1 involved slow shearing through the peak to the residual strength and it was then followed by stages 2 to 5, carried out at alternating fast and slow rates of displacement. At the end of stage 5, the normal stress on the sample was reduced to 206 kPa.

Shearing was commenced again in slow stage 6 and continued at alternating fast and slow rates of displacement during stages 7 to 15. Stages 9 and 13 were both carried out at 400 mm/min with (stage 8) and without (stage 13) the slow residual state having been previously established respectively. The intention was to investigate the dependence of the threshold strength on the static strength, mobilized prior to fast shearing. The test terminated with the fast stage 15.

The water content of the sample at the end of the test was as follows:

average : 27.2%

near the shear surface : 27.6%

Description of the test specimen

The shear surface was well polished and heavily striated in the

direction of shear. The striations were probably caused by small lumps of drier soil (1-2mm in diameter) which were seen to be present in modest numbers and which were a product of the method of initial sample preparation.

Test 2

Test results

After the soil had been placed in the apparatus, it was consolidated to a normal stress of 988 kPa and swelled to 241 kPa.

The stress ratio versus cumulative displacement is plotted in Fig.A2.3 and results from the test are presented in Table A2.2.

Stage 1 involved shearing at 400 mm/min in order to measure the threshold strength of the unsheared remoulded sample. Stage 2 followed at 0.01 mm/min. After stage 2, the sample was consolidated to 987 kPa and swelled to 593 kPa.

Slow shearing was commenced again in stage 3. A slight increase of the stress ratio with displacement was observed at the end of stage 3, probably due to the build up of friction in the gap. No recording was performed during stage 4, carried out at 400 mm/min. The slow residual strength was established again during stage 5. The stress ratio was still decreasing with displacement when fast shearing at 400 mm/min during the last shearing stage 6 was performed.

The water content of the sample at the end of the test was as follows:

shear zone :	25.5%
average :	25.6%

Description of the test specimen

The shear surface was striated circumferentially and polished. Observations of thin sections, prepared from the test specimen, under an optical-microscope showed a shear zone with the clay particles orientated parallel to the shear direction. Some shears were orientated at 20° to 25° with respect to the principal displacement shear.

Tests 3,4,5,6,7,8 , clayely siltstone A

Operator : Lemos L.J.L.

Reference : Lemos (1986)

General

The results of six ring shear tests on a light brown clayely siltstone are considered here. This soil was the first tested soil from the Kalabagh dam site, showing a significant drop of strength below the slow residual during fast shearing. This unusual behaviour, observed during fast shearing at 400 mm/min in Test 3, prompted the intensive investigation of the behaviour of the clayely siltstone. The purpose of carrying out Test 4 was to investigate the effect of initial water content on the drop of strength to less than the slow residual strength during fast shearing. Test 5 involved first shearing at 400 mm/min. In Test 6 the effect of the compliance of the proving ring, measuring the side-friction (and controlling the size of gap), on the behaviour of soil during fast shearing was assessed. The proving ring was made stiffer during fast shearing by clamping its deflecting sides, as described in section 3.2.3. Test 7 was performed in order to investigate the build-up of shearing resistance immediately after fast shearing, using a slow rate of displacement. The technique of stiffening the proving ring, measuring the side friction, was also used to controll the size of the gap during fast shearing. Finally, Test 8 involved soil shearing against a rough glass interface. A thermocouple and piezometric probe were installed in the interface in order to measure both the temperature and the pore water pressure during fast shearing. Both instruments, however, failed to give meaningful results because of problems, associated with their setting.

Mineralogy and Chemical Analysis

Analysis of the mineralogy of the clayely siltstone gave the following results (Skempton and Lemos, 1984):

Clay minerals	40%	(predominantly illite + kaolinite)
Quartz	28%	
Calcite	4%	
Muscovite, biotite	4%	
Chlorite	3%	
Haemonite, Limonite	3%	

Feldspar 6%

Chemical analysis showed 4% of calcium carbonate $[CaCO_3]$

Sample Preparation

The soil had been cut from a block of sample and it was in an air-dry condition. It was divided into two batches (one batch : Tests 3,4,5, and another batch: Tests 6,7,8) and prepared in exactly the same way as the soil used for Tests 1 and 2.

Index properties

The index properties of the soil, prepared as for the ring shear tests, were as follows:

LL=39, PL=21, PI=18%

Gradings

Particle size distributions were determined preparing the sample in different ways:

1. The soil was prepared in the same manner as the samples for the ring shear tests, and then it was dispersed and shaken by hand.
2. The soil was prepared in the same manner as the samples for the ring shear tests, and then it was pretreated, dispersed, and stirred using a high speed liquefier for fifteen minutes.
3. Soil from the Test 6 specimen was dispersed.

Method	sand %	silt %	<2 μ m %	<5 μ m %	pretreatment losses %
1	22	58	20	32	-
2	18	61	21	37	2.9
	30	58	12	25	-
3	18	55	27	45	-

The grading curves which were determined by using the second and third method are shown in Fig. A2.1a

Test 3

Test results

The soil was placed in the apparatus at a water content of 28%. It was then consolidated to a normal stress of 922 kPa and swelled to 593 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.04 mm/min would ensure 95% dissipation of pore water pressure during shear in a displacement of 1mm. The slow standard rate of displacement, used throughout the test, was 0.01 mm/min.

Each shearing stage is summarized in Table A2.3. The stress-ratio versus cumulative displacement is shown in Fig. A2.4a, A2.4b and A2.4c.

Stage 1 involved slow shearing through the peak to the residual strength and it was followed by the series of alternating slow and fast stages 2 to 7. At the end of stage 7, the normal stress was reduced to 242 kPa.

Shearing was commenced again in stage 8, carried out at 0.01 mm/min and 0.05 mm/min. The gap was kept closed during shearing at 0.05 mm/min. The stress ratio increased initially and then attained a steady value. It increased slightly later, possibly due to the friction build-up in the gap. The remaining stages of the test, 9 to 17, were carried out at alternating slow and fast rates of displacement. Stage 17, carried out at 400 mm/min with the gap closed, was performed in order to assess the influence of the loss of soil through the gap on the drop of strength below the slow residual strength during fast shearing. Though the amount of soil loss was reduced and some friction in the gap might have been developed, a significant loss of strength was observed.

The final water content of the sample was as follows:

shear zone	:	20.0%
average	:	20.5%

Description of the test specimen

The shear surface was strongly striated in the direction of shear. The striations appeared to be polished. They had been probably caused by small lumps of drier soil.

Test 4

Test results

The soil was placed in the apparatus at a water content of 20.8% (as opposed to 28% in Test 3). It was then consolidated to a normal stress of 878 kPa and swelled to 557 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.04 mm/min would ensure 95% dissipation of pore water pressure in 1mm travel.

The stress ratio versus cumulative displacement is plotted in Fig. A2.5 and test results are presented in Table A2.4.

Shearing of the sample involved two stages. Stage 1 involved shearing through the peak to the residual strength at 0.01 mm/min. Stage 2 was carried out at 400 mm/min

The only apparent effect of the lower initial water content on the stress ratio-displacement curve during fast shearing was the increase of the displacement at which a significant drop of strength to less than the slow residual strength was observed.

The final water content of the sample was as follows:

shear zone	:	18.7%
average	:	19.3%

Description of the test specimen

It was difficult to separate the sample through a shear surface, because it crumbled easily. A thin zone which was darker than the rest of the sample was present. A shear surface was also observed during the visual inspection. It was not polished, probably because the last stage of the test involved fast shearing.

Test 5

Test results

The sample was consolidated to a normal stress of 878 kPa and swelled to 241 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.04 mm/min would ensure 95% dissipation of pore water in a displacement of 1mm.

The stress ratio versus cumulative displacement is plotted in Fig.

A2.6a A2.6b and A2.6c and the results from the test are presented in Table A2.5.

Stages 1 to 6 involved shearing at 400mm/min. A pause was allowed after each stage for the dissipation of pore water pressure, generated during fast shearing. This pause varied from thirty minutes (time slightly higher than that required for 95% dissipation of pore water pressure) to forty eight hours. The sample was unloaded in shear while at rest.

Stages 7 and 8 involved pulse loading at 133 mm/min with a pause of a few seconds between the pulses. The intention was to simulate earthquake loading and observe its effect on the stress ratio-displacement curve. The rate of displacement of 133 mm/min was chosen because it could be imposed to the sample by means of the fast motor M1 enabling a quick speed-up.

Stage 9 involved shearing at 0.0022 mm/min for a displacement of 2mm only. The test ended with stage 10, carried out at 133 mm/min.

The final water content of the sample at the end of the test was as follows:

average : 20.4%
shear zone : 20.3%

Test 6

In this test the technique, described in section 3.2.3, was used for controlling the size of the gap during fast shearing. During the slow stages preceding the fast shearing the deflection sides of the proving ring, measuring the side friction, were clamped together, so that the size of the gap would remain relatively constant during fast shearing and the loss of soil through the gap would be minimized.

Test results

The soil was placed in the apparatus at a water content of 26.5%. It was then consolidated to a normal stress of 988 kPa and swelled to 593 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.04 mm/min would ensure 95% dissipation of pore water pressure in a displacement of 1mm. A rate of displacement of 0.01

mm/min was used as the slow standard throughout the test.

The stress ratio versus cumulative displacement is plotted in Fig. A2.7 and the results from the test are presented in Table A2.6.

Shearing through the peak to the residual strength was performed at the slow rates of displacement of 0.0077, 0.051 and 0.01 mm/min during stage 1 and continued at 400 and 0.01 mm/min alternatively during the remaining stages of the test.

The slow residual coefficient of friction ($\tan\phi'_r = 0.255-0.265$), measured in this test, was lower than that determined in the previous tests on the clayey siltstone A (i.e ,Test 3 : $\tan\phi'_r = 0.330-0.360$). Index properties and a grading (Fig.A2.1) of the sample, determined at the end of the test, indicated a higher plasticity for the soil used in this test.

Test 6 (end) :	LL=49%,	PL=22%,
Initially determined :	LL=39%,	PL=21%,

The stress ratio-displacement curve during fast shearing, was unaffected by the use of a stiffer proving ring, which kept the size of the gap between the confining rings relatively constant.

The final water content of the sample was as follows:

Shear zone :	19.4%
average :	19.6%

Test 7

Test results

The soil was placed in the apparatus at a water content of 24.6%. It was then consolidated to a normal stress of 1010 kPa and swelled to 593 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.04 mm/min would ensure 95% dissipation of pore water pressure in a displacement of 1mm.

The shearing stages are summarized in Table A2.7 and the stress ratio versus cumulative displacement is plotted in Fig A2.8a and A2.8b.

Stage 1 involved shearing at 0.004 mm/min for a displacement of 2.4mm. The peak strength had not been yet mobilized, when stage 2 was performed at 400 mm/min. Stage 3 followed immediately after stage 2 at 0.36 mm/min. The rate of 0.36 mm/min was chosen because it was the slowest rate that could be applied immediately after fast shearing without stopping the motor. The slow residual strength was established during stage 4.

Stage 5 involved four pulses at 400 mm/min to simulate earthquake loading. The motor was kept running at a constant speed and was connected to or disconnected from the worm drive of the apparatus. Stage 6 was carried-out at 0.35 mm/min immediately after stage 5. Stage 7 involved shearing at 0.006 mm/min. The slow residual strength had not been yet established, when stage 8 was carried out.

Stage 8 involved ten pulses at 400 mm/min. After the sample had been relieved from the shear load, shearing started at 1.55 mm/min immediately after stage 8. The rate of 1.55 mm/min was chosen so that the residual strength of the soil would be mobilized in the shortest time. The rate of displacement was reduced to 0.007 mm/min in stage 10 following immediately after stage 9. At the end of stage 10, the clamps were removed. Shearing ended with stage 11, carried out at 0.01 mm/min.

Lemos (1986) concluded that the time needed to mobilize the residual strength during slow shearing, performed immediately after a fast stage was independent of the shearing rate and approximately equal to the consolidation time of the soil.

The final water content of the sample, determined at the end of the test, was:

average : 18.9%
shear zone : 19.1%

Test 8

Interface : Glass

Initial roughness : C.L.A = 8.50 μ m

Test results

The soil was placed in the apparatus at a water content of 26%. It was then consolidated to a normal stress of 590 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.01 mm/min would ensure 95% dissipation of pore water pressure in a 1mm displacement. A rate of 0.006 mm/min was used as the slow standard throughout the test.

The stress ratio versus cumulative displacement is plotted in Fig A2.9a and A2.9b. Each shearing stage is summarized in Table A2.8.

Stage 1 involved shearing at 400 mm/min. The slow residual strength was established in stage 2. Stages 3 and 4 were carried out at a fast and slow rate of displacement respectively. The normal stress was reduced to 165 kPa and 80 kPa in the remaining slow stages 5 and 6 respectively. After stage 6, the sample was removed from the apparatus.

The final water content of the sample was as follows :

average : 20.5%

shear zone : 21.5%

Description of the test specimen

The sample when pulled away from the interface separated quite easily. The surface in contact with the interface was not polished. A well defined, polished and slightly striated shear surface was observed approximately 0.2mm away from the interface.

Microstructural observations

Lemos (1986) carried out an intensive investigation of the microstructure of clayely siltstone A. He concluded that during fast shearing a powerful remoulding action takes place, leaving the clay matrix in a loose state. Pre-existing shear surfaces were disrupted by round massive particles and the preferred orientation of the clay particles was disturbed. For a detailed description of the microstructure observations, reference should be made to Lemos (1986).

Test 9, claystone B

Operator : Lemos L.J.L

Reference : Lemos (1986)

Mineralogy and chemical analysis

Analysis of the mineralogy of the claystone gave the following results (Skempton and Lemos 1984):

Clay minerals	38% [predominantly illite + kaolinite]
Quartz	6%
Calcite	50% [carbonate, as concretions and nodules]
Muscovite, biotite	1%
Chlorite	1%
Haemonite, limonite	2%
Feldspar	1%

Chemical analysis showed about 16% of calcium carbonate

Sample preparation

The sample, as received, had been cut from a block sample and was in an air-dry condition. It was shredded and soaked in distilled water and then subjected to three or more cycles of wetting and drying. On the last wetting cycle it was left for two days soaked in distilled water and thoroughly worked using a spatula on a glass plate. During this stage only gravel-size nodules were removed. They constituted a small proportion of the sample. After working with the spatula, the soil was wet-sieved through a 63 μ m sieve in order to remove only coarse sand size particles or aggregates which had caused some rutting on the shear surface of the previously tested samples. These were added to the larger nodules already removed, and then weighed. As a proportion by weight, all the material coarser than 63 μ m constituted approximately 6%. The wet-sieved material was then allowed to dry by evaporation to a water content above the plastic limit and was then tested in the ring shear apparatus.

Index properties

The index properties of the soil, prepared as for the ring shear test, were as follows:

LL=56.2, PL=25.0, PI=31.2%

Gratings

Particle size distributions were determined following different ways of sample preparation:

1. The soil was prepared as for the ring shear test, and then was dispersed and shaken for at least half an hour
2. The soil was prepared as for the ring shear test, and then was pretreated (with H_2O_2 and HCl), dispersed and stirred for fifteen minutes.

Method	sand %	silt %	<2 μ m %	<5 μ m %	pretreatment losses %
1	14	49	36	57	-
2	11	49	40	65	9.7
>>	11	46	43	68	-

The grading curve which was determined by using the second method is shown in Fig.A2.1.

Test results

The soil was placed in the apparatus at a water content of 28.9%. It was then consolidated to a normal stress of 987 kPa and swelled to 214 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.02 mm/min, would ensure 95% dissipation of the pore water pressure in a displacement of 1mm. The slow standard rate of displacement, used throughout the test, was 0.01 mm/min.

Each shearing stage is summarized in Table A2.9 and the stress ratio versus cumulative displacement is shown in Fig A2.10a to A2.10d.

Shearing was commenced at 400 mm/min in stage 1 and continued with the series of stages 1 to 8, carried out at alternating slow and fast rates of displacement. The slow residual coefficient of friction kept decreasing during stages 4,6 and 8. After stage 8, the normal stress on the sample was increased to 535 kPa.

Shearing was commenced again in slow stage 9. The remaining stages of the test, 10 to 18, involved alternating slow and fast shearing.

The final water content of the sample was as follows:

shear zone : 22.4%

average : 21.6%

Description of the test specimen

The sample separated on a polished, striated, single surface. Round particles, having a diameter of approximately 0.5 mm, existed near the shear surface, which was 3 to 4 mm away from the blades of the top porous stone. The shear zone had a different colour from the rest of the sample.

A thin section, viewed under polarized light, showed a shear zone approximately 1 mm thick, bounded by two discontinuous shear surfaces. The clay particles within the shear zone were orientated at 10^0 to 15^0 with respect to the shear direction. For a detailed description of the observations, reference should be made to Lemos (1986).

Test 10, claystone C

Operator : Lemos, L.J.L

Reference : Lemos (1986)

Sample preparation

The sample, as received, had been cut from a block sample and was in an air-dry condition. It was prepared in exactly the same way as claystone B. The proportion by weight of the material that was coarser than $63\mu\text{m}$ was 1% approximately.

Index properties

The index properties of the soil, prepared in the same manner as the sample for the ring shear test, were as follows:

LL=54.6, PL=27.5 PI=27.1

Gradings

Particle-size distributions of the soil, prepared as for the ring shear test, pretreated with H_2O_2 and HCl, dispersed and stirred for fifteen minutes, gave the following results (Fig A2.1):

Test No.	sand %	silt %	<2 μ m %	<5 μ m %	pretreatment losses %
1	-	55	38	73	17
2	-	55	40	70	-

Test results

The soil was placed in the apparatus at a water content of 25.5%. It was then consolidated to a normal stress of 992 kPa and swelled to 248 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.028 mm/min, would ensure 95% dissipation of pore water pressure in a displacement of 1 mm. The slow standard rate of displacement, used throughout the test, was 0.01 mm/min.

The stress ratio versus cumulative displacement is plotted in Fig. A2.11a, A2.11b and A2.11c and the results from the test are summarized in Table A2.10.

Stage 1 involved shearing at 400 mm/min. Stages 2 to 8 were carried out at alternating slow and fast rates of displacement. At the end of stage 8, the normal stress on the sample was increased to 544 kPa.

Slow shearing was started again in stage 9. Shearing during stage 10, carried out at 400 mm/min, had to be stopped prematurely after a displacement of 48mm, because the U/V recording system ran out of paper at a displacement of about 8mm. Stage 11 was carried out at 400 mm/min too, without previously establishing the slow residual state. The remaining stages of the test, 12 to 17, were carried out at alternating slow and fast rates of displacement.

The final water content of the sample at the end of the test was as follows:

shear zone : 24.0%
average : 25.5%

Description of the test specimen

More than one shear surfaces were present within the sample. The sample separated at random through either an unstriated but highly polished shear surface, or through a shear surface that was striated

but not polished. A thin section, viewed under polarized light, showed the clay particles to be orientated at approximately 30^0 with respect to the shear direction.

Test 11, siltstone

Operator: Lemos, L.J.L

Reference: Lemos (1986)

Sample preparation

The sample, as received, had been cut from a block sample and was in an air-dry condition. It was prepared in exactly the same way as claystone B. The proportion by weight of the material that was coarser than $63\mu\text{m}$ was 3% approximately.

Index properties

The index properties which were determined on soil, prepared as for the ring shear test, were as follows:

LL=40, PL=26.6 PI=13.4

Gradings

Particle-size distributions of the soil, prepared as the sample for the ring shear test, and then pretreated with H_2O_2 and HCl, dispersed, and stirred, gave the following results (Fig A2.1):

Test No	Time of Stirring (min)	sand %	silt %	<2 μm %	<5 μm %	pretreatment loss %
1	15	20	78	2	7	—
2	15	20	79	1	7	6
3	30	15	82	3	9	—
4	5	15	83	2	7	—

Test results

The soil was placed in the apparatus, consolidated to a normal stress of 992 kPa and swelled to 248 kPa. From the consolidation stage, it was estimated that the rate of displacement of 0.10 mm/min would ensure 95% dissipation of pore water pressure in a shear displacement of 1 mm. The slow standard rate of displacement, used throughout the test, was 0.01 mm/min.

The stress ratio versus cumulative displacement is plotted in Fig. A2.12a and A2.12b and the results from the test are presented in Table A2.11.

Shearing was started at 400 mm/min in stage 1 and continued at alternating slow and fast rates of displacement during stages 2 to 8. At the end of stage 8, the normal stress on the sample was increased to 493 kPa.

Shearing was commenced again in slow stage 9. The remaining stages of the test, 10 to 15, involved alternating slow and fast shearing.

The final water content of the sample was as follows:

shear zone : 23.9%
average : 23.6%

Description of test specimen

A shear zone which was approximately 2mm thick and had a different colour from the rest of the sample, was present. The sample was separated on a well-defined, un-polished surface.

Tests 12,13,14, clayey siltstone B

Operator : Lemos, L.J.L

Reference : Lemos (1986)

The results of three ring shear tests on a clayey siltstone are considered here. In Test 12, the mobilization of strength immediately after fast shearing was investigated by means of slow probes. In Test 13, fast rates of displacement up to 6000 mm/min were imposed to the sample by means of the motor M5. In Test 14, shearing of the soil was performed against a rough glass interface in which a thermocouple and a pore pressure probe were installed for the temperature and pore water pressure measurement during fast shearing.

Sample preparation

The soil was received in a mechanically ground condition. It was remoulded to a water content above the plastic limit and left to hydrate for twenty four hours.

Index properties

The index properties of the soil, prepared as the sample for the ring shear test, were as follows:

LL=44, PL=23, PI=21%

Gradings

Particle size distributions determined on soil, prepared as for the ring shear test, gave the following results (Fig A2.1):

Test No.	sand %	silt %	<2 μ m %	<5 μ m %	pretreatment loss %
1	18	74	8	18	8.8
2	16	74	0	20	9.0

Lemos (1986) commented that the clay fraction (% <2 μ m) of the soil was surprisingly small for a plasticity index of 21%. It might have indicated that clay particles were larger than 2 μ m.

Test 12

Test results

The soil was placed in the apparatus at a water content of 26.4%. It was then consolidated to a normal stress of 1037 kPa and swelled to 609 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.0134 mm/min would ensure 95% dissipation of the pore water pressure in a 2mm displacement.

The stress ratio versus cumulative displacement is plotted in Fig. A2.13a and A2.13b and the test results are summarized in Table A2.12.

Stage 1 involved slow shearing through the peak to the residual strength. The increase of the strength immediately after the fast stages 2,5 and 8 was investigated by shearing the sample slowly at 0.35 mm/min during stages 3,6 and 9 respectively. The time, needed for the full mobilization of strength in stage 3, was identical to the time, needed for the 95% dissipation of the excess pore water pressure, observed during the consolidation stage. The strength of the soil at the end of stages 3, 6 and 9 was probed during the slow stages 4, 7 and 10. At the end of stage 10, the normal stress on the sample was

reduced to 200 kPa.

Shearing was commenced again in slow stage 11. The mobilization of the strength immediately after the fast stage 12 was investigated by shearing the sample at 0.35 mm/min during stage 13. The remaining stages of the test, 14 to 19, involved shearing at alternating slow and fast rates of displacement. The sample was unloaded in shear whenever it was left at rest.

The final water content of the sample was as follows:

average : 20.0%

shear zone : 21.5%

Description of the test specimen

A shear zone which was 1 to 2mm thick and had a different colour from the rest of the sample was observed at the mid-height of the sample. The material within the shear zone seemed to be more plastic. The sample separated through an un-polished, non-planar shear surface. Outside the shear zone sand-size lumps of dryer soil were present. A thin section, viewed under the polarized microscope, showed a shear zone approximately 0.8mm thick with the clay particles orientated at about 15° with respect to the shear direction. The silt grains had their axes preferentially orientated with respect to the shear direction too.

Test 13

Test results

The soil was placed in the apparatus at a water content of 31.4%. It was then consolidated to a normal stress of 992 kPa and swelled to 608 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.021 mm/min would ensure 95% dissipation of the excess pore water pressure during shearing in a displacement of 1mm.

The stress ratio versus cumulative displacement is illustrated in Fig. A2.14a, A2.14b, and A2.14c and the test results are summarized in Table A2.13.

Stage 1 involved shearing through the peak to the residual strength at 0.01 mm/min and 0.05 mm/min with the gap opened and closed

respectively. Stages 2 to 9 were carried out at alternating slow and fast rates of displacement. In the fast stages 4 and 5, the rates of displacement were imposed to the sample by means of the motor M5, connected directly through a magnetic clutch to the worm drive of the apparatus. The motor was switched on initially to build-up the full speed and then it was connected to the sample through the magnetic clutch. Soon after the shearing had been started, the magnetic clutch kept slipping and the motor was disconnected from the worm drive of the apparatus. From then on, the magnetic clutch was taken off and the motor was directly connected to the worm drive through a rigid connection. At the end of stage 9, the normal stress was reduced to 200 kPa.

Shearing was commenced again in stage 10. Stages 10 to 15 involved alternating slow and fast shearing. The increase of strength after the fast stage 15 was probed in the stages 16 and 17 both carried out at 0.327 mm/min, with a short pause allowed between them (Fig A2.14c). The slow residual strength was established during again during stages 19 and 20. The test terminated with stage 21.

The final water content of the sample was as follows:

average : 20.0%
shear zone : 21.5%

Description of the test specimen

The structure of the test specimen was identical to that observed for the test specimen in Test 12.

Test 14

Interface: Glass

Initial roughness: (It was not reported)

Consolidation stage

The soil was placed in the apparatus at a water content of 31%. It was then consolidated to a normal stress of 594 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.03 mm/min would ensure 95% dissipation of pore water pressure during shearing in 1mm travel.

The response of the pore water pressure probe to an increment of the normal stress during the consolidation stage was reasonably good ($\Delta u/\Delta \sigma_n = 0.70-0.85$), indicating a high degree of saturation. It was found, however, that the transducer was measuring a residual pore water pressure, after the consolidation had been completed, due to the interference between the porous stone of the transducer and its diaphragm.

Test results

The stress ratio versus cumulative displacement is shown in Fig. A2.15. The shearing stages are summarized in table A2.14.

Stage 1 involved shearing at 0.03 mm/min. The remaining stages of the test, 2 to 6, were carried out at slow and fast rates of displacement.

There was a considerable variation of the pore water pressure during the slow shearing because of the shear stresses transmitted to the tip of the pore pressure probe (the tip was approximately 2mm below the interface surface, Lemos, 1986). During the fast shearing oscillations of higher amplitude and larger frequency were observed, suggesting the unreliability of the pore-water pressure measurement. As soon as fast shearing during stages 2, 4 and 5 had been stopped, the pore water pressure, measured by the probe, increased. It dissipated afterwards attaining a residual value.

The final water content of the sample was not reported.

Description of the test specimen

The soil which was close to the interface had a darker colour than the rest of the sample. This indicated that a sorted thin layer probably richer in clay and small round particles had been formed during shearing. There was slight evidence of striations on the soil surface in contact with the interface. A shear surface which was highly polished, striated, and 0.5 mm away from the interface surface was present.

Test 15, clayey siltstone C

Operator: Lemos, L.J.L

Reference: Lemos (1986)

Sample preparation

The soil was received in a mechanically ground condition. It was remoulded to a water content above the plastic limit and left to hydrate for 24 hours.

Index properties

The index tests carried out on soil, prepared as the sample for the ring shear test, gave the following results:

LL=39.4, PL=22, PI=17.4%

Gradings

A particle size distribution of the soil, prepared as for the ring shear test, gave the following results (Fig A2.1)

silt	:	90%
% <2 μ m	:	10%
% <5 μ m	:	25%
pretreatment loss	:	19.7%

Test results

The soil was placed in the apparatus at a water content of 27.1%. It was then consolidated to a normal stress of 1037 kPa and swelled to 608 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.01 mm/min would ensure 95% dissipation of pore water pressure during shearing in a displacement of 1mm.

The stress ratio versus cumulative displacement is plotted in Fig A2.16a and A2.16b. Each shearing stage is summarized in Table A2.15.

Stage 1 involved slow shearing through the peak to the residual strength. Stages 3 and 6 were carried out at 0.329 and 0.393 mm/min respectively, immediately after fast stages 2 and 5, in order to investigate the mobilization of strength after fast shearing. The strength, mobilized at the end of stage 6, was surprisingly low, though the stage was prolonged for 22 min, twice the time needed for 95% dissipation of the excess pore water pressure. The slow residual state was established again during stages 4 and 7. Stage 8 was carried out at 10 mm/min. Stage 9 involved slow shearing to the residual

conditions. At the end of stage 9, the normal stress on the sample was reduced to 200 kPa.

Slow shearing was commenced again in stage 10. The remaining stages of the test 11 to 18 were carried out at alternating fast and slow rates of displacement.

The final water content of the sample was as follows:

average : 20.7%

shear zone : 20.9%

Description of the test specimen

The sample separated through a continuous, un-polished shear surface. A shear zone which was 1 to 2mm thick and had a different colour from the rest of the sample was present at the mid-height. The soil was friable and crumbled easily when lightly pressed between the fingers. A thin section, viewed under polarized light, showed a uniform mass of clay, and fine and medium silt. A shear zone approximately 0.7mm thick was present at the mid-height of the sample. The clay particles within the shear zone were orientated at 30° approximately with respect to the shear direction. There was no evidence of a continuous shear surface.

Test 16 : sandstone

Operator : Tika, T.M.

Soil description and sample preparation

The soil had a greyish colour. It was received in a mechanically ground condition (powdered). It was mixed with distilled water and left to hydrate for twenty four hours before testing in the ring shear apparatus.

Mineralogy

Results from an X-ray diffraction of soil are summarized as follows (University college, Dept. of Geological studies):

Clay minerals (% <4 μ m)	{	Montmorillonite	90%
		Muscovite	5%
		Kaolinite	5%
		(+ Sodic plagioclase, Hornblende, Quartz)	

Coarse fraction { Muscovite, Hornblende, Kaolinite,
 (% >4 μ m) { Sodic plagioclase, Calcite, Quartz.

Gradings

Particle size distributions of the soil, determined prior to and after testing in the ring shear apparatus, with and without pretreatment, gave the following results:

Grading	sand %	silt %	<2 μ m %	pretreatment loss %
Prior to testing	86	13	1	8.5
After testing	68	26	6	10.0
Prior to testing	90	9	1	} without pretreatment
After testing	73	19	8	

A grading curve of the soil, determined prior to testing in the ring shear apparatus, is shown in Fig A2.1. According to the grading curve shown, the soil is classified as a well graded clayey sand (SWC). The soil, observed under a microscope, consisted of angular particles with relatively rough surfaces.

Test results

The soil was placed in the apparatus at a water content of 22.4%. It was then consolidated to a normal stress of 1000 kPa and swelled to 500.7 kPa. The compression during the consolidation stage was equivalent to a decrease of the water content of 2.3% approximately. From the consolidation stage, it was estimated that the slow standard rate of displacement of 0.01 mm/min, used throughout the test, would ensure 95% dissipation of pore water pressure during shearing in a displacement of 0.20 mm.

Shearing stages are summarized in Table A2.16 and the plots of the stress ratio versus cumulative displacement are given in Fig. A2.17b to A2.17c.

Stage 1 involved slow shearing through the peak strength. It was followed by stages 2 to 9, carried out at alternating slow and fast rates of displacement. The residual strength was not fully established

nor had the sample been unloaded in shear prior to shearing at 10 mm/min. At the end of stage 9, the nominal normal stress on the sample was reduced to 242.6 kPa.

Slow shearing was commenced again in stage 10. Stages 11 to 19 involved alternating slow and fast shearing. No recording was performed during stage 17, carried out at 5400 mm/min, due to failure of the recording system. Stage 18 followed at 5400 mm/min too, without previously establishing the slow residual state. The test terminated with the slow stage 19.

The final water content of the sample was as follows:

shear zone (central third) : 17.4%

Outer thirds : 17.0%

Description of the test specimen

The sample exhibited a discrete shear zone which was approximately 5mm thick across the mid-radius of the sample and reduced to 3mm towards the sample sides. The material within the shear zone appeared to have been degraded and compacted and had a darker colour from the rest of the sample. The shear zone of the sample was observed to spread asymmetrically with respect to the mid-plane, growing into the upper, non-moving part of the sample. Mandl et al (1977) studied the textural changes in densely packed granular materials under shear in a ring shear apparatus. They attributed the asymmetric spread of the shear zone to two reasons, partly to the asymmetry in the construction of the ring shear apparatus (which allows the sample to dilate only by an upward movement), and partly to the tendency of the shear zone to spread away from the zone of segregation of smaller grains.

Verification of the visually observed degradation was obtained by the particle size distributions, determined at the end of the ring shear test, and the observations of the microstructure of the sample. The amount of particle breakage during shearing is higher than that indicated by the results of the particle size distributions before and after testing, since the gradings were performed on the whole sample and not only on the material from the shear zone. Soil from the shear zone observed under a microscope showed a well-graded grading which was finer than that of the initial material. The particles had a

subangular shape and relatively smooth surfaces. A thin section, prepared from the sample, is presented in Plate 1 (photographs 1 to 3).

Test 17,18,19,20,21, clayey siltstone D

Operator: Tika, T.M

General

The results of five ring shear tests on a clayey siltstone from the Kalabagh dam site are considered here. Test 17 was a standard ring shear test in which the influence of fast shearing on the residual strength was investigated. Tests 18,19 and 20 were performed in order to study the microstructure of the shear zone which was formed when the slow residual, the fast peak and the fast residual strengths respectively operated. In Test 21 the effect of the change of normal stress during fast shearing on the behaviour of soil was investigated.

Sample preparation

The soil had a brownish colour. It was received in a mechanically ground condition (powdered). It was remoulded with distilled water and left to hydrate for twenty four hours before testing in the ring shear apparatus.

Mineralogy

Results from an X-ray diffraction of the soil are as follows (University College, Dept of Geological studies):

Clay minerals	{	Illite	50%
(% <4 μ m)		Montmorillonite	35%
		kaoline	15%
Coarse fraction	{	Quartz, Sodic plagioclase,	
(% >4 μ m)		Muscovite, Calcite.	

Index properties

The index properties of the soil were as follows:

LL=44.2, PL=20.1, PI=24.1%

Gradings

Particle size distributions of the soil were determined following different ways of sample preparation

Gradings A and B were determined using the standard hydrometer method ,with and without soil pretreatment respectively (Fig.A2.1).

For grading C, the soil was soaked in distilled water for forty eight hours. It was then dried in order to remove the excess water and worked on a glass plate with a steel spatula in order to crush any remaining lumps of soil. The soil was then dispersed and mechanically shaken for twelve hours.

For grading D, the soil was soaked in distilled water for forty eight hours. The slurry formed was put into a blender for ten minutes. The sample was then dried to remove the excess water and worked on a glass plate with a steel spatula in order to crash any remaining lumps of soil. It was then dispersed and mechanically shaken for twelve hours.

Grading	sand %	silt %	<2 μ m %	<5 μ m %	pretreatment loss %
A	20	62	18	34	6.4
B	5	71	25	38.5	
C	7	69	24	40.0	
D	7	67	26	42.0	

The grading curves without pretreatment indicated a higher silt and clay content, suggesting the presence of calcium carbonate in the silt and clay size particles. For gradings C and D, the procedures suggested by Heley and MacIver were followed (1971).

Observations of the material, remaining on the 63 μ m sieve with a microscope, indicated that the majority of the particles had a platy and elongated shape. The large particles were porous. Quartz particles had an angular shape.

Test 17

Test results

The soil was placed in the apparatus at a water content of 29.1%. It was then consolidated to a normal stress of 1000 kPa and swelled to 527.9 kPa. The compression of the sample during the consolidation stage was equivalent to a decrease of the water content of about 12.2%. From the consolidation stage, it was estimated that the slow standard rate of displacement, 0.01 mm/min, used throughout the test, would ensure 95% dissipation of pore water pressure in 0.4 mm travel.

Each shearing stage is summarized in Table A2.17 and plots of the stress ratio versus cumulative displacement are shown in Fig.A2.18a to A2.18d.

Stage 1 involved slow shearing through the peak to the residual strength. Stages 2 to 7 were carried out at alternating fast and slow rates of displacement. During the fast stage 6, the gap between the confining rings was closed to minimize soil extrusion from the sample sides. Although the loss of soil had been minimized and some friction might have been developed in the gap, the drop in strength to less than the slow residual persisted. At the end of stage 7, the nominal normal stress on the sample was reduced to 264.6 kPa.

Slow shearing resumed in stage 8. The slight increase of the stress ratio with displacement, observed during this stage, was due to friction developed in the gap. Stage 9 followed at 95 mm/min. The proving ring used for the measurement of the side friction broke during this stage. Results from this stage are not presented therefore, since they are unreliable. After the proving ring had been replaced, the slow residual state was established again in stage 10. Stage 11 to 14 involved alternating slow and fast shearing.

Stages 15,16 and 17 were carried out at 2000 mm/min with a pause allowed between them for the dissipation of the pore water pressure, generated during the fast shearing. It was intended to investigate the response of the sample to a fast rate of displacement after a fast stage and a pause for consolidation. Slow shearing was commenced again in stage 18. The test ended with the fast stage 19.

The final water content of the sample was as follows:

shear zone : 15.0%

lower part : 16.4%

upper part : 17.8%

Description of the test specimen

The sample separated along a rugged and heavily-striated, non-planar shear surface which was formed closer to the upper loading platen. The shear zone was approximately 1.5 mm thick at the mid-radius of the sample and 1mm thick at the sample sides.

Test 18

Test Results

The soil was placed in the apparatus at a water content of 31.60%. It was then consolidated to a normal stress of 966.8 kPa and swelled to 264.6 kPa. The compression of the sample during the consolidation stage was equivalent to a decrease of the water content of about 11.3%.

The plot of the stress ratio versus cumulative displacement is given in Fig.A2.19 and the test results are presented in Table A2.18

Stage 1 involved shearing at 370 mm/min. The test terminated with stage 2, carried out at 0.0053 mm/min in order to establish the slow residual strength.

The final water content of the sample was as follows:

shear zone : 20.5%

overall : 22.2%

Description of the test specimen

The same observations as for the test specimen of Test 17 were made.

Microstructural observations

Plate 2 illustrates the microstructure of the test specimen. The observations of the microstructure showed a shear zone, bounded by two discontinuous shear surfaces approximately 0.50 mm apart (photographs 8 to 10). The clay particles and the shear surfaces within the shear zone were orientated parallel to the direction of shear (photographs 11 to 15).

Test 19

Test results

The soil was placed in the apparatus at a water content of 30.80%. It was then consolidated to a normal stress of 966.7 kPa and swelled to 264.6 kPa. The compression of the sample during the consolidation stage was estimated to be equivalent to a decrease of the water content of about 11%. From the consolidation time, it was found that the slow standard rate of displacement of 0.014 mm/min, used throughout the test, would ensure 95% dissipation of the pore water pressure during shearing in a displacement of 0.6 mm.

The stress ratio versus cumulative displacement is plotted in Fig.A2.20, and the test results are presented in Table A2.19.

Stage 1 involved shearing at 12 mm/min. The slow residual strength of the soil was established in stage 2. Stage 3, performed at 400 mm/min, was followed immediately by stage 4, carried out at 0.3430 mm/min. A short pause of 1.75 min was only allowed in order to change the configuration of the gear boxes, engaged to the worm drive of the apparatus. The rate of displacement used in stage 4 was chosen so that the displacement after 45 min (consolidation time of soil) of shearing would be approximately 15mm. This is the displacement at which the residual strength is established during slow standard shearing, following a fast stage after a pause for consolidation. The slow residual strength of the soil was established again in stage 5. Stages 6 to 11 involved alternating fast and slow shearing. The test terminated with stage 12, carried out at 400 mm/min only for a relatively short displacement.

The final average water content of the sample was 21.8%.

Description of the test specimen

The sample separated along a non-planar shear surface which had formed close to the upper loading platen.

Microstructural observations

Plate 3 illustrates the microstructure of the test specimen. The observations of the microstructure showed a shear zone present in the mid-height of the sample. The shear zone was thicker than that formed

when the slow residual strength operated (compare photographs 16 and 17 with 8 and 9 respectively). It was bounded by two discontinuous shear surfaces which were approximately 0.70 mm apart and considerably less distinct than those formed at the slow residual state. The clay particles and the shear surfaces within the shear zone were orientated at approximately 30° to the direction of shear.

Test 20

Test Results

The soil was placed in the apparatus at a water content of 31.10%. It was then consolidated to a normal stress of 966.8 kPa and swelled to 264.6 kPa. The decrease in the water content during the consolidation stage was about 11%.

The stress ratio versus the cumulative displacement is plotted in Fig.A2.21, and the test results are presented in Table A2.20.

Stage 1 involved a shearing at 100 mm/min. It was followed by stage 2, carried out at 0.01 mm/min in order to establish the slow residual strength of the soil. The test was terminated with stage 3 which involved shearing at 400 mm/min. The water was siphoned out of the perspex bath just before stage 3 commenced.

The average water content of the sample at the end of the test was 20.9%

Description of the test specimen

The sample separated along a non-planar shear surface which was rugged and striated circumferentially.

Microstructural observations

Plate 4 illustrates the microstructure of the test specimen. The observations of the microstructure showed a shear zone at the mid-height of the specimen which was considerably thicker than that formed at the slow residual and the fast peak states (compare photographs 9, 17 and 25). The shear zone was bounded by two discontinuous shear surfaces approximately 1.40mm apart. The structure of the shear zone appeared to be more homogeneous than those corresponding to the slow residual and the fast peak strengths. The

elongated massive grains within the shear zone had their long axes orientated parallel to the direction of shear and they were separated by parallel also orientated clay particles. The shear surfaces within the shear zone were less and smaller than those observed at the slow residual and fast peak states.

Test 21

Test Results

The soil was placed in the apparatus at a water content of 33.2%. It was then consolidated to a normal stress of 264.6 kPa. The decrease in the water content due to the compression was approximately 13%. From the consolidation stage, it was estimated that the slow standard rate of displacement of 0.01 mm/min, used throughout the test, would ensure 95% dissipation of the pore water pressure in a displacement of less than 0.5 mm.

Each shearing stage is summarized in Table A2.21 and stress ratio versus cumulative displacement is shown in Fig. A2.22.

Stage 1 involved shearing at 400 mm/min. The slow residual strength of the soil was established in stage 2. Stages 3 and 4 involved fast and slow shearing respectively. As soon as stage 5, carried-out at 100 mm/min, had been completed the nominal normal stress on the sample was increased to 527.9. Shearing was then commenced immediately at the same rate of displacement. The intention was to investigate the effect of a change of the normal stress during fast shearing on the behaviour of soil. The test terminated with the stage 6.

The average water content of the sample at the end of the test was 24.3%.

Description of the test specimen

The same observations as for the test specimen of Test 20 were made.

Test 22 , claystone D

Operator: Tika, T. M

Sample preparation

The soil ^{had} a light brownish colour. It was received in a mechanically ground condition. It was remoulded with distilled water and left to hydrate for at least twenty four hours before testing in the ring shear apparatus.

Mineralogy

Results of the X-ray diffraction of the soil are as follows (University College, Dept. of Geological studies):

Clay minerals (% < 4 μ m)	{	Illite : 40%
		Kaolinite : 30%
		Montmorillonite : 30%
Coarse fraction: (% >4 μ m)	{	Quartz, Muscovite, Calcite,
		Sodic plagioclase

Index properties

The index properties of the soil were as follows:

$$LL = 52.6, \quad PL = 27.0, \quad PI = 25.6\%$$

Gradings

Particle size distributions of the soil were determined following different procedures of sample preparation.

Gradings A, B, C and D were determined using the standard hydrometer method, with and without soil pretreatment.

For grading E, the soil was soaked in distilled water for forty eight hours and the excess water was then allowed to evaporate. The soil was then worked on a glass plate with a steel spatula in order to crush any remaining lumps. The soil was then dispersed and mechanically shaken for twelve hours.

For grading F, the soil was soaked in distilled water for forty eight hours. The slurry was put into a blender for ten minutes. The surplus water was left to evaporate. The soil was then worked with a steel spatula on a glass plate to crush any remaining lumps. The soil was then dispersed and mechanically shaken for twelve hours.

Grading	sand %	silt %	<2 μ m %	<5 μ m %	pretreatment loss %
A	1	53	46	68	8
B	7	53	40	64	9
C	8	46	46	66	10
D	13	50	37	57	—
E	12	52	36	57	—
F	13	50	37	61	—

The loss of soil during pretreatment was of the order of 9%. Grading curves without pretreatment gave lower clay fractions, suggesting the presence of calcium carbonate in the sand-size particles. Sample preparation for gradings D and E followed the procedures, suggested by Heley and MacIver (1971). Gradings D, E and F are quite similar, suggesting that the dispersing agent was efficient in separating the soil-particles and no further degradation occurred when the sample was soaked in distilled water for forty eight hours and subsequently blended.

Observations of the material remaining on the 63 μ m sieve with a microscope showed that most of the particles were elongated and platy (they were more platy than the particles of clayey siltstone D, but less elongated than those).

Test results

The soil was placed in the apparatus at a water content of 36.1%. It was then consolidated to a normal stress of 966.8 kPa and swelled to 527.9 kPa. The decrease in the water content during the consolidation stage was about 10.2%. From the consolidation stage, it was estimated that the slow standard rate of displacement, 0.01 mm/min, used throughout the test, would ensure 95% dissipation of pore water pressure during shear in a displacement of less than 2mm.

The soil was tested at a wider range of normal stresses (527.9, 264.6, 132.9 and 89.0 kPa) than the soils tested previously in order to investigate the normal stress dependence of the rate effects on the residual strength.

Each shearing stage is summarized in table A2.22 and the plots of the stress ratio versus cumulative displacement are shown in Fig A2.23a to A2.23f

Stage 1 involved slow shearing through the peak to the residual strength. The increase of the stress ratio at the end of the stage was due to the friction developed in the gap. Stage 2 followed at 9.5 mm/min for a displacement of only 5.5mm. No readings were taken during this stage (the U/V recorder ran out of paper). After a pause, stage 3 was carried out at 9.5 mm/min, without previously establishing the slow residual strength. The shear resistance dropped to a value as low as 50% of the slow residual strength. Slow shearing was commenced in stage 4. Stage 5 was performed at 9.5 mm/min in order to verify the behaviour observed in stage 3. The shear resistance initially dropped to a value lower than the slow residual strength of the soil, but then increased towards it. Slow shearing resumed in stage 6. Stages 9 to 12 involved shearing at alternating fast and slow rates of displacement. At the end of stage 12, the nominal normal stress on the sample was reduced to 264.6 kPa.

Slow shearing to the residual conditions was commenced in stage 13. Stage 14 was carried out at 9.5 mm/min for a relatively short displacement, because of the failure of the recording system. Stage 15 followed at 9.5 mm/min too, without previously establishing the slow residual strength. Stages 16 to 19 involved alternating slow and fast shearing. After a pause for consolidation, stage 20 was performed at the same rate of displacement as stage 19. The nominal normal stress was then reduced to 132.9 kPa.

Slow shearing resumed in stage 22. Stages 23 to 30 involved alternating fast and slow shearing. At the end of stage 30, the nominal normal stress was reduced to 89.0 kPa.

Shearing was resumed in slow stage 31. The increase of the stress ratio during this stage was due to the friction developed in the gap. Stages 32 to 37 involved alternating fast and slow shearing. No measurements were taken during the fast stage 38 due to failure of the recording system. The slow residual strength was established again in stage 39. Stages 40 and 41 were carried out at a fast and slow rate of

displacement respectively.

The nominal normal stress was then increased back to 264.6 kPa in order to test the shear zone of the sample at 360 mm/min and clarify the uncertainty about the peak stress ratio, observed in stage 19, carried out at 2600 mm/min (the traces of the shear load on the U/V recorder paper were not clear). Slow stage 42 was used to establish the residual strength at the new normal stress. Stages 43 to 47 involved alternating fast and slow shearing.

At the end of stage 47, the nominal normal stress was further increased to 527.9 kPa in order to re-test the shear zone of the sample at 9.5 mm/min and clarify the discrepancy of the results, obtained in stages 3 and 5. While the results from stage 47 corroborated those obtained from stage 3, the results from stage 5 were similar to those obtained from the stages, carried out at the same rate of displacement but at lower normal stresses.

The final water content of the sample was as follows:

average : 22.7%
shear zone : 20.3%

Description of the test specimen

The sample separated along a heavily-striated, polished, non-planar shear surface. The shear surface was formed asymmetrically with respect to the mid-plane of the sample, being closer to the upper loading platen. Rotund particles intruded the shear surface. A shear zone which was 1mm thick and had a darker colour from the rest of the sample was present.

A2.2 TILLS

Test 29 , Lower Cromer Till

Operator : Tika, T.M

Soil description

The soil used in the test came from the same batch of soil, as that used by previous researchers (El-Chamrawy, 1979, Lupini, 1981,

Takahashi, 1981, Gens 1982, De-Campos, 1983, Hight, 1983 and Lemos (1986). The samples were obtained from the base of the coastal cliffs in the North Norfolk coast near Happisburgh.

The geological sequence of the exposed cliffs has been given by Baswell (1923) and reported by El-Chamrawy (1978) as follows:

Era : Kainozoic

Period : Quaternary

		Description
Holocene:		Alluvium, brown sand, shingle...e.t.c.
Epochs:	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), consisting of
	Cromer Till	Upper tills (unsorted and unlaminated)
		Interglacial tills (laminated)
		Lower tills (unsorted and unlaminated).

Mineralogy

The results of a mineralogical analysis, carried out by the X-ray diffraction Laboratory at the Geology Department of Imperial College, are as follows (El-Chamrawy, 1979, Lupini, 1981):

Whole Sample:

Major components : quartz
 Minor components : calcite, feldspar, clay minerals
 traces : mica, muscovite type
 possibly including some dolomite.

Clay fraction:

Major components : calcite, illite
 Minor components : smectite, kaolinite, chlorite.
 traces : quartz, feldspar possible.

Sample Preparation

The soil was in an air-dry condition. It was mechanically ground, remoulded with distilled water to a relatively plastic state and then left to hydrate for at least twenty four hours before testing in the ring shear apparatus.

Index properties

The index properties of the soil were as follows:

LL=26, PL=12, PI=14%

Gradings

Particle size distributions of the soil with and without pretreatment are shown in Fig. A2.24. The pretreatment loss of the soil was of the order of 17.80%. The pretreated soil gave a coarser particle size distribution, indicating the presence of calcite in the range of silt and clay size particles. Using the British Soil classification system for engineering purposes (B.S 5930, 1981), the soil is classified as CLS, a sandy clay of low plasticity.

Test results

The soil was placed in the apparatus at a water content of 18.10%. It was then consolidated to a normal stress of 966.8 and swelled to 571.8 kPa. The decrease of the water content of the sample due to the compression was approximately 7.4%. From the consolidation stage, it was estimated that the slow standard rate of displacement, 0.025 mm/min, used throughout the test, would ensure 95% dissipation of the pore water pressure during shearing in less than 2mm of displacement.

Fig.A2.25a to A2.25d illustrate the stress ratio versus cumulative displacement. The results from the test are summarized in Table A2.23.

Stage 1 involved slow shearing though the peak to the residual strength and was followed by stage 2, carried out at 133 mm/min. Stage 3 was carried out at the slow standard rate of displacement only for a displacement of 3.1mm ,because of problems arising with the motor M1. The high shear force applied on the sample, due to the high values of both the normal stress and the coefficient of friction, caused the displacement of the motor itself from its fixed position. The nominal normal stress was then decreased to 484 kPa.

Slow shearing was resumed in stage 4. Stage 5 followed at 1200 mm/min. At the end of the stage 5, the sample was relieved from the shear load for a period of 26 days. After this period, the nominal normal stress on the sample was reduced to 264.6 kPa.

Slow shearing was commenced again in stage 6. Stages 7 to 9 involved alternating fast and slow shearing. Slow shearing at 0.1333 mm/min resumed immediately after the fast stage 9. A short pause of 10.2 min was only allowed in order to change the motor, engaged to the worm drive of the apparatus. The rate of 0.1333 mm/min was chosen so that after shearing for a period of about 60 min it would result in the same displacement at which the slow residual strength would be mobilized, had the slow standard rate of displacement been applied, after a rest for consolidation (settlements observation at the end of stage 7 had indicated a consolidation time of about 60 min). The slow residual strength was established again in stage 11. Stages 12 and 13 involved fast and slow shearing respectively. At the end of stage 13, the nominal normal stress on the sample was reduced to 132.9 kPa.

Slow stage 14 was used to establish the residual strength of the soil at the new normal stress. Stages 15 to 17 involved alternating fast and slow shearing. After a pause for consolidation, stage 18 followed stage 17 at the same rate of displacement. At the end of stage 18, the nominal normal stress on the sample was reduced to 67.1 kPa.

Slow shearing was commenced in stage 19. No recording was performed during the fast stage 20 (the U/V recorder ran out of recording paper). Stages 21 and 22 were both carried out at 133 mm/min with a pause allowed between them for consolidation. The slow residual strength was established during stage 23. Stages 24 and 25 involved fast and slow shearing respectively. The test terminated with the fast stage 26.

The final water content of the sample was as follows:

shear zone : 13.1%
average : 13.3%

Description of the test specimen

The sample separated along a rugged shear surface which was formed closer to the upper loading platen. Sand-size particles intruded the

shear surface. Photograph 32 in Plate 5 illustrates a thin section of the test specimen.

Test 24 , *Magnus Till*

Operator: Lemos L.J.L

Reference: Lemos (1986)

Site and Geological description

The soil came from the Magnus platform site in the North sea. A comprehensive geological and geotechnical description of the site had been given by Rigden and Semple (1983) and Jardine (1985). Briefly it is a glacial till, most probably a glaciomarine clay deposited some ten to fifty thousands years ago.

Sample preparation

The soil was air-dried and mechanically ground. It was remoulded with distilled water to a water content slightly higher than the plastic limit and left to hydrate for twenty four hours.

Index properties and grading

The index properties of the soil were as follows:

LL=32, PL=17, PI=15%

A particle size distribution of the soil is shown in Fig. A2.24.

Test results

The soil was placed in the apparatus at a water content of 20%. It was then consolidated to a normal stress of 530 kPa.

The test results are presented in Table A2.24 and the stress ratio versus cumulative displacement is plotted in Fig. A2.26.

Stage 1 involved shearing at 0.0169 mm/min and 0.058 mm/min. Stage 2 was carried-out at 131 mm/min. During this stage the motor was stoped for a few seconds and switched on again at the same speed. The test terminated with stage 3, carried out at 0.017 and 0.068 mm/min.

The final water content of the sample was not reported.

Test 25 , weathered Cowden Till

Operator : Lemos, L.J.L

Reference : Lemos (1986)

Soil description

The soil used for the test was supplied by the Building Research Establishment from a test site at Cowden (located on the east coast of Britain approximately 23 kilometers north-east of Hail). A comprehensive laboratory investigation of the Cowden Till has been given by Gens and Hight (1979). The purpose of carrying-out the test was to measure the residual strength and to investigate the effect of the rate of shearing and reversing the shear direction on it. The last effect is relevant to the negative skin friction of piles.

Sample preparation

The soil was air-dried, mechanically ground and remoulded with distilled water to a water content slightly higher than the plastic limit.

Index properties and grading

The index properties of the soil were as follows:

LL=39, PL=18.0 PI=21%.

A particle size distribution of the soil is shown in Fig. A2.24.

Tests results

The soil was placed in the apparatus at a water content of 22% and consolidated to a normal stress of 50 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.01 mm/min would ensure 95% dissipation of the pore water pressure during shearing in a 1mm displacement.

Shearing consisted of 45 stages with the normal stress ranging from 18 to 400 kPa. The test results are presented in Table A2.25 and the stress ratio versus cumulative displacement is plotted in Fig. A2.27a to A2.27d.

Stage 1 involved shearing through the peak to the residual strength at

0.003 mm/min and 0.05 mm/min, with the gap opened and closed respectively. Stages 2,3,4,5 and 6 followed at 0.06, 0.005, 0.1, 1.0 and 0.006 mm/min respectively. At the end of stage 6, the torque was released and the normal stress on the sample was increased to 100 kPa.

Stage 7 was performed at 0.0035 and 0.05 mm/min and was followed immediately by stage 8, carried out at 0.1 mm/min. After a pause, stages 9 and 10 followed at 1.0 and 0.0001 mm/min respectively. The shear load was then removed and the sample was consolidated to 150 kPa.

Shearing was commenced at 0.005 mm/min in stage 11. Stages 12 to 15 were carried out at increasing rates of displacement from 0.1 to 100 mm/min. A pause for the dissipation of the pore water pressure, generated during the shearing, was allowed after each stage. The displacements during these stages were too short and it is uncertain whether the ultimate conditions had been established. The slow residual state was re-established during stage 16. The shearing direction was reversed in stage 17, carried out at 0.5 mm/min. The displacement needed to mobilize the shear resistance increased and the residual strength decreased slightly. When the forward movement was re-imposed during stage 18, a significant increase of strength was observed. A considerable displacement was needed to establish the residual state again. At the end of stage 18, the torque was released and the sample was consolidated to 200 kPa.

Slow shearing at 0.01 mm/min resumed in stage 19. Stages 20 to 27 involved shearing at alternating fast and slow rates of displacement. The pattern of behaviour observed was that of an immediate gain in strength, followed by a further gain with both displacement and shearing rate. The displacements, involved in the fast stages, however, were small and a steady state was not established during fast shearing. According to Lemos (1986), the increase of shear resistance with displacement, observed in the fast stages, could be explained either by an increase in the effective stress due to generation of negative pore water pressure, or by structural changes of the soil. To clarify this argument, Lemos (1986) performed the series of stages 28 to 38. They consisted of a sequence of shearing at two rates of displacement of 0.37 and 10 mm/min, with or without a pause between

them and without any torque-release. The results obtained, indicated the following:

- (i) The difference between the slow peak and the slow residual strength immediately after fast shearing was of the same magnitude as the increase of strength over its immediate value during fast shearing (i.e, compare stages 29 and 30).
- (ii) Had the slow stage immediately after the fast shearing involved only a small displacement for the shear resistance to be mobilized and was followed by a pause for consolidation and a slow stage again, the slow peaks measured immediately after the fast shearing and after the pause were the same (compare stages 33 and 34). This indicated the absence of any excess pore water pressure at the end of the fast shearing. The increase of strength therefore had been caused by structural changes, taken place during the fast shearing. This was further supported by the following evidences.
- (iii) If fast shearing resumed again after the fast residual strength had been established and time for consolidation had been allowed, the fast residual strength recovered almost immediately (compare stages 36 and 37).
- (iv) The time for full consolidation of the soil was about 100 min, and the time needed for 50% of consolidation was approximately 10 min. The duration of the stages, carried out at 10 mm/min, varied from 8.6 to 18.8 min, ensuring a high degree of consolidation.

The slow residual strength was established again during stage 39, carried-out at 0.01 mm/min. The remaining stages of the test, 40 to 45, involved slow shearing at 0.015 mm/min and at the normal stresses of 446 kPa, 207, 103, 48 and 18 kPa respectively.

The water content of the sample at the end of the test was as follows:

shear zone : 21.6%
average : 21.4%

Description of the test specimen

The sample separated through a non-planar, un-polished shear surface. The shear zone was 1 to 2 mm thick and had a lighter colour than the rest of the sample. A thin section, viewed under polarized light, showed a shear zone approximately 1mm thick. The shear zone contained discontinuous shear surfaces. The clay particles on the shear surfaces were orientated parallel to the shear direction. They terminated either at the edges of the massive particles, or enclosed them. The shear zone was richer in fine massive particles. The same degree of orientation of the clay particles was observed both within the shear zone and the rest of the sample, apart from the shear surfaces, referred to above. Because of the mobility of the massive particles, the shear surfaces must have been formed and destroyed continuously during shear. The elongated massive particles had their long axis orientated parallel in the direction of shear.

A2.3 RESIDUAL SOILS

Test 26, residual soil from Fiji

Operator : Lupini, L.F

Reference : Lupini (1981)

Soil type and mineralogy

The tested soil was a weathered volcanic red-brown soil from the site of Ngondro Dam, Fiji. It was a porous soil with a total unit weight around 1.54 Mg/m^3 (the total unit weight of anisotropically normally consolidated lower Cromer Till samples is 2.08 Mg/m^3 , Takahashi, 1981). The natural water content of the samples was about 84%.

The clay fraction of the soil consisted of partly hydrated-poorly ordered kaolinite (not well crystallized). Some iron oxides were found in the clay size fraction, such as maghemite and hematite.

Index properties

The index properties of the soil were as follows:

LL=90, PL=53, PI=37%

Grading

Grading analysis in the dispersed (deflocculant used) and non dispersed form gave similar results. The grading of the dispersed soil was as follows:

sand size : 23%
silt size : 37%
clay fraction: 40%

Test results

The soil was placed in the ring shear apparatus in a remoulded state. It was first consolidated to a normal effective stress of 176.7 kPa which was then increased to 352.2 kPa. During this stage a nominal drained rate was obtained so as to ensure 95% dissipation of excess pore water pressure at a shearing displacement of 2mm. This rate was calculated to be 0.0171 mm/min. The actual nominal drained rate used however was slightly lower 0.0157 mm/min.

Stage 1 was commenced at this last rate (Fig. A2.26a). This drained rate was maintained until a displacement of 346mm when it was decreased to 2.5×10^{-4} mm/min (by a factor of 63). No significant rate effects were observed and the rate was re-established to the previous value (0.0157 mm/min). This procedure was repeated again at displacements of 702mm, 804mm, 1178mm, but now the rate was decreased even further to 2.5×10^{-5} mm/min (a factor of 630). Again no significant rate effects were observed and the sample was relieved from the torque and the normal effective stress was reduced to 176.7 kPa.

Stage 2 was initiated at a shearing rate of 0.0023 mm/min (after full swelling of the specimen had occurred, 0.72%). The rate was then increased to the nominal drained value 0.0157 mm/min and as expected (from the previous stage) no significant variations of the stress ratio were observed. In addition this stage did not show a peak stress ratio despite the soil being overconsolidated by a factor of two. This indicated that the soil had formed a sliding shear surface. However, the residual angle of friction was quite high, $\phi'_r \approx 17^\circ$.

Stage 3 consisted of different sub-stages at fast and very fast rates and can be followed in Fig. A2.26b. In this figure displacements are cumulative from the beginning of stage 2.

At first the rate was increased by a factor of 100 to 1.659 mm/min. (Sub-stage 3a). A remarkable increase in the stress ratio resulted, its values approaching 0.44. This sub-stage lasted for 40 min. The time for 95% consolidation of the whole soil specimen for an increase in normal effective stress from 176.71 to 352.17 kPa is 23 min.

Sub-stage 3b was carried out at a faster rate, 17.72 mm/min and the stress ratio increased to more than 0.60. The duration of sub-stage 3b was 9.4 min, which is less than a half for 95% of average degree of consolidation of the whole specimen to occur. Uncertainties about pore pressures therefore exist.

Sub-stage 3c was carried out at a very fast rate, 177.2 mm/min. The stress ratio dropped dramatically to values fluctuating around 0.20 (maximum 0.24, minimum 0.167). It would appear that the undrained strength of the material was reached. The first part of the sub-stage was carried out for 6 min until a cumulative displacement of 1320 mm was reached. A consolidation stage was allowed for 27 min (no torque on the sample) to check whether the drop in stress ratio was caused by a build up of pore water pressure. The consolidation settlements were about 0.50% of the total thickness of the specimen. Shearing was then re-initiated at the same rate (177.2 mm/min) and the same behaviour was observed with low values of the stress ratio ($0.20 \pm 15\%$).

Sub-stage 3d was immediately started by changing the shearing rate to 17.72 mm/min without allowing any consolidation to occur. A clear trend of increasing stress ratio was observed until at a displacement of 2410 mm the test was stopped for 15 min but with the shear stresses acting. It was then re-started at the same rate until the values of the stress ratio were stable at 0.60. Sub-stage 3d was complete. The state of effective stresses existing at the end of sub-stage 3d was studied in stage 4 by firstly allowing for a period of more than 5 times the consolidation of the whole specimen with the torque removed, and secondly by carrying a slow drained shear stage in order to study the structure formed by previous shearing during sub-stage 3d. If the increase in the stress ratio was due to a generation of negative pore water pressure, then the difference between the value of the stress ratio at the end of stage 3d and stage 4 would give a rough estimate of its magnitude.

Stage 4 was then carried out at the nominal drained rate, of 0.0157 mm/min (see Fig. A2.26a and Fig. A2.26b). The peak stress ratio for stage 4 was very close to the stress ratio at the end of sub-stage 3b, 3d and to the peak stress ratio of stage 1. This suggests two things: firstly, the excess pore water pressure at the end of stage 3d was probably very small; secondly, at the end of sub-stage 3d the soil was not being sheared in a sliding mode. During stage 4 a new shear surface had to be formed this process requiring displacements of the same magnitude as stage 1, when the soil was initially sheared in a remoulded stage.

The final water content of the sample was not reported.

Description of the test specimen

A thin section of the soil, viewed in the optical microscope under crossed polarized light, showed two shear surfaces formed. The principal shear surface (top) was very planar. The secondary shear surface was irregular and wavy. It bounded together with the principal one, a shear zone of variable thickness.

Test 27, residual soil from Piji

Operator: Tika, T.M.

Soil type

The elapsed time between this test and that performed by Lupini (1981) was at least four years. Though the original block of the undisturbed soil had been enveloped in wax coating and stored, mineralogical changes of the soil might have taken place during this period, as it is indicated by the difference in the index properties and the residual coefficient of frictions of the two tested samples.

Mineralogy

Clay minerals (% <4 μ) : kaolinite

Coarse fraction (% >4 μ m): Muscovite, Magnetite.

Index properties

The index properties of the soil were as follows:

LL=85.2, PL=63.3, PI=21.9%

Gradings

An envelope of the particle size distribution of the soil (without pretreatment) is shown in Fig. A2.29.

Tests results

The soil was placed in the apparatus at a water content of 81.4%. It was then consolidated to a nominal normal stress of 922.9 kPa and swelled to 484 kPa. The decrease of the water content of the sample during the consolidation stage was about 26.8%. From the consolidation stage, it was calculated that the slow standard rate of displacement, 0.0153 mm/min, used throughout the test, would ensure 95% dissipation of the pore water pressure in 0.4 mm of shearing displacement.

The results from the test are presented in Table A2.26 and the stress ratio versus cumulative displacement plots is given in Fig. A2.30a and A2.30b.

Stage 1 involved slow shearing through the peak to the residual strength and was followed by stage 2, carried out at 133 mm/min. The slow residual strength was established again in stage 3. Fast shearing of the sample at 133 mm/min was repeated during stages 4 and 5, as the anticipated loss of strength during fast shearing according to Lupinis results didn't take place. Stages 6,7 and 8 involved alternating slow and fast shearing. At the end of stage 8, the nominal normal stress on the sample was reduced to 220.7 kPa.

Slow shearing was commenced again in stage 9. Despite the soil being overconsolidated, no increase of strength was observed during this stage. This indicated that the soil was shearing in a sliding mode. Stages 10 to 14 were carried out at alternating fast and slow rates of displacement. No recording was performed during the fast stage 14 (the U/V recorder ran out of paper). The test ended with the fast stage 15.

The final water content of the sample was as follows:

average : 55.4%

shear zone : 58.2%

Description of the test specimen

After the test, the sample exhibited two shear surfaces which bounded a

variable thickness shear zone (about 6mm at the mid-radius of the sample and 1mm at the sample sides). The sample was split through the top (principal) shear surface which was non-planar and had light striations. The material within the shear zone had a darker colour than the rest of the sample.

Test 28, residual soil from Mauritius

Operator: Tika, T,M

Soil Type

Undisturbed blocks of the soil (brownish colour) were obtained from the site of the Champagne hydro-electric project, Mauritius.

Mineralogy

Clay minerals ($\% < 4\mu\text{m}$):	{	Kaolinite trace only
	{	(+ Gibbsite, Quartz)
Coarse fraction ($\% > 4\mu\text{m}$):	{	Kaolinite, Muscovite, Quartz,
	{	Gibbsite, Ilmenite.

Sample preparation

The soil was air-dried, mechanically ground and sieved through the $425\mu\text{m}$ sieve. It was remoulded with distilled water to a plastic state and left to hydrate for twenty four hours before testing in the ring shear apparatus.

Index properties

The index properties of the soil, prepared as the sample as for the ring shear test, were as follows:

LL=48.6, PL=42.3, PI=6.2%,

Gradings

Particle size distributions of the soil, determined prior to and after testing in the ring shear apparatus, are shown in Fig. A2.29. The loss of weight of the pretreated soil was 6.5 %.

Test results

The soil was placed in the apparatus at a water content of 46.1%. It

was then consolidated to a nominal normal stress of 264.6 kPa. The decrease of the water content of the sample due to the compression was about 9.5%. From the consolidation stage, it was estimated that the slow standard rate of displacement, 0.01 mm/min, used throughout the test, would ensure 95% dissipation of the pore water pressure in a shear displacement of 0.02 mm.

The complete sequence of shearing can be followed in Fig. A2.31a and A2.31d. Each shearing stage is summarized in Table A2.27.

Stage 1 involved slow shearing through the peak to the residual strength and was followed by stage 2, carried out at 360 mm/min. The slow residual strength was established again in stage 3. After the completion of stage 3, it was intended to perform a fast stage. The fast shearing, however, ended prematurely with a displacement of less than 0.1 mm due to mechanical disengagement of the motor M2 unit. The slow residual strength was further established in stage 4. Since no significant rate effects on the residual strength were observed in stage 2, the rate of displacement was further increased to 1000 mm/min in stage 5. A neutral type of behaviour with respect to rate effect was observed again. At this point, the effect of relatively slow rates of displacement on the residual strength was investigated. Stages 7 to 14 involved alternating slow and fast shearing. Stage 7 was carried out only for a displacement of 12.8 mm, as shearing had to stop prematurely, because of the failure of the recording system. After the completion of stage 14, the nominal normal stress was increased to 527.9 kPa.

Shearing was commenced at the slow standard rate of displacement in stage 15. Stage 16 followed at 12 mm/min. The sample was slowly resheared to the residual state in stage 17. The shearing rate was increased nine-fold in stage 18 (0.095 mm/min), and was reduced to the slow standard one in stage 19. Stages 20 and 21 involved fast and slow standard shearing. The rate of displacement was increased by a factor of approximately six in stage 22, carried out at 0.058 mm/min. Stage 23 was performed at 90 mm/min in order to clarify an uncertainty existing about the peak stress ratio observed in stage 20, due to the sudden uneven redistribution of the shear loads shortly after fast shearing had started. Slow standard shearing was resumed in stage 24

and was increased six-fold in stage 25. Stages 26 to 28 were carried-out at alternating fast and slow rates of displacement. The drop of strength to less than the slow residual, observed in the fast stage 28, was further verified during stages 30 and 32.

The results obtained from stages 5 and 28 both carried out at 1000 mm/min indicated a transition in the type of the behaviour of the soil during fast shearing from a rate-independent to a rate-dependent one with increasing normal stress. Existing results from other soils suggested, however, the opposite view, that is, for soils showing a loss of strength during fast shearing, the loss of strength either becomes more pronounced at decreasing normal stress or is independent of the normal stress. To clarify this apparent contradiction, it was decided to reduce the nominal normal stress back to 264.6 kPa and investigate the behaviour of the soil during fast shearing again. A rate-independent behaviour would indicate that the observation, made on the previously tested soils, did not apply in the case of this weathered volcanic soil, whereas a rate-dependent behaviour would show a change in the material properties within the shear zone of the sample.

Slow standard shearing was resumed in stage 34. Stages 35 to 42 involved alternating fast and slow shearing. No recording of the shear loads was performed during stage 37, due to failure of the recording system. The loss of strength, observed in stages 35 and 39, indicated that a change in the material properties within the shear zone had caused the transition of the behaviour of the soil from a rate-independent to a rate-dependent one. Stage 41, carried out at 1000 mm/min, involved a displacement of only 16.3 mm. The peak stress ratio, observed in the slow stage 42, suggested that the fast peak strength is associated with both structural changes, taking place within the shear zone, and generation of negative pore water pressure. The test terminated with the stage 42.

The final water of the sample was as follows:

average	:	32.40%
shear zone	:	31.60%

Description of the test specimen

The sample separated along a non-planar shear surface. A shear zone which had variable thickness (the maximum thickness was 2 mm at the mid-radius of the sample) was present. The shear zone was bounded by two shear surfaces; the upper (top) one was the main slip surface. The material within the shear zone had a contrasting darker colour and was markedly finer than the rest of the sample which appeared to be intact.

A thin section of the sample, viewed under the optical microscope, showed a much finer texture within the shear zone than for the rest of the sample (Plate 5, photographs 34 and 35). A well-defined shear zone with discontinuous shear surfaces at its boundaries was observed. The massive particles within the shear zone had their long axes orientated parallel to the direction of shear.

Discussion of the results

While degradation of the soil within the shear zone had affected the undrained soil behaviour during fast shearing, it had no effect on the drained residual strength of the soil which remained constant (compare the results from the slow stages 1 to 14 with those from the stages 34 to 40).

Bishop (1974) considered the particle breakdown of crustal materials and its effects on the residual strength. The breakdown of rock fragments on shearing became of particular interest at the time of the inquiry into the Aberfan Disaster. For rockfills and colliery waste, continuous particle degradation occurs with strain even at relatively low stresses, and in the case of mudstones and shales the residual material may be a fine grained cohesive material of measurable plasticity. This was first observed during the investigation into the Aberfan Disaster (Bishop et al, 1969). In this case, the material passing the 200 sieve and giving $\phi'_p=39.5^\circ$ in drained triaxial tests, degraded to a cohesive material with $PI=16\%$ and $\phi'_r=17.5-18.5^\circ$ on the lower section of the slip surface where a displacement of 23.10 m was estimated. Attempts to reproduce this phenomenon in the laboratory in the ring shear apparatus, using material passing the No 7 sieve, have reduced $\phi'_r=37^\circ$ to $\phi'_r=20^\circ$ at a displacement of one third of a kilometer. Bishop et al (1969) concluded that the larger displacement, required to approach but not fully to achieve the field value,

suggested either a difference in the material properties or an additional agency in the field, such as chemical weathering resulting from the presence of pyrites. The Aberfan flowslide velocity was estimated to be 10-20 mph (Bishop, 1973). It is possible that the material from the Aberfan showed hints of the behaviour, exhibited by the weathered volcanic soil from Mauritius, and a ring shear test at the appropriate rate of displacement would enable to achieve the value of the residual strength observed in the field.

Mandl et al (1977) simulated the grain breakage in tectonic faulting by carrying a series of ring shear tests on densely packed sugar grains both dry and kerosene-saturated. The experiments were carried out under different normal stresses (3 and 9 bar). A compact shear band was formed and its pore space was filled with fine material, produced by grain breakage. The reduction in pore space inside the shear zone, caused by the filler material, increased with the normal stress. They concluded that grain breakage is more likely to be a consequence of the shearing motion itself. To achieve comparable crushing in the absence of shearing motion a compressive load several times as high would be required. The shearing motion continuously alters the position of an individual particle such that it may eventually be fractured under the combined action of the local contact forces. Another mechanism that might contribute to grain crushing is the continual and changing formation of grain bridges.

Tests 29, 30, residual soil from Japan

Operator: Tika, T.M.

Site and soil description

The soil was obtained from the exposed surface of the huge Mt. Ontake landslide area, Japan. Medium to large scale landslides took place in the area of Mt. Ontake at the time of the Naganoken-Seibu earthquake of 1984 in Japan. Its magnitude of 6.8 placed this event among the greatest to have occurred in this area in historic times. By far the most spectacular event in this earthquake was a cataclysmic debris flow that originated on the south flank of Mount Ontake. A vast amount of soil and rock, as much as 35 millions m³, was detached from the flank of the mountain between the altitudes of 1850 and 2250 m, and after

bouncing off several times it plummeted down the gorge through a distance of about 12 km. The travel time for the debris flow was estimated to have been of the order of 8 min and the average speed was therefore 90 km/h. A detailed description and analysis of the landslides is given by Ishihara et al (1986) and Ishihara and Hsü (1986).

One of the features common to all the landslides, occurring in the area of Mt. Ontake, is that a wind-blown yellowish pumice deposit of pliocene age exists beneath the scoria and lava, which are the products of a series of volcanic activities in the subsequent Pleistocene era. This pumice is highly porous and weak and is thought to be responsible for triggering the slides. The pumice was identified as partly saturated sandy silts, composed of highly porous halloysite, and having a water content as high as 70-120%. No movement had taken place prior to earthquake and the slides were therefore newly activated on a fresh slip surface mainly due to the inertia forces of the earthquake.

The results of two ring shear tests are considered here.

Mineralogy

Clay minerals ($\% < 4\mu\text{m}$) : Biotite

Coarse fraction ($\% > 4\mu\text{m}$) : Sodium plagioclase, Hornblende, Biotite,
Quartz, Magnetite.

Test 29

Sample preparation

The soil had a yellowish colour and a natural water content of 77%. Because of the large size of the massive particles, it was air-dried, mechanically ground (grinding in a mortar with a pestle was found to be ineffective) and sieved through the $425\mu\text{m}$ sieve. It was mixed then with distilled water and left to hydrate for at least twenty four hours before testing in the ring shear apparatus.

Index properties

The index properties of the soil, prepared as the sample for the ring shear test, were as follows:

LL=63.4, PL=57.5, PI=5.9%

Gradings

Particle size distributions of the soil as initially received, with and without pretreatment, and as used for the ring shear test are shown in Fig. A2.29.

Tests results

The soil was placed in the apparatus at a water content of 77.1%. It was then consolidated to a normal stress of 264.5 kPa. The decrease of the water content due to the compression was approximately 13%. From the consolidation stage, it was estimated that the slow standard rate of displacement, 0.0117 mm/min, used throughout the test, would ensure 95% dissipation of the pore water pressure during shearing in less than 0.001 mm of displacement.

The stress ratio versus cumulative displacement is shown in Fig.A2.32a to A2.32c and the results from the test are summarized in Table A2.28

Stage 1 involved slow shearing through the peak to the residual strength. The peak stress ratio was not recorded, because of the failure of the data recording system (Orion). Stage 2 followed at 120 mm/min. The residual strength of the soil was established again during slow stage 3. Stage 4 was carried-out at 1000 mm/min. No significant rate effect on the residual strength of the soil was observed. In the subsequent stages 5 to 13 the effect of lower (stages 6 and 8), intermediate (stage 10) and faster (stage 12) rates of displacement on the residual strength of the soil was investigated. At the end of the slow stage 13, the nominal normal stress was increased to 527.9 kPa.

Slow shearing was commenced again in stage 14 and was followed by fast shearing at 1000 mm/min during stage 15. Stages 16 to 26 involved alternating slow and fast shearing. After the completion of stage 26 it was intended to shear the sample at 6000 mm/min. The fast motor M5, however, could not apply the shear force, required for the initiation of shearing due to both the high coefficient of friction and normal stress on the sample. After five trials (substages 27a to 27e) which resulted in a displacement of 4.3mm of the sample, a rate of displacement of 3500 mm/min was finally used during stage 27f. The slow residual strength was established again in stage 28.

Stage 29 was carried out at the same rate of displacement, as stage 25, in order to check whether there was a change in the type of the rate effect on the residual strength of soil with increasing cumulative displacement. No such change was observed and at the end of the stage, the nominal normal stress was reduced to 440.1 kPa in order to use faster rates of displacement. The shear force, required for the initiation of shearing was still high. A rate of displacement of 4400 mm/min was finally used in stage 31b. The test ended with stage 32, carried out at 4400 mm/min.

The final average water content of the sample was 54%.

Description of the test specimen

The sample when viewed under a microscope exhibited a shear zone. The shear zone had variable thickness (it was approximately 2.5 mm at the mid-radius of the sample) and a finer texture than the rest of the sample.

A thin section of the sample (Plate 6, photographs 36 to 38) showed that the elongated massive grains within the shear zone had their long axes orientated parallel to the shear direction. There was no evidence of preferred orientation of the clay particles within the shear zone, neither of any shear surfaces at the boundaries of the shear zone.

Discussion of results

An independent investigation into the effect of rate of shearing on the friction angle of the soil from the Ontake avalanche was reported by Sassa (1987). Sassa (1987) found that the friction angle remained constant within a range of displacement rates between 0.6 and 600 mm/min. The value of the friction angle was found to be 34.7° which compares well with the values of 32.4° and 33.2° at the normal stresses of 200 kPa and 400 kPa respectively, obtained from Test 29.

Test 30

Sample Preparation

The soil used for this test came from the initial batch of the soil, obtained from the exposed surface of the Mt. Ontare landslide. It had, however, a higher clay fraction than the soil used in Test 29. It was intended to investigate whether an increase in the clay fraction would

change the rate-independent behaviour of the soil used in Test 37, to a rate-independent one.

The clay-fraction was increased by sedimenting part of the soil and adding the suspended fine material to the remaining (the fine material, $\% \leq 4\mu\text{m}$, extracted from each 100 gr of sedimented soil was added to 100 gr of the remaining).

Index properties and gradings

The index properties of the soil, prepared as the sample for the ring shear test, were as follows:

$$\text{LL} = 62.8, \quad \text{PL} = 50.0, \quad \text{PI} = 12.8\%$$

Grading curves of the soil are shown in Fig A2.29.

Test Results

The soil was placed in the apparatus at a water content of 60.9%. It was then consolidated to a nominal normal stress of 527.9 kPa. The decrease of the water content of the sample due to the compression was approximately 8.2%. From the consolidation stage, it was estimated that the slow standard rate of displacement, 0.0925 mm/min, used throughout the test, would ensure 95% dissipation of pore water pressure during shearing within 0.1 mm of displacement.

Each shearing stage is summarized in Table A2.29. The complete sequence of shearing can be followed in Fig. A2.33a to A2.33d.

Stage 1 involved slow shearing through the peak to the residual strength and was followed by stages 2 to 9, carried out at alternating fast and slow rates of displacements. At the end of stage 9, the nominal normal stress was reduced to 264.6 kPa.

Slow shearing was commenced in stage 10. The slight increase of the residual coefficient of friction during this stage was due to the friction developed in the gap. Stages 11 to 16 involved alternating fast and slow shearing. Stages 17 and 18 were both carried out at 6000 mm/min with a pause of 18 min allowed between them. In stage 18, the immediate increase in the stress ratio was followed by the recovery of the same low value, observed during stage 17. The slow residual strength was established again during stage 19. The slow standard rate

of displacement was reduced to 0.0118 mm/min in stage 20. Stages 21 and 22 were both carried-out at 6000 mm/min with a pause of 1.3 min allowed between them. The same pattern of behaviour, as in stages 17 and 18, was observed. Stages 23 and 24 were performed at 1.8945 and 0.0925 mm/min respectively. Stage 25 was carried out at 1000 mm/min, as stage 13, in order to investigate any change of the behaviour of soil at increasing cumulative displacement. No such change was observed. Stages 26 to 30 involved shearing at slow and fast rates of displacement. At the end of the stage 30, the nominal normal stress was reduced to 132.9 kPa.

Slow standard shearing was resumed in stage 31. The remaining stages of the test, 32 to 36, were carried out at alternating fast and slow rates of displacement. After stage 36, the sample was removed from the apparatus.

The final average water content of the sample was 50.5%.

Description of the test specimen

The same observations, as for the test specimen from Test 29, were made.

A2.4 OTHER NATURAL SOILS

Tests 31, 32 *Tuff from Panana*

Operator : P. Martins

Reference : Vaughan (1981), Martins (1983)

The results from two ring shear tests on Tuff from Panana are considered here. Test 32 was conducted on a sample of weathered material, whereas Test 33 was performed on a sample of mechanically degraded parent rock, the intention being to test the parent material with possibly a slightly different mineralogy.

Procedure

Outlined below is a general description of the procedures followed. For more specific details reference should be made to the section dealing with the individual tests.

Soil passing through a 425 μ m sieve was mixed with distilled water and allowed to hydrate for several days. The soil was then placed in the machine at a water content close to the plastic limit and consolidated in stages to the maximum normal stress available (1000 kPa). The sample was then swelled back to the first operating stress (either 100 kPa or 300 kPa).

The sample was sheared slowly to peak strength. The rate was then increased to 0.1 mm/min and the sample was sheared until residual conditions were achieved (The residual strength was measured at a displacement rate of 0.006 mm/min). The sample was then unloaded in shear and allowed to consolidate fully prior to performing a fast stage. After the fast stage, the sample was again unloaded in shear and was usually allowed to consolidate fully before slowly reshearing to residual conditions. Fast stages were monitored using a recording oscilloscope. In order to further investigate the nature of the rate effects, various probes were conducted during the tests. These probes were conducted by reshearing the sample at a different rate from that immediately preceeding and monitoring the immediate response. Probes were conducted either after allowing the sample to fully consolidate after the previous stage, or immediately afterwards so that pore pressure effects could be investigated.

Once testing had been completed at the first normal stress level, the sample was unloaded in shear and then subjected to a new normal stress level so that the stress level dependence of the various rate effects could be ascertained. After a further series of stages, the sample was removed, the shear surface examined, and the index properties determined.

Test 31

Index properties

LL=58, PL=41, PI=17%

It should be noted that the material used for determining the index properties was prepared in the same manner as the test specimen. Therefore the properties measured are bulk values and do not represent conditions on the shear surface itself where local degradation leads to an increased plasticity index (See Heley and MacIve, 1971).

Sample preparation

As described in the section on procedures.

Test results

The first series of stages was performed at a normal stress at 300 kPa. The angles of shearing resistance at peak and at residual conditions (at 0.006 mm/min) were determined as follows:

σ'_n (kPa)	ϕ'_{peak}	ϕ'_r
300	33.0°	9.0°
100	-	11.2°

Two stages at a displacement rate of 100 mm/min showed that the pre-slip strength had increased by about 30%, and that the angle of friction increased further with post-slip displacement. A slow probe following the second stage showed an immediate decrease in shearing resistance roughly equal to the immediate gain at the beginning of the fast stage. After shearing slowly to residual conditions, a stage at 10 mm/min was performed. This indicated a more modest increase in strength. The sample was resheared to the residual strength and then unloaded in shear and swelled to 100 kPa.

A series of fast stages was performed and the increased strength noted. Slow probes following fast tests again indicated that the immediate decrease in strength roughly equalled the immediate gains on fast shearing. Fig. A2.34a to A2.34d illustrate typical results during fast and slow shearing.

Description of the tested sample

The shear surface was seen to be highly polished, as would have been expected from the low angles of friction observed.

Test 32Index properties

LL=50, PL=36, PI=14%

(See note concerning Atterberg limits in description of Test 31).

Sample preparation

The parent rock was subjected to six cycles of wetting and drying after

which the material was mechanically ground. The soil was then sieved and that passing a 425 μ m sieve was used in the manner described earlier.

Test results

The first series of stages was performed at a normal stress of 100 kPa. The angles of shearing resistance at peak and at residual conditions were determined as follows:

σ'_n (kPa)	ϕ'_{peak}	ϕ'_r
100	44.0 $^{\circ}$	23.0 $^{\circ}$
300	-	17.5 $^{\circ}$

Truly residual conditions were achieved only after displacements in excess of one meter, and so the first stage at 100 mm/min, conducted after a displacement of 700 mm, had a strength exceeding the lowest residual value.

Fast stages again indicated the existence of an immediate increase in strength, but unlike the first series of tests, the shearing resistance decreased after slippage had occurred. Slow probes conducted immediately after this first fast stage indicated that in fact the angle of shearing resistance had increased (as observed in the first series), but that positive pore pressures generated by fast shearing had reduced the effective stresses and thereby the available strength. This pore pressure phenomenon was observed in all stages with the degraded parent rock and represents another rate dependent effect. The above observations were confirmed by the subsequent test stages. Altogether three stages at 100 mm/min, two at 10 mm/min and one at 1 mm/min were performed at a normal stress of 100 kPa; and various probes were performed to investigate the pore pressure effect, and also the effect of the initial strength on the immediate gain in strength. The results are shown in Fig.A2.35a. The sample was then consolidated to 300 kPa and a further series of stages with probes was performed. The results are shown in Fig.A2.35b.

Description of the test specimen

The shear surface was striated circumferentially and was slightly polished.

Tests 33,34, *Soil from the Senerchia and Buoninvetre slides*

Operator : Lemos, L.J.L

Reference : Lemos (1986)

Soil description

The soils came from Irpinia, South of Italy. The area had been subjected to an earthquake in the evening of the 23 of November 1980 with a magnitude of 6.8. Following the earthquake, a series of old landslides which remained apparently stable over a long period of time were triggered. The most important cases of reactivation, such as those at Senerchia and Buoninvetre, seem to have occurred as a result of strong seismic tremors, while even violent rainstorms appear to have no effect on these slides at least for 30 years, as can be deduced from the age of houses built in the area (Cotecchia and Del Prete, 1984). It was reported that the Senerchia and Buoninvetre slides did not run down immediately following the main shaking from the earthquake, but started to move slowly a few minutes to several hours after the shaking of the earthquake. (Cotecchia and Del Prete, 1984).

A comprehensive description of the site and geological history has been given by Maugeri et al (1982). The baserock and mountains in this area are composed of tectonically overthrust and faulted units of dolomites and limestones of the Trias to Miocene era. The rock formations are overlaid by the deposits of variegated clay named varicoloured clay which is of marine origin. On the top of this clay deposit, there exists a series of colluvium of chaotically positioned sandy clays containing calcareous and marly pebbles of different size. It was discovered that the basal sliding surface ran mainly through the deposit of the variegated clay at a depth of 40 to 50 m from the ground surface well below the elevation of the ground table. The mineralogical composition of variegated clay is essentially kaolinitic and smectitic (Cotecchia and Del Prete, 1984). It is a grey overconsolidated marly clay with prominent flat laminae containing small-size calcareous pebbles. It is sometimes interbedded with calcareous marls, green clays and cemented fine-grained sandstones. An envelope of the particle size distribution of the clay, quoted by Battista et al (1986), is shown in Fig. A2.36.

Lemos (1986) performed three ring shear tests on soil from the Senerchia and Bouninvetre slides. The results of two of the tests which involved fast shearing are considered here. Test 33 was conducted on soil from the Severchia slide (referred to as Senerchia 2 by Lemos, 1986) whereas Test 34 was conducted on soil from the Bouninvetre slide.

Sample preparation

The sample preparation was common for the two tests. The soil was air-dried, mechanically ground and sieved through a 425 μ m sieve. It was remoulded to a water content slightly wetter than the plastic limit and left to hydrate for twenty four hours before being placed in the ring shear apparatus.

Test 33

Index properties

The index properties and the clay fraction of the soil (light brown), used for the ring shear test, were as follows.

LL=51.7, PL=23.8, PI=27.9 and CF=37.0%

Tests results

The soil was placed in the apparatus at a water content of 28.4%^{and} was then consolidated to a normal stress of 920 kPa. From the consolidation stage, it was estimated that a rate of displacement of 0.0035 mm/min would ensure 95% dissipation of the pore water pressure during shearing within 2 mm of displacement.

The stress ratio versus cumulative displacement is plotted in Fig.A2.37a and Fig.A2.37b and the results from the test are presented in Table A2.30.

Stage 1 involved slow shearing through the peak to the residual strength. Stage 2 was carried out at 3.69 mm/min. The slow residual state was established again in stage 3. Stage 4 consisted of three sub-stages, carried out at 133 mm/min, with and without a pause between them.

Stages 6,7 and 8 involved slow shearing under the normal stresses of 485, 915, 480 kPa and 232 kPa respectively. The displacement needed to

establish again the residual conditions was considerable, suggesting that some disturbance of the alignment of the clay particles in the shear zone occurred when the normal stress was decreased. Stages 9 and 10 involved shearing at 133 mm/min and 0.003 mm/min respectively. Stage 11 consisted of two substages, carried out at 133 mm/min, with a pause between them and the shear load acting on the sample. The test terminated with the stage 12, carried out 0.0033 and 0.01 mm/min.

The final average water content of the sample was 26.2%.

Description of the test specimen

The sample separated through at least two well-defined and highly polished shear surfaces, one of them being horizontal and the other non-planar.

Test 34

Index properties

The index properties and the clay fraction of the soil (light brown) were :

LL=53.3, PL=27.0, PI=26.3 and CF=39.0%

Tests results

The soil was placed in the apparatus at a water content of 31.5% and was then consolidated to a nominal normal stress of 960 kPa. From the consolidation stage, it was estimated that the slow standard rate of displacement of 0.003 mm/min would ensure 95% dissipation of pore water pressure due to shearing in a displacement of 1mm.

The complete sequence of shearing can be followed in Fig. A2.38a to A2.38d. The results of the test are presented in Table A2.31.

Stage 1 involved slow shearing through the peak to the residual strength. At the end of the stage, the normal stress was reduced to 490 kPa.

Stage 2 was carried out at the slow standard rate of displacement. No increase in the stress ratio was observed. This indicated that a sliding surface had been formed. Stages 3 to 6 involved shearing at 0.10, 0.003 and 1 mm/min respectively. Stage 7, carried out at 10.7

mm/min, consisted of three substages with a pause of 5 sec between them. The pauses had no significant effect on the stress ratio which remained constant. In stage 8, the shear direction was initially reversed by mistake, but no readings were taken. The sample was unloaded in shear and allowed to consolidate. Shearing was then resumed at 10.7 mm/min in the forward direction. Stage 9 followed immediately afterwards at 0.10 mm/min. Stage 10 was performed at 0.05 mm/min. Stage 11 consisted of 5 substages at 100 mm/min with a short pause between them. The general pattern of the behaviour observed was that of an immediate increase in the stress ratio, followed by a further increase and a small decrease afterwards. Stage 12 was carried out at 0.05 mm/min immediately after stage 11. After a pause, stage 13 was performed at 0.003 mm/min. In the following stages 14 to 17, carried out at alternating fast and slow rates of displacement, the influence of the static stress ratio, established prior to fast shearing was investigated. At the end of stage 17, the normal stress on the sample was reduced to 200 kPa.

Slow standard shearing was commenced in stage 18. Stage 19 was carried out at 0.003 and 0.05 mm/min alternately, with the gap between the confining rings opened and closed respectively. The remaining stages of the test involved shearing at fast and slow rates of displacement. After the completion of stage 26, the sample was removed from the apparatus.

The final water content of the sample was as follows:

shear zone :	25.5%
average :	25.9%

Description of the test specimen

The sample separated through a well-defined, highly polished, non-planar shear surface. For a detailed description of the observations of a thin section, prepared from the sample, reference should be made to Lemos (1986).

A2.5 ARTIFICIAL SOILS

Test 35, clayey silt

Operator : Tika, T.M

Soil description

The soil used for this test was a mixture of an artificial silt and Speswhite kaolin (silt : 79%, kaolin : 21% of dry unit weight). The purpose was to produce a mixture having a grading similar to that of the clayey siltstones from the Kalabagh dam site with a clay fraction of about 20%.

Sample preparation

The silt was mixed thoroughly with the kaolin in a dry state to produce a uniform material. The mixture was then saturated with distilled water, remixed again, and left to hydrate for twenty four hours before testing in the ring shear apparatus.

Index properties and grading

The index properties of the soil were as follows:

LL=24.6, PL=15.8, PI=8.8%

Particle size distributions of the soil, determined before and after the ring shear test, were identical (Fig. A2.39).

Test results

The soil was placed in the apparatus at a water content of 24.2%. It was then consolidated to a normal stress of 966.8 kPa and swelled to 527.9 kPa. The compression of the sample during the consolidation stage was equivalent to a decrease of the water content of 8.4%. A rate of displacement of 0.0102 mm/min was chosen as the slow standard, so as to ensure 95% dissipation of the pore water pressure during shearing in a displacement of 0.2 mm.

Each shearing stage is summarized in Table A2.32. The complete sequence of shearing can be followed in the Fig. A2.40a and A2.40b.

Stage 1 involved slow shearing through the peak to the residual state. It was followed by the series of stages 2 to 7 involving alternating fast and slow shearing. At the end of stage 7, the nominal normal stress on the sample was reduced to 89 kPa.

Slow shearing was started again in stage 8. Stages 9 to 14 were

carried out at alternating fast and slow rates of displacement. In order to verify the drop of strength to less than the slow residual, observed in the fast stage 13, stage 15 was carried out at 2500 mm/min. Though the results from the stage 13 and 15 agreed qualitatively, the state of the pore water pressure during these stages is uncertain due to their short duration. In the following stages 16 and 17, the sample was retested slowly at the nominal normal stresses, 89 and 527.9 kPa respectively, in order to check whether a change in the material properties within the shear zone of the soil had taken place due to segregation. No such change was observed. Stage 18 followed at 9.3 mm/min. The test terminated with the slow stage 19.

The final water content of the sample was as follows:

average : 16.3%

shear zone : 19.0%

Description of the test specimen

No separation of the test specimen on the shear zone was observed. The shear zone had been formed closer to the upper loading platen of the apparatus.

Test 36, sandy clay

Operator : Tika, T.M

Soil description

The soil used for the test was a mixture of a medium to fine sand (identical to that used in Test 43) and Sphewhite kaolin (sand : 66%, kaolin : 34% of dry unit weight). The purpose was to perform a test on a sand-clay mixture (as opposed to the silt-clay mixture used in Test 36).

Sample preparation

The mixture was produced by thoroughly mixing the sand and the kaolin in a dry state. Care was taken to produce a uniform material and to avoid segregation of the sand grains. The mixture was re-mixed again, after the addition of distilled water, and left to hydrate for twenty four hours.

Index properties and grading

The index properties of the soil were as follows:

LL=22.5, PL=12.0, PI=10.5%

A particle size distribution of the soil is shown in Fig. A2.39.

Test results

The soil was placed in the apparatus at a water content of 32.1% and consolidated to a nominal normal stress of 264.6 kPa (consolidation time : 5 min, $C_v = 9.1 \text{ m}^2/\text{yr}$). The decrease of the water content due to the compression was approximately 8.4%. From the consolidation stage, it was estimated that the slow standard rate of displacement of 0.1020 mm/min, used throughout the test, would ensure 95% dissipation of the pore water pressure in a displacement of 0.4 mm.

The stress ratio versus cumulative displacement is plotted in Fig. A2.41a A2.41b and A2.41c. The results from the test are presented in Table A2.33.

Shearing through the peak to the residual state was started in slow stage 1 and continued during stages 2 to 5 at alternating fast (130, 1100) and slow rates of displacement. After the completion of the slow stage 5, the nominal normal stress was increased from 264.5 Pa to 527.9 kPa and stage 6 was performed immediately afterwards at 1100 mm/min. The intention was to investigate the effect of a change of the normal effective stress during fast shearing on the behaviour of soil. Stage 7 involved slow reshearing to the residual conditions. Stages 8 to 15 were carried out at alternating fast (1100, 100, 3 and 2500 mm/min) and slow rates of displacement. At the end of stage 15, the sample was swelled back to a nominal normal stress of 264.6 kPa.

Slow shearing was commenced again in stage 16. Stages 17 to 22 followed at alternating fast and slow rates of displacement. During stage 23 the procedure, followed in stage 6, was repeated. Fast shearing at 100 mm/min was commenced immediately after the nominal normal stress was increased from 264.6 to 703.5 kPa. After stage 23 and a pause for consolidation, the nominal normal stress was increased to 527.9 kPa. Stage 24 was used to establish again the slow residual state. The test ended with stage 25 during which the reverse procedure of that followed in stages 6 and 23 was implemented. Fast shearing at 130 mm/min

started immediately after the nominal normal stress was reduced from 527.9 to 264.6 kPa (swelling time : 8 min, $C_s = 4.3 \text{ m}^2/\text{yr}$).

The final average water content of the sample was 14.5%.

Description of the test specimen

A shear zone was present within the sample. It had a variable thickness (5mm at the mid-radius of the sample) and had been formed close to the upper loading platen of the apparatus.

A2.6 SOILS OF VERY HIGH PLASTICITY

Test 37, kaolin

Operator : Lupini, L.F

Reference : Lupini (1981)

Soil type

The soil used for this investigation was a kaolin commercially available under the name Speswhite.

Index properties and grading

The index properties of the tested soil were as follows:

LL=72, PL=36, PI=36%

A grading curve of the soil is shown in Fig.A2.42 (Martins, 1983)

Test results

The kaolin was prepared in a very wet state at a water content of 132% (which is approximately twice its liquid limit). Because of the unusually wet state and in order to have a sample 20 mm thick after consolidation, the consolidation stage had to be started on a sample twice as thick (roughly 40 mm). This means that the upper yoke could not be placed until the upper platen did not clear the plane at the top part of the upper confining rings. Also considerable squeezing of this very wet material occurred, in particular at very low normal stresses, during the consolidation stages. This squeezing was between the loading platen walls and the upper confining rings. This excess material was washed and siphoned out of the water bath to avoid unnecessary corrosion problems.

The consolidation stage were dictated by the amount of material which was squeezed out. Up to a normal effective stress of 18.8 kPa, The subsequent loading stages were as follows and no further squeezing occurred.

Stage	σ'_n (kPa)
i	18.8
ii	40.7
iii	68.9
iv	176.7
v	352.2

During consolidation stages iv and v, readings were taken so that a drained nominal rate could be chosen to give 95% dissipation of the excess pore water pressure at a displacement of 1.0 mm. Because this material is a relatively coarse kaolin, it consolidated so quickly that it was impossible to read the settlement dial gauges for the first 30 sec. Consolidation times were accordingly very small. Stage iv: 8.5 min, stage v: 4.5 min, approximately.

Shearing was started at a drained rate of 0.015 mm/min and continued at a normal stress of 352.2 kPa until a residual angle of friction approaching 12.3° was reached.

The complete sequence of shearing can be followed in Fig. A2.43a and Fig. A2.43b and is discussed below.

After completion of the first shearing stage the torque was relieved and the normal effective stress was increased to 518.9 kPa and was maintained at this level for all the remaining stages of the test. Stage 2 was carried out at the same rate, 0.015 mm/min, and gave residual values of the friction angle between 11.3° and 11.8° , suggesting that for this material the residual coefficient of friction is stress level dependent. Stage 3 was carried out at a faster rate of 0.430 mm/min. A significant increase in strength occurred and since the stage lasted for more than four times the full consolidation of the specimen it is believed that this increase is probably due to 'viscous' effects (rather than an incomplete dissipation of induced excess pore

water pressure). Stage 4 was carried out at 3.60 mm/min, the maximum rate possible with the gear boxes of ring shear No. 1. Here again an increase in the torque was observed. This shearing stage, however, only lasted just over 26 min (about 3 times that for full consolidation of the specimen), but the stress-displacement curve did not show significant variation except for a slight tendency to increase. A small peak was observed during the first part of this stage. This overall behaviour supports the view that non pore pressure effects were responsible for the increase in strength. A consolidation stage was carried out after this stage (after relieving the torque from the sample). Small consolidation movements were observed ($<0.25\%$) suggesting that if any excess pore pressure was induced during fast shearing, this may have been confined to the shear surface zone or alternatively it must have been very small. In addition, during re-shearing at the drained rate (stage 5, 0.0015 mm/min) after a 1 hr consolidation time the strength was practically unaltered from the values of stage 2. (A small peak of 12.2° was observed, this value decaying very slowly to 11.7°).

Stage 6 was carried out at a rate of 130 mm/min (the torque was not relieved between stage 5 and 6). In order to achieve this very fast rate the gear box mechanism was disconnected and a special engine was attached directly to the ring shear at the drive to the worm shaft. Stage 6 was commenced and readings were taken for 5 min approximately. Unfortunately this time is too short to draw any conclusions about development and dissipation of excess pore pressure. However the readings were quite stable for displacements over 100 mm. The reason why this stage was not carried out for a longer time was because a loss of material for a significant displacement could have ended the test prematurely. (The cumulative displacement since the beginning of the test until the end of stage 6 was over 1200 mm). Consolidation movements were again insignificant. The extent of the damage to the structure due to this very fast rate was studied during stage 7. Stage 7 was a drained probe (0.015 mm/min) which showed that the use of the very fast rate did not change or disturb significantly the alignment of the platy particles in the shear surface. A small peak however was observed ($\phi' = 12.4^\circ$).

At this point only faster or equal to the nominal drained rate were

used. In order to establish a more complete picture slower rates were used during stages 8 and 9. These were 0.001 mm/min and 0.0001 mm/min, the last one being the slowest possible rate in the ring shear machine No. 1. Stage 8 gave values of the order of 11.5° . Stage 9 gave values fractionally smaller, 11.2° to 11.3° .

During stage 10 the nominal drained rate was re-established and values of ϕ'_r of the order of 11.9° were obtained. During the last part of stage 7 and throughout all further stages (8,9 and 10) the levelling screws of the horizontal loading arm had run out of travel. A qualitative assessment of stages 8, 9 and 10 can be given on the observed difference in the stress ratio between, for instance, stage 10 and 9 and 10 and 8. All of these last stages were very short for any appreciable loss of material to occur. The deviation of the loading arm from the horizontal should have been very small.

The average water content of the sample at the end of the test was 40.8%.

Description of the test specimen

The sample could not be separated on one continuous shear surface. Several shear surfaces seemed to be present in the mid-part of the sample. A thin section at the mean radius of the sample showed that a large shear zone, bounded by two irregular shear surfaces, had been formed. The fabric in the shear zone was rather complex. However the clay platelets were strongly orientated in the direction of shear. Outside the shear zone the clay platelets were, in places, orientated at approximately 45° to the shear direction.

Tests 38,39,40,41,42, blue London Clay

Operator : Tika, T.M

The results of five ring shear tests on blue London clay are considered here. Tests 38 and 39 were standard ring shear tests in which the effect of fast shearing on the residual strength was investigated. Tests 40 to 42 involved soil shearing against a rough stainless steel interface.

Soil description

The soil used for the tests was blue London clay from Canons Park, North London. It came from the same batch of soil, as that used by Lemos (1986). The engineering behaviour of blue London clay both in-situ and laboratory has been investigated by Jardine (1985).

Mineralogy

Results from an X-ray diffraction of the soil (University College, London) gave the following results:

coarse fraction : Quartz as impurities
($\% > 4\mu\text{m}$)

clay fraction ($\% < 4\mu\text{m}$) :

{	Smectite	50%
{	Kaolinite	25%
{	Muscovite	25%

Index properties and grading

The index properties of the soil were as follows:

LL=71, PL=26, PI=45%

A particle size distribution of the soil is shown in Fig A2.42

Sample preparation

The soil was air-dried and mechanically ground. It was remoulded with distilled water to a plastic state and left to hydrate for at least two weeks before testing in the ring shear apparatus.

Tests 38,39

Consolidation stage

The soil was placed in the apparatus at a water content of 59.3% and 60% in Tests 39 and 40 respectively. It was then consolidated to a normal stress of 922.9 kPa and swelled to the first operating nominal normal stress (either 264.6 kPa or 527.9 kPa). The compression of the sample during the consolidation stage was equivalent to a decrease of the water content of 33.7% and 36.8% for Tests 39 and 40 respectively. A rate of 0.00625 mm/min was chosen as the slow standard rate of displacement, so as to ensure 95% dissipation of the excess pore water pressure during shearing in less than 0.9 mm of displacement.

Test 38

Test results

The test was carried out at two nominal normal stresses, 527.9 kPa and 440.1 kPa. The complete sequence of shearing can be followed in Fig. A2.44a and A2.44b. Results from the test are presented in Table A2.34.

Slow shearing was commenced through the peak to the residual strength in stage 1. In order to obtain a large displacement quickly, the rate of shearing was increased by a factor of $\times 2$ initially and $\times 23$ afterwards. Because of the tendency of the gap between the upper and lower confining rings to open during this speeding-up process and the loss of soil by squeezing (the loss of soil increases with increasing normal stress on the sample), the normal stress on the sample was reduced to 440.1 kPa.

Slow shearing was commenced again in stage 2 with the confining rings completely closed initially. When a large displacement had been accumulated, the gap was re-opened and the rate of shearing was reduced to the slow standard one. The remaining stages of the test, 2 to 11, were carried-out at alternating fast and slow rates of displacement. After the completion of stage 12, the sample was removed from the apparatus.

The average water content of the sample at the end of the test was 29.8%.

Description of the test specimen

A shear zone, bounded by two shear surfaces, was present within the sample. The principal shear surfaces was non-planar, polished and slightly striated.

Photographs 39 and 40 in Plate 6 illustrate scanning electron-micrographs from the principal shear surface of the specimen.

Test 39

Test results

The test was performed at a nominal normal stress of 264.6 kPa. The complete sequence of shearing can be followed in Fig. A2.45a, A2.45b and A2.45c. Each shearing stage is summarised in Table A2.35.

Stage 1 involved slow shearing through the peak to the residual state.

In order to obtain a large displacement quickly, the shearing rate was increased by a factor of $\times 70$. The confining ring gap was kept closed during this procedure in order to minimise the loss of soil by squeezing. When the desired large displacement had taken place, the gap was reopened and the rate of shearing was reduced to the slow standard one. The remaining stages of the test, 2 to 13 were carried-out at alternating fast and slow rates of displacement. During the stages 6 and 8 the rotational potentiometer, described in section 3.2.3, was used for the measurement of the displacement during the fast shearing (Fig A2.45d). After stage 13, the sample was removed from the apparatus.

The average water content of the sample measured at the end of the test was 34%.

Description of the tested specimen

The sample was split through a well-defined, polished, slightly striated, non-planar shear surface.

Discussion of the results

The drained residual friction angle, measured in Tests 38 and 39, compares well with available data in the literature which are presented here:

Reference	LL (%)	PI (%)	CF (%)	σ'_n (kPa)	ϕ'_R	Comments
La-Gatta (1971)	71	49	57	400	7.7°	
Bishop et al (1971)	72	43	57	40-280	9.2°-9.5°	Wraysbury, montmorillonite consisted 10% of clay fraction
Lupini (1981)	75	51	48	220-352	6.1°-7.1°	South Ockendon, montmorillonite consisted approximately 42% of the clay fraction
Lupini (1981)	57	36	36	177-352	19°-20°	South Ockendon, Silty London clay
Lemos (1986)	71	45	57	410	10.5°	Canons Park, soil-on-glass interface test (C.L.A = 2.2 μm)
This study	71	45	53	250-450	10.5°-11°	Canons Park, smectite consisted approximately 50% of the clay fraction

Considering the values of ^{the} drained residual friction angle of London clay from Wraysbury and Canons Park, it seems that either the mineralogical analysis of Canons Park London clay overestimates the smectite content of the clay or that the pore water chemistry of the soils might have been different.

Tests 40,41,42

Interface: Stainless steel

Initial roughness :C.L.A = $8.40\mu\text{m}$

The choice of the roughness of the interfaces used in the tests was based on roughness measurements of pile surfaces, found in literature and carried-out during this study. They are summarized in Table A2.36. The indirect roughness measurements reported in Table A2.36 consisted in measuring the roughness of replicas of pile surfaces (the replicas were made by means of a Taylor-Hobson replica kit).

The data, shown in Table A2.36, indicate that the roughness, chosen for the series of tests, is representative of a wide range of measurements. It matches also the roughness used in a current research programme on the behaviour of displacement piles in overconsolidated clays (Bond, 1989) with which the test results were to be compared.

Test procedure

Outlined below is a general description of the procedure followed in the tests. For more specific details, reference should be made to the section dealing with the individual test.

The soil was placed in the apparatus at a water content of 60.9%, 64.9% and 56.5% for Tests 40, 41 and 42 respectively. It was then consolidated to a normal stress of 967 kPa and swelled to 484 kPa. The decrease of the water content during the consolidation stage was 30%, 32% and 29.5% for Tests 40, 41 and 42 respectively.

Shearing involved a series of successive fast stages (pulses), carried out for a displacements of half to one meter, at a constant rate of displacement and with a pause of about 10 min allowed between them, to simulate pile driving operation. After the completion of the series of the successive fast stages and a pause for consolidation, slow drained probes were carried out in order to further investigate the nature of the rate effects. The rates of displacements which were used to study the rate effects in Tests 40, 41 and 42 were 110, 1100 and 2600 mm/min respectively. In Test 42, after the completion of the series of the successive fast stages, the effect of fast shearing on the residual soil-steel shear resistance was further investigated.

Test 40

Test results

The complete sequence of shearing can be followed in Fig. A2.46a and A2.46b. Each shearing stage is summarized in Table A2.37.

Shearing was commenced with the series of successive fast stages 1 to 5, carried out at 110 mm/min, and with a pause of 10 min between them, simulating pile driving operation. At the end of stage 5 and after a pause of 5 hr, the nominal normal stress on the sample was reduced to 352.4 kPa to simulate the reduction of radial stresses around a pile after installation. Stage 6 followed at 0.00328 mm/min, representing pile movement due to loading. At the end of stage 6, the nominal normal stress was increased back to 484 kPa. The test terminated with stage 7, involving slow drained shearing to the residual state at 0.0053 mm/min.

The final water content of the sample was not measured.

Description of the test specimen

The sample when pulled away from the interface tended to separate through a well defined shear surface, formed within the soil, leaving a thin layer (approximately 0.1 mm) attached to the interface. The layer in touch with the interface was smooth, whereas the shear surface had heavy striations. Quartz particles intruded the shear surface (i.e. compare with the smooth and with the light striations shear surface observed in the soil-on-soil Tests 39 and 40).

Photographs 41 to 45 in Plate 6 illustrate scanning electron-micrographs from the shear surface and the layer of soil attached to the interface.

Final roughness : C.L.A = 7.40 μm

Test 41

Test results

The complete sequence of shearing can be followed in Fig. A2.47a and A2. 47b. Each shearing stage is summarized in Table A2.38.

Shearing involved initially the series of fast stages 1 to 5, carried out at 1100 mm/min and with a pause of 10 min between them. After a pause of 7.5 hr at the end of stage 5, fast shearing resumed again in

stage 6. The test terminated with a slow drained probe (stage 7), carried-out at 0.00484 mm/min in order to investigate the state of stress ratio at the end of fast stage 6.

The final average water content of the sample was 31%.

Description of the test specimen

The same observations, as in Test 40, were made. The striations on the principal shear surface were not as heavy as in Test 40, probably due to the fact that the slow residual strength had not been fully established in the last stage of the test.

Final roughness : C.L.A = 8.30 μ m

Test 42

Test results

The complete sequence of shearing can be followed in Fig. A2.48a, A2.48b and A2.48c. Results from the test are presented in Table A2.39.

Shearing was commenced with the series of successive fast stages 1 to 5, carried out at 2600 mm/min. After a pause of 6 days, the series of stages 6 to 9 was performed at the same rate of displacement. A short pause of 10 min was allowed after each stage in the two series of fast shearing. The state of the soil-interface shear resistance^{at} the end of fast stage 9 was checked in the slow stage 10, carried out at 0.00487 mm/min. Stage 11 involved fast shearing at 2600 mm/min. Slow shearing to the residual state resumed in stage 12 at 0.00487 mm/min in order to investigate the changes of the structure of the soil induced by the preceding fast stage.

The remaining stages of the test, 13 to 20, involved shearing at alternating fast and slow rates of displacement in order to investigate the effect of fast shearing on the residual soil-interface strength. A displacement of about 1m, was allowed, during each fast stage. After the completion of stage 20, the sample was removed from the apparatus.

The final average water content of the sample was 19.8%

Description of the test specimen

The same observations, as in Tests 40 and 41, were made. The striations on the shear surface were as heavy as in Test 40.

Final roughness : C.L.A = 9.10

A2.7 SANDSTest 43, medium sand

Operator : Tika, T.M

Soil description

The soil used was a medium to fine, rounded, uniformly graded, silica sand (Fig A2.48).

Test results

The sand was mixed with distilled water twenty four hours before testing in the ring shear apparatus. It was then placed in the apparatus in a dense state (bulk unit weight about 20 kN/m³) and consolidated to a normal stress of 264.6 kPa. A rate of displacement of 0.0142 mm/min was used as the slow standard throughout the test, so as to ensure 95% dissipation of the excess pore water pressure during shearing in a displacement of 0.01 mm.

Shearing of the soil involved four stages at alternating slow and fast rates of displacement. Fig.A2.50 shows the stress ratio plotted against displacement and Table A2.40 summarizes the test results.

No particle degradation took place during shearing, as it was indicated by a grading curve of the soil determined at the end of the ring shear test.

Tests 44,45,46,47, fine sand

Operator : Tika, T.M

Soil description

The soil, used for the series of the tests, was a fine to medium,

subrounded, uniformly graded, silica sand (SiO_2 not less than 97%, Fig A2.48).

Sample preparation and consolidation stage

The sand was mixed with distilled water twenty four hours before testing in the ring shear apparatus. It was placed in the apparatus at a relatively dense state (bulk unit weight : 18-19 kN/m^3) and consolidated in increments to the level of the operating nominal normal stress in the particular test. In all tests the consolidation of the samples was almost immediate (consolidation time less than 1 min).

Test 44

Test results

The test was performed at a nominal normal stress of 484 kPa. The slow standard rate of displacement, used throughout the test was 0.0917 mm/min. The complete sequence of shearing can be followed in Fig. A2.51a. Each shearing stage is summarized in Table A2.41.

Stage 1 involved slow shearing through the peak to the residual state. The remaining shearing stages of the test, 2 to 11, involved shearing at alternating fast and slow rates of displacement. During the fast stages 2 and 8 the rotational potentiometer was used for the measurement of the displacement during fast shearing. Fig. A2.51b shows the stress ratio against displacement during fast shearing, as measured by the rotational potentiometer and as computed assuming a constant average velocity.

Possible particle degradation was checked by determining the particle size distribution of the sample at the end of the test. It was identical with the initially determined one. Particle degradation was assumed to be negligible also for the following tests 45 to 47, carried-out at lower normal stresses.

Test 45

Test results

The test was carried out at a nominal normal stress of 264.6 kPa. A rate of 0.0940 mm/min was chosen as the slow standard one. The stress ratio versus cumulative displacement is plotted in Fig. A2.52 and the results from the test are summarized in Table A2.42.

Stage 1 involved slow shearing through the peak to the residual state. Stage 2 followed at 100 mm/min. Slow re-shearing to the residual conditions was performed during stage 3. The slow standard rate of displacement was further reduced to 0.0112 mm/min in stage 4. Stage 5, followed at 360 mm/min. The remaining shearing stages of the test were carried out at alternating slow and fast rates of displacement. No readings were recorded during the slow stage 6 due to the failure of the recording system (Orion). After stage 11, the sample was removed from the apparatus.

Test 46

Test results

The test was performed at a nominal normal stress of 132.9 kPa. The slow standard rate of displacement, used throughout the test, was 0.0926 mm/min. The complete sequence of shearing can be followed in Fig. A2.53. Test results are presented in Table A2.43.

Shearing was commenced at 0.0117 mm/min through the peak to the residual strength in stage 1 and continued at the slow standard rate of displacement during stage 2. The following shearing stages of the test were carried-out at alternating fast and slow rates of displacement. The test had to be terminated prematurely because of the excessive loss of soil by squeezing through the gap between the confining rings during fast shearing, following stage 6 (results from this fast stage are not presented).

Test 47

Test results

The test was carried out at two nominal normal stresses, 440.1 kPa initially (stages 1 to 3) and 132.9 kPa afterwards. A rate of 0.0935 mm/min was used as the slow standard throughout the test. The complete sequence of shearing can be followed in Fig. A2.54. Table A2.44 summarizes the results from the test.

Stage 1 involved slow shearing through the peak to the residual strength and was followed by fast stage 2, carried-out at 360 mm/min. Slow residual conditions were established again during stage 3. At the end of stage 3, the sample was swelled to 132.9 kPa.

Slow shearing was commenced again during stage 4. Stage 5 followed at 1200 mm/min. The shear loads were not recorded during this stage (their traces were outside the range of the U/V recording paper), and stage 6 was carried-out in order to establish again the slow residual strength of the sand. The remaining stages of the test, 7 to 11, involved shearing at alternating fast and slow rates of displacement.

A2.8 TESTS TO INVESTIGATE THE LOSS OF STRENGTH DURING FAST SHEARING

Test 48, *clayey siltstone D*

Operator: Tika, T.M

Interface: Glass

Initial roughness: C.L.A = $7.30\mu\text{m}$

This test on the clayey siltstone D was performed in order to investigate the role of pore water pressure during fast shearing. The last stage of the test was prolonged for a period equal to the consolidation time of the soil.

Test results

The soil was placed in the apparatus at a water content of 32.9%. It was then consolidated to a normal stress of 1000 kPa and swelled to 527.9 kPa. The decrease of the water content during the consolidation stage was approximately 10.3%. A rate of displacement of 0.01 mm/min was used as the slow standard throughout the test.

The stress ratio versus cumulative displacement is shown in Fig. A2.55 and the results from the test are summarized in Table A2.45

Stage 1 involved slow shearing through the peak to the residual strength. The increase of the residual stress ratio with displacement was due to the friction developed in the gap. At the end of stage 1, the nominal normal stress was reduced to 264.6 kPa.

Slow shearing was resumed in stage 2. Stage 3 was carried out at 300 mm/min in order to assess the soil behaviour prior to prolonged

shearing. The settlements recorded at the end of the stage indicated a consolidation time of 45 min (coefficient of consolidation : $1.50 \text{ m}^2/\text{yr}$). The residual strength was established again during stage 4. The fast shearing was prolonged for 51 min in stage 5. After stage 5, the sample was removed from the apparatus.

The average water content of the sample was 23.4%.

Description of the test specimen

The sample separated through a well-defined shear surface, formed within the sample and away from the soil-interface boundary at a distance of approximately 0.2 mm. The shear surface was striated circumferentially. Coarse grains intruded the shear surface.

Final roughness: C.L.A = $7.0 \mu\text{m}$

Test 19, clayey siltstone D

Operator: Tika, T.M

This test on the clayey siltstone D was performed on a sample from the original supplied soil, but with the clay fraction reduced. It was intended to investigate whether a reduction of the clay fraction would change the rate independent-behaviour of the soil to rate-dependent one. The clay fraction of the soil was reduced by sedimenting it and discarding the fines in suspension. The role of the pore water pressure during fast shearing was also investigated by performing successive fast stages at increasing and decreasing rates of displacement with a short pause allowed between them.

Index properties and grading

The index properties of the soil were as follows:

LL=41.5, PL=20.0, PI=21.5%

A particle size distribution of the soil is shown in Fig. A2.1.

Test results

The soil was placed in the apparatus at a water content of 25.5%. It was then consolidated to a normal stress of 966.8 kPa and swelled to 264.6 kPa. The decrease of the water content during the consolidation

stage was about 10%. From the consolidation stage, it was estimated that a rate of displacement of 0.024 mm/min would ensure 95% dissipation of the pore water pressure in a displacement of 1.00 mm.

Each shearing stage is summarized in Table A2.46. The stress ratio is plotted against displacement in Fig.A2.56a and A2.56b, for stages 1 to 22, and against cumulative displacement in Fig.A2.56c and A2.56d, for the remaining stages of the test.

Stage 1 involved slow shearing through the peak to the residual strength. Stages 2 and 3 were both carried out at 133 mm/min. Readings were not taken during stage 2, because of the failure of the recording system. Stages 4 to 22 involved shearing at rates of displacement varying from 0.0014 to 2.3816 mm/min. No significant rate effect on the residual strength was observed. Stage 23 followed at 1300 mm/min. No recording was performed during this stage.

After a pause for consolidation, stages 24 to 37 were performed at increasing rates of displacement from 900 to 6000 mm/min and a pause of 2.5 min allowed between them. The duration of the series of the fast stages was 33.79 min and the degree of consolidation from its start to its end was about 95%. The fast minimum strength, however, showed no indication of increasing. The pattern of the behaviour observed was that of an initial increase of strength after the 2.5 min pause, followed by a further decrease of strength up to a rate of displacement of 3000 mm/min. Thereafter, the minimum strength seemed to be insensitive to the change of the rate of displacement from 3000 to 6000 mm/min.

After a pause, another series of fast stages was carried out with a pause of 2.5 min allowed between them. Stages 38 and 39 were carried out at 3700 and 2000 mm/min respectively, followed by stages 40 to 54 at decreasing rates of displacement from 5400 to 1200 mm/min. The duration of the series of the fast stages was 41.06 min and the degree of consolidation from its start to its end was about 95%. The test was terminated with the stages 55 and 56, carried out at 0.0409 and 0.1573 mm/min respectively.

The final water content of the sample was 16.3%.

Description of the test specimen

The sample separated through a non-planar, rugged shear surface. A shear zone which had a darker colour from the rest of the sample was present.

Test 50, clayey siltstone D

Operator: Tika, T.M

Interface: Stainless steel

Initial roughness: C.L.A = $8.3\mu\text{m}$

Test results

The soil was placed in the apparatus at a water content of 30%. It was then consolidated to a normal stress of 966.8 kPa and swelled to 264.6 kPa. The decrease of the water content of the sample due to the compression was about 10%. From the consolidation stage, it was calculated that the slow standard rate of displacement, 0.015 mm/min, would ensure 95% dissipation of the pore water pressure in less than 1 mm of shear displacement.

Results from the test are summarized in Table A2.47. The shearing stages of the test can be followed in Fig. A2.56.

Stage 1 involved slow shearing through the peak to the residual strength and was followed by stages 2 to 7, carried out at alternating fast and slow rates of displacement. At the end of stage 7, the nominal normal stress was increased to 527.9 kPa. Slow shearing was commenced again in stage 8. The remaining stages of the test involved alternating fast and slow shearing.

The final water content of the sample was as follows:

shear zone : 19.6%

Outer thirds: 18.8%

Description of the test specimen

The same observations as for the test specimen of Test 48 were made.

Final roughness : C.L.A = $8.1\mu\text{m}$

Test 51, clayey siltstone D**Operator**: Tika, T.M**Interface**: Glass**Initial roughness**: C.L.A = 7.4 μ m

This test on the clayey siltstone D involved soil shearing against an instrumented glass interface for the measurement of the pore water pressure and temperature changes during fast shearing.

Test results

The soil was placed in the apparatus at a water content of 29.4%. It was then consolidated to a normal stress of 922.9 kPa and swelled to 154.8 kPa. The decrease of the water content due to the compression was about 10.8%. From the consolidation stage, it was estimated that the slow standard rate of displacement of 0.015 mm/min, used throughout the test, would ensure 95% dissipation of the pore water pressure within 0.6 mm of shear displacement.

Each shearing stage is summarized in Table A2.48. The results from the test are plotted in Fig.A2.57a and A2.57b.

Stage 1 involved slow shearing through the peak to the residual state. Stage 2 followed at 380 mm/min. After a rest for consolidation, the nominal normal stress was increased to 264.6 kPa.

Slow shearing was commenced again in stage 3. Stage 4 was carried out at 370 mm/min. After a pause of 7.5 min (required to change the motor connected to the worm drive of the apparatus), shearing was resumed at 0.6123 mm/min in stage 5 in order to monitor ^{the} mobilization of the strength, immediately after the fast shearing. Slow standard residual conditions were established again in stage 6. Stage 7 followed at 0.6295 mm/min. At the end of stage 7, the nominal normal stress was increased to 440.1 kPa.

The slow residual strength was established in stage 8. The test terminated with stage 9, carried-out at 370 mm/min.

The final water content of the sample was not measured.

Description of the test specimen

The same observations as for the test specimens of Tests 48 and 50 were made.

Final roughness: C.L.A = $7.0\mu\text{m}$

Test 52, claystone D

Operator: Tika, T.M

Interface: Glass

Initial roughness: C.L.A = $7.4\mu\text{m}$

The results from a test on the claystone D, involving soil shearing against an instrumented glass interface for the measurement of the pore water pressure and temperature changes during fast shearing, are considered here.

Test results

The soil was placed in the apparatus at a water content of 41.8%. It was then consolidated to a normal stress of 922.9 kPa and swelled to 264.6 kPa. The decrease of the water content due to the compression was about 20%. From the consolidation stage, a rate of displacement of 0.0141 mm/min was chosen as the slow standard, so as to ensure 95% dissipation of pore water pressure in 0.6 mm of shear displacement.

The results from the test are summarized in Table A2.49 and stress ratio versus cumulative displacement is plotted in Fig.A2.58a and A2.58b.

Shearing was commenced at the slow standard rate of displacement in stage 1. Stage 2 to 6 involved shearing at alternating fast (400 mm/min) and slow rates of displacement. Stage 7 followed immediately after fast stage 6 at 1.5379 mm/min (pause of 6 min was allowed for the change of the motor engaged to the worm drive of the apparatus). Slow standard shearing was restarted in stage 8. At the end of the stage 8, the nominal normal stress was increased to 527.9 kPa.

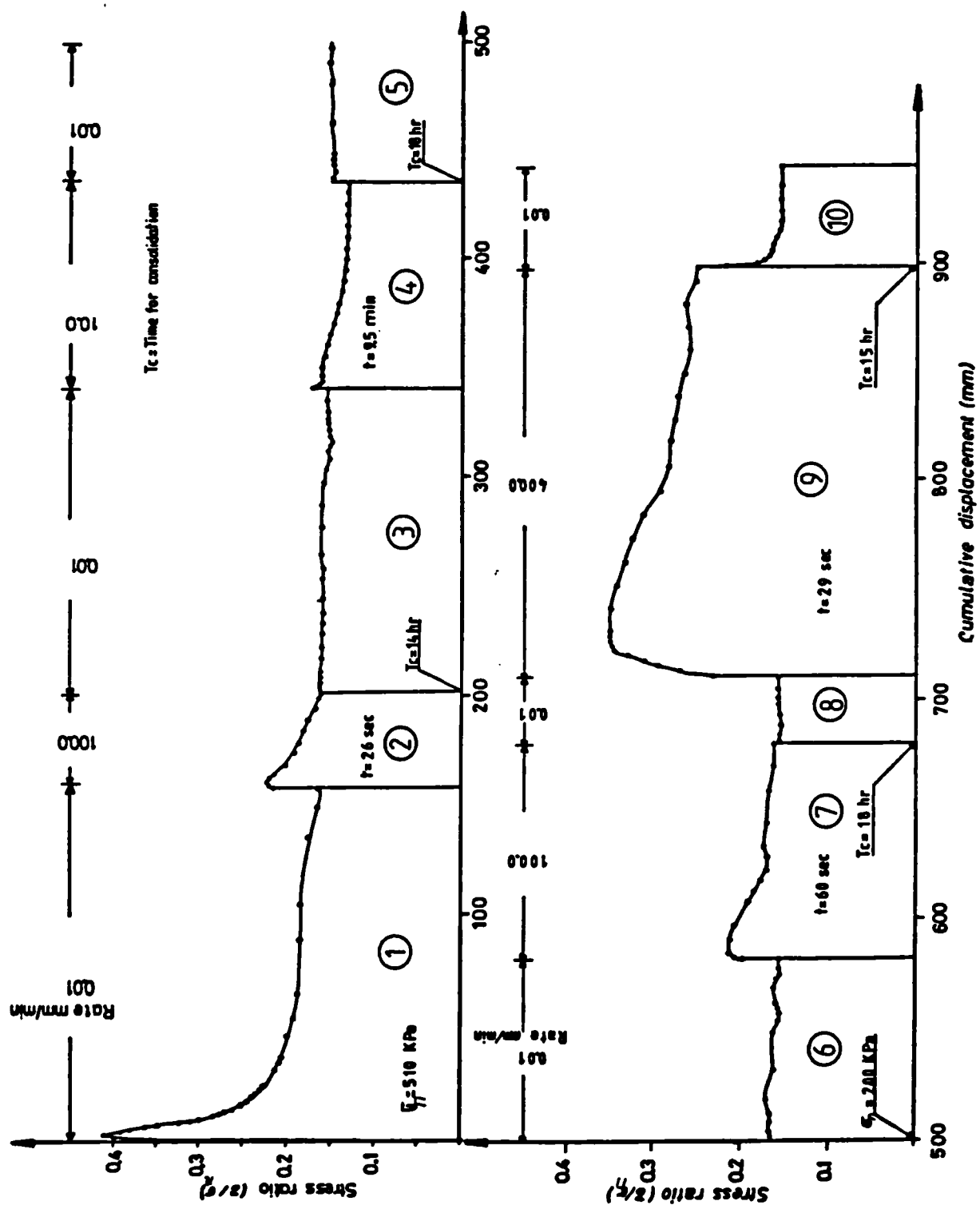


Fig. A2.2a Ring shear test 1 on claystone A

Table A2.1 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
11	400	1081.0	211.0	*		
12	0.01	1082.0	210.0	e:0.230 0.230	1.0	
13	400	1151.0	210.0	b:0.315 c:0.320 0.260	0.0 2.0 69.0	
14	0.003	1153.0	210.0	e:0.225 0.160	13.0	
15	10	1265.0	211.0	b:0.172 c:0.170 d:0.140	0.0 0.0 99.0	

b: threshold, c: fast peak, d: fast residual, e: fast peak after fast shearing

* No recording was performed

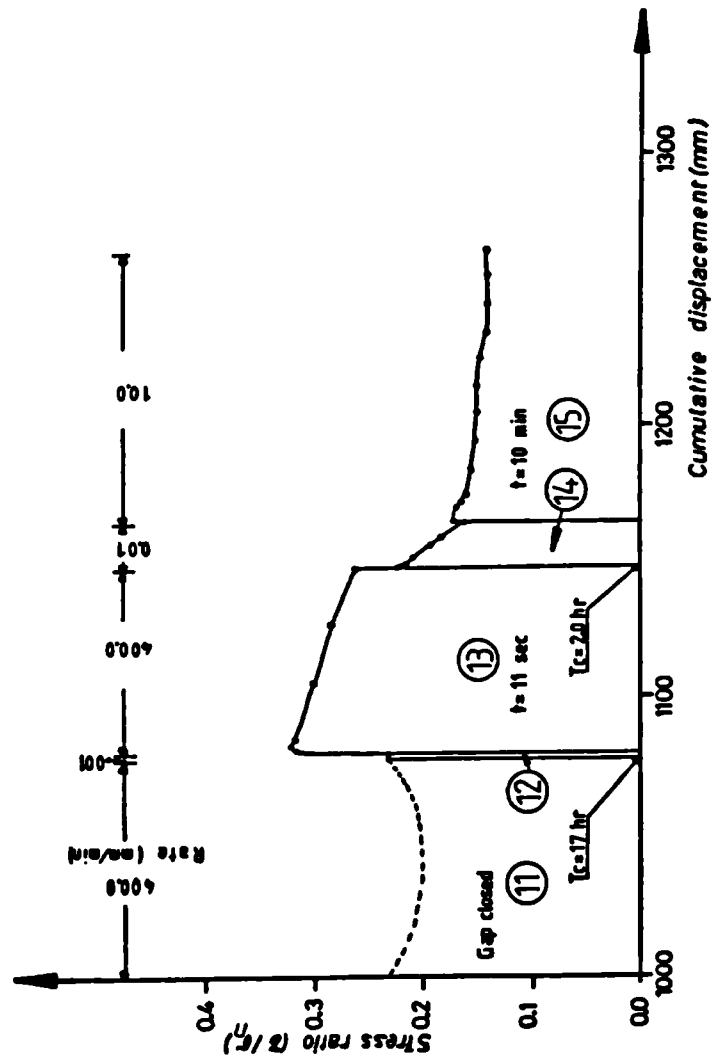


Fig. A2.2b Ring shear test 1 on claystone A

Table A2.2 Results from ring shear test 2 on claystone A

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	400	197.0	201.0	b,c:1.023 d:0.210	0.0 197.0	
2	0.01	301.0	198.0	e:0.335	1.0	
3	0.06	364.0	541.0	0.170 0.200	104.0	
4	400	506.0	546.0	0.165 *	20.0	
5	0.014	516.0	553.0	e:0.210 0.165	10.0	
6	400	709.0	553.0	b,c:0.250 d:0.190	0.0 193.0	

b: threshold, c: fast peak, d: fast residual, e: fast peak after fast shearing
* No recording was performed

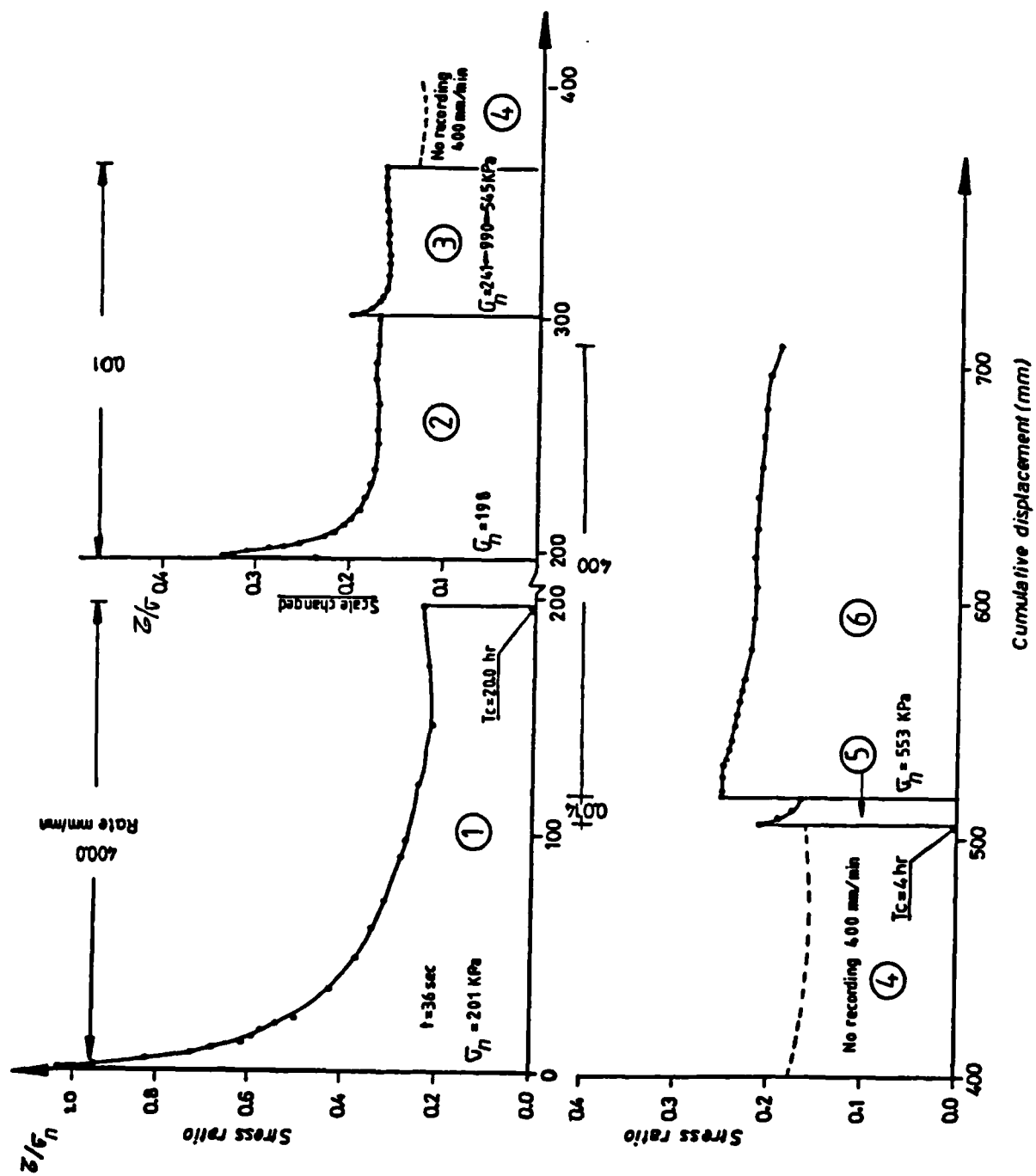


Fig. A2.3 Ring shear test 2 on claystone A

Table A2.3 Results from ring shear test 3 on clayey siltstone A

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.01	240.0	500.0	peak:0.540 0.440 0.360	8.0 40.0 240.0	
2	100	340.0	495.0	b:0.420 c:0.430 d:0.320	0.0 0.0-10.0 60.0	
3	0.01	424.0	497.0	e:0.400 0.350		
4	400	605.0	493.0	b,c:0.440 d:0.170	0.0-10.0 100.0	
5	0.01	718.0	495.0	e:0.380 0.330		
6	10	830.0	498.0	b:0.360 c:0.370 d:0.370	0.0	
7	0.01	837.0	496.0	e:0.344 0.330		

b: threshold, c: fast peak, d: fast residual, e: fast peak after fast shearing

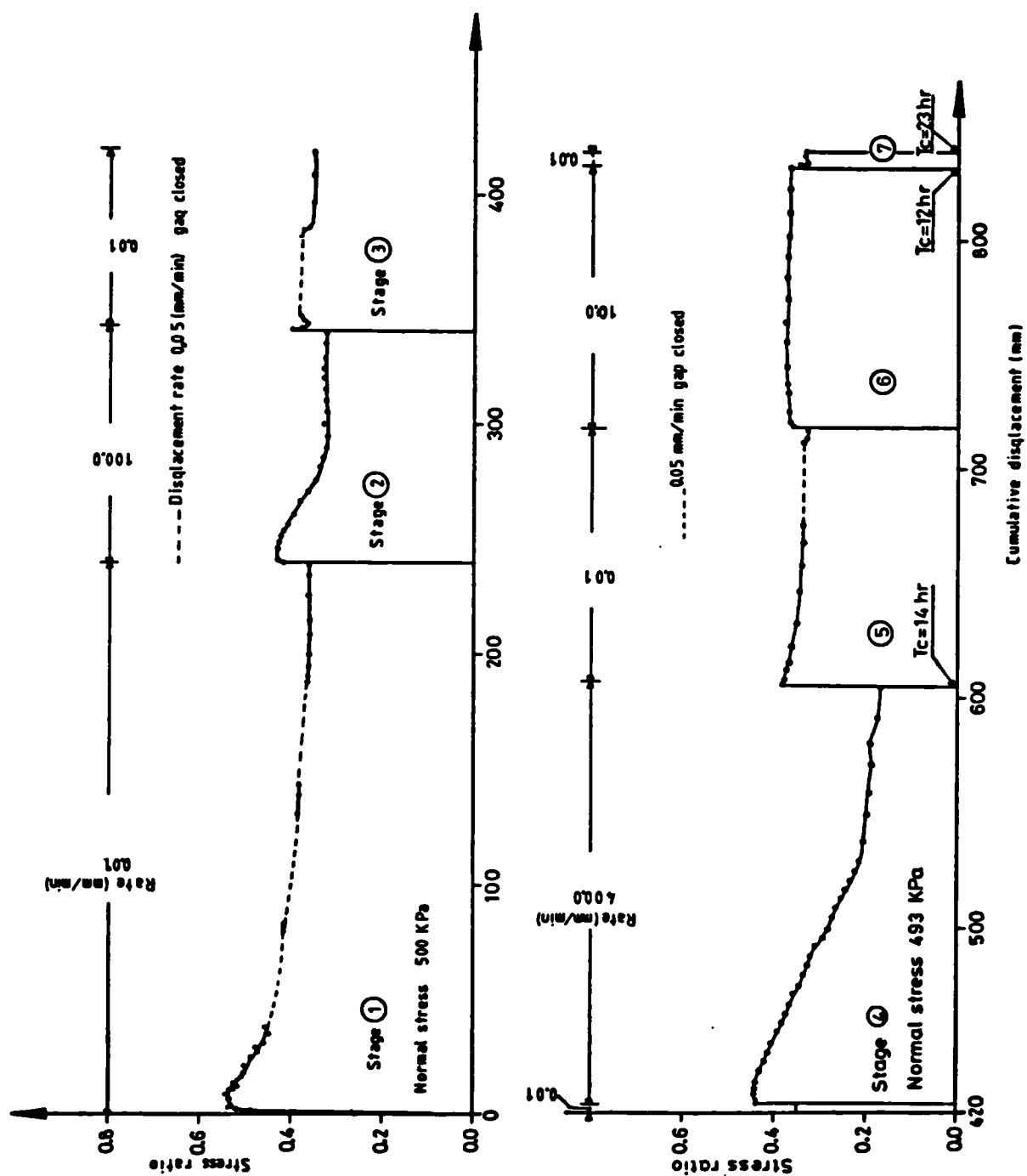


Fig. A2.4a Ring shear test 3 on clayey siltstone A

Table A2.3 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
8	0.01	998.0	195.0	peak:0.474 0.370	1.0 8.0	
9	400	1204.0	201.0	b:0.500 c:0.510 d:0.160	0.0 170.0	
10	0.01	1264.0	202.0	e:0.413 0.375		
11	100	1390.0	202.0	b,c:0.470 d:0.270	0.0 100.0	
12	0.01	1424.0	206.0	e:0.410 0.380		
13	10	1544.0	205.0	b,c:0.435 d:0.430	0.0	
14	0.01	1640.0	218.0	e:0.400 0.340		

b: threshold, c: fast peak, d: fast residual, e: fast peak after fast shearing

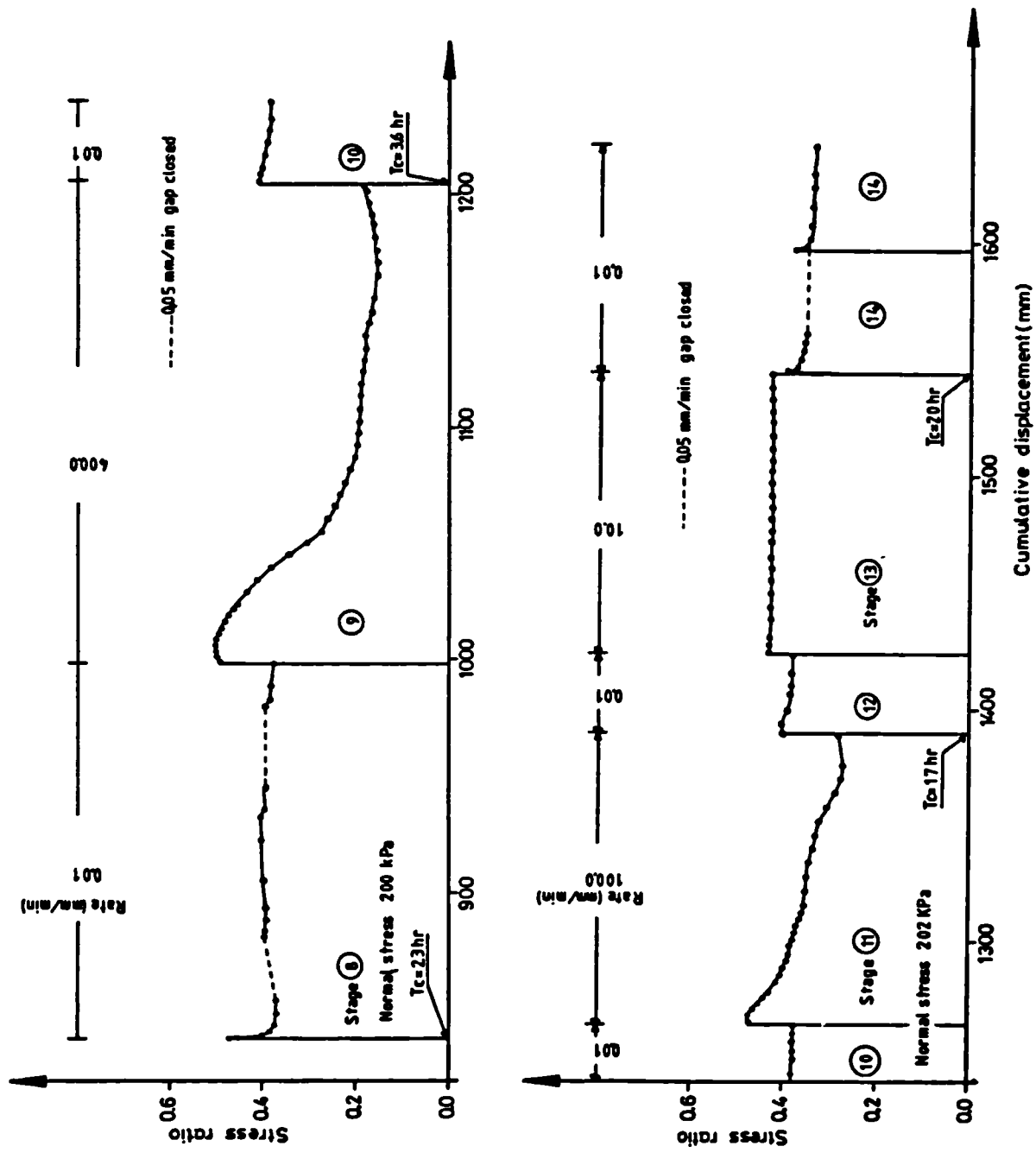


Fig. A2.4b Ring shear test 3 on clayey siltstone A

Table A2.3 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
15	700	1822.0	218.0	b,c:0.500 d:0.150	0.0 80.0	
16	0.01	1852.0	222.0	e:0.435 0.400		
17	400			No recording was performed		

b: threshold, c: fast peak, d: fast residual, e: fast peak after fast shearing

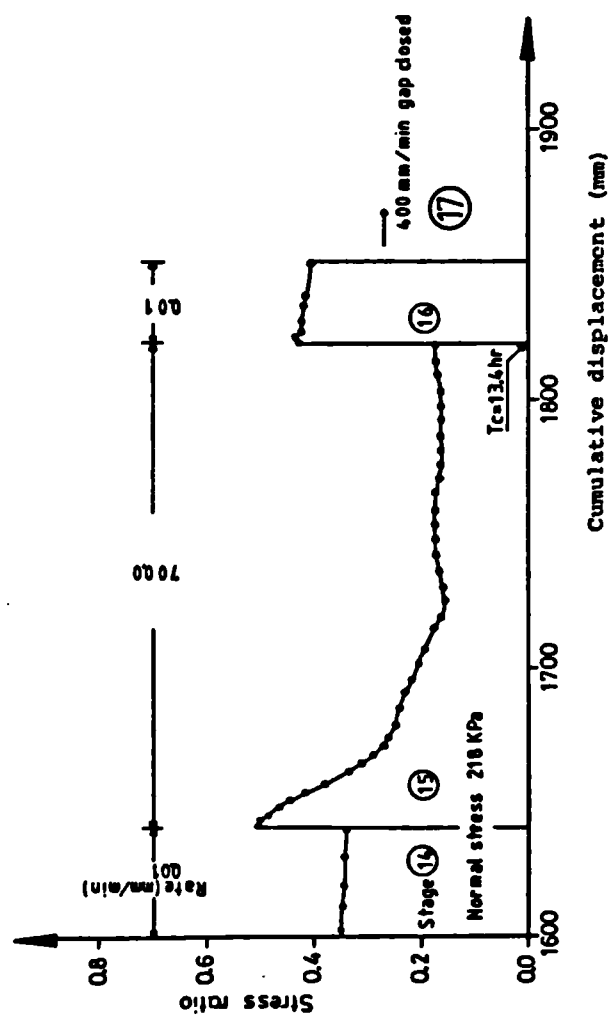


Fig. A2.4c Ring shear test 3 on clayey siltstone A

Table A2.4 Results from ring shear test 4 on clayey siltstone A

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0034	428.0	440.0	peak:0.560	4.0	
	0.0034			0.540	2.9	
	0.015			0.430	80.0	
	0.015			0.360	428.0	
2	400	906.0	430.0	b:0.455	0.0	
				c:0.467	2.0	
				d:0.100	370.0	
				0.150	478.0	

b: threshold, c: fast peak, d: fast residual

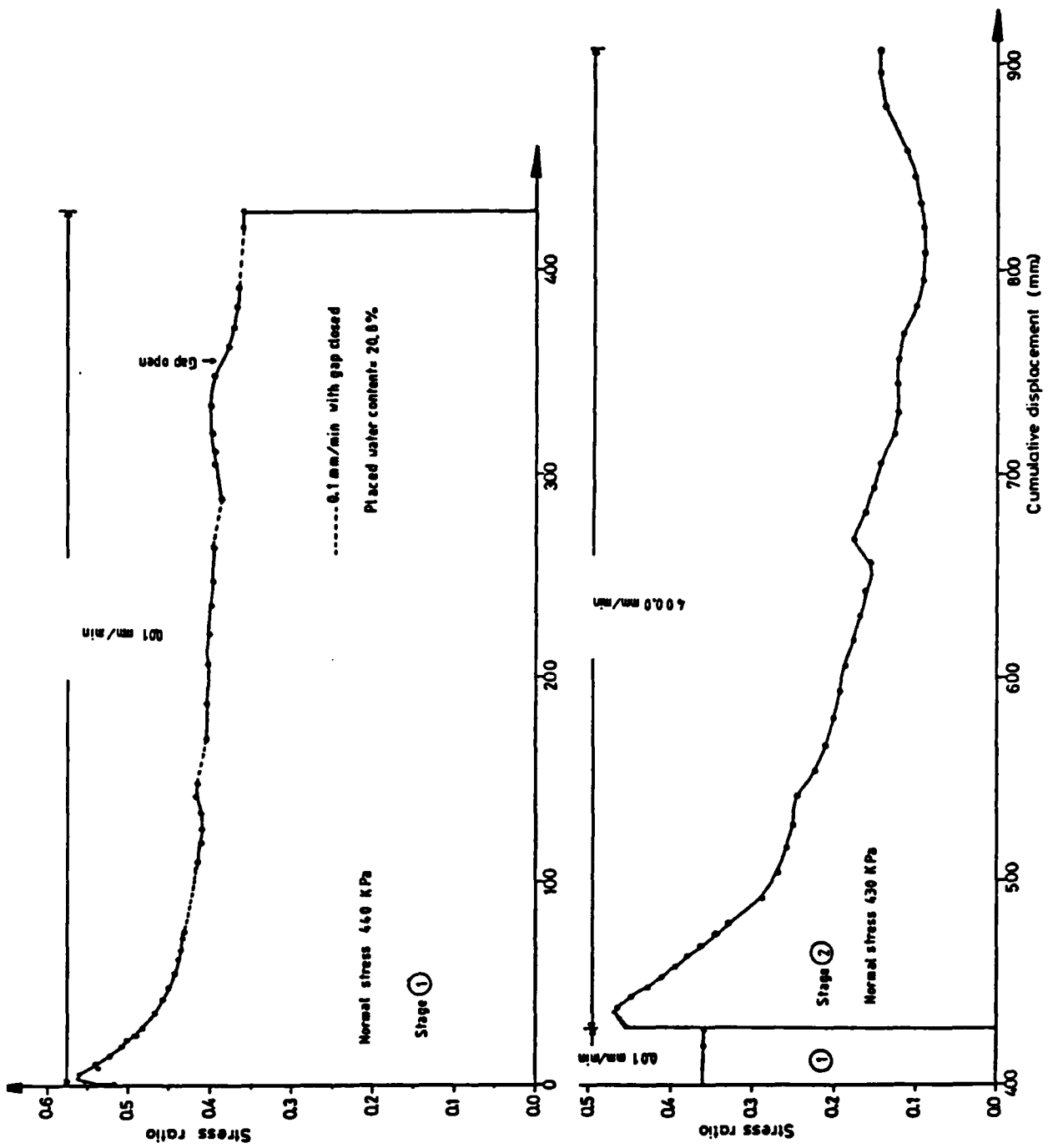


Fig. A2.5 Ring shear test 4 on clayey siltstone A

Table A2.5 Results from ring shear test 5 on clayey siltstone A

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
1	400	498.0	189.0	b,c:1.200 0.100 d:0.090	0.0 200.0 498.0	75
2	400	628.0	189.0	b,c:0.340 d:0.100	0.0 100.0	19.8
3	400	830.0	195.2	b,c:0.400 d:0.120	0.0 120.0	30.6
4	400	1056.0	196.0	b,c:0.400 d:0.150	0.0 120.0	34.2

b: threshold, c: fast peak, d: fast residual

* duration of the particular stage in sec

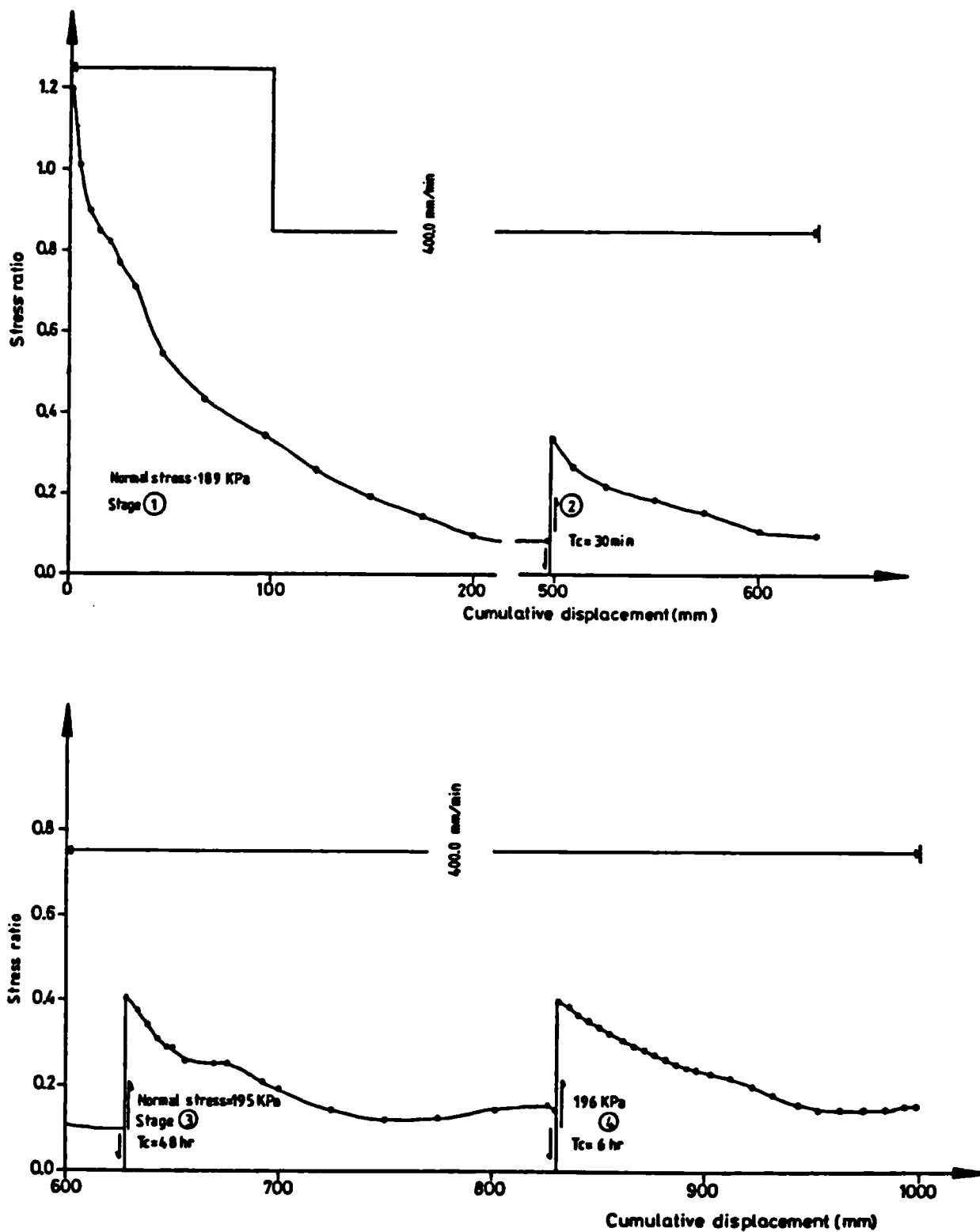


Fig. A2.6a Ring shear test 5 on clayey siltstone A

Table A2.5 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
5	400	1122.0	196.3	b,c:0.980 0.155	0.0 66.0	10.2
6	400	1180	196.0	b,c:0.380 0.290	0.0	9.0
7	133	1190.0	201.0	b,c:0.430 0.380	0.0 2.0	4.8
	133	1199.0	201.0	b,c:0.380 0.365	0.0 90.0	4.2
	133	1210.0	201.0	b,c:0.370 0.350	0.0 11.0	4.8
	133	1221.0	201.0	b,c:0.360 0.350	0.0 11.0	4.8
	133	1226.0	201.0	b,c:0.350 0.350	0.0 5.0	2.4
8	133	1249.0	201.0	b,c:0.480 0.430	0.0 230.0	10.2
		1284.0	201.0	b,c:0.430 0.370	0.0 35.0	34.8
		1288.0	201.0	b,c:0.370 0.360	0.0 4.0	1.8
		1310.0	201.0	b,c:0.390 0.390	0.0 22.0	3.6
		1385.0	201.0	b,c:0.400 0.345	0.0 75.0	33.6

b: threshold, c: fast peak

* duration of particular stage or pulse in sec

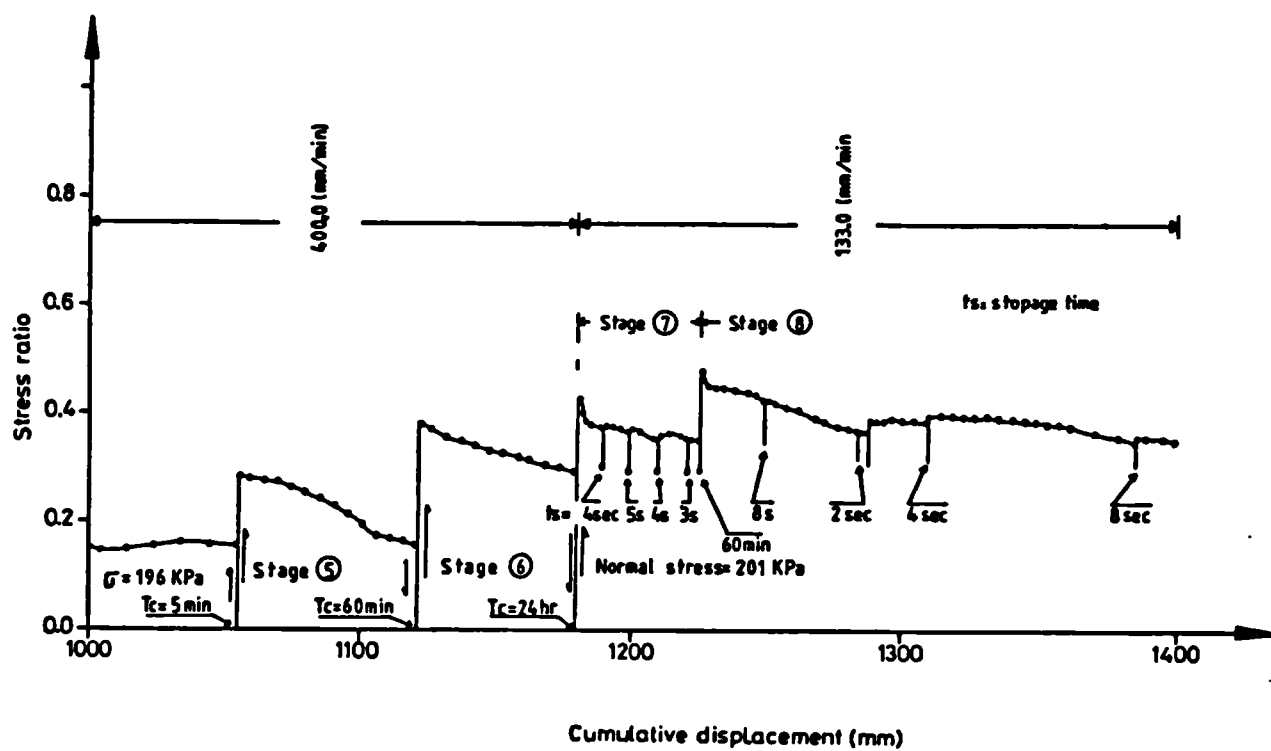


Fig. A2.6b Ring shear test 5 on clayey siltstone A

Table A2.5 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
8	133	1434.0	201.0	b,c:0.355	0.0	22.2
				0.330	49.0	
		1628.0	201.0	b,c:0.350	0.0	87.6
				d:0.310		
				0.340	194.0	5.4
		1640.0	201.0	b,c:0.340	0.0	
				0.340	12.0	3.6
		1648.0	201.0	b,c:0.345	0.0	
				0.345	8.0	3.6
		1656.0	201.0	b:0.345	0.0	
				c:0.350		2.7
				0.350	8.0	
		1662.0	201.0	b:0.350	0.0	2.7
				c:0.352		
				0.352	6.0	2.7
		1668.0	201.0	b,c:0.355	0.0	
				0.355	8.0	2.7
9	0.0022	1670.0	210.0	e:0.450		
				0.430	2.0	
10	133	1752.0	210.0	b,c:0.532		
				0.395	82.0	37.2

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
 * duration of particula pulse or load in sec

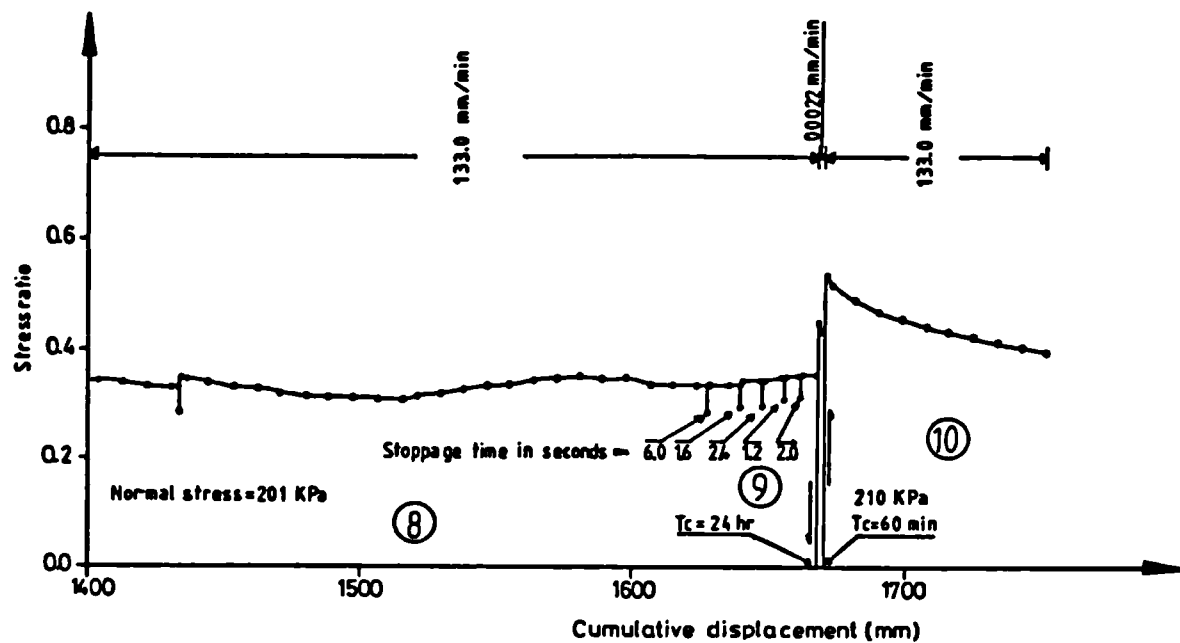


Fig. A2.6c Ring shear test 5 on clayey siltstone A

Table A2.6 Results from ring shear test 6 on clayey siltstone A

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0077	10.0	503.0	peak:0.565	3.0	
				0.490	13.0	
	0.051	324.0		0.250	324.0	
	0.01	336.0		0.255	336.0	
2	400	626.0	542.0	b:0.370	0.0	
				c:0.385	4.0	
				0.170	80.0	
				d:0.105	290.0	
3	0.01	674.0	518.0	e:0.340	352.9	
				0.265	20.0	
4	400	994.0	519.0	b:0.415	0.0	
				c:0.430		
				d:0.100	320.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

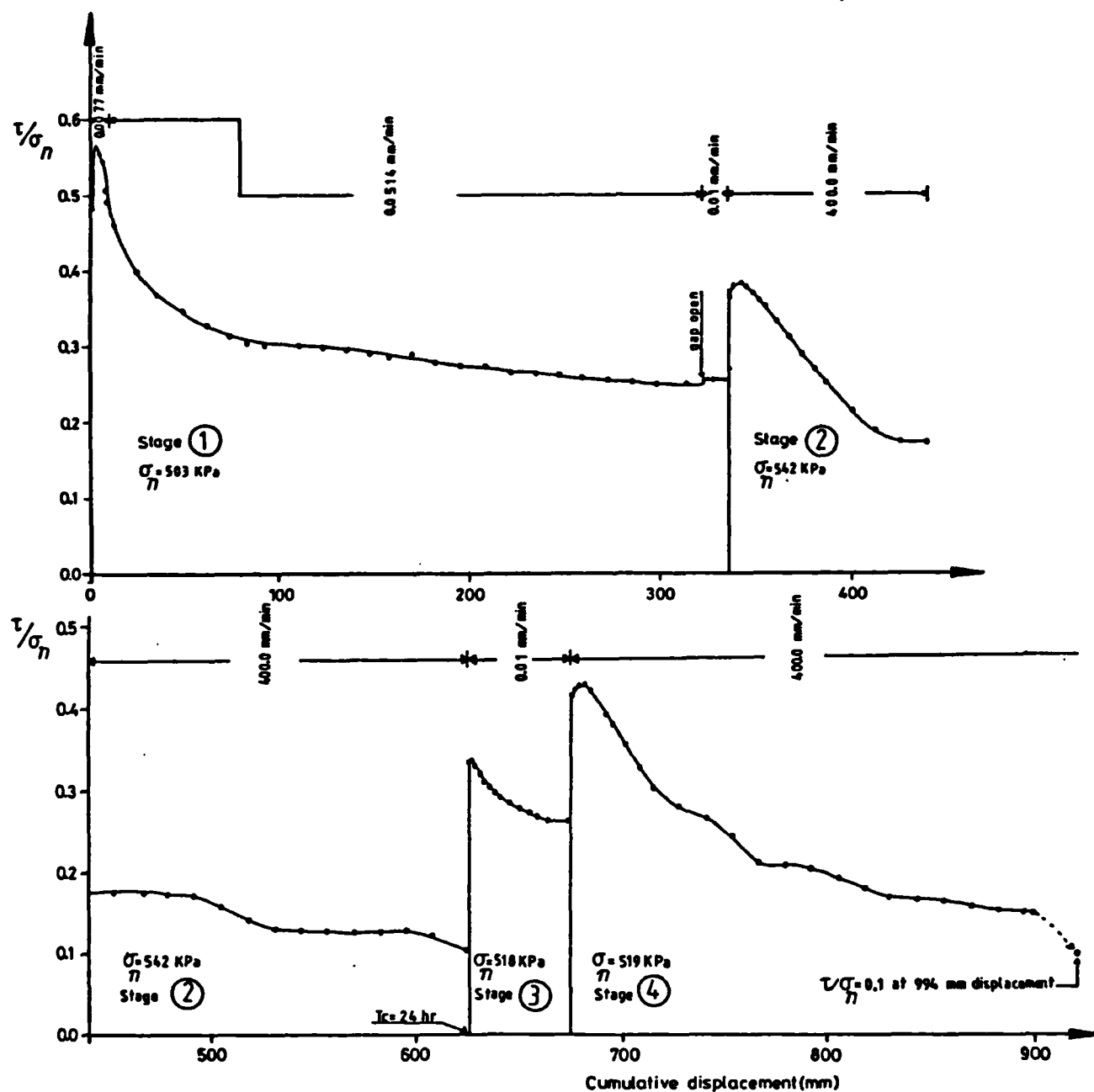


Fig. A2.7 Ring shear test 6 on clayey siltstone A

Table A2.7 Results from ring shear test 7 on clayey siltstone A

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
1	0.004	2.4	483.0	peak:0.488 0.487	2.4	
2	404	503.2	487.0	b,c:0.737 0.710 0.340 No further recording	0.89 16.76 164.0	
3	0.36	536.3	487.0	0.167 0.190 0.260 0.330	0.4 0.8 6.8 33.1	20 min 92 min
4	0.01	615.1	486.0	e:0.344 0.288	0.5 78.8	
5a	400	813.8	504.3	b:0.420 c:0.440 0.159	0.0 4.0 130.0	
5b	400	911.5	504.0	b:0.160 c:0.166 0.09	0.0 97.7	
5c	400	1031.0	504.0	b:0.107 c:0.108 0.115	0.0 119.5	
5d	400	1647.0	504.0	b,c:0.125 d:0.080	0.0 65.0	
6	0.35	1167.0	504.0	0.094 0.145 0.210 0.262 0.277	0.0 1.0 5.0 13.0 20.0	58.1 min
7	0.006	1174.0	504.0	e:0.325 0.300	1.0 7.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
 * elapsed time from the start of the particular stage

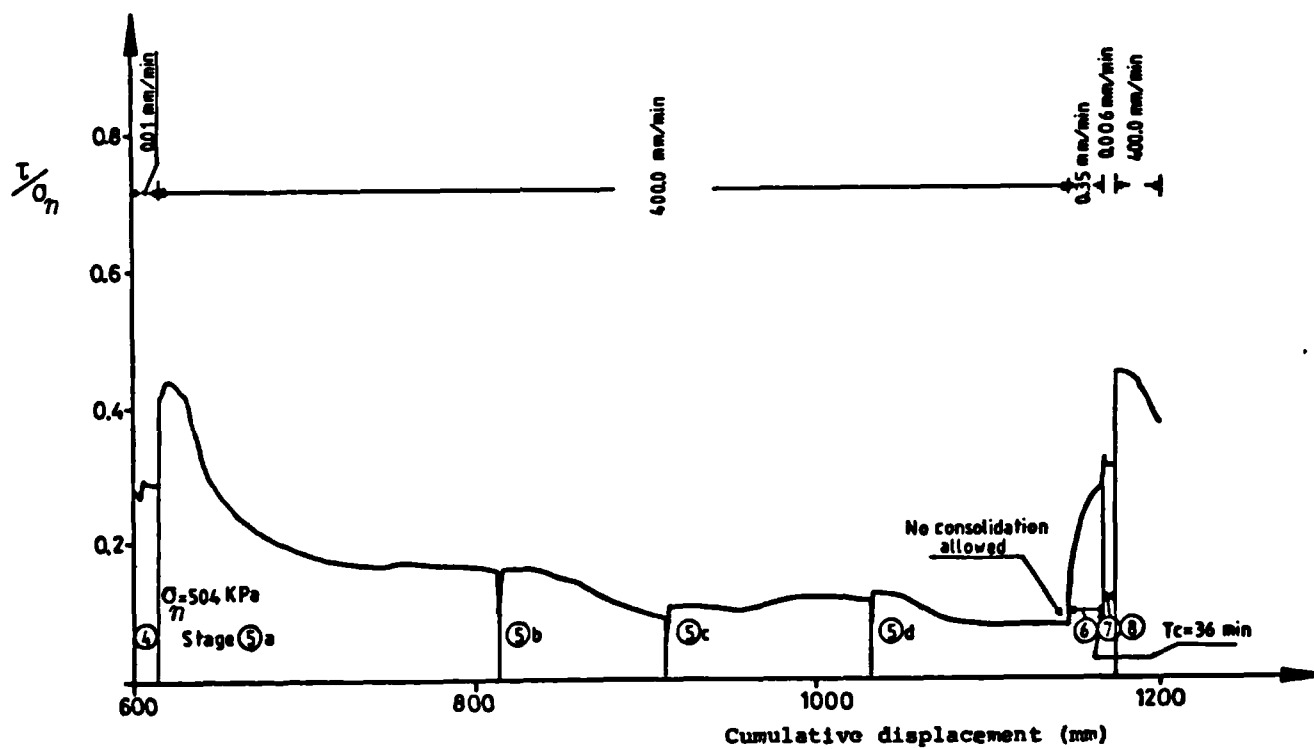
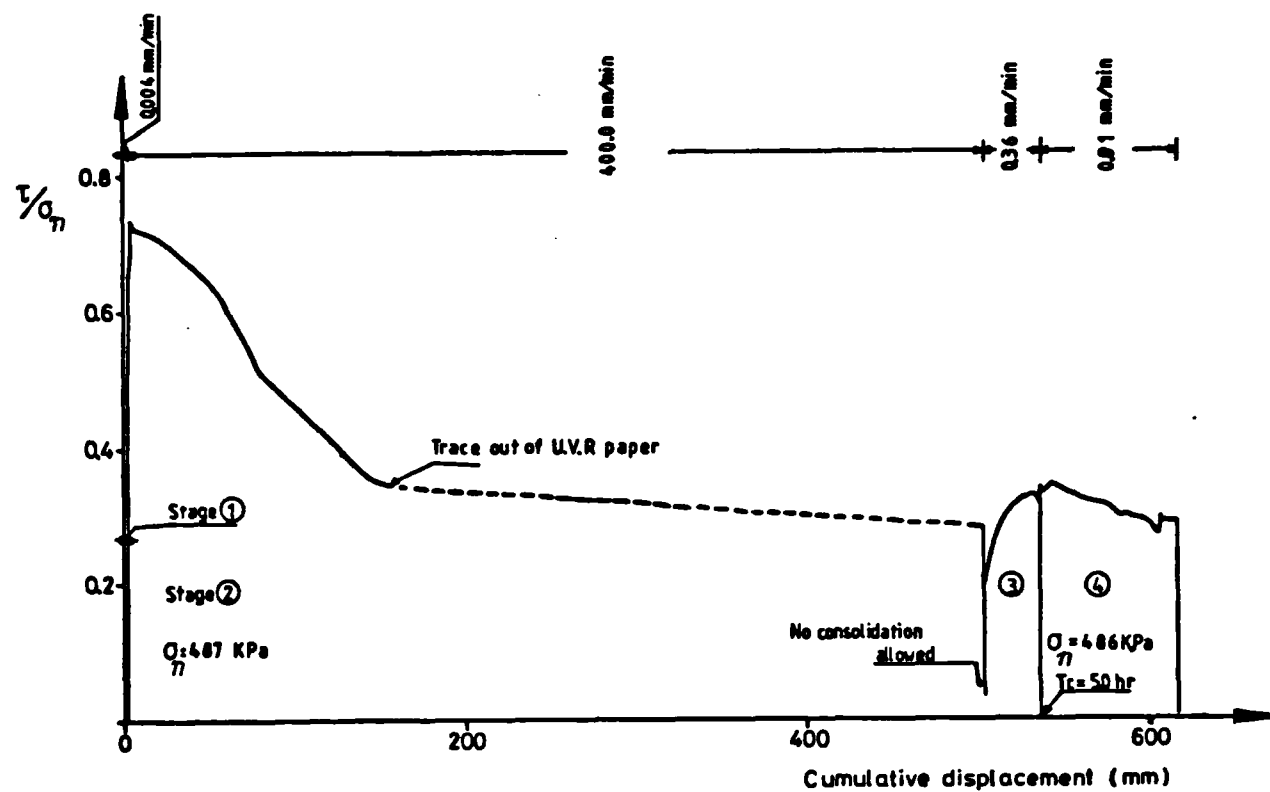


Fig. A2.8a Ring shear test 7 on clayey siltstone A

Table A2.7 (Continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
8a	400	1238.0	504.0	b,c:0.450 0.239	0.0 64.0	9.6
8b	400	1270.0	504.0	b,c:0.262 0.168	0.0 32.0	4.8
8c	400	1300.0	504.0	b,c:0.184 0.157	0.0 30.0	4.8
8d	400	1339.0	504.0	b,c:0.710 0.164	0.0 39.0	6
8e	400	1375.0	504.0	b:0.176 c:0.183	0.0 36.0	5.4
8f	400	1410.0	504.0	b:0.190 c:0.196 d:0.172	0.0 35.0	5.4
8g	400	1444.0	504.0	b,c:0.182 0.090	0.0 34.0	5.4
8h	400	1476.0	504.0	b,c:0.102 d:0.070	0.0 32.0	4.8
8i	400	1511.0	504.0	b,c:0.080 d:0.078	0.0 35.0	5.4
8j	400	1577.0	504.0	b:0.095 0.150	0.0 66.0	9.6
9	1.55	1580.0	504.0	0.154 0.181	3.0	
10	0.007	1630.0	504.0	0.218 0.247 0.295 0.311 0.287	0.0 1.0 4.0 9.0 50.0	
11	0.01	1787.0	504.0	e:0.287 0.218	157.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

* duration of particular substage sec

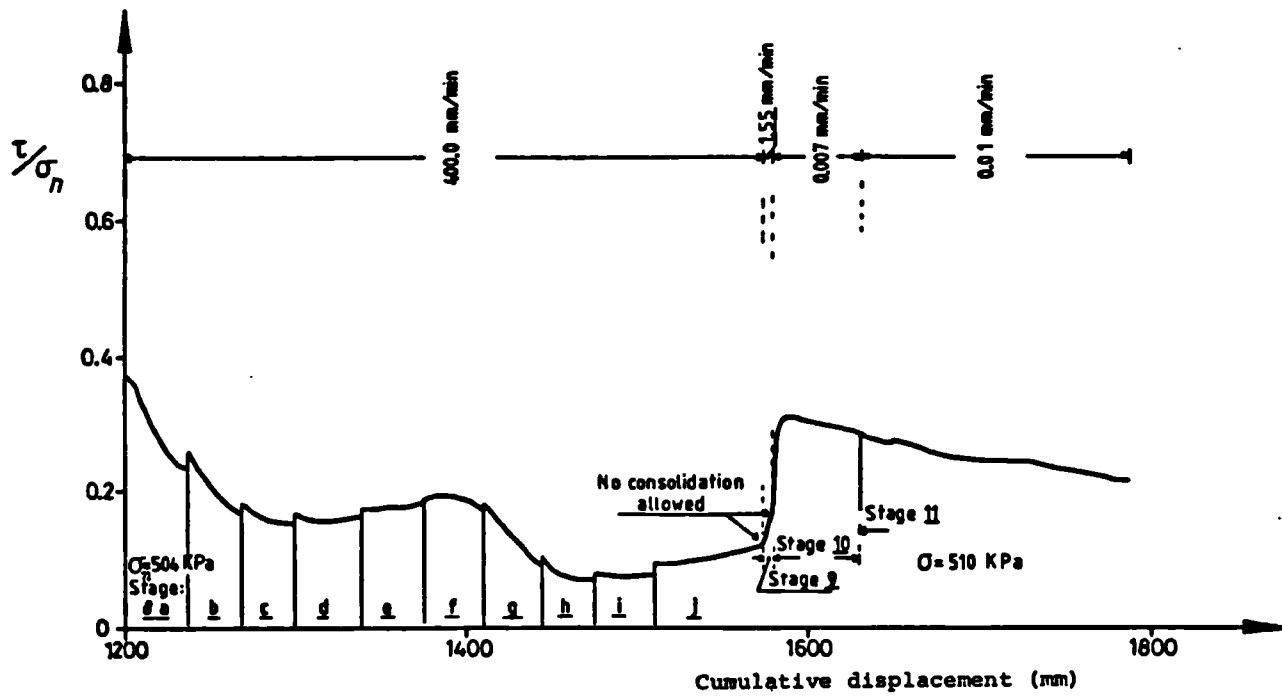


Fig. A2.8b Ring shear test 7 on clayey siltstone A

Table A2.8 Results from ring shear test 8 on clayey siltstone A

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	460	535.5	474.0	b,c:0.630 d:0.265	0.0 535.5	
2	0.006	635.2	490.0	e:0.285 0.237	100.0	
3	430	1489.0	484.0	b:0.350 c:0.360 0.257 0.219 d:0.206 0.242	0.0 48.0 774.0 1848.0 569.8 853.8	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

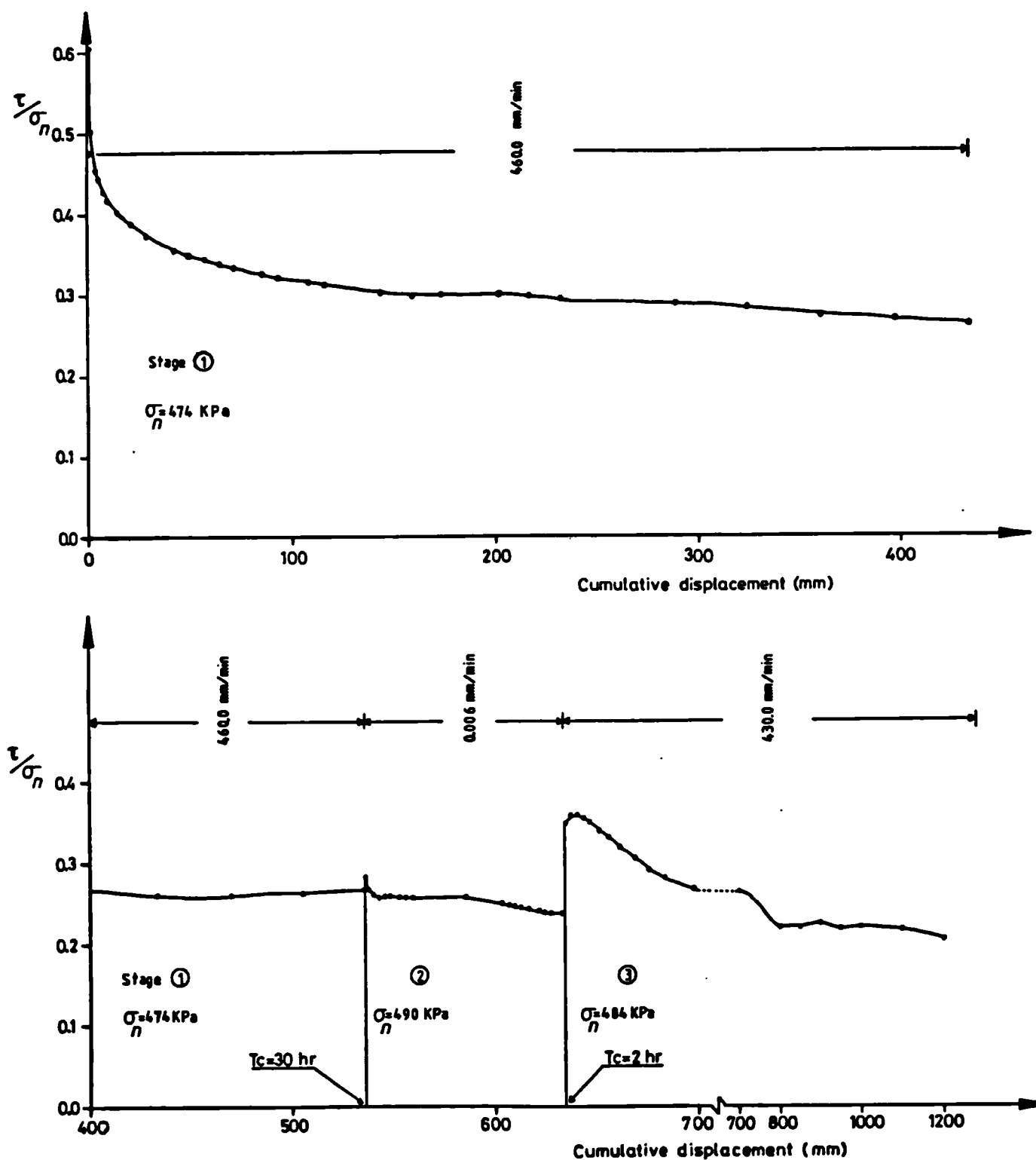


Fig. A2.9a Ring shear test 8 on clayey siltstone A
(soil-glass interface)

Table A2.8 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
4	0.006	1497.0	483.0	e:0.273 0.242	8.0	
5	0.006	1520.0	165.0	peak:0.350 0.270	23.0	
6	0.006	1533.0	80.0	peak:0.355 0.300	13.0	

e: slow peak after fast shearing

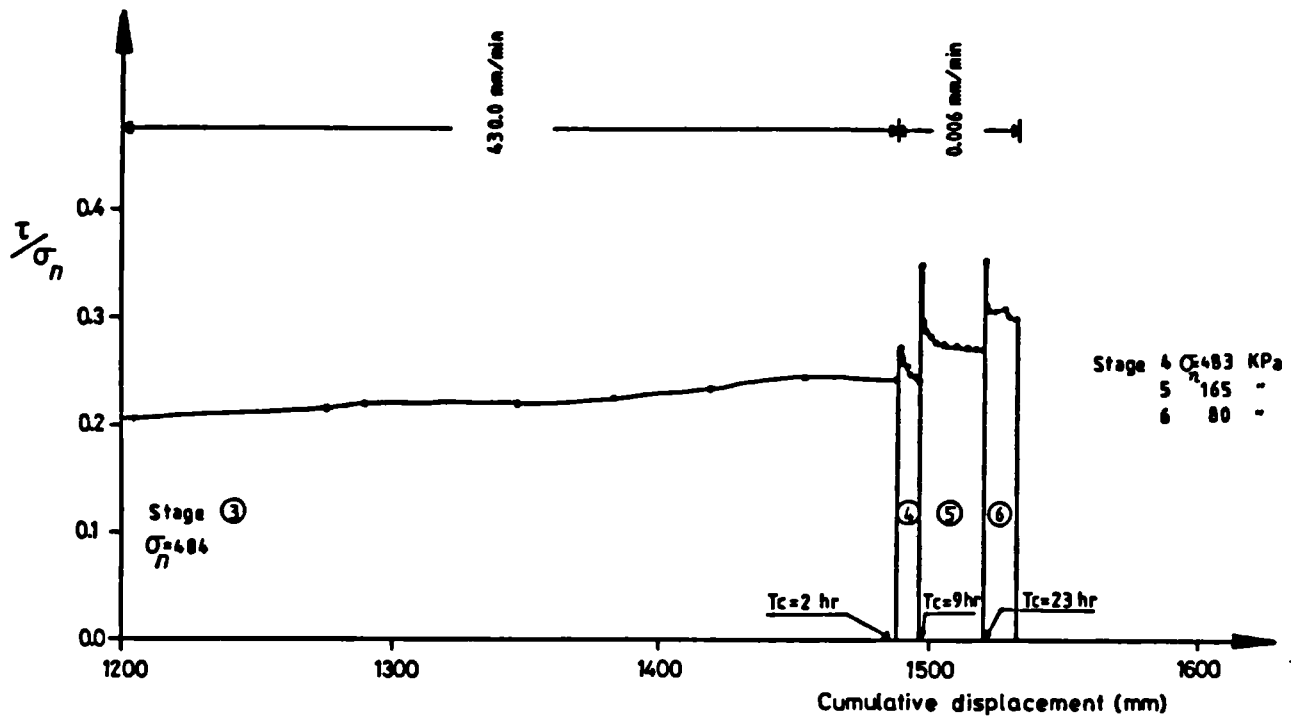


Fig. A2.9b Ring shear test 8 on clayey siltstone A
(soil-glass interface)

Table A2.9 Results from ring shear test 9 on claystone B

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
	1 400					
2		200.0	188.0	b,c: 1.330 0.340	0.0 200.0	
	0.0065	240.0	188.0	e: 0.545		(1)
	0.036	244.0		0.440		
	0.0065	292.0		0.350		
	0.0361	336.0		0.270		(1)
	0.0065	346.0		0.250		
	0.01	518.0		0.220	265.0	

b: threshold, c: fast peak, e: slow peak after fast shearing
(1) Gap closed

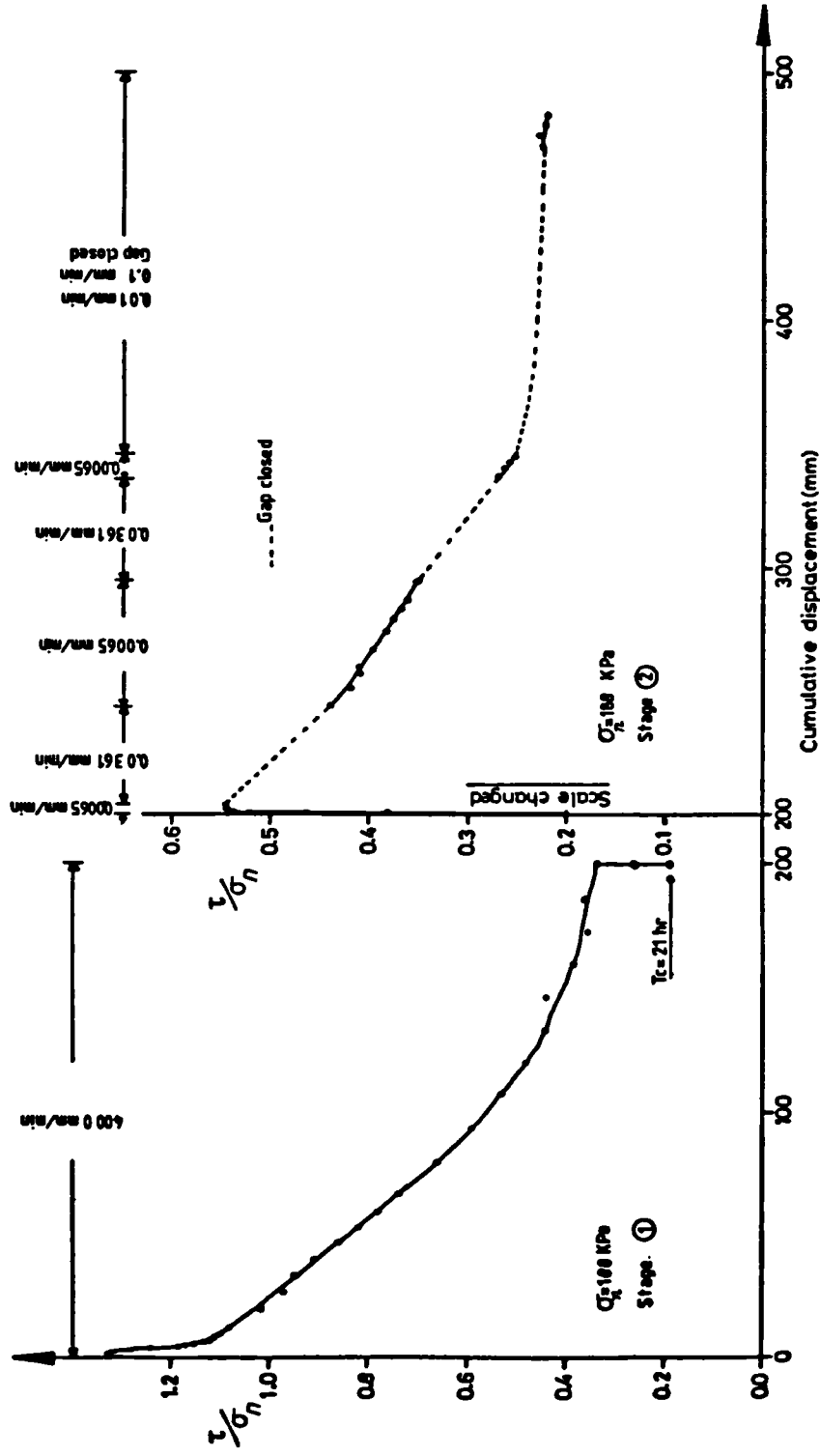


Fig. A2.10a Ring shear test 9 on claystone B

Table A2.9 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
3	400	792.0	196.0	b:0.275 c:0.400 d:0.210	0.0 200.0	
4	0.01	990.0	190.0	e:0.395 0.200		
5	100	1176.0	183.0	b,c:0.250 d:0.215	0.0 120.0	
6	0.01	1284.0	183.0	e:0.252 0.190		
7	10	1413.0	183.0	b,c:0.20 d:0.180	0.0 45.0	

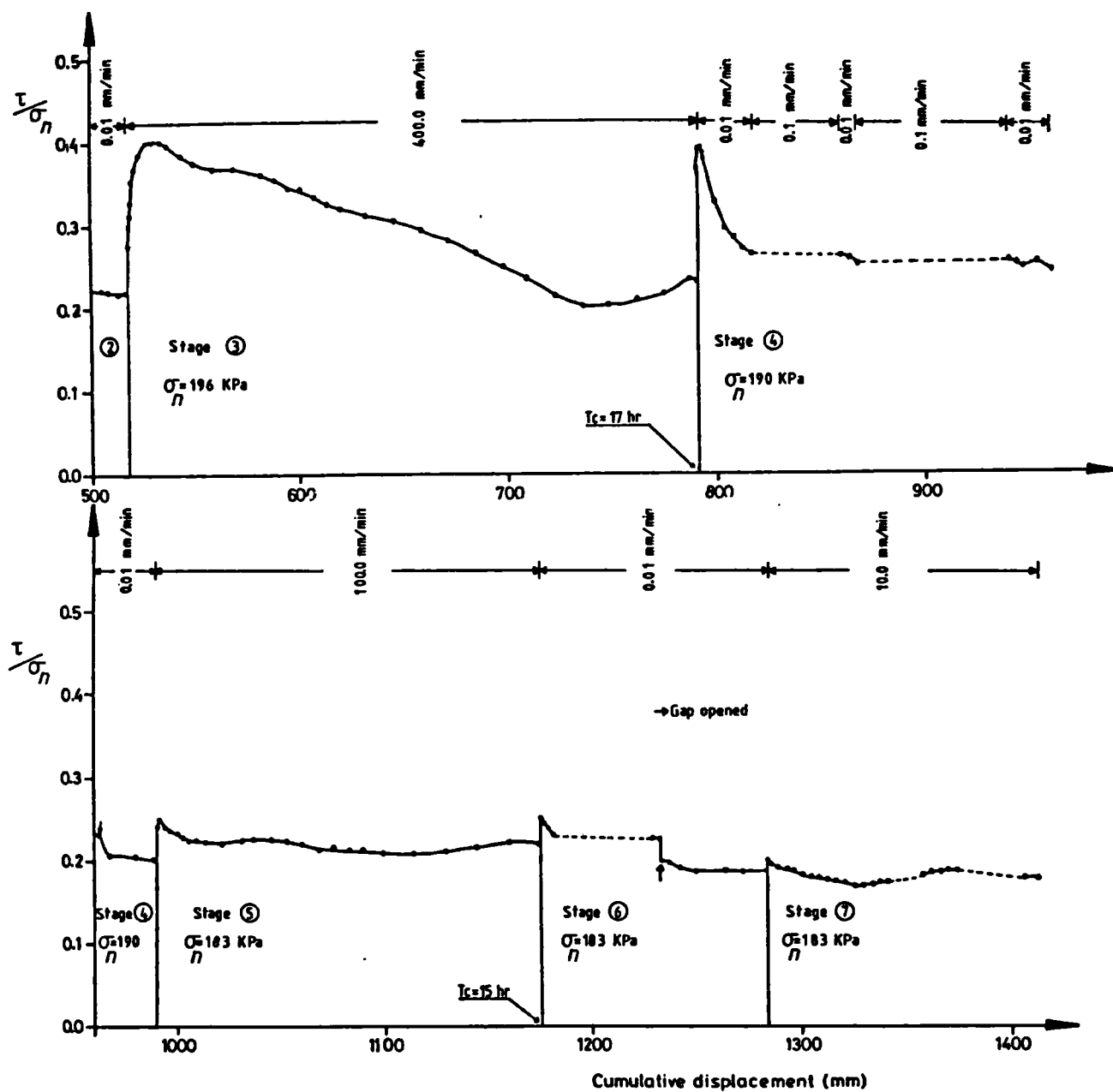


Fig. A2.10b Ring shear test 9 on claystone B

Table A2.9 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
8	0.01	1417.0	199.0	e:0.180		
9	0.01	1458.0	535.0	0.180	4.0	
10	400	1609.0	535.0	*	41.0	
11	0.01	1650.0	525.0	e:0.282		
				0.190	41.0	
12	400	1650.0	525.0	b:0.305		
				c:0.335		
				d:0.270	400.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

* No recording was performed

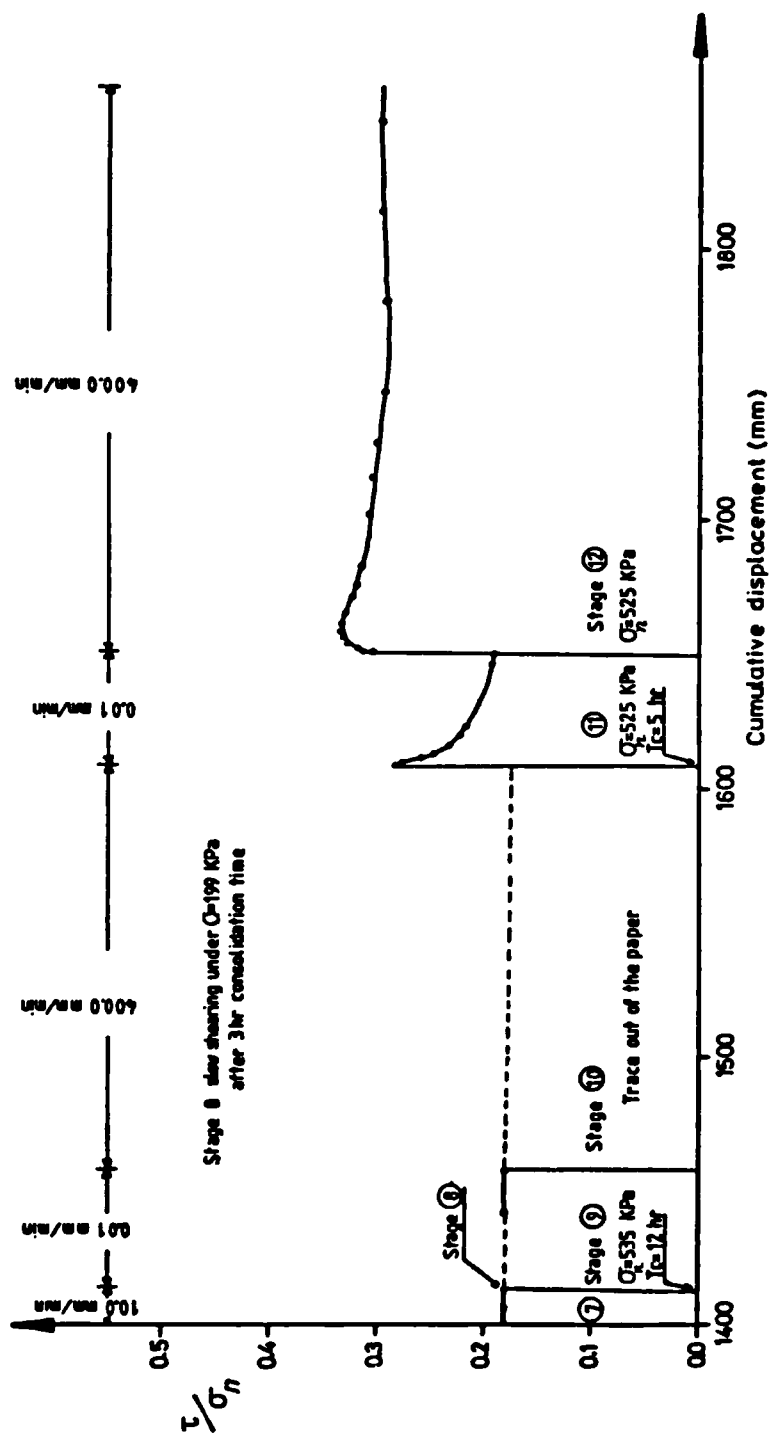


Fig. A2.10c Ring shear test 9 on claystone B

Table A2.9 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
13	0.01	1956.0	531.0	e:0.270 0.190	44.0	
14	100	2109.0	548.0	b,c:0.245 d:0.200	0.0 110.0	
15	0.01	2124.0	539.0	e:0.210 0.190	15.0	
16	10	2244.0	539.0	b,c:0.210 d:0.150	0.0 120.0	
17	0.01	2264.0	537.0	e:0.160 0.170	20.0	
18	774	2478.0	537.0	b:0.345 c:0.420 d:0.335	0.0 6.0 214.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

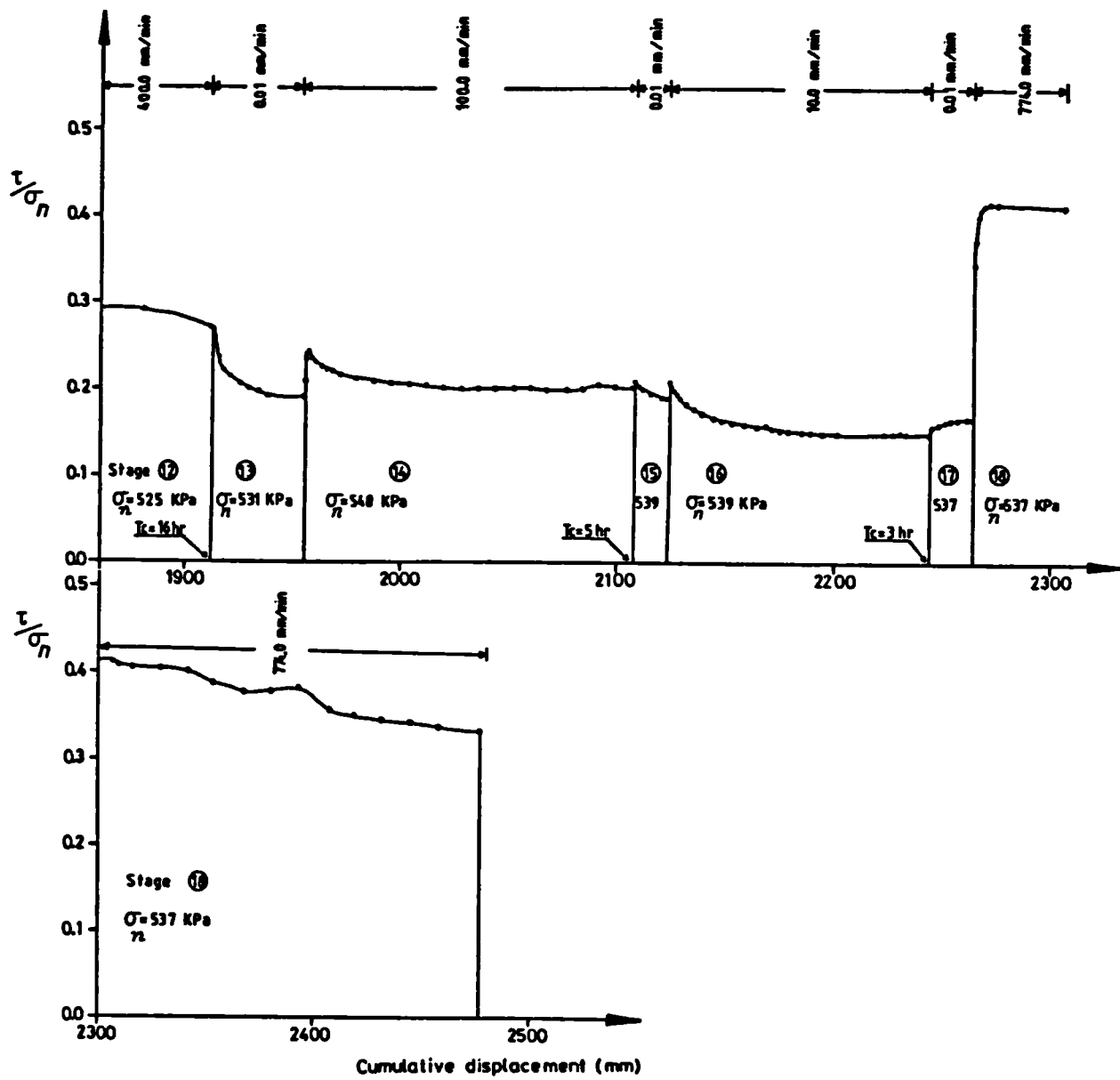


Fig. A2.10d Ring shear test 9 on claystone B

Table A2.10 Results from ring shear test 10 on claystone C

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	400	188.0	193.0	b:1.100 c:1.200 0.320	0.0 4.0 188.0	
2	0.01	436.0	200.0	e:0.520 0.180		
3	400	638.0	204.0	b:0.300 c:0.380 d:0.220	0.0 187.3	
4	0.01	730.0	200.0	e:0.300 0.190		
5	100	922.0	204.0	b,c:0.245 d:0.140	0.0 170.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

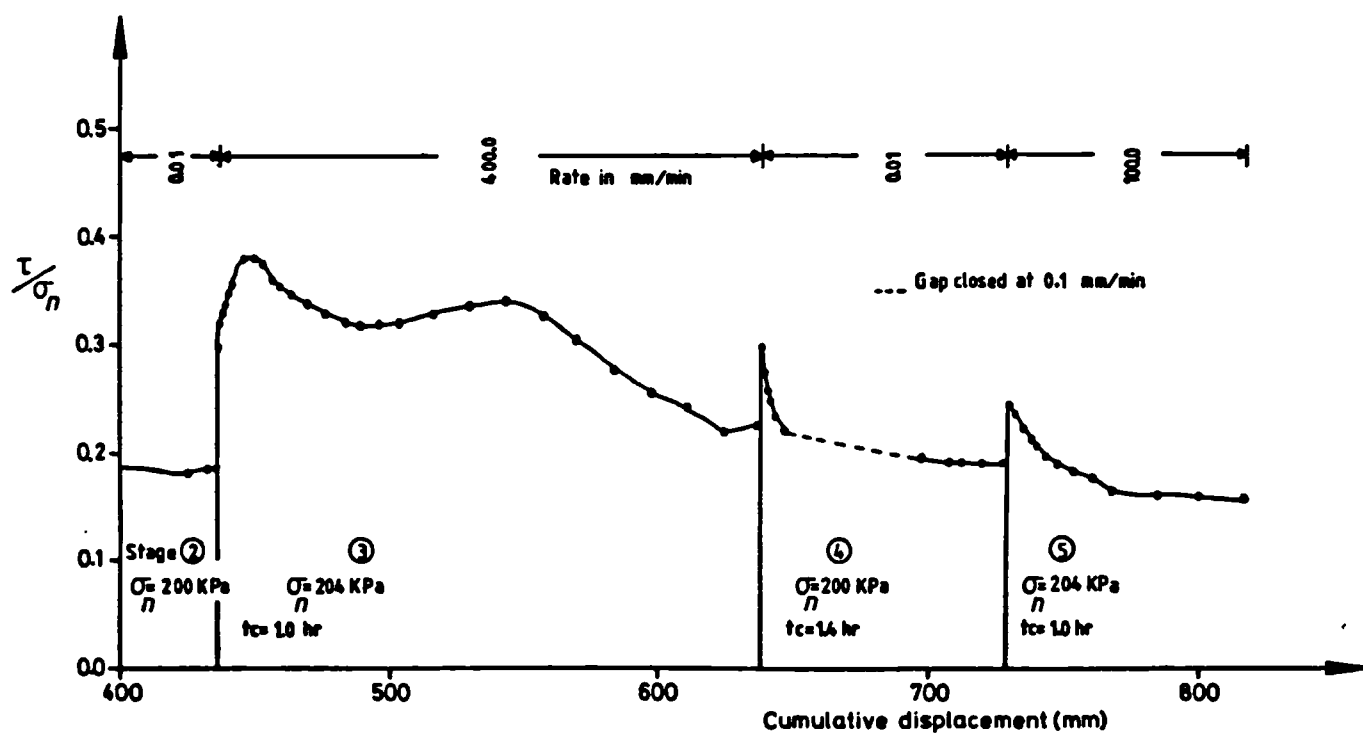
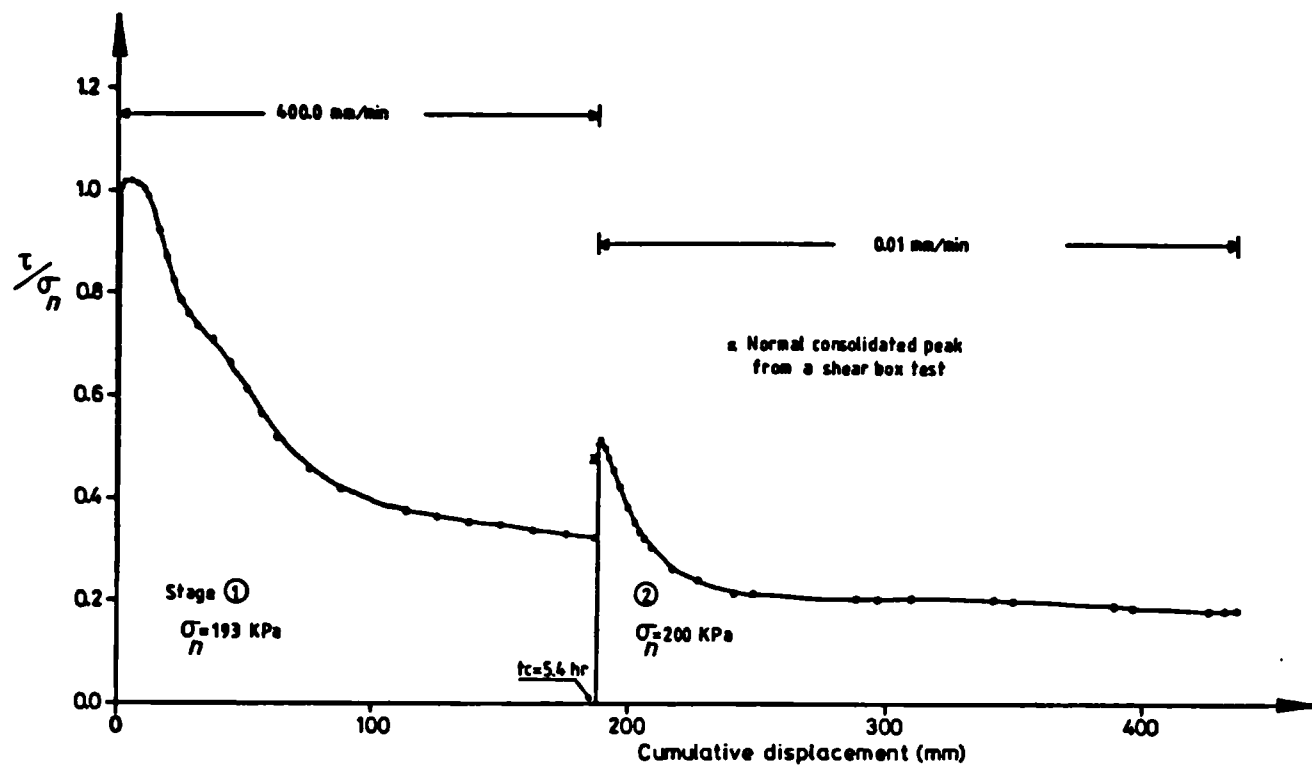


Fig. A2.11a Ring shear test 10 on claystone C

Table A2.10 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
6	0.01	935.0	196.0	e:1.186 0.180	35.0	
7	10	1032.0	197.0	b,c:0.192 d:0.170	0.0 97.0	
8	0.01	1040.0	199.0	0.175		
9	0.01	1062.0	529.0	0.162 0.166		
10	400	1110.0	544.0	b:0.260 c:0.295 no further recording	0.0 80.0	
11	400	1386.0	544.0	b,c:0.300 d:0.127	0.0 276.0	
12	0.01	1442.0	518.0	e:0.227 0.180	12.0	
13	100	1636.0	520.0	b:0.225 c:0.250 d:0.140	0.0 160.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

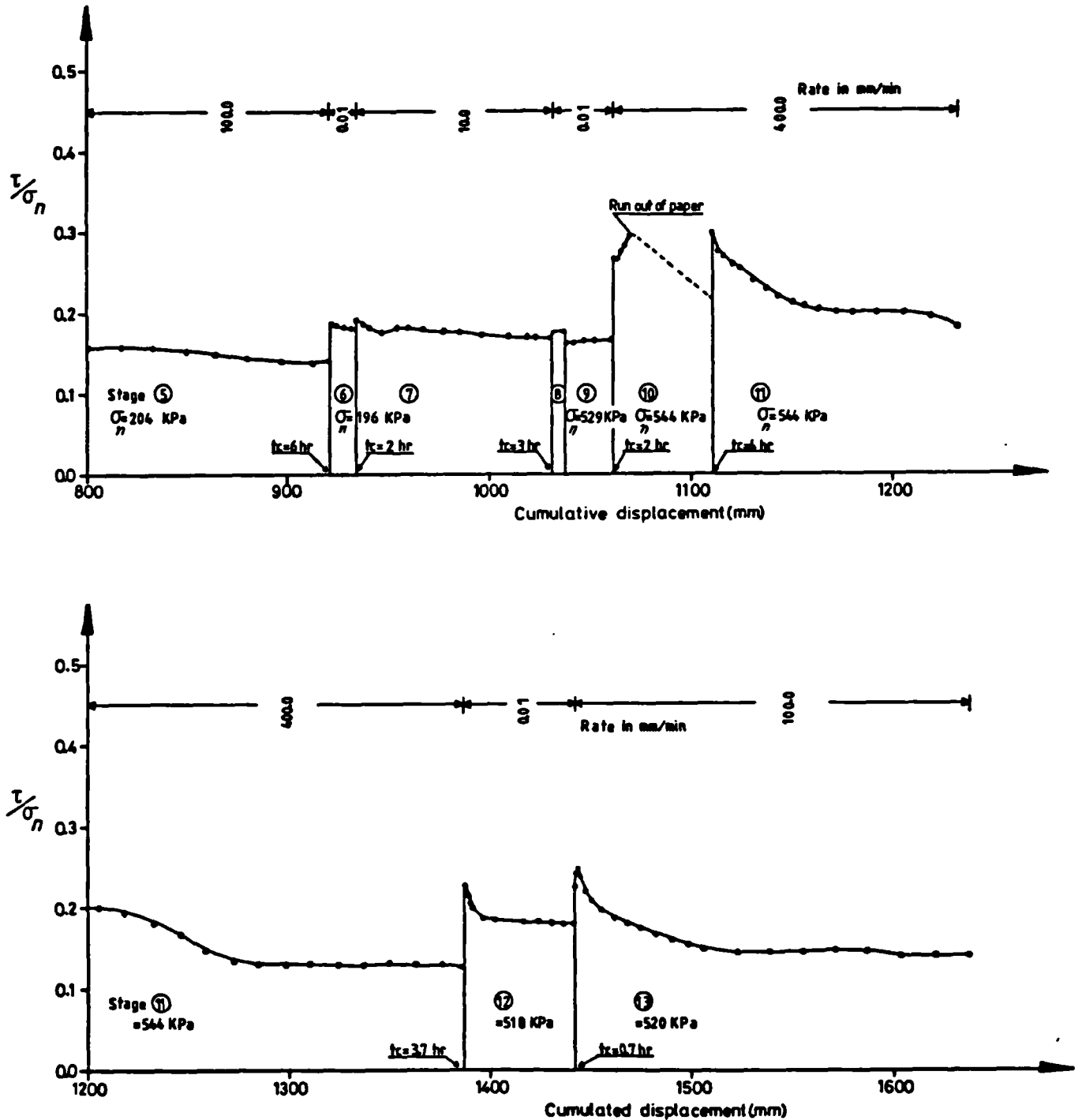


Fig. A2.11b Ring shear test 10 on claystone C

Table A2.10 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
14	0.01	1644.0	519.0	e:0.170 0.170		
15	10	1766.0	522.0	b,c:0.185 d:0.160	0.0 30.0	
16	0.01	1788.0	520.0	0.170		
17	810	2058.0	541.0	b:0.280 c:0.365 d:0.140	0.0 4.0 270.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

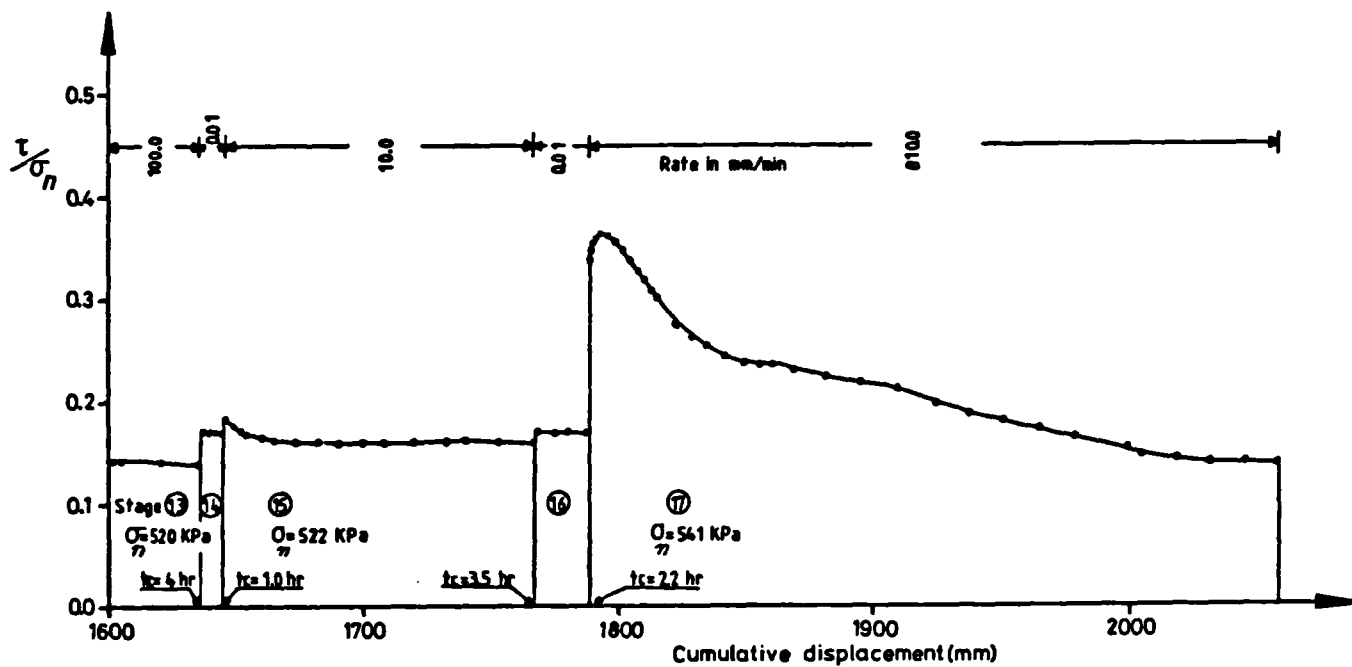


Fig. A2.11c Ring shear test 10 on claystone C

Table A2.11 Results from ring shear test 11 on siltstone

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	400	78.0	169.0	b:1.330 c:1.420 0.860	5 78.0	
2	0.01	142.0	181.0	e:0.635	0.0-64.0	
3	400	394.0	177.0	b,c:1.050 0.562	252.0	
4	0.01	405.0	179.0	e:0.635	0.0-11.0	
5	100	628.9	179.0	b,c:0.862 d:0.635	0.0 223.9	
6	0.01	644	188.0	e:0.630	15.1	
7	10	756.0	188.0	b,c:0.800 d:0.600	0.0 112.0	
8	0.01	764.0	177.0	e:0.640 0.625	8.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

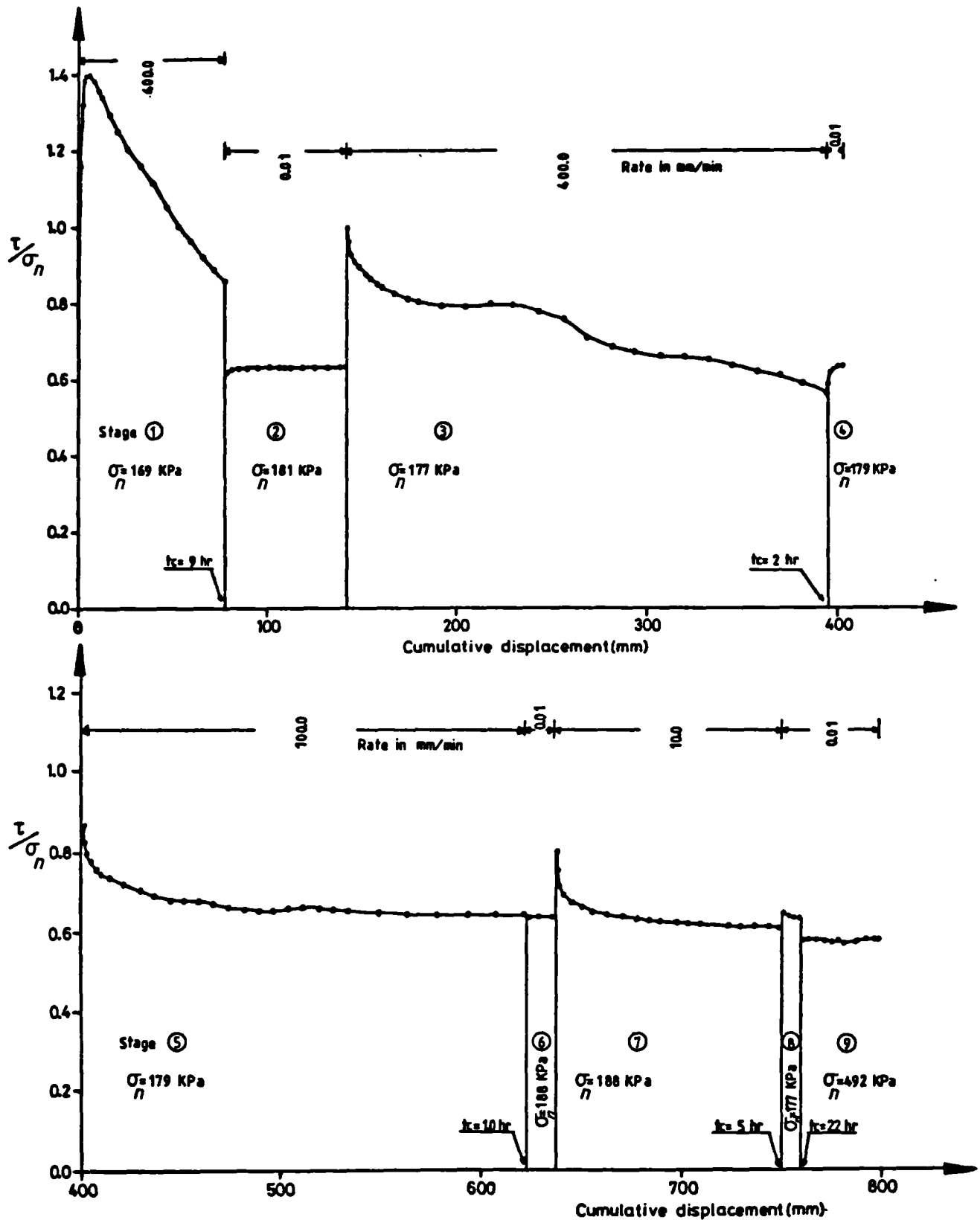


Fig. A2.12a Ring shear test 11 on siltstone

Table A2.11 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
9	0.01	803.0	493.0	0.570	41.0	
10	400	1052.0	490.0	b,c:0.840 d:0.540	0.0 249.0	
11	0.01	1078.0	487.0	e:0.550 0.515	26.0	
12	100	1260.0	487.0	b,c:0.750 d:0.525	0.0 182.0	
13	0.01	1297.0	477.0	e:0.540 0.515	37.0	
14	100	1533.0	479.0	b,c:0.530 d:0.510	0.0 236.0	
15	0.01	1650.0	474.0	e:0.510 0.506	117.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

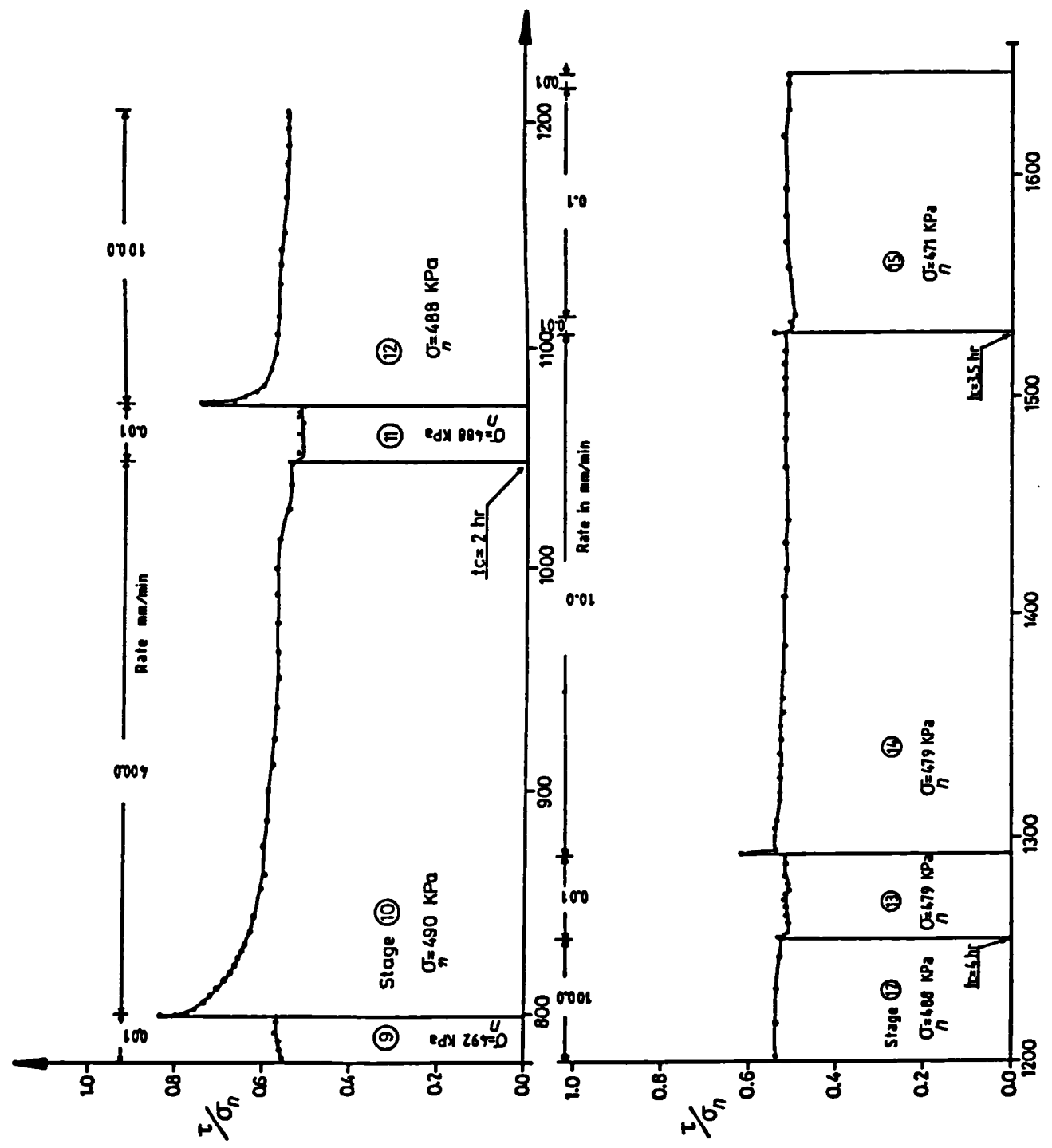


Fig. A2.12b Ring shear test 11 on siltstone

Table A2.12 Results from ring shear test 12 on clayey siltstone B

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
1	0.006 0.01	7.5 334.0	510.0 510.0	peak:0.490 0.481 0.325	3.6 7.5 334.0	
2	436	625.0	496.0	b:0.442 c:0.452 d:0.079	0.0 8.0 291.0	
3	0.35	658.0	496.0	0.252 0.303 0.312	9.0 22.0 33.0	27.0 62.0 94.0
4	0.02	812.0	486.0	0.331 0.320	154.0	
5	100	1006.0	524.0	b:0.405 c:0.408 0.253	194.0	
6	0.35	1008.0	524.0	0.283	0.35	5.0
7	0.32 0.02	1019.0 1044.0	524.0	0.341 0.335 0.341 0.325	0.3 25.0	14.2
8	11.5	1159.0	524.0	b,c:0.378 0.326	115.0	
9	0.36 1.44	1164.0 1170.0	524.0	0.324 0.322	5.0	0.0 14.2-18.4
10	0.026	1174.0	526.0	0.330 0.312	4.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
 * Elapsed time from the start of the particular stage in min

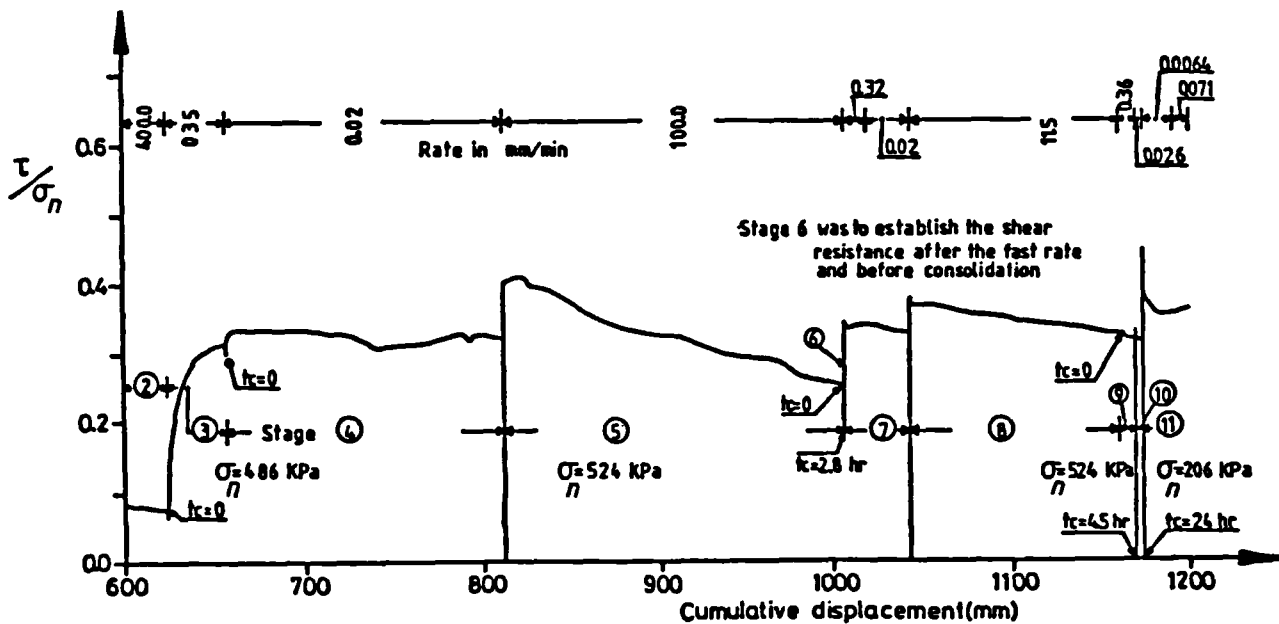


Fig. A2.13a Ring shear test 12 on clayey siltstone B

Table A2.12 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
11	0.0064	1190.0	206.0	peak:0.446		
				0.349		
	0.071 0.0062	1218.0 1225.0		0.345	7.0	
12	464	1457.0	205.0	b:0.503 c:0.530 0.175	0.0 232.0	
13	0.35	1458.0	205.0	0.173 0.243	1.5	5 min
14	0.0065	1459.0	203.0	0.389	1.3	
	0.014	1471.0		0.373	12.0	
	0.0127	1474.0		0.372	3.0	
15	100.0	1670.0	198.0	b,c:0.487 d:0.276	0.0 196.0	
16	0.00125	1672.0	198.0	e:0.381		
	0.0340	1702.0	198.0	0.354	30.0	
17	11.9	1821.0	198.0	b,c:0.415 d:0.397	0.0 10.0-119.0	
18	0.317	1828.0	198.0	e:0.395 0.372	7.0	
19	926	2136.0	198.0	b:0.550	0.0	
				c:0.580		
				d:0.170	308.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
 * elapsed time from the start of the particular stage

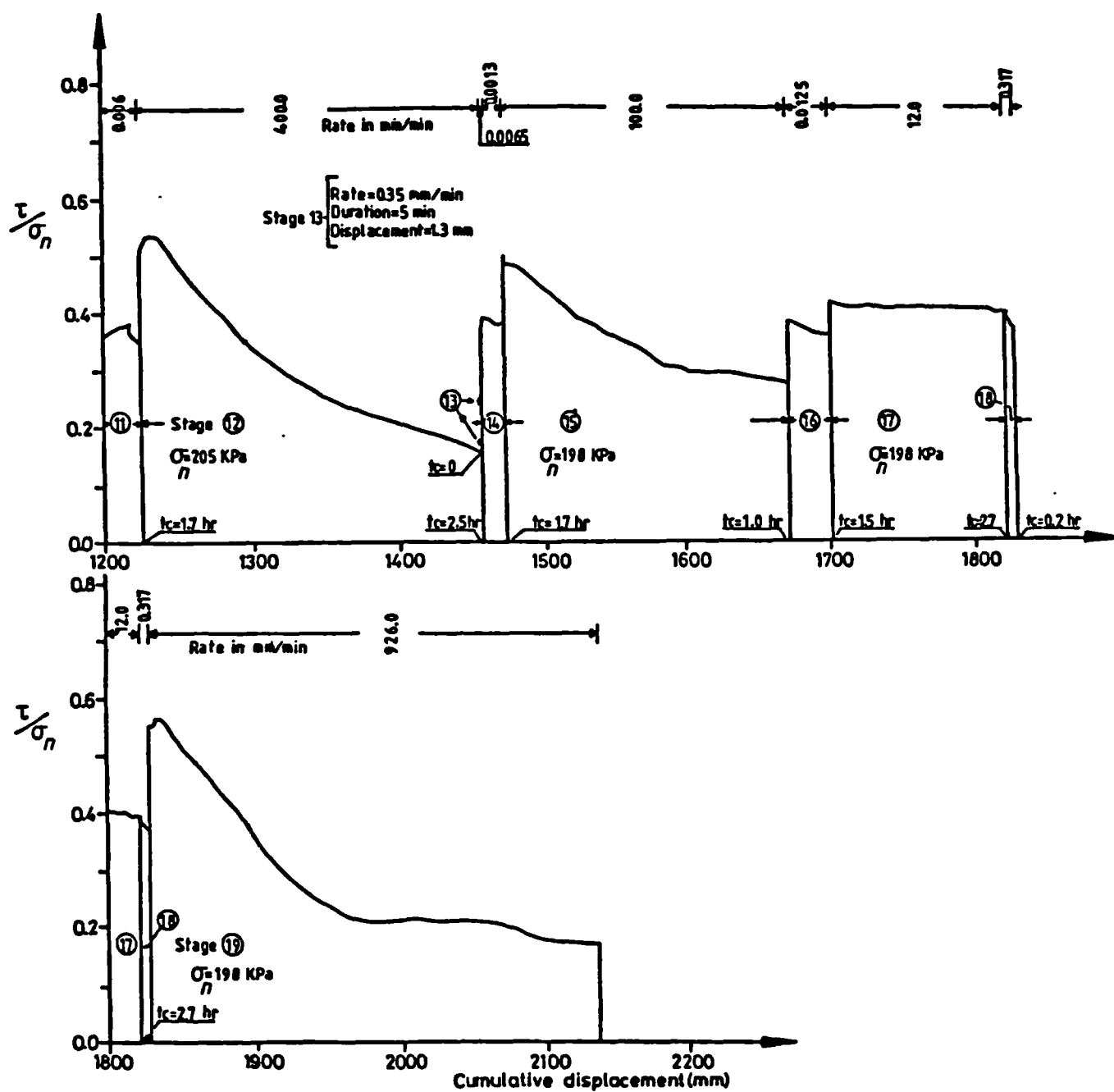


Fig. A2.13b Ring shear test 12 on clayey siltstone B

Table A2.13 Results from ring shear test 13 on clayey siltstone

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.01	292.0	507.0	peak:0.512 0.332	292.0	
2	400	677.0	515.0	b,c:0.443 d:0.130	0.0 385.0	
3	0.01	746.0	517.0	e:0.340 0.322	69.0	
4	2000	752.0 756.0	512.0	b,c:0.493 b,c:0.493	0.0-6.0 0.0-4.0	
5	4500	809.0	512.0	b,c:0.506 0.462	0.0 53.0	
6	1000	992.0	512.0	b,c:0.522 d:0.158	183.0	
7	0.02	1017.0	517.0	e:0.349 0.339	25.0	
8	2100	1297.0	527.0	b,c:0.518 d:0.130	0.0 280.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

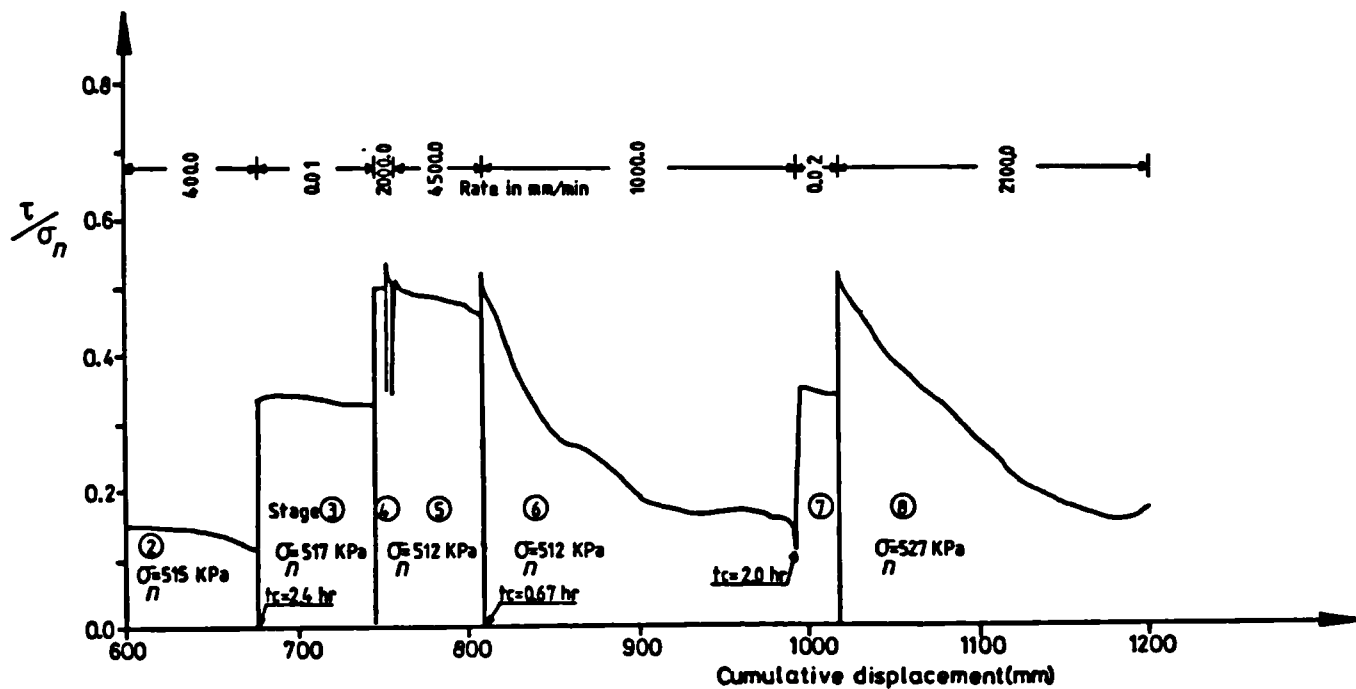
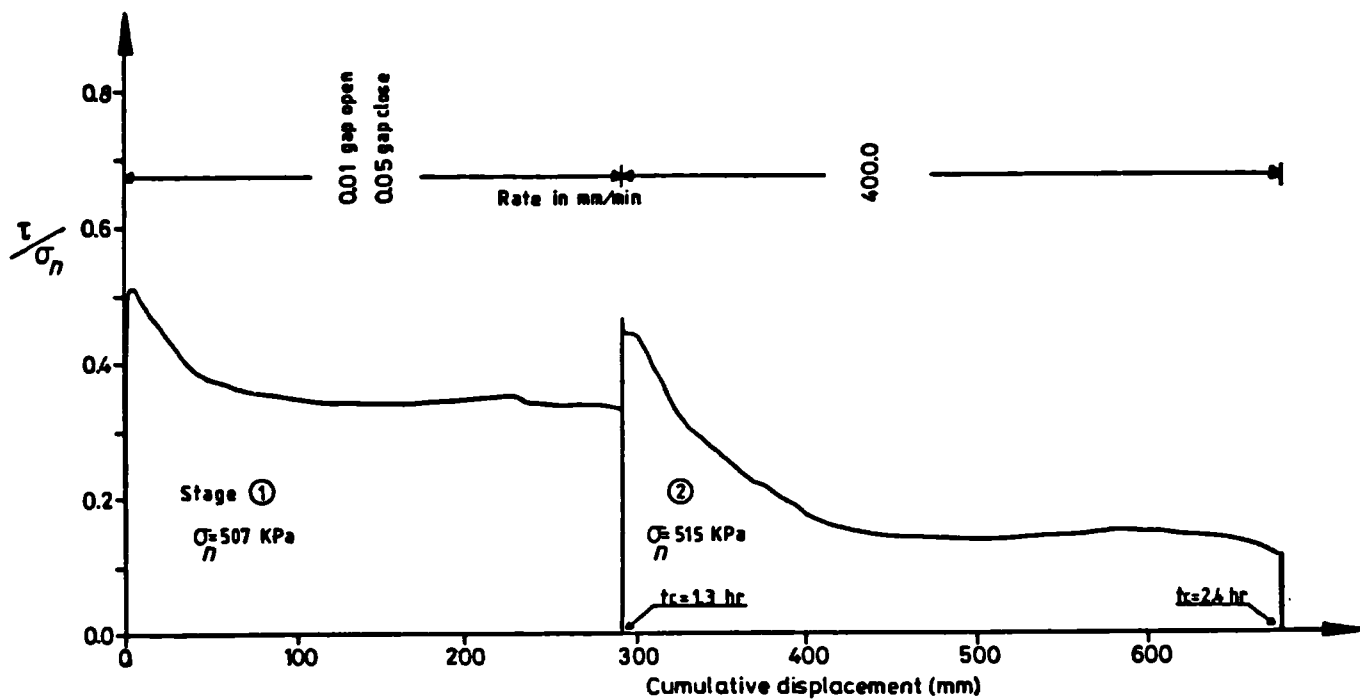


Fig. A2.14a Ring shear test 13 on clayey siltstone B

Table A2.13 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
9	0.014	1311.0	534.0	e:0.360 0.342	14.0	
10	0.0148	1423.0	201.0	peak:0.506 0.346	112.0	
11	400	1558.0	204.0	b,c:0.496 d:0.188	0.0 135.0	19.8 sec
12	0.0175	1602.0	192.0	e:0.380 0.362		
13	5600	1883.0	203.0	b,c:0.645 d:0.148	0.0 281.0	3 sec
14	0.017	1907.0	202.0	e:0.396 0.367	24.0	
15	870	1246.0	214.0	b,c:0.546 d:0.104	0.0 239.0	16.2 sec
16	0.327	2148.0	214.0	0.162 0.215	2.0	5.8 min
17	0.327	2150.0	214.0	0.284 0.303	0.07 2.0	6.4 min
18	0.327	2155.0	214.0	0.389 0.346	0.06 5.0	15 min
19	0.021	2157.0	214.0	0.370 0.366	2.0	
20	0.021 0.0061	2294.0 2297.0	207.0 205.0	0.380 0.343 0.348		
21	2150	2578.0	205.0	b,c:0.608 d:0.130	0.0 281.0	7.8 sec

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
 * duration of the particular stage

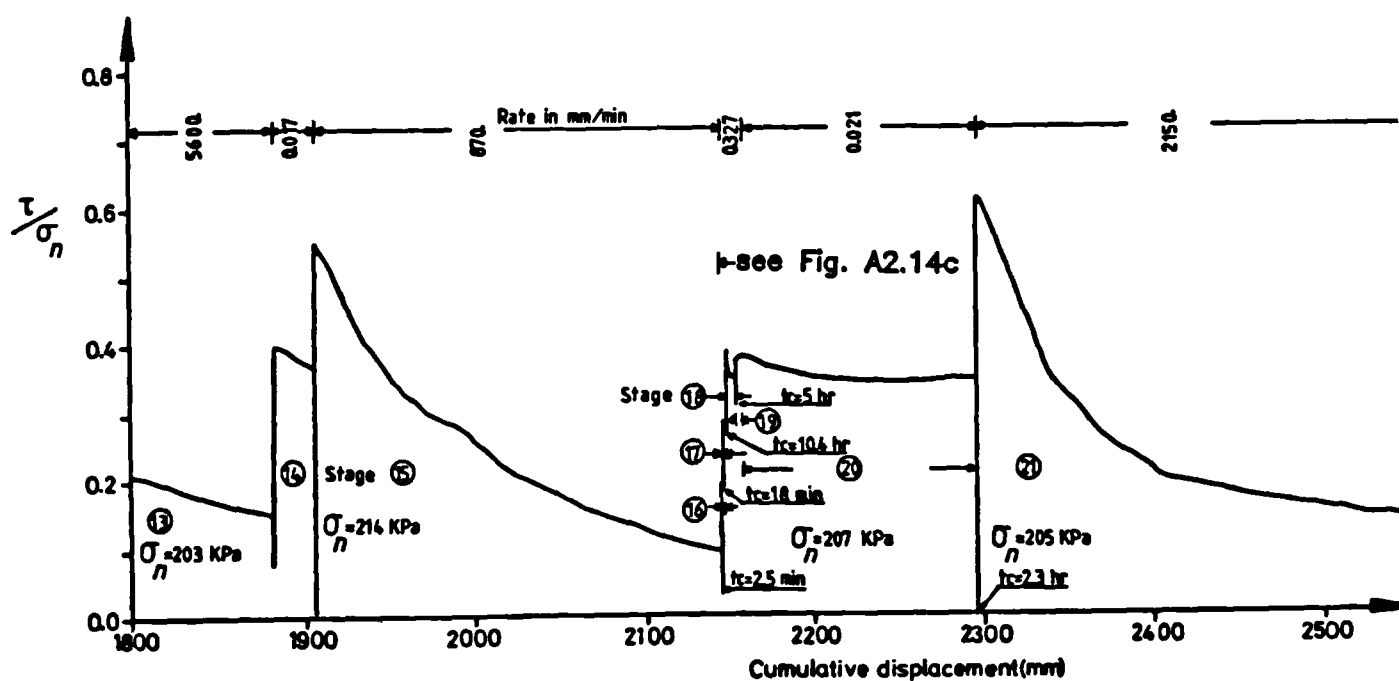
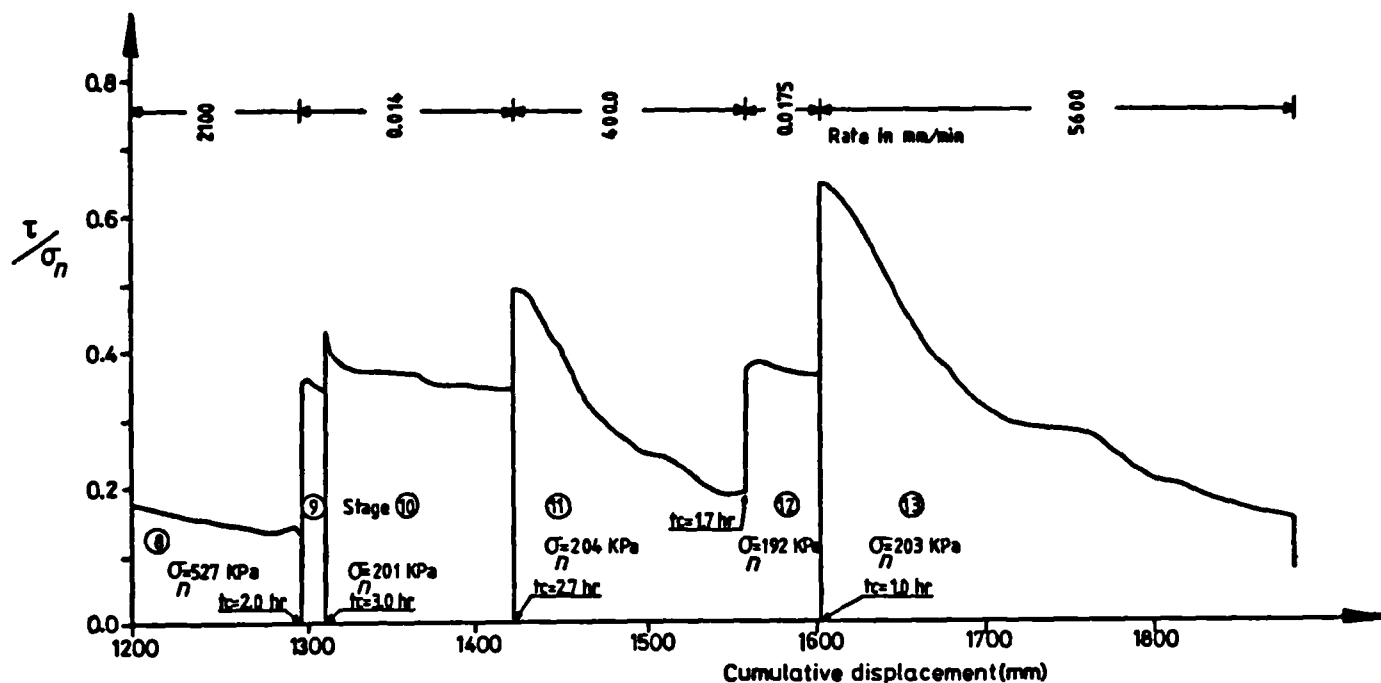


Fig. A2.14b Ring shear test 13 on clayey siltstone B

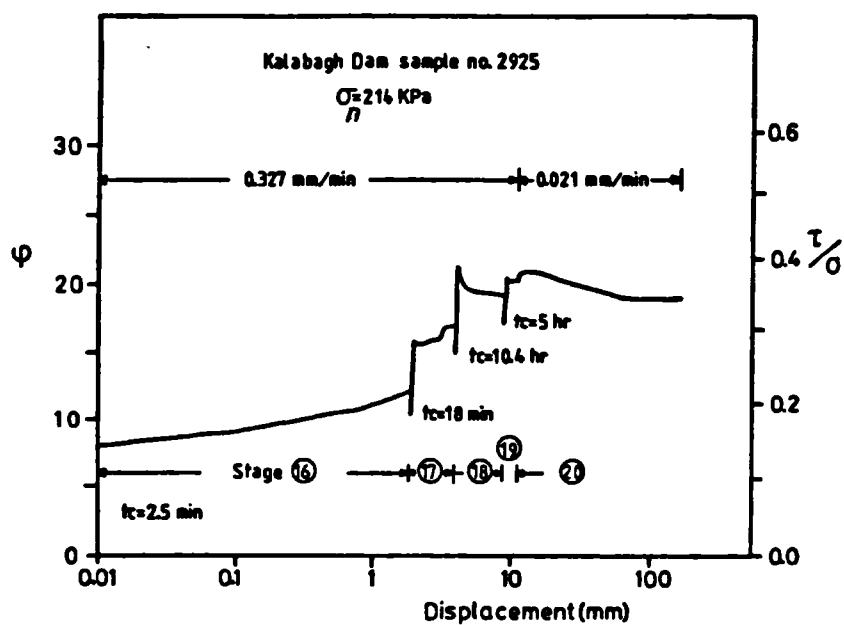


Fig. A2.14c Ring shear test 13 on clayey siltstone B

Table A2.14 Results from ring shear test 14 on clayey siltstone B

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.03	325.0	500.0	peak:0.449 0.326 0.310 0.288	1.82 26.0 107.0 325.0	
2	800	1116.0	492.0	b,c:0.426 0.288 0.180 d:0.143	0.7 187.0 480.0 791.0	
3	0.0066	1269.0	506.0	e:0.302 0.279 0.274	4.0 84.0 133.0	
4	6200	2240.0	500.0	b,c:0.475 0.278 d:0.091	1.0 203.0 971.0	
5	6200	3429.0	512.0	b,c:0.244 d:0.055	0.0 1189.0	
6	0.0139	3446.0	504.0	e:0.330 0.314	17.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

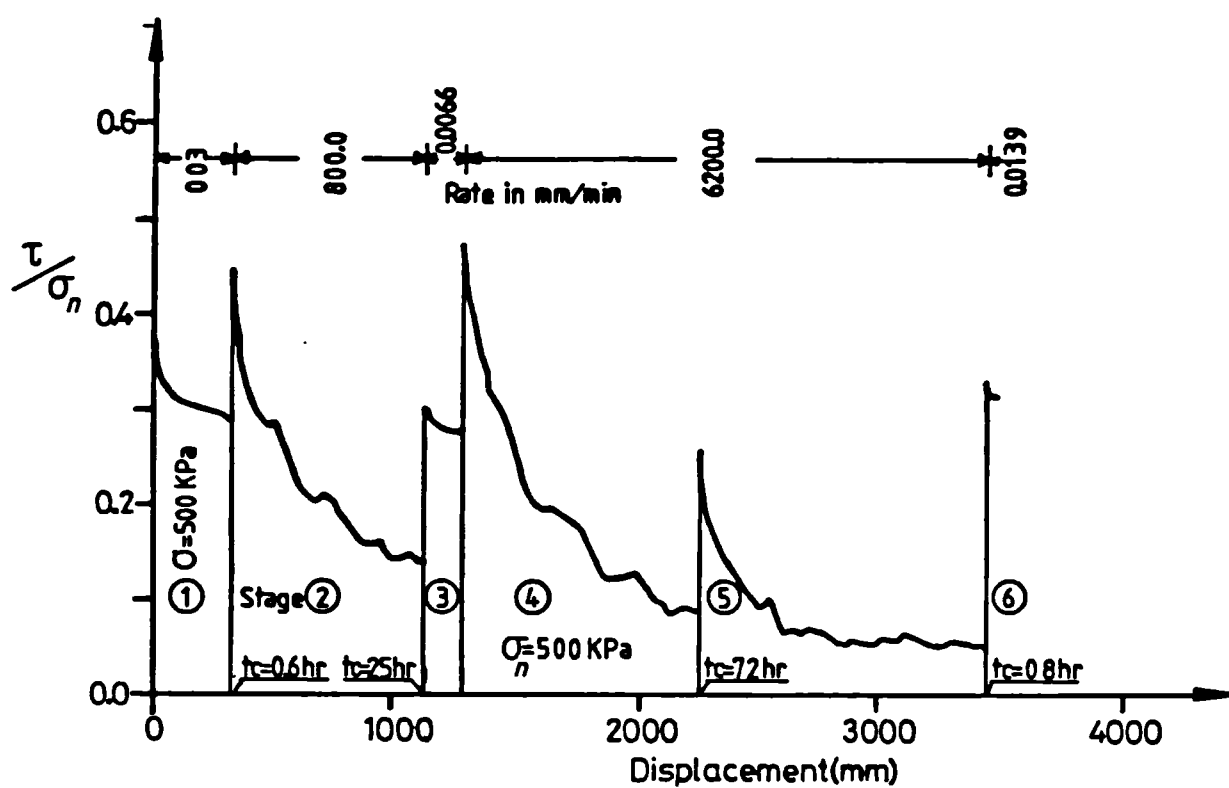


Fig. A2.15 Ring shear test 14 on clayey siltstone B
(soil-glass interface)

Table A2.15 Results from ring shear test 15 on clayey siltstone C

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
1	0.01	164.0	507.0	peak:0.627 0.500	688.0 164.0	
2	400	463.0	527.0	b,c:0.780 d:0.185 0.264	0.0 212.0 299.0	
3	0.319	467.0	527.0	0.272 0.485	0.0 4.6	6 min
4	0.0135	491.0	544.0	0.471 0.427		
5	100	683.0	544.0	b,c:0.610 d:0.231	0.0 192.0	
6	0.393	693.0	543.0	0.235 0.285	86.0	22 min
7	0.0129	706.0	528.0	0.510 0.498		
8	10.0	819.0	527.0	b,c:0.655 0.634 d:0.520	0.0 2.0 30.0-113.0	
9	0.016	821.0	525.0	e:0.527 0.500	2.0	
10	0.01	884.0	200.0	peak:0.696 0.527		
11	440	1105.0	206.0	b,c:0.936 0.750 0.699 d:0.153	1.0 2.0 3.0 221.0	
12	0.015	1108.0	205.0	0.516	3.0	
13	100	1306.0	222.0	b,c:0.792 d:0.356	1.0 198.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

* Elapsed time from the start of the particular stage

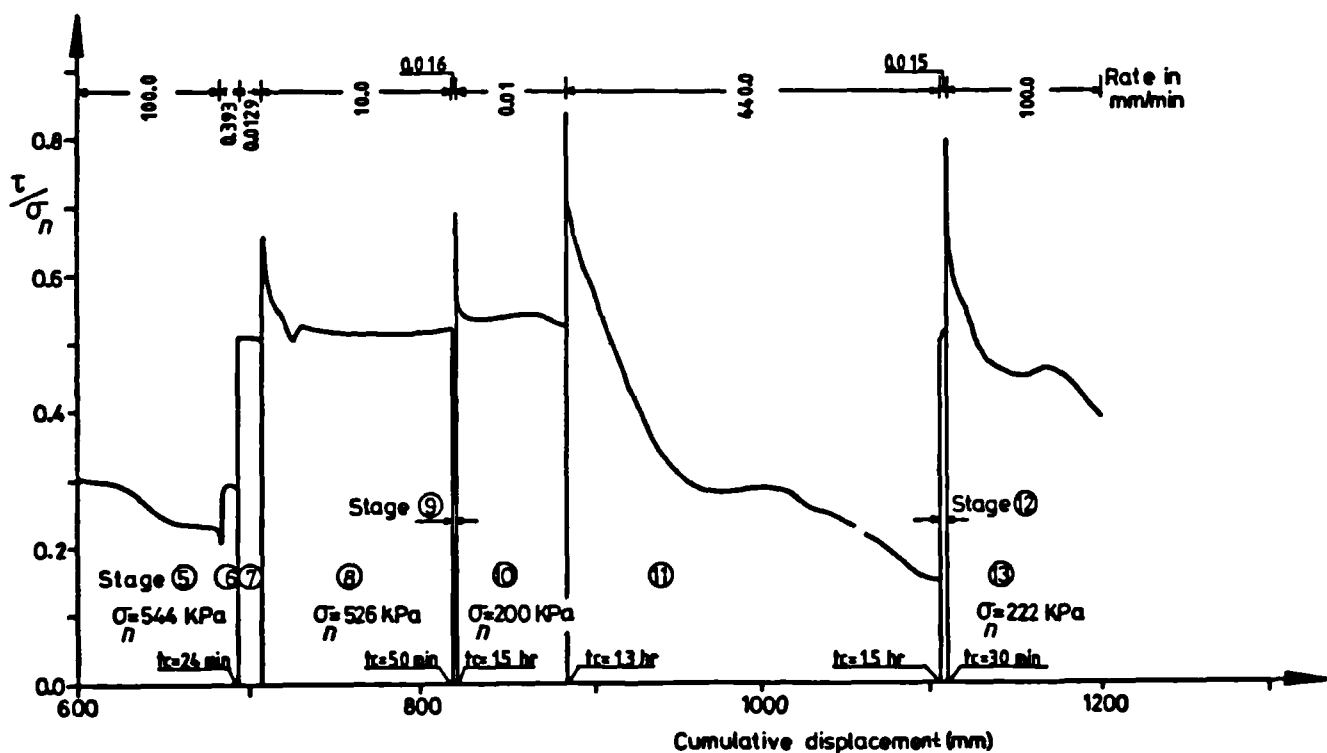


Fig. A2.16a Ring shear test 15 on clayey siltstone C

Table A2.15 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
14	0.015	1317.0	207.0	e:0.520 0.523		
15	12	1437.0	207.0	b,c:0.717 0.611 d:0.544	0.0 1.0 120.0	
16	0.015	1441.0	207.0	e:0.563 0.536	4.0	
17	828	1713.0	205.0	b,c:0.811 0.775 d:0.178	1.0 1.0 272.0	
18	0.014 0.115 0.0286	1726.0 1765.0 1770.0	221.0	0.516 0.523 0.544	13.0 5.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

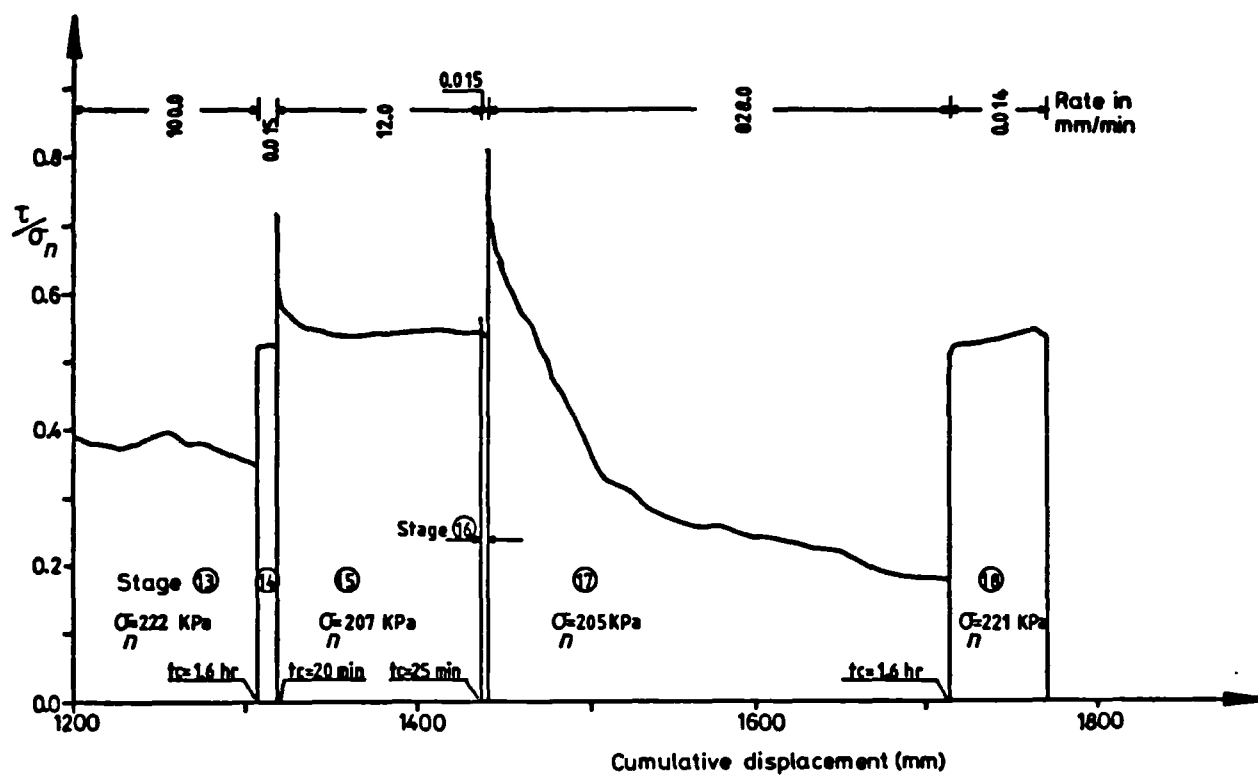


Fig. A2.16b Ring shear test 15 on clayey siltstone C

Table A2.16: Results from ring shear test 16 on sandstone

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.01	12.1	427.8	peak:0.683 0.680	8.5 12.1	
2	10	198.5	423.8 417.9	b,c:0.690 d:0.650	0.0 87.9-185.4	(1)
3	0.01	224.6	417.2	e:0.650	4.1-26.1	
4	100	411.0	417.6 417.6	b,c:0.780 d:0.700	1.5 123.4-186.4	
5	0.01	436.6	419.7	e:0.670	0.9-25.6	
6	400	607.8	423.5 423.5	b,c:0.840 d:0.660	0.9 171.2	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
(1) : residual conditions had not been established previously

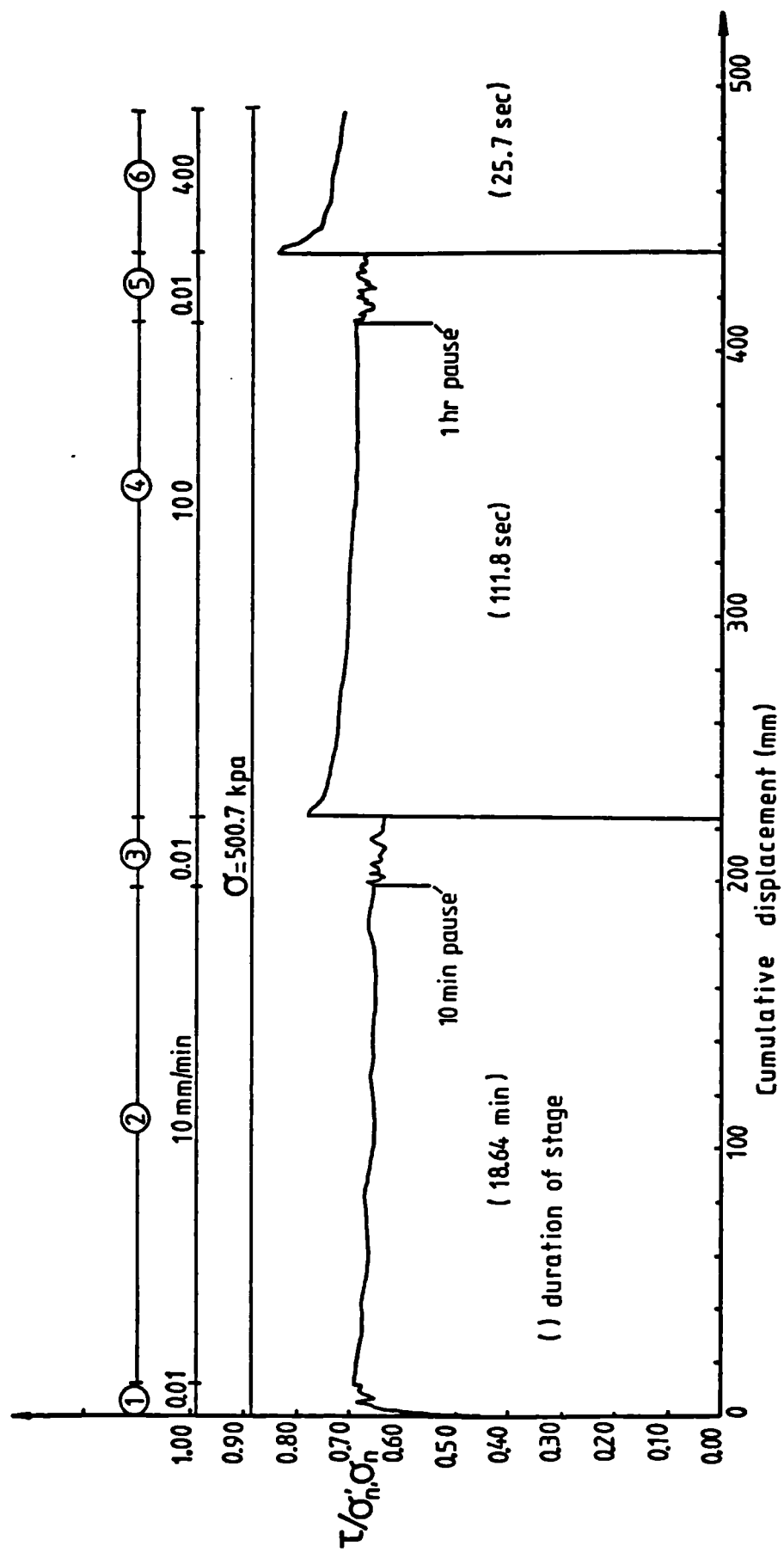


Fig. A2.17a Ring shear test 16 on sandstone

Table A2.16 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
7	0.01	657.6	469.9	e:0.650	1.5-49.8	
8	2000	1015.3	458.2	b,c:0.860 0.690 d:0.660	1.3 180.0 357.8	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

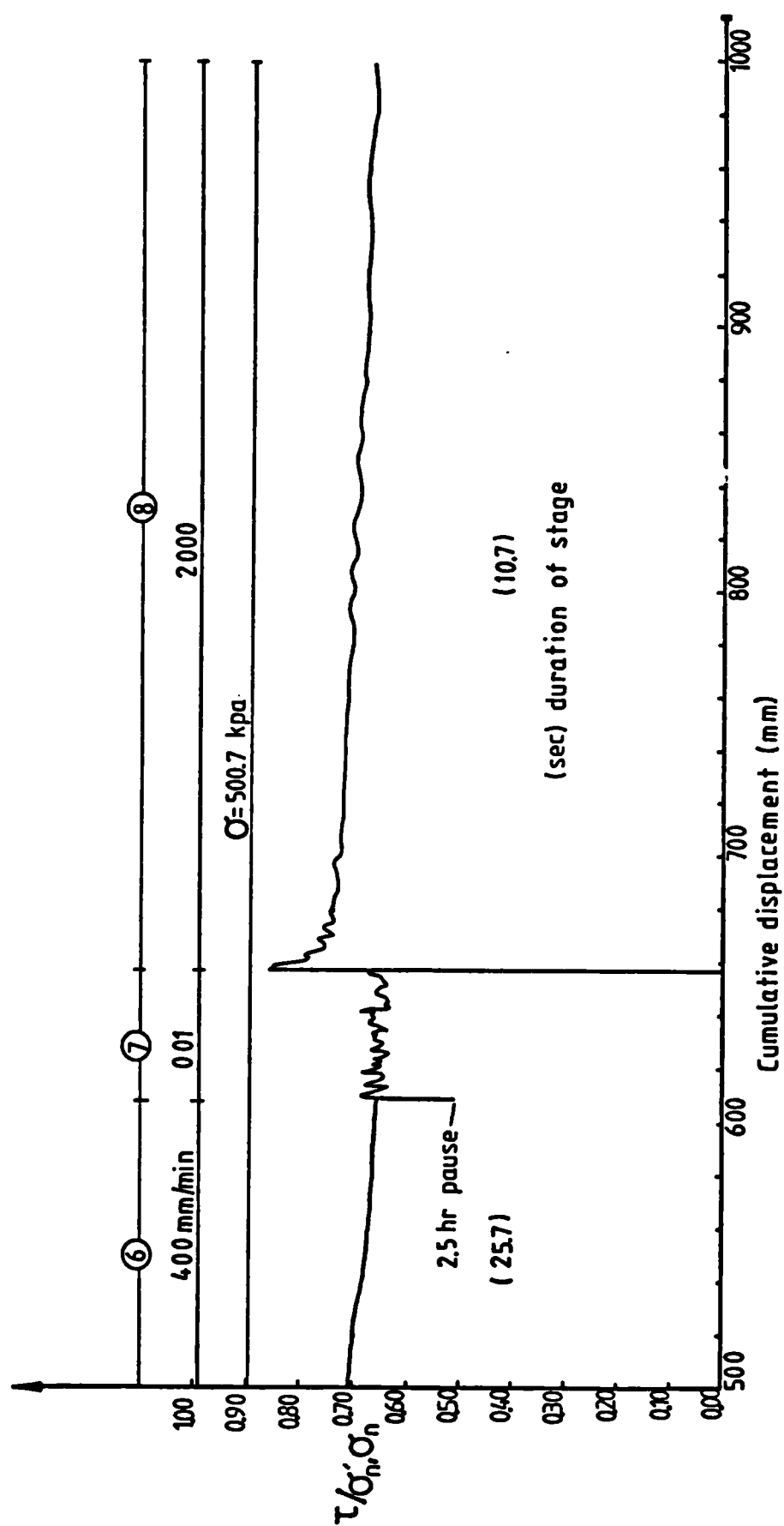


Fig. A2.17b Ring shear test 16 on sandstone

Table A2.16 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
9	0.01	1063.5	405.9	e:0.650	0.8-50.0	
10	0.01	1131.5	161.2	peak:0.850	0.2	
11	100	1319.2	157.4	0.680 b,c:0.930	22.2-68.0 0.7	
12	0.01	1359.4	224.3	d:0.760 e:0.710	138.5-187.7 4.9	
13	400	1584.5	230.6	0.660 b,c:0.940 d:0.650	11.7-40.2 0.7 180.0 225.1	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

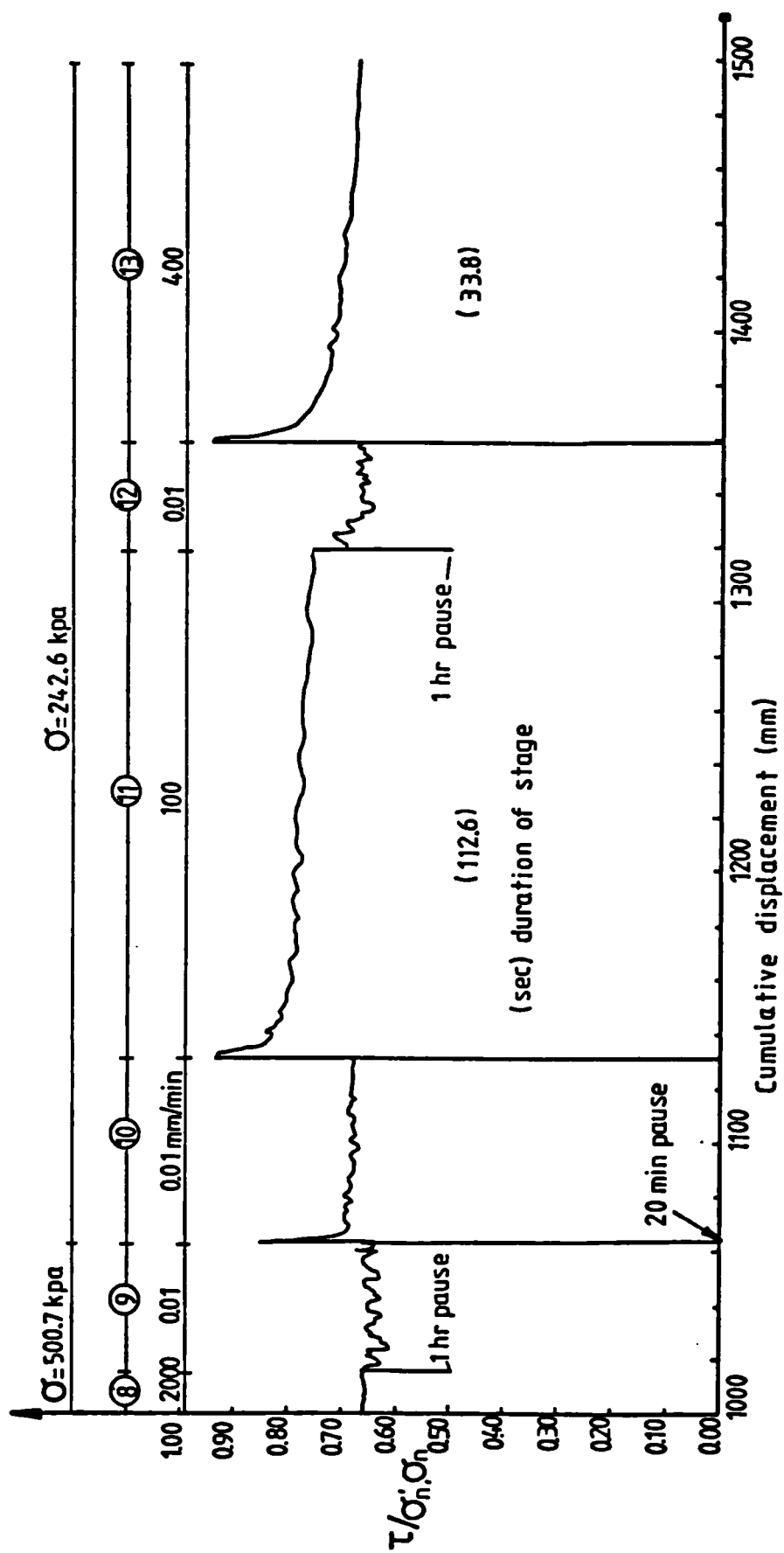


Fig. A2.17c Ring shear test 16 on sandstone

Table A2.16 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
14	0.01	1637.9	219.5	e:0.660	1.6-53.4	
15	2000	1944.7	220.6	b,c:0.990 0.718	0.4 180.0	
16	0.01	2022.7	207.6	d:0.700 e:0.710 0.660	306.8 0.6 2.1-77.9	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

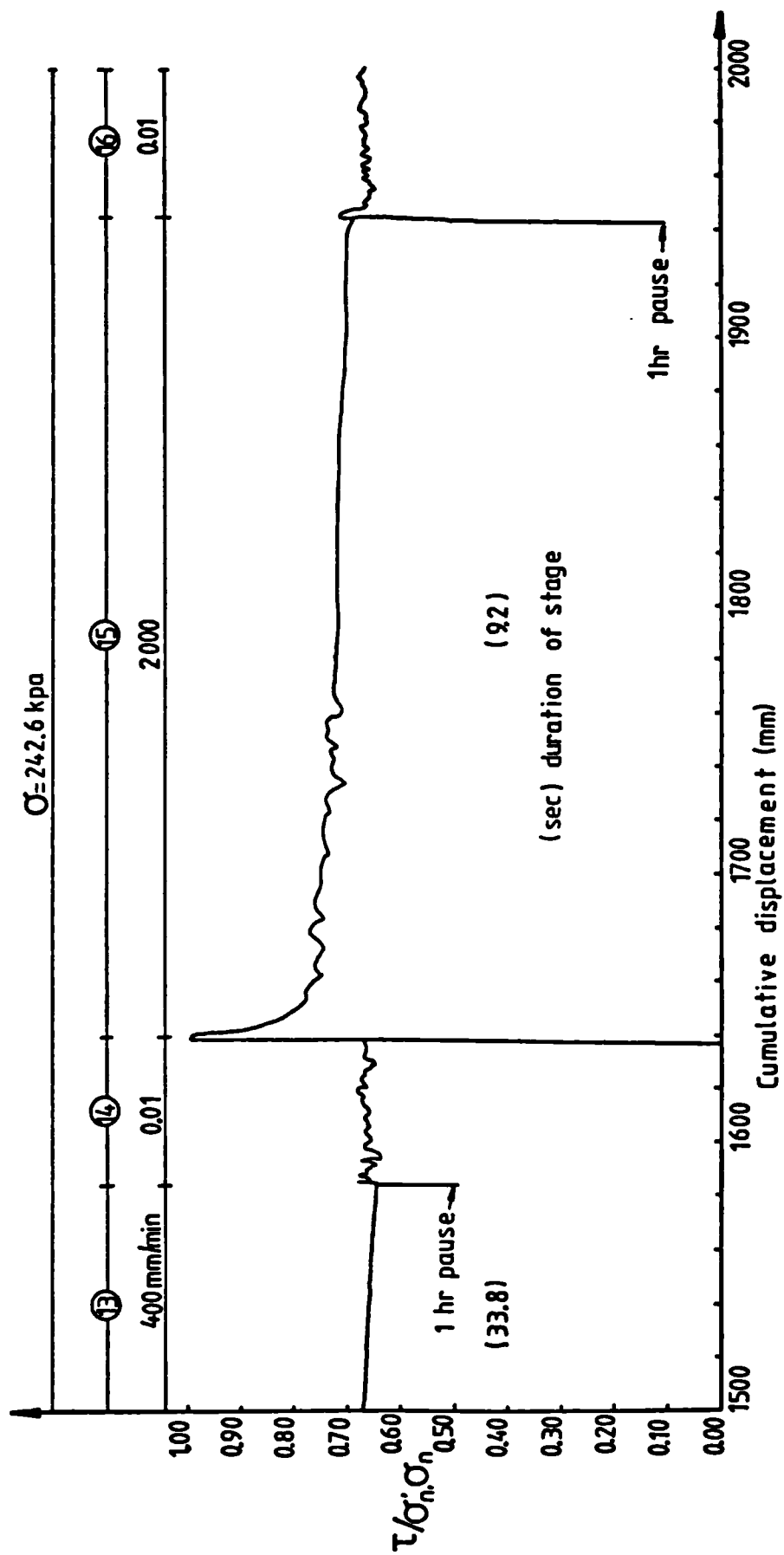


Fig. A2.17d Ring shear test 16 on sandstone

Table A2.16 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
17	5400	2374.5	187.3	no recording was performed		
18	5400	2770.4	187.3	b, c: 0.950 0.740 0.660	0.9 180.0 395.9	
19	0.01	2783.2	187.3 235.1	e: 0.710 0.650	0.6 10.0-12.8	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

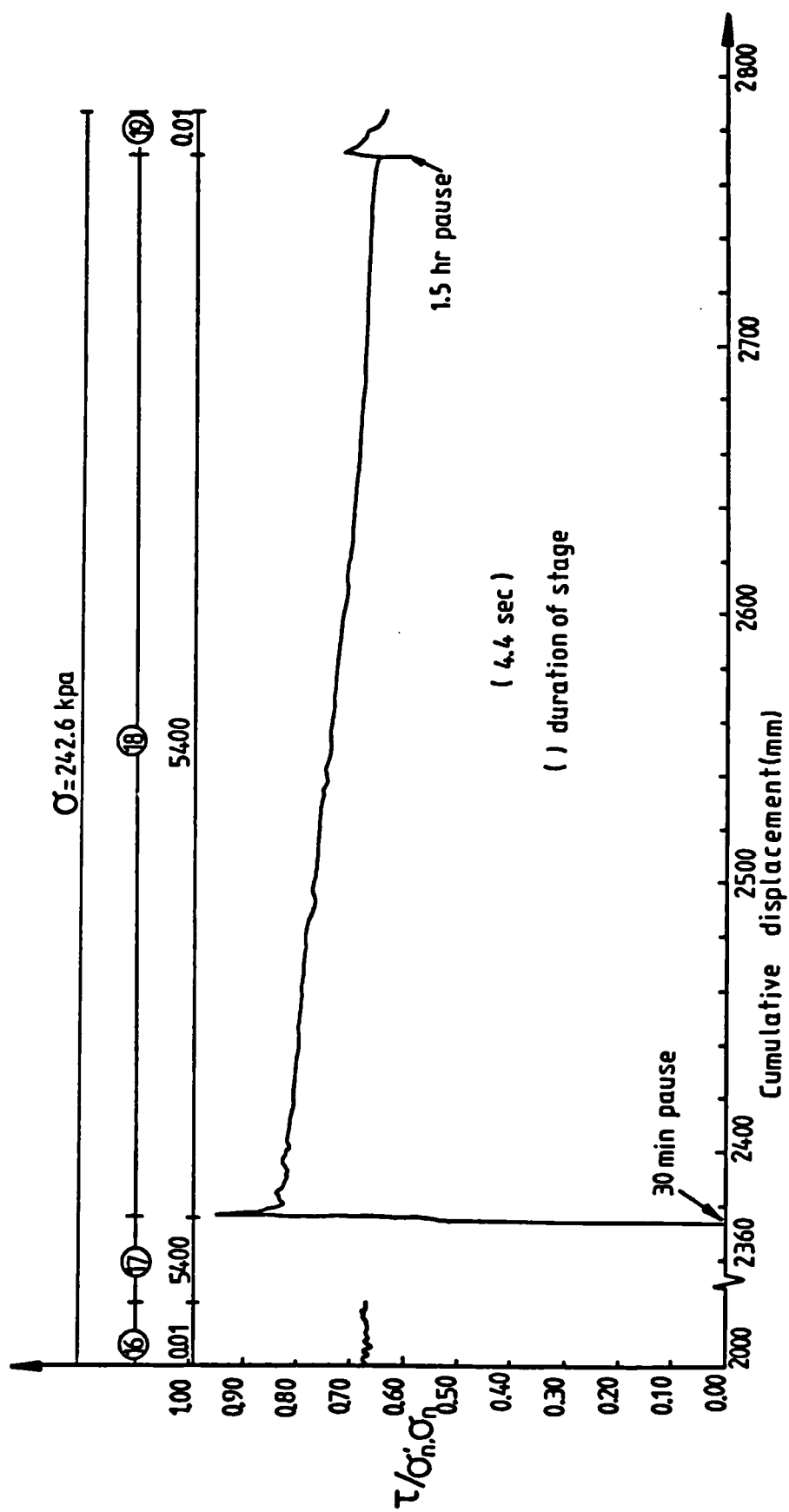


Fig. A2.17e Ring shear test 16 on sandstone

Table A2.17 Results from ring shear test 17 on clayey siltstone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.01	143.9	504.8	peak:0.500 0.310	2.6 98.8	
2	95	323.9	495.1	b:0.360 c:0.391 d:0.220-0.260	0.5 8.9 111.0-179.9	
3	0.01	403.3	506.0	e:0.367 0.311	0.9 63.5-79.4	
4	400	1188.4	447.6	b:0.367 c:0.454 d:0.050-0.150	2.0 17.3 146.7-785.1	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

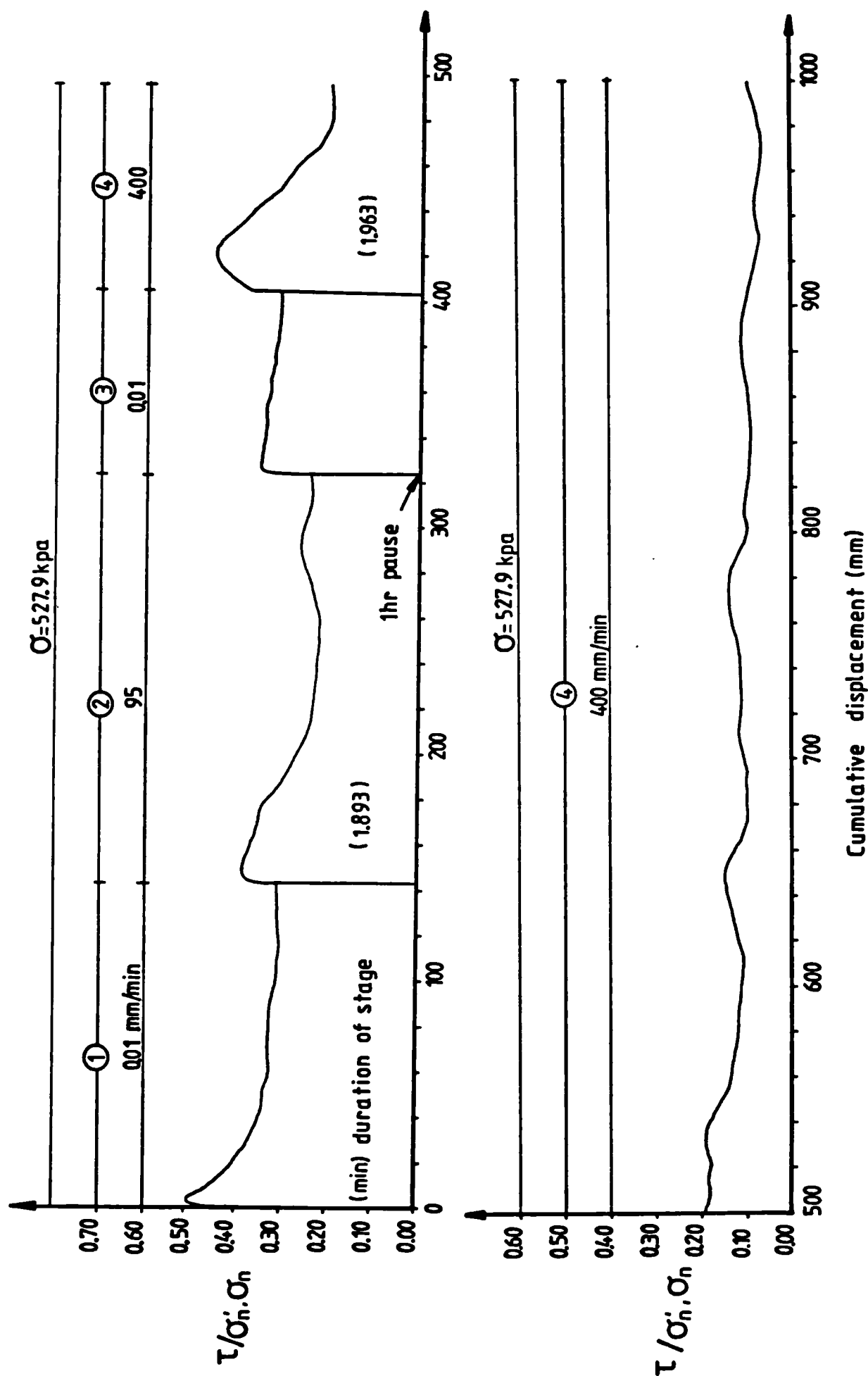


Fig. A2.18a Ring shear test 17 on clayey siltstone D

Table A2.17 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
5	0.01	1278.3	503.8	e:0.344 0.295	2.0 33.5-89.9	
6	2000	1502.1	581.8	b:0.433 c:0.527 d:0.140 0.210	0.3 1.7 125.9 181.7-223.8	0.450 0.560 0.138 0.210
7	0.01	1641.2	507.6	e:0.415 0.315	1.8 67.9-139.1	
8	0.01	1960.5	243.0	peak:0.374 0.338	0.01 12.1	
9	95	2144.0	194.3	accident with the proving ring		

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
 (*) Values of stress ratio corrected for the friction in the gap at $\sigma = 527.9$ kPa

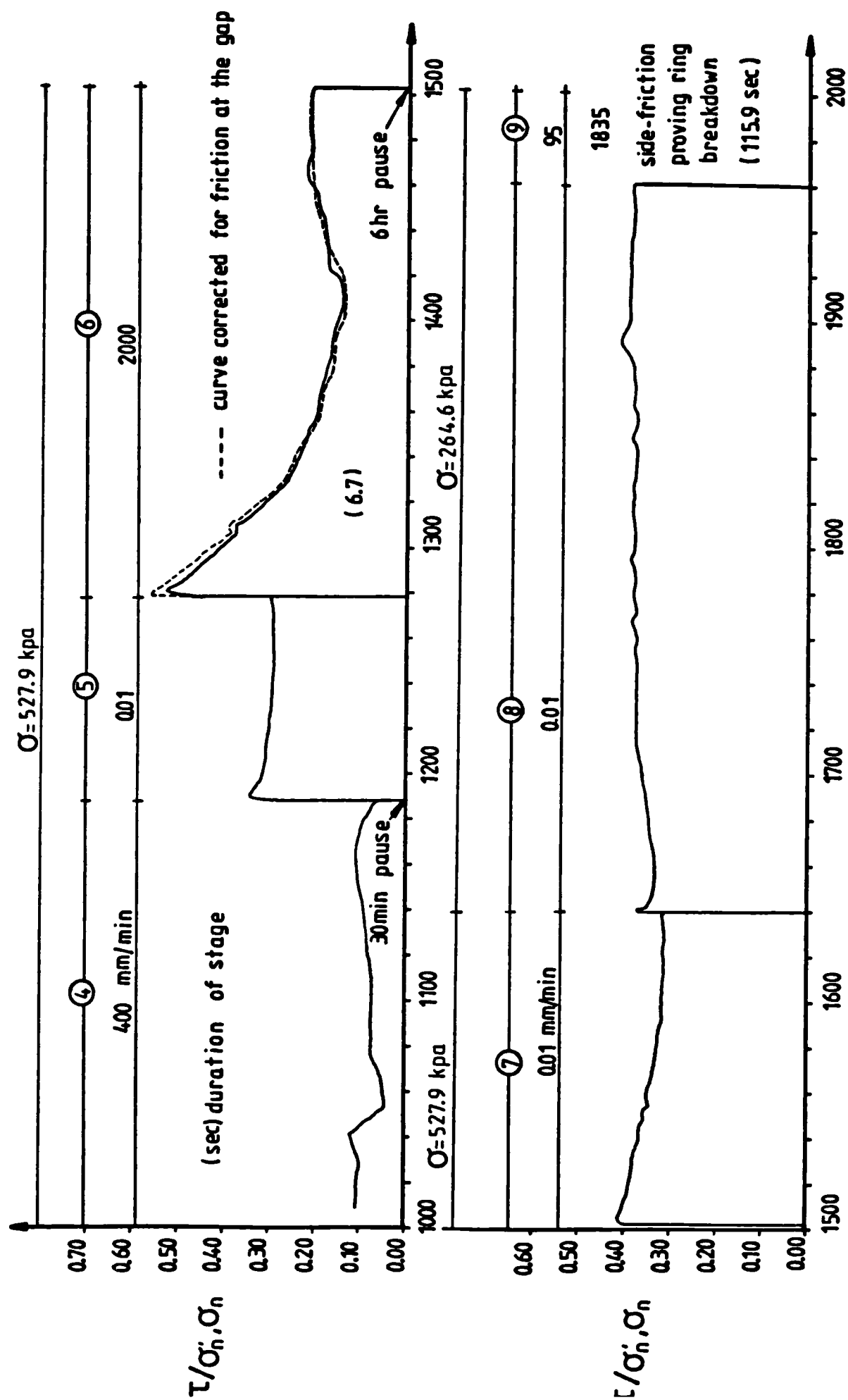
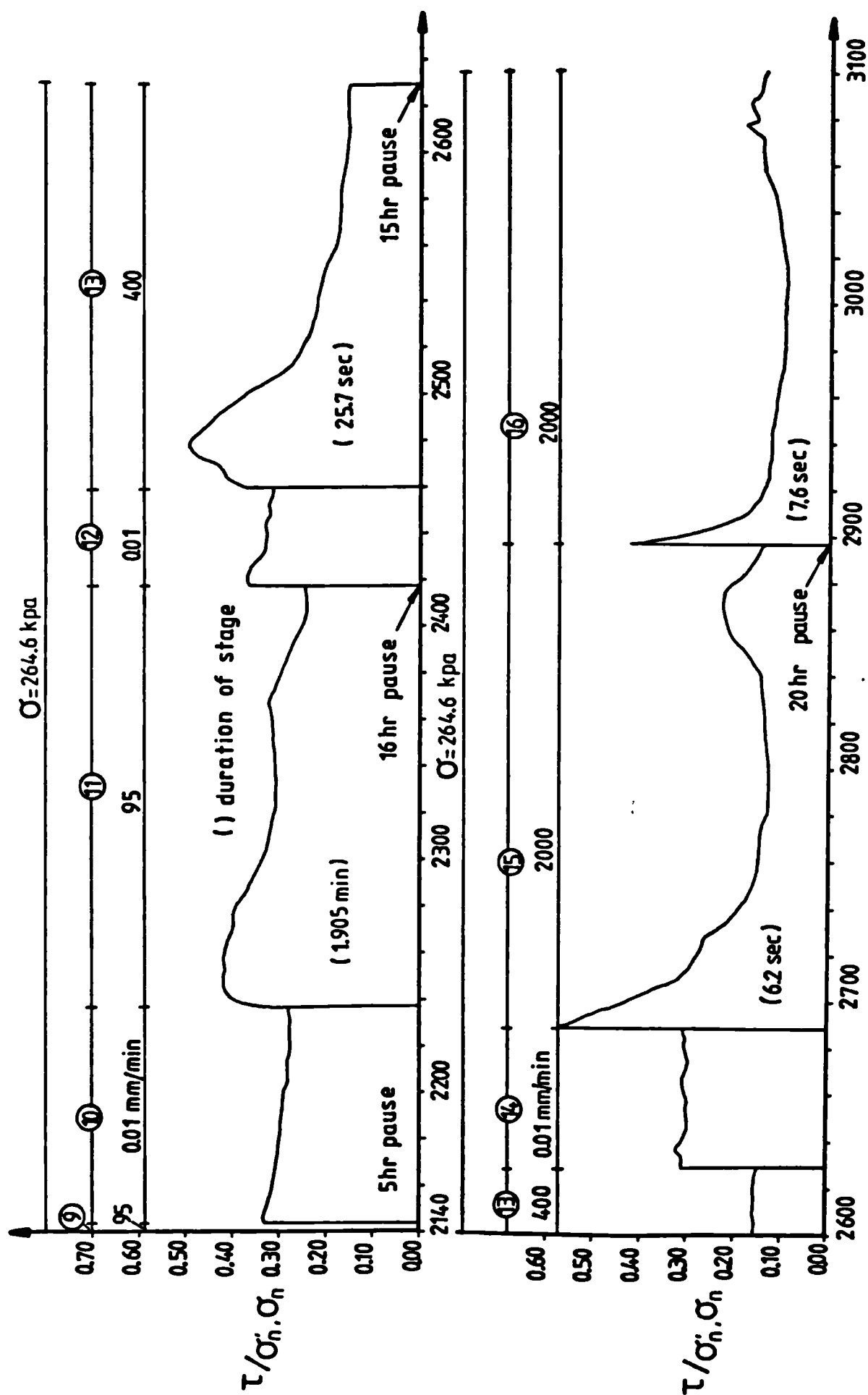


Fig. A2.18b Ring shear test 17 on clayey siltstone D

Table A2.17 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
10	0.01	2237.0	262.8	e: 0.336 0.285	1.0 58.0-262.8	
11	95	2418.0	225.5	b: 0.334 c: 0.422 d: 0.240	0.3 7.1 181.0	
12	0.01	2460.6	260.5	e: 0.370 0.315	0.80 39.0-42.6	
13	400	2630.0	212.2	b: 0.363 c: 0.449 d: 0.150	1.3 18.1 169.4	
14	0.01	2689.9	260.9	e: 0.318 0.306	2.1 49.1-59.9	
15	2000	2897.4	259.3	b, c: 0.570 d: 0.126 0.225	0.30 94.0 207.4	
16	2000	3152.5	259.3	b, c: 0.426 d: 0.098 0.159	0.0 79.5-121.0 255.1	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing



Cumulative displacement (mm)

Fig. A2.18c Ring shear test 17 on clayey siltstone D

Table A2.17 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
17	2000	3387.5	259.3	b,c:0.428 0.099	0.0 158.5	
18	0.01	3415.7	260.0	e:0.324	0.5-28.2	
19	5400.00	3632.1	244.3	b,c:0.543 d:0.080 0.220	0.0 120.0-155.0 194.0-216.4	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

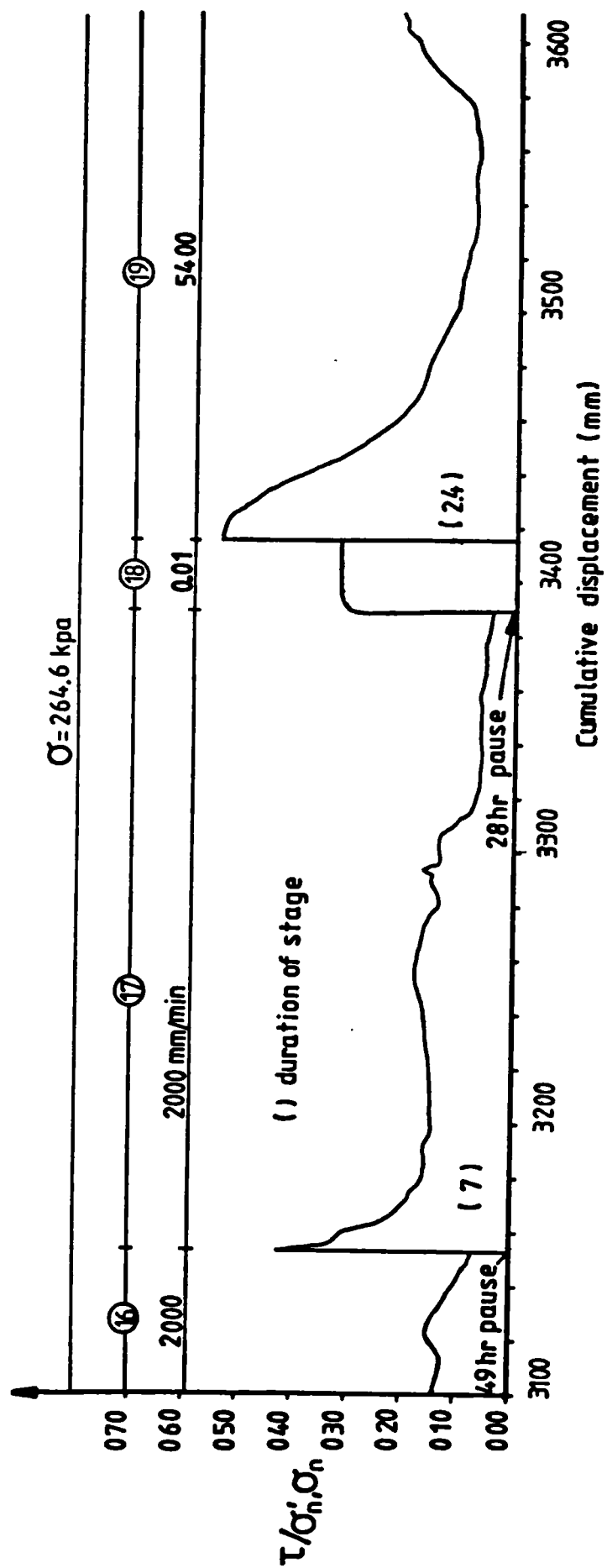


Fig. A2.18d Ring shear test 17 on clayey siltstone D

Table A2.18 Results from ring shear test 18 on clayey siltstone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	370	195.1	232.7 236.3 261.2	b:1.010 c:1.060 0.265	1.2 1.8 195.1	
2	0.0053	221.7	215.9	0.382	21.6-26.6	(1)

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
 (1) the peak stress ratio was not recorded

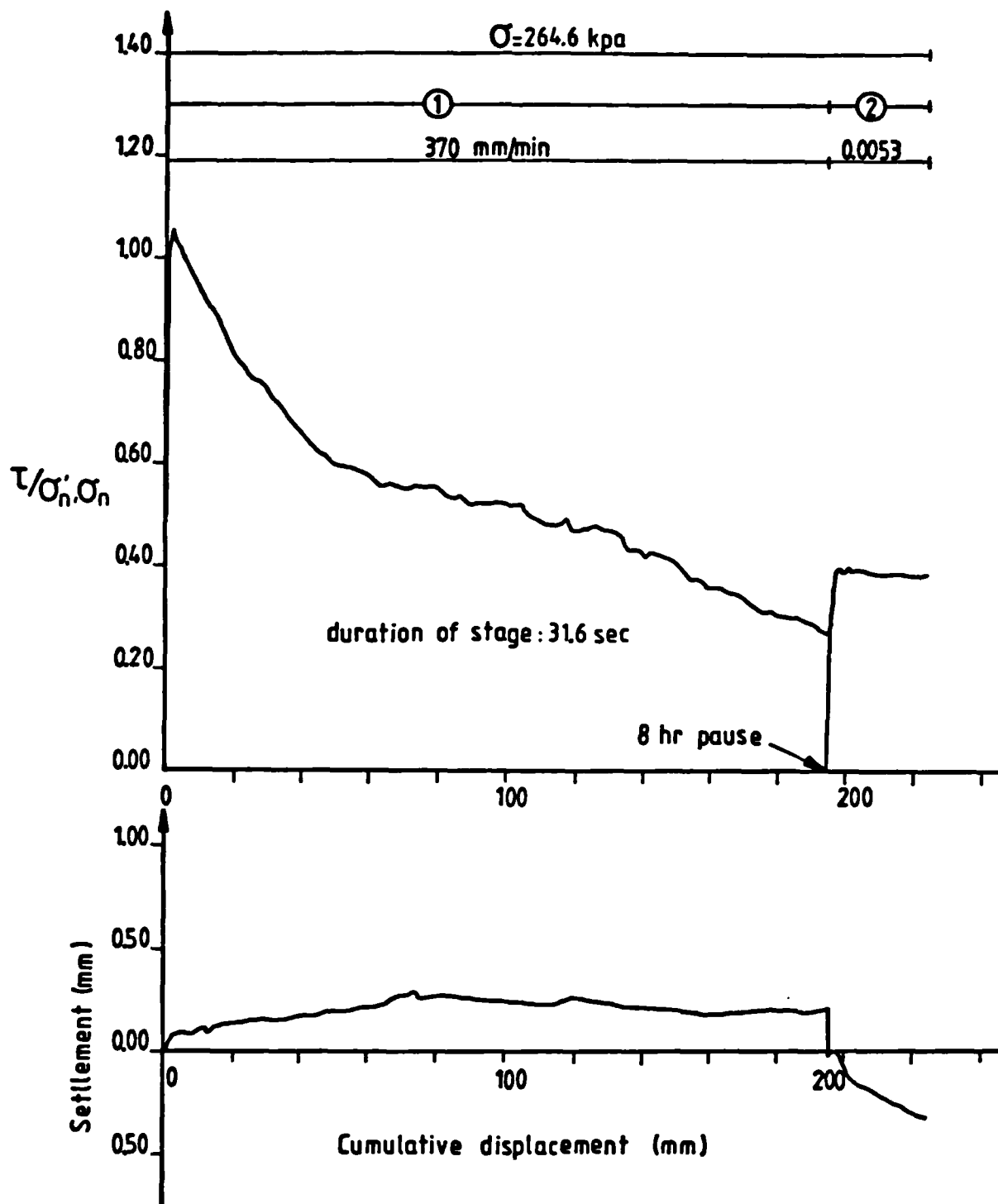


Fig. A2.19 Ring shear test 18 on clayey siltstone D

Table A2.19 Results from ring shear test 19 on clayey siltstone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	12	235.4	229.1	b,c:0.931 d:0.345 0.376	1.3 182.2 233.6	
2	0.014	532.1	225.0	e:0.376	1.2-2.4 278.2-296.7	
3	400	922.7	242.0	b:0.357 c:0.476 0.150 d:0.085	1.5 12.9 180.4 330.4	
4	0.343	941.8	227.0	0.307 0.311	14.0 15.9-19.1	
5	0.014	994.2	228.5	e:0.331 0.307	1.0 52.4	
6	0.095	1167.5	233.3	c:0.311 d:0.278 0.283	0.1-0.2 152.3 172.3-177.3	
7	0.014	1223.1	239.5	e:0.295 0.292	12.7 51.9-55.6	
8	1.2	1377.4	241.4	b:0.283 c:0.305 d:0.270	0.3 2.8-7.7 70.1-154.3	
9	0.014	1436.8	237.0	e:0.285	56.1-59.4	
10	11	1630.0	240.7	b:0.324 c:0.350 d:0.326 0.340	0.8 14.7-15.0 75.7-101.0 193.2	
11	0.014	1661.5	240.0	e:0.332 0.301	1.7 31.5	
12	400	1669.9	251.0	b:0.342 c:0.512	0.7 7.5	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

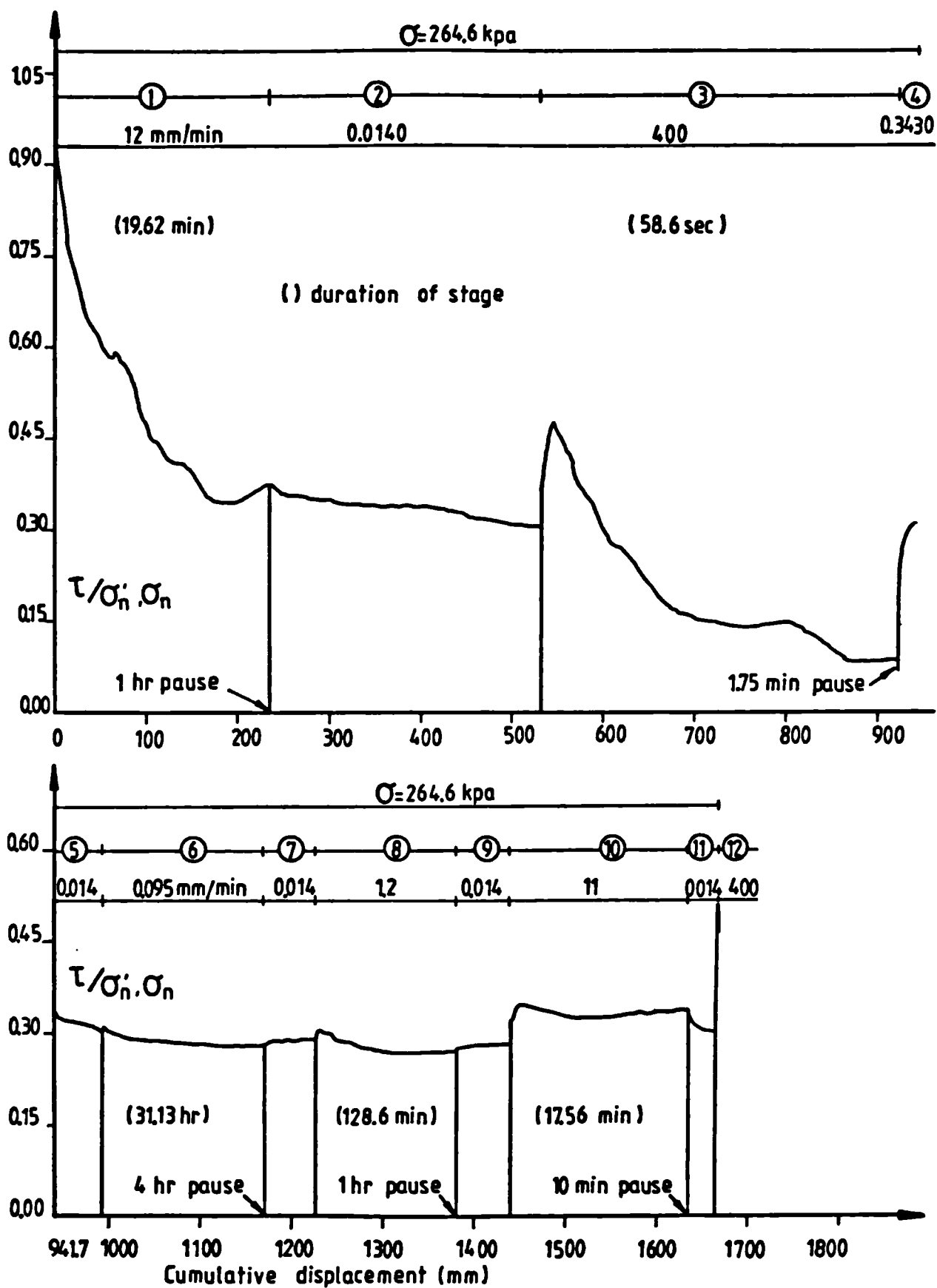


Fig. A2.20 Ring shear test 19 on clayey siltstone D

Table A2.20 Results from ring shear test 20 on clayey siltstone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	100	213.7	213.3 226.9	b,c:1.098 0.333	2.5 213.7	
2	0.01	346.1	213.6 213.7	e:0.400 0.331	2.1 118.3	
3	400	1148.2	221.0 221.0 239.2 228.4 242.1 237.2	b:0.396 c:0.482 0.236 0.345 d:0.203 0.325	3.5 16.1 156.3 282.9 352.9 802.1	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

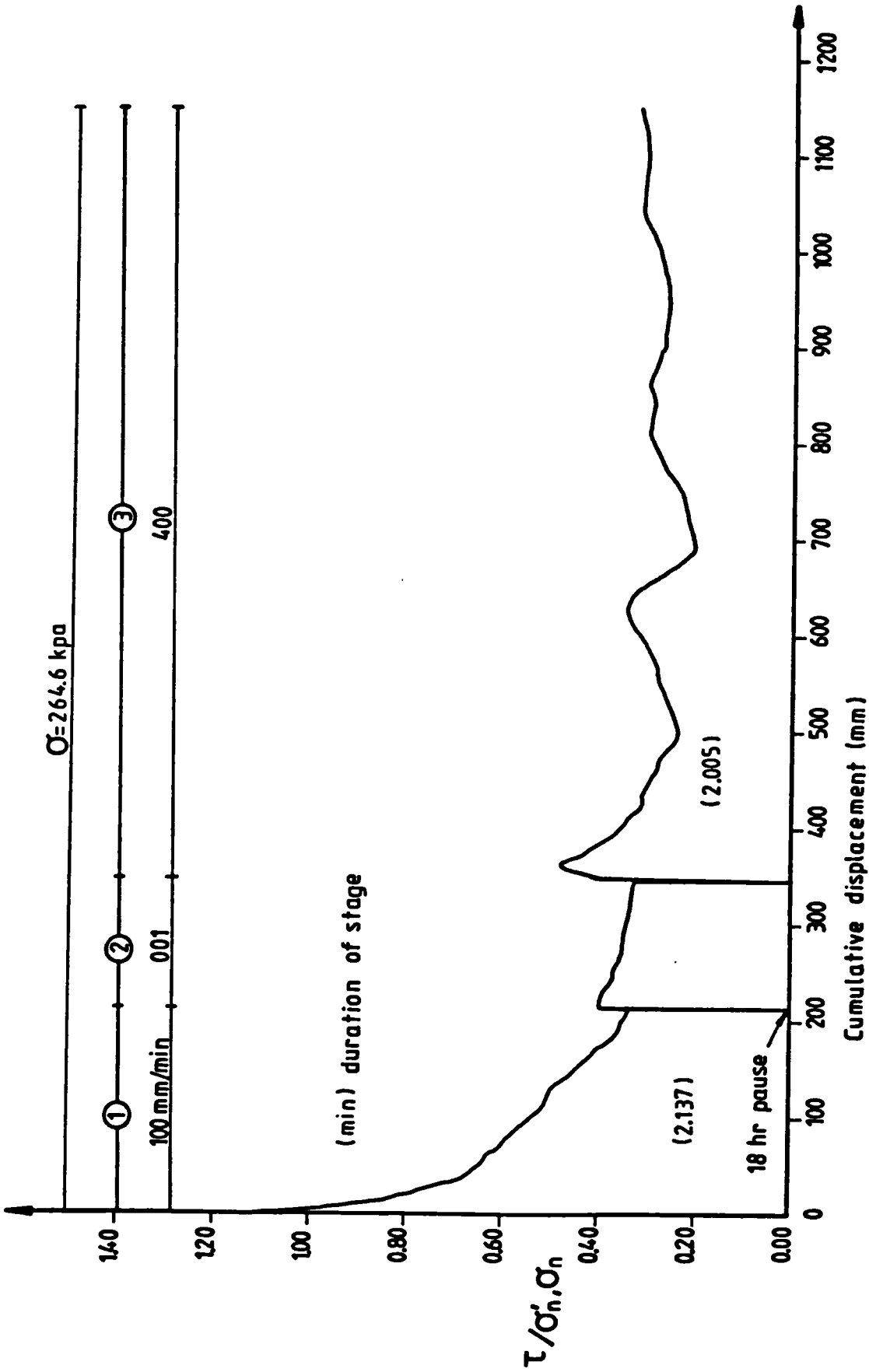


Fig. A2.21 Ring shear test 20 on clayey siltstone D

Table A2.21 Results from ring shear test 21 on clayey siltstone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	400	194.9	240.0	b:0.322 0.423 0.512	1.8 10.1 192.1	
2	0.01	285.9	236.4	e:0.451 0.354	2.9 88.9	
3	12	487.5	222.4 228.5 237.2	b:0.410 c:0.450 d:0.442	0.4 115.5-146.9 176.2-201.6	
4	0.01	569.1	235.7 237.7	e:0.383 0.333	0.3 78.1	
5	100	759.4	239.5	b:0.409 c:0.475 d:0.260 0.312	0.7 7.0-8.6 117.0 190.3	
6	100	1426.4	475.0 475.0 477.0	0.110 0.195 0.182	0.0 583.6 667.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

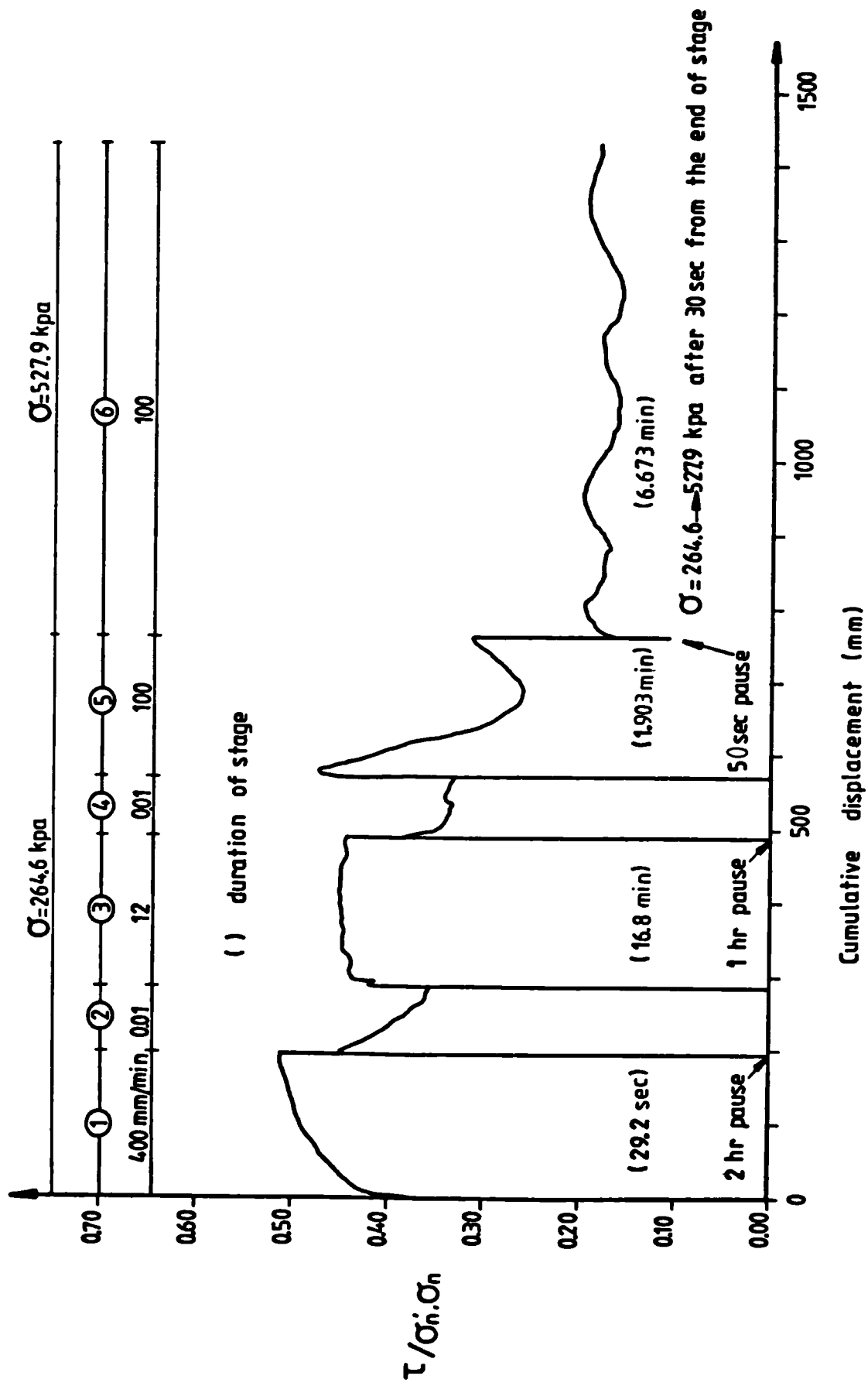


Fig. A2.22 Ring shear test 21 on clayey siltstone D

Table A2.22 Results from ring shear test 22 on claystone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.01	208.5	463.5	PEAK:0.494 0.188	2.2 55.7	
2	19.5	214.0		no recording was performed		
3	9.5	406.0	509.5 501.4	b,c:0.200 0.080	2.70 119.9-192.0	
4	0.01	472.2	489.3	e:0.177 0.160	3.6 33.0-66.2	
5	9.5	664.4	495.2 507.3 514.9	b,c:0.174 d:0.144 0.180	0.2 69.2-75.5 124.6-192.2	
6	0.01	693.8	496.9 494.0	e:0.190 0.182	3.7 29.4	
7	95	871.0	496.6 494.7	b:0.202 c:0.240	0.3 2.0-2.7	
8	0.01	917.6	491.6 486.0 498.3	d:0.120 e:0.146 0.158	92.1-177.2 0.3 40.4-46.6	
9	360	1085.7	497.7	b:0.184 c:0.326 d:0.283	1.0 18.0 103.9-168.1	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

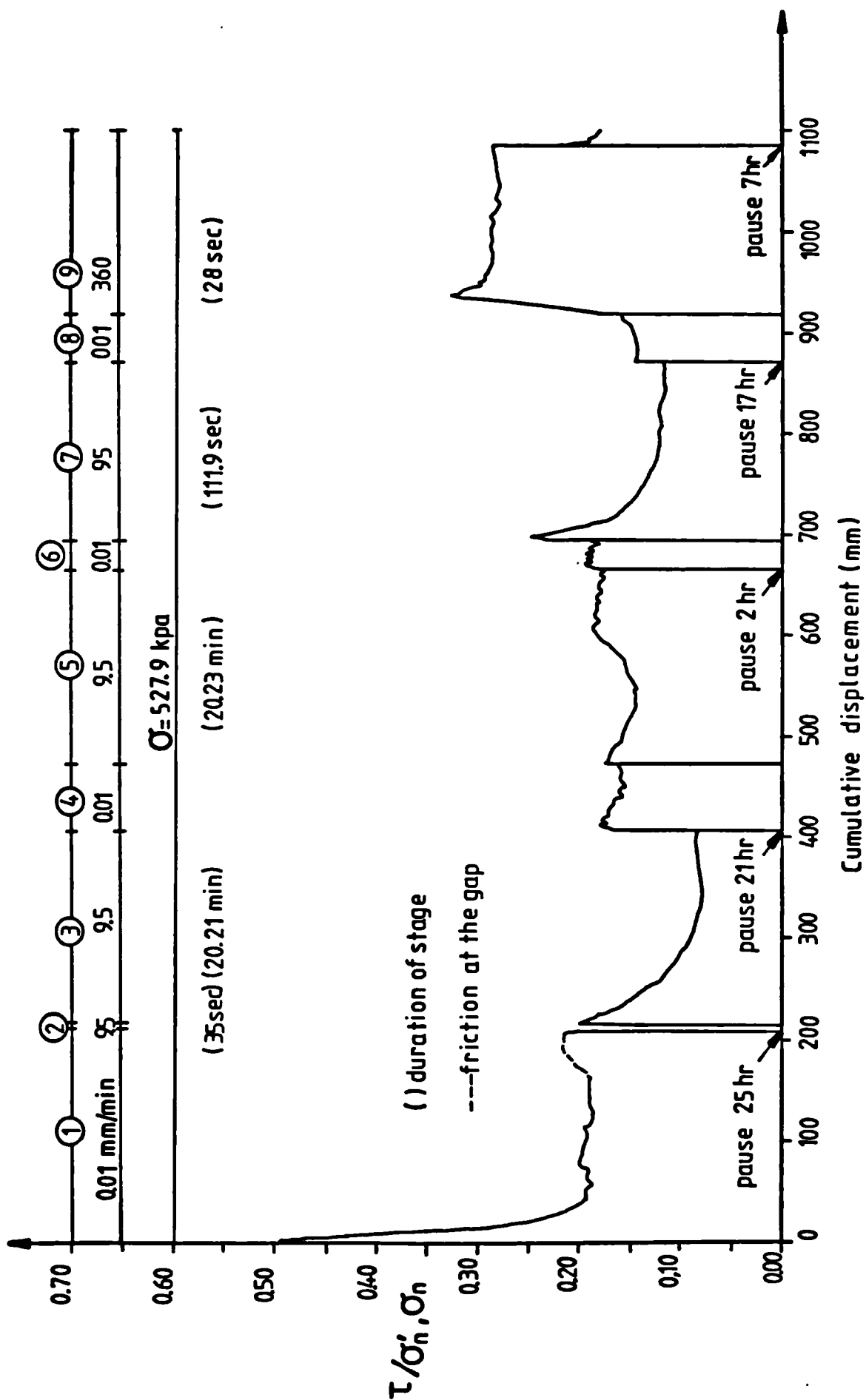


Fig. A2.23a Ring shear test 22 on claystone D

Table A2.22 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
10	0.01	1161.8	499.2 491.2	e:0.237 0.165	0.30 73.1-76.1	
11	2600	1445.1	490.9 490.9 501.6 505.0 505.0	b:0.326 c:0.423 0.229 0.255 0.139	1.3 3.2 111.0 169.7-180.6 283.3	
12	0.01	1466.9	491.9 485.6	e:0.262 0.191	0.4 17.0-20.8	
13	0.01	1574.8	230.3 239.2	peak:0.195 0.150	0.4 103.7-108.9	
14	9.5	1615.2	238.4 240.5	b,c:0.168 0.122	0.09 40.4	
15	9.5	1810.0	239.2 240.2 246.8	b,c:0.156 d:0.124 0.170	0.25 33.7 167.3-194.8	
16	0.01	1884.9	235.5	e:0.174	74.9	
17	95	2063.9	235.5 235.5	b:0.197 c:0.210 d:0.153	0.3 2.1 109.3-179.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

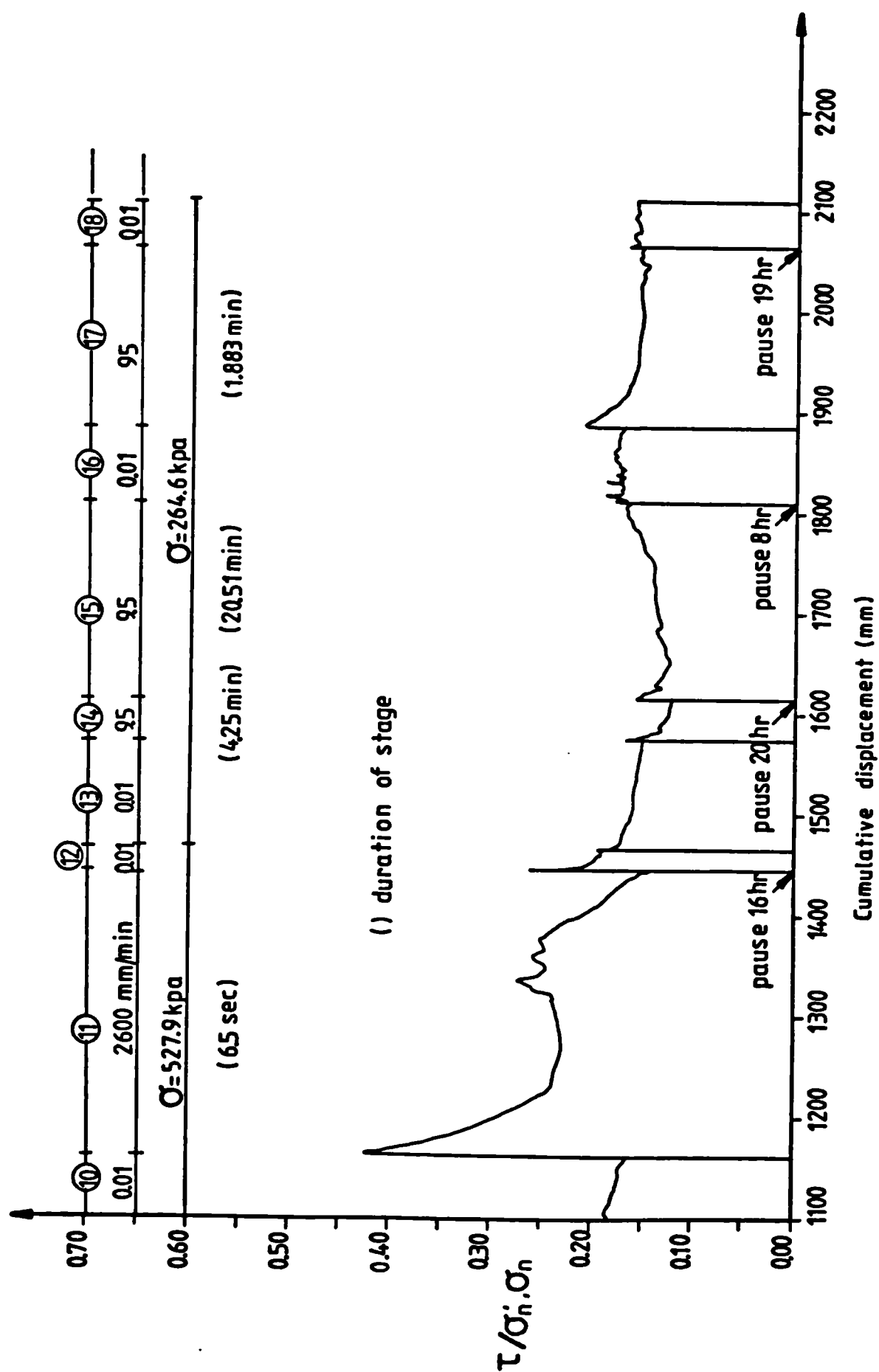


Fig. A2.23b Ring shear test 22 on claystone D.

Table A2.22 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
18	0.01	2109.1	230.1 232.4	e:0.168 0.160	0.30 3.7-45.2	
19	2600	2450.8	234.9 237.4 235.0	b,c:0.585 0.130 0.050	3.2 181.0 341.7	
20	2600	2783.0	237.0 237.4 237.5	b,c:0.273 0.100 d:0.072-0.117	180.0 203-332.2	
21	0.01	2814.2	239.2 235.8	e:0.274 0.191	0.4 293-312	
22	0.01	2905.4	108.8 113.8	peak:0.230 0.186	0.2 29.7	
23	9.5	3095.0	110.8 113.4 116.9	b,c:0.210 d:0.143 0.176	0.06 87.84.9 166.6-189.6	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

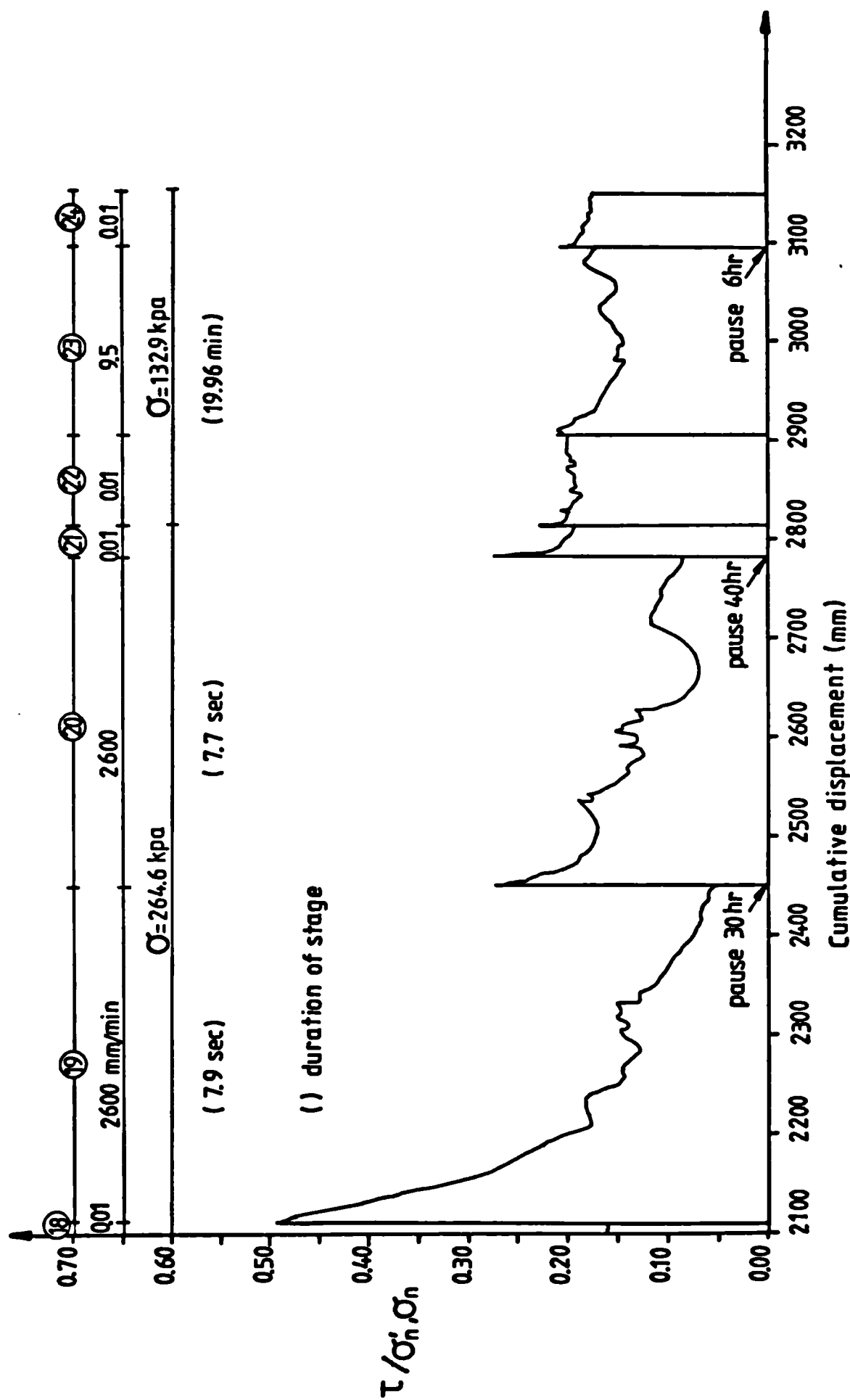


Fig. A2.23c Ring shear test 22 on claystone D

Table A2.22 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
24	0.01	3142.0	117.7 112.9	e:0.205 0.171	0.2 47.0	
25	95.0	3326.5	112.4 111.7 111.70	b:0.214 c:0.217 d:0.190	0.09 8.8 95.6-184.5	
26	0.01	3368.7	110.5 113.6	e:0.220 0.200	3.1 5.2-42.2	
27	360	3529.8	111.1 111.3 115.8 117.8 116.3	b:0.240 c:0.387 0.236 0.257 d:0.189	0.5 16.5 79.9 130.7-140.3 161.1	
28	0.01	3573.0	112.8 112.9	e:0.298 0.210	1.0 31.0-43.2	
29	2600	3886.1	116.0 121.6	b,c:0.540 d:0.146	0.9 94.4-313.1	
30	0.01	3918.6	115.7 111.8	e:0.317 0.211	0.3 32.5	
31	0.01	4082.0	73.9 76.7	peak:0.228 0.188	0.46 21.8	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

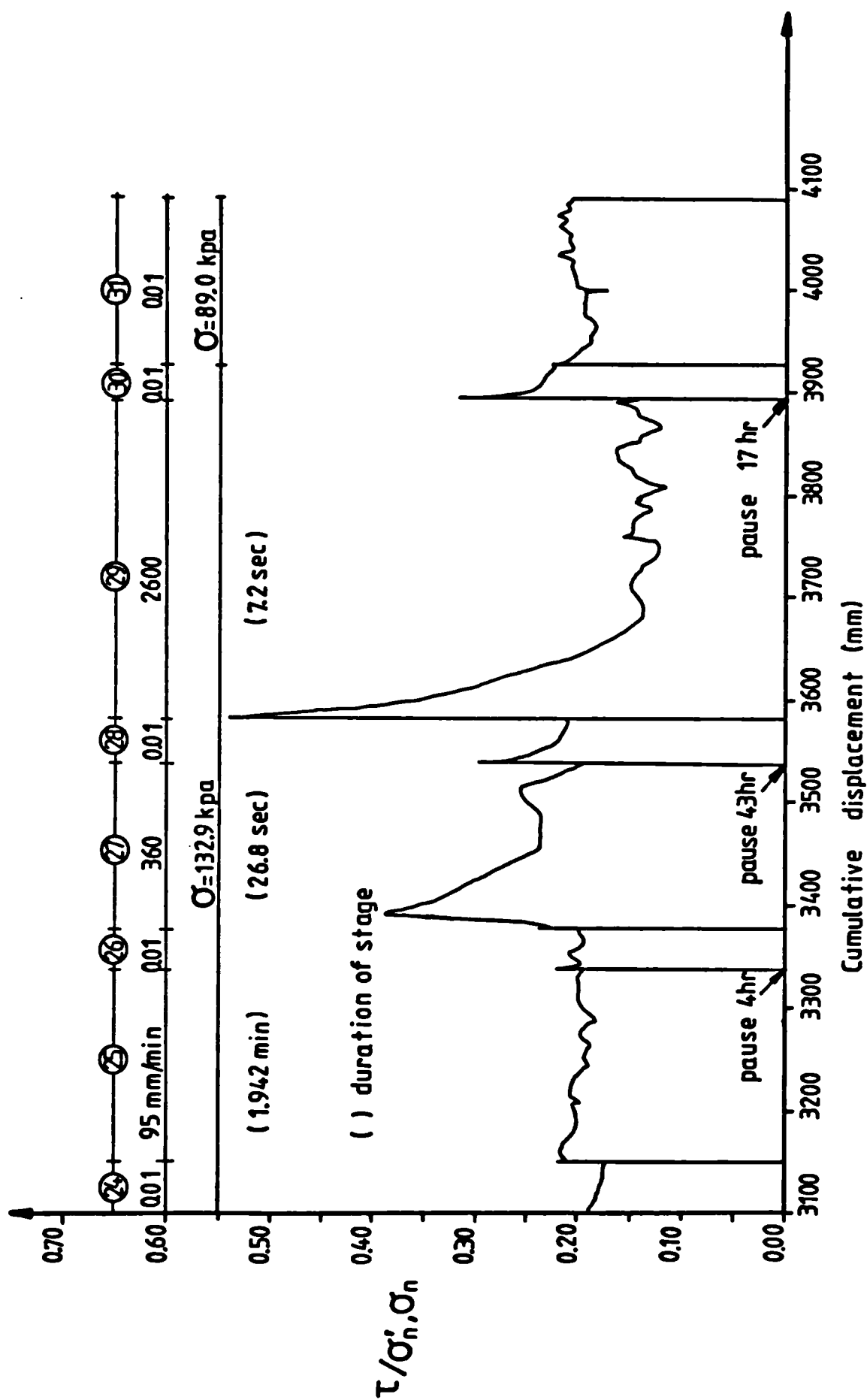


Fig. A2.23d Ring shear test 22 on claystone D

Table A2.22 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
32	9.5	4274.6	63.0 66.2 68.7	b,c:0.214 d:0.155 0.178	0.05-0.10 126.5 192.6	
33	0.01	4346.0	65.3 62.0	e:0.193 0.172	0.30 57.5-71.4	
34	95	4528.2	62.2 64.1 68.0	b:0.230 c:0.300 d:0.281	0.1 22.3 46.2-182.2	
35	0.01	4599.6	62.7 59.5	e:0.262 0.192	0.20 71.2	
36	360	4758.1	60.3 60.3 64.5	b:0.234 c:0.432 0.231	0.4 15.6 158.5	
37	0.01	4833.2	58.9 58.5	e:0.310 0.244	0.30 12.4-75.1	
38	1200	4859.1	no recording performed			
39	0.01	4917.2	60.4 59.0	e:0.373 0.200	0.40 58.1	
40	2600	5192.2	61.6 61.6 69.5	b:0.591 c:0.646 d:0.116	0.50 3.50 129.2-275.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

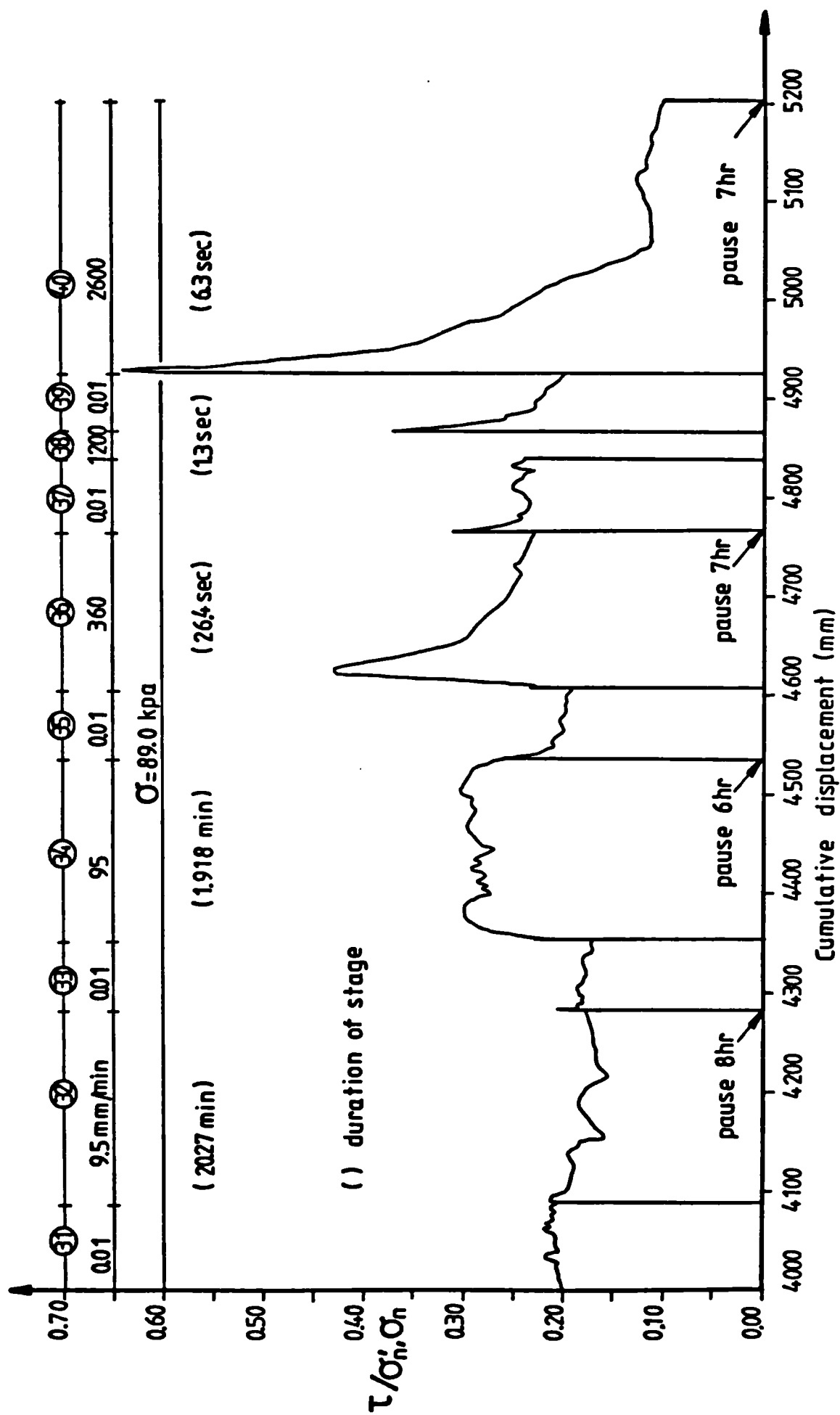


Fig. A2.23e Ring shear test 22 on claystone D

Table A2.22 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
41	0.01	5217.7	59.0	e:0.343	0.5	
42	0.01	5246.7	59.0	0.240	25.5	
43	360	5401.9	223.9	0.199	0.5	
			222.1	0.165	20.6-29.0	
			222.2	b:0.199	0.7	
			222.6	c:0.352	16.8	
			227.0	0.222	144.4-155.2	
44	0.01	5438.4	224.7	e:0.259	0.4	
				0.168	36.5	
45	360.0	5590.2	222.9	b:0.201	0.5	
			222.9	c:0.309	14.5	
			230.0	0.191	151.8	
46	0.01	5629.2	228.4	e:0.260	0.3	
			219.8	0.180	39.0	
47	2600	5846.2	225.0	b:0.439	0.5	
			225.1	c:0.489	2.7	
			232.1	d:0.077	217.0	
48	0.01	5892.5	483.3	0.261	0.7	
			477.6	0.170	40.5-463	
49	9.5	6079.7	475.3	b,c:0.172	0.3	
				d:0.105	60.2-187.2	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

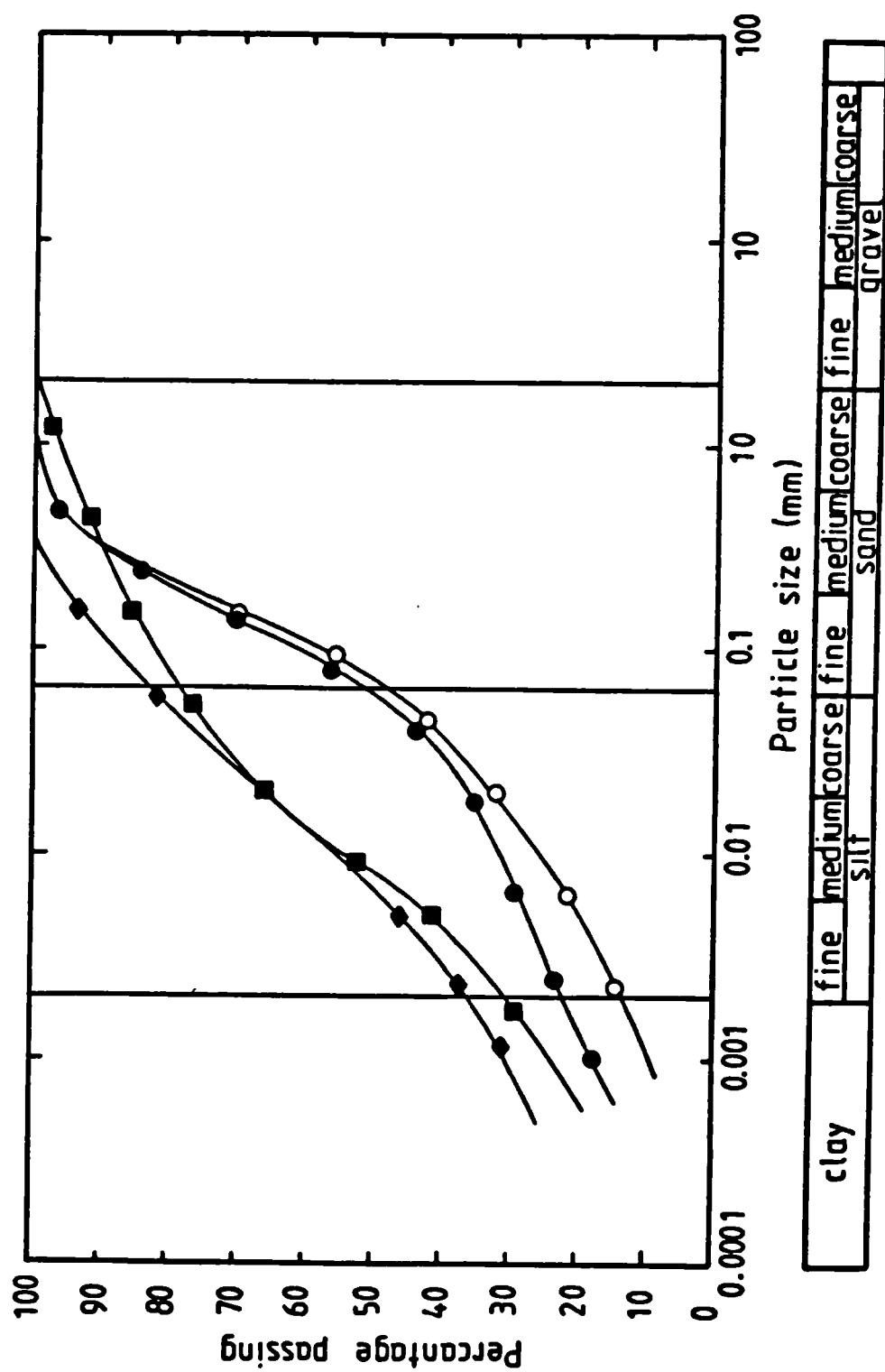


Fig. A2.24 Grading curves of Tills

- Lower Cromer Till
- >> (pretreated soil)
- Weathered Cowden Till
- ◆ Magnus Till

Table A2.23 Results from ring shear test 23 on Lower Cromer Till

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.025	163.9	580 525 496 475	peak:0.545 0.578 0.590 0.595	5.5-7.1 16.5-34.3 36.0-70.0 70.0-164.9	
2	133	556.3	543.9 574.4 561.0	b,c:0.860 0.290 d:0.195	0.9 200.0 383.4-392.4	
3	0.025	559.5	551.6	e:0.551	2.9-3.2	
4	0.025	599.9	412.3 415.0	0.714 0.585	0.4 1.4-40.3	
5	1200	965.3	451.1 466 440	b,c:0.922 0.350 d:0.342	0.6 200.0 303.4-365.4	
6	0.025	1536.9	225.8 241.1 230.0	peak:0.604 0.490 0.518	0.4 15.2 538.4-571.5	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

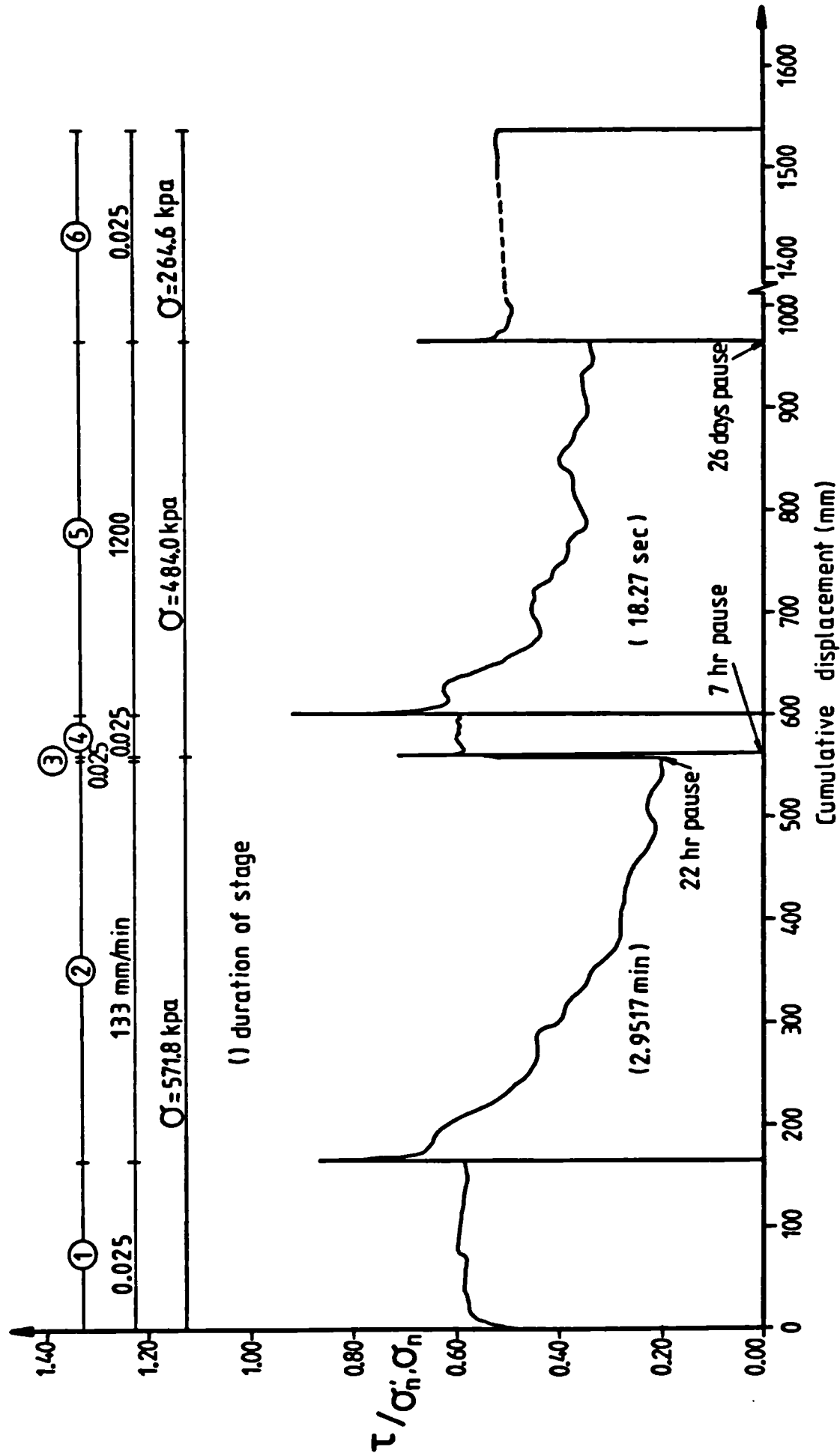


Fig. A2.25a Ring shear test 23 on Lower Cromer Till

Table A2.23 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
7	133	1797.8	233.3 237.6 242.5	b,c:0.705 0.293 d:0.293	0.3 200.0 260.9	
8	0.025	1984.8	230.0	e:0.515	10.0-187.2	
9	133	2246.4	191.7 208.0 216.0	b,c:0.755 0.390 d:0.370	0.3 200.0 261.4	
10	0.1333	2363.9	211.3 210.0 208.7 221.0	0.220 0.462 0.507 0.546	0.0 0.7 2.7 34.5-117.5	
11	0.025	2405.7	220.6 221.0	e:0.567 0.547	0.5 17.4-41.8	
12	1200	2746.3	220.0 229.7 228.2 221.3	b,c:0.894 d:0.357 0.386 0.444	0.3 130.0 200.0 340.6	
13	0.025	2755.5	224.1 225.0	e:0.604 0.565	0.9 1.3-9.2	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

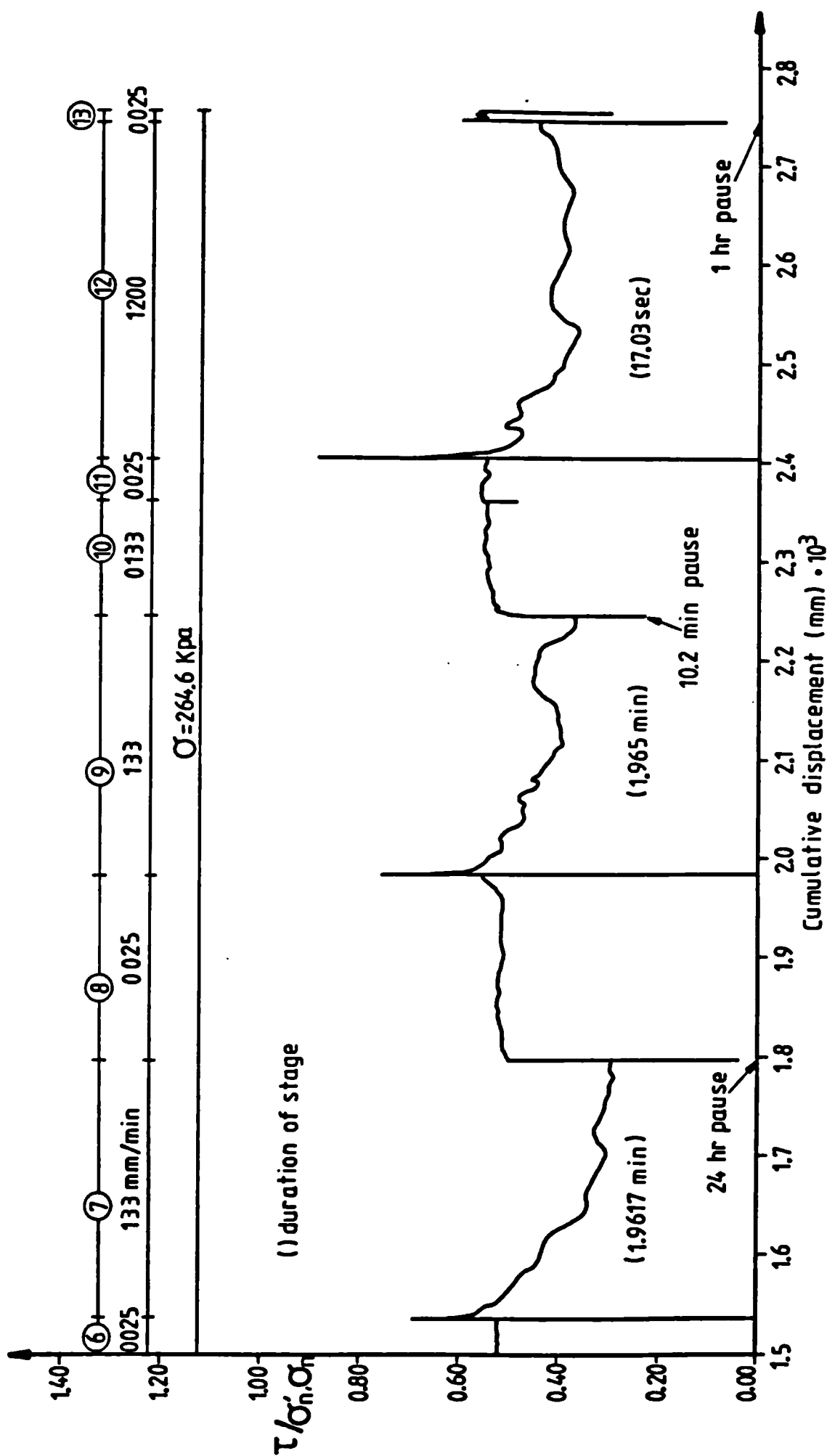


Fig. A2.25b Ring shear test 23 on Lower Cromer Till

Table A2.23 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
14	0.025	2898.2	113.5 118.0	peak:0.877 0.610	0.5 8.2-35.0	
15	133	3177.3	116.0 116.0	b,c:0.837 d:0.368	0.2 159.0-264.0	
16	0.025	3293.9	116.8 118.0 115.0 97.0	e:0.534 0.510 0.535 0.580	0.2 1.4-6.9 27-75.3 76.0-104.5	
17	1200	3638.4	103.6 97.5 102.0	b,c:1.064 0.276 d:0.303	0.0 200.0 344.6	
18	1200	4019.9	96.4 96.4	b,c:0.480 d:0.304±0.024	0.0 75.0-381.5	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

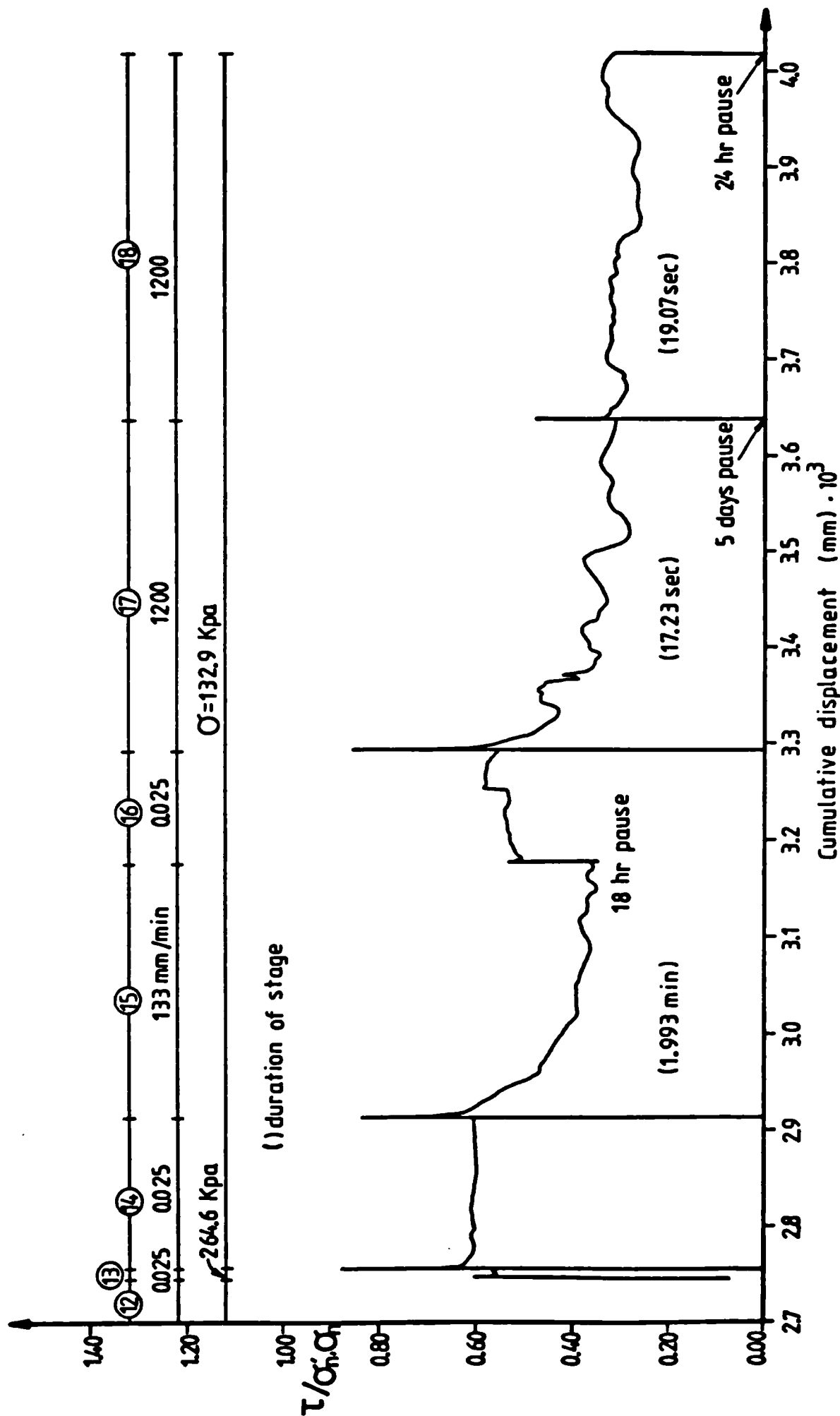


Fig. A2.25c Ring shear test 23 on Lower Cromer Till

Table A2.23 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
19	0.025	4231.8	50.6 55.0	peak:0.746 0.610	0.5 17.5-211.9	
20	133	4244.7		no recording was performed		
21	133	4408.7	47.7 56.2	b,c:1.025 0.353	0.5 152.0-164.0	
22	133	4684.4	48.0 42.1 36.1	b,c:0.694 0.246 d:0.225	0.3 200.0 274.4	
23	0.025	4865.5	46.1 52.0 48.0	e:0.640 0.561 0.605	0.0 1.7-3.2 7.0-181.0	
24	133 9.5	5105.9 1615.2	44.7 65.0 57.8	b,c:0.978 d:0.310 0.343	0.2 200.0 240.4	
25	0.025	5255.3	54.0 52.0	e:0.549 0.575	0.0-6.0 6.0-149.4	
26	1200	5638.2	68.6 58.5 52.5 45.7	b,c:1.248 d:0.381±0.021 0.512 0.426	0.0 80.0-179.3 200.0 382.9	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

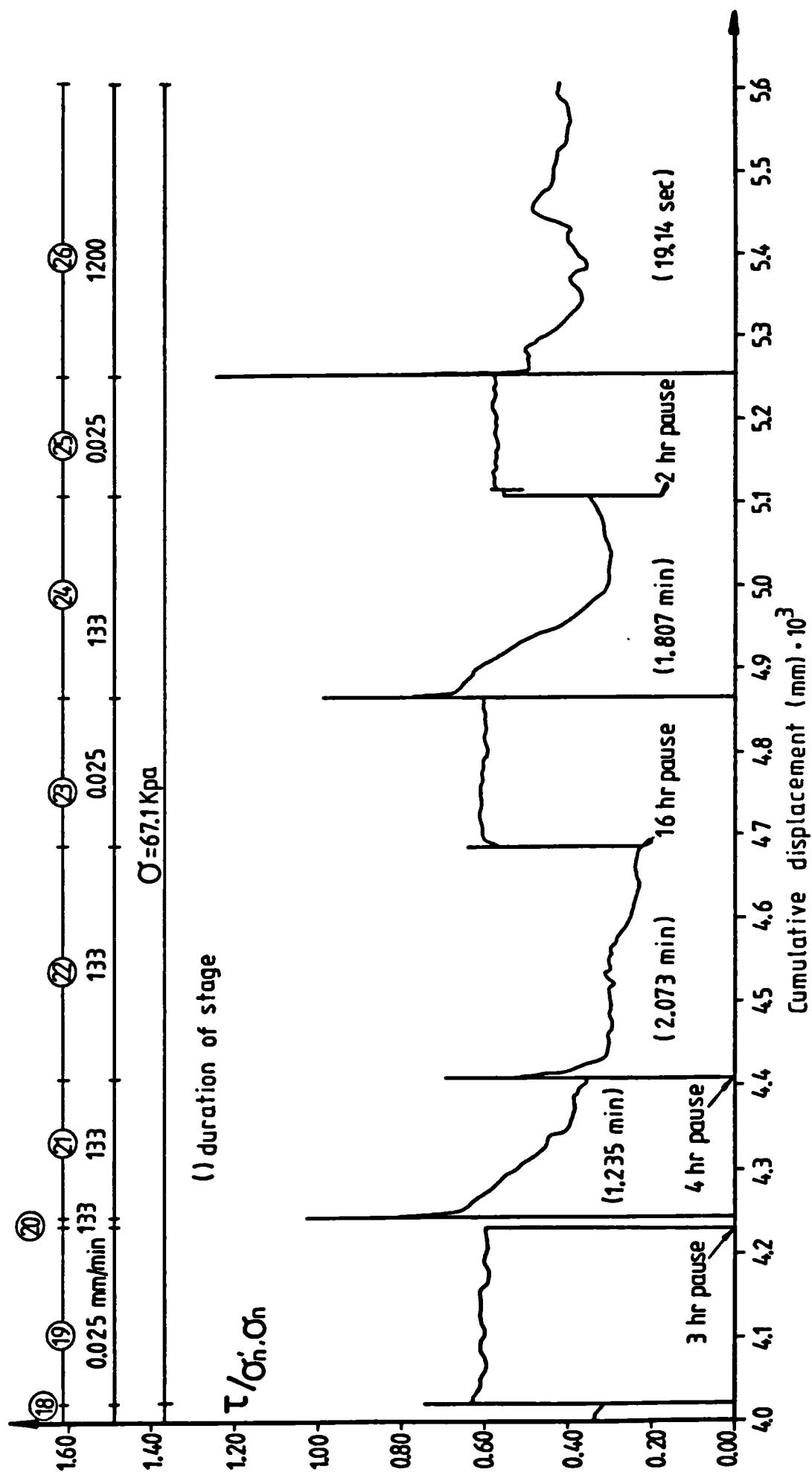


Fig. A2.25d Ring shear test 23 on Lower Cromer Till

Table A2.24 Results from ring shear test 24 on Magnus Till

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0169	25.0	497.0	peak:0.570 0.550 0.520		
2	0.058 131	440.0 616.0	497.0	b,c:0.710 d:0.620	20.0	
3	0.017 0.086	656.0 812.0	497.0	e:0.500 0.510	0.0-40.0 40.0-156.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

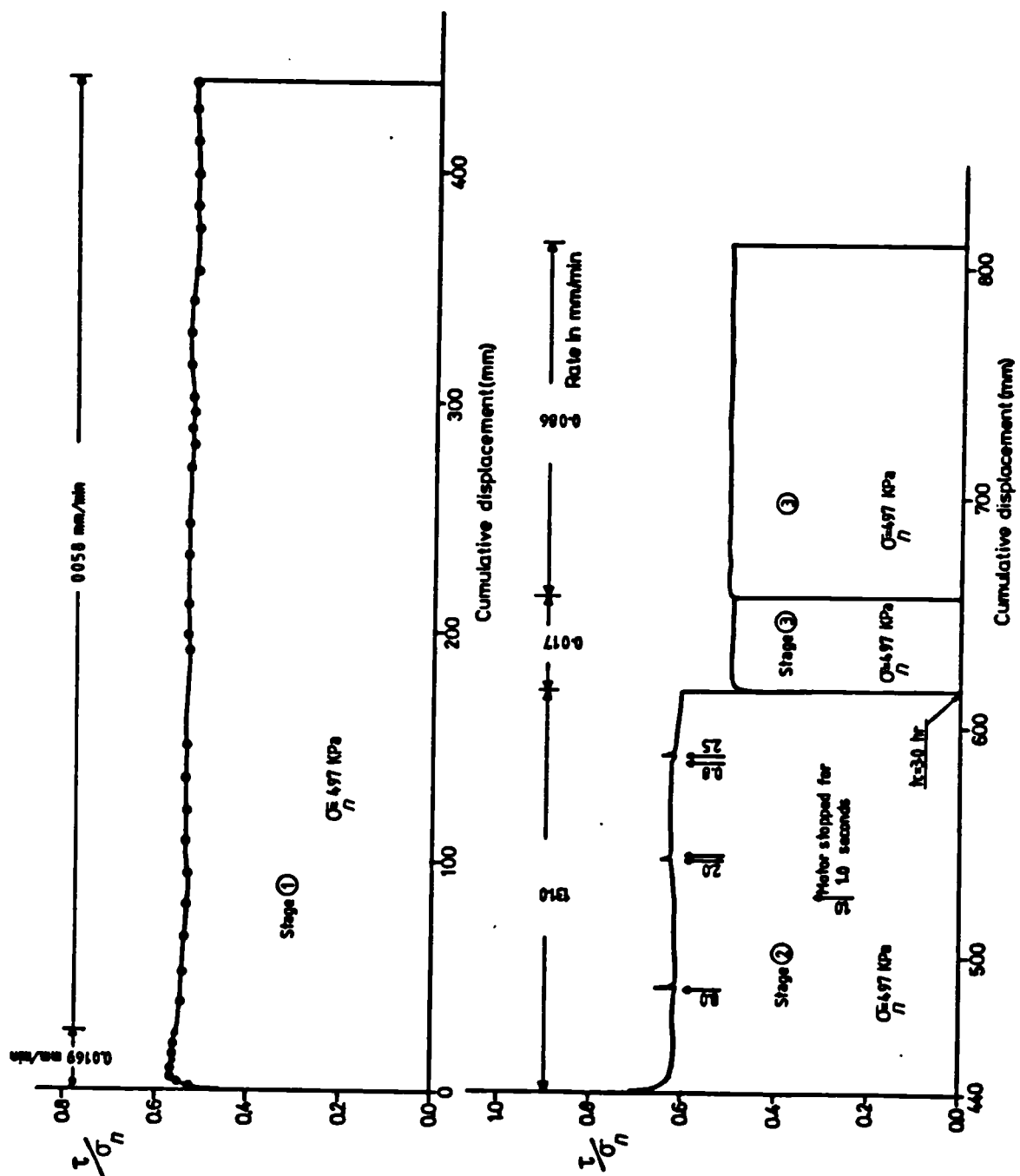


Fig. A2.26 Ring shear test 24 on Magnus Till

Table A2.25 Results from ring shear test 25 on weathered Cowden Till

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0035	135.0	50.0	peak:0.750 0.540		
2	0.0600	157.0	50.0	0.520		
3	0.0050	177.0	50.0	0.576		
4	0.1000	189.0	50.0	0.546		
5	1	198.0	50.0	0.578		
6	0.0006	203.0	50.0	0.634		
7	0.0035	290.0	100.0	0.517		
8	0.1000	302.0	100.0	0.502		
9	1	310.0	100.0	c:0.580 d:0.520 e:0.590 0.550		
10	0.0001	316.0	100.0			

c: fast peak, d: fast residual, e: slow peak after fast shearing

Table A2.25 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
11	0.005	407.0	150.0	c:0.498 0.490		
12	0.1	423.0	150.0	c:0.600 d:0.483		
13	1	428.0	150.0	c:0.566 d:0.507		
14	10	444.0	150.0	b,c:0.604 d:0.585		
15	100	468.0	150.0	b,c:0.732 0.712		
16	0.005	555.0	150.0	e:0.507 0.480		
17	0.05	562.0	150.0	c:0.490 0.468	7.0	Reverse
	0.5	594.0	150.0	0.463		
18	0.05	601.0	150.0	e:0.533 0.531		Forward
	0.50	642.0	150.0	0.480		

c: fast peak, d: fast residual, e: slow peak after fast shearing

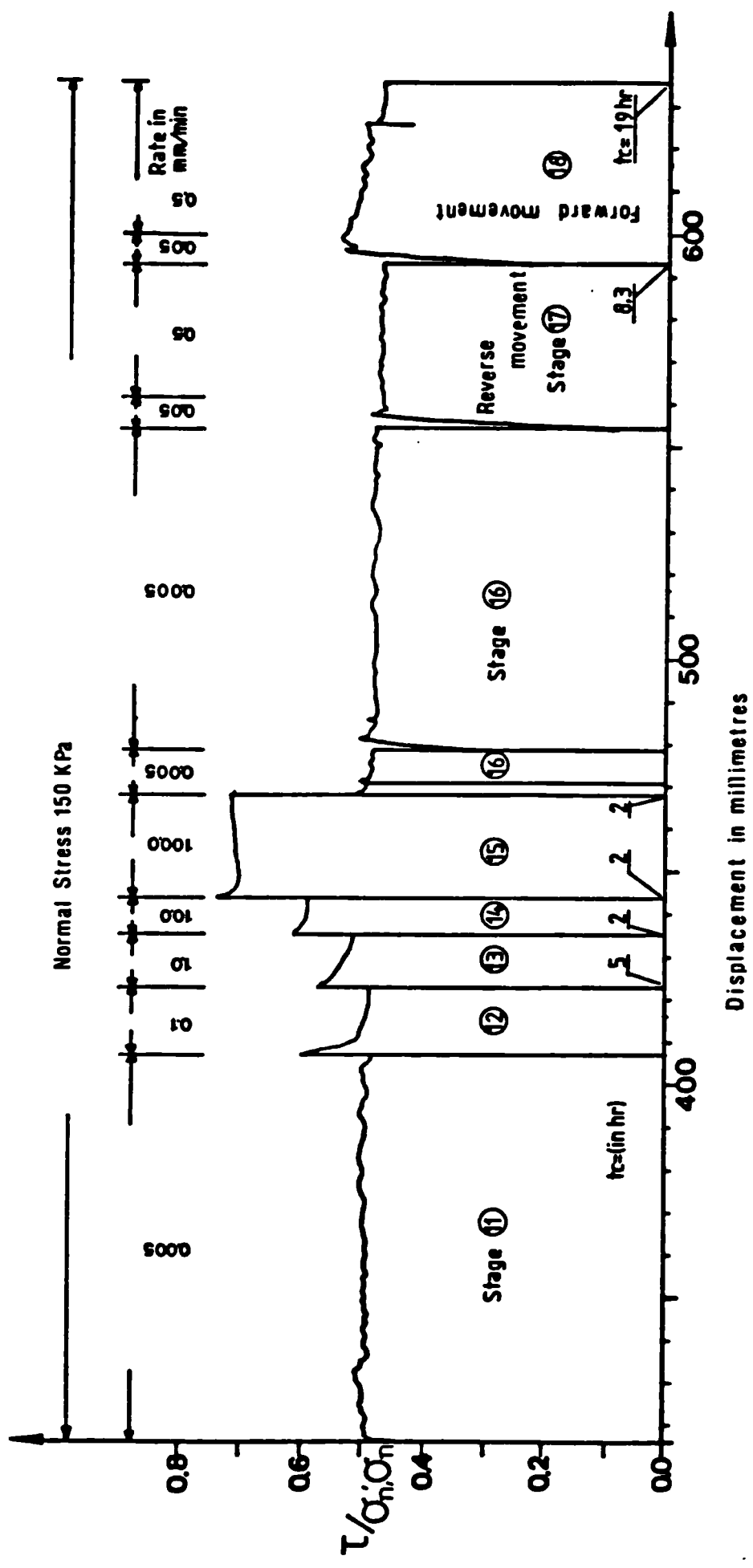


Fig. A2.27b Ring shear test 25 on weathered Cowden Till

Table A2.25 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
19	0.01	685.0	200.0	0.460	0.43	
20	400	807.0	200.0	b:0.560 c:0.730	0.0 122.0	
21	0.01	822.0	200.0	e:0.500 0.460	15.0	
22	200	922.0	200.0	b:0.580 c:0.620	0.0 100.0	
23	0.01	929.0	200.0	e:0.485 0.470	7.0	
24	100	959.0	200.0	b:0.570 c:0.595	0.0 30.0	
25	0.01	984.0	200.0	e:0.498 0.460	25.0	
26	10	1034.0	200.0	b:0.520 0.550	0.0 50.0	
27	0.01	1092.0	200.0	e:0.504 0.454		
28	0.37	1129.0	200.0	c:0.473 0.459		
29	10	1317.0	200.0	b:0.470 c,d:0.527	0.0 85.0-188.0	
30	0.37	1332.0	200.0	e:0.512 0.459	15.0	2hr pause
31	0.37	1342.0	200.0	c:0.468 0.459		

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

Table A2.25 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
32	10	1497.0	200.0	b:0.475 c:0.527	0.0	
33	0.37	1499.0	200.0	e:0.517	2.0	4hr pause
34	0.37	1506.0	200.0	c:0.517 0.463	0.0 7.0	15hr pause
35	0.37	1515.0	200.0	c:0.478 0.459	0.0 9.0	
36	10	1687.0	200.0	b:0.473 c:0.541	0.0	6hr pause
37	10	1773.0	200.0	b, c:0.546 0.520	0.0	18hr pause
38	0.37	1798.0	200.0	e:0.551 0.459	25.0	
39	0.01	1845.0	200.0	e:0.498 0.449		
40	0.015	1898.0	446.0	0.430		
41	0.015	1936.0	207.0	peak:0.576 0.450		
42	0.015	1974.0	103.0	peak:0.576 0.510		
43	0.015	2003.0	48.0	peak:0.605 0.540		
44	0.015	2045.0	18.0	peak:0.751 0.624		

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

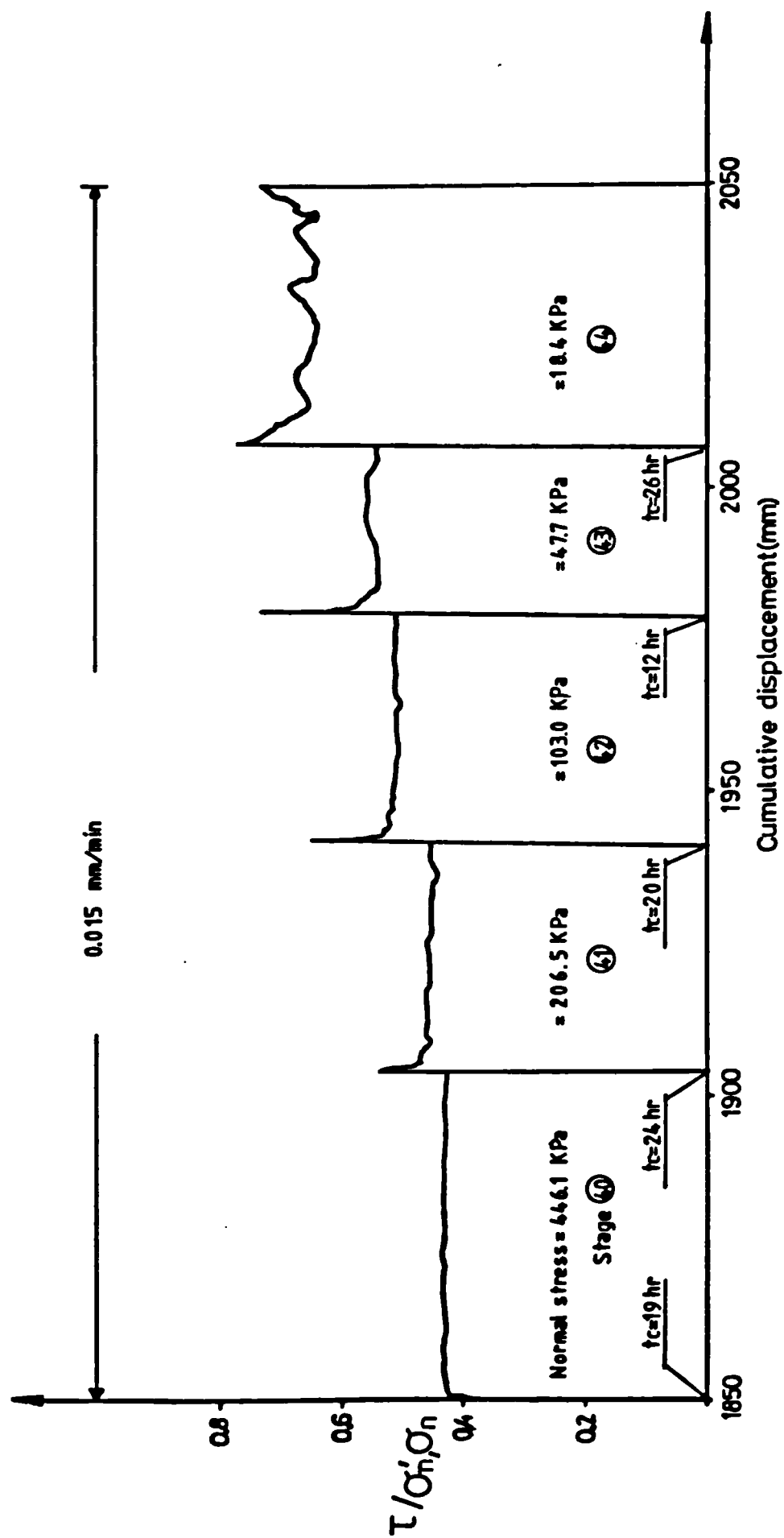


Fig. A2.27d Ring shear test 25 on weathered Cowden Till

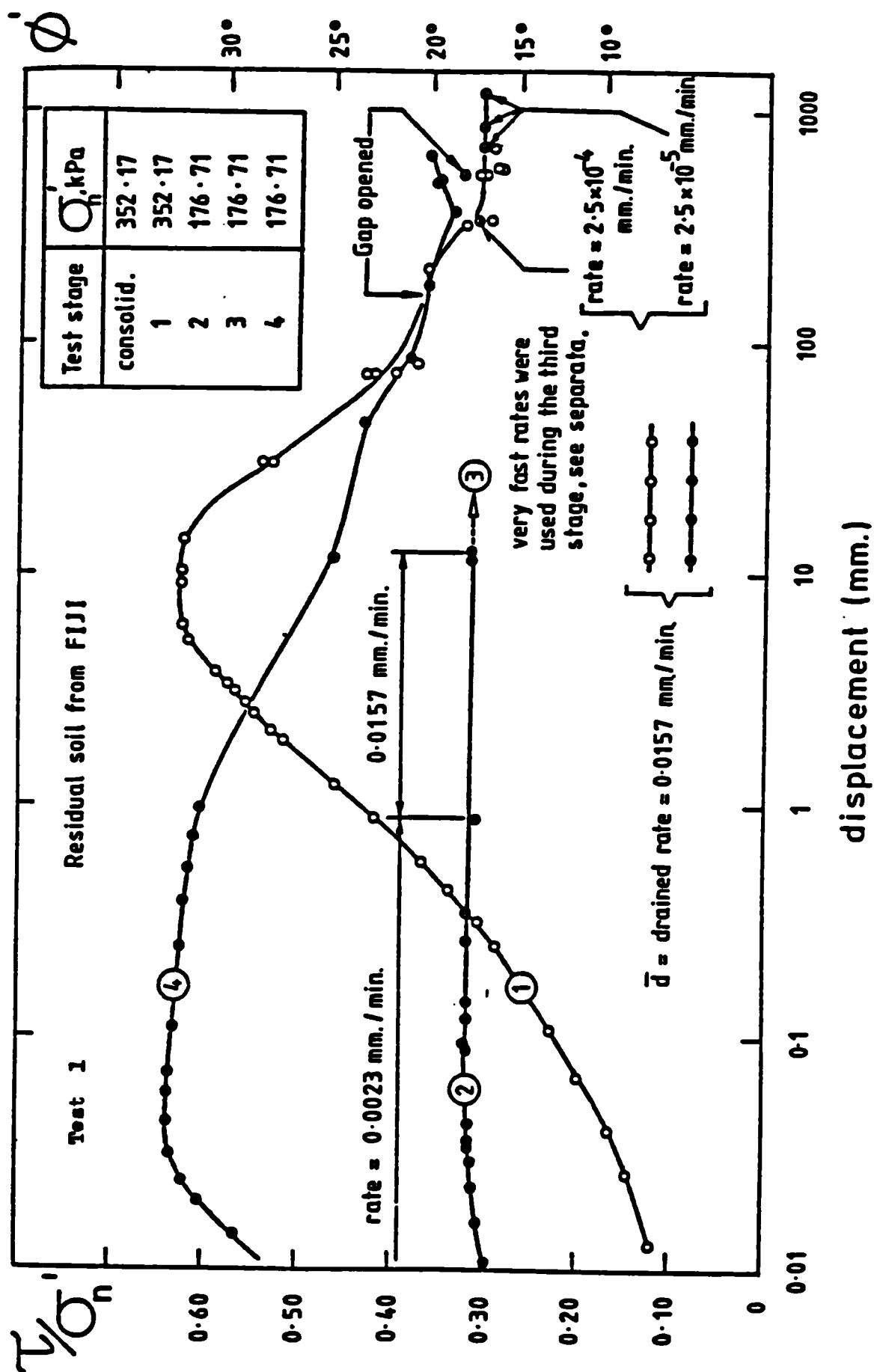


Fig. A2.28a Ring shear test 26 on the residual soil from Fiji

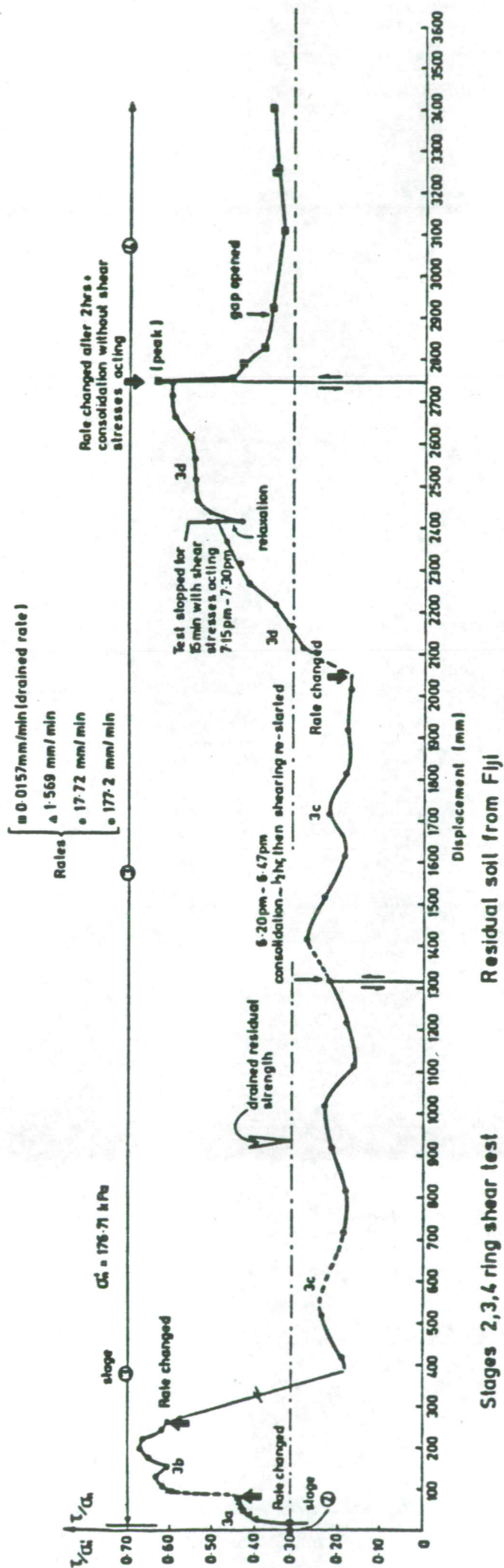


Fig. A2.28b Ring shear test 26 on the residual soil from Fiji

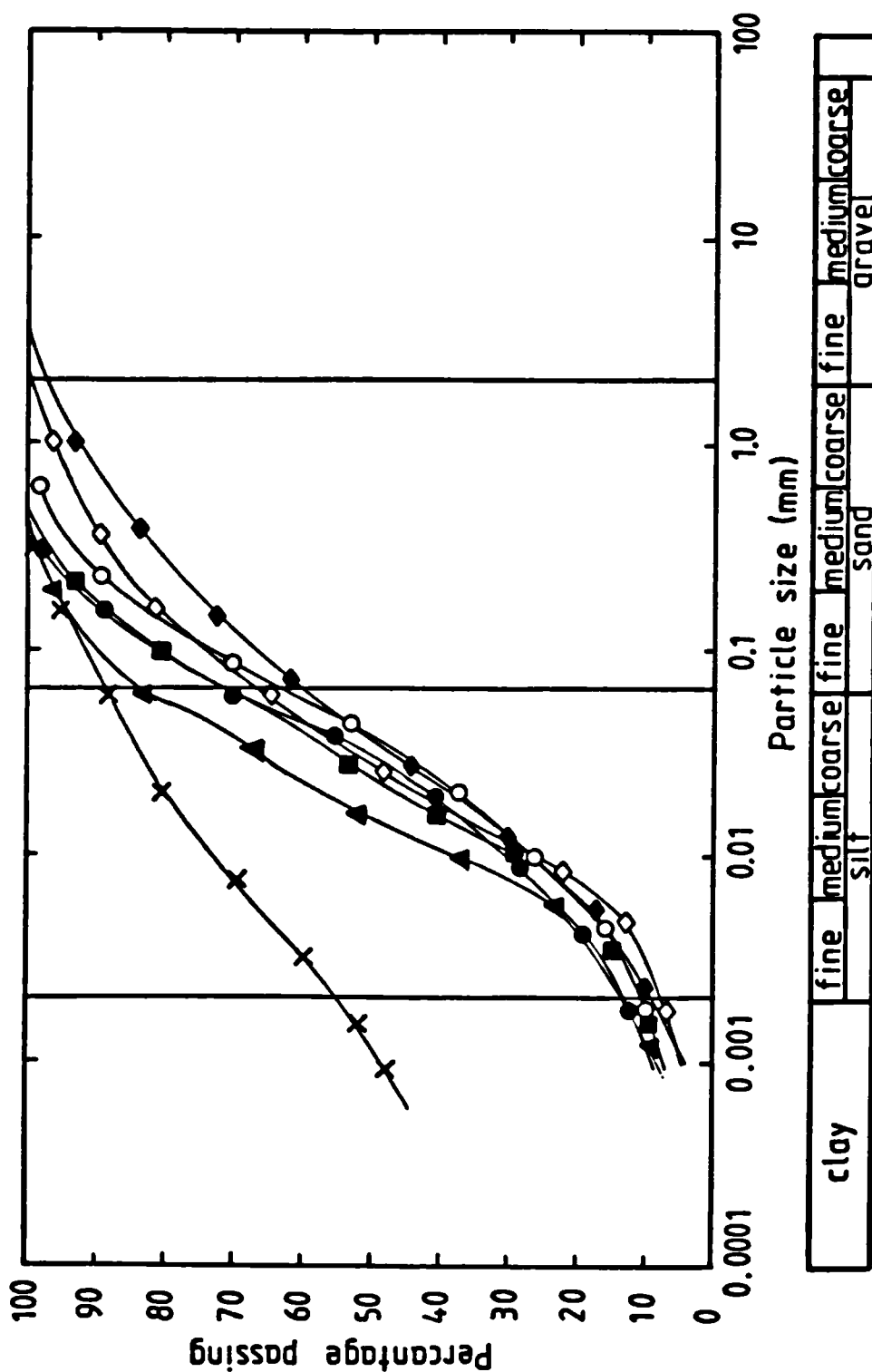


Fig. A2.29 Grading curves of residual soils

- X Fiji
- Mauritius, as supplied (pretreated soil)
- >> ,after the ring shear test
- ◆ Japan, as supplied
- ◇ >> (pretreated soil)
- Japan (Test 29)
- ▲ Japan (Test 30)

Table A2.26 Results from test 27 on residual soil from F1.11

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.016	179.9	437.7 470.0	peak:0.616 0.195	3.6 131.2	
2	133	439.6	469.7 470.0 483.0 467.5 461.0	b:0.268 c:0.363 0.217 0.277 0.240	0.2 3.6 636-84.4 149.6 200.0	
3	0.016	661.8	466.2 461.3	0.144 e:0.379 0.196	259.4 0.6 97.0-102.7	
4	133	923.4	450.4 457.5 479.0 480.4	b:0.270 c:0.359 d:0.269 0.291	0.2 6.1-12.9 171.3 200.0	
5	133	1184.6	478.4 441.2 441.1	0.294 b:0.390 c:0.394	261.6 0.3 0.9	
6	0.016	1339.1	435.0 447.0 455.0	0.251 e:0.341 0.195	200.0-261.2 0.5-0.7 135.4-154.5	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

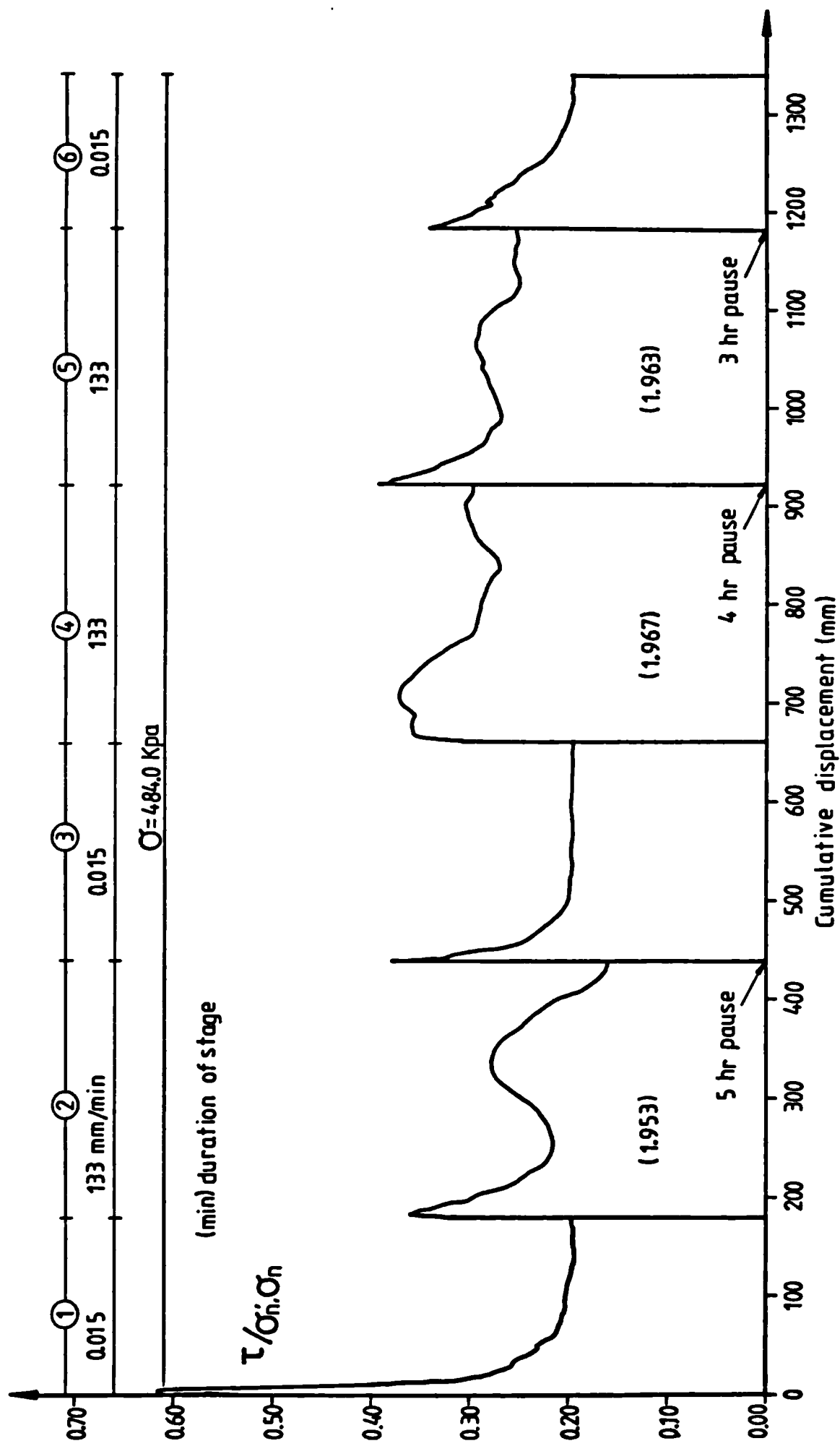


Fig. A2.30a Ring shear test 27 on the residual soil from Fiji

Table A2.26 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
7	1100	1579.6	458.9 446.2 446.2 440.1	b:0.441 c:0.500 0.200 0.149	0.7 2.4-3.5 200.0 240.5	
8	0.016	1650.2	449.5	e:0.344 0.200	0.8 70.6	
9	0.015	1734.6	191.7 183.0	peak:0.225 0.220	0.4 11.8-84.4	
10	133	1932.2	185.6 188.8 200.8 196.9	b:0.288 c:0.523 d:0.352 0.369	0.1 15.5 162.3 197.6	
11	0.015	2012.7	186.0	e:0.438 0.252	0.7 80.5	
12	1100	2177.7	185.9 185.9 205.6	b:0.514 c:0.620 0.205	0.5 5.1 165.0	
13	0.015	2360.8	191.4	e:0.390 0.247	1.0 93.9-183.1	
14	2400	2536.3	no recording was performed			
15	2400	2764.1	200.3 195.0 187.5 186.5	b,c:0.560 d:0.137 0.150 0.162	0.4 122-142.3 200.0 227.8	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

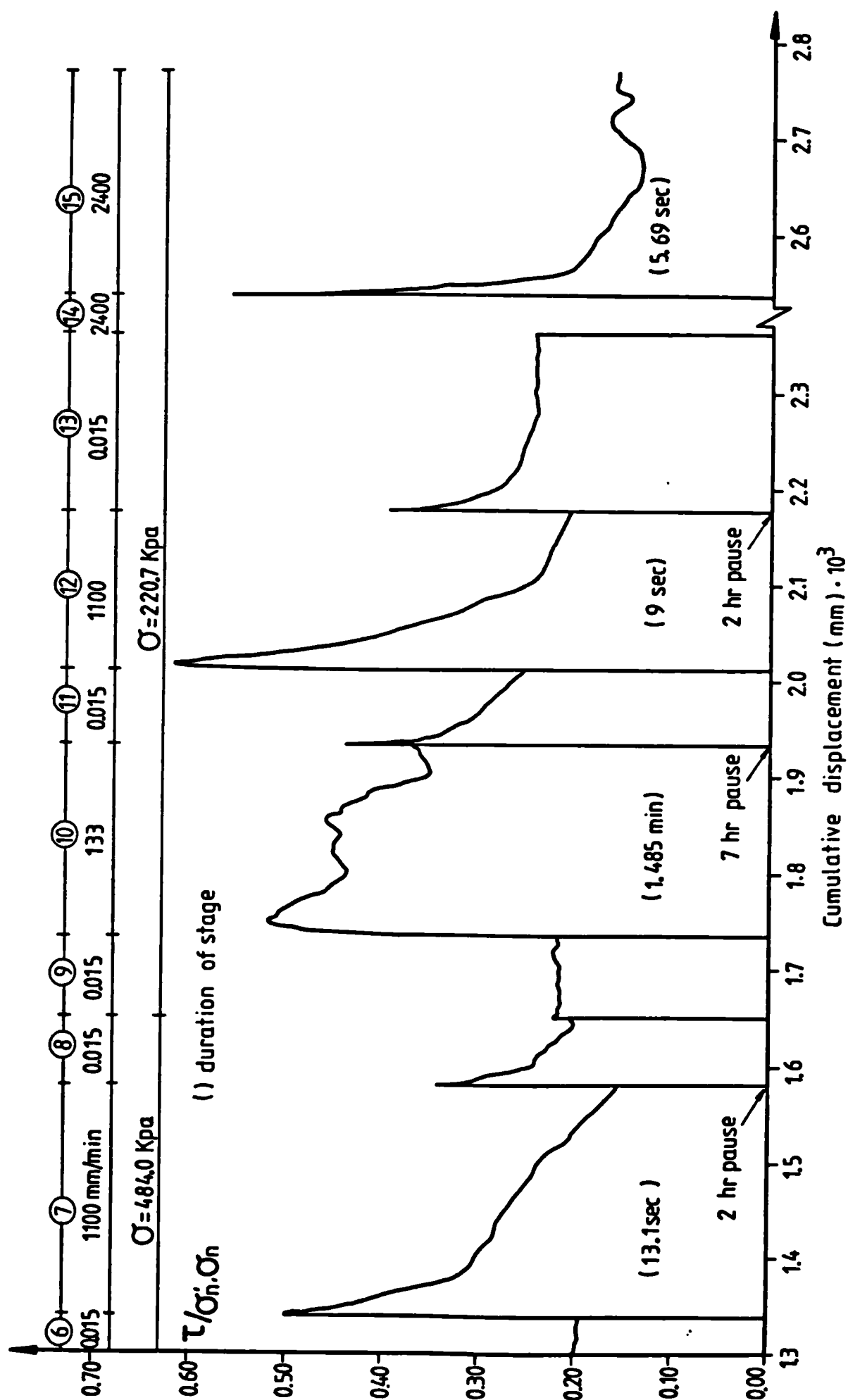


Fig. A2.30b Ring shear test 27 on the residual soil from Fiji

Table A2.27 Results from ring shear test 28 on residual soil from Mauritius

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.010	55.2	214.5 208.9	peak:0.780 0.717	7.4-8.1 43.5	
2	360	219.4	212.5 211.6	b,c:0.938 d:0.646	4.6-5.3 164.2	
3	0.01	246.3	231.0 227.0	e:0.700 0.686	1-6.6 19.9-26.8	
4	0.01	273.7	231.0 230.0	e:0.717 0.710	1.7-3.8 4.7-27.5	
5	1000	448.0	232.3 232.4	b,c:1.080 d:0.674	0.6 174.2	
6	0.01	486.6	217.5	e:0.718	5.2-38.7	
7	8.8	540.8	215.7 223.8	b,c:0.859 d:0.700	0.5 53.5-54.2	
8	0.01	553.6	224.5 223.3	e:0.727 0.690	0.2 12.8	
9	0.095	579.5	224.3 224.7	c:0.747 0.689	0.1 3.0-25.9	
10	0.01	596.1	223.5	e:0.706 0.693	0.1 2.0-16.6	
11	0.932	625.1	223.1 223.2	c:0.714 0.685	0.1 0.2-28.9	
12	0.01	637.0	220.0 219.5	e:0.718 0.686	0.01 2.4-12	
13	12	874.4	224.3 220.5 225.0 225.0	b,c:0.817 d:0.662 0.710 0.676	0.4 116.7-126.7 200.0 237.4	
14	0.01	888.2	229.1 227.0	e:0.710 0.678	5.6-13.7	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

Table A2.23 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
15	0.01	928.6	457.0 444.0	0.700 0.707	1.3-10.7 10.7-40.5	
16	12	1162.5	446.6 446.9	b,c:0.859 d:0.710	0.8 199.3-211.3	
17	0.01	1176.8	445.8 445.4	e:0.717 0.679	0.4 6.0-14.3	
18	0.095	1220.0	452.5	0.690	4.0-43.2	
19	0.01	1228.7	450.6 450.0	e:0.712 0.676	0.1 3.3	
20	90	1395.6	451.8 457.2	b,c:0.941 d:0.720	1.5 107.1-117.6	
21	9.5	1408.3	455.0 452.0	e:0.715 0.675	0.4 6.5-12.8	
22	0.058	1436.3	453.0	d:0.705	13.2-28.3	
23	90	1609.2	451.9 461.2	b,c:0.919 d:0.726	1.9 172.7	
24	0.01	1621.2	460.5 457.5	e:0.716 0.680	0.2 1.4	
25	0.058	1697.8	457.5	d:0.685	57.8-76.6	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

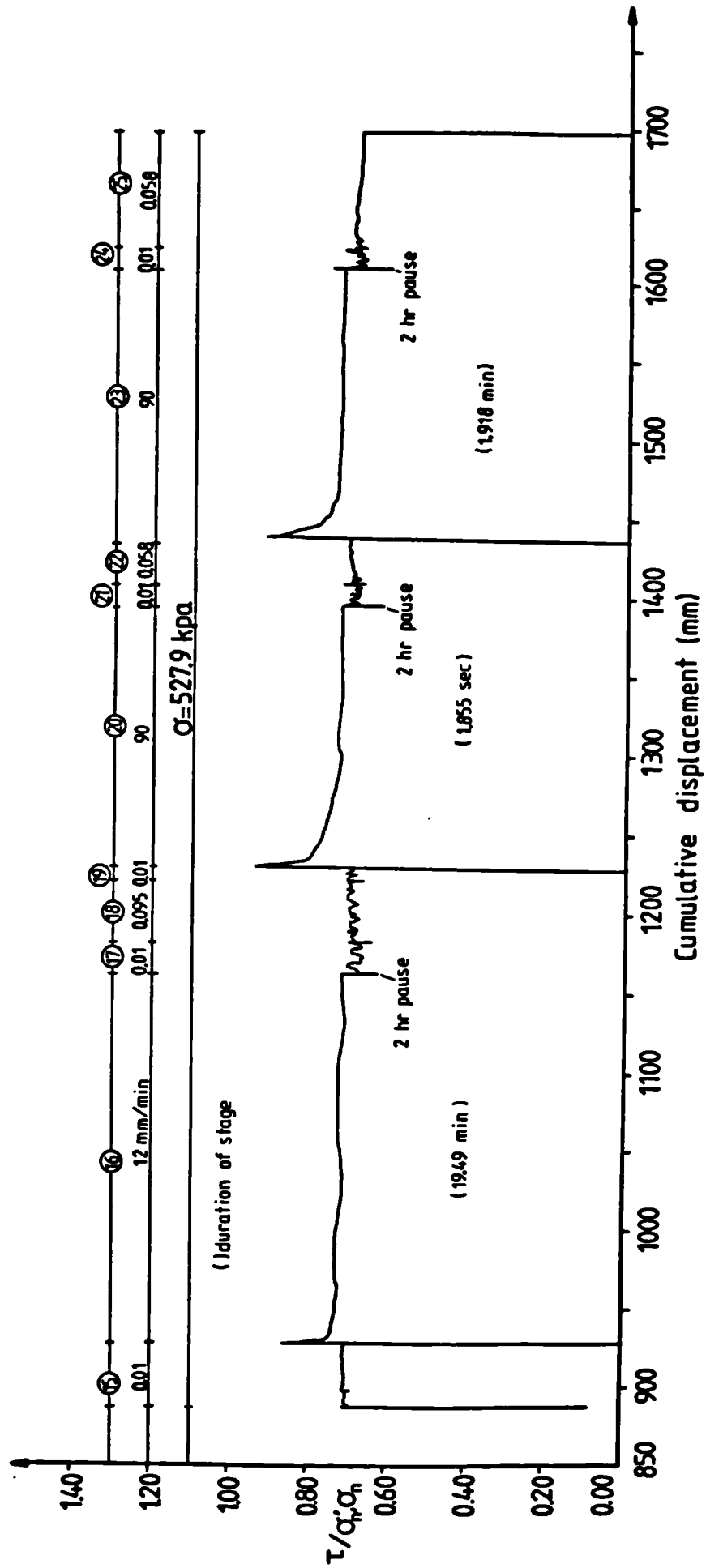


Fig. A2.31b Ring shear test 28 on the residual soil from Mauritius

Table A2.27 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
26	360	1849.7	453.7 466.7	b,c:0.897 d:0.747	7.7-8.2 129.2-152.0	
27	0.01	1876.5	464.0 460.8	e:0.710 0.686	2.9-4.7 22.3	
28	1000	2087.9	477.7 466.0	b,c:0.993 d:0.250	0.5 200.0-211.5	
29	0.01	2121.3	463.0 456.7	e:0.673 0.687	2.3 15.8-17.8	
30	1000	2336.3	459.0 473.0	0.690 b,c:0.993	17.8-334 0.6	
31	0.01	2389.9	526.2 505.0	d:0.370-0.400 e:0.650	139.7-215.0 2.9-6.2	
32	2000	2578.9	490.0 480.0	0.673 0.659	12.6-30 45.1-536	
33	0.01	2601.2	477.1 486.9 486.9	b,c:1.024 d:0.266 0.330	1.0 135.2 189.0	
			496.1 492.0 490.0	e:0.685 0.670 0.658	3.1 4.8 11.7-22.3	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

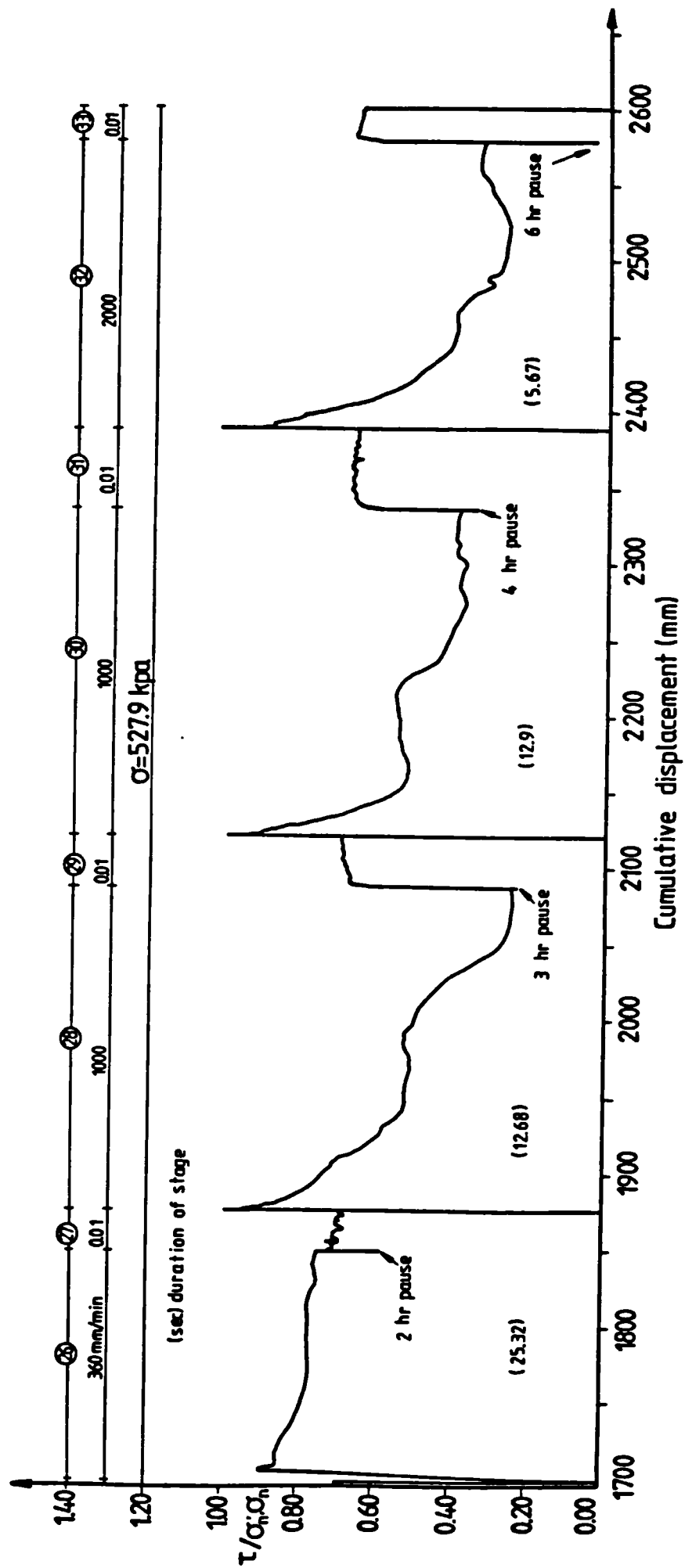


Fig. A2.31c Ring shear test 28 on the residual soil from Mauritius

Table A2.27 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
34	0.01	2661.4	240.6 242.2 240.0	peak: 0.853 0.769 0.691	0.6 1.0 57.9-60.0	
35	2000	2826.9	242.2 225.7	b,c: 1.234 d: 0.265-0.297	0.2 100.0-165.8	
36	0.01	2860.5	214.0	0.707	2.6-33.6	
37	1000	3006.0		no recording was performed		
38	0.01	3032.3	223.8	0.687	2.1-26.3	
39	1000	3200.8	230.3 230.0 219.6	b,c: 1.150 d: 0.313 0.386	0.4 102.5-128.3 168.5	
40	0.01	3231.6	218.4 217.0	e: 0.706 0.689	2.4 10.9-30.8	
41	1000	3247.9	231.3 233.7	b,c: 1.120 0.874	0.4 16.3	
42	0.01	3273.5	230.5 227.0	e: 0.740 0.730	2.1-3.1 10.7-25.3	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

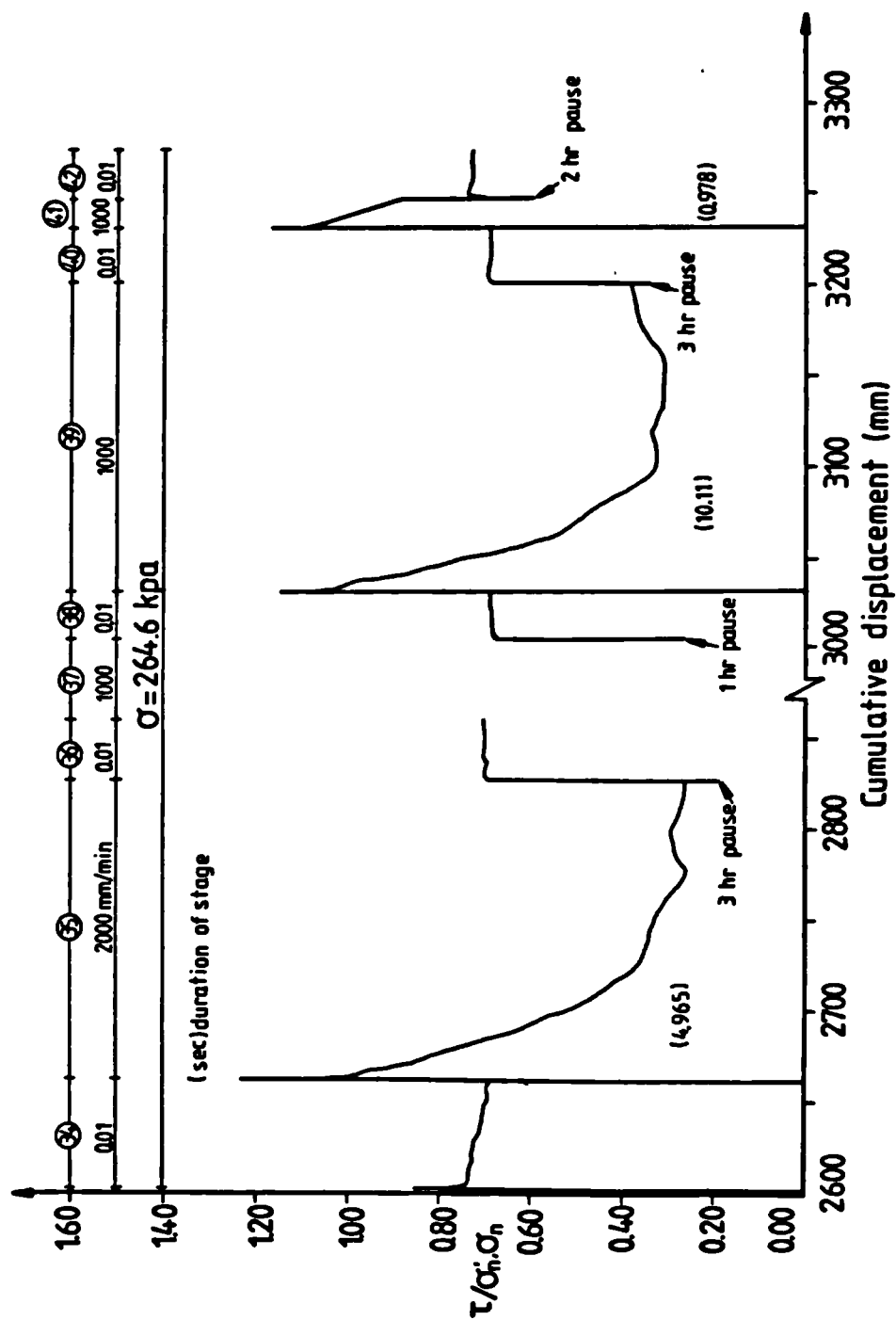


Fig. A2.31d Ring shear test 28 on the residual soil from Mauritius

Table A2.28 Results from ring shear test 29 on residual soil from Japan

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)		Notes
					peak missed		
1	0.0117	75.1	202.7	0.679	75.1		
2	120	272.9	261.9	b,c:1.000	2.1		
3	0.0117	305.0	237.4	d:0.634	197.7		
4	1000	493.5	239.7	e:0.651	0.3		
			236.0	0.610	29.5-31.8		
5	0.0117	529.1	246.8	b,c:1.131	10.0		
			238.0	d:0.651	73.3-188.8		
6	10	718.7	235.9	e:0.708	0.3		
			230.0	0.631	14.7-35.5		
7	0.0117	761.6	237.7	b,c:0.838	0.7		
			234.8	d:0.638	189.6		
8	0.9333	950.1	244.4	e:0.703	0.4		
			239.0	0.629	25.1-42.9		
9	0.0117	982.3	peak missed				
			239.5	d:0.625	1.0-14.0		
10	360	1158.4	230.0	0.640	72.8-188.5		
			229.4	e:0.693	0.1		
11	0.0117	1187.3	231.0	0.647	1.0-32.1		
			237.2	b,c:0.959	3.8		
12	5000	1408.9	241.3	d:0.628	173.0		
			244.5	e:0.735	0.3		
13	0.0117	1434.5	240.0	0.642	1.0-28.9		
			238.6	b,c:1.358	2.4		
13	0.0117	1434.5	241.0	d:0.691	186.3-221.6		
			241.1	e:0.760	0.4		
			257.0	0.660	7.8-25.6		

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

Table A2.28 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
14	0.0117	1455.4	481.5 475.0	peak:0.789 0.701	1.2 7.4-20.9	
15	1000	1633.6	479.7 470.9	b,c:1.112 d:0.659	1.2 178.2	
16	0.0117	1648.0	473.6 468.0	e:0.736 0.670	0.6 4.0-14.4	
17	0.9333	1826.5	477.1 473.0 469.0	b,c:0.672 0.664 d:0.648	0.5 0.6-129.3 129.3-178.4	
18	0.0117	1869.4	471.9 474.0	e:0.674 0.646	0.8-1.2 1.3-42.9	
19	10	2049.0	466.1 452.6 446.6	b,c:0.799 d:0.644 0.654	0.7 113.3 179.7	
20	0.0117	2081.6	447.4 445.0	e:0.753 0.667	0.4 1.0-32.6	
21	120	2256.5	470.8 454.4	b,c:0.829 d:0.651	2.4 174.9	
22	0.0117	2310.5	457.3 455.5	e:0.732 0.639	0.4 51.2-54.0	
23	360	2467.0	481.5 474.6	b,c:0.827 d:0.644	6.3 156.5	
24	0.0117	2493.9	476.0 470.0	e:0.748 0.658	0.3 13.3-26.9	
25	2500	2706.7	473.5 475.3	b,c:1.254 d:0.635	1.2 212.8	
26	0.0117	2745.0	442.7 443.0	e:0.699 0.652	0.6 5.2-38.3	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

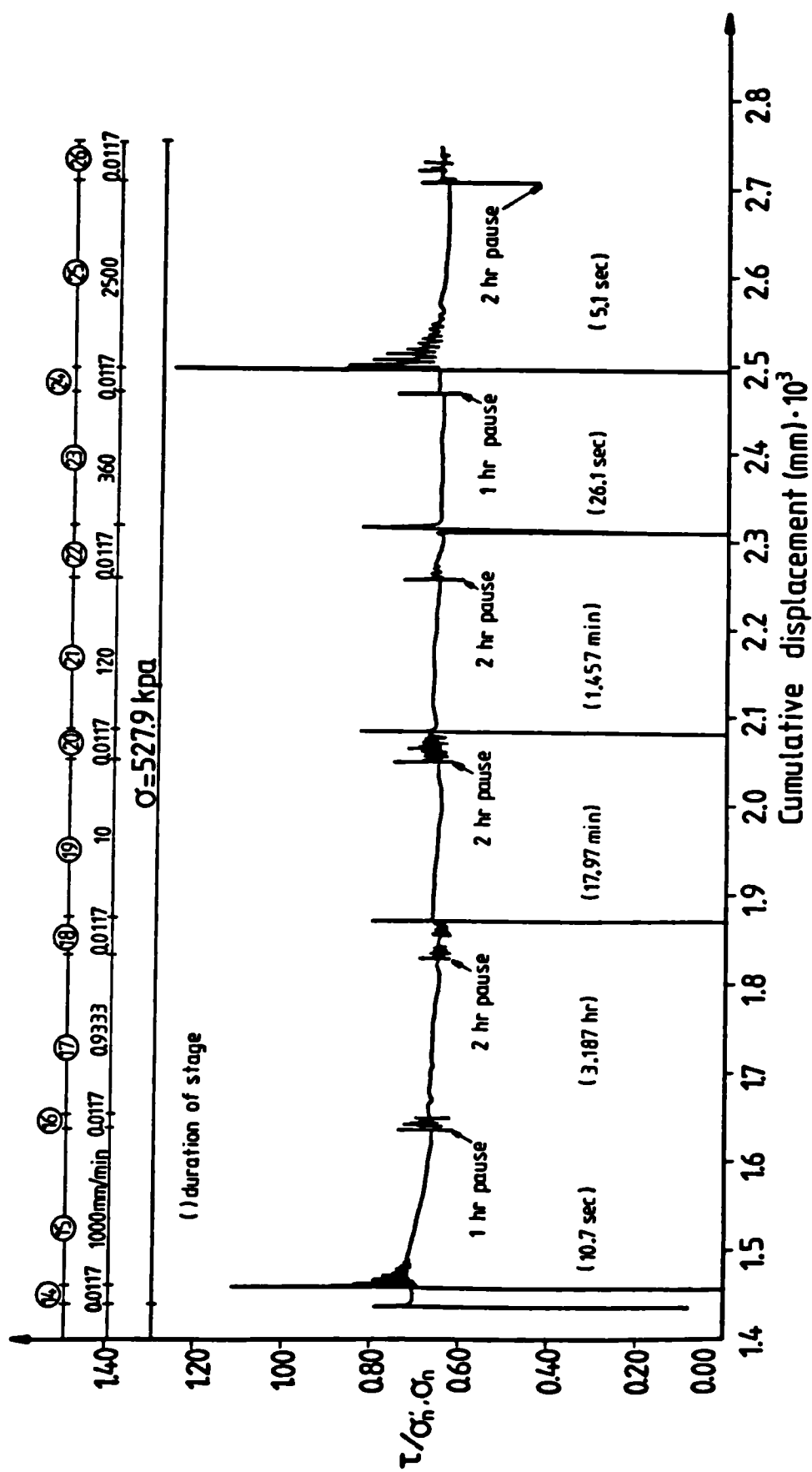


Fig. A2.32b Ring shear test 29 on the residual soil from Japan

Table A2.28 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
27a	10	2745.1	470.2 484.8	b,c:0.068 0.668	0.01 0.17	
27b	10	2745.5	478.2 483.4	b,c:0.895 0.635	0.01 0.31	
27c	10	2745.6	482.7 475.8	b,c:1.104 0.706	0.00 0.13	
27d	120	2747.6	475.3 468.7	b,c:1.168 0.733	0.08 1.99	
27e	120	2749.4	471.2 490.2	b,c:1.066 0.690	0.09 1.78	
27f	3500	3158.9	474.9 515.2 514.0	b,c:1.262 0.644 d:0.633	1.70 170.1 413.9	
28	0.0117	3184.4	514.5 510.0	e:0.669 0.645±0.013	0.3 2.5-25.4	
29	2500	3553.6	503.0 517.1 517.1	b,c:1.175 0.639 d:0.627	1.2 172.4 369.3	
30	0.0117	3585.6	427.4 423.5	e:0.787 0.637±0.022	0.7 1.0-31.9	
31a	15	3585.9	428.3 429.0	b,c:1.145 0.643	0.01 0.3	
31b	4400	3954.7	430.5 424.7 419.6	b,c:1.151 0.629 d:0.613	2.1 220.8 368.7	
32	4400	4434.6	424.5 424.7 422.6 422.0	b,c:0.971 0.624 d:0.610 0.620	2.1 189.0 424.1 460.6	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

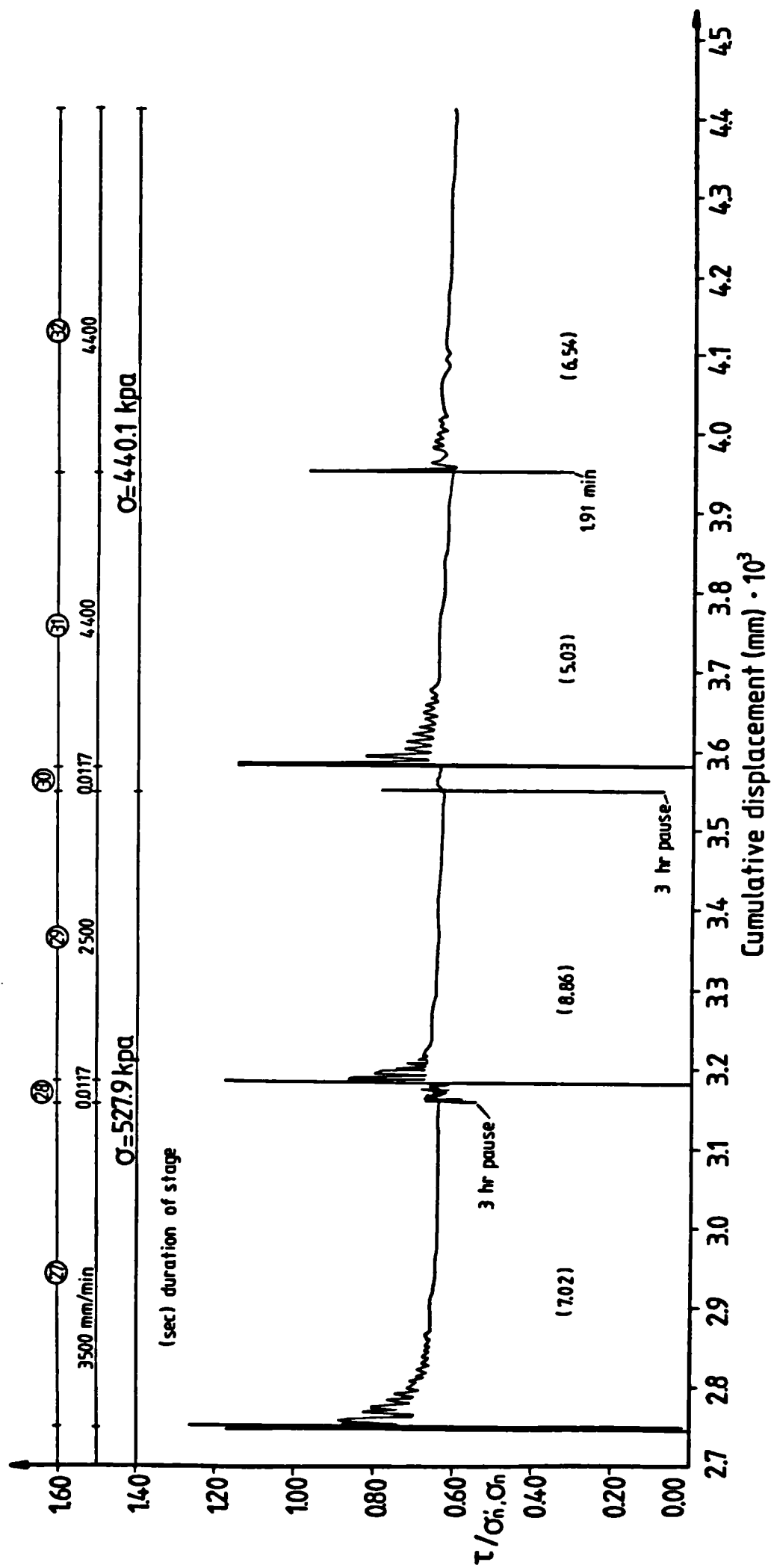


Fig. A2.32c Ring shear test 29 on the residual soil from Japan

Table A2.29 Results from ring shear test 29 on residual soil from Japan

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0925	98.1	439.7 388.0	peak:0.832 0.689	5.8-10.9 79.3-98.1	
2	100	264.8	487.6 425.7	b,c:0.909 d:0.677	3.0 166.7	
3	0.0925	386.5	433.1 429.5	e:0.753 0.645	0.4 97.3-121.6	
4	360	558.0	499.9 486.0	b,c:0.843 d:0.632	7.7 171.6	
5	0.0925	686.8	479.0 435.4	e:0.714 0.623	1.5 127.9-128.8	
6	1000	869.0	496.2 470.3	b,c:0.926 d:0.621	1.5 122.8-182.2	
7	0.0925	1038.2	458.3 435.0	e:0.680 0.617	0.5 40.2-169.2	
8	2500	1224.1	503.3 468.4	b,c:0.992 d:0.611	1.2 164.0	
9	0.0925	1327.4	466.4 475.1 467.9	0.644 e:0.688 0.588	185.9 0.3 103.3	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

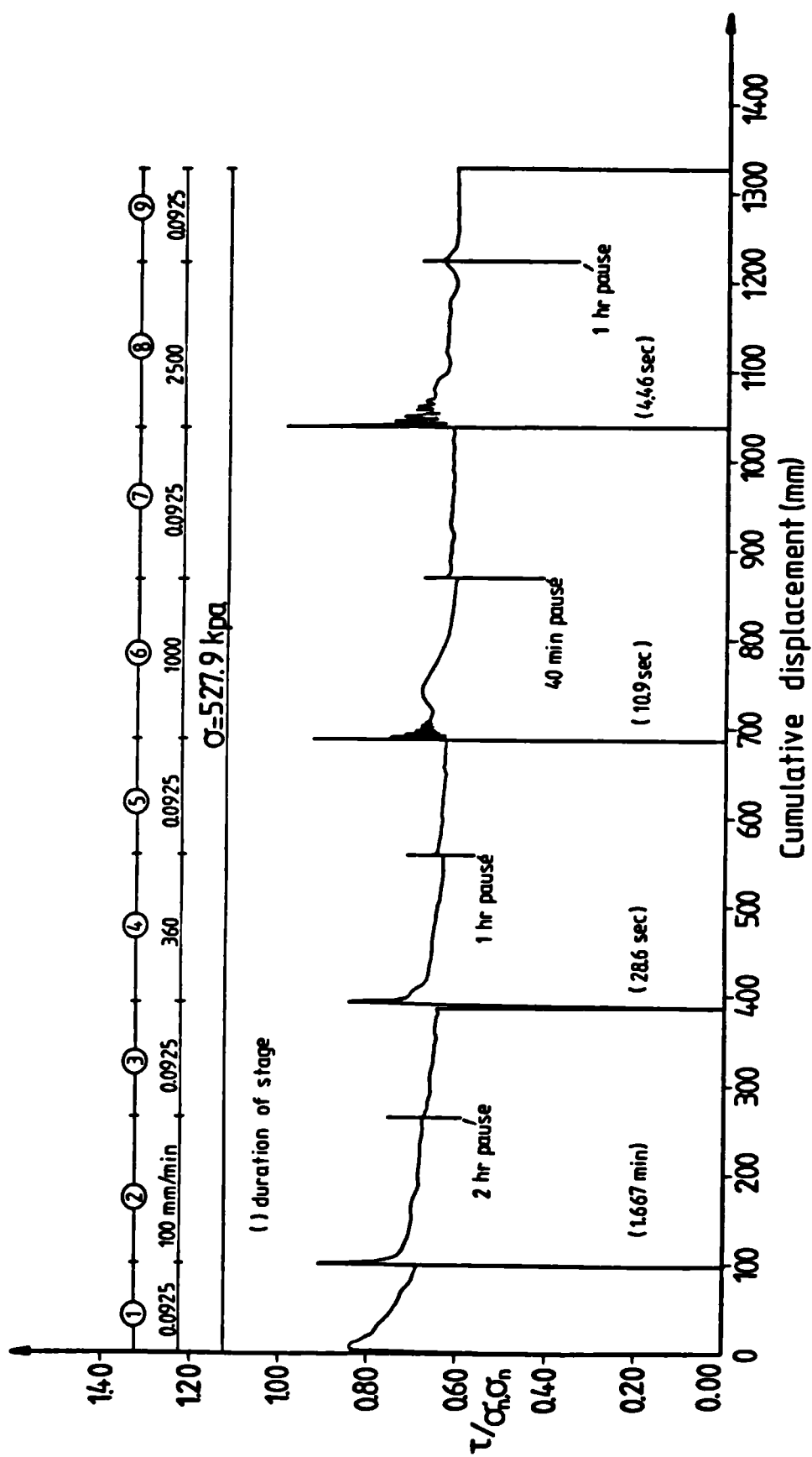


Fig. A2.33a Ring shear test 30 on the residual soil from Japan

Table A2.29 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
10	0.0925	1476.8	261.5 259.6 237.0	peak:0.796 0.590 0.623	0.8 10.3 149.4	
11	100	1642.2	252.9 244.4	b,c:0.923 d:0.603	1.2 165.4	
12	0.0925	1760.8	257.5 251.2	e:0.715 0.585	0.5 49.3-118.6	
13	1000	1936.6	260.2 254.5	b,c:1.177 d:0.556	0.9 175.9	
14	0.0925	2046.9	255.4 253.0	e:0.739 0.572	0.2 5.0-110.3	
15	2500	2273.7	256.1 255.0	b,c:1.222 d:0.553	0.9 54.7-226.8	
16	0.0925	2326.4	255.3 253.0	e:0.664 0.582	0.2 25.6-52.7	
17	6000	2478.3	259.7 257.7	b,c:1.129 d:0.480	2.7 151.8	
18	6000	2718.5	260.3 256.0	b,c:0.939 d:0.451	1.9 132.4-240.3	
19	0.0925	2785.5	257.6 255.5	e:0.642 0.592	0.9 38.7-66.9	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

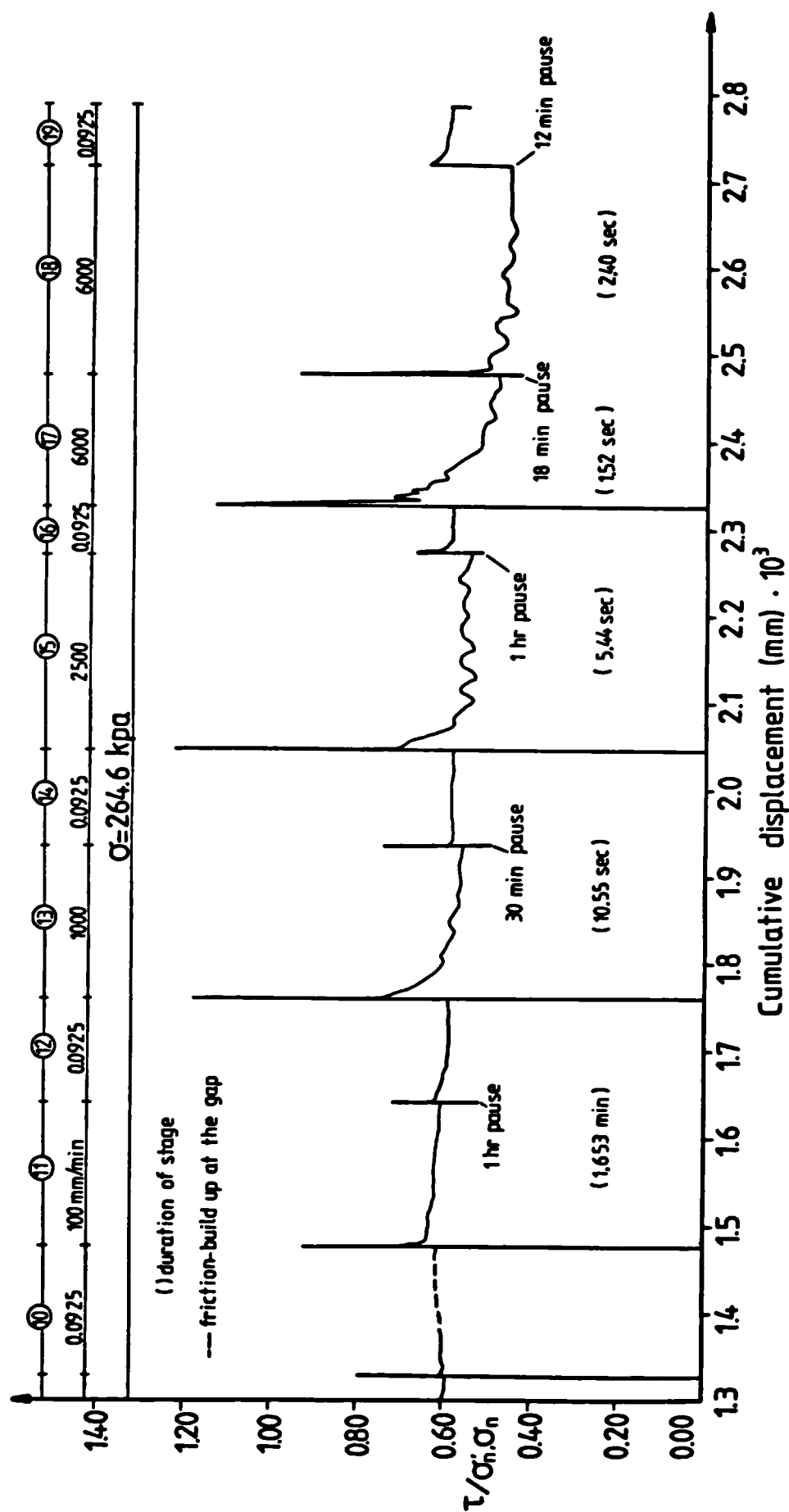


Fig. A2.33b Ring shear test 30 on the residual soil from Japan

Table A2.29 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
20	0.0115	2800.7	260.0 259.5	e:0.739 0.602	0.1 3.1-15.2	
21	6000	2956.0	260.7 258.8	b,c:1.197 d:0.446	0.3 155.3	
22	6000	3010.2	258.3 260.0	b,c:0.817 d:0.451	2.0 41.1-54.2	
23	1.8945	3066.5	260.3 258.5	0.599 0.588	0.3 16.5-56.3	
24	0.0925	3096.8	258.5 258.0	e:0.622 0.583	0.3 15.4-30.3	
25	1000	3211.3	258.4 257.0	b,c:1.225 d:0.537	0.7 114.5	
26	0.0925	3245.9	258.2 256.5	e:0.669 0.572	0.3 226.0-346.0	
27	0.0925	3888.5	130.1 129.5	peak:0.768 0.674	0.3 12.0-39.7	
28	1000	4080.8	129.1 129.1	b,c:1.342 d:0.387	0.5 171.0-192.3	
29	0.0925	4213.2	129.7 129.8 128.3	e:0.571 0.547 0.607	0.3 0.6 118.1-132.4	
30	6000	4403.8	129.3 128.3	b,c:1.315 d:0.212	2.9 162.7-190.6	
31	0.0925	4521.5	128.5 129.5 128.8	0.540 e:0.611 0.579	0.5 12.7-59.3 84.5-117.7	
32	360	4672.3	129.7 128.5 128.1	b,c:1.044 d:0.453 0.483	5.1 107.2-112.0 150.7	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

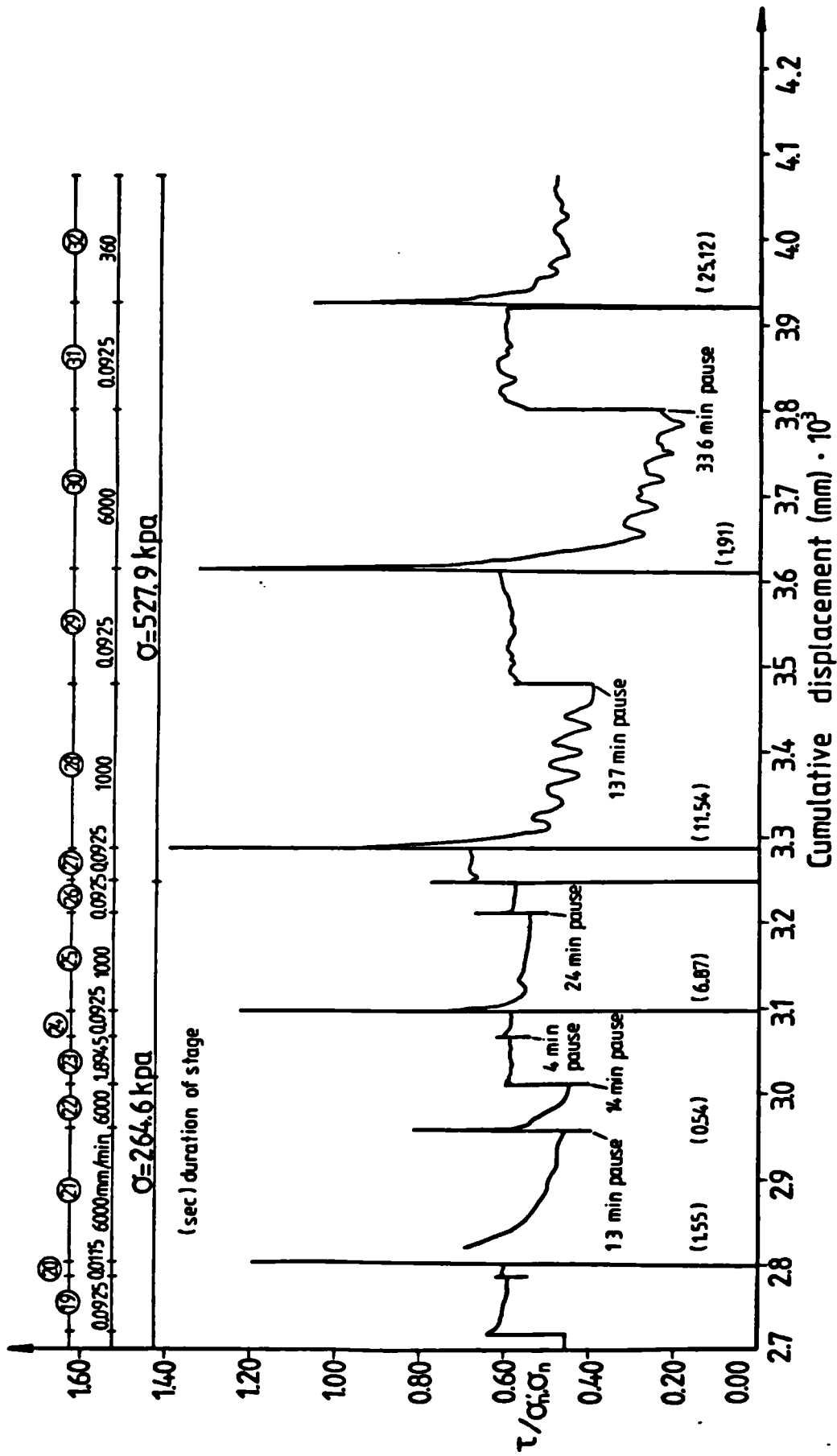
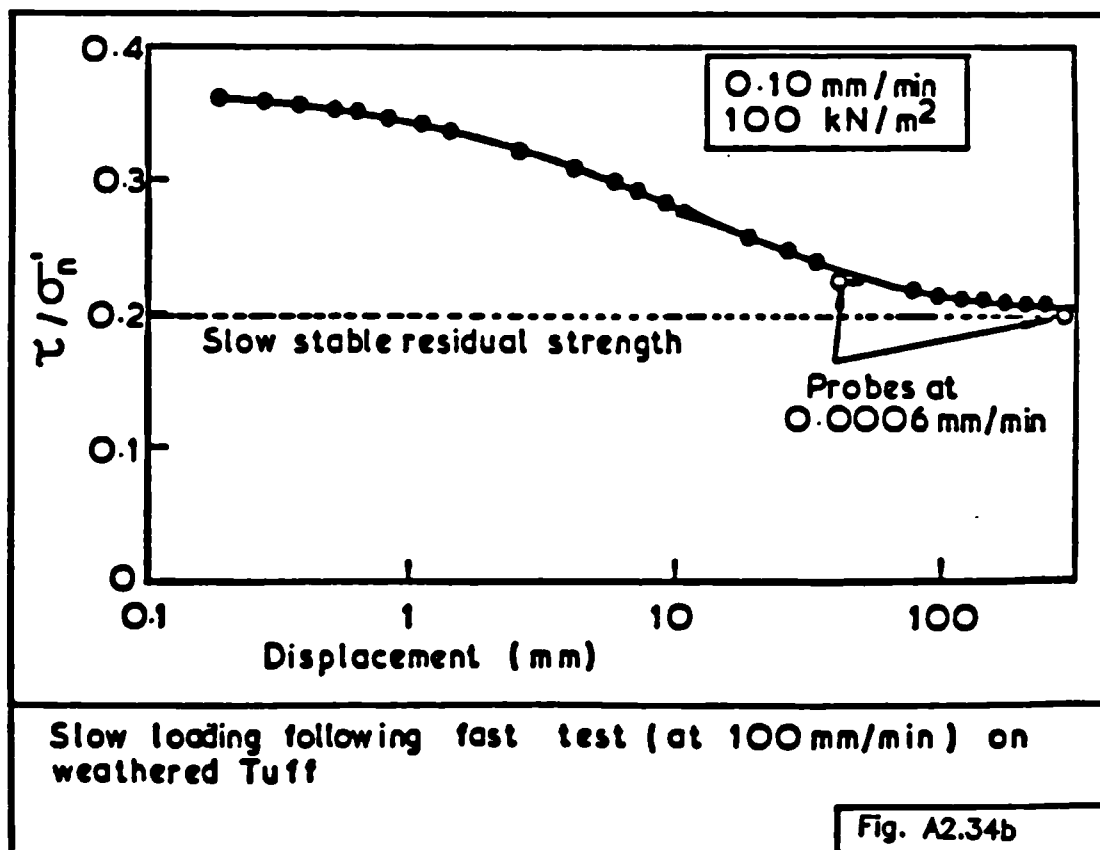
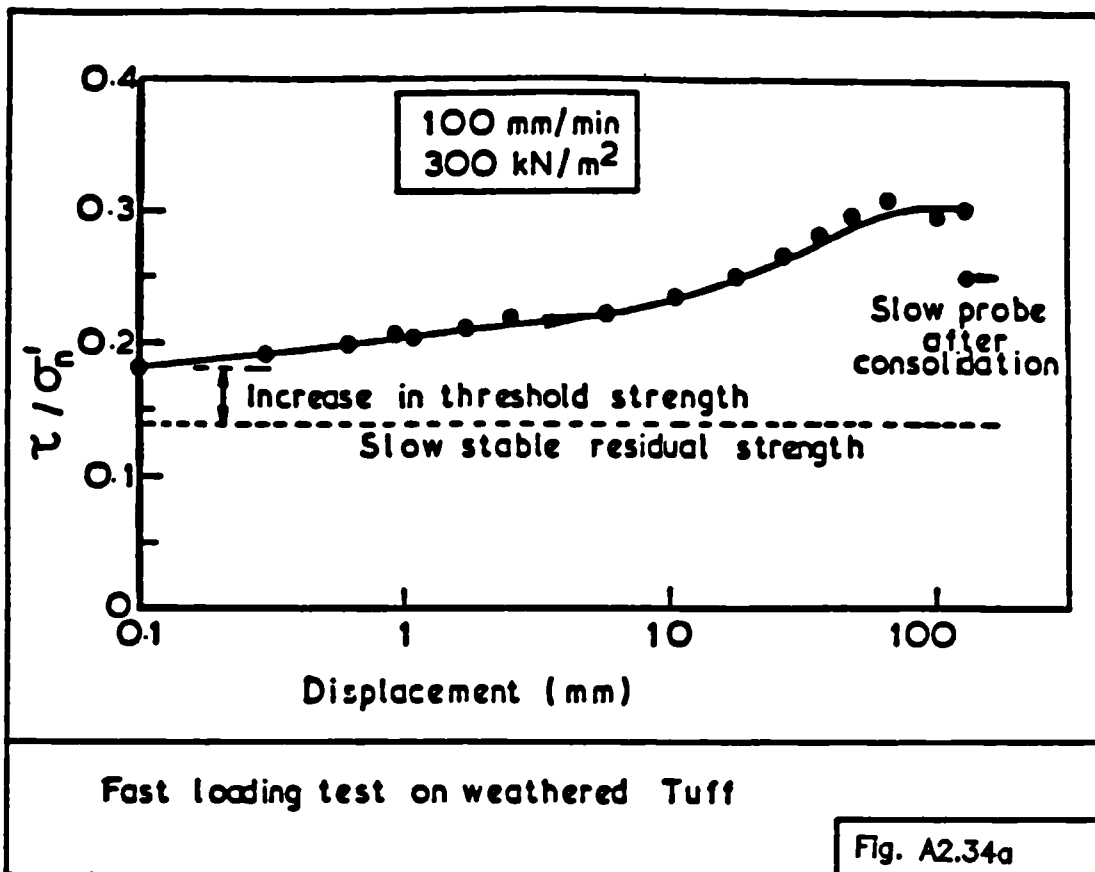


Fig. A2.33c Ring shear test 30 on the residual soil from Japan



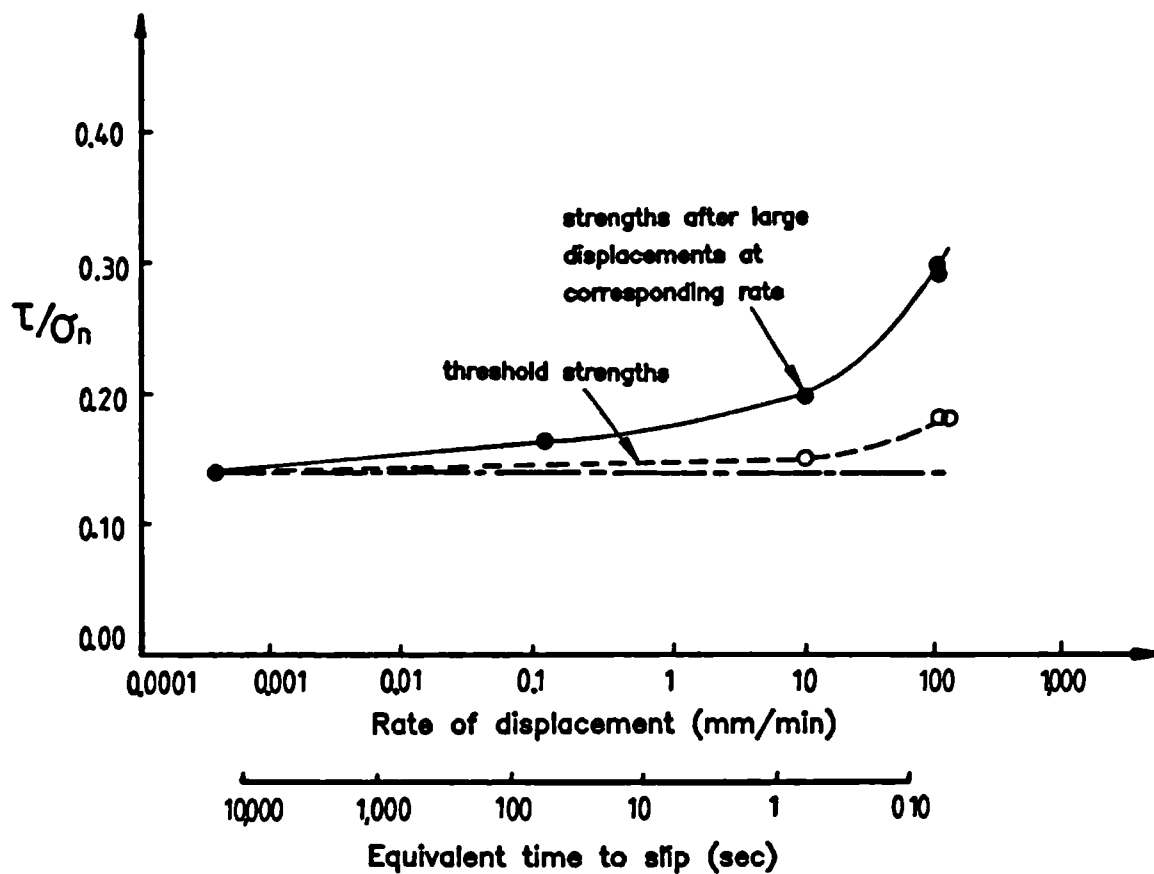


Fig. A2.34c Fast stages on weathered Tuff ($\sigma_s = 300 \text{ kPa}$)

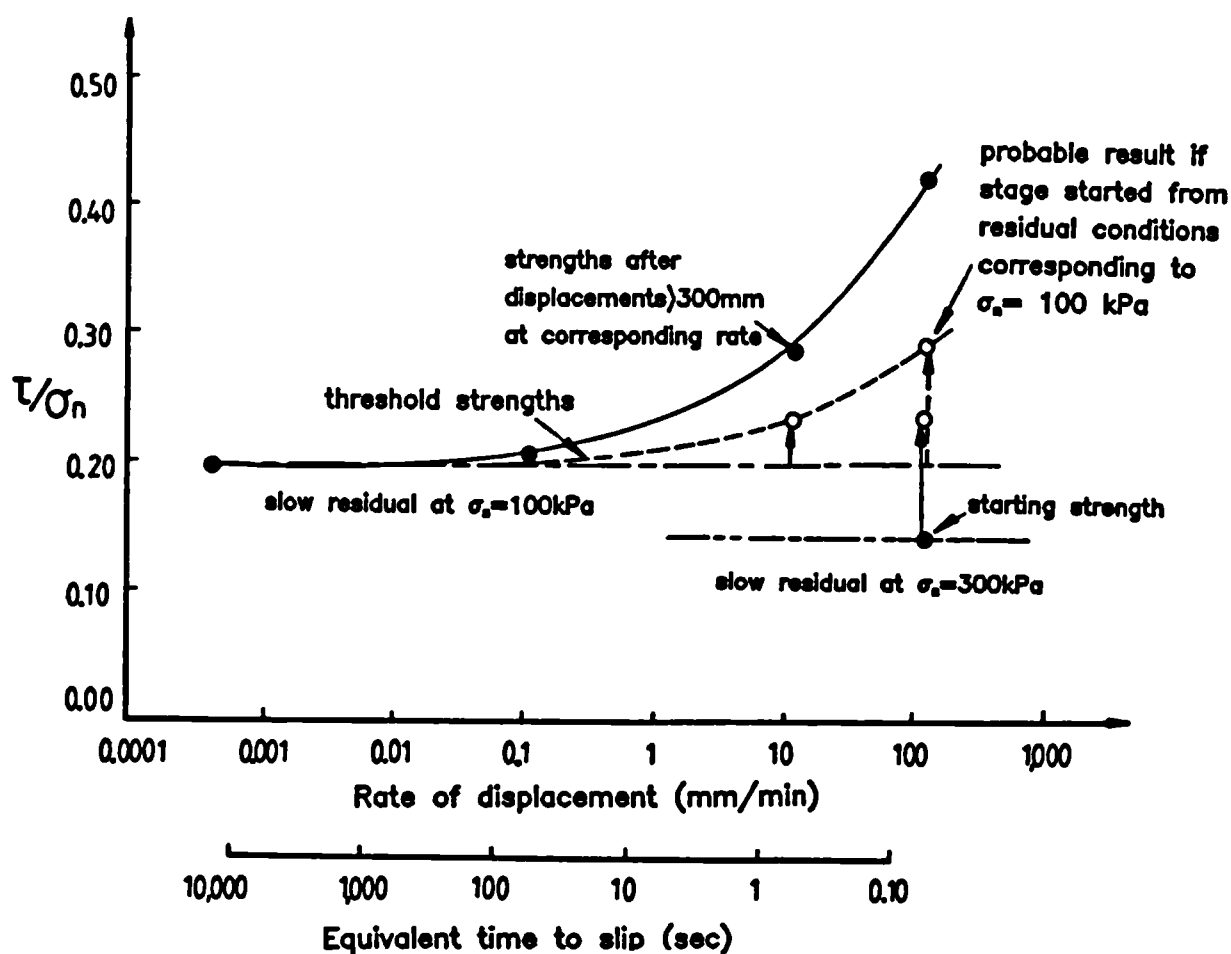


Fig. A2.34d Fast stages on weathered Tuff ($\sigma_s = 100 \text{ kPa}$)

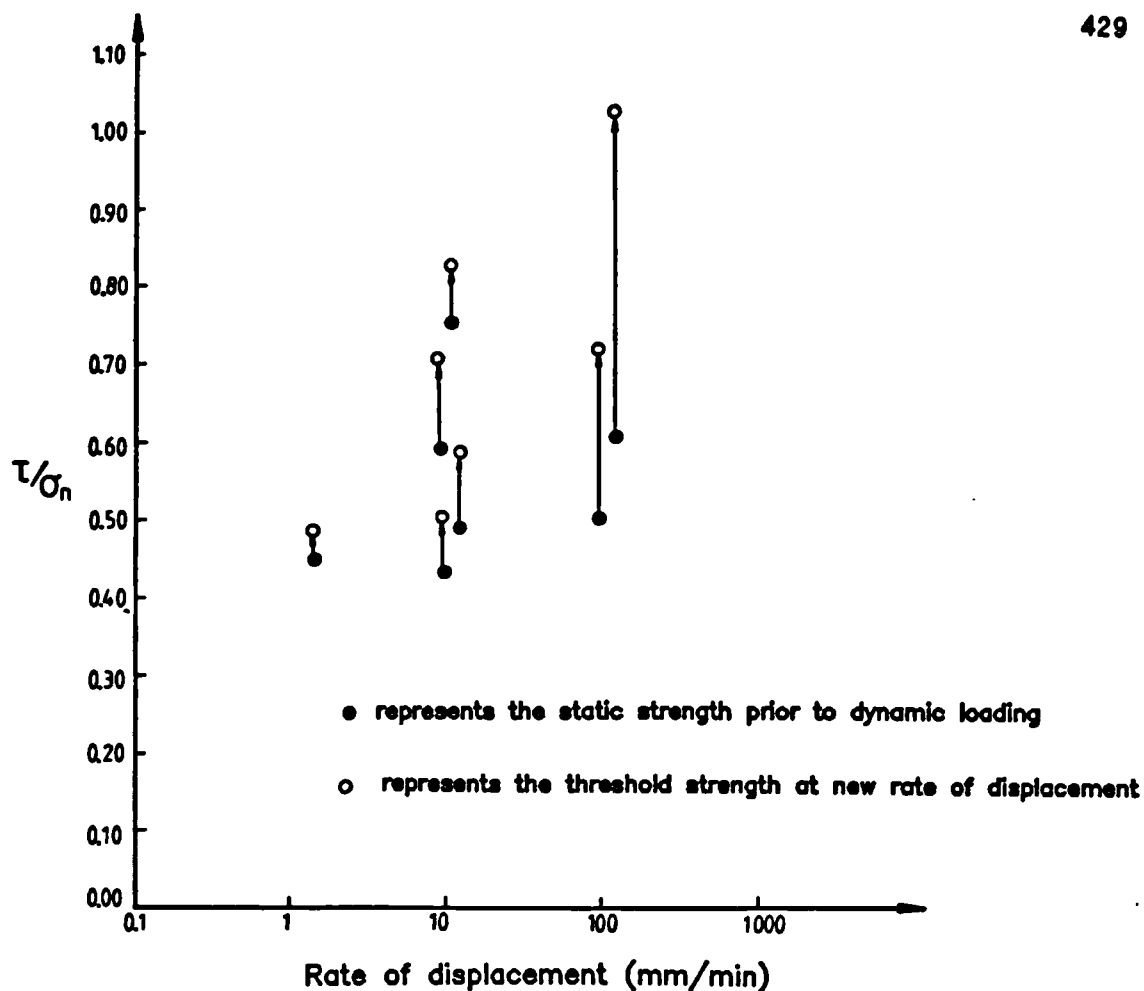


Fig. A2.35a Results of various stages on degraded Tuff ($\sigma_n = 100$ kPa)

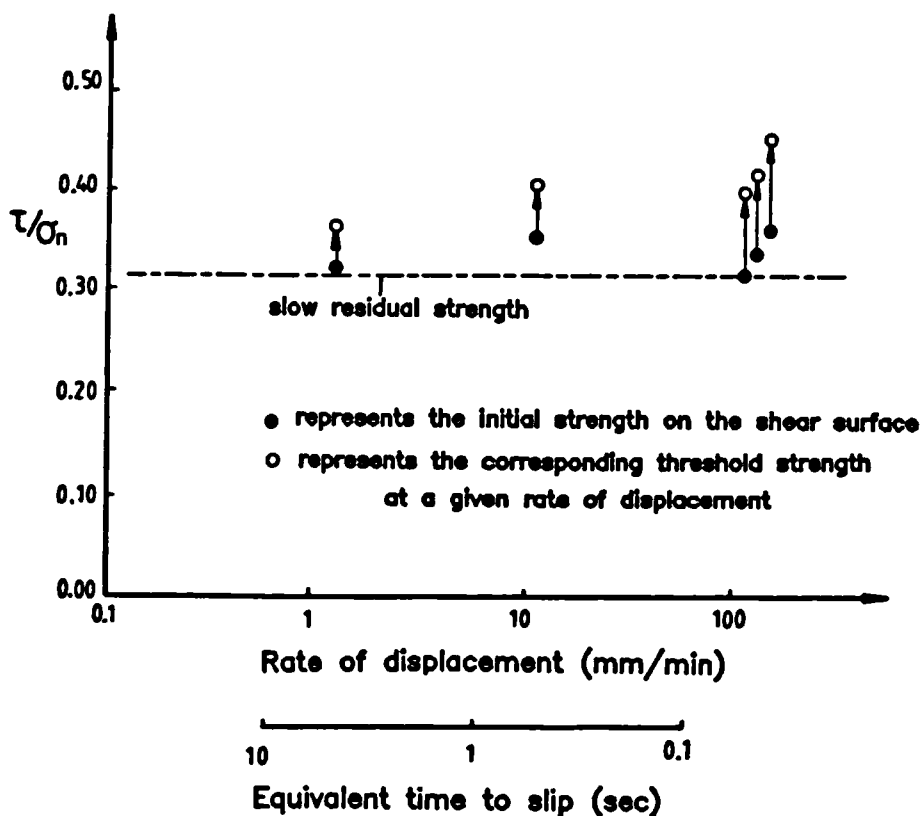


Fig. A2.35b Results of various stages on degraded Tuff ($\sigma_n = 300$ kPa)

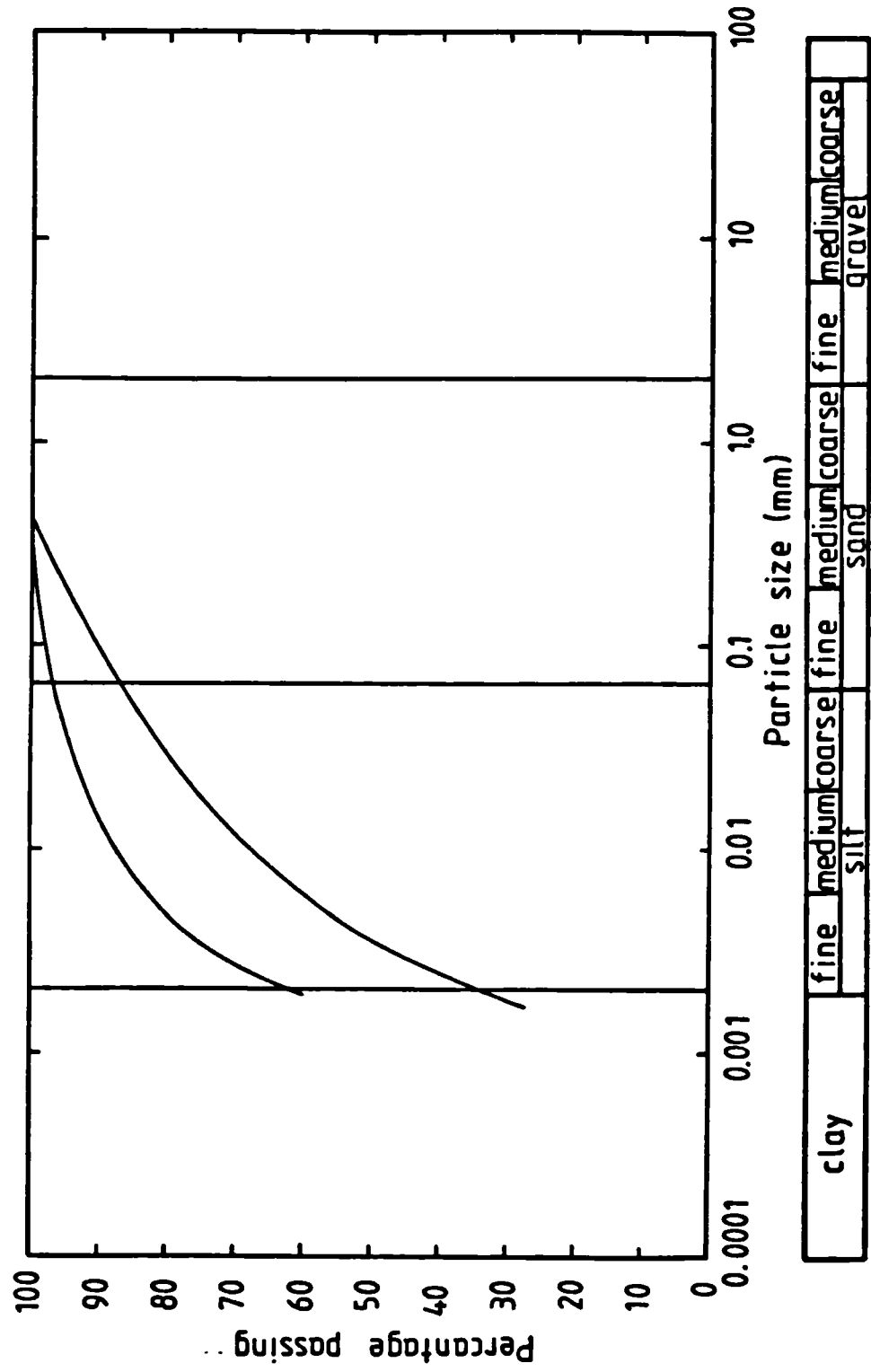


Fig. A2.36 Particle size distribution of varicoloured clays (Battista et al, 1986)

Table A2.30 Results from ring shear test 33 on soil from Senerchia slide

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
1	0.0015	4.0	920.0	peak:0.315 0.300 0.155 0.135 0.102	2.0	
2	3.69	280.0		b,c:0.105 d:0.088 e:0.103	0.0-8.0 46.0	12.5 min
3	0.003	286.0		b:0.125 c:0.140 d:0.113	0.0 136.0	1.02 min
4a	133	422.0		b,c:0.120 d:0.120	0.0 46.0	21 sec
4b	133	468.0		b,c:0.125 d:0.123	0.0 14.0	
4c	133	482.0		0.085 0.095		
5	0.003	486.0	485.0			
6	0.003	562.0	915.0			
7	0.003	802.0	480.0	peak:0.090 0.088		

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
* duration of the particular stage or substage

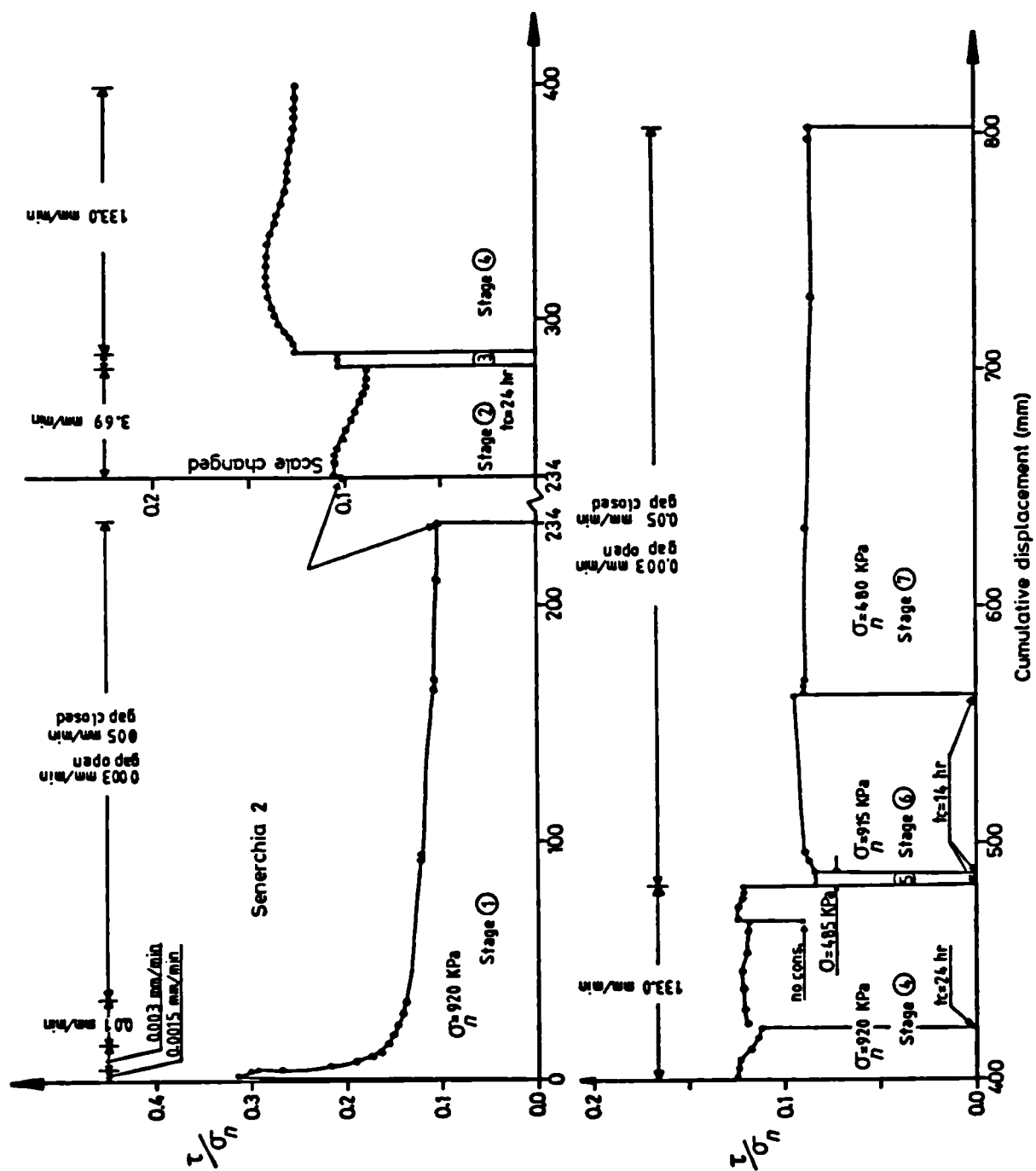


Fig. A2.37a Ring shear test 33 on soil from Senerchia slide

Table A2.30 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
8	0.003	963.0	232.0	peak:0.105 0.085	0.0 160.0-161.0	
9	133	1202.0	232.0	b:0.123 c:0.145 d:0.135	0.0 20.0 40.0-239.0	1.8 min
10	0.003	1218.0	232.0	e:0.105 0.085	16.0	
11	133	1326.0	232.0	b:0.120 c:0.143 d:0.140	0.0 60.0-108.0	48.6 sec
	133	1448.0	232.0	b:0.157 c:0.162 0.138	0.0 122.0	55.2 sec
12	0.003	1459.0	234.0	e:0.125 0.080		
	0.010	1593.0		0.080		

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
* duration of the particular stage or substage

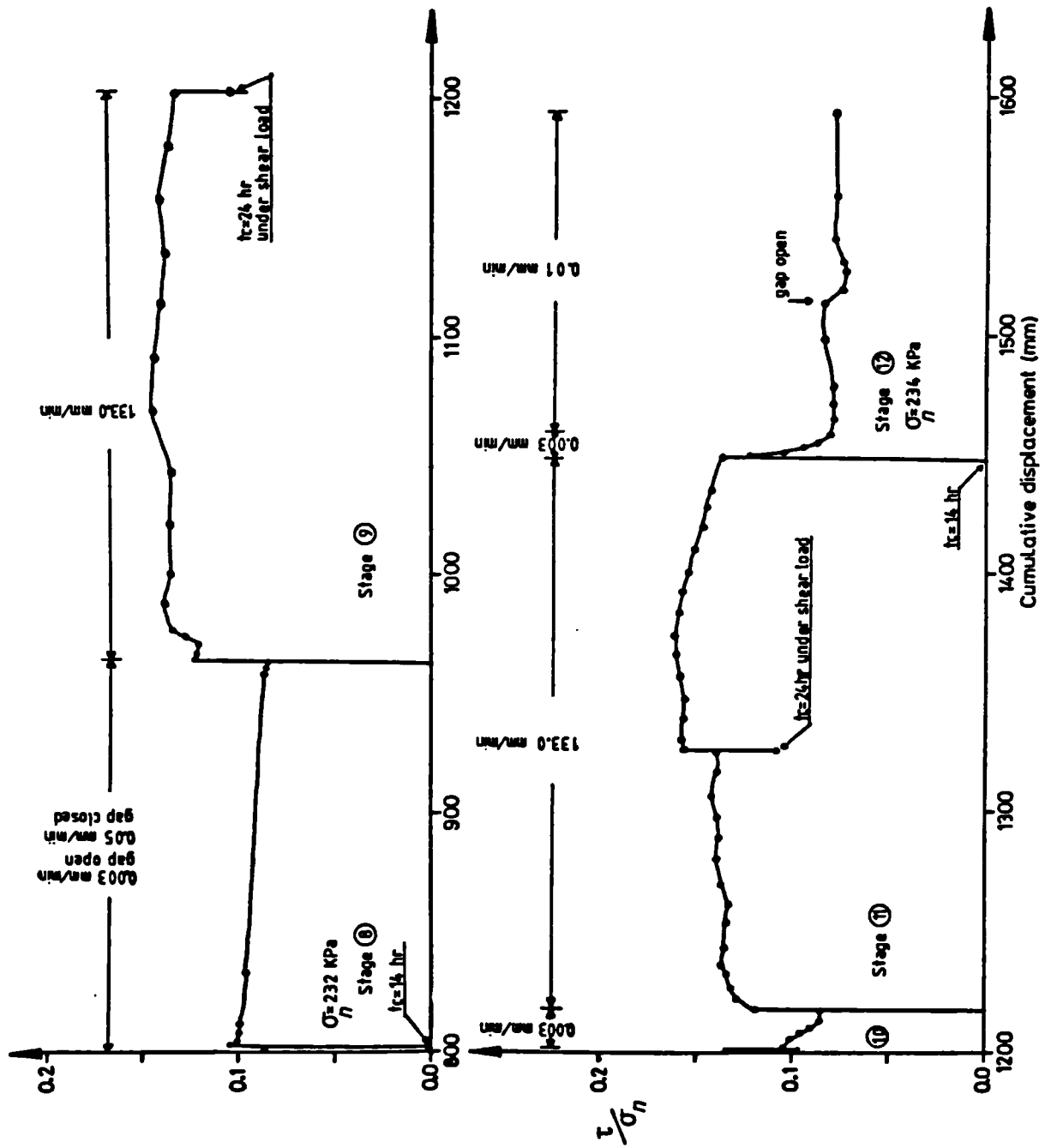


Fig. A2.37b Ring shear test 33 on soil from Senerchia slide

Table A2.31 Results from ring shear test 34 on soil from Buoninvetre slide

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
1	0.001	4.0	910.0	peak:0.380 0.370		
	0.003	38.0		0.185		
	0.003	470.0		0.180		
2	0.003	642.0	490.0	0.190		
3	0.10	650.0	490.0	c:0.190 0.185	0.0 8.0	80 min
4	0.003	654.0	490.0	e:0.185 0.180		
5	1.0	674.0	490.0	c:0.195 0.180		
6	0.003	680.0	490.0	e:0.180 0.180	20.0	20 min
7	10.7	798.0	490.0	b,c:0.220 d:0.200	0.0 20.0-118.0	11 min
	10.7	800.0	490.0	b:0.200 c:0.205 d:0.205	0.0	11.4 sec
	10.7	801.0	490.0	b,c:0.205 d:0.205	0.0	6 sec
	10.7	803.0	490.0	b,c:0.205 d:0.205	0.0	11.4 sec

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
* duration of the particular stage or substage

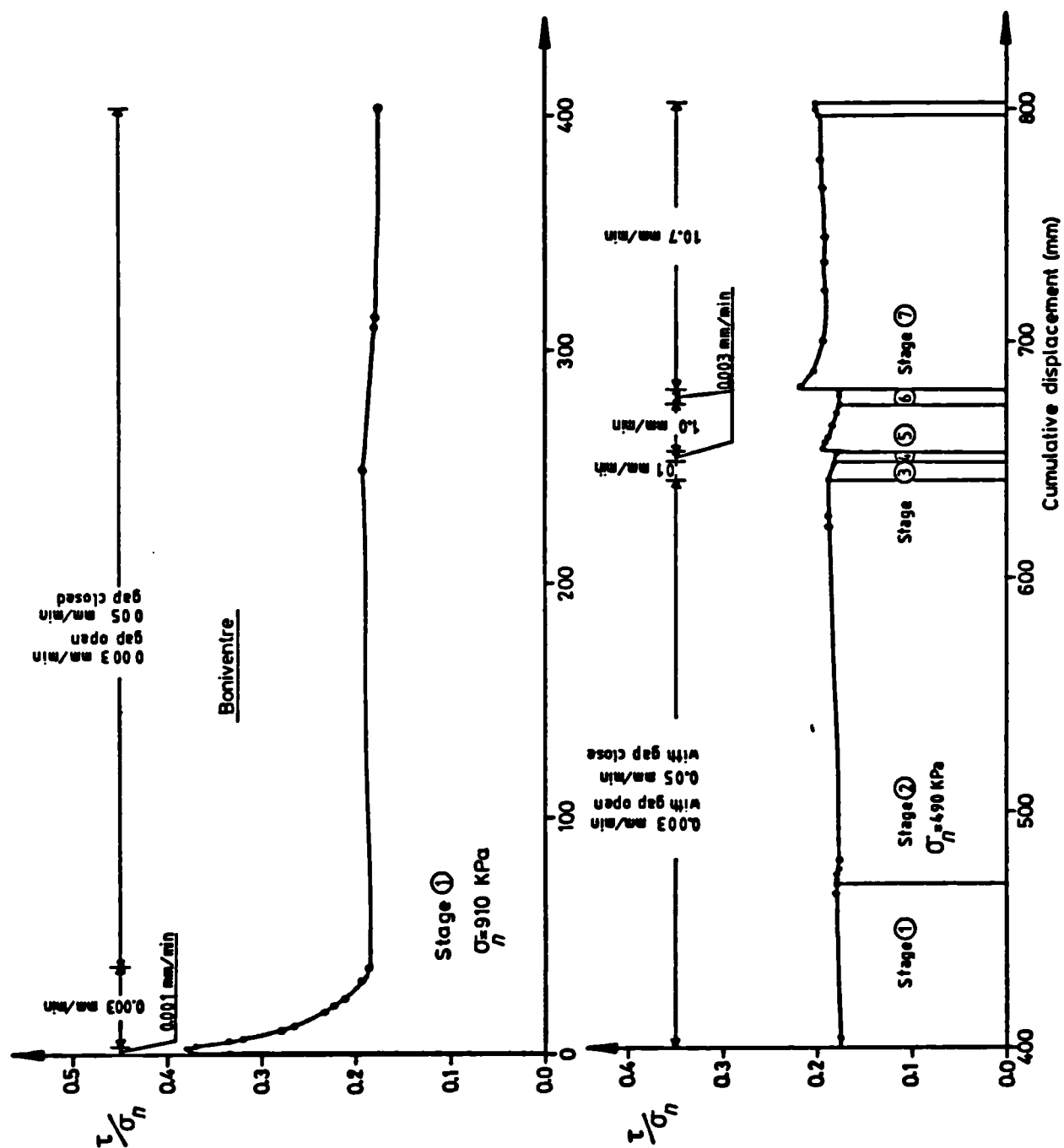


Fig. A2.38a Ring shear test 34 on soil from Buoninvetre slide

Table A2.31 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
8	10.7	810.0	490.0	b,c:0.237 0.220	0.0 7.0	39 sec
9	0.10	816.0	490.0	0.180		
10	0.05	836.0	490.0	e:0.190 0.180	10.0-20.0	
11	100	1065.0	490.0	b:0.260 c:0.360 d:0.293	0.0 229.0	5.4 sec
	100	1074.0	490.0	b,c:0.305 0.300	0.0 9.0	7.8 sec
	100	1087.0	490.0	b,c:0.305 0.300	0.0 13.0	9 sec
	100	1102.0	490.0	b,c:0.305 0.297	0.0 15.0	8.4 sec
	100	1116.0	490.0	b,c:0.310 0.302	0.0 14.0	
12	0.05	1119.0	490.0	e:0.270 0.252		60 min
13	0.003	1208.0	490.0	e:0.232 0.172		
14	400	1402.0	490.0	b:0.322 c:0.382 0.165	0.0 194.0	29.4 sec
15	0.05	1404.0	490.0	e:0.265		
16	400	1483.0	490.0	b,c:0.455 0.262	0.0 79.0	12 sec

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
* duration of the particular stage or substage



Fig. A2.38b Ring shear test 34 on soil from Buoninvetre slide

Table A2.31 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
17	0.003	1685.0	490.0	e:0.275 0.180		
18	0.003	1714.0	200.0	0.182 0.170		
19	0.003	2010.0	200.0	0.180 0.160	296.0	
20	100	2230.0	200.0	b:0.247 c:0.320 d:0.252	0.0	2.2 min
21	0.89	2234.0	200.0	e:0.310 0.282		4.5 min
22	0.89	2237.0	200.0	e:0.320 0.290		3.4 min
23	200	2476.0	200.0	b.c:0.475 d:0.170	0.0 60.0-239.0	1.2 min

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
* duration of the particular stage

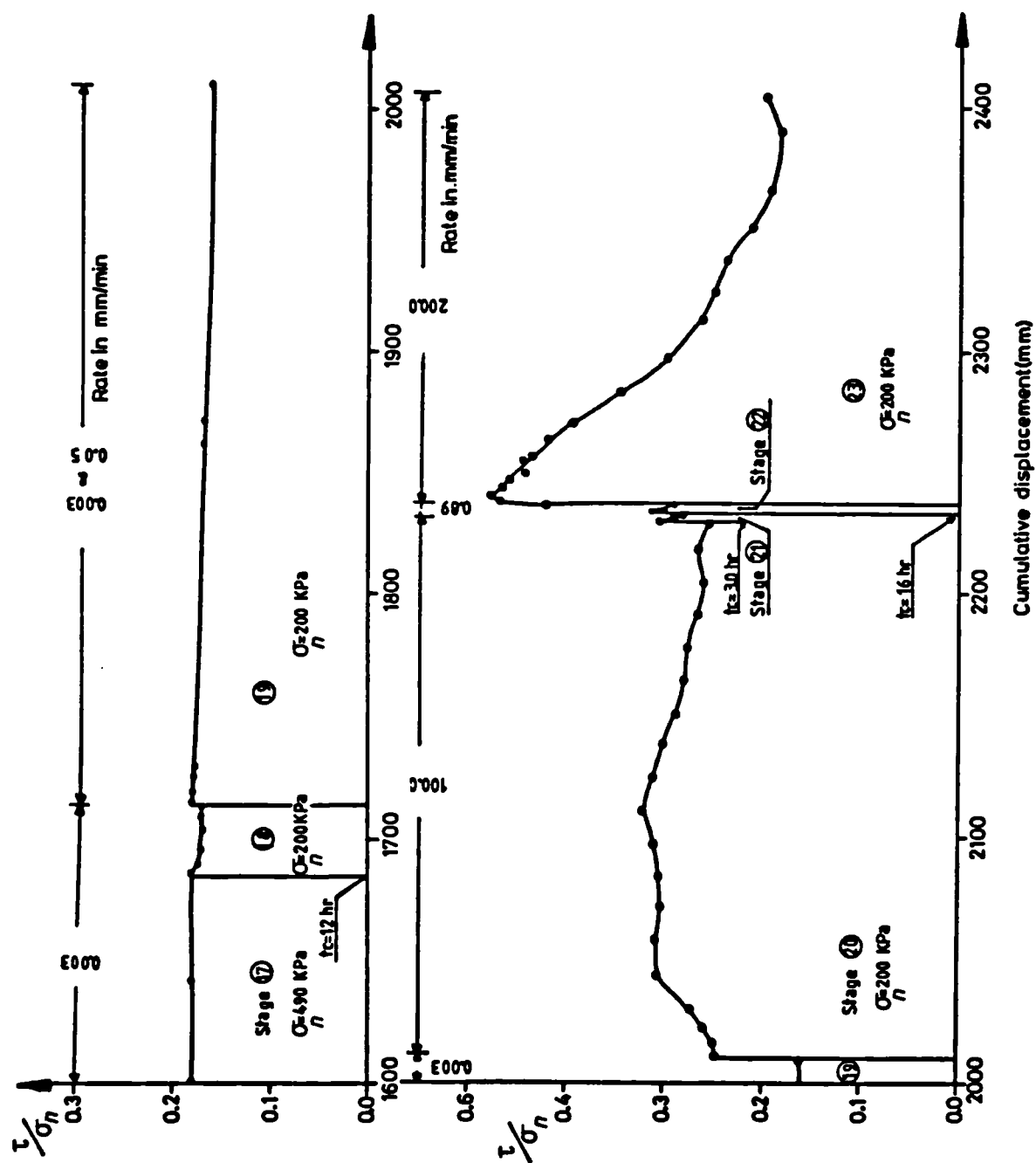


Fig. A2.38c Ring shear test 34 on soil from Buoinvetre slide

Table A2.31 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes *
24	0.003	2498.0	200.0	e:0.332 0.222		
25	400	2608.0	200.0	b:0.400 c:0.475 d:0.215	0.0 110.0	16.8 sec
26	0.005	2612.0	200.0	e:0.335 0.315		

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
* duration of the particular stage

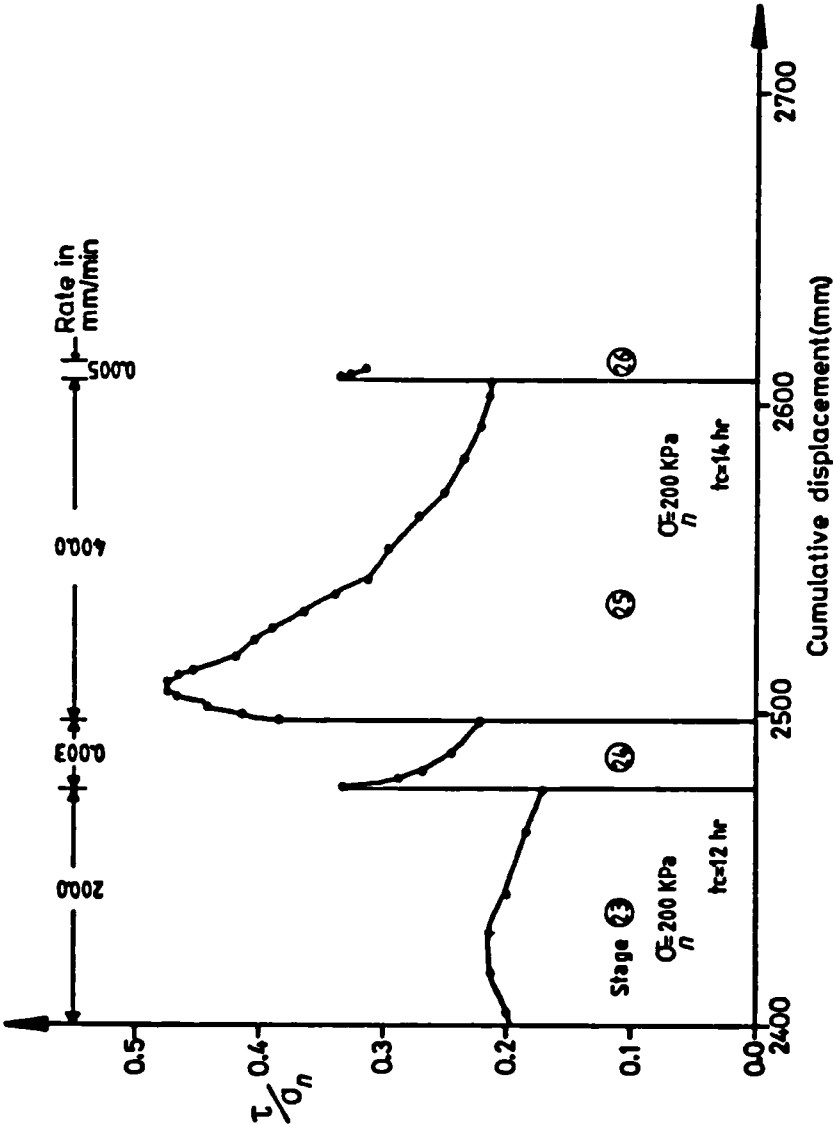


Fig. A2.38d Ring shear test 34 on soil from Buoninvetre slide

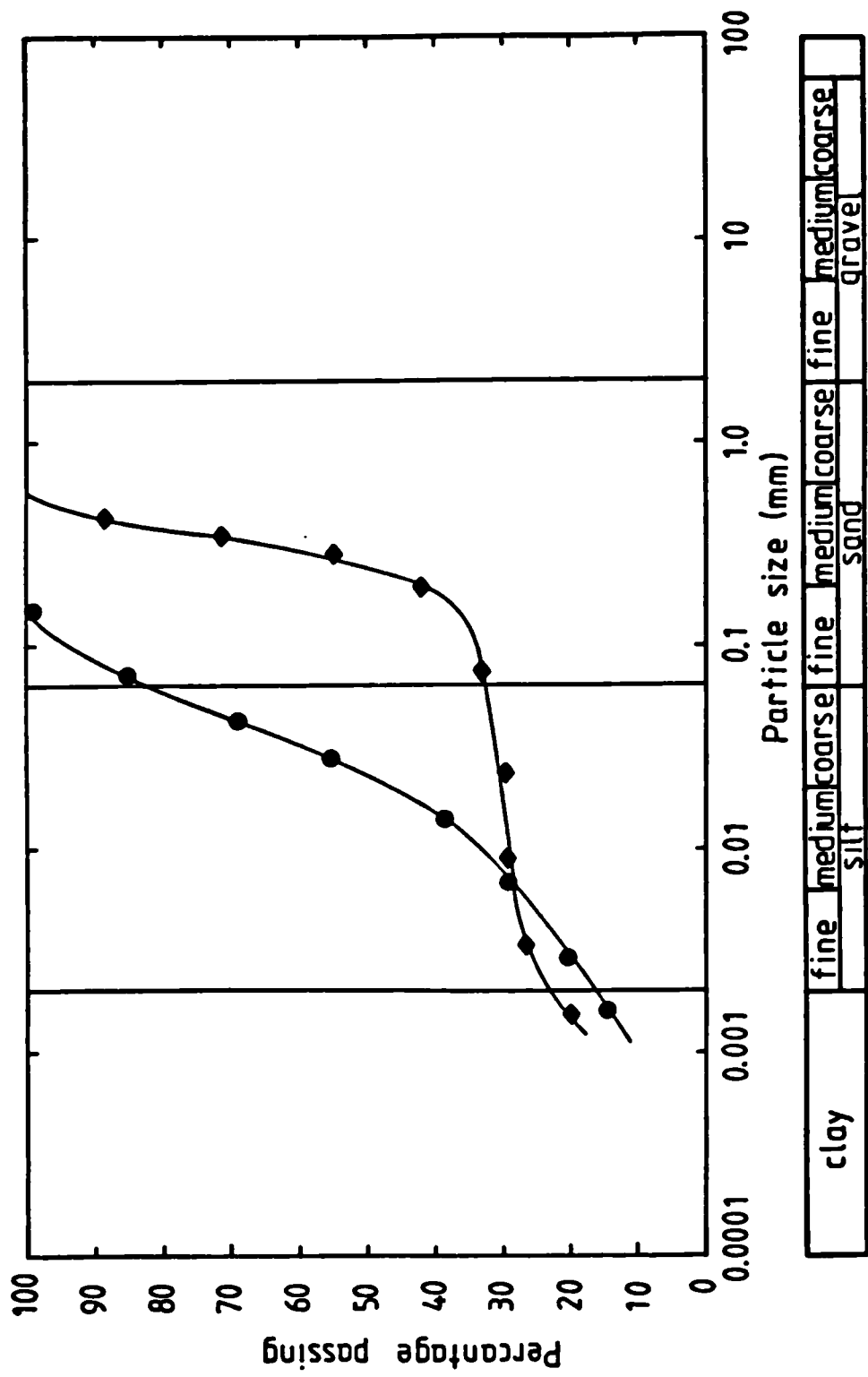


Fig. A2.39 Grading curves of artificial soils

- ◆ sandy clay
- clayey silt

Table A2.32 Results from ring shear test 35 on clayey silt

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0102	75.4	489.7 494.0	peak:0.667 0.637	9.5-10.8 50.6-75.4	
2	370	235.0	453.1 461.1	b,c:0.894 0.688	6.9-7.5 159.5	
3	0.0102	267.6	455.0	e:0.654	15.0-32.7	
4	1100	432.0	460.5 467.9	b,c:1.012 0.783	1.1 164.3	
5	0.0102	460.0	461.4 458.5	e:0.678 0.670	5.4-5.7 20.4-28.0	
6	2500	626.2	467.5 482.0 484.0 486.6	b,c:1.154 d:0.696 0.770 0.729	1.0 78.5 130.9 165.5	
7	0.0102	654.4	472.0	e:0.660	6.0-28.1	
8	0.0102	683.4	59.5 66.3	peak:0.853 0.672	0.3 26.7-29.0	
9	370	831.2	65.40 61.0 62.3	b,c:1.197 d:0.994 1.008	2.8 86.8 147.8	
10	0.0102	949.0	61.0 58.0	e:0.690 0.655	0.7 10.0-117.8	
11	6.0	1066.3	54.0 55.0	b,c:0.936 d:0.639	0.2 0.3 74.2-86.2	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

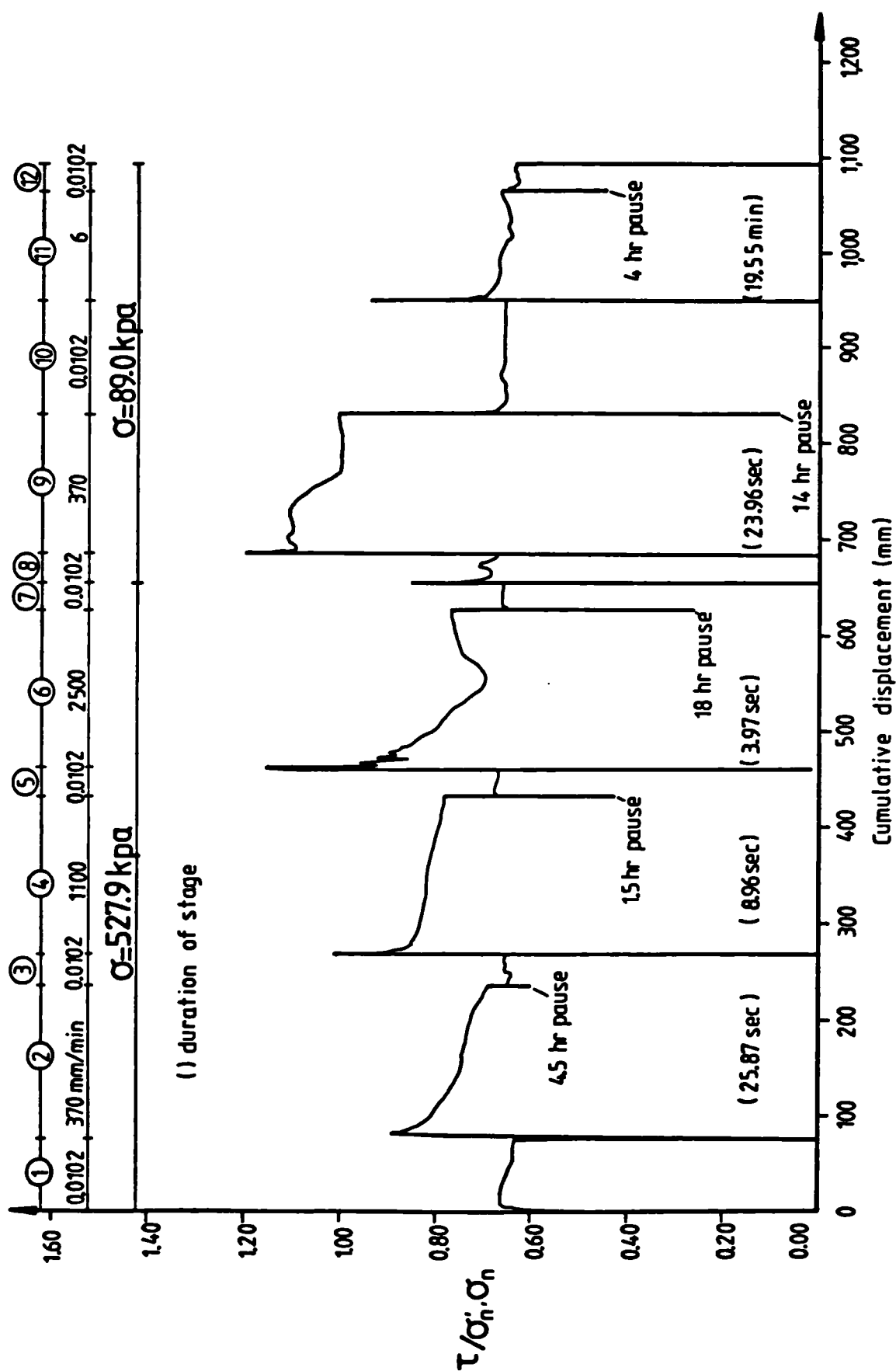


Fig. A2.40a Ring shear test 35 on clayey silt

Table A2.32 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
12	0.0102	1094.2	54.7 54.5	e:0.653 0.630	0.7 4-27.9	
13	1100	1259.5	59.6 65.0	b,c:0.300 0.515	0.1 165.3	
14	0.0102	1296.7	58.0 58.5	e:0.678 0.680	6.9-20 20.0-37.2	
15	2500	1531.4	59.1 64.5	b,c:1.750 d:0.397-0.387	0.2 135-234.7	
16	0.0102	1561.0	64	e:0.678	8.0-29.5	
17	0.0102	1580.9	488.8 484.7	0.674 0.672	2.8-3.2 36.0-19.8	
18	9.3	1765.0	477.2 477.4 469.6	b,c:0.888 d:0.660 0.674	0.9 74.4 184.1	
19	0.0102	1786.4	468.0	e:0.650	22.0-21.3	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

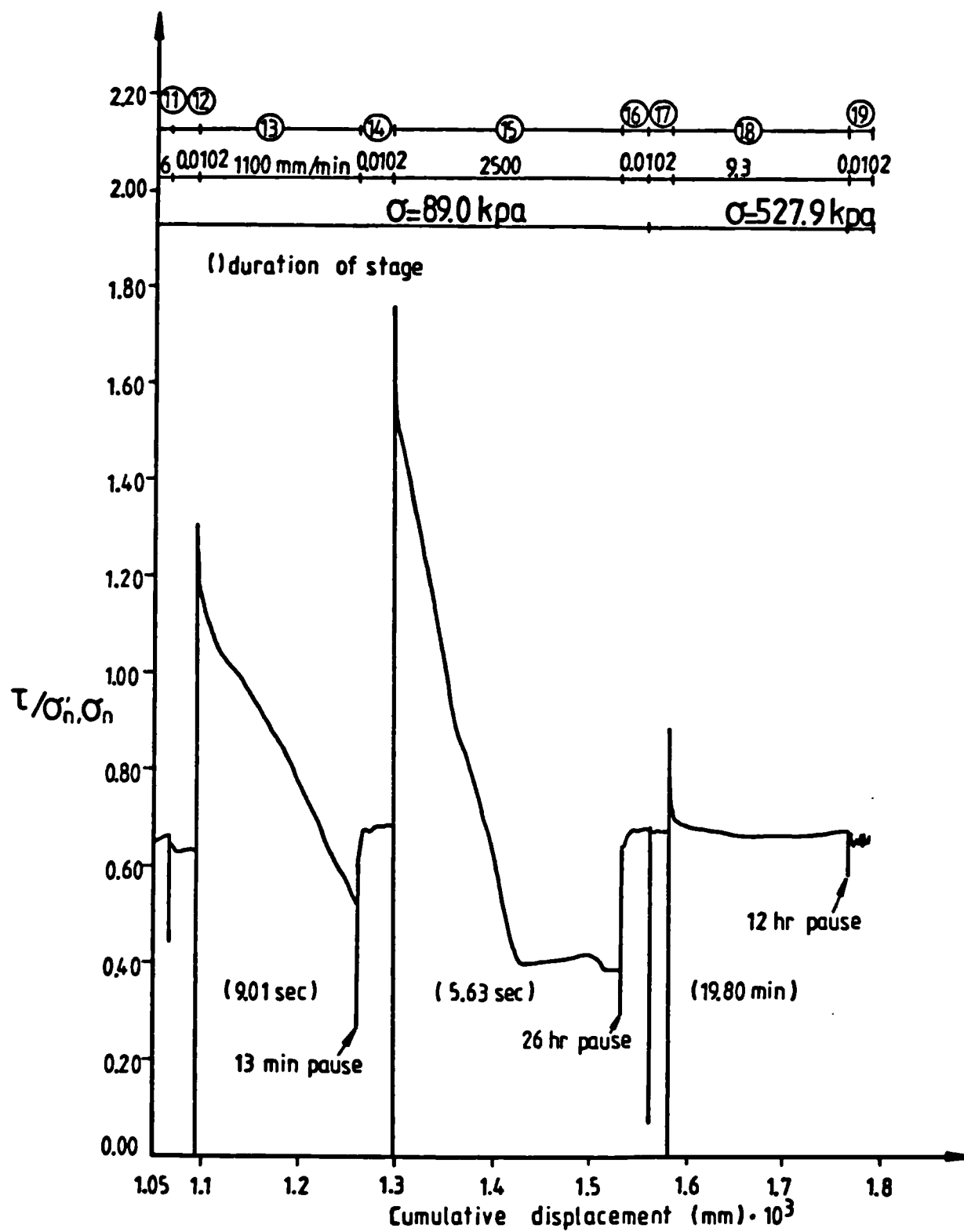


Fig. A2.40b Ring shear test 35 on clayey silt

Table A2.33 Results from ring shear test 36 on sandy clay

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.1020	54.0	195	peak:0.560	11.0-54.0	
2	130	267.8	204.2 203.0 205.0	b,c:0.657 d:0.538 0.578	0.4 23.0-26.9 194.1-213.8	
3	0.1020	412.4	204.8 196.0 200.0	e:0.555 0.565 0.579	0.3 2-32 41-144.6	
4	1100	587.7	175.0 203.4 199.0	b,c:0.815 d:0.681 0.693	0.5-1.9 148.6-159.6 165.0-175.2	
5	0.1020	729.0	196.6 196.5 180.0	e:0.582 0.541 0.565	0.5 1.5 7.3-141.3	
6	1100	932.4	458.3 451.8	0.465 0.570	0.0 203.4	
7	0.1020	1032.7	443.9 435.0	e:0.528 0.577	0.7 76.0-100.4	
8	1100	1227.2	398.7 447.2	b,c:0.822 d:0.688	1.1 170.0-193.9	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

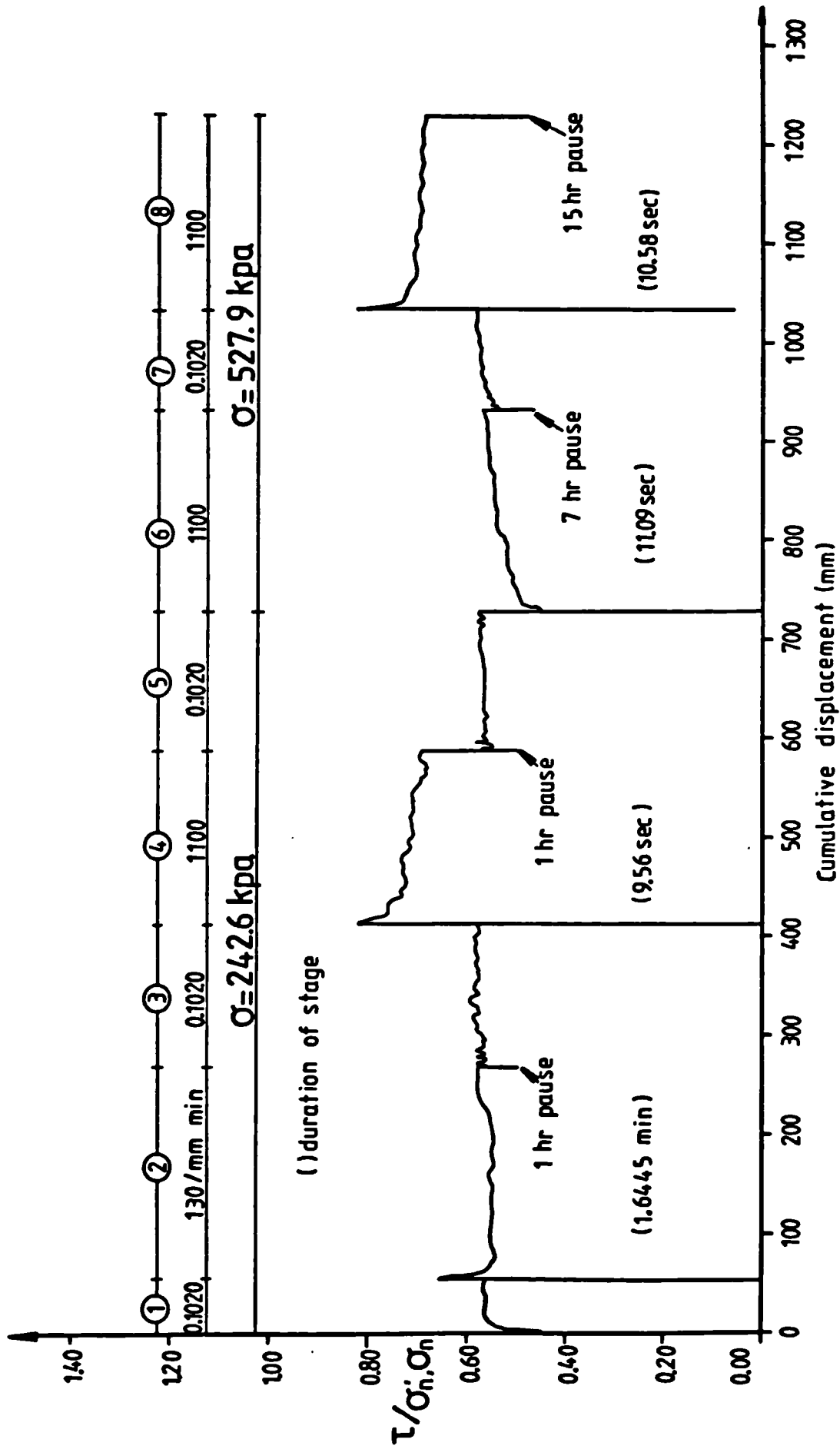


Fig. A2.41a Ring shear test 36 on sandy clay

Table A2.33 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
9	0.1020	1422.4	457.0	e:0.545	2.6-16.5	
10	100	1576.8	443.0	0.570	122.9-195.2	
11	0.1020	1693.8	448.9	b,c:0.748	0.7	
			470.0	d:0.605	153.5-154.4	
12	3	1887.5	466.6	e:0.548	0.4	
			455.0	0.565	25.8-116.9	
13	0.1020	2050.7	461.3	b,c:0.630	1.2	
			470.0	d:0.578	5-193.7	
14	2500	2337.1	465.0	e:0.581	0.5	
			460.0	0.575	5.0-163.2	
15	0.1020	2342.3	462.9	b,c:0.867	0.8	
			489.6	d:0.682	112.3	
16	0.1020	2470.4	480.2	e:0.593	1.0	
			478.0	0.580	4.0-104.5	
			228.3	peak:0.724	1.1	
			248.5	0.605	4.0-128.6	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

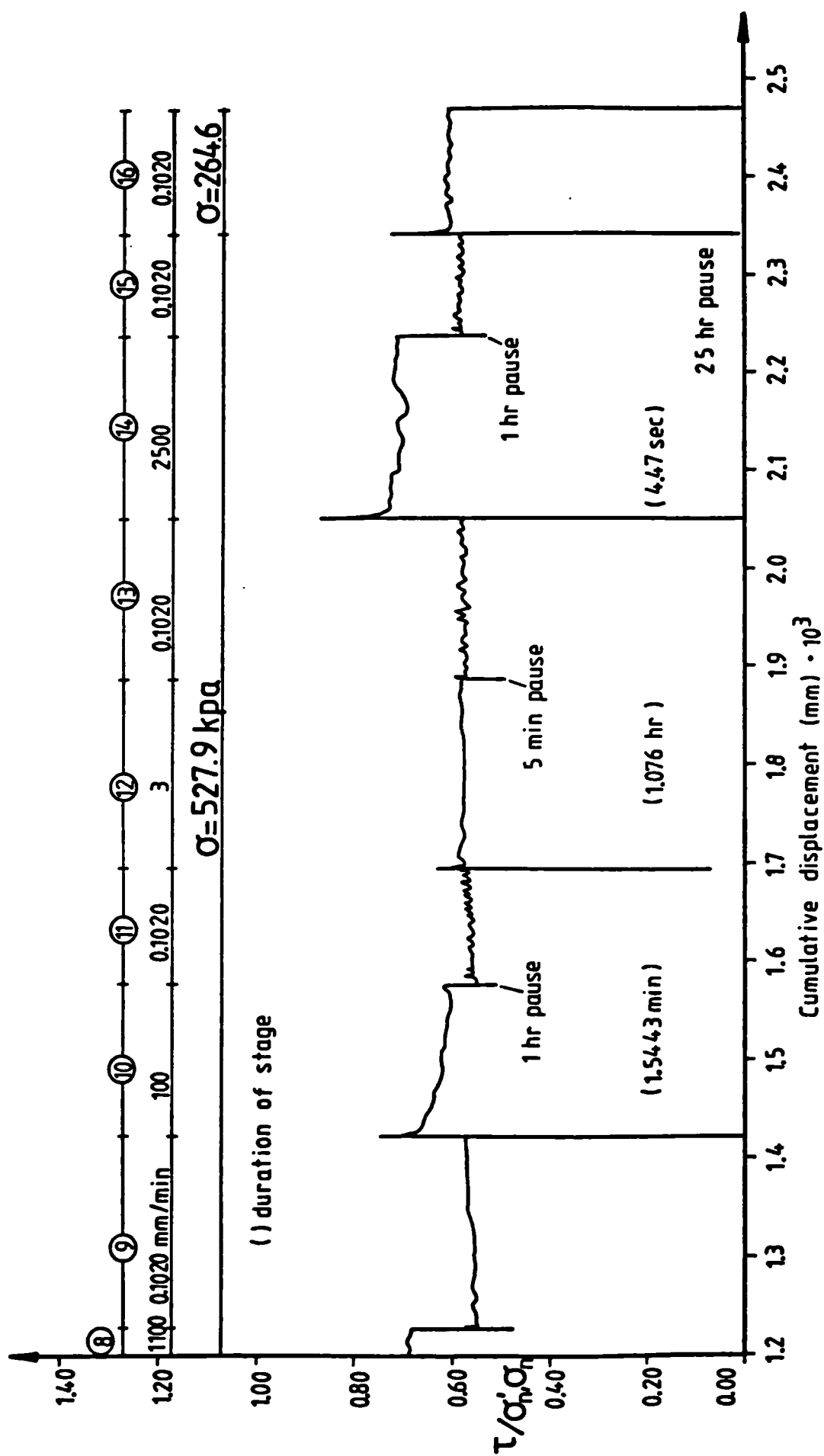


Fig. A2.41b Ring shear test 36 on sandy clay

Table A2.33 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
17	1100	2647.9	215.4 237.0	b,c:0.853 d:0.645	0.7 146.7-177.5	
18	0.1020	2700.0	221.0	e:0.610	1.0-52.1	
19	2500	2893.5	223.5 232.5	b,c:0.843 0.719	0.8 20.0-40.3	
20	0.1020	2967.1	237.9 228.9	d:0.607 e:0.612	193.5 2.3	
21	3	3143.4	220.8 214.9	0.600 b,c:0.663	3.0-73.6 0.5	
22	0.1020	3213.0	220.0 218.0	d:0.600 e:0.622	10.0-176.3 0.5-0.7	
23	100	3362.9	212.0 631.6	0.600 0.424	9.7-69.6 0.0	
24	0.1020	3463.8	629.5 428.5	0.585 peak:0.617	139.3-149.8 0.6	
25	130	3663.5	442.0 222.5	0.600 b,c:1.246	0.8-101.0 0.8	
			249.5	d:0.630	99.4-199.7	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

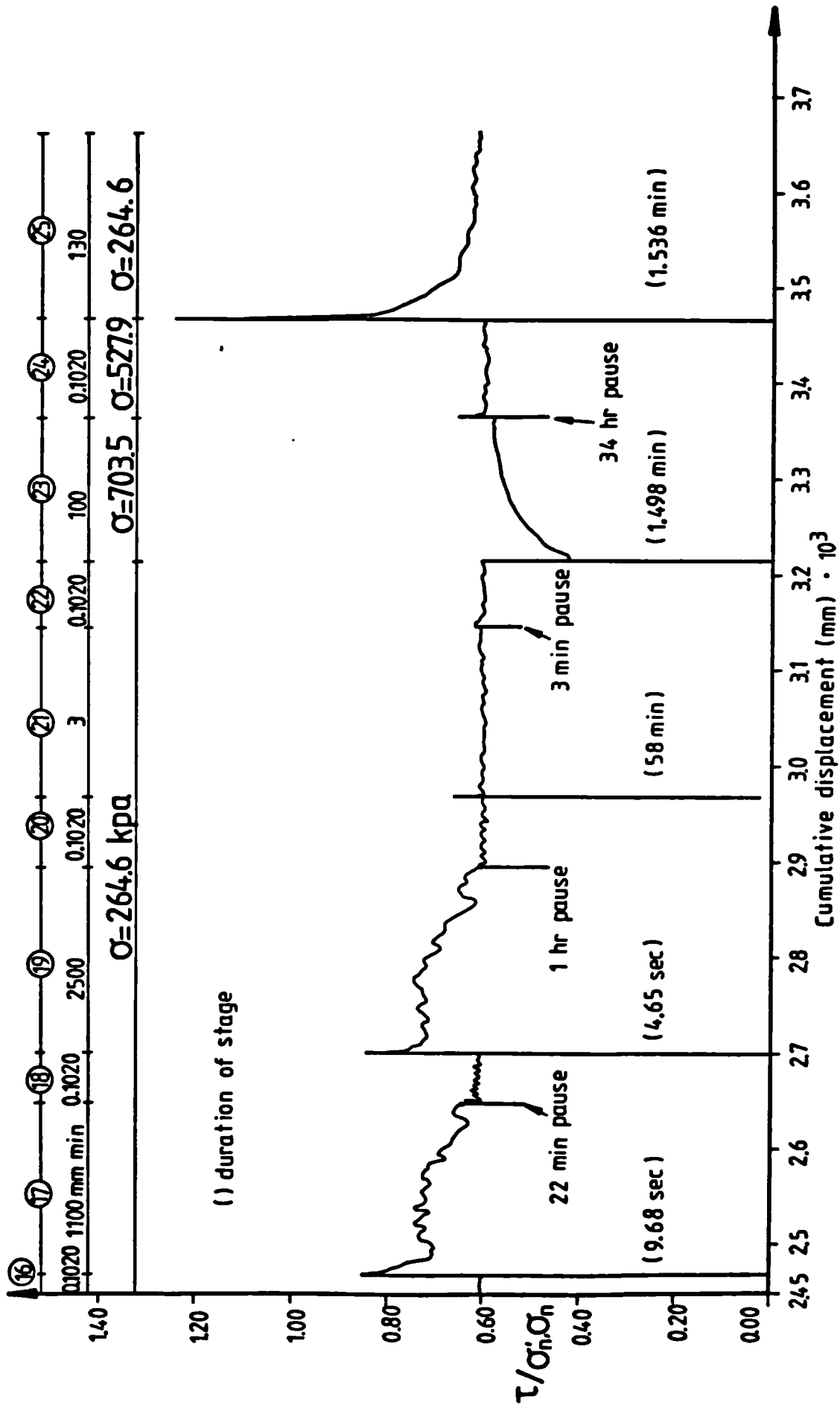


Fig. A2.41c Ring shear test 36 on sandy clay

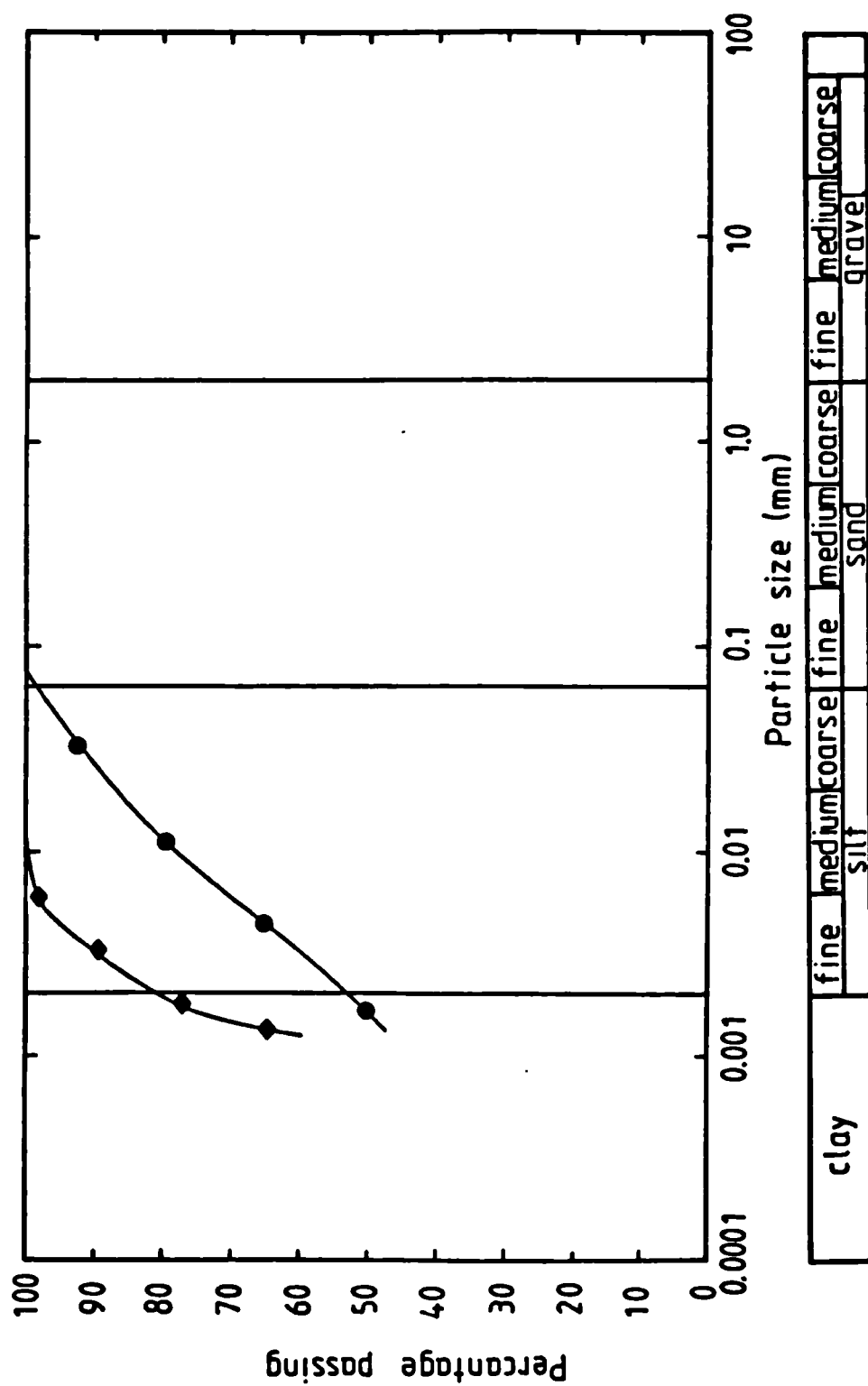


Fig. A2.42 Grading curves of kaolin and blue London clay

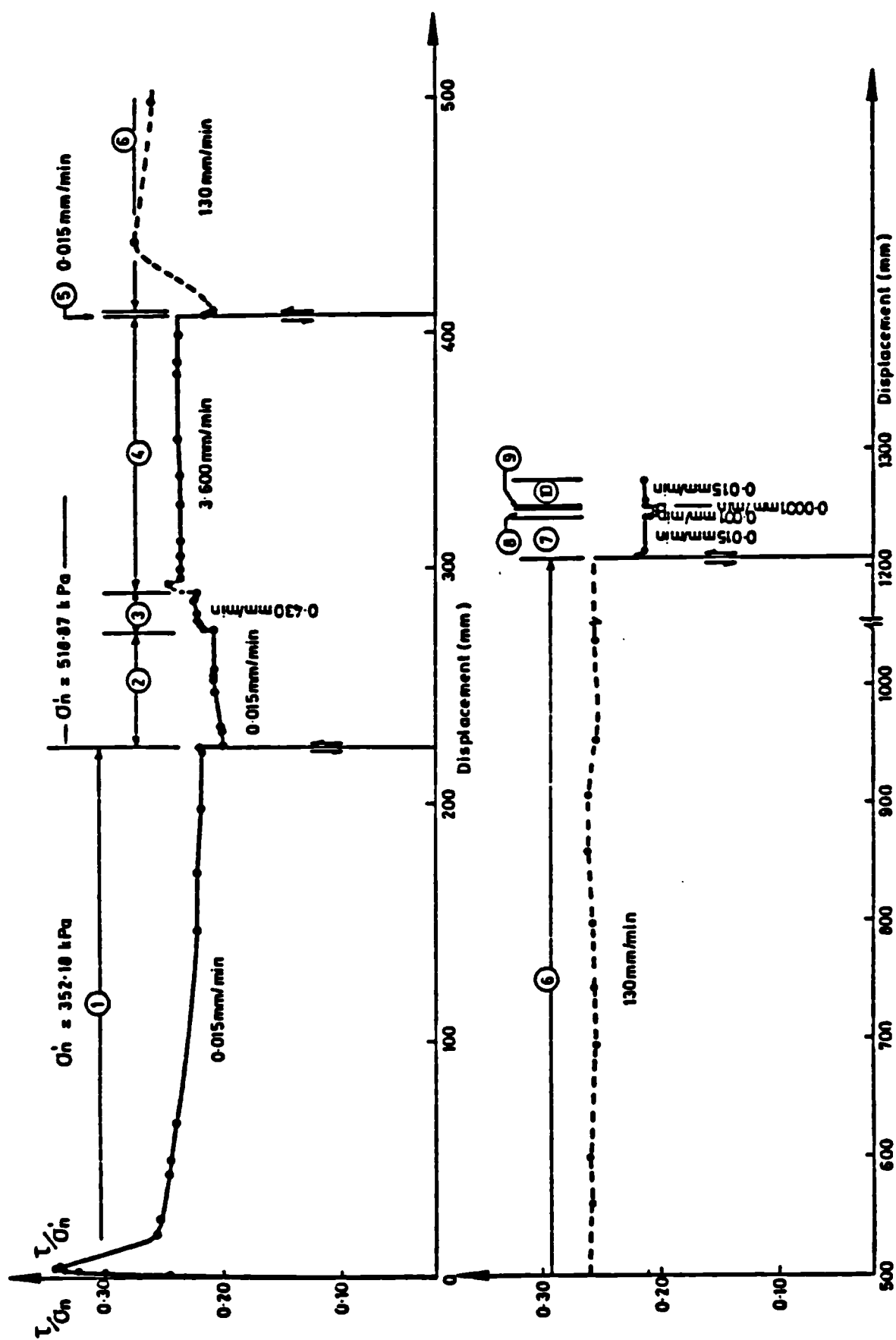


Fig. A2.43a Ring shear test 37 on kaolin

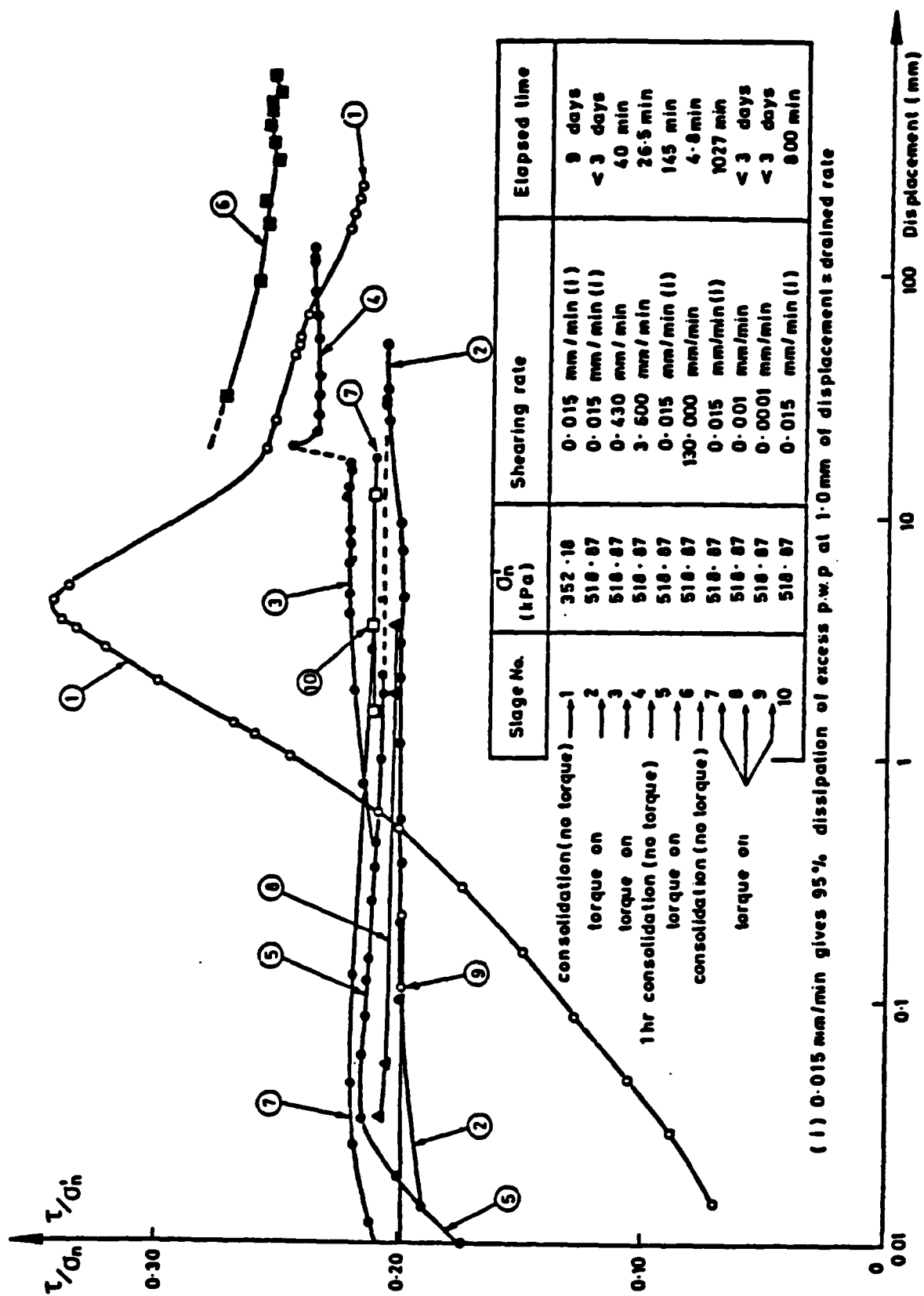


Fig. A2.43b Ring shear test 37 on kaolin

Table A2.34 Results from test 38 on blue London clay

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.00625	23.7	459.7	peak 0.395	2.7	
			474.0	0.222	23.7	
	0.01362	166.5	497.0	0.197	166.5	
			490.0	0.178	556.9	
2	0.1440	981.5		Gap closed		
	0.00625	1005.8	415.0	0.195		

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

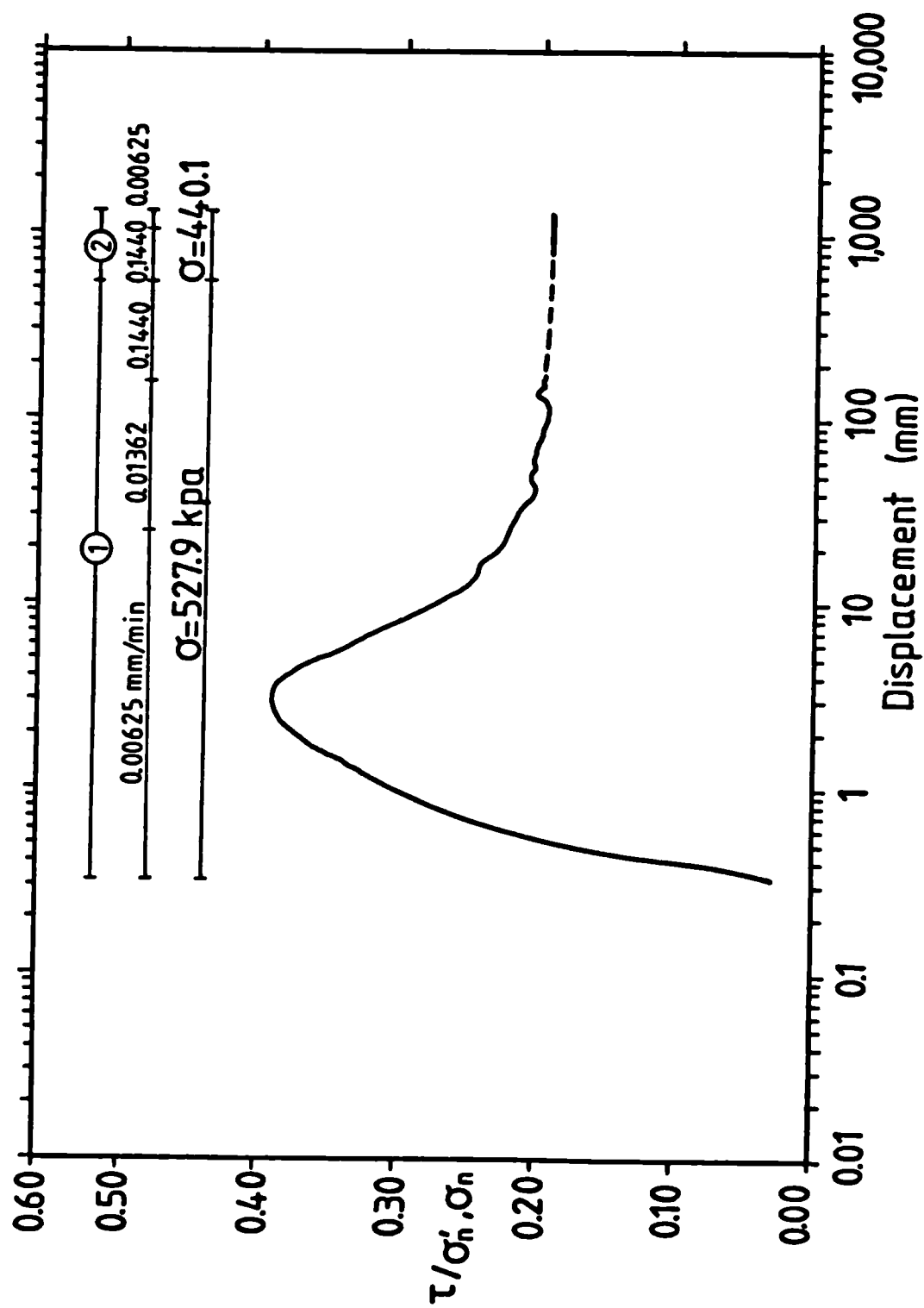


Fig. A2.44a Ring shear test 38 on blue London clay

Table A2.34 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
3	1.1395	1228.5	429.5 431.5	b,c:0.208 0.195	0.6-0.8 36.0-222.8	
4	0.00625	1262.6	428.0 427.0 427.0	e:0.186 0.185 0.193	0.2 0.6 21.7-34.0	
5	11	1459.0	430.1 430.0 419.5	b:0.210 c:0.215 d:0.192	0.1 8.0-10.0 123.5-196.4	
6	0.00625	1484.8	422.9 424.0	e:0.181 0.195	0.5 18.9-206.7	
7	100	1691.6	426.0 422.6 438.0	b:0.214 c:0.307 d:0.197	0.5 7.2-17.9 202.1-206.7	
8	0.00625	1720.5	411.5 400.0	e:0.188 0.181	0.2 0.3-28.9	
9	370	1917.0	402.8 396.7 414.0	b:0.194 c:0.312 d:0.250	2.4 40.3 191.0-196.6	
10	0.00625	1948.5	411.5 395.0	e:0.227 0.190	0.3 9.9-31.5	
11	1100	2171.94	398.8 402.0 418.0	b:0.301 c:0.357 d:0.202	0.0 10.5-11.6 177.2-223.4	
12	0.00625	2216.0	415.1 410.0 400.0	e:0.224 0.195 0.184	0.4-0.5 4.0-5.0 11.0-44.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

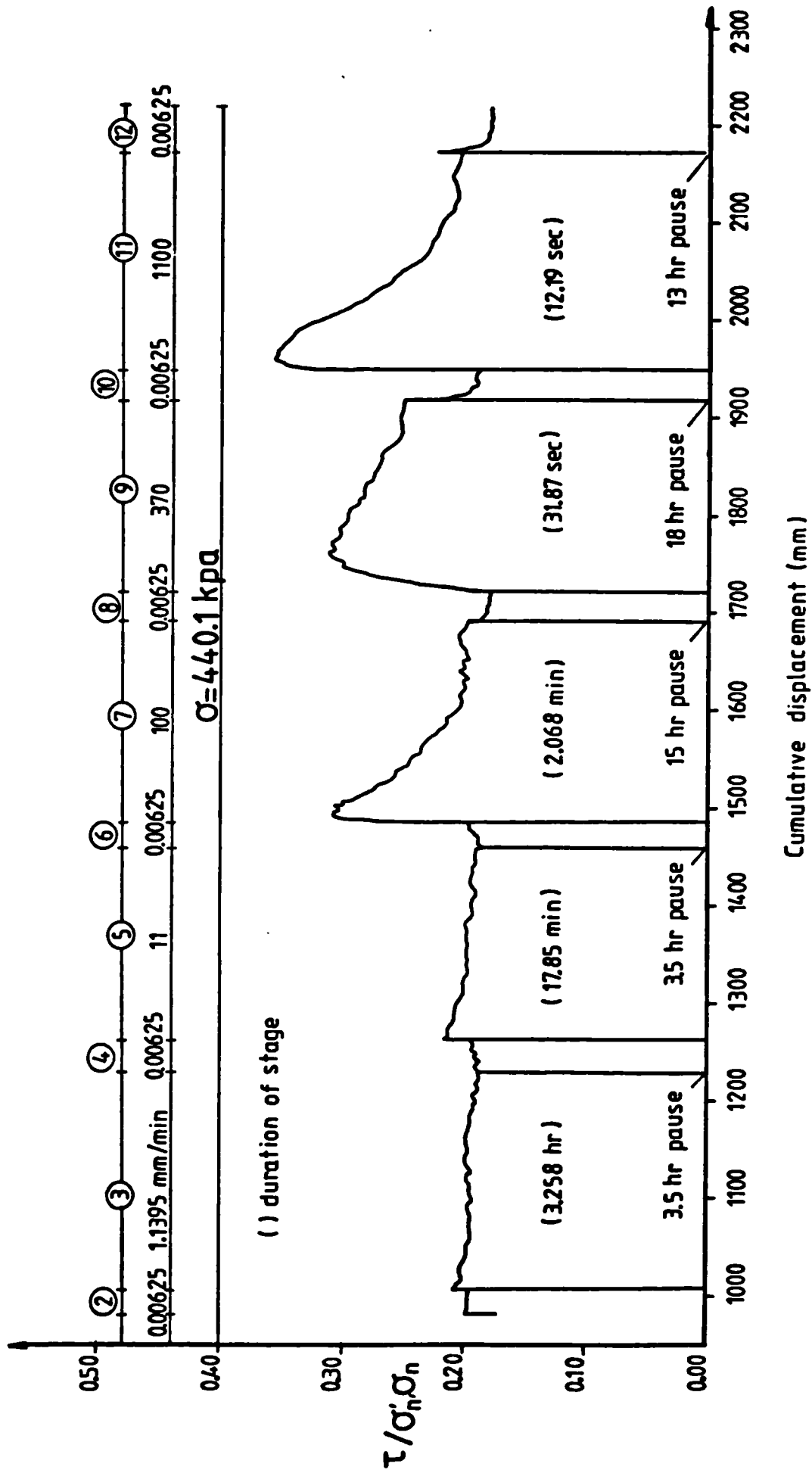


Fig. A2.44b Ring shear test 38 on blue London clay

Table A2.35 Results from ring shear test 39 on blue London clay

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.00625	17.2	249.7	peak: 0.459	1.4	
	0.44061	1862.5	226.5	0.240	17.2	
	0.00625	1937.2	248.0	Gap closed 0.203	1862.5–1937.2	

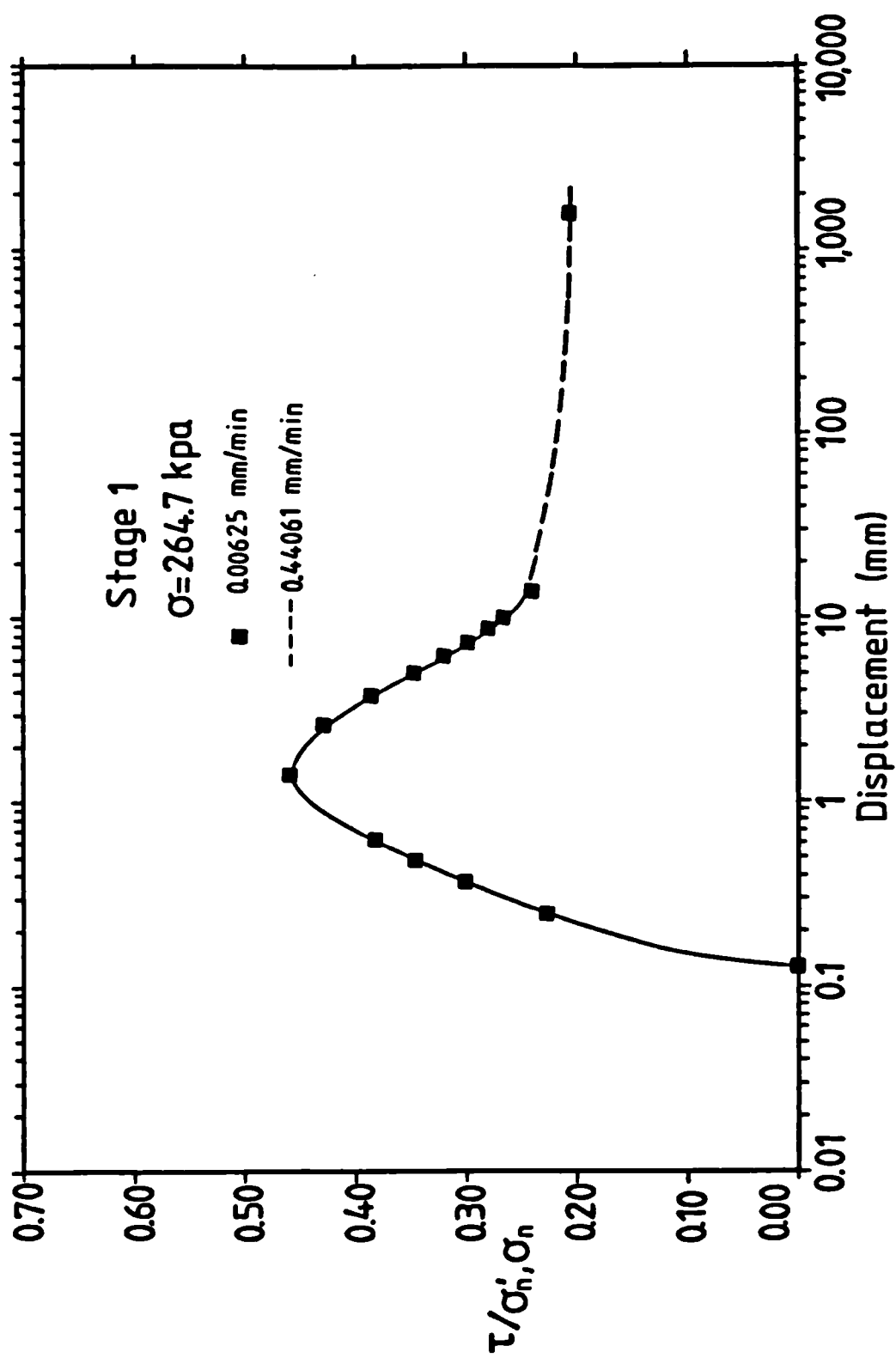


Fig. A2.45a Ring shear test 39 on blue London clay

Table A2.35 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
2	1.1218	2170.5	250.5 255.0	c:0.197 0.195	0.3 0.7-233.3	
3	0.00625	2226.5	250.1 247.0 249.0 244.0	e:0.203 0.191 0.197 0.205	0.4 4.0 8-9 30-55.9	
4	11	2429.6	244.2 244.5 244.9	b:0.235 c:0.237 d:0.211	0.11 24.8-50.4 168.5-203.1	
5	0.00625	2469.5	247.0	e:0.190	0.8-39.9	
6	100	2683.6	251.6 253.4 252.5	b:0.238 c:0.340 0.223	0.2 18.0-24.8 214.2	
7	0.00625	2735.3	251.6 247.2	e:0.225 0.192	0.2 18.3-51.6	
8	370	2994.6	250.5 253.0 257.0 259.0	b:0.232 c:0.371 0.239 d:0.212	1.2 21.6-22.1 200.0 235.0-259.3	
9	0.00625	3044.6	255.2 252.0	e:0.243 0.205	1.0 24.5-50.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

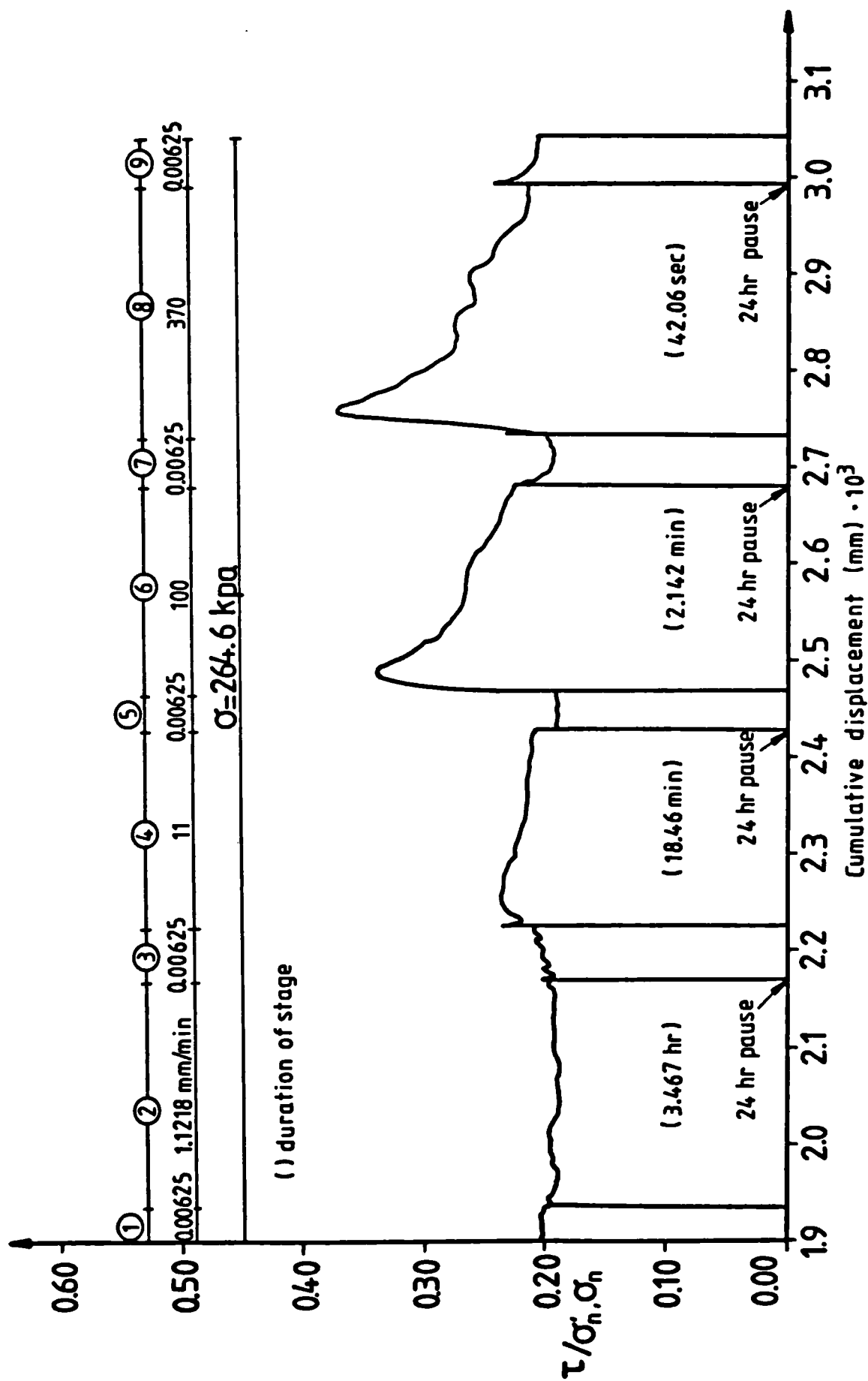


Fig. A2.45b Ring shear test 39 on blue London clay

Table A2.35 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
10	1100	3272.9	252.5 252.5 252.5	b:0.301 c:0.380 d:0.186	0.18 8.6 162.0-228.3	
11	0.00625	3326.6	254.2 252.0	e:0.225 0.187	0.8 14.0-53.7	
12	6200	3604.1	259.6 253.5 255.9 238.5	b:0.403 c:0.430 0.229 0.282	0.4 21.7 112.2 147.1	0 14.4
13	0.00625	3756.7	253.0 256.9 253.0	d:0.177-0.200 e:0.236 0.178	225.9-277.5 0.5 29.7-97.2	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

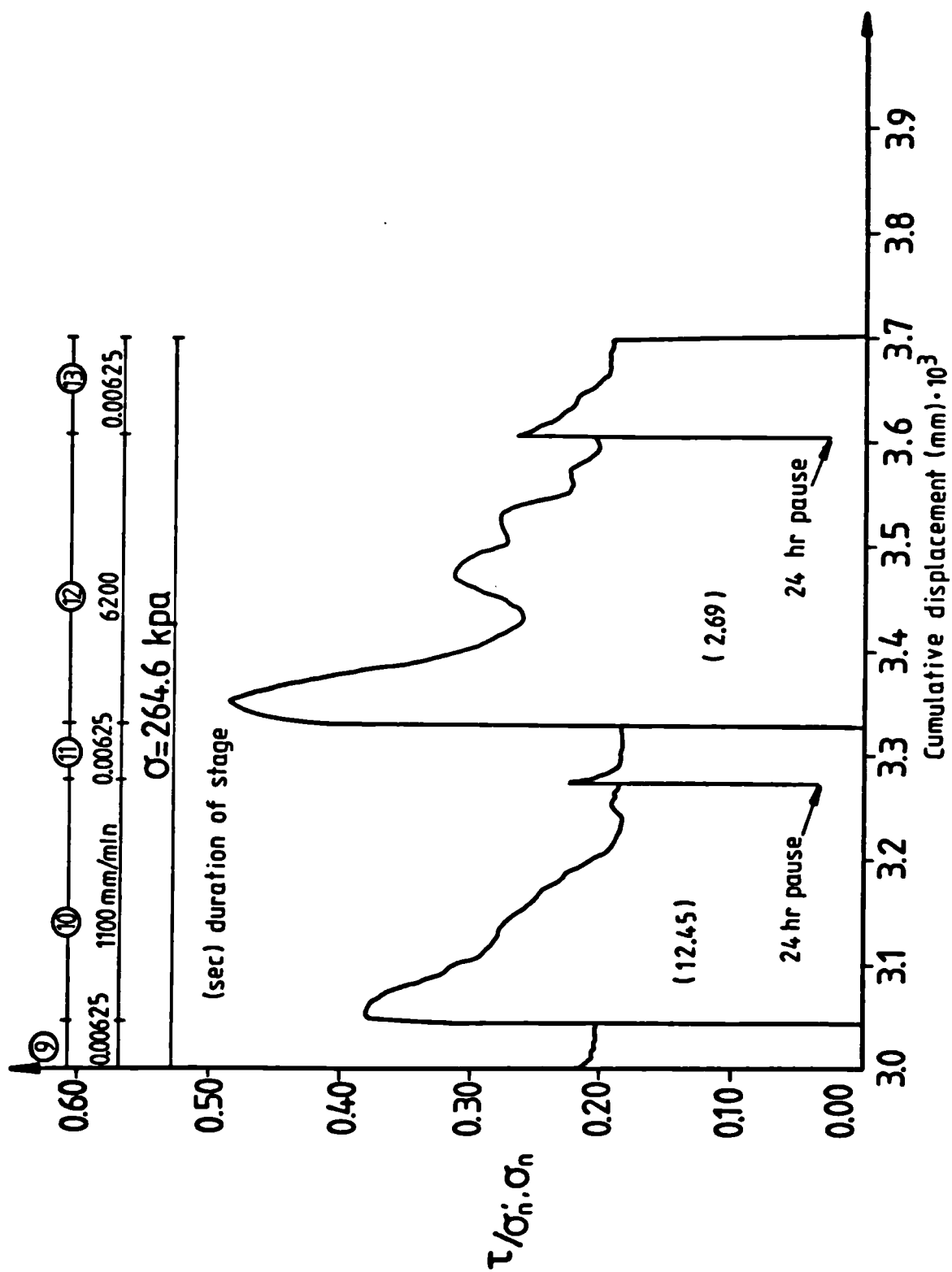


Fig. A2.45c Ring shear test 39 on blue London clay

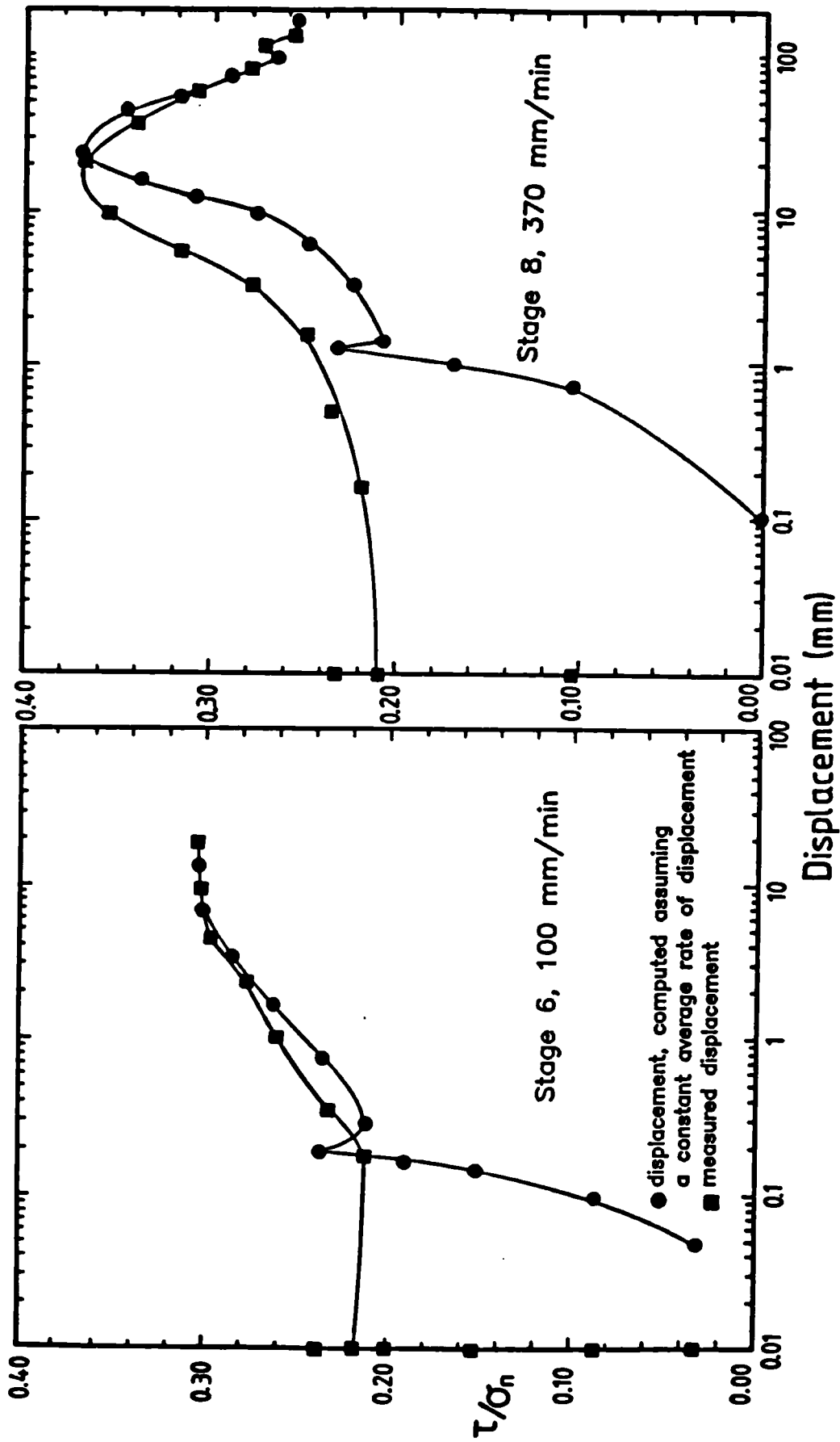
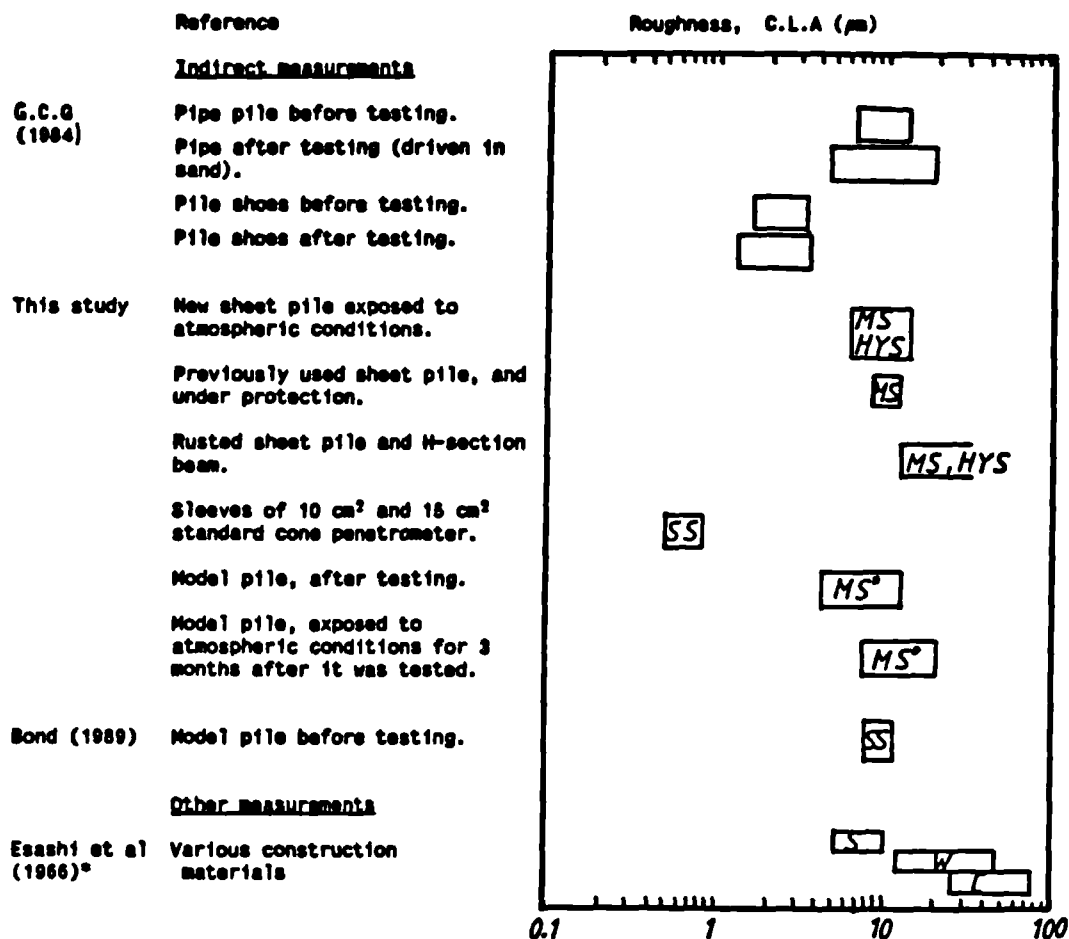


Fig. A2.45d Ring shear test 39 on blue London clay

Table A2.36 Measurements of roughness of pile-surfaces



Notes

MS : mild steel	S : Steel
MS ^p : hot-rolled mild steel	W : Wood
HYS : High-yield steel	C : Concrete
SS : Stainless steel	□ : Unspecified

* : as reported by Yoshimi et al (1981). They used the R_{max} roughness parameter, defined as the distance between the lowest trough and the highest peak of the surface. The values indicated in the table are half the values reported by the authors ($R_{max}/2$).

Table A2.37 Results from ring shear test 40 on blue London clay

Stage	Rate of displacement (mm/min)	Cummulative displacement (mm)	Net normal stress (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	110	1104.7	453.2 444.4 441.5	b,c:0.347 d:0.125 0.136	0.8 577.605.0 1050-1104.7	
2	110	2249.0	440.3 442.0 442.0 444.4	b,c:0.182 d:0.132 0.132 0.150	0.1 397.0-442.0 612.0-697.0 1088.0-1144.3	
3	110	3367.7	443.3 445.0 441.2	b,c:0.201 d:0.144 0.159	0.1 240.0 1030.0-1118.2	
4	110	4487.9	438.3 437.0	b,c:0.206 d:0.122	0.1 1114.0-1120.2	
5	110	5614.6	435.4 438.3	b,c:0.196 d:0.129	0.1 811.0-1126.0	

b: threshold, c: fast peak, d: fast residual

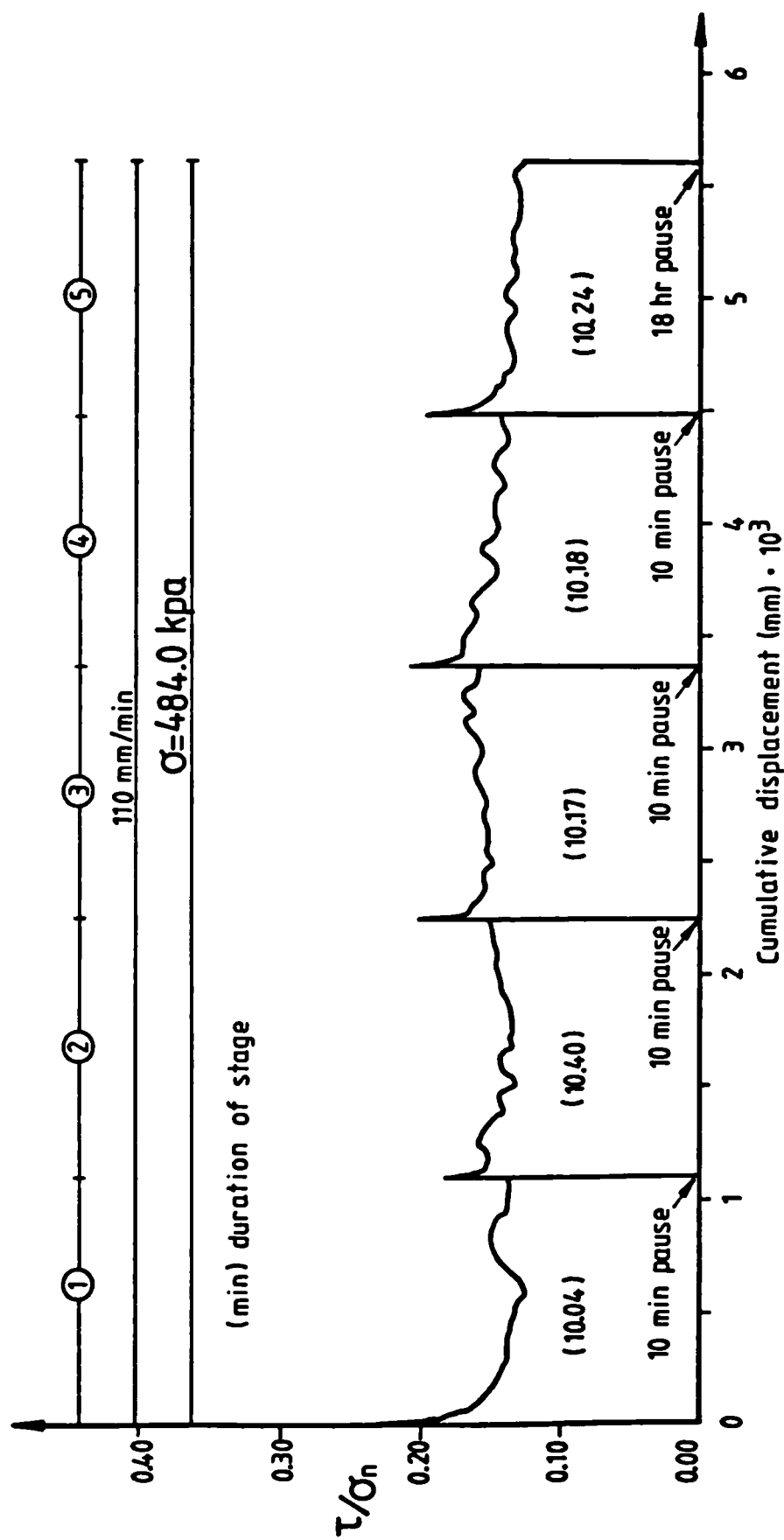


Fig. A2.46a Ring shear test 40 on blue London clay
(soil–stainless steel interface)

Table A2.37 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
6	0.00328	5647.7	312.3	0.195	0.1	
7	0.00530	5663.1	317.0	0.158	25.0-33.1	
			440	0.152	0.7-15.4	

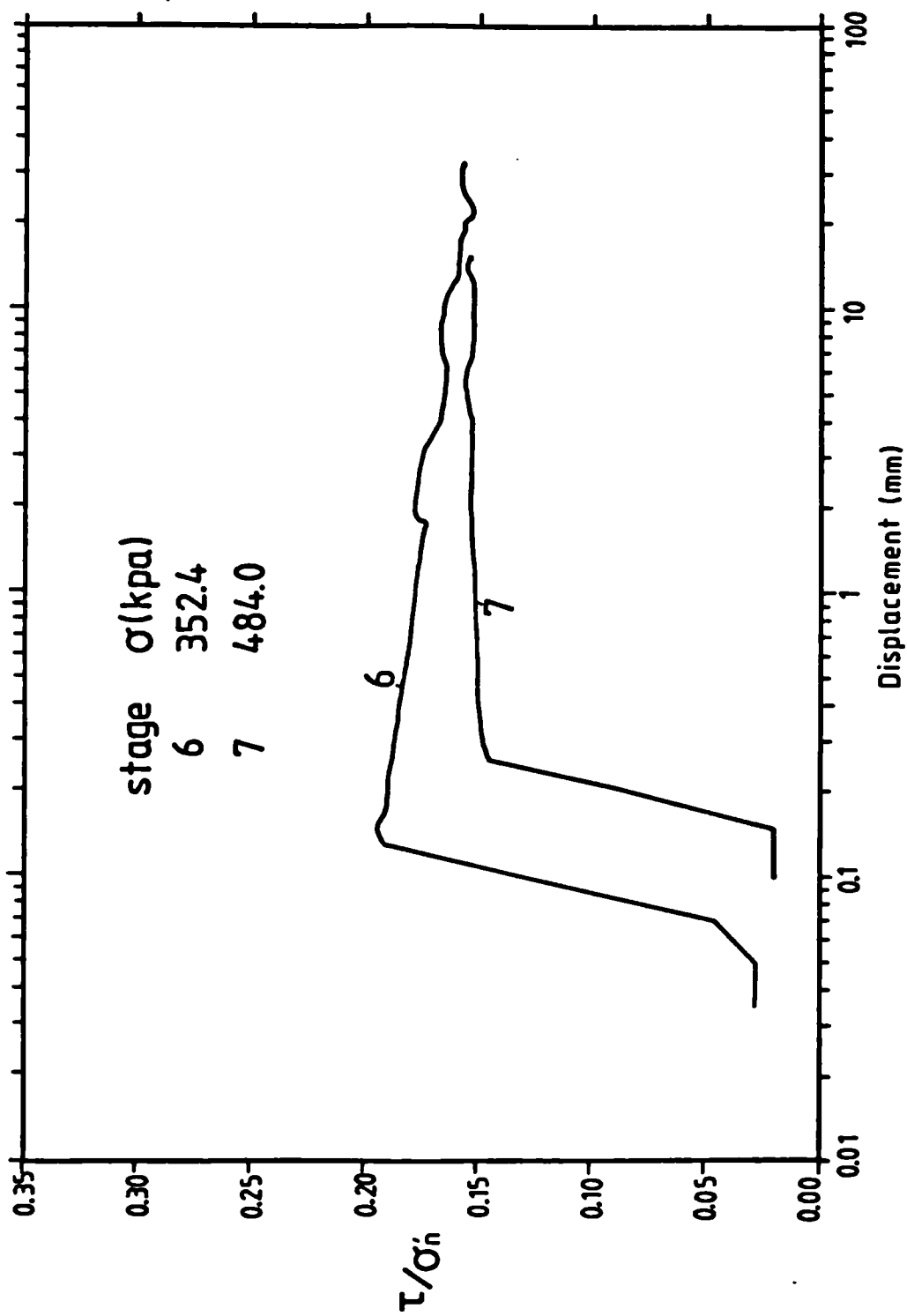


Fig. A2.46b Ring shear test 40 on blue London clay
(soil-stainless steel interface)

Table A2.38 Results from ring shear test 41 on blue London clay

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	1100	1050.7	424.5 421.8 414.2	b,c:0.567 0.282 d:0.272±0.009	0.5 102.1 756.0-1050.7	
2	1100	2100.4	414.8 415.0 422.0	b,c:0.397 d:0.190 0.217±0.005	0.0 690.0-707.0 1007.0-1049.8	
3	1100	3137.8	420.3 420.0 422.0	b,c:0.346 d:0.149 0.186±0.004	0.0 443.5 743.0-1037.4	
4	1100	4193.6	420.9 425.5 425.0	b,c:0.326 d:0.205 0.214±0.003	0.0 750.0-820.0 846.4-1055.8	
5	1100	5227.1	428.1 436.3	b,c:0.341 d:0.251±0.011	0.3 243.0-1033.5	
6	1100	6262.0	433.0 432.6 449.5	b,c:0.425 d:0.282 0.317±0.014	0.0 8.6 93.0-1034.9	

b: threshold, c: fast peak, d: fast residual

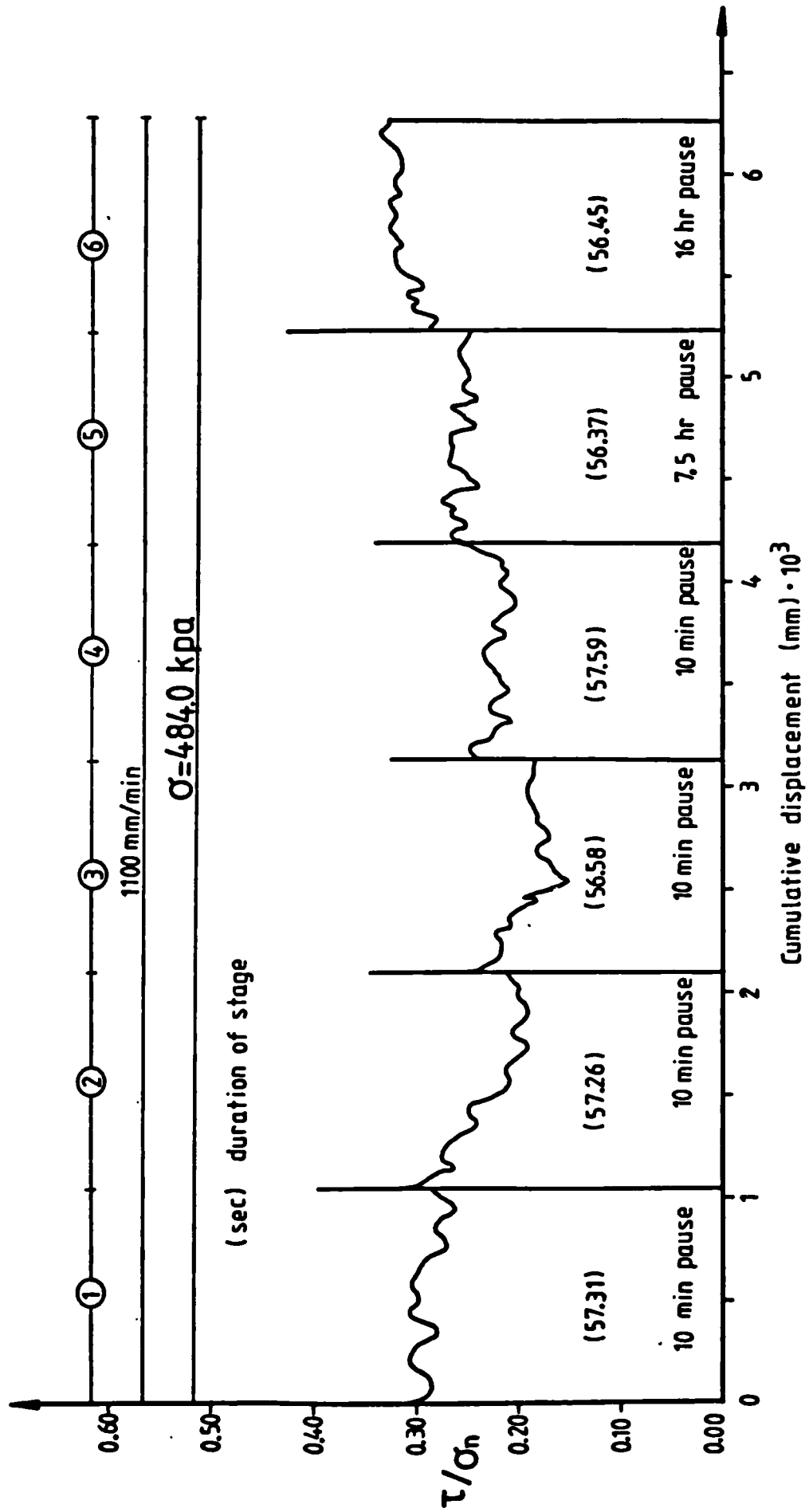


Fig. A2.47a Ring shear test 41 on blue London clay
(soil-stainless steel interface)

Table A2.38 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
7	0.00484	6277.74	449.1 440.0	e:0.262 0.195	0.4 9.2-15.7	—

e: slow peak after fast shearing

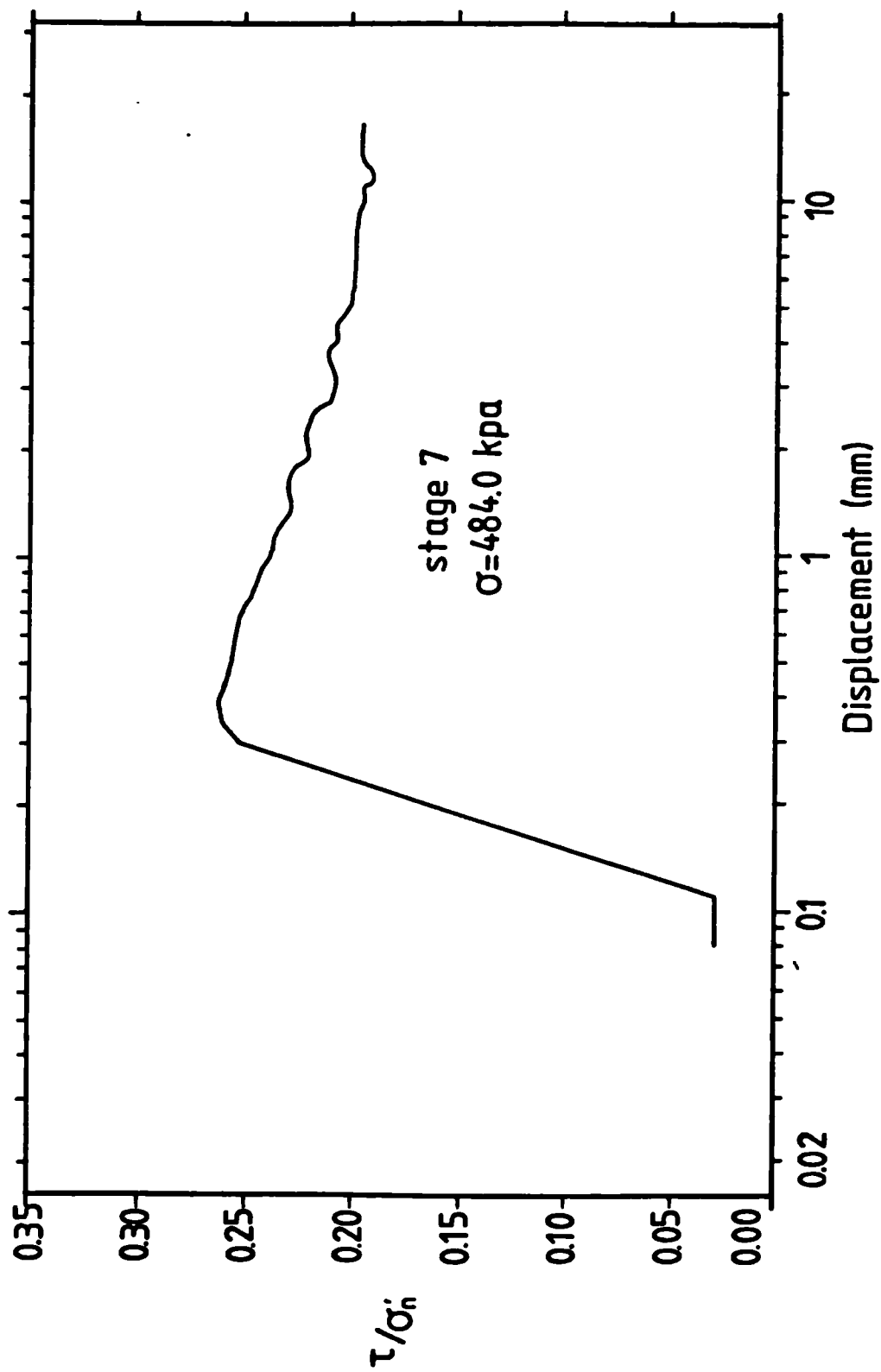


Fig. A2.47b Ring shear test 41 on blue London clay
(soil–stainless steel interface)

Table A2.39 Results from ring shear test 42 on blue London clay

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	2600	667.1	413.4 395.9 403.6 405.6 402.5	b,c:0.510 d:0.130 0.177 0.132 0.233	0.0 294.6 366.0 475.2 661.5-667.1	
2	2600	1231.5	403.5 397.9 403.0	b,c:0.361 d:0.263 0.350	0.7 42.0-67.4 564.5	
3	2600	1852.4	403.3 402.0	b,c:0.466 d:0.325	0.0 620.9	
4	2600	2343.5	404.8 411.6	b,c:0.459 d:0.325	0.5 491.9	
5	2600	2784.8	412.1 409.0	b,c:0.462 0.315	0.5 441.3	
6	2600	3479.9	412.6 422.9	b,c:0.563 0.282	0.6 695.1	
7	2600	4194.9	423.4 414.6	b,c:0.388 0.300	0.6 715.0	
8	2600	4920.8	414.6 428.0	b,c:0.434 0.317	0.6 695.0-725.9	
9	2600	5633.2	423.3 421.8 424.0	b,c:0.438 d:0.281 0.345	0.6 643.0 712.3	
10	0.00487	5655.2	see the following table			
11	2600	6089.7	409.6 409.5 424.2	b:0.418 c:0.428 d:0.279	0.6 1.9 393.7-433.8	

b: threshold, c: fast peak, d: fast residual

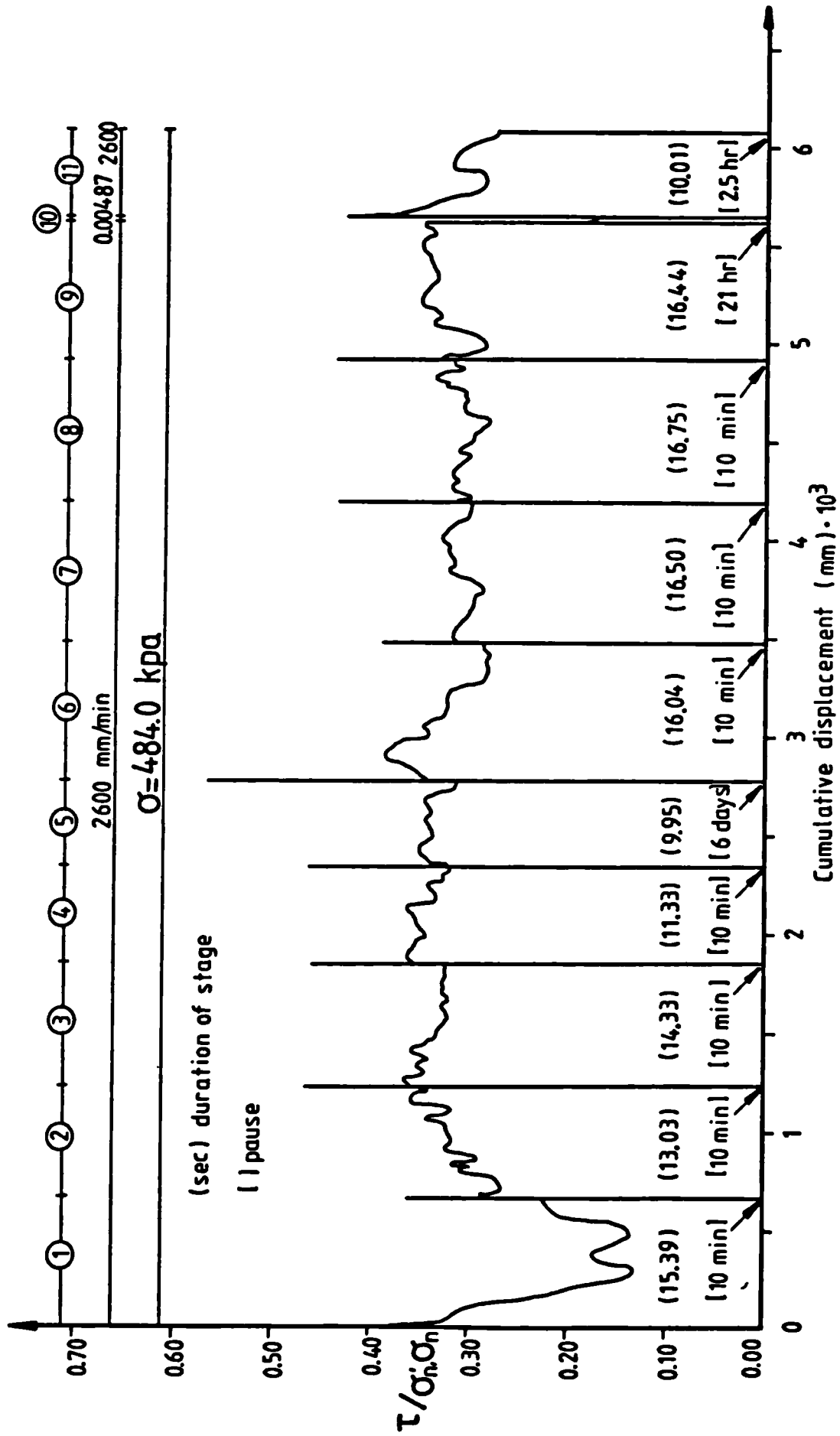


Fig. A2.48a Ring shear test 42 on blue London clay
(soil-stainless steel interface)

Table A2.39 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
10	0.00487	5655.9	426.8 421.5 419.0	e:0.251 0.175 0.167	0.5 10.0 15.6-22.7	
12	0.00487	6362.9	422.8 435.5	e:0.250 0.154	0.5 17.3-273.2	

e: slow peak after fast shearing

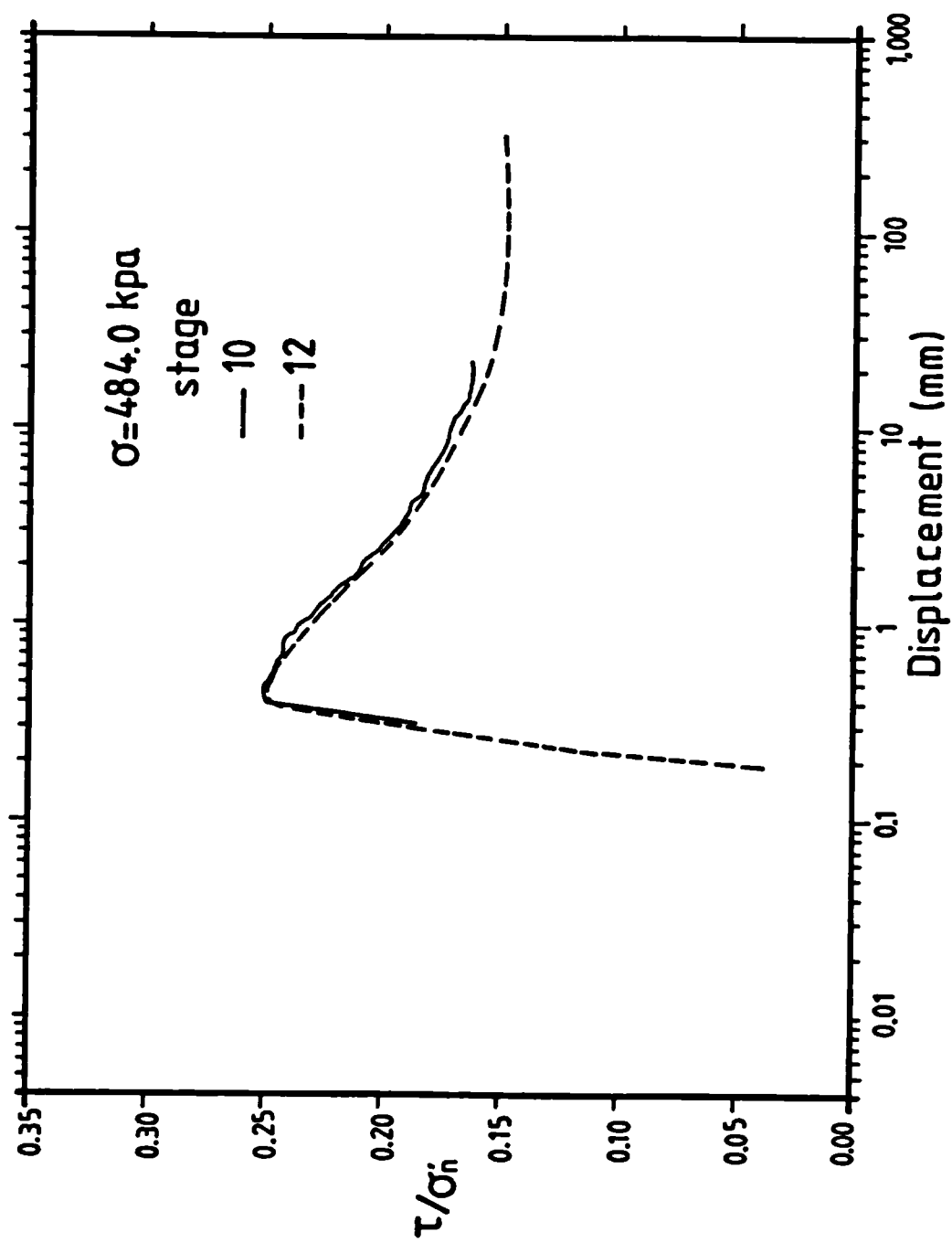


Fig. A2.48b Ring shear test 42 on blue London clay
(soil-stainless steel interface)

Table A2.39 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
13	2600	7373.1	436.2 436.2 439.6 441.6	b:0.391 c:0.394 0.271 d:0.166	0.6 0.9 194.4 967.5	
14	0.00628	7391.3	440.2 431.7	e:0.232 0.160	0.4 17.4-18.2	
15	1100	8397.1	440.5 440.5 442.4	b:0.273 c:0.357 d:0.130±0.005	0.0 0.3 527.0-1005.8	
16	0.00628	8426.4	441.0 449.0	e:0.225 0.162	0.4 19.0-25.1	
17	80	9432.0	451.3 451.4 450.0	b:0.163 c:0.225 d:0.141±0.008	0.2 1.8 290.4-1005.6	
18	0.00628	9466.7	450.7 450.6	e:0.159 0.155	0.3 0.4-34.7	
19	400	10475.1	454.9 457.0 454.2	b:0.191 c:0.309 d:0.095±0.009	0.9 10.0-19.7 243.7-1008.4	
20	0.00628	10534.9	437.2	e:0.217 0.153	0.2 17.3-59.7	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

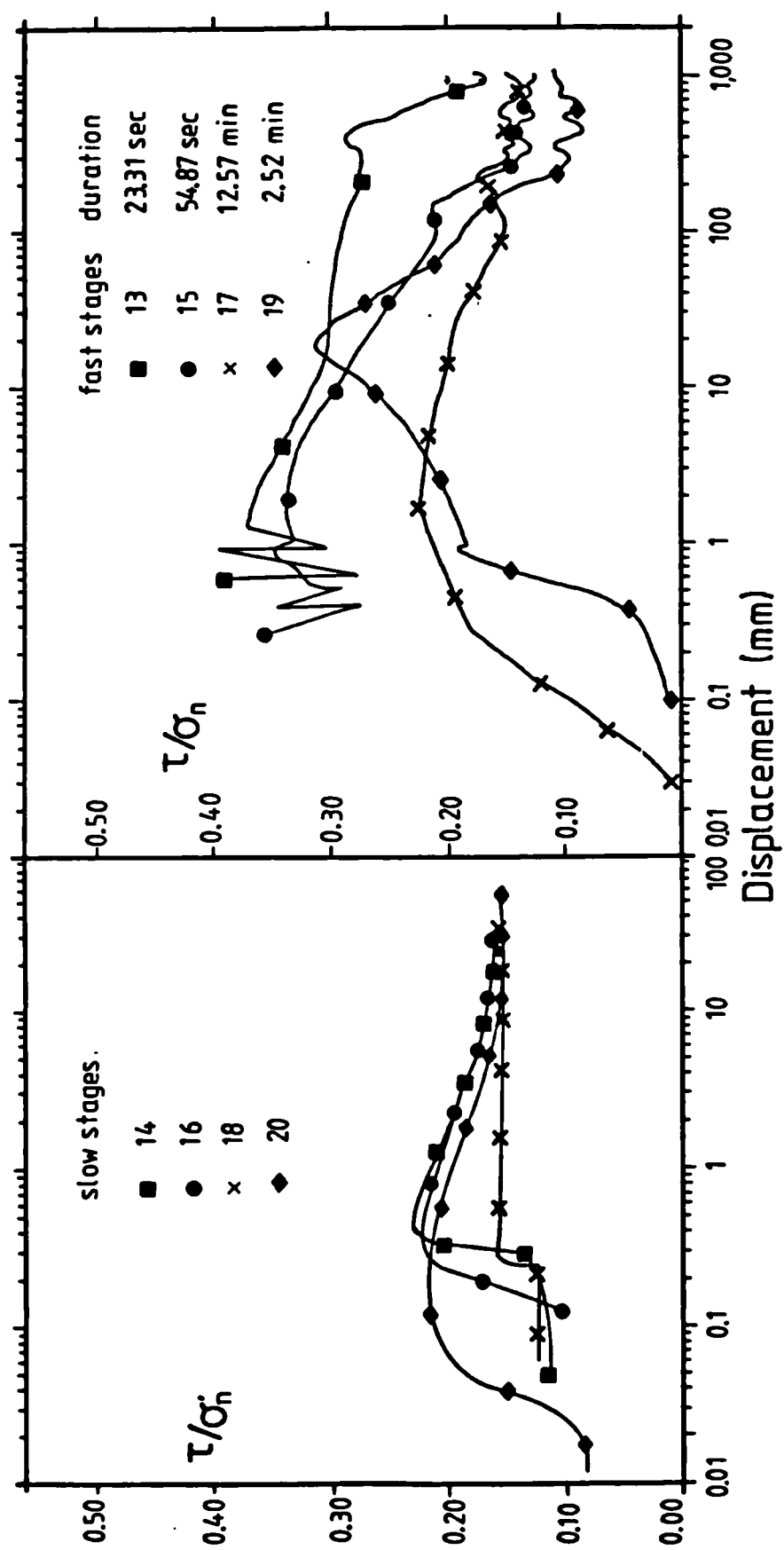


Fig. A2.48c Ring shear test 42 on blue London clay
(soil–stainless steel interface)

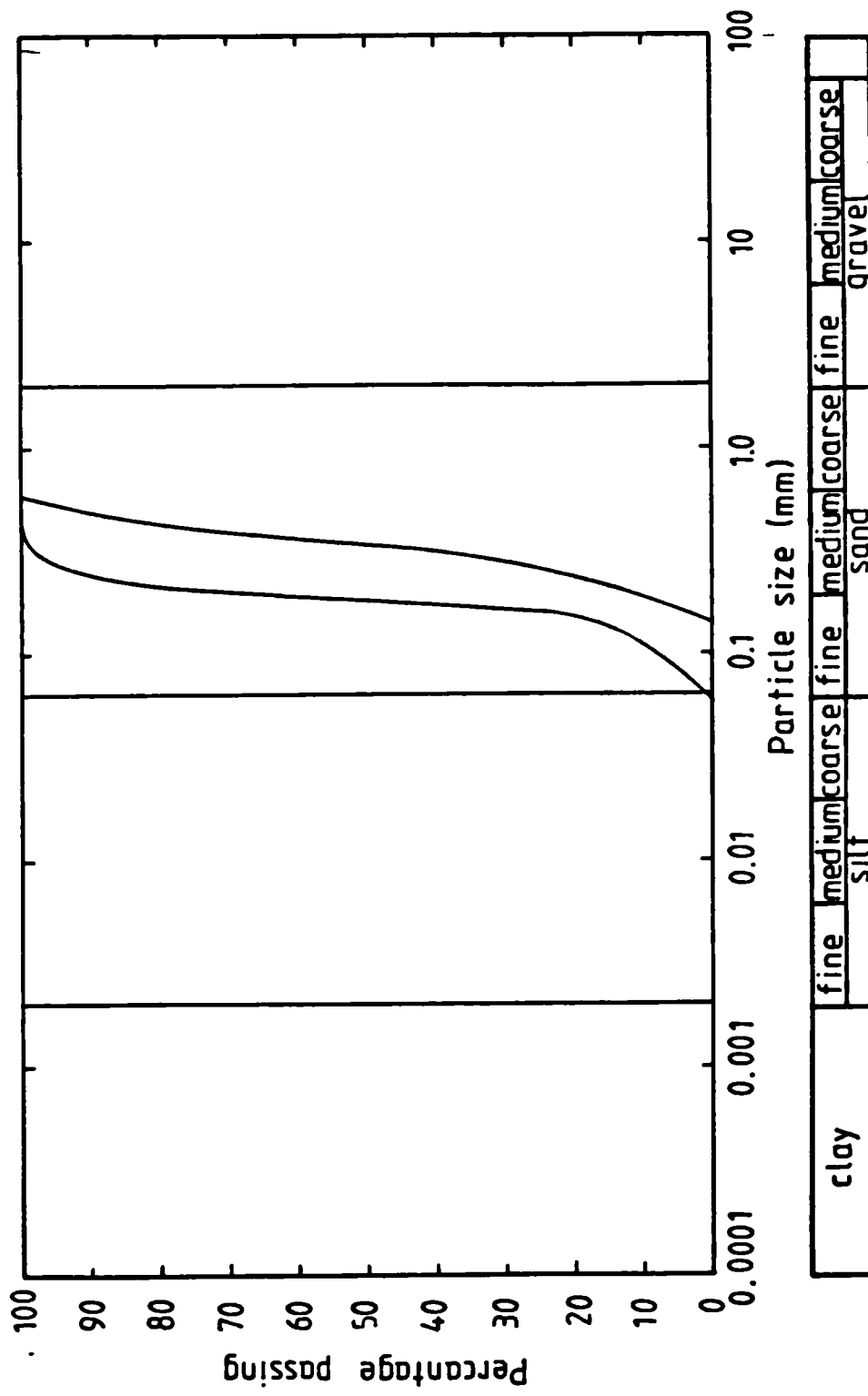


Fig. A2.49 Grading curves of tested sands

Table A2.40 Results from ring shear test 43 on medium sand

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0142	61.8	241.3 200.0	peak:0.700 0.566	1.6 8.0-61.8	
2	2600	468.8	206.1 220.0	b,c:0.735 d:0.560	1.3 11.4-407.0	
3	0.0142	504.5	232.4 210.0	e:0.584 0.566	0.4 10.0-35.7	
4	5500	852.1	247.7 237.0	b,c:0.679 d:0.554	2.7 20.0-347.6	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

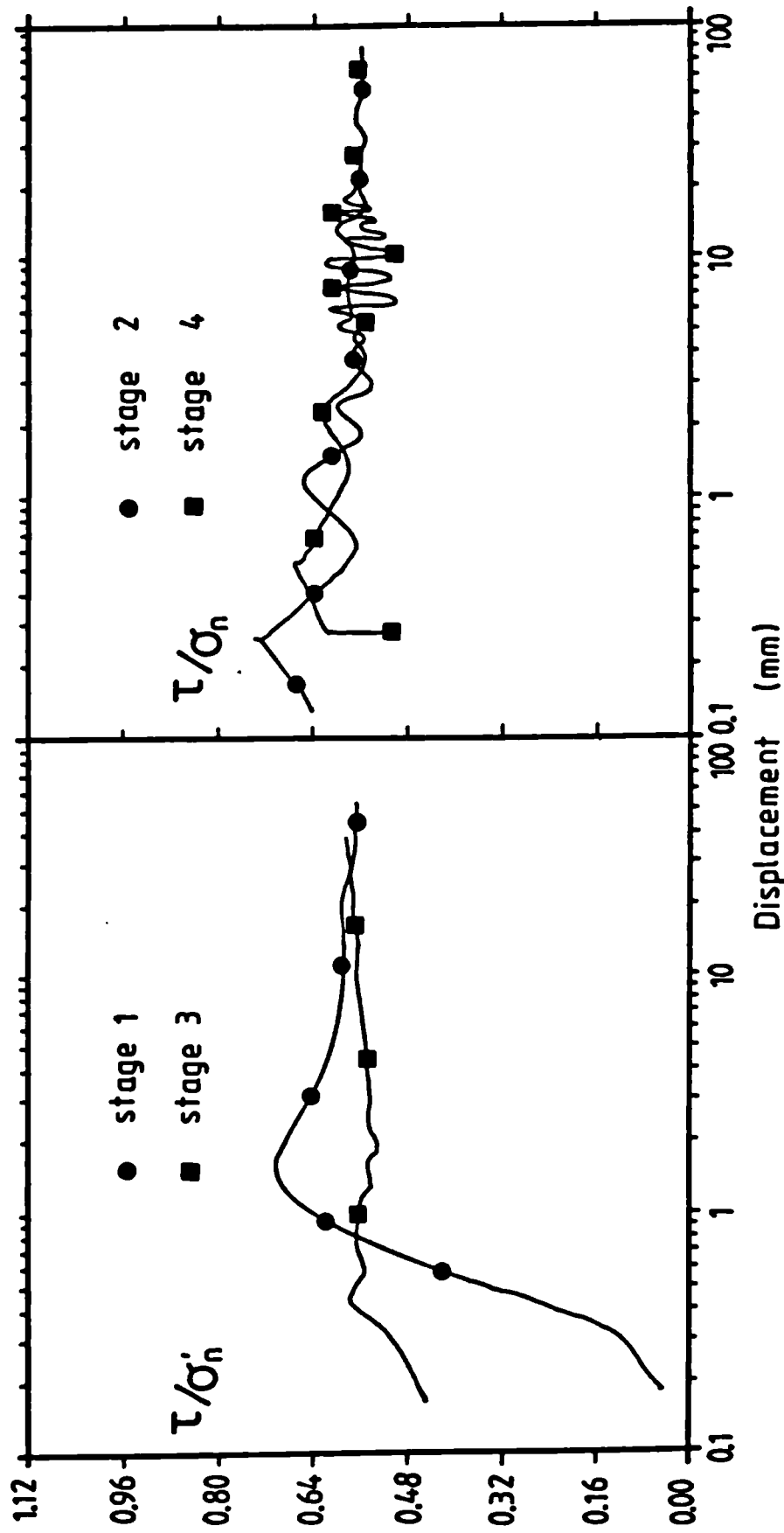


Fig. A2.50 Ring shear test 43 on medium sand

Table A2.41 Results from ring shear test 44 on fine sand

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0917	96.9	445.0 390.0	peak: 0.646 0.580	2.4-33 37.6-96.9	
2	100	302.9	441.4 442.0 441.0	b,c: 0.610 d: 0.528 0.537	2.1 82.8 185.0-206.0	
3	0.0917	360.6	442.7 445.0	e: 0.595 0.558	2.9-3.6 44.6-57.7	
4	360	561.2	440.5 443.2 443.2	b,c: 0.623 d: 0.555 0.563	5.8-11.8 146.5-171.2 200.7	
5	0.0917	660.7	443.2 438.0	e: 0.626 0.569	3.5 56.0-99.4	
6	100	860.1	443.6 444.1 441.8	b,c: 0.658 d: 0.565 0.589	2.4 99.7 199.5	
7	0.0917	916.2	444.0 443.0	e: 0.638 0.590	0.7-1.2 560.0	
8	2500	1128.9	446.0 441.6 443.2	b,c: 0.634 d: 0.584 0.594	0.89 81.4 212.7	
9	0.0917	1243.2	448.7 440.0	e: 0.640 0.580	2.8 30.0-114.4	
10	3.2759	1469.3	448.3 448.0	b,c: 0.565 d: 0.550-0.570	11-28 4.8-226.0	
11	0.0917	1599.3	444.0 445.0	e: 0.579 0.575	0.4 0.8-130.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

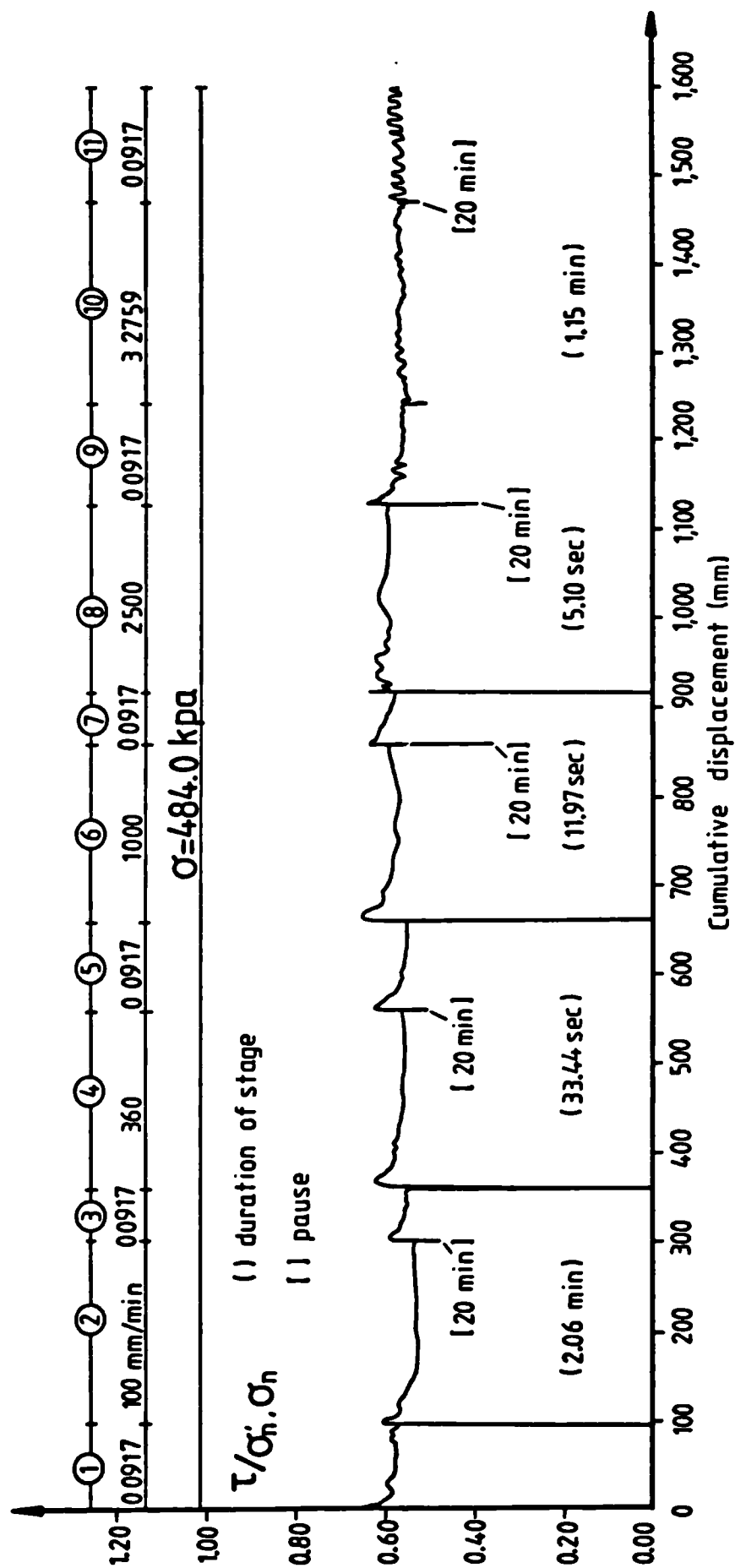


Fig. A2.51a Ring shear test 44 on fine sand

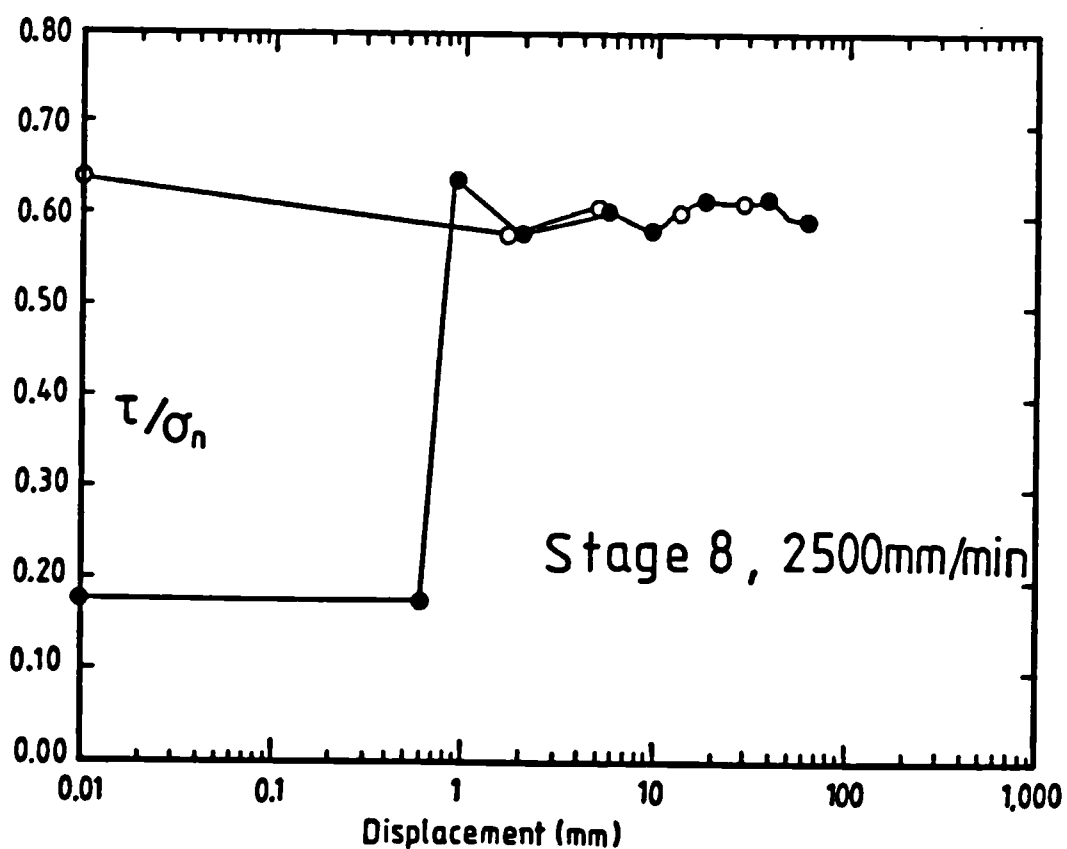
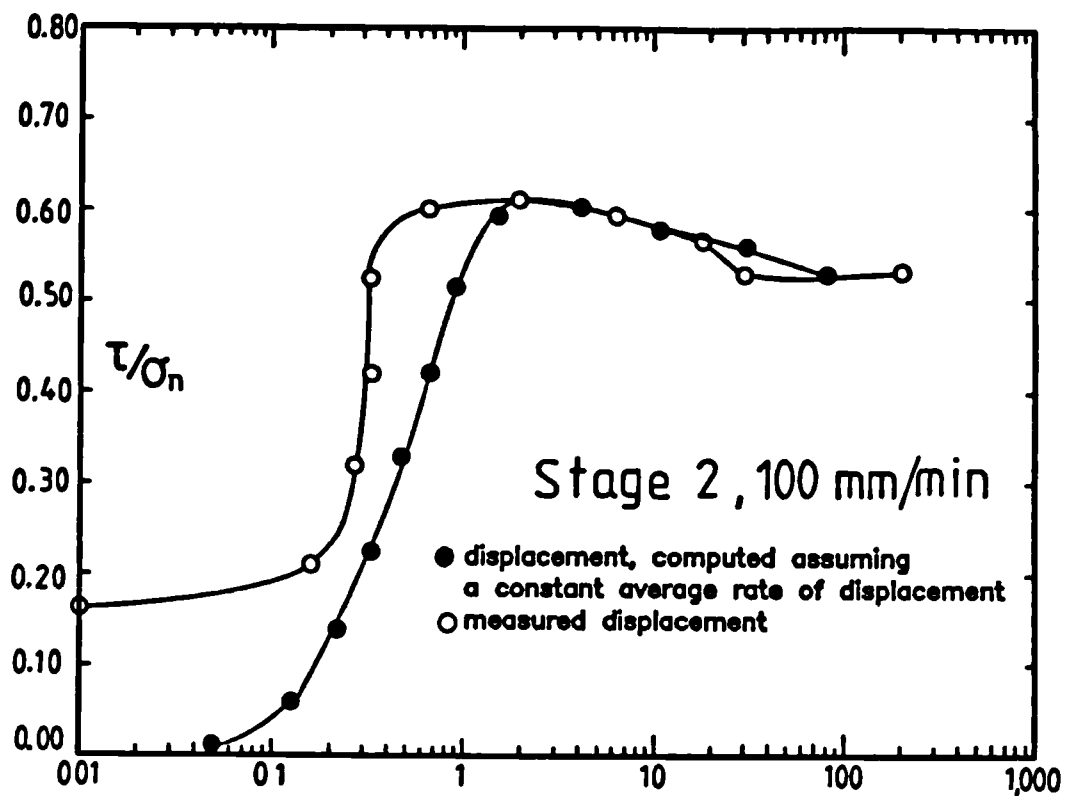


Fig. A2.51b Ring shear test 44 on fine sand

Table A2.42 Results from ring shear test 45 on the fine sand

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0940	150.6	251.5 205.0	peak:0.707 0.600	1.6 18.6-150.5	
2	100	335.4	233.9 231.3 201.7	b,c:0.633 d:0.593 0.627	2.4 17.1 184.5	
3	0.940	406.6	203.5 197.0	e:0.640 0.620	0.4 30.2-71.3	
4	0.01124	422.6	225.0	0.630	3.6-15.9	
5	360	580.5	238.0 228.6 228.4	b,c:0.636 d:0.579 0.594	4.8 137.1 157.9	
6	0.0940	684.9	No recording	was performed		
7	1000	876.8	217.3 230.0 222.8	b,c:0.677 d:0.601 0.611	0.5 9.0-10.8 151.9-191.8	
8	0.0940	978.0	229.0 222.0	e:0.670 0.589	0.8 37.9-103.2	
9	6000	1180.2	222.7 223.4	b,c:0.704 d:0.611 0.612	3.8 191.4 200.3	
10	0.0940	1226.8	229.0 225.0	e:0.661 0.609	0.6 10.1-46.5	
11	2.6358	1450.8	222.0 222.0 222.0	b,c:0.613 d:0.565 0.590	0.4 113.3-145.0 224.0	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

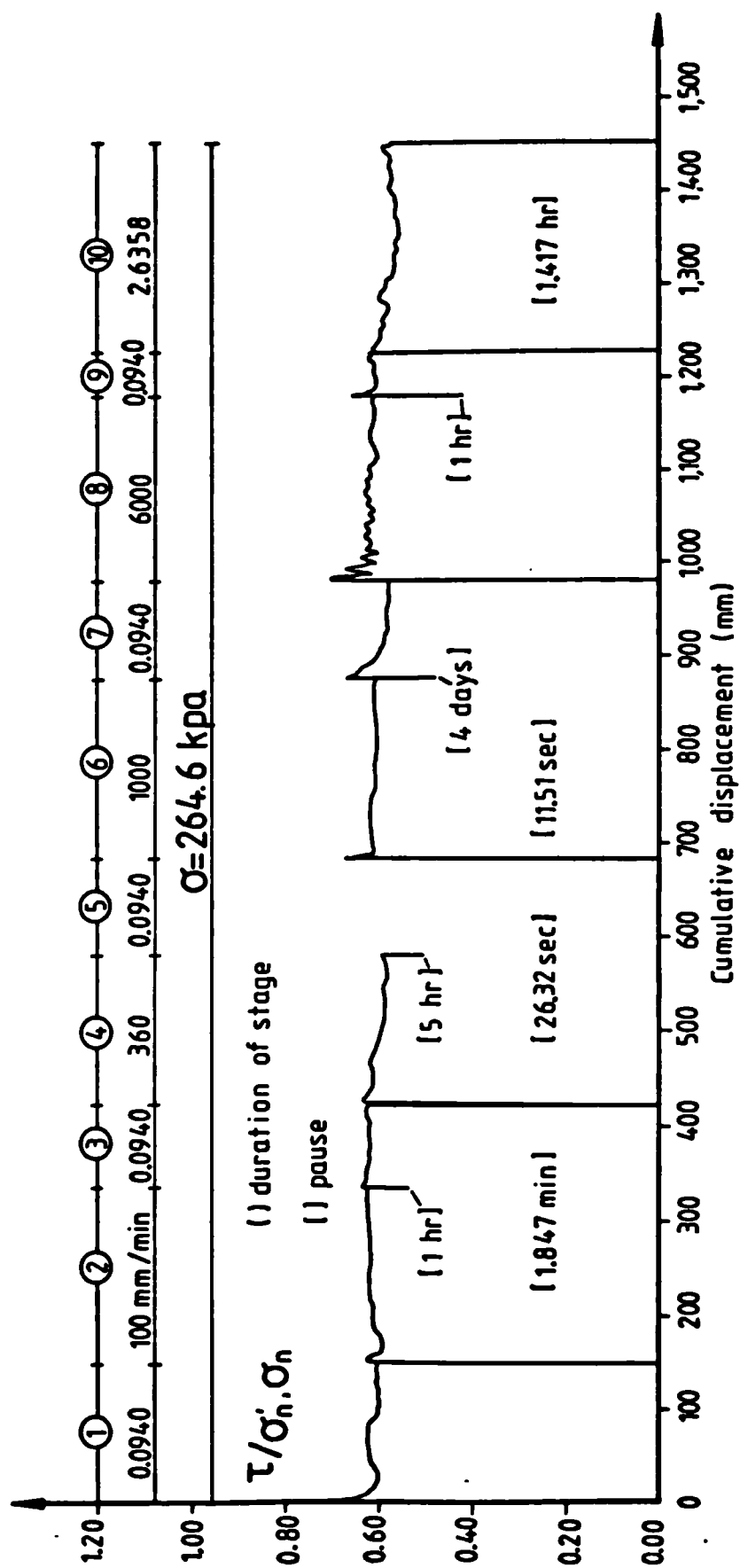


Fig. A2.52 Ring shear test 45 on fine sand

Table A2.43 Results from ring shear test 46 on fine sand

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0117	25.2	116.7 110.0 92.5	peak:0.638 0.585 0.600	2.2 12.5-15.0 22.7-25.2	
2	0.0926	75.6	89.4 83.1	0.621 0.555	0.4 19.0-50.4	
3	120	259.6	115.0 95.0 98.3	b,c:0.608 0.555 0.573	1.8-2.0 28.3 111.0-183.9	
4	0.0926	324.1	125.9 90.2	e:0.703 0.562	29.0 32.7-64.4	
5	400	516.6	112.4 101.0 107.0	b,c:0.604 d:0.585 0.615	4.6 75-80 128.3-192.5	
6	0.0926	579.2	129.5 67.5	e:0.591 0.578	1.2 40.0-62.6	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

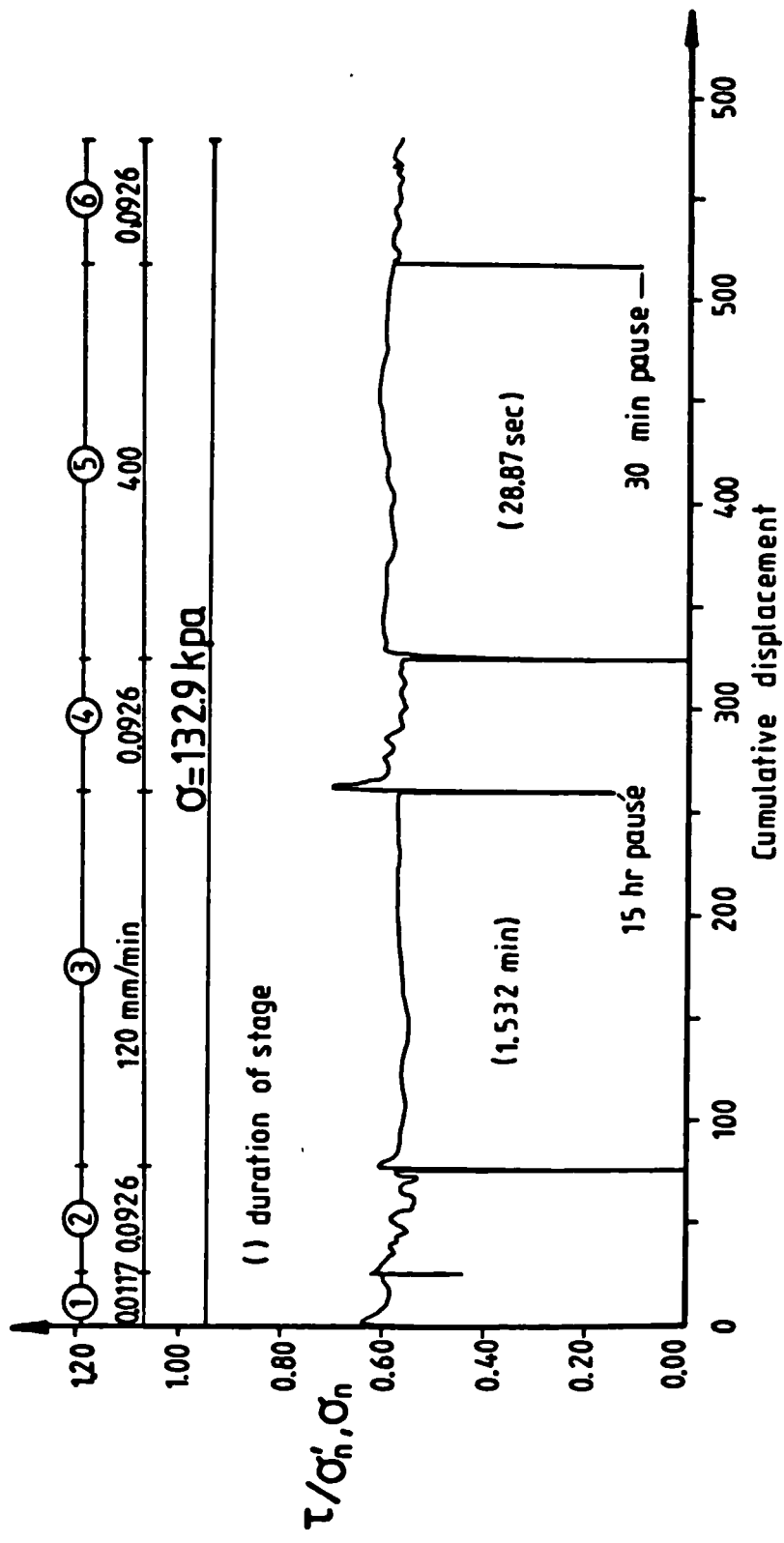


Fig. A2.53 Ring shear test 46 on fine sand

Table A2.44 Results from ring shear test 47 on fine sand

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0935	89.1	435.0 378.0	peak:0.686 0.585	1.7-1.9 31.0	
2	360	262.7	344.0 333.9	b,c:0.638 d:0.589	5.4-5.6 150.0-173.4	
3	0.0935	308.1	348.8 349.5	e:0.640 0.623	0.7 9.7-45.4	
4	0.0935	453.3	96.1 92.2	peak:0.723 0.595	0.5 20.0	
5	1200	649.6	No recording was performed	0.616	62.7-145.2	
6	0.0935	716.3	91.1 88.0 83.0	e:0.660 0.585 0.590	0.4-0.6 10.4-20.0 40.0-66.7	
7	1200	910.5	122.2 127.2 98.0	b,c:0.630 d:0.573 0.610	0.6 14.3-17.3 130.0-194.2	
8	0.0935	1084.2	88.0 80.0	e:0.650 0.575	0.5-0.6 4.8-173.8	
9	6000	1295.2	121.8 126.4 102.0	b,c:0.691 d:0.530 0.570	4.4 35.2 127.5-211.0	
10	0.0935	1515.5	107.3 98.0	e:0.616 0.575	0.5 6.6-170.7	
11	3.4677	1720.1	108.3	b,c:0.600 0.573	0.6 1.0-204.6	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

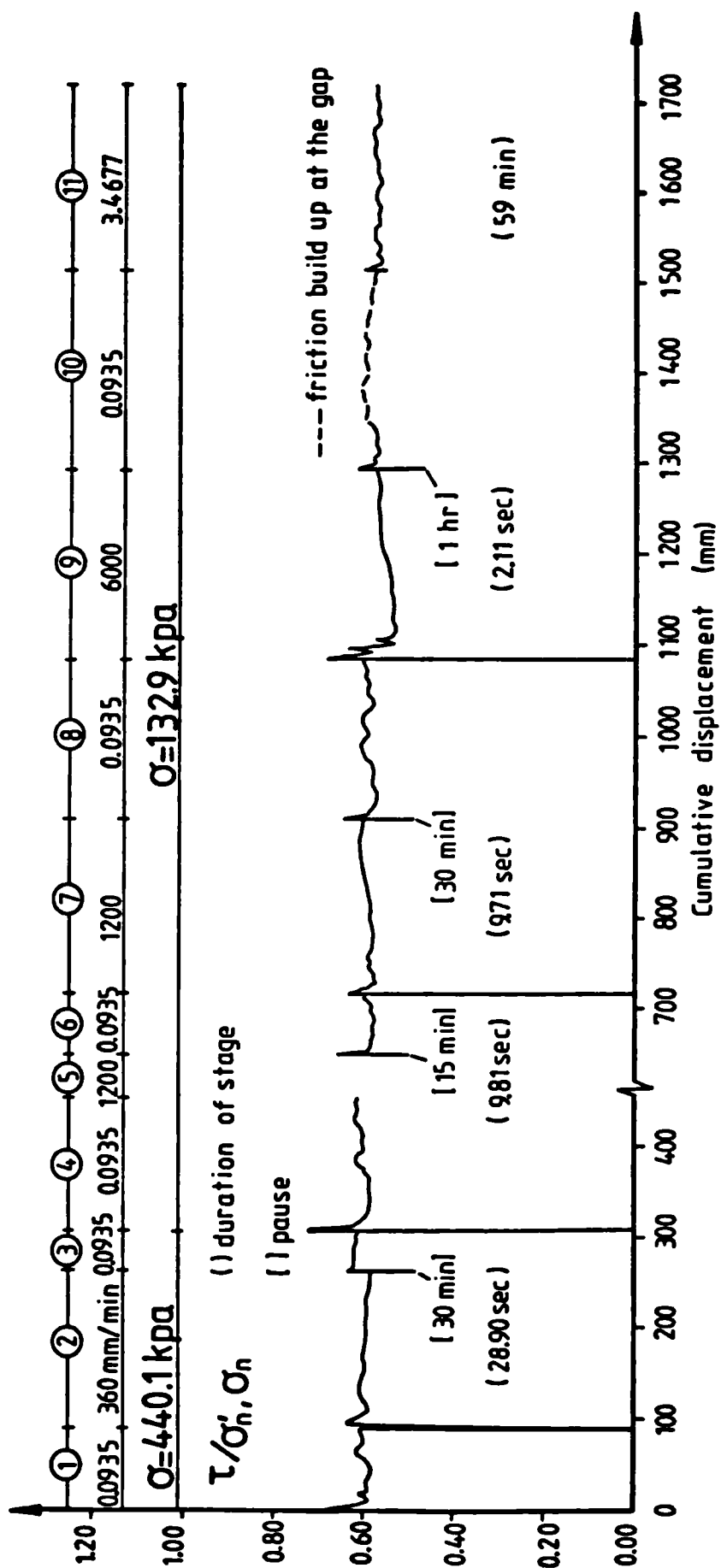


Fig. A2.54 Ring shear test 47 on fine sand

Table A2.45 Results from ring shear test 48 on clayey siltstone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.01	219.1	524.0	peak:0.454 0.326	1.3 133.4	
2	0.01	252.5	255.0	peak:0.291 0.264	0.4 2.0-33.4	
3	300	520.0	228.7	b:0.337 c:0.380 d:0.126	1.1 12.2 226.1	
4	0.01	604.2	230.7 226.0	e:0.314 0.295	1.2 17.1-84.2	
5	300	18089.9	225.6	b:0.375 c:0.396 0.156 d:0.115±0.019 0.177±0.012 0.152±0.011	0.4 9.3 221.3 340-1750 3500-12100 12100-15000	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

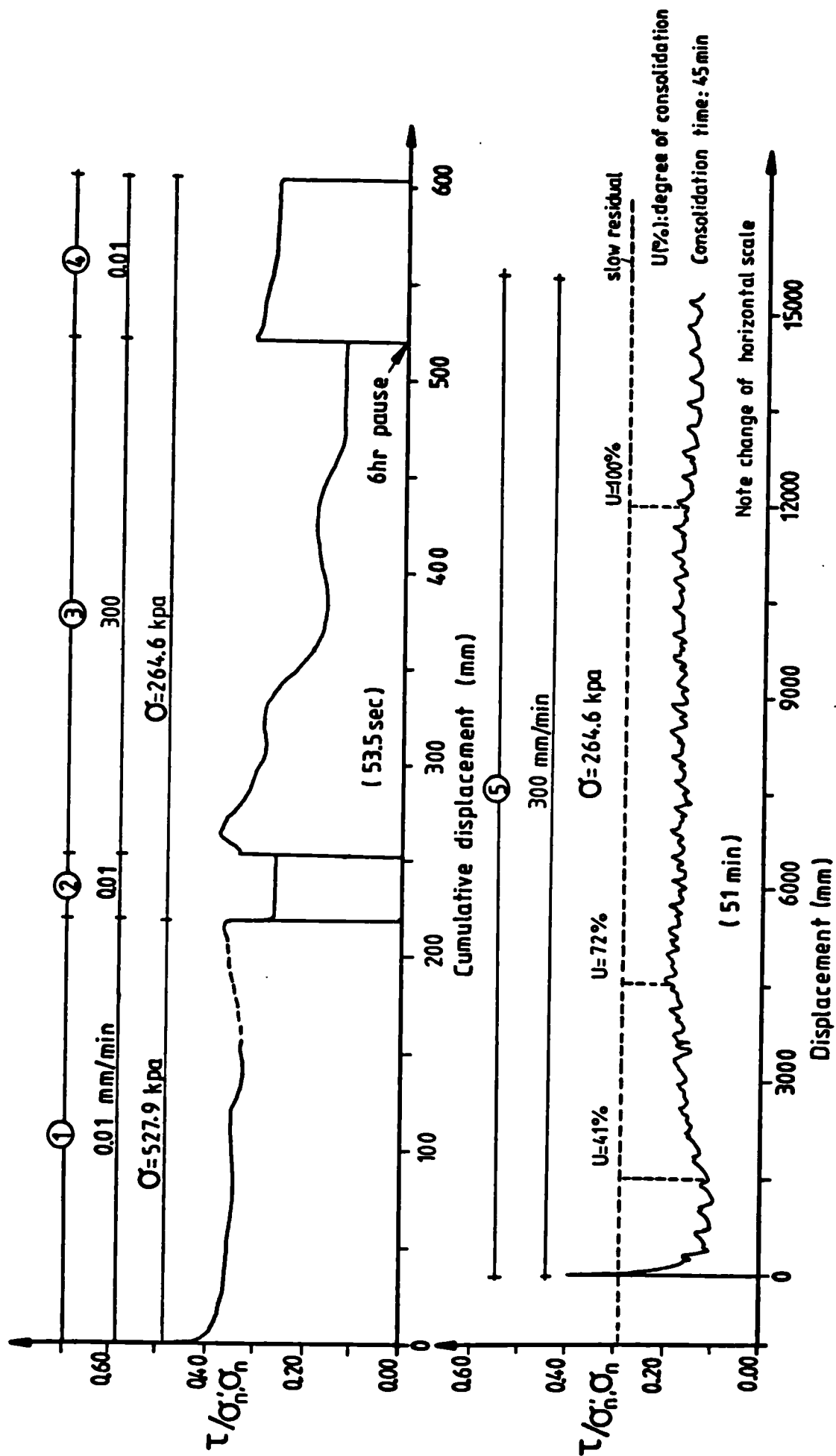


Fig. A2.55 Ring shear test 48 on clayey siltstone
(soil-glass interface)

Table A2.46 Results from ring shear test 49 on clayey siltstone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0220	211.7	260.0 232.0	peak:0.570 0.415	0.9 211.7	
2	133	476.7	No recording was performed			3hr pause
3	133	688.3	243.50 243.50 242.30 230.0	b:0.364 c:0.369 d:0.127 0.233	0.2 0.3 119.0 211.6	1hr pause
4	0.0026	691.6	242.6	e:0.409 0.390	1.0 3.2	
5	0.0038	696.0	234.2 235.0	c:0.407 0.394	0.01 0.12	
6	0.0049	699.3	235.2	c:0.428 0.388	0.01 3.2	
7	0.0052	703.3	235.0	c:0.396	1.2-4.0	
8	0.0740	744.9	242.0	c:0.399	3.2-41.6	
9	0.095	974.4	243.0	c:0.400	0.0-201.7	
10	0.1202	995.7	No recording was performed			

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

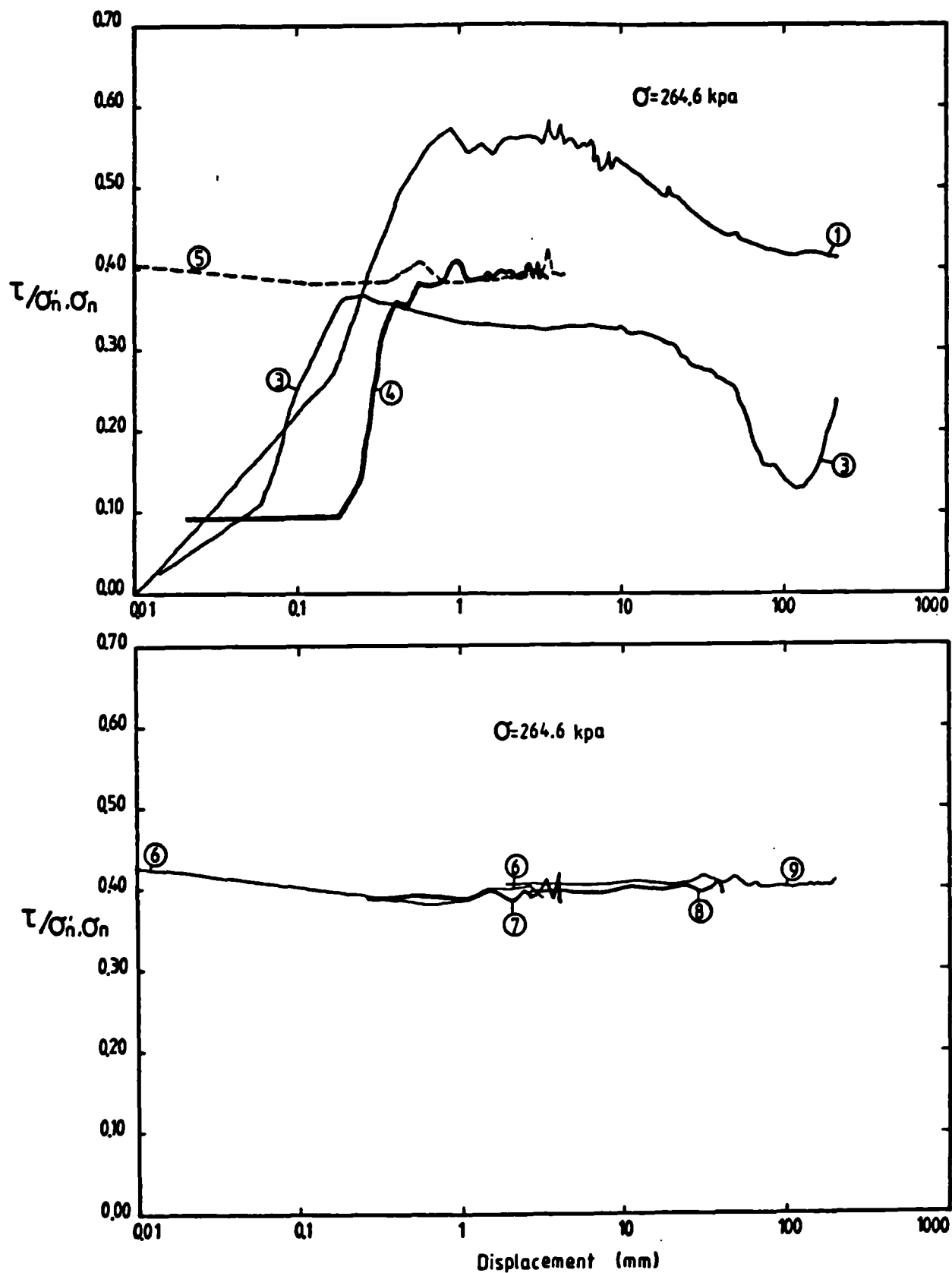


Fig. A2.56a Ring shear test 49 on clayey siltstone D

Table A2.46 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
11	0.1356	1166.1	240.0	c:0.410	8.1-162.7	
12	0.3897	1212.5	No recording was performed			
13	0.4975	1258.8	245.0	c:0.422	11.1-46.3	
14	0.6370	1392.6	227.0	c:0.425 0.417	0.0 133.8	
15	0.3064	1701.2	233.5	c:0.422 0.418	0.0 15.9-308.6	
16	1.8791	1885.3	230.8 237.5	c:0.428 0.417	0.0 112.7-184.1	
17	2.3816	2094.9	235.0	c:0.429	47.6-209.6	
18	1.3941	2412.8	235.0	0.427	69.7-317.9	(1)
19	1.0017	2571.1	235.0	c:0.438 0.430	0.0 59.1-158.3	
20	0.0014	2573.9	240.8 241.2 246.2	e:0.429 0.421 0.392	1.0 1.3 2.6-2.8	
21	0.0130	2579.5	245.0	c:0.409 0.407	0.7 3.0-5.6	
22	0.0660	2661.0	237.0 245.0	c:0.410 0.404	2.2 6.1-81.5	
23	1300	2787.8	No recording was performed			

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

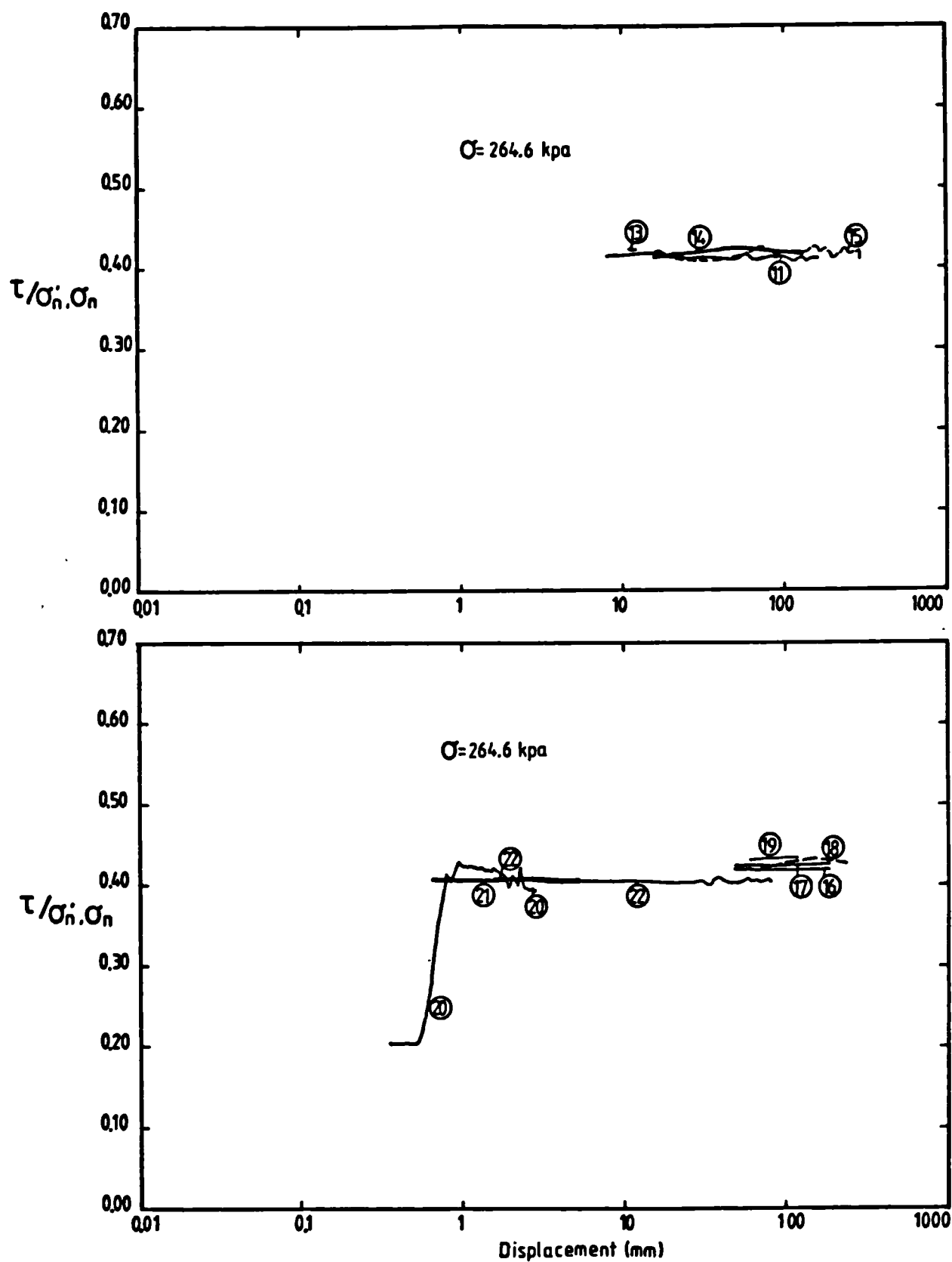


Fig. A2.56b Ring shear test 49 on clayey siltstone D

Table A2.46 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
24	900	2981.3	240.0	b,c:0.760 0.399	0.2 183.2	
25	1200	3142.3	251.3	b,c:0.507 0.386	0.3 50.7	
26	1500	3311.4	262.4	b,c:0.448 0.332	0.7 162.1	
27	1700	3488.5	260.0	b,c:0.398 0.256	0.4 177.1	
28	2000	3669.6	260.0	b,c:0.354 0.249	0.0 178.7	
29	2300	3872.5	259.0	b,c:0.328 0.152	0.0 202.9	
30	2600	4110.3	255.6	b,c:0.238 0.118	0.6 237.8	
31	3000	4365.6	260.0	b,c:0.202 d:0.106	0.0 139.0	
32	3500	4709.1	261.0	b,c:0.226 d:0.079	0.0 172.6	
33	4000	5015.4	255.0	b,c:0.174 0.179 0.068	2.0 59.7 277.0	
34	4000	5258.2	252.5	b,c:0.142 c:0.151 0.079	1.0 60.4 242.8	
35	4400	5546.9	254.5	b,c:0.155 d:0.091 0.125	0.0 174.9 288.7	
36	5400	5755.9	248.6	b:0.158 c:0.202 0.074	0.0 13.1 209.0	
37	6000	5948.1	253.3	b:0.144 c:0.157 d:0.098 0.125	0.0 14.0 130.0 192.2	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

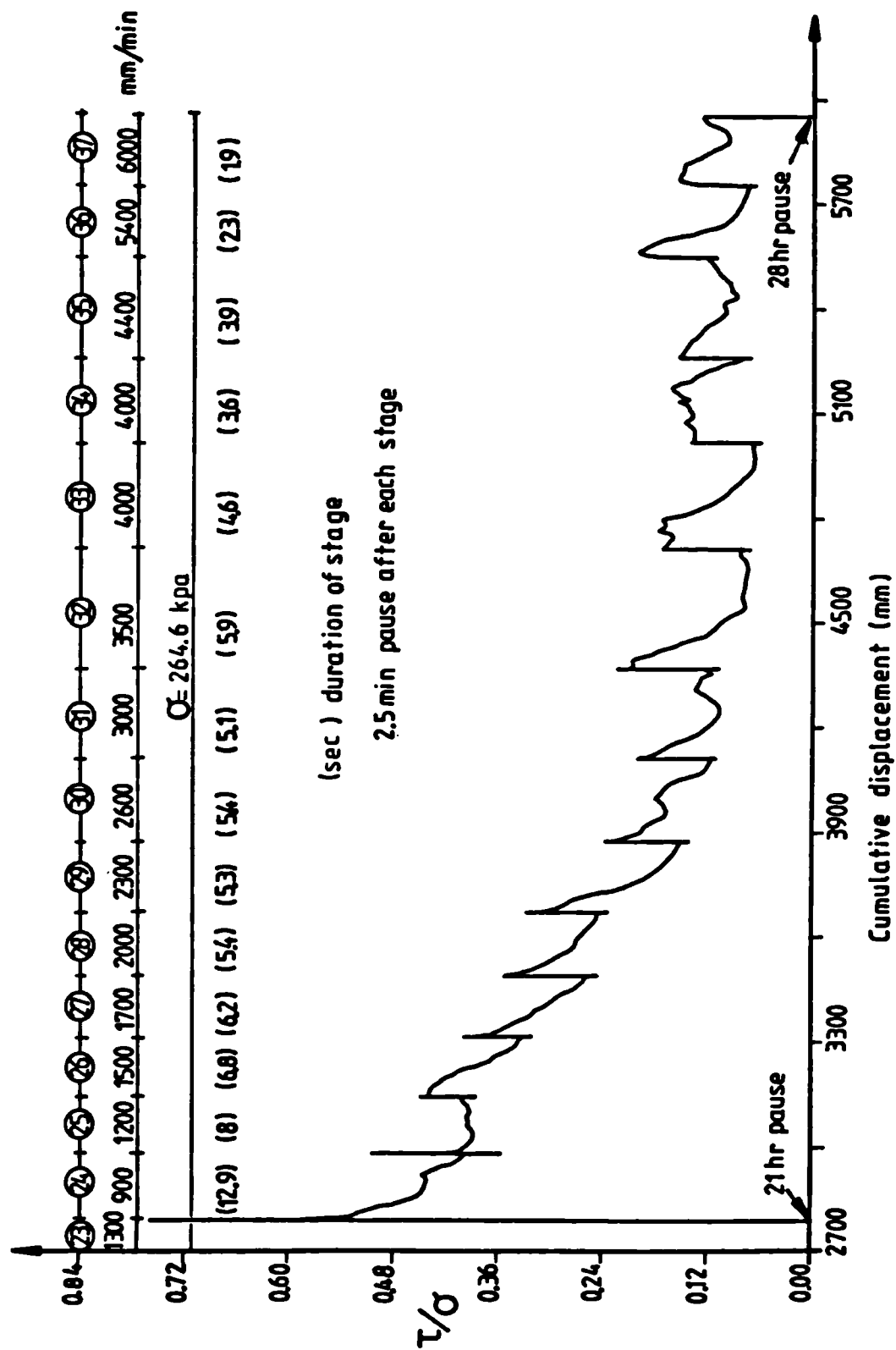


Fig. A2.56c Ring shear test 49 on clayey siltstone D

Table A2.46 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
38	3700	6194.6	248.5	b,c:0.442 d:0.071	0.9 248.5	
39	2000	6198.0	244.8	b,c:0.278 d:0.218	0.0 3.4	
40	6400	6510.4	253.0	b:0.358 c:0.367 d:0.077	0.0 1.3 312.4	
41	4700	6742.1	256.5	b,c:0.245 d:0.066 e:0.098	0.0 142.3 222.6	
42	4300	7005.6	254.0	b:0.213 c:0.232 d:0.083	1.0 7.3 124.9	
43	3700	7264.7	253.6 253.6 259.0	b:0.159 c:0.181 d:0.050	0.0 0.9 259.1	
44	3200	7456.7	258.0	b,c:0.151 d:0.051	0.8 192.0	
45	3200	7650.6	267.9	b:0.167 c:0.165 d:0.048	0.8 59.4 183.8	
46	3000	7846.5	266.5	b,c:0.154 d:0.056	0.7 196.0	
47	2600	8028.8	255.2 260.7	b,c:0.220 d:0.048	0.0 183.3	
48	2300	8184.4	254.9	b,c:0.191 d:0.058	0.0 155.6	
49	2000	8311.1	248.5	b,c:0.192 d:0.076	0.0 126.7	
50	1900	8452.4	245.9 246.4	b,c:0.191 d:0.072	0.0 74.8	
51	1600	8570.9	251.6 258.2	b,c:0.189 d:0.054	0.0 118.5	
52	1500	8676.4	246.6	b,c:0.146 d:0.136 e:0.164	0.0 104.8 81.3	
53	1400	8762.0	248.8	b,c:0.214 d:0.050	0.0 85.8	
54	1200	8879.1	250.4	b,c:0.140 d:0.144 e:0.125	0.0 83.2 117.1	
55	0.0409	8932.0	238.0	e:0.392 d:0.382	1.6 7.2	
56	0.1573	9172.6	250.0	0.421	44.4-240.6 (1)	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing
(1) the peak stress ratio was not recorded

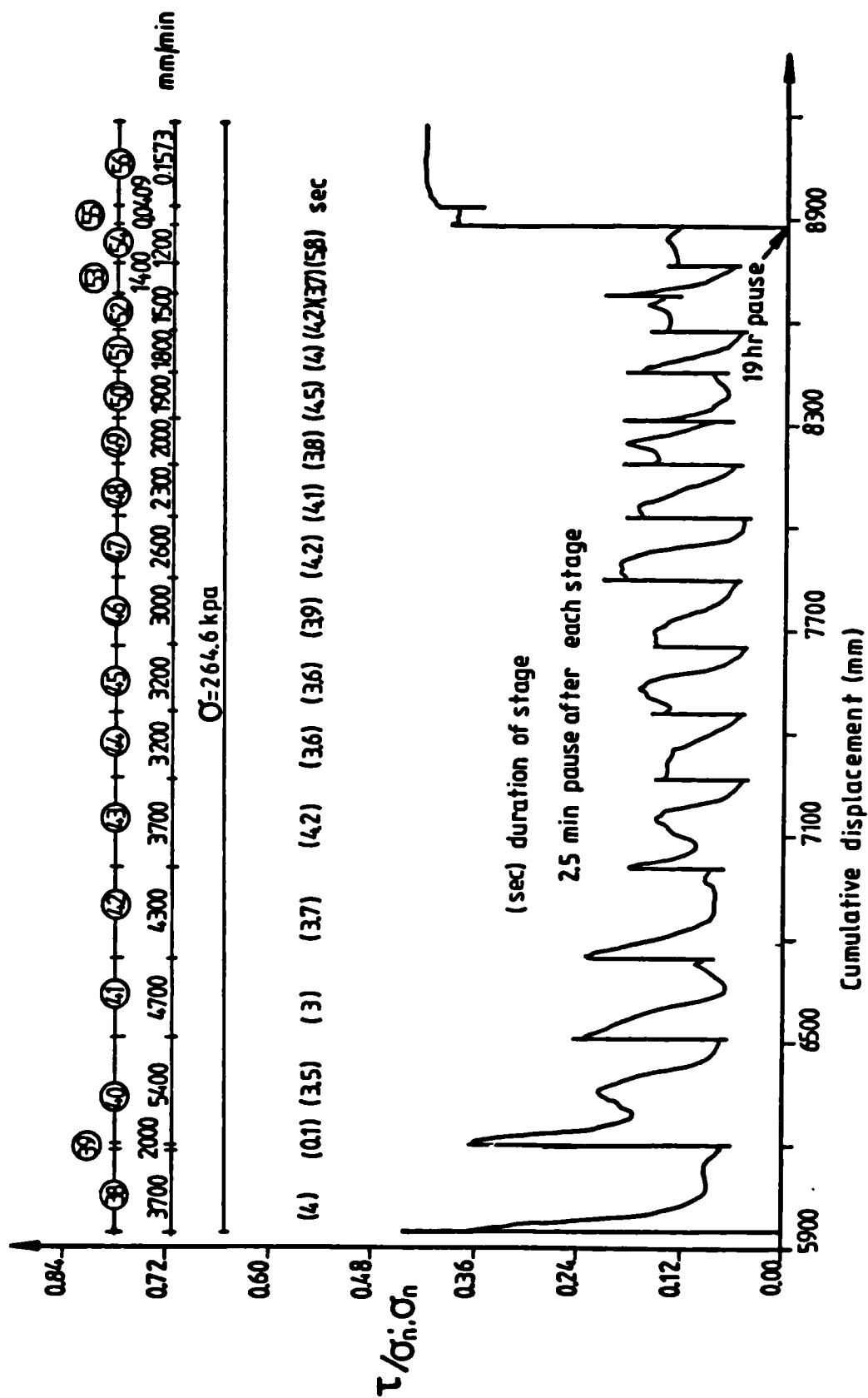


Fig. A2.56d Ring shear test 49 on clayey siltstone D

Table A2.47 Results from ring shear test 50 on the clayey siltstone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0157	297.5	200.3 228.7	peak:0.571 0.281	0.4 216.0-297.5	
2	133	560.1	218.3 219.3 235.5 229.6 231.3	b:0.446 c:0.446 0.379 0.410 0.362	0.2 4.0-10.6 50.0-188 210.0-216 257.0-262.8	
3	0.0157	628.8	230.9 228.1	e:0.341 0.283	0.4 52.8-68.7	
4	1200	816.0	228.3 253.6 238.7	b,c:0.528 d:0.140 0.162	0.4 107.0-117.5 187.2	
5	0.0157	888.1	236.7 231.5	e:0.335 0.264	0.9 56.8-72	
6	350	1063.5	228.3 228.3 238.5	b:0.391 c:0.438 d:0.194	0.3 2.1-2.6 158.1-175.4	
7	0.0157	1140.3	237.9 228.0	e:0.345 0.256	0.7 75.0-76.8	
8	0.0157	1182.0	463.8	0.250	21.4-41.7	
9	133	1416.8	469.7 469.7 485.0	b:0.311 c:0.369 d:0.270	0.2 4.5-5.4 111.6-182.0	
10	0.0157	1456.7	479.4 471.6	e:0.304 0.272	0.2 33.3-39.9	
11	800	1644.0	470.0 470.0 475.0	b:0.383 c:0.435 d:0.300	0.8 5.6 66.2-137.7	
12	0.0157	1688.4	477.5 469.9	e:0.297 0.259	1.0 28.9-44.3	
13	350	1865.4	474.6 480.5 477.1	b:0.329 c:0.385 d:0.271	0.3 1.6-3.9 70.3	
14	0.0157	1901.6	473.6 473.0	e:0.298 0.269	1.0 24.4-36.2	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

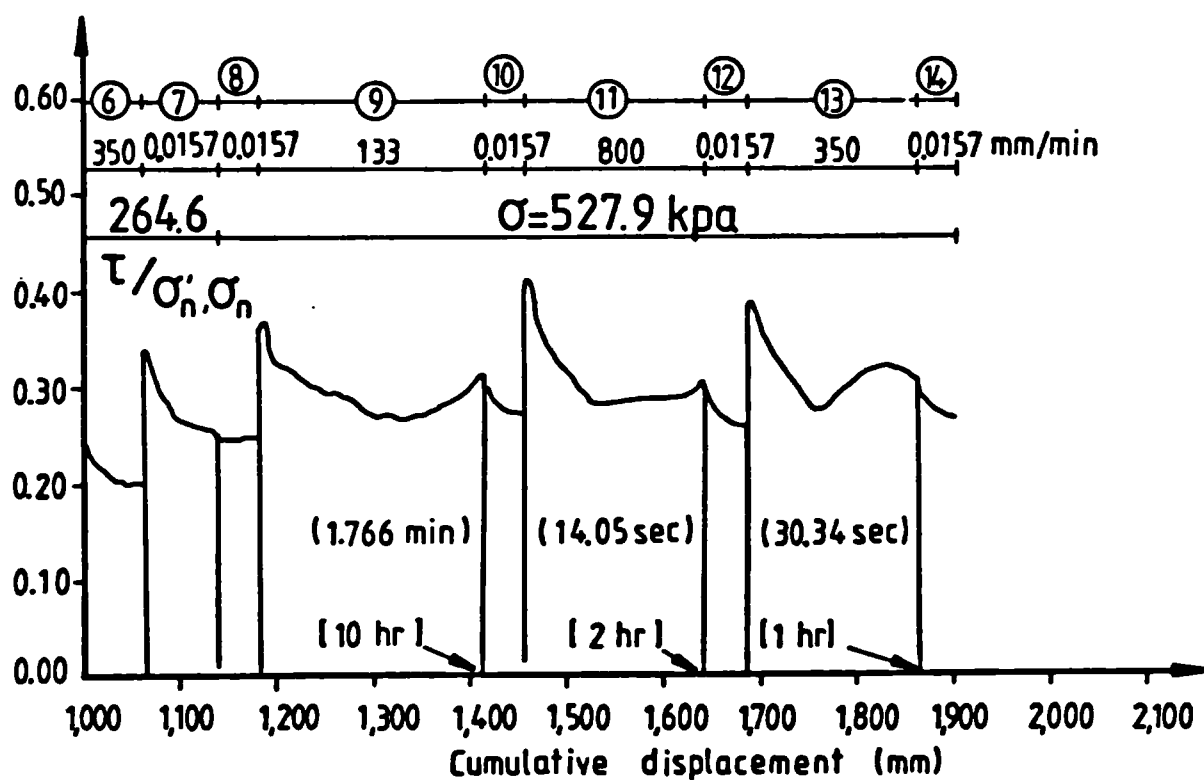
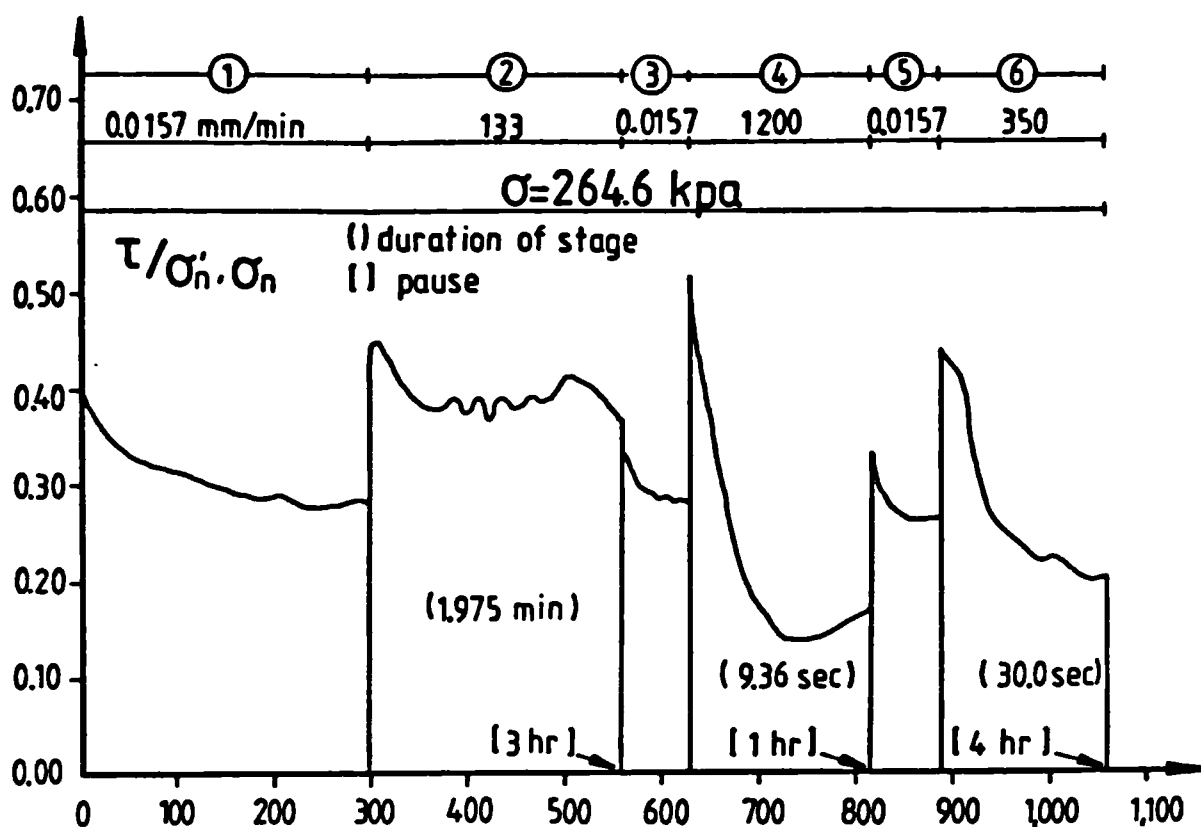


Fig. A2.57 Ring shear test 50 on clayey siltstone D (soil-steel interface)

Table A2.48 Results from ring shear test 51 on clayey siltstone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0150	93.4	121.8	peak:0.712 0.400	0.2 40.0-93.4	
2	380	812.2	136.9 132.0	b,c:0.595 d:0.140	0.7 592.3-812.2	
3	0.0150	966.3	197.3 195.0	e:0.403 0.340	4.3 28.0-60.7	
4	370	2965.0	195.0 195.0	b,c:0.507 0.165	0.6 500.0-1998.7	
5	0.6123	27.4	192.0 195.0	0.164 0.305	0.0 6.0-27.4	
6	0.0150	3027.0	201.0 203.0	e:0.323 0.310	1.9-3.0 30.0-34.7	
7	0.6295	3105.4	189.9 192.0	c:0.340 0.290	0.7 58.3-78.4	
8	0.0150	3134.4	379.5	e:0.263	10.8-28.6	
9	370	3746.8	383.0 389.0	b,c:0.414 d:0.247	0.5 284.7-612.3	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

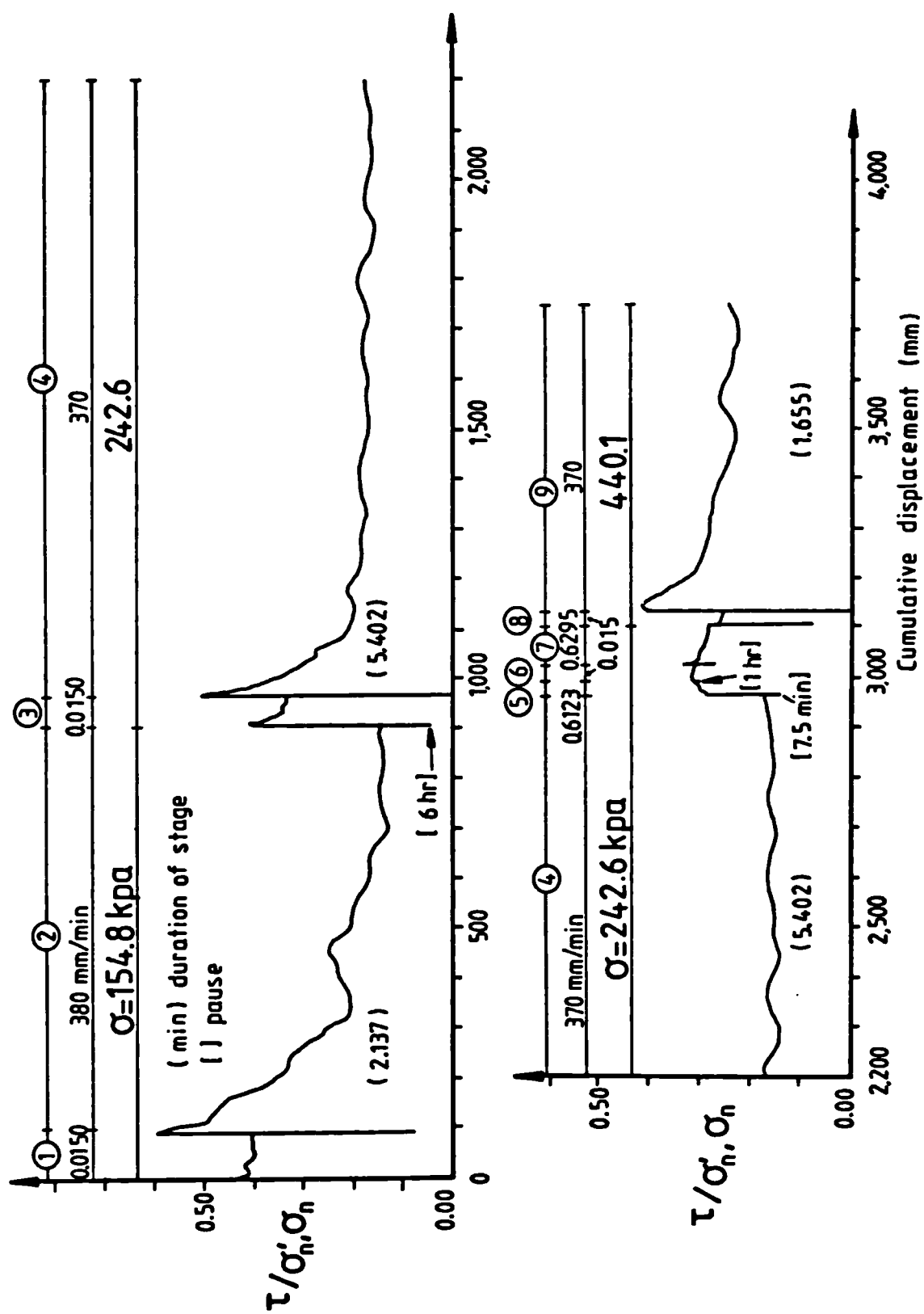


Fig. A2.58a Ring shear test 51 on clayey siltstone D
(soil-glass interface)

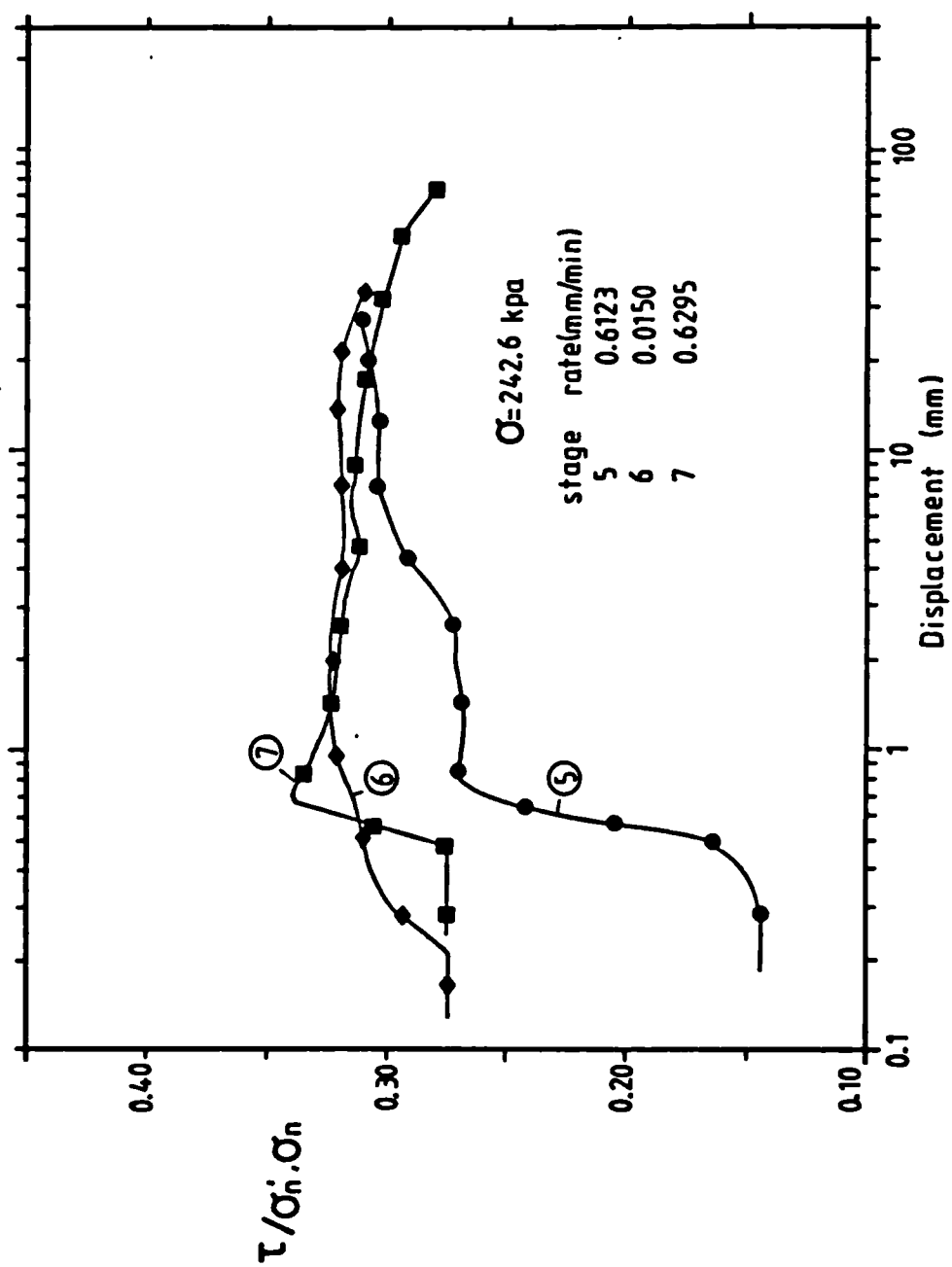


Fig. A2.58b Ring shear test 51 on clayey siltstone D
(soil-glass interface)

Table A2.49 Results from ring shear test 52 on claystone D

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
1	0.0141	122.7	238.3 239.0	peak:0.497 0.226	0.4 113.0-122.7	
2	400	1285.2	237.4 237.7 225.4	b:0.261 c:0.340 d:0.100	0.1 0.4 284.5-1162.5	
3	0.0141	1641.84	243.2 240.0	e:0.256 0.200	0.7-2.3 30.0-356.6	
4	370	3347.0	223.9 225.0 223.0	b:0.255 c:0.354 d:0.092	0.7 4.2 239.8-1705.1	
5	0.0141	3554.0	233.0 217.0	e:0.301 0.186	2.6-2.8 203.9-207.0	
6	400	4817.2	233.4 236.2 231.6	b:0.259 c:0.348 d:0.082	0.2 3.1-3.8 172.9-1263.2	
7	1.5379	4914.6	221.8 233.0	0.195 0.144	0.1 81.5-97.5	
8	0.0141	5346.0	225.5	e:0.184 0.180	2.8-3.6 5.0-431.4	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

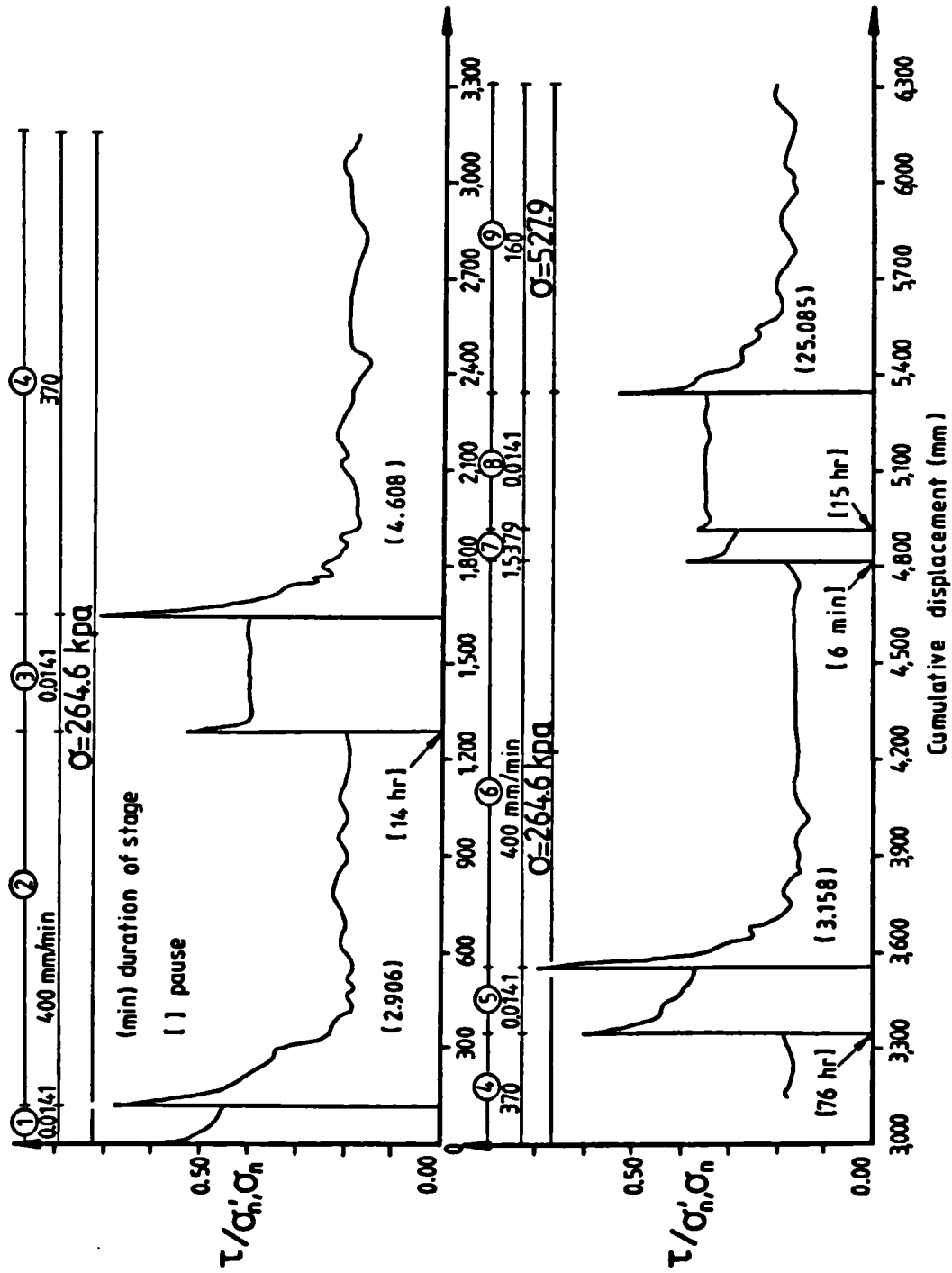


Fig. A2.59a Ring shear test 52 on claystone D
(soil-glass interface)

Table A2.49 (continued)

Stage	Rate of displacement (mm/min)	Cumulative displacement (mm)	Net normal stress σ_n (kPa)	Stress ratio	At a displacement of (mm)	Notes
9	160	9359.6	498.7 490.0	b,c:0.266 d:0.083	0.1 400.0-4013.5	
10	0.0050	9453.1	490.2 485.0	e:0.211 0.185	0.6-0.7 34.0-93.4	

b: threshold, c: fast peak, d: fast residual, e: slow peak after fast shearing

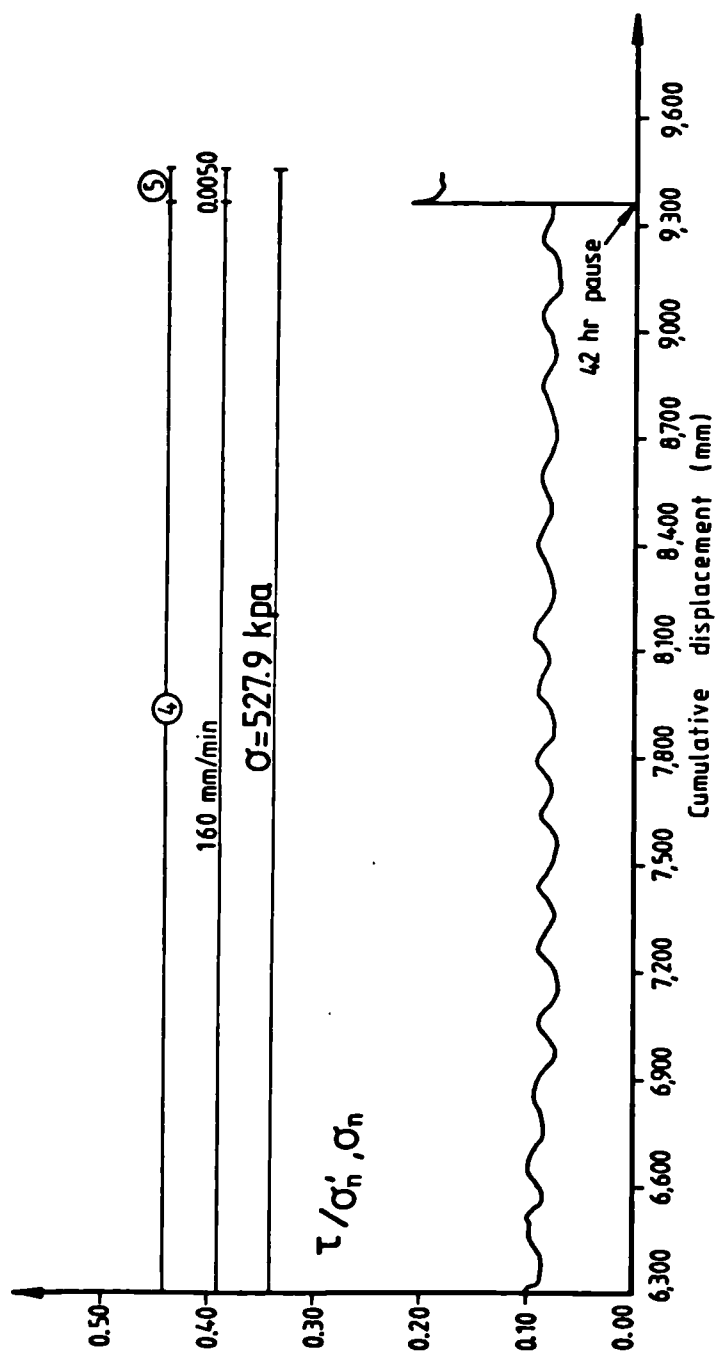


Fig. A2.59b Ring shear test 52 on claystone D
(soil-glass interface)

APPENDIX 3

SHEAR BOX TESTS

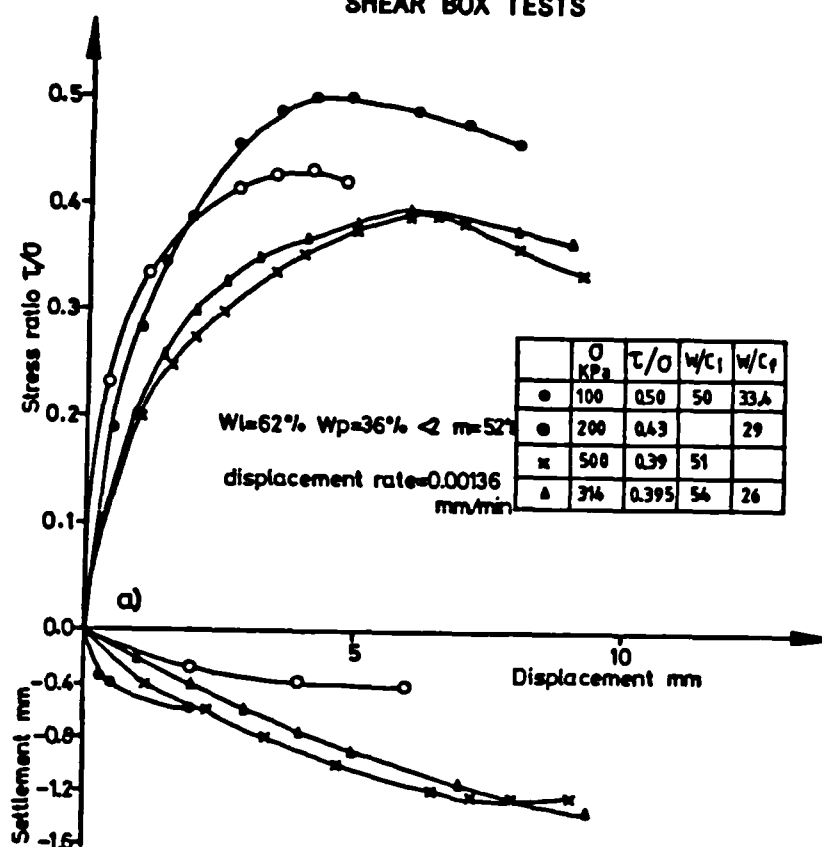


Fig. A3.1 Results from a set of shear box tests on claystone A

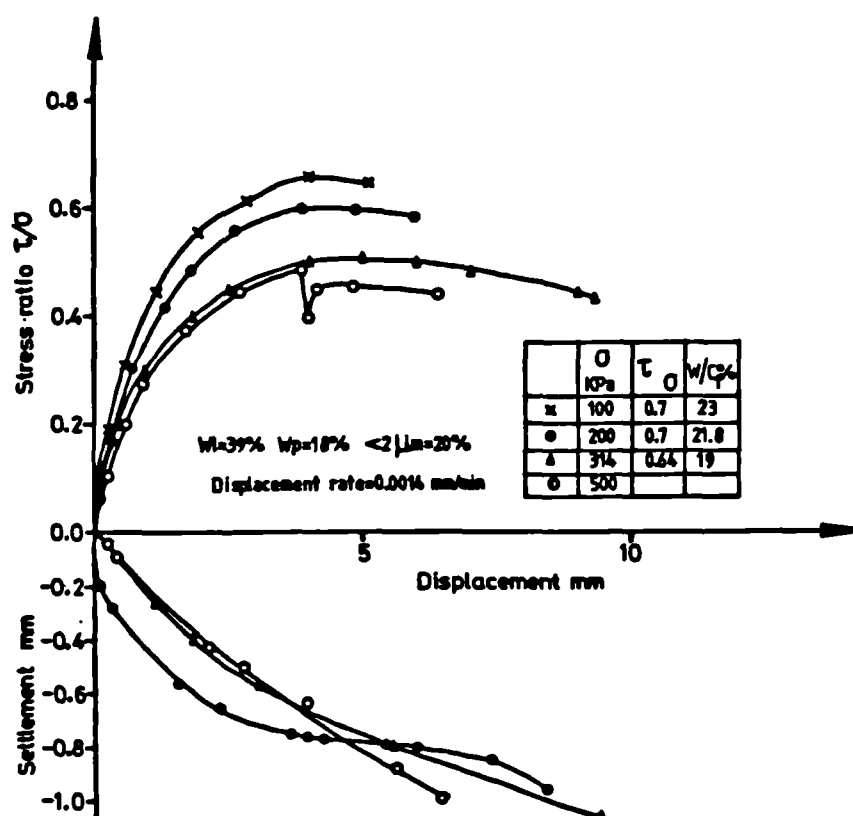


Fig. A3.2 Results from a set of shear box tests on clayey siltstone A

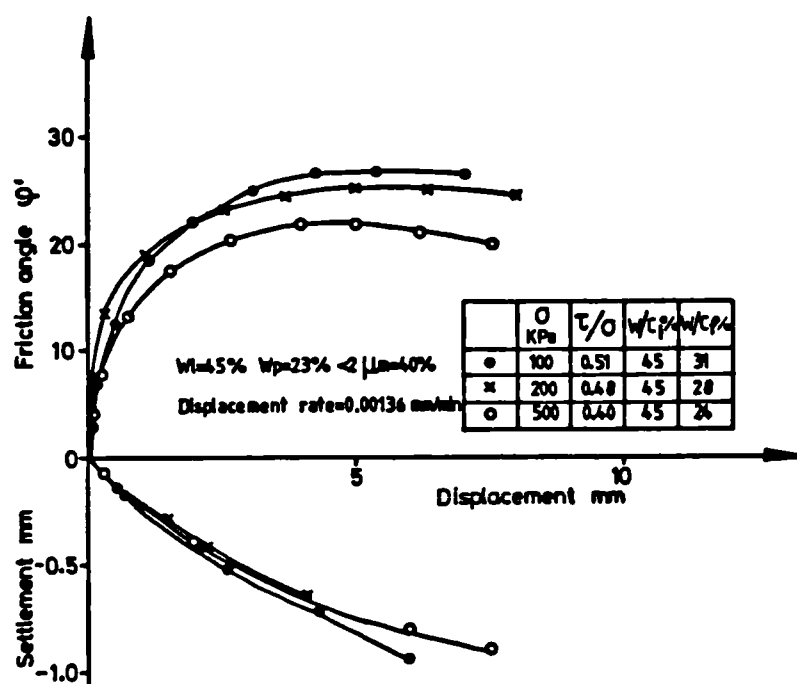


Fig. A3.3 Results from a set of shear box tests on claystone B

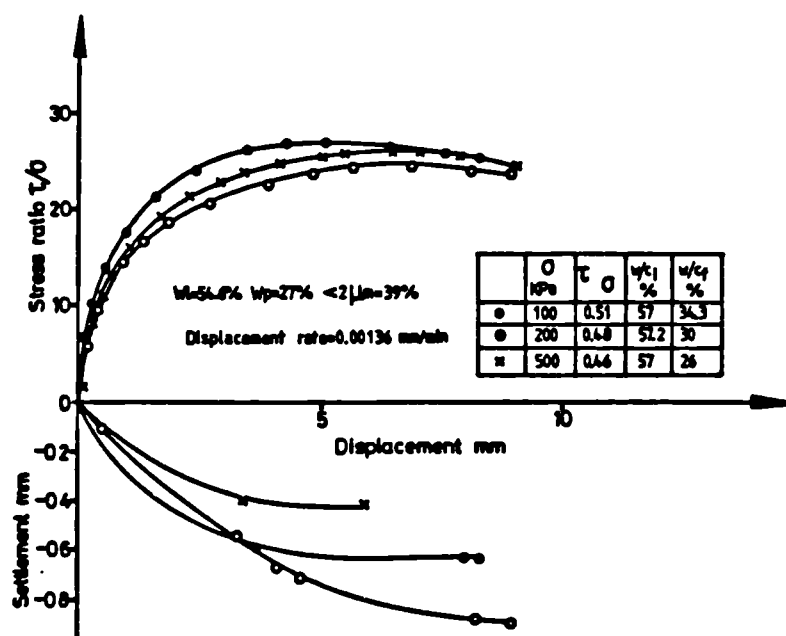


Fig. A3.4 Results from a set of shear box tests on claystone C

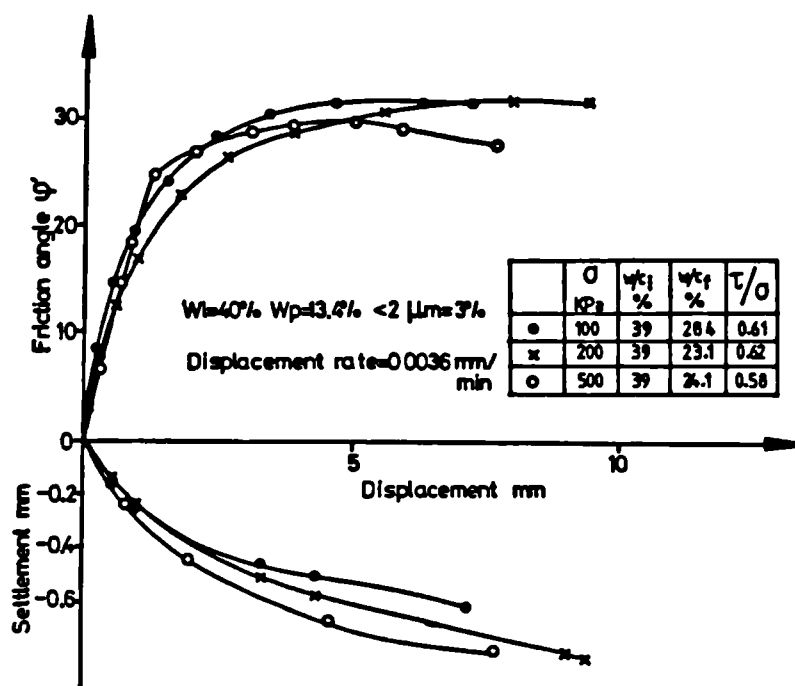


Fig. A3.5 Results from a set of shear box tests on siltstone

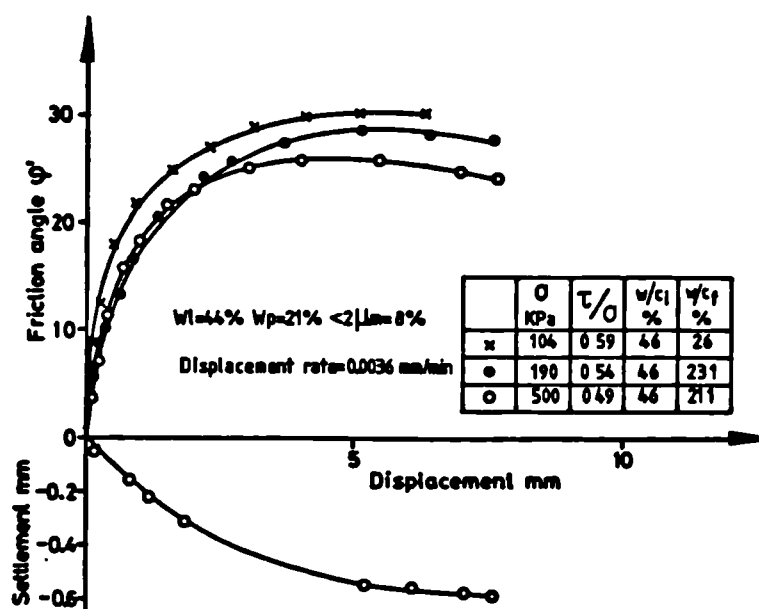


Fig. A3.6 Results from a set of shear box tests on clayey siltstone B

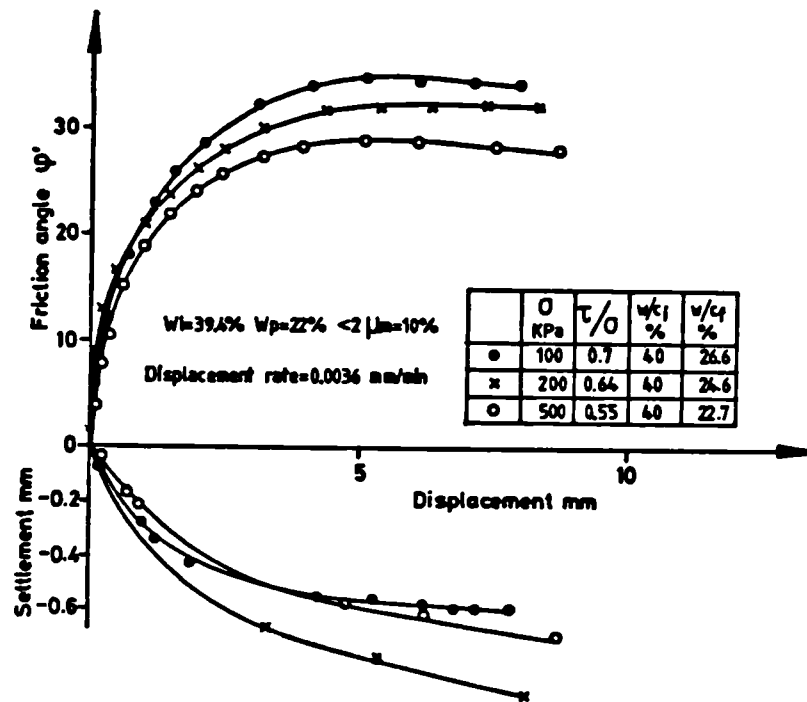


Fig. A3.7 Results from a set of shear box tests on clayey siltstone C

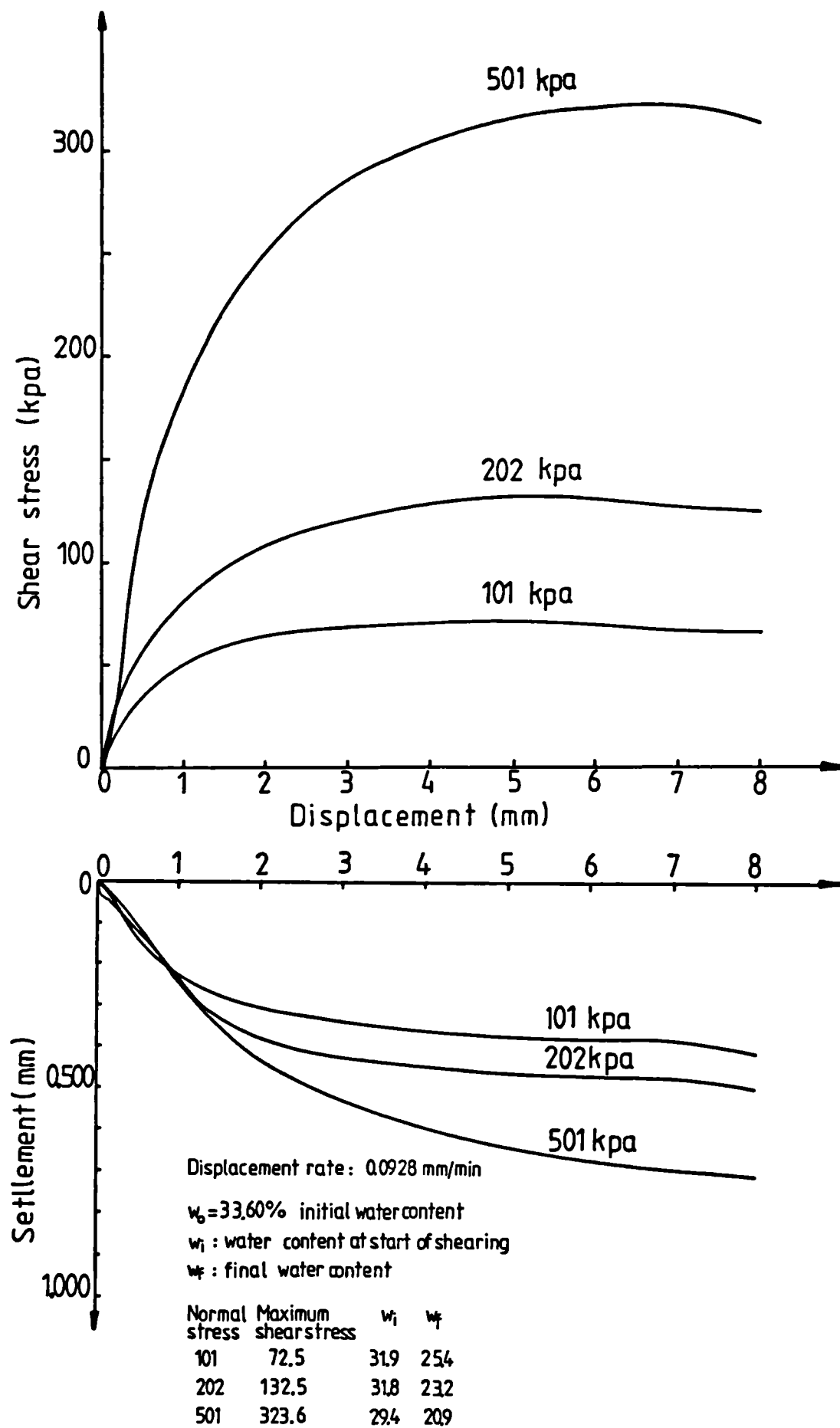


Fig. A3.8 Results from a set of shear box tests on sandstone

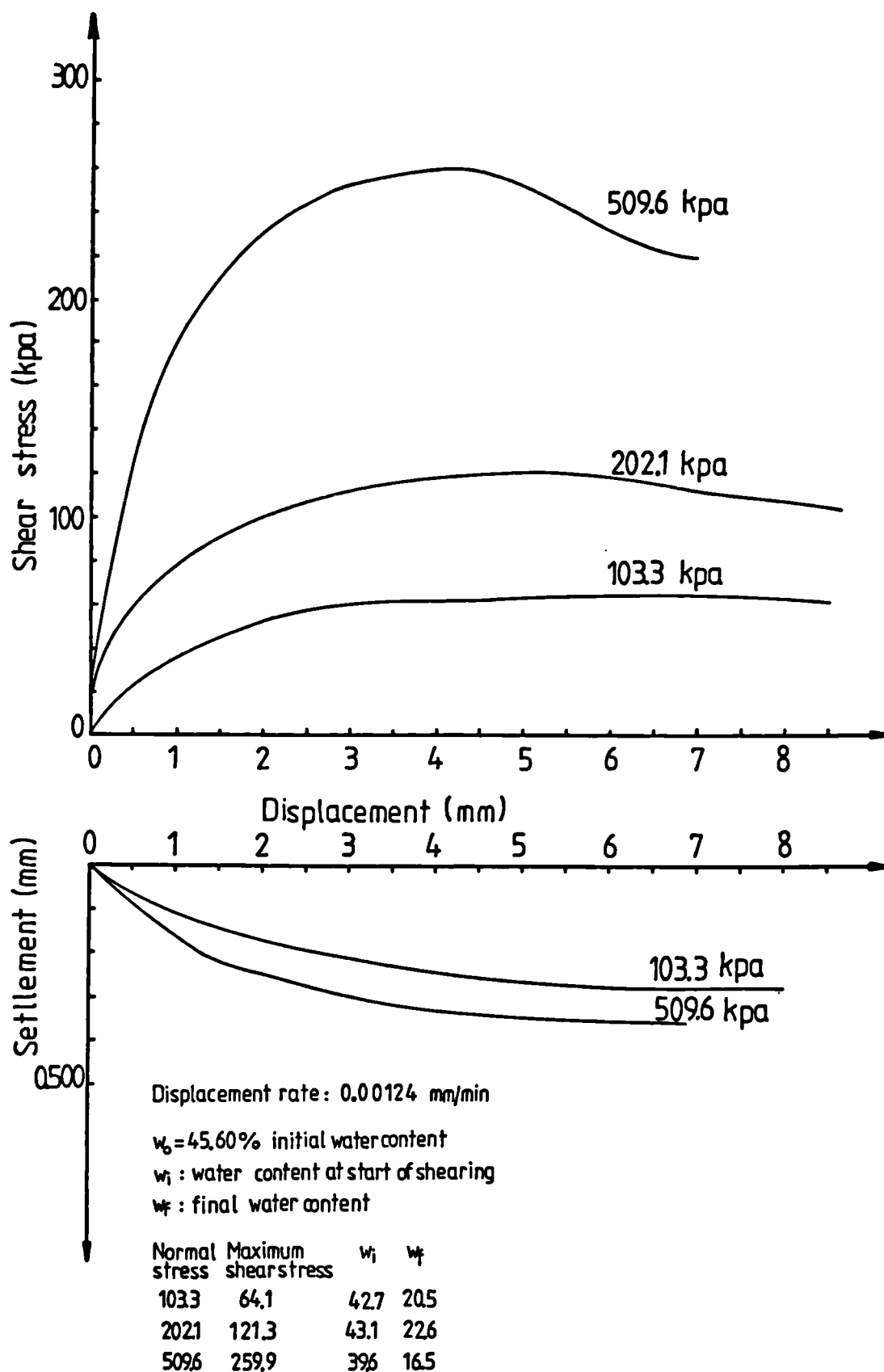


Fig. A3.9 Results from a set of shear box tests on clayey siltstone D

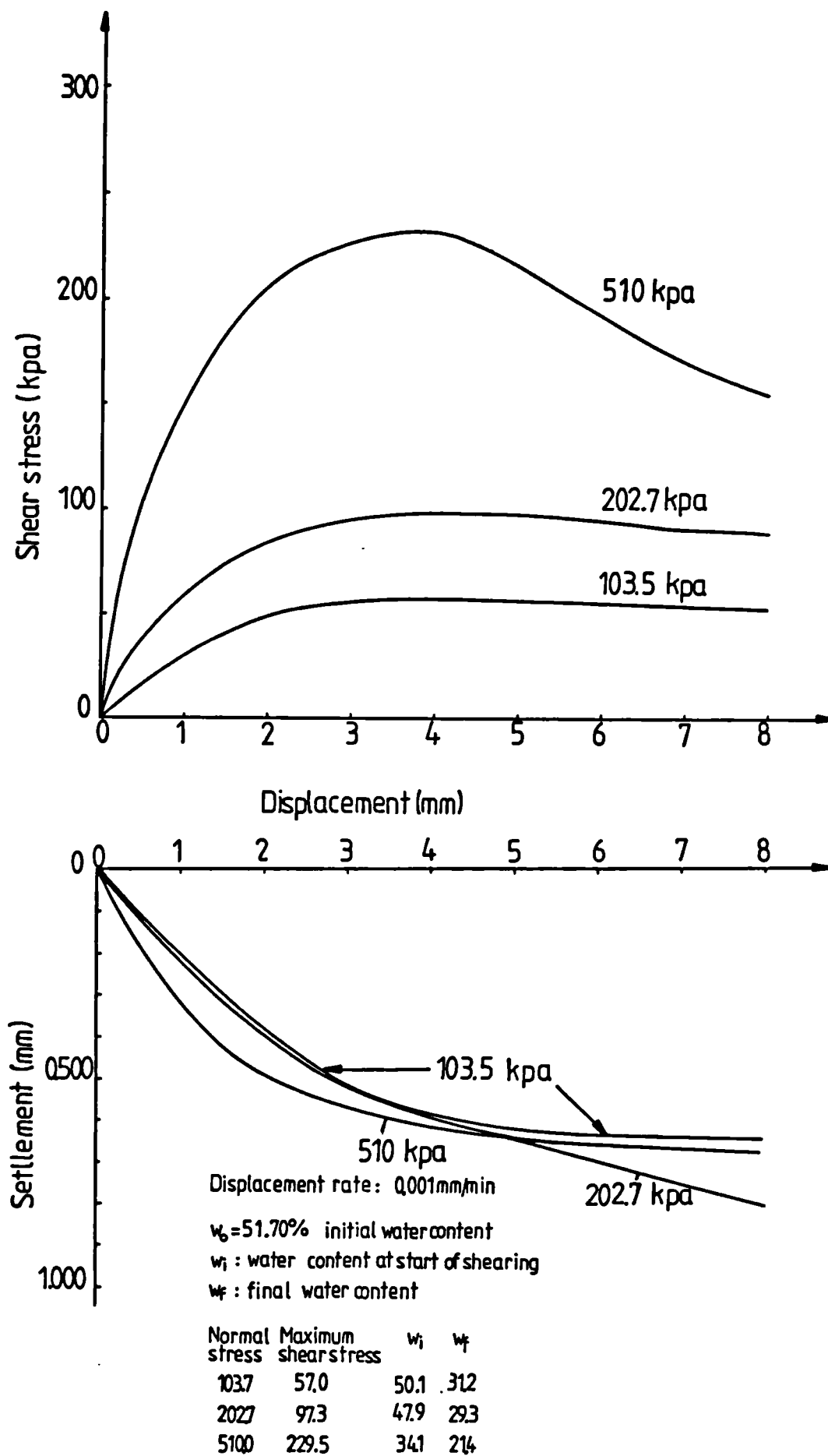


Fig. A3.10 Results from a set of shear box tests on claystone D

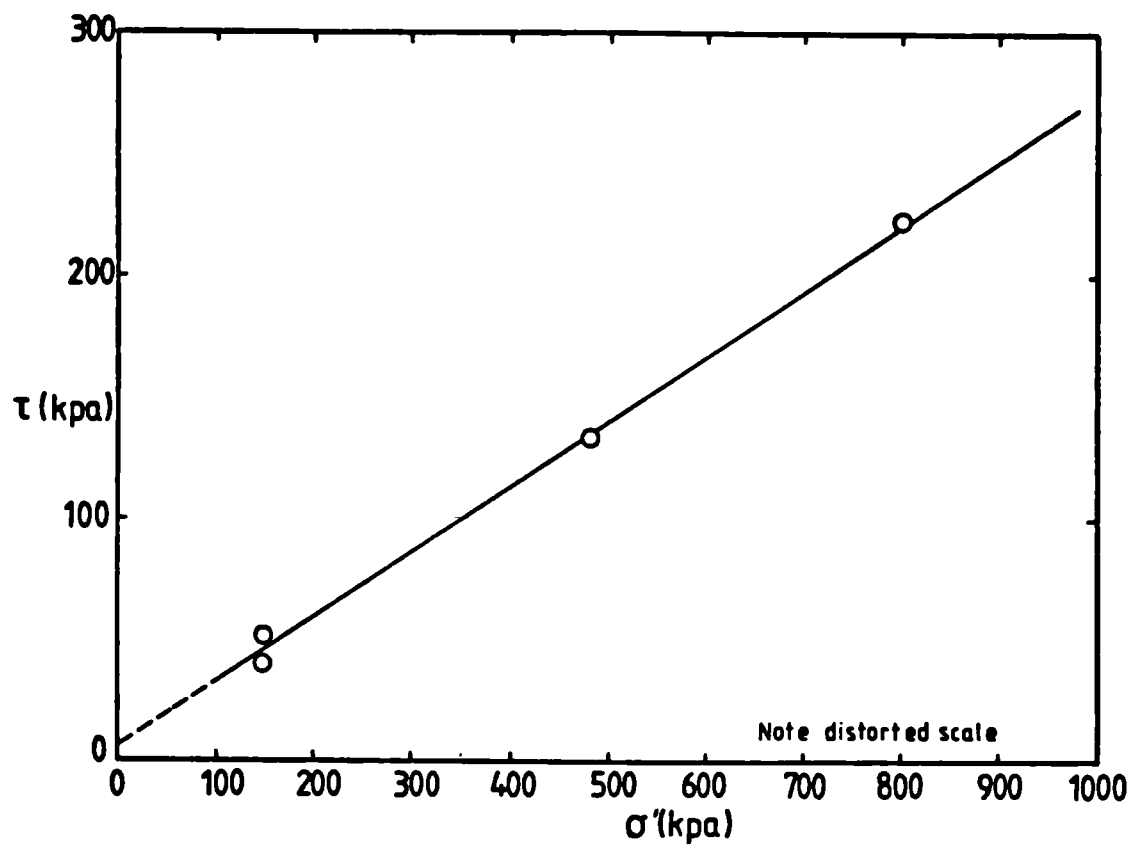


Fig. A3.11 Normally-consolidated peak strength envelopes of soil from Senerchia slide

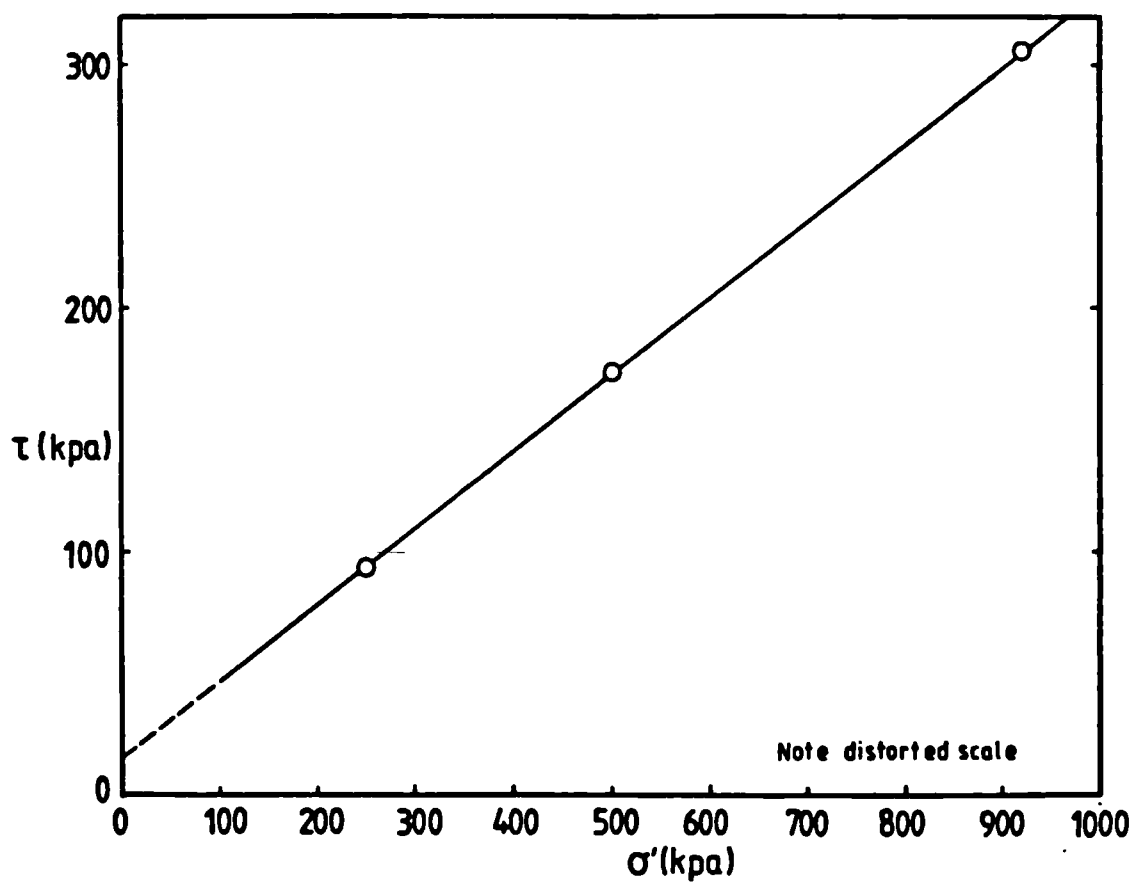


Fig. A3.12 Normally-consolidated peak strength envelopes of soil from Buoninvetre slide

APPENDIX 4

RESPONSE TIME OF PORE WATER PRESSURE MEASURING SYSTEM

Water must flow from the shear zone into the measuring system to compress the water present between the shear zone and the diaphragm of the transducer and to deflect the diaphragm.

The volume of the water in the measuring system (porous stone, hypodermic stainless steel tubes, valves, and cavity between the porous stone and the diaphragm) is 539.2 mm^3 . Assuming that the shear zone is formed 1mm away from the interface surface and that the porosity of soil is 35%, the total volume of water between the shear zone and the diaphragm of the transducer is:

$$V_w = 583.5 \text{ mm}^3 \quad (\text{A4.1})$$

The volume of water that must flow into the measuring system to compress the water is:

$$\Delta V_w = V_w \cdot C_w \cdot p \quad (\text{A4.2})$$

where C_w is the compressibility of de-aired water,

$C_w = 4.20 \times 10^{-7} / \text{kPa}$ (Lange, 1966) and

p is the pressure inside the measuring system, at given time, when a pressure p_0 is applied externally to the system.

Differentiating equation (A4.2) with respect to time, we obtain:

$$\frac{\Delta V_w}{\Delta t} = V_w \cdot C_w \cdot \frac{\Delta p}{\Delta t} \quad (\text{A4.3})$$

or

$$\frac{\Delta V_w}{\Delta t} = 24.51 \times 10^{-14} \frac{\Delta p}{\Delta t} \quad (\text{A4.4})$$

The flow rate of water from the shear zone and through the porous stone is:

$$Q = \frac{\Delta V}{\Delta t} = -k \cdot l \cdot A \quad (\text{A4.5})$$

where k is the permeability which for the tested soils is within the range of 3×10^{-11} to 2×10^{-10} m/sec, and

i is the hydraulic gradient,

$$i = \frac{1}{\gamma_w} \frac{(p-p_0)}{l} \quad (\text{A4.6})$$

where γ_w is the unit weight of water,

l is the sum of the distance of the shear zone from the interface and the thickness of the porous stone, and

A is the porous stone surface (diameter : 0.50 in)

Substituting the values of k , i and A , we obtain:

$$Q = \frac{\Delta V}{\Delta t} = -(9.29 \times 10^{-14} \sim 61.93 \times 10^{-14}) (p-p_0) \quad (\text{m}^3/\text{sec}) \quad (\text{A4.7})$$

Since $\frac{\Delta V_w}{\Delta t} = \frac{\Delta V}{\Delta t}$, we have:

$$-(9.29 \times 10^{-14} \sim 61.93 \times 10^{-14}) (p-p_0) = 24.51 \times 10^{-14} \frac{\Delta p}{\Delta t} \quad (\text{A4.8})$$

or

$$24.51 \frac{\Delta p}{\Delta t} + (9.29 \sim 61.93) (p-p_0) = 0 \quad (\text{A4.9})$$

or

$$p + C \left(\frac{\Delta p}{\Delta t} \right) = p_0 \quad (\text{A4.10})$$

with $C = 0.396 \sim 2.638$

For a unit applied pressure, $p_0 = 1$, equation (A4.10) gives:

$$p + C \left(\frac{\Delta p}{\Delta t} \right) = 1 \quad (\text{A4.11})$$

and

$$p = 1 - e^{-t/C} \quad (\text{A4.12})$$

For 95% equalization of the pore water pressure ($p/p_0=0.95$), the response time of the measuring system is:

$$t = 2.996 C \quad (\text{A4.13})$$

or

$$t = 1.20 \sim 7.90 \text{ sec} \quad (\text{A4.14})$$

APPENDIX 5

TEMPERATURE INDUCED PORE WATER PRESSURE CHANGES IN SATURATED SOILS

Following the approach adopted by Bishop (1973b), and Campanella and Mitchell (1968), the pore water pressure change accompanying a temperature change under undrained conditions of constant total stress can be derived as follows:

The volume changes of the pore water and solid material due to both total stress and temperature changes must be equal to the sum of the volume changes of the total soil mass due to both temperature and pressure changes:

$$(\Delta V_w)_{\Delta\theta} + (\Delta V_s)_{\Delta\theta} + (\Delta V_w)_{\Delta\sigma} + (\Delta V_s)_{\Delta\sigma} = (\Delta V)_{\Delta\theta} + (\Delta V)_{\Delta\sigma} \quad (A5.1)$$

in which ΔV_w is the volume change of the pore water,
 ΔV_s is the volume change of the solid material
 ΔV is the volume change of the soil mass,
 and the subscripts $\Delta\theta$ and $\Delta\sigma$ represent temperature and total stress changes respectively.

The volume changes due to temperature changes are:

$$(\Delta V_w)_{\Delta\theta} = a_w \cdot V_w \cdot \Delta\theta \quad (A5.2)$$

$$(\Delta V_s)_{\Delta\theta} = a_s \cdot V_s \cdot \Delta\theta \quad (A5.3)$$

where, a_w and a_s are the thermal coefficients of cubical expansion of water and soil grains respectively, which are both temperature and confining pressure dependent, and

$$(\Delta V)_{\Delta\theta} = a_s \cdot V \cdot \Delta\theta + (\Delta V_{st})_{\Delta\theta} \quad (A5.4)$$

as the components of the volumetric strain along any continuous path across the solid skeleton within a soil element are equal to those within the solid material of the skeleton.

The term $(\Delta V_{st})_{\Delta\theta}$ represents the structural volume change of the total soil mass, caused by the change in the interparticle forces due to the

reorientation or relative movement of the soil grains to permit the soil structure to carry the same effective stress.

The volume changes due to total stress changes are:

$$(\Delta V_w)_{\Delta\sigma} = m_w \cdot V_w \cdot \Delta u \quad (A5.5)$$

where, m_w is the compressibility of water, and

Δu is the pore water pressure change under undrained conditions

$$(\Delta V_s)_{\Delta\sigma} = m_s \cdot V_s \cdot \Delta u + m'_s \cdot V_s \cdot (\Delta\sigma - \Delta u)$$

or

$$(\Delta V_s)_{\Delta\sigma} = m_s \cdot V_s \cdot \Delta u + m'_s \cdot V_s \cdot \Delta\sigma' \quad (A5.6)$$

where, m_s is the compressibility of the mineral solid material under an isotropic stress,

m'_s is the compressibility of the mineral solid material under a deviatoric stress, which is of the same order as m_s ($m_s \approx m'_s$), and

$$(\Delta V)_{\Delta\sigma} = m \cdot V \cdot (\Delta\sigma - \Delta u)$$

or

$$(\Delta V)_{\Delta\sigma} = m \cdot V \cdot \Delta\sigma' \quad (A5.7)$$

where, m denotes the compressibility of the soil mass

If the total stress remains constant ($\Delta\sigma=0$) during the temperature change $\Delta\theta$, then:

$$\Delta\sigma' = -\Delta u \quad (A5.8)$$

Substituting equations (A5.2) to (A5.8) into (A5.1) yields:

$$a_w \cdot V_w \cdot \Delta\theta + a_s \cdot V_s \cdot \Delta\theta - a_s \cdot V \cdot \Delta\theta - (\Delta V_{st})_{\Delta\theta} = -m \cdot V \cdot \Delta u - m_w \cdot V_w \cdot \Delta u \quad (A5.9)$$

since $V = V_s + V_w$ and $n = V_w/V$

equation (A5.9) yields:

$$\Delta u = \frac{-n \cdot \Delta\theta(a_w - a_s) + \frac{(\Delta v_{st})\Delta\theta}{v}}{m + n \cdot m_w} \quad (\text{A5.11})$$

$$\Delta u = \frac{-n \cdot \Delta\theta(a_w - a_s) + a_{st} \cdot \Delta\theta}{m + n \cdot m_w} \quad (\text{A5.11})$$

where, $a_{st} = \frac{(\Delta v_{st})\Delta\theta}{v \cdot \Delta\theta}$ is the physical - chemical coefficient of structural volume change, caused by a change in temperature.

APPENDIX 6

PLATES

PLATE 1*Photographs 1 to 3, sandstone*

(Ring shear test 16)

Photograph 1 Thin section after the ring shear test, viewed in the camera set-up under crossed polars; showing some degradation of particles inside the shear zone.

Photographs 2,3. Thin section after the ring shear test, viewed in the optical microscope under crossed polars. Photograph 2 shows the structure of the upper part of the sample far away from the shear zone. Photograph 3 shows the structure at the shear zone.

Photographs 4 to 7, clayey siltstone D

(Soil consolidated in an oedometer from a remoulded state)

Photographs 4,5. Thin section of the sample, showing the structure of the remoulded material. Photograph 4 shows a view in the camera set up under crossed polars. Photograph 5 shows a view in the optical microscope under crossed polarized light.

Photographs 6,7. Scanning-electron micrographs of the sample; showing clay particles aggregation.

PLATE 2*Photographs 8 to 15, clayey siltstone D*

(Ring shear test 18, structure of the shear zone formed at the slow residual state).

Photograph 8. Thin section after the ring shear test, viewed in the camera set-up under crossed polars. Shear zone and discontinuous shear surfaces are shown.

Photographs 9, 10. Thin section of the sample after the ring shear

test, viewed in the optical microscope under crossed polars. Photograph 9 shows the shear zone, bounded by discontinuous shear surfaces. The upper shear surface is partly infilled with Carbowax. Shear zone contains discontinuous shear surfaces. Parallel orientation to the direction of shear of clay particles and shear surfaces within the shear zone is shown. Photograph 10 is an enlargement of photograph 9 at the boundary of the shear zone.

Photograph 11. Scanning-electron micrograph of the principal shear surface after the ring shear test, viewed normal to the surface. Striations direction and clay particles alignment are clear. Clay particles leave gap in places. Silt size particles are seen to intrude the shear surface.

Photographs 12 to 15. Scanning-electron micrographs of the shear zone after the ring shear test 18. The shear surfaces are normal to the plane of the photograph. Shear direction is from the right to the left. Photograph 12 shows discontinuous shear surfaces and preferred particle orientation within the shear zone. Photographs 13 and 14 are details of photograph 12 at the boundary of the shear zone with the lower part of the sample. Photograph 15 is an enlargement of photograph 12; showing the structure of the soil within the shear zone.

PLATE 3

Photographs 16 to 23, clayey siltstone D

(Ring shear test 19, structure of the shear zone formed when the fast peak strength operated).

Photograph 16. Thin section after the ring shear test, viewed in the camera set-up under crossed polars. Shear zone is shown.

Photographs 17 and 18. Thin section after the ring shear test, viewed in the optical microscope under crossed polars. Photograph 17 shows the shear zone, bounded by discontinuous shear surfaces. The shear zone is thicker than that formed at the slow residual state (compare photographs 16 and 17 with 8 and 9 respectively). Shear surfaces and clay particles within the shear zone are orientated at 30° with respect to shear direction. Photograph 18 is an enlargement of photograph 17 at the boundary of the shear zone.

Photograph 19. Scanning electron-micrograph of the principal shear surface after the ring shear test 19, viewed normal to the shear surface. The lack of the preferred clay particles orientation, observed when the slow residual strength operates, is noticeable (compare with photograph 11)

Photographs 20 to 22. Scanning electron micrographs from the boundary of the shear zone. The shear surfaces are normal to the plane of the photographs. Shear direction is from the right to the left of the photograph. Photographs 20 and 21 show the orientation of the clay particles and elongated massive grains with respect to shear direction. Photograph 22 is an enlargement of photograph 20; showing the structure within the shear zone. The structure is more porous than that formed at the slow residual state (compare photographs 15 and 22). Photograph 23 is an enlargement of photograph 20 at the boundary of the shear zone.

PLATE 4

Photographs 24 to 31, clayey siltstone D

(Ring shear test 20, structure of the shear zone formed when the fast residual strength operated).

Photograph 24. Thin section after the ring shear test, viewed in the camera set-up under crossed polars. Shear zone and shear surfaces are shown.

Photographs 25, 26 and 27. Thin section after the ring shear test, viewed in the camera set-up under crossed polarized light. Photograph 25 shows shear zone, bounded by discontinuous shear surfaces. Shear zone is considerably thicker than those formed when the slow residual and the fast peak strengths operate (compare photographs 24 and 25 with 8 and 9, and 16 and 7 respectively). Clay particles and elongated massive grains are preferentially orientated to the direction of shear. The shear surfaces within the shear zone are less and smaller than those observed at the slow residual and the fast peak states. Photograph 26 shows the structure within the shear zone. Photograph 27 is an enlargement of photograph 26.

Photograph 28. Scanning-electron micrograph of the principal shear surface after the ring shear test. View is normal to the principal

shear surface. Clay particles alignment is intermediate between those corresponding to the slow residual and the fast peak strengths.

Photographs 29, 30 and 31. Scanning electron-micrographs from the shear zone after the ring shear test. Shear surfaces are normal to the plane of the photographs. Shear direction is from the left to the right. Photograph 29 shows clay particles orientation in the shear zone. Massing grains intrude the shear plane. Photographs 30 and 31 are enlargements of photograph 29 at the boundary of the shear zone.

PLATE 5

Photograph 32, Lower Cromer Till

(Ring shear test 23)

Photograph 32. Thin section after the ring shear test, viewed in the camera set-up under crossed polars; showing the shear zone. Shear direction is from the upper left to the mid-right of the photograph.

Photographs 33, 34 and 35, residual soil from Mauritius.

(Ring shear test 28).

Photograph 33. Thin section after the ring shear test; viewed in the camera set-up under crossed polars; showing degradation of the particles within the shear zone.

Photographs 34 and 35. Thin section after the ring shear test; viewed in the optical microscope under polarized light. Photograph 34 shows the structure of the soil away from the shear zone. Photograph 35 shows the shear zone, bounded by discontinuous shear surfaces.

Photographs 36, 37 and 38, residual soil from Japan

(Ring shear test 29).

Photographs 36, 37 and 38. Thin section after the ring shear test, viewed in the optical microscope under crossed polarized light. Photograph 6 shows the shear zone. Its boundaries are not planar due to the interference of the massive grains. Elongated massive grains are preferentially orientated within the shear zone. Photograph 37 is an enlargement of photograph 36; showing the structure of the shear

zone. Photograph 38 shows the structure of the soil far away from the shear zone.

PLATE 6

Photographs 39 and 40, blue London clay
(Ring shear tests 38 and 40).

Photographs 39, 40. Scanning-electron micrographs after the ring shear test 38. View normal to the principal shear surface. Photograph 39 shows preferred orientation of clay particles on the shear plane and slight striations. Clay particles leave gaps in places. Silt size particles intrude the shear surface. Photograph 40 is an enlargement of photograph 39.

Photographs 41 to 45. Scanning-electron micrographs after the ring shear test 40 (soil-steel interface). View normal to the principal shear surface. Photograph 41 shows clay particles orientation and direction of striations in the shear plane. Striations are heavier than those observed in the ring shear test 38 (photograph 39). Silt size particles intrude the shear surface. Photograph 42 is an enlargement of photograph 41. Photograph 43 shows the transition zone between the principal shear surface and the interface. Clay particles are orientated parallel to the shear direction. Photograph 44 is an enlargement of photograph 43. Photograph 45 shows the soil in touch with the interface. Striations are absent.

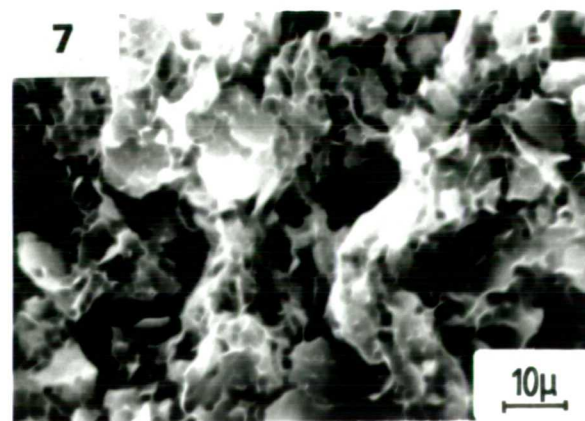
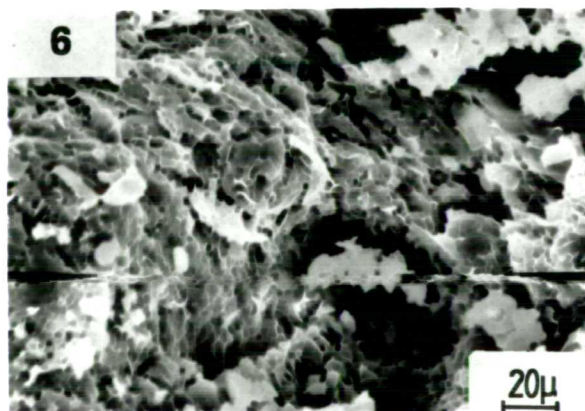
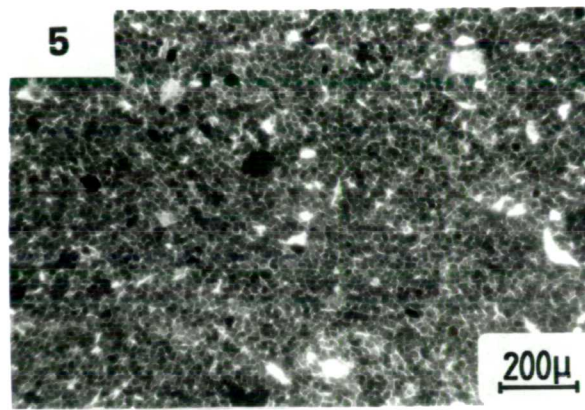
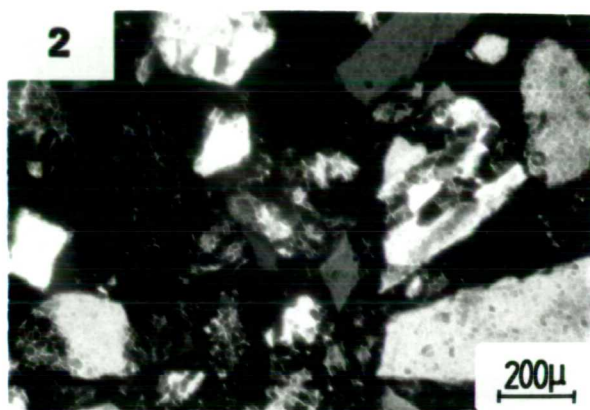
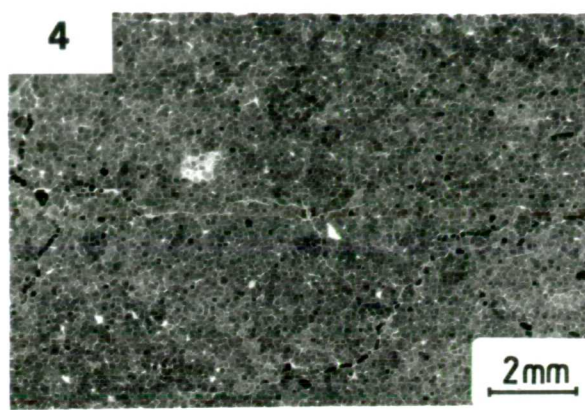
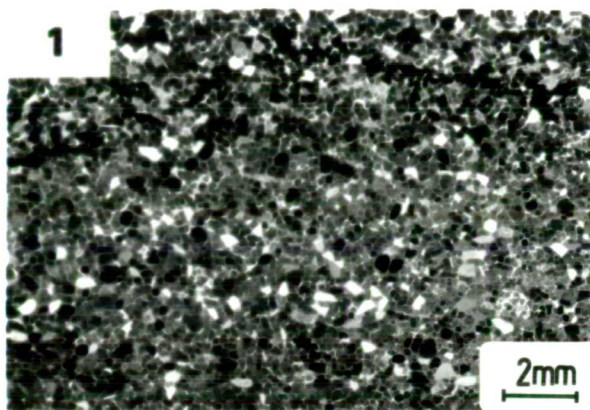
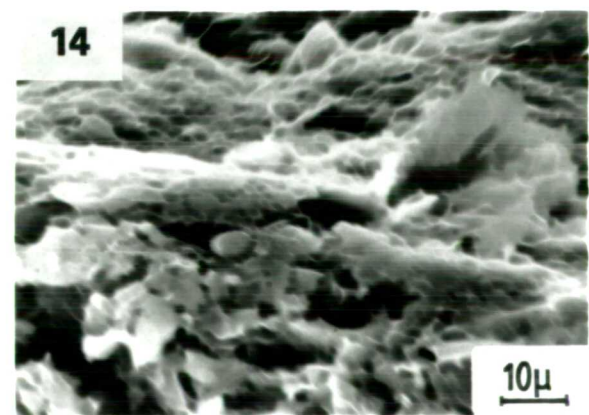
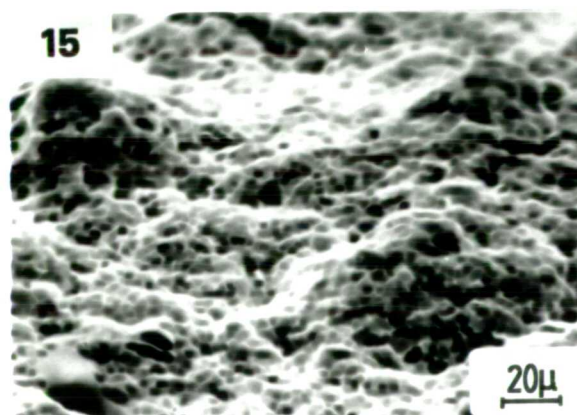
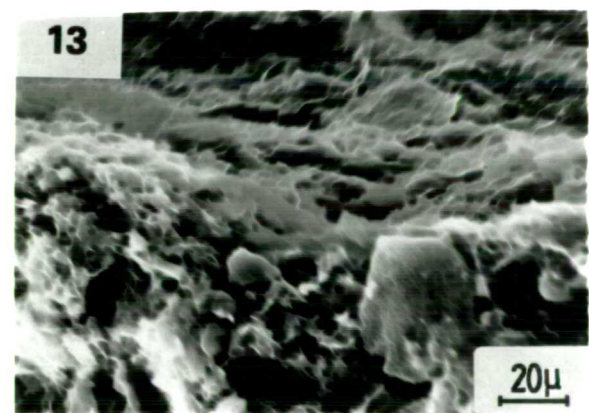
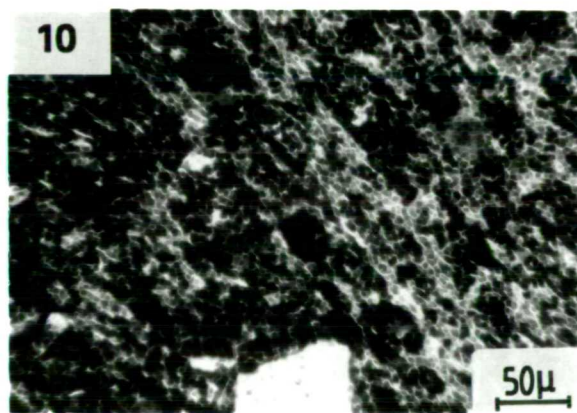
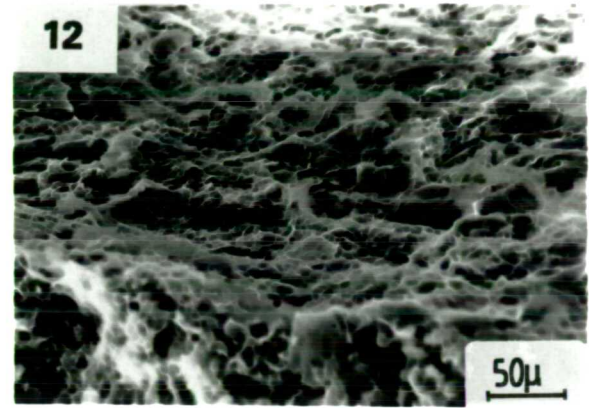
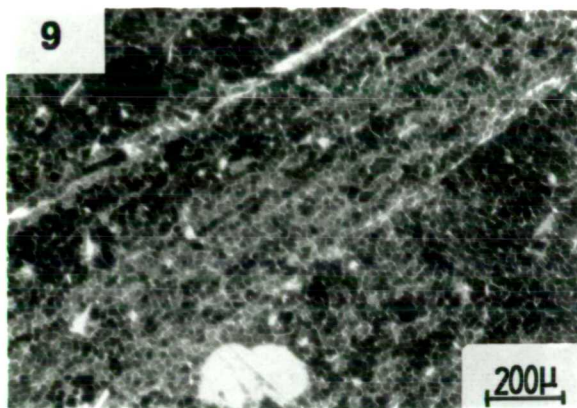
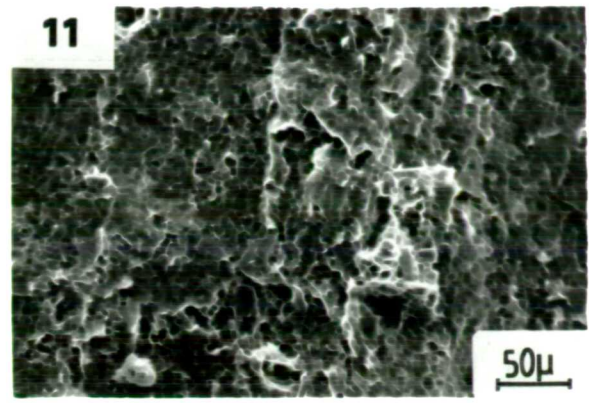
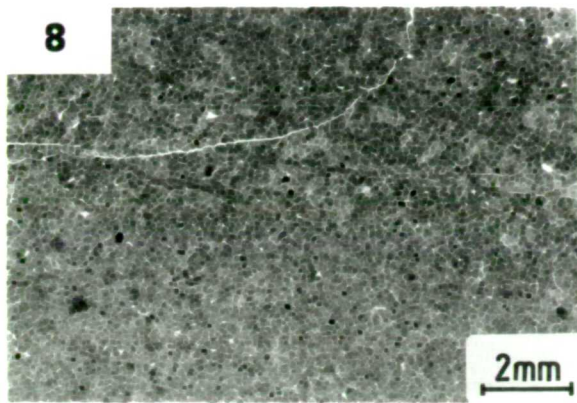
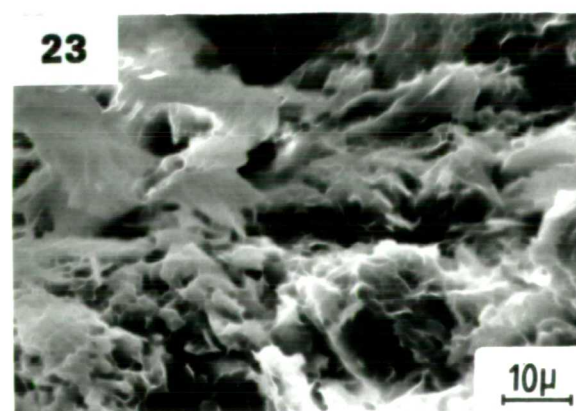
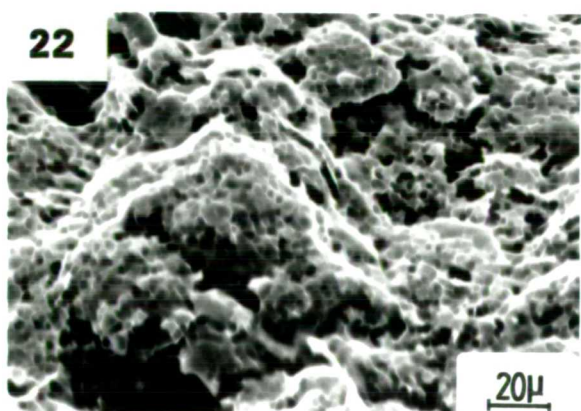
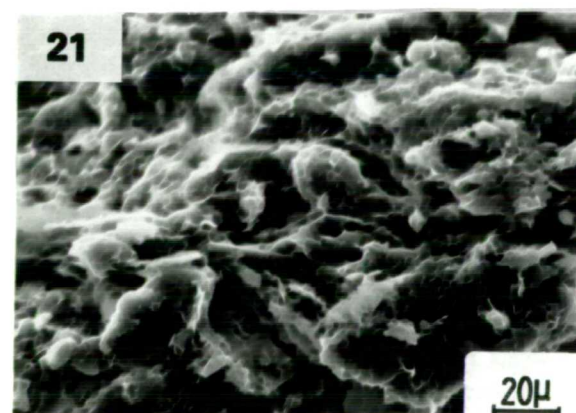
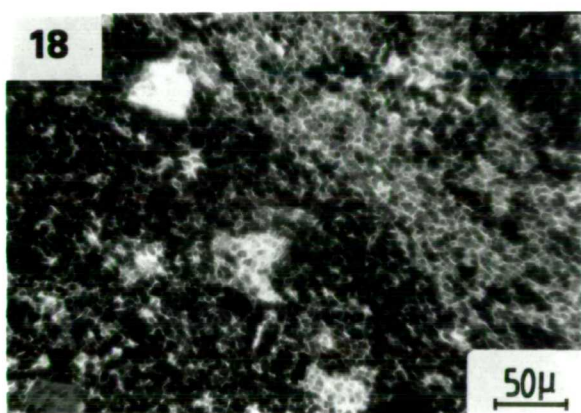
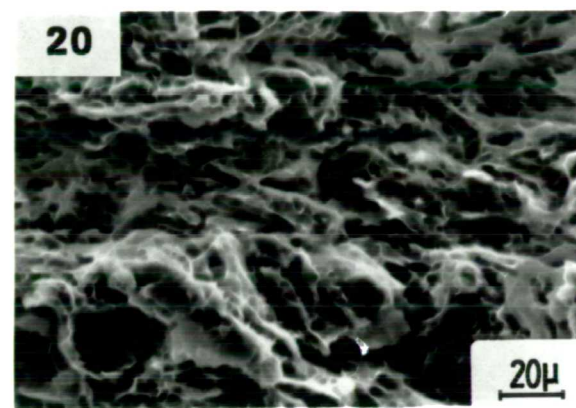
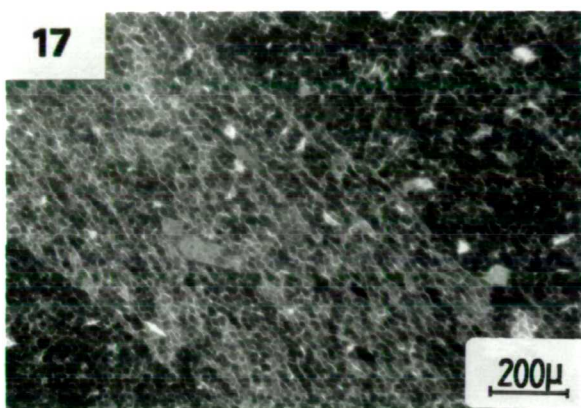
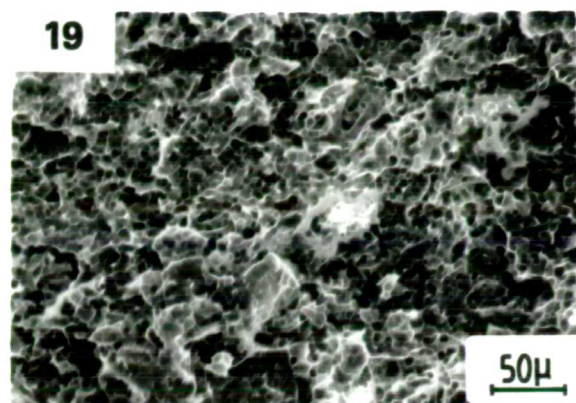
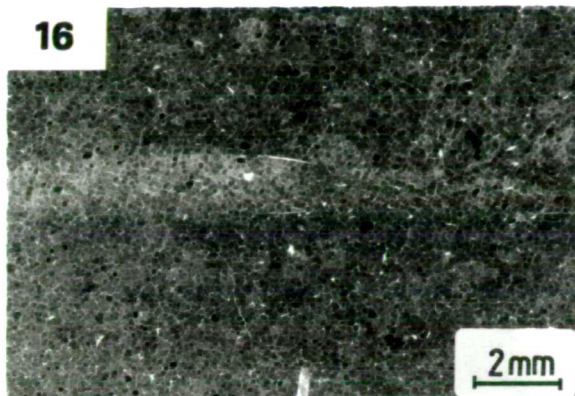


Plate 1





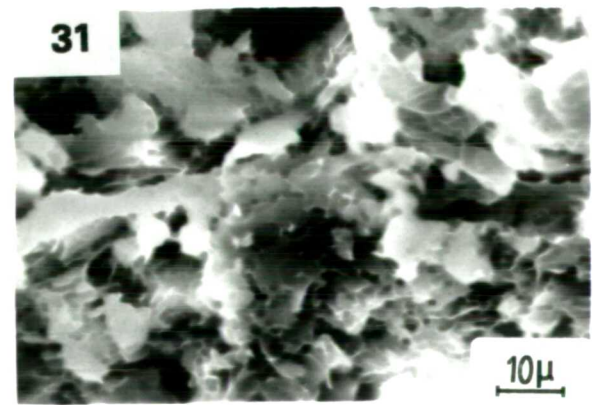
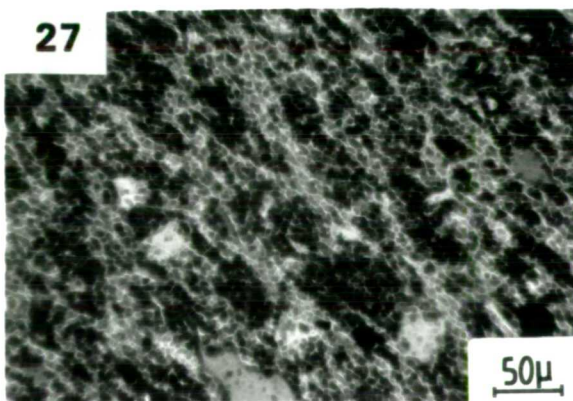
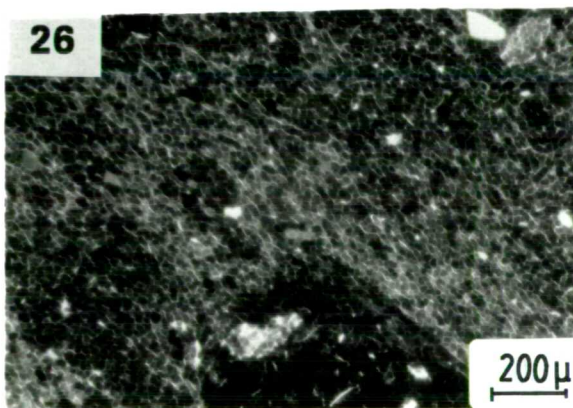
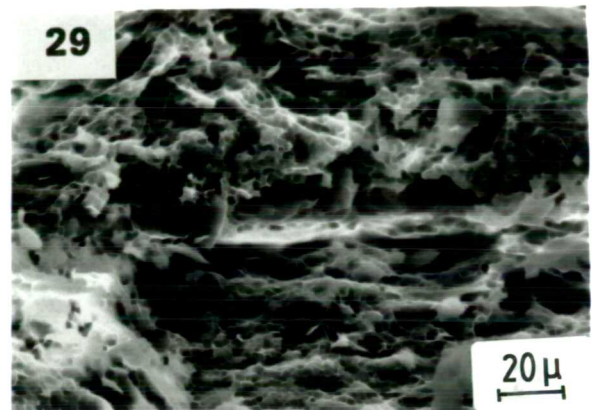
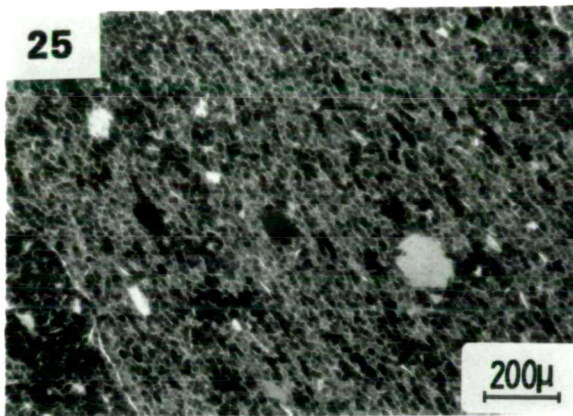
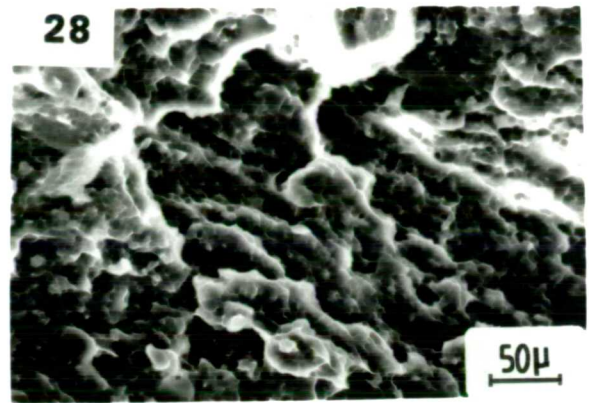
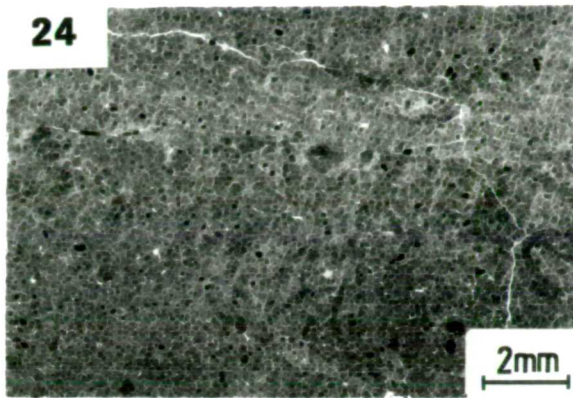


Plate 4

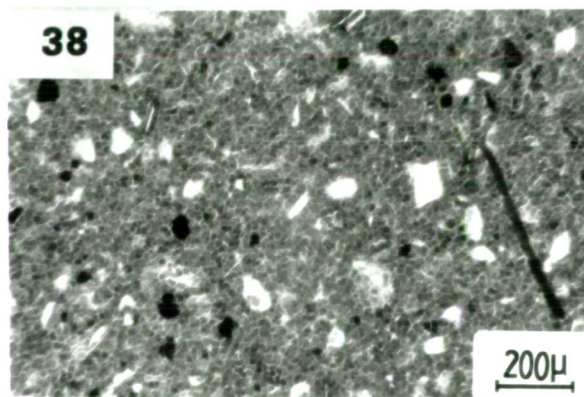
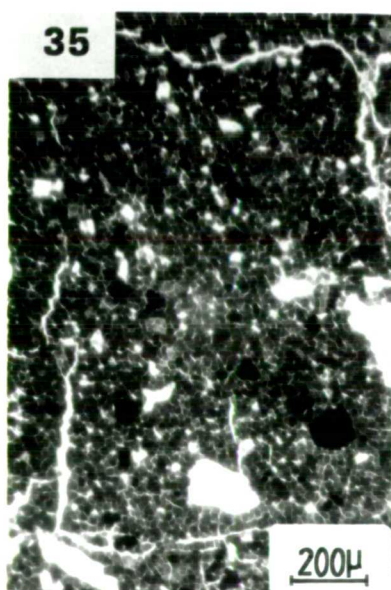
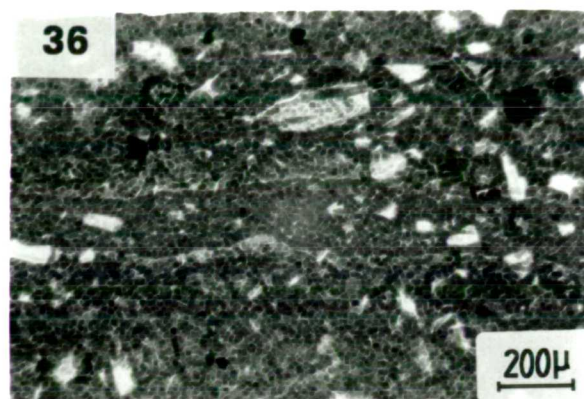
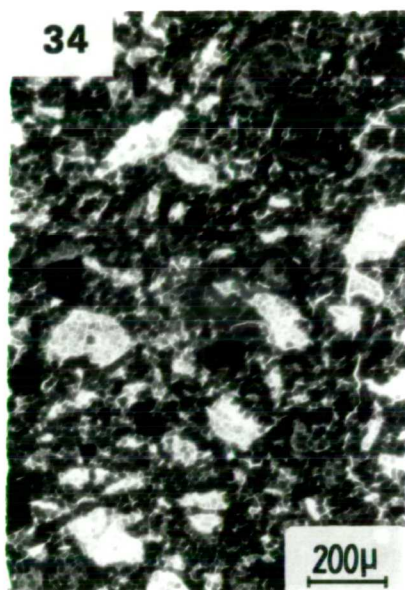
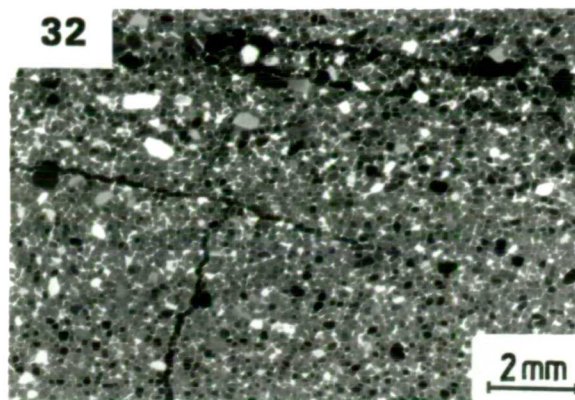
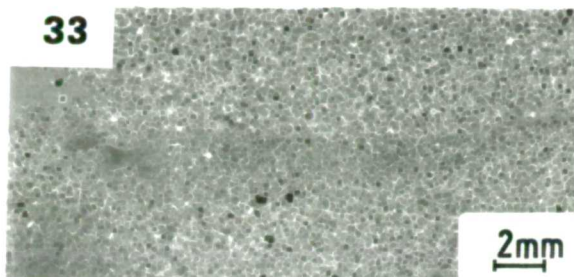


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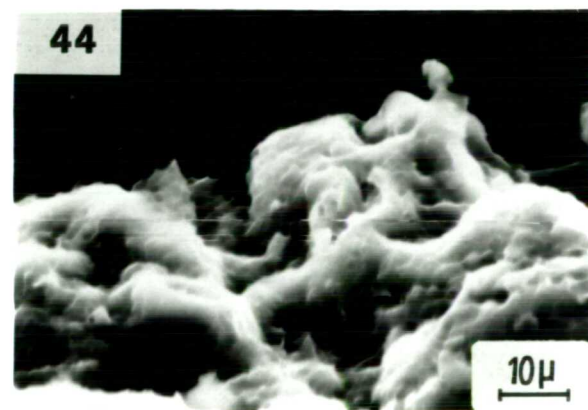
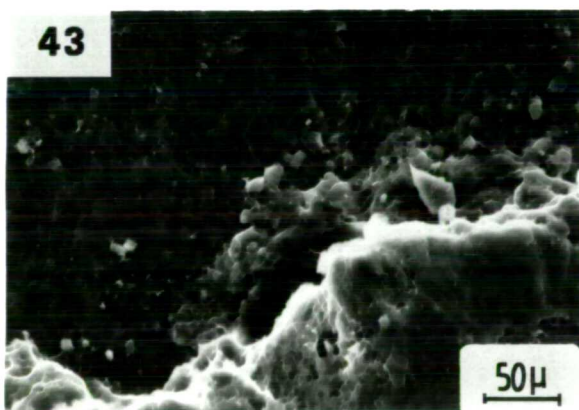
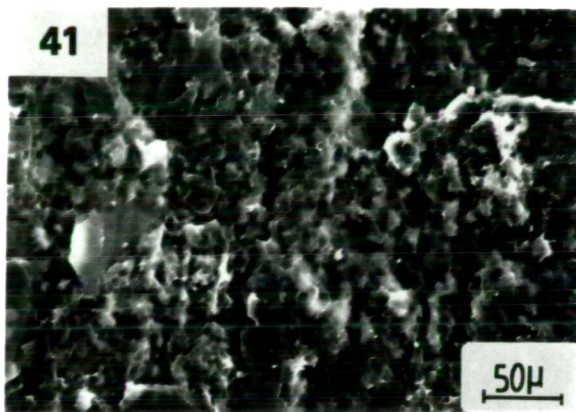
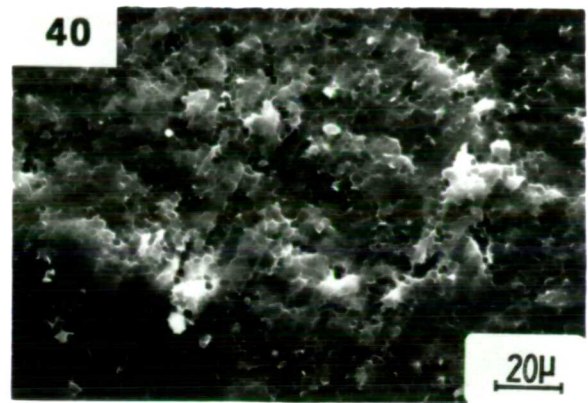
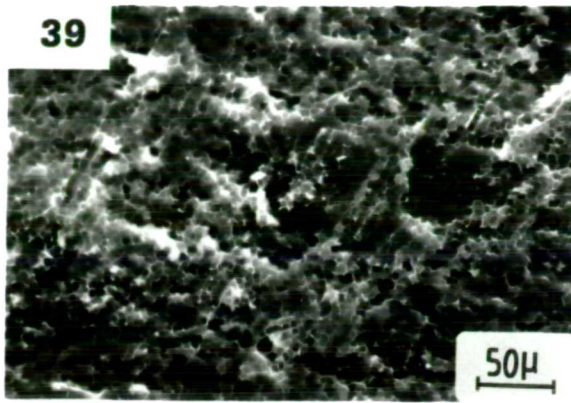


Plate 6