

STRESS STRAIN AND STRENGTH BEHAVIOUR OF
VERY SOFT SOIL SEDIMENT

by

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If the going is tough and the pressure is on;
if reserves of strength have been drained and the summit still not in sight
then the quality to see in a person is neither great strength nor quickness
of hand, but rather a resolute mind firmly set on its purpose
that refuses to let its body slacken or rest.

- Sir Edmund Hillary

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CHAPTER 1 INTRODUCTION

When fine grained natural sediments or artificially produced waste materials are transported and deposited through water, several different phases of behaviour are observed. These have been described variously as suspension, free settling, hindered settling, compression settling, intermediate and consolidating soil phases. Transitions between each are not always distinct in terms of material properties or behaviour and time spent in the early phases before a conventional soil state is attained can be a significant proportion of the total period of engineering interest. The eventual state attained following procession through these stages may be very different to that if the soil had been remoulded at the same final density. Standard engineering models exist which can describe soil behaviour well in separate phases under particular conditions, but these are of limited validity when extended to more general conditions and wider volumetric ranges than those for which they were formulated. The number of factors required to describe the entire range of behaviour is consequently larger than that for any one phase, and many of these factors are more familiar in fields of chemistry, geology or sedimentology than in classical soil mechanics.

This thesis discusses, in engineering terms, the engineering behaviour observed in a particular soil during the general sedimentation and self weight consolidation process. In the second chapter existing knowledge about behaviour at zero or low stresses is reviewed and evaluated with respect to common assumptions made, often implicitly, in formulating predictive models. It is shown that while these models have been extensively developed to a stage where they can approximate many aspects of soil behaviour, the lack of fundamental investigations carried out in parallel with their development has often led to inadequate appreciation of the causes of discrepancies between modelled and real behaviour. This has occurred particularly where standard

geotechnical testing equipment and methods devised for stiff soils have been used to obtain global average relationships between engineering parameters. Even where modified tests have been developed, instrumentation has sometimes been inadequate and measurements too infrequent, so that data available have necessarily been analysed only in terms of constitutive forms assumed already.

In chapter three experimental techniques are proposed which, where possible will allow soil behaviour to be examined under the least restrictive conditions of one dimensional compression so that basic engineering concepts may be analysed. Chapter four describes the testing programme and presents direct results of experiments. Chapter five analyses compression behaviour and establishes some trends which can be observed for particular parameters and relationships, and which exist between experiments under different initial and boundary conditions. Similar analysis of strength behaviour is undertaken in chapter six, where results obtained using different testing methods are compared. In the final chapter the general relevance of these results and their implications for engineering problems are discussed. Some suggestions are made for future work.

Areas of application

Improved knowledge about cohesive waterborne sediments can result in considerable savings for related industries. In the United Kingdom the annual cost of maintenance dredging is £25m (I.C.E. Coastal Engineering Research Panel, 1985). In East Coast ports alone reduction of the distances travelled by each dredger would lead to a saving of £270,000 per annum, per kilometre reduction. Studies at Rotterdam Europort (Kirby, Parker, van Oostrum, 1979) show that although a channel dredged recently may quickly refill with sediment to a depth which echo-sounding techniques might indicate to be unnavigable, the strength may be so low as to allow passage of vessels virtually unimpeded. A density of 1.2 Mg/m^3 is now used by the Rijkswaterstaat to define the "Nautical Depth" of a channel, stated to be "a

density within the suspension above whose altitude vessels can safely sail." Dredging control using information from gamma ray densimeters has enabled production increases of up to 50% to be obtained in the Europort area. In the United States \$30m was spent in a 5 year period on a dredging research programme aimed at improving disposal methods (Haliburton, 1977). Considerable volumes of waste material are also produced by the mining industry. The phosphate industry in Florida produces 40 million tons by dry mass per annum at an initial 3% solids by mass which even after two years retains void ratios around 10, due to the high content of attapulgite, a clay mineral consisting of long fibrous particles with large specific surface. Disposal areas for these clays occupy over 50,000 acres and are surrounded by 300 miles of dams, posing significant environmental and safety problems (Bromwell, Oxford, 1977).

Failures of underwater slopes have been well documented. In muds deposited recently in the Mississippi Delta area very low shear strengths combine with apparently high excess pore pressures and presence of gas bubbles to cause instability for slope angles less than 1° . Recent research carried out at Oxford suggests that presence of gas may cause high excess pressures to be deduced where none exist. Duncan and Buchignani (1973) analysed a slope failure in San Francisco Bay which occurred during cutting of a slope from a normally consolidated clayey silt. The importance of accurate determination of in situ parameters for analysis was shown by the estimated saving of \$200,000 through using a slope of 7:8 rather than 1:1, decreasing the supposed safety factor from 1.26 to 1.17. Analysis of error sources showed that an error of only 4% in the soil density could reduce this safety factor by 10%. Similar problems due to changes in loading or boundary conditions occur where natural changes, such as increase in water current, cause erosion of a sediment layer which might, for example, be supporting an underwater cable or pipeline.

In all these areas in situ property determination in soils of low density provides major problems. Density is often the only quantity that can be measured both accurately and continuously and then only when a stable platform can be maintained. Recovery of high quality samples from these layers is virtually impossible, so that there is a strong need for correlations between density and other properties such as strength and compressibility.

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2.1 Basic concepts, definitions, relationships and classifications

Conventional geotechnical engineering employs several basic concepts to describe soil behaviour. Effective stress rather than total pressure changes are assumed to cause deformations. The fluids phase and solids phase are usually defined independently and the total volumetric state is described by an appropriate parameter linking the two separate components, with the solids phase composition described further by its particle size distribution. Two constitutive relationships, one defining structural compressibility and the other describing fluid flow or permeability, are also used universally and a simple relationship of each type is often sufficient. Other engineering properties such as standard classification test results, compression/time behaviour, strength behaviour and anisotropy of stress or strength are related theoretically or empirically to these concepts.

Although they can be used to describe soil behaviour well under many conditions, the basic concepts are not fundamental but are influenced by or should be considered together with other instantaneous factors including interparticular forces, microstructure fabric and boundary conditions and with sampling and testing methods and initial conditions which are often grouped as "stress history" effects.

In soft sediments the mass of dissolved salts in the water may not be negligible compared to the mass of solids and wherever ambiguity is possible, density parameters should be defined specifying whether the salt has been assumed in the fluids or solids phase. This is particularly important when calculating moisture or fluid content from drying/weighing, where corrections must be applied to obtain the fluid content as used in this thesis. Noorany (1984) notes that this is rarely done and gives conversions as well as corrections for saturation. Some symbols applied in this thesis are defined here for reference, assuming full saturation:

w = fluid content = (mass of water + salts)/(solids mass excluding salts)

ρ = density ρ_w = fluid density (inc salt) ρ_s = solids density (exc salt)

C_m = solids concentration (solids mass exc. salts/total volume)

S_m = solids content (solids mass exc. salts/total mass)

n = porosity (voids volume/total volume)

v = specific volume (total volume/solids volume)

Void ratio is used most widely in marine geotechnical literature:

$$e = v-1 = \frac{\rho_s}{\rho_w} w = \frac{n}{1-n} = \frac{\rho_s - \rho}{\rho - \rho_w} = \frac{\rho_s}{C_m} - 1 = \frac{\rho_s}{\rho_w} \left(\frac{1}{S_m} - 1 \right) \quad (2.1.1)$$

Comparative values of some of these parameters are shown in figure 2.1. The error in fluid content or void ratio due to ignoring the salt fraction for a drying test on marine soil is about 4% at $w = 20\%$ 10% at $w = 200\%$ ($e=5$) and 30% at $w = 1000\%$ ($e=30$). Even for a soil mixed in fresh tap water, typically of salinity around 4 ppt by mass, 3% error results at a void ratio of 20, a value typical of many natural or artificial suspended sediments during settling. Use of these density parameters to define the volumetric state implies homogeneity of the soil structure for different elements at the same density provided that the average stress and recent previous stresses are the same. This homogeneity principle becomes increasingly inapplicable with decreasing density, when a global macrostructure should be considered to consist of one or more levels of microstructure, with separate stress states existing in each.

In a dilute soil-water state, most common soils containing clay minerals exist as a flocculated suspension. The nature of particle agglomeration depends on the specific surface of the clay mineral and the comparative attractive (Coulomb or London-Van der Waals) and repulsive (double layer) forces and the size of agglomerates depends particularly on the shearing rate. The character of these forces and relevance to soil flocculation, settling and consolidation behaviour have been discussed by Michaels and Bolger (1962a), Stevenson (1972), Arulanandan and Smith (1973) and Yong (1984) and shown to depend on factors including particle concentration and arrangement, fluid salt concentration, pH, Eh, conductivity and dielectric constant, while the shearing

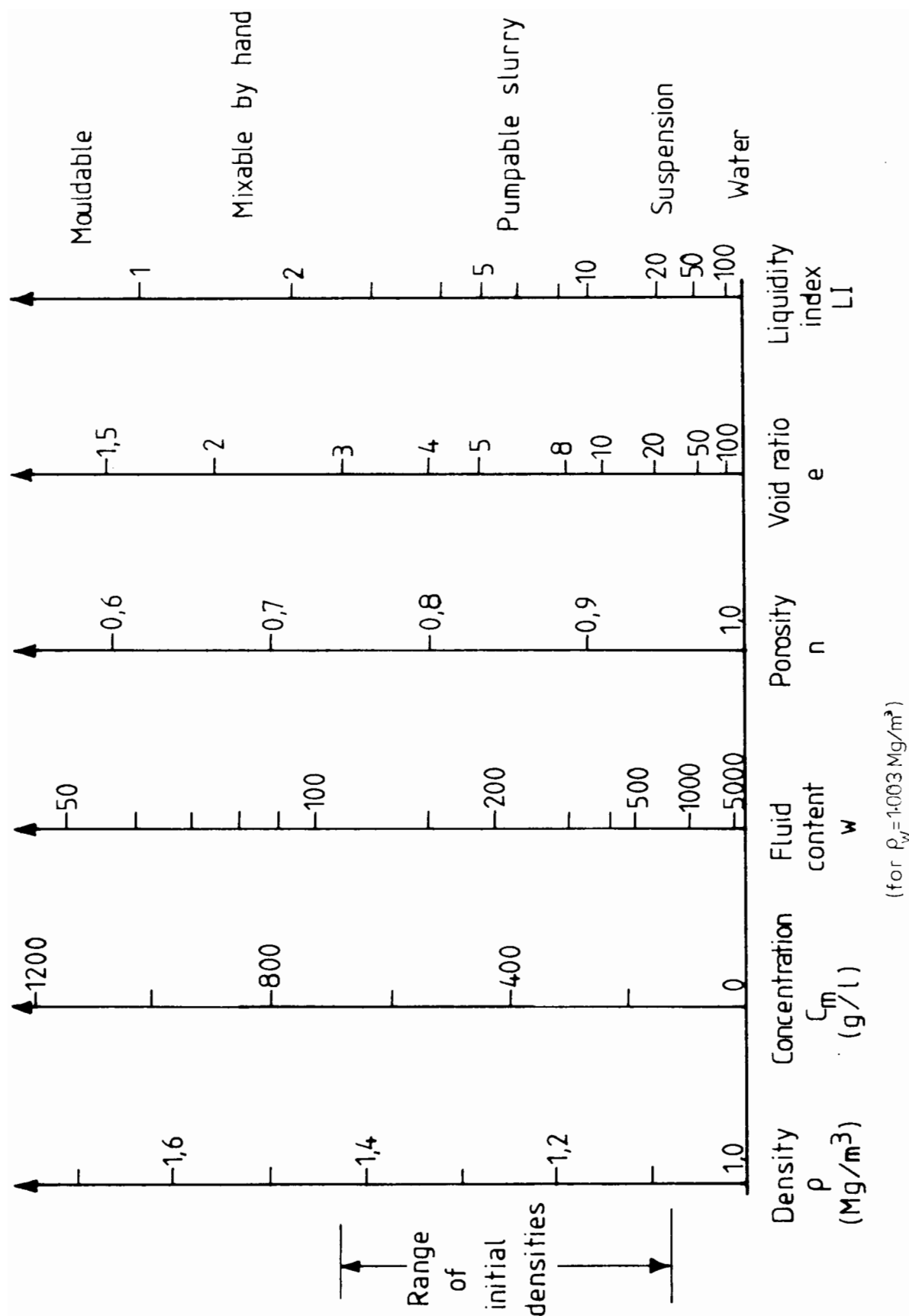


Figure 2.1 Comparison of soil density parameters, Combwich soil.

rate will depend on external mixing processes and on relative fluid-agglomerate velocities. A hierarchy of agglomerate units may exist simultaneously, with "domains" of clay particles arranged in "clusters" or "flocs" which are themselves arranged in "peds", with silt size particles serving as focal points and becoming encased in shells of clay particles. These groupings have been identified by Aylmore and Quirk (1960), Bowles et al (1969), Barden (1971 and 1972) and Smart and Tovey (1981) and terms used are those recommended by Yong and Warkentin (1975), although many different clay particle arrangements have been observed within or in addition to these groups. The implications are that full description of soil states requires stress and volumetric terms for each grouping separately. Global average parameters, if used alone, may be adequate to describe behaviour under certain conditions but in general relationships between these parameters will appear not to be unique and will certainly exhibit strong variations between different soils. Even a parameter such as the liquidity index, designed to reduce dependence of relationships on soil types, can be expected to achieve this only over a limited range of soil states.

The principle of effective stress, that the effective stress defined as the difference between total stress σ and pore fluid pressure u

$$\sigma = \sigma' + u \quad (2.1.2)$$

governs soil deformation is of prime importance in standard soil mechanics. Bishop (1959) stated in two hypotheses that volume change, deformation and strength of soils depended on the effective stress thus defined. The existence of effective stress greater than zero has been used to define the existence of a consolidating soil rather than a settling suspension (Monte and Krizek, 1976; Been, 1980; Been & Sills, 1981) while Sills and Elder (1984) preferred the term "structured phase" for this condition. Equation 2.1.2 states nothing about the nature of the forces contributing to σ' . Mitchell (1976) considered the vertical equilibrium of all forces acting on a curved surface passing through

the voids and points of interparticle contact only (the surface considered by Bishop) to show that

$$\sigma\alpha = (u_0+R)(\alpha-\alpha_c) - A(\alpha-\alpha_c) + C'\alpha_c - A'\alpha_c \quad (2.1.3)$$

where α is the average total interparticle area along a horizontal plane served by each interparticle contact (of effective average area α_c). Area α is equal to the total horizontal area divided by the number of interparticle contacts along the curved surface. $\sigma\alpha$ is then the vertical force on α due to applied total stress σ . Component u_0 is the pore pressure measured externally at the same elevation and is lower than the interparticle pore pressure u by an osmotic or solute pressure quantity R which is a measure of the double layer repulsion, so that $(u_0+R)(\alpha-\alpha_c)$ is the force carried by the hydrostatic component u . $A(\alpha-\alpha_c)$ is the long range attractive force (Van der Waal's and electrostatic) and $A'\alpha_c$ the short range attraction at the contact from chemical bonding and cementation. Component $C'\alpha_c$ is the short range repulsive force due to direct solid to solid contact and surface hydration. In soft sediments the actual average contact area α_c is likely to be much less than α . Bishop (1960) showed for lead shot where R , A and A' are negligible and equation 2.1.3 reduces to

$$\sigma = u_0(1 - \frac{\alpha_c}{\alpha}) + C' \frac{\alpha_c}{\alpha} \quad (2.1.4)$$

that even where α_c was of significant magnitude, volume change and shear strength behaviour were still governed by $\sigma' (= \sigma - u_0)$ so that the contact areas had no effect on observed behaviour. This may be an indication that on a microscopic scale the contact areas remained very small. Regardless, Mitchell assumed that $\alpha_c \ll \alpha$ and dividing equation 2.1.3 through by σ gives

$$\sigma \approx u_0 + R - A + (C' - A') \frac{\alpha_c}{\alpha} \quad (2.1.5)$$

where each term represents a force per unit area of cross section. The last term is the net force at the contact divided by the total horizontal area served by the contact, i.e. the "average intergranular pressure" and can be denoted σ'_i . The conventional effective stress, $\sigma - u_0$, is then

$$\sigma' = \sigma'_i + R - A \quad (2.1.6)$$

Some evidence exists that shear strength is controlled by σ'_1 alone but that volume changes may be governed additionally by R and A. If deformation of any kind should actually be governed by σ'_1 then assumed dependence on σ' will only be approximate unless either R and A are both negligible or $R = A$, in which cases the conventional effective stress principle will be valid.

Mesri and Olson (1970, 1971) tested montmorillonites and reported that for sodium montmorillonite, which has a very low friction angle ($\approx 4^\circ$), the effective stress failure envelope was unaffected by electrolyte concentration which controlled R, but the water content attained at any particular effective stress was affected. Virtually no direct interparticle contact occurs in this clay ($\sigma'_1 \approx 0$) and attractive forces are thought to be relatively small, so that $\sigma' \approx R$. This accounts for the small strength variations observed to occur with increase in effective stress. Sridharan and Rao (1973) also postulated a relationship of the form in equation (2.1.6) and termed σ'_1 the "modified effective stress" and $R-A$ the "intrinsic effective stress". They tested four different soils in one dimensional compression using nine pore fluids to vary the dielectric constant. This and later work (Sridharan and Rao, 1979) showed consistent trends of compressibility and shear strength with dielectric constant at constant void ratio or (conventional) effective stress. This agreed with theoretical predictions that an increase in dielectric constant would cause R to increase (and A to decrease) so that σ'_1 and therefore strength would decrease. It is worth noting that if σ'_1 controlled shearing resistance it would also govern the shearing component of one dimensional compression which would then not be solely σ' dependent. Other observations by Bolt (1956), Kenney, Moun and Berre (1967) and Kulkarni (1973) showed similar trends with other factors which caused interparticle physico-chemical forces to vary. It should be noted that the effect of these pore fluid changes may also have been to alter the fabric formed at initial deposition or mixing, so that without accompanying microstructural examinations some of these results may not be totally conclusive. Such effects are discussed later in the chapter. In

any particular soil-water system the physico-chemical forces will vary with density and with fabric type at constant density, but may also vary with relative fluid/solids velocity. Rosenquist (1961) observed that the viscosity of clay porewater increases with closeness to the mineral surface so that an increase in average velocity of the free pore fluid would decrease the thickness of the adsorbed water layers and reduce repulsion.

Index tests and Atterberg limits

Index tests were introduced originally to enable classification of different soil types and more recently the Atterberg limits have become widely used in correlations with particular engineering properties of soils. They are essentially strength tests. Of the two general types of liquid limit test, the Casagrande cup apparatus yields the most variable results, mainly due to operator related causes (Norman, 1958) but remains most widely used in the U.S.A. The cone penetrometer method was shown to be more reliable by Sherwood and Ryley (1970) who recommended use of the 30° 80 g cone required currently by British Standard 1377. Correlations with Atterberg limits are of two types; dependence of soil behavioural properties on the limits and variation in the limits with changes in the soil water system. Since comparisons of different cones are primarily strength test comparisons they are considered in a later section although their effects on the limits reported by different authors are relevant here. Test results are dependent on soil composition (Budhu, 1985; Dumbleton and West, 1966) but the main factors are physico-chemical in many clays and in recognition of this the activity was defined by Skempton (1953) as the plasticity index divided by the percentage of clay size particles. Sridharan and Rao (1975) continued their work varying clay type and fluid dielectric constant to show, as expected, similar trends to those for strength. Chassefiere and Monaco (1983) tested over 200 marine samples and showed decrease in liquid limit and plasticity index both with an increase in smectite, a clay mineral with a very high ion exchange capacity, and with the use of higher salinity water for remoulding. This is in contrast to results

obtained by Locat and Lefebvre (1985). Bjerrum (1954) showed that both parameters decreased when a normally consolidated marine clay was subjected to leaching by fresh water. This suggests that different processes were at work; leaching causes little change in fabric (Kazi and Moum, 1973) but large changes in interparticle electrical forces, whereas the fluid used for mixing a sample initially affects the fabric on a microstructural scale. Piakowski (1981) demonstrated a linear correlation between the cone index, a fall cone parameter similar to the liquid limit, and the specific surface of soils, and suggested redefinition of the liquidity index to yield more consistent results. A final source of variability has not been discussed elsewhere but might be deduced from cone test data obtained by Hansbo (1957) who measured times to full penetration for several cones which were never greater than 0.1 seconds. BS1377 recommends a 5 second penetration time, which might allow additional penetration to occur with either consolidation or undrained creep.

The main conclusion to be drawn from this section is that interparticular forces and microstructural fabric may influence soil behaviour in ways which will not be reflected by macrostructural parameters. Derived relationships may be difficult to obtain in unique forms if these parameters are used alone or without modification.

2.2 Very soft clays. General properties

The previous section considered ways in which conventional parameters might depend on microstructural effects. In this and the next section a general review is made of engineering behaviour of very soft clayey soils. Few data exist for states above the liquid limit as investigated in this thesis. To establish a framework into which experiments and results might be fitted a summary is presented first of known behaviour and models for stiffer soils of similar types. Data fall into two categories; those relating different soils and those obtained from detailed analyses of particular soils, which are sometimes used to provide models applicable to general classes of soil.

A complete soil model will consider consolidation, stiffness, strength and deformation as different aspects of the same behaviour due to different loading conditions. This is consistent with the microstructural dependence described already. The critical state models are particular examples developed largely on the basis of isotropic tests using kaolin, a clay soil of low specific surface, medium plasticity and low activity, although Lawrence (1980) investigated effects of variation in clay content, salinity and saturation and effects due to leaching, within the critical state framework. The general models are restricted to normally and lightly over-consolidated states and a particular assumption is that behaviour described in terms of normalised stresses is similar at different stress states. This leads to the postulated distinction between soft and stiff states as one of degree of overconsolidation alone, regardless of actual density (Parry and Wroth, 1981). Such a classification may not be relevant to soft sediments. Definition of the overconsolidation ratio (OCR) presents a major problem for many natural clays where the effective overburden stress is often not equal to the preconsolidation stress obtained from a reloading test despite lack of geological evidence for previous overburden. If ground water changes can be rejected as in most recent sediments this behaviour is usually attributed to secondary compression. Considerable debate exists as to whether soils states are similar at the same stresses and density regardless of whether stress unloading or secondary compression has taken place. Wroth (1975) predicts and observes an increase in K_0 , the ratio of the horizontal to vertical effective stresses at rest, during unloading, as observed by many other authors (e.g. Mayne & Kulhawy, 1982) and Parry and Wroth suggest that K_0 might remain constant during secondary compression. McRoberts (1984), on the basis of the macropore-micropore model for consolidation developed by de Josselin de Jong (1968), supports this view, quoting as indirect evidence the increase in pore pressures following cessation of drainage after laboratory consolidation noted by Holzer et al (1973). Nagaraj (1984) and Allam and Sridharan (1984) perhaps

confuse shear stress and strength when they argue that shear strength increases during secondary compression but since σ'_v remains constant σ'_h must decrease (for the Mohr's circle radius to increase). Soydemir (1984) uses viscoelastic analysis to suggest that K_o will increase during aging. Data from Hamamdshev (1981) support this. Kavazanjian and Mitchell (1984) provide some evidence that K_o will eventually approach 1.0 for all soils since the isotropic state represents that of minimum energy with no deviator shearing stress. Values of K_o measured in situ are often inaccurate but are commonly around 0.8 to 1.0 although values much lower have been measured, especially in quick clays (Kenney, 1967; Berre and Bjerrum, 1973).

Both the critical state models and the SHANSEP procedure for testing and design (Ladd and Foott, 1974) are based on independence of normalised stress strain behaviour on stress level and allow relationships to be established among stiffness, undrained strength, stress and OCR (e.g. Wroth et al, 1979) which are constant for a particular soil over wide stress ranges but quite limited volumetric ranges, generally between the liquid and plastic limits. Departures from these models occur due to structural and stress anisotropy but more significantly at extremes of the data ranges on which the models were based and most importantly due to differences between clays in natural and remoulded states. The relatively low activity of kaolin (0.3 to 0.4) limits further the applicability of kaolin based models to the problem soils discussed in chapter 1.

Large property variations occur in very soft clay soils deposited by sedimentation. Submarine accumulation rates have been estimated to vary between 1 cm/1000 years and 2 m/year (Monney, 1971). Grim (1962) suggested that slow sedimentation rates allow cementation to occur between particles before deformation occurs due to increases in effective stress, resulting in high void ratio deposits in which density is nearly constant with depth. This theory explains the presence of metastable deposits where the void ratio remains high until the effective stress becomes sufficiently great to overcome cementation.

Collapse then occurs and high pore pressures are generated, as observed by Denissov (1965), Chang et al (1973), Hirst and Richards (1976) and Crawford and Burn (1976), all of whom reported pore pressures nearly equal to the vertical total stress after periods up to 22 years. Crooks et al (1984) reviewed fifty field cases where anomalous pore pressure behaviour occurred, often accompanied by lack of strength increase with compression. Effective stresses remain low when this behaviour occurs, causing a creep effect to be observed as in secondary compression. Similar behaviour was observed by Leonards and Ramiah (1960) who introduced the term "quasi-preconsolidation", Leonards and Altshaeffl (1964) who recorded apparent overconsolidation ratios up to 1.4 for clays sedimented artificially at rates as low as 0.6 mm solids per day, Bumrungrasup and Moh (1972) and by Crawford (1965) and Bjerrum (1967) in natural clays. The opposite effect or reduction of this effect occurs when the cementing agents are leached from soil by replacement of pore fluid and has been demonstrated by Kenney (1967). Moore et al (1977) observed that a recent marine sediment which showed overconsolidation characteristics behaved as though normally consolidated when leached with NaOH solution. One net result in any particular case of sedimentation and consolidation is a dependence of compression behaviour on rate of deformation or stress increase (Crawford, 1964). Leroueil et al (1983) observed a 15% increase in the measured preconsolidation pressure for intact natural clays with a tenfold increase in strain rate. Hawley (1977) claimed that use of rate parameters is preferable to use of time when modelling compression and suggested that a combination of rate effects and time hardening might lead to compression which is greater for a given stress at lower loading rates but less at very low rates.

Changes in compression behaviour are usually accompanied by changes in strength behaviour, where metastability or aging may result in high sensitivities or ratios of initial undisturbed strength to remoulded strength. Many authors have observed these effects together (e.g. Shen et al, 1973; Krizek and Salem, 1977; Costa Filho et al, 1977, and Silva et al, 1984) since

they were demonstrated first by Bjerrum et al (1958) for natural clays, although Bjerrum and Lo (1963) and others have also reported strength increase with aging in soils which did not exhibit secondary compression. Bjerrum and Wu (1960) and Karlsson and Pusch (1967) reported increased strength with aging which was removed by reconsolidation. Bjerrum (1954) first showed that leaching of a marine clay reduces the undisturbed strength slightly but the remoulded strength markedly, causing a large increase in sensitivity. The absence of similar sensitivity values in clays sedimented artificially at different salinities caused Bjerrum and Rosenqvist (1956) to conclude that leaching was mainly responsible for producing the quick clays of very high sensitivity found in Scandinavia and Canada. Mitchell and Houston (1969) identified eight different mechanisms responsible for sensitivity development of which most are time dependent. Allam and Sridharan (1979) aged two isotropically consolidated clay soils of very different activity (2.5 and 0.37), the first showing high and the second negligible secondary compression, for periods up to 30 days, and found that the angles of shearing resistance for peak strength increased while the extrapolated cohesion intercepts, although small, tended to decrease with time. Thixotropy is a term commonly used to describe hardening following remoulding although it was originally applied only to reversible softening and settling in colloidal suspensions, and as such is not strictly applicable to age strengthening in soils. The definition given and discussed by Mitchell (1960) however has become widely accepted and implies a difference between the two only of degree, although it should be used solely where physical remoulding (rather than, for example, sedimentation) occurs, and volumes remain constant. Often, however, in particular studies where leaching was not involved, the aging process has been neglected and it has been implied simply that sensitivity or peak strength are unique soil properties depending only on stress or density once some equilibrium state is attained, e.g. Skempton and Northey (1953), Kulkarni (1973) and Howat (1985). Houston and Mitchell (1969) obtained relationships among sensitivity, liquidity index and

effective stress although they recognised the influence of other factors such as aging. Locat and Lefebvre (1985) suggested that a relationship exists between the sensitivity and plasticity of unleached soils and that secondary compression inhibits rather than increases sensitivity. On the basis of this work Leroueil et al (1985) stated that four distinct structural states may be encountered in clay soils; the intact or natural state, the destructured state observed when an intact soil undergoes large volumetric or shear deformation, the remoulded state of minimum strength and the resedimented state obtained by remoulding in a slurry, followed by deposition and self weight consolidation. Skempton (1984) distinguished between critical state strength and the lower residual strength which occurs on shear surfaces at very large displacements (Skempton and Bishop, 1950). Pyles (1984) observed a constant ratio of residual to peak strength for a soil at states between the plastic and liquid limits. Yong and Tang (1983) discuss remoulding processes and indicate that although the sensitivity of some quick clays is extremely high, the energy required for total remoulding is sufficiently high that the susceptibility to strength loss is lower than for other less sensitive soils. This is supported by Eden and Kubota (1961), who measured different sensitivities using different methods.

Rate effects observed for consolidation may have an even stronger influence over strength behaviour and are greater for higher clay content or plasticity (Borgesson, 1981). In many strength tests however strains are not known accurately and in addition drainage may lead to observations of apparent rate effects due to non uniform pore pressures or localised consolidation (Wilson, 1963; Aas, 1965). Cheng (1980) observed an increase in undrained strength with strain rate up to an ultimate value only. Parry and Wroth (1981) concluded that for triaxial (CU) tests a low rate of loading may not produce a distinct stress-strain peak but a tenfold increase in testing rate will usually cause a strength increase of 5% for remoulded clays and up to 15% for undisturbed clays, while many authors have reported more significant

variability with rate in vane testing, which is discussed later. As an opposite extreme, the strain rate reduction to zero at cessation of testing is followed by slow relaxation of stresses, partly due to consolidation but mainly due to undrained creep (Lacerda and Houston, 1973). Undrained creep at constant shear stress has also been described by Bjerrum and Lo (1963), Hyde and Brown (1976) and Sherif et al (1980) and various theories have been developed for prediction of creep magnitude, e.g. Komamura and Huang (1974), Ter-Stepanian (1975) and Kavazanjian and Mitchell (1980) who produced a general time dependent deformation model for clays containing both volumetric and deviatoric components.

Singh and Mitchell (1969) indicate that time dependent soil properties described above can be summarised by placing soils in three categories, those that lose strength with time, those that gain strength with time and those whose strength is independent of time under particular conditions.

2.3 Settling and sedimentation

Natural sedimentation and artificial deposition are often irregular processes due to variations in soil characteristics, supply rate, containment boundaries and external stress conditions such as near bed flow rates. Consequently, although field studies are necessary to validate predictive models, specific aspects of behaviour are difficult to evaluate and most non theoretical studies are simulations in which the conditions above may be controlled. These have often been in areas outside geotechnical engineering, but many of the conclusions are relevant. Investigations using clay minerals or soils are considered primarily here and may broadly be divided into two types, those of settling processes in which settlements and velocities are considered usually as functions of concentration and those in which stresses are considered by measurement of pore pressures together with density profiles. The latter experiments have usually been undertaken in tall settling columns.

2.3.1 Settling studies

Most early investigations attempted to relate the settling velocity in a suspension to the concentration, using the Stokes velocity as a constant for the particular soil/water system. Michaels and Bolger (1962a) using kaolin and Peirce and Williams (1966) using four natural muds, including one from the Severn estuary similar to that studied in this thesis, obtained good fits to data using this type of relationship of the specific form suggested by Richardson and Zaki (1954) for the settling velocity, v

$$v = v_{st} n^r \quad (2.3.1)$$

where v_{st} is the Stokes velocity for a dilute suspension, n is the porosity and r an empirical coefficient varying with particle shape but generally between 4 and 6. McLaughlin (1959) and Owen (1970) both found settling velocities variable with the suspension depth, and Owen showed that the velocity increased with concentration up to a salinity dependent critical value and then decreased. Fitch (1966) recognised a disruptive mechanism whereby rapid pore water drainage could occur to the surface up fine vertical channels within the suspension, carrying some soil particles at low concentrations and increasing the net downward solids flux. Kynch (1951) developed a theory describing hindered settling at high concentrations using continuity and the assumption that fall velocity depends only on the local solids concentration. Particles are assumed not to be in contact, so that no interparticle forces occur, and to be identical. The discussion of section 1.2 suggests that the second assumption will be restrictive only where flocculation is insufficient to prevent segregation. McRoberts and Nixon (1976) studied hindered settling for various soils but mixed them in distilled water so that flocculation was probably minimal and nevertheless assumed segregation would not occur. Tiller (1981) modified Kynch's theory, recognising that the characteristics of constant concentration originate not at the base of the soil but at the surface of the rising sedimented layer, where the upward fluid velocity is non zero. "Settling in compression" was described by Tory and Shannon (1965) where settling rates are reduced by particle contacts but allowed for only by

empirical modifications to Kynch's theory and none of the other authors above considered consolidation effects or pore pressures.

2.3.2 Settled bed and settling column studies

Note: In this section density values reported by different authors are all converted to solids concentrations for ease of comparison.

Gaudin and Fuerstenau (1958) reported the first studies of the entire sedimentation process from suspension settling to bed consolidation. They used an X-ray absorption technique to obtain density profiles at different times during deposition from a 1 m deep kaolin suspension and observed steps in density from the initial suspension to the rising bed below and then to a denser region near the column base. The upper two regions merged after about two hours but the lower transition became more gradual as time proceeded. These density discontinuities were also observed in later studies by Krone (1962), Migniot (1968), Imai (1980,1981), Been (1980) and Been & Sills (1981). Krone found the concentration to which the upper transition occurred varied with initial concentrations which ranged up to 40 g/l ($e = 65$). Migniot tested a large number of muds at initial concentrations around 50 g/l and gave a typical example in which maximum concentrations of 220, 320 and 330 g/l ($e = 11, 7.3, 7.1$) were attained in the three distinct phases up to 10 hours, 20 days and 10 years respectively. Owen (1970b) used initial suspension depths from 2 to 10 m at concentrations from 1 to 16 g/l and salinities from 0.3 to 32 g/l, for a clay of fairly high plasticity but medium activity. He took measurements during the post suspension stages only and observed that for suspensions with different initial concentration and height if the bed heights were the same at any time then density profiles' shapes were almost identical. He noted that the surface concentration increased slowly with time but was very dependent on bed thickness. Keshian et al (1977) sedimented phosphatic clay at concentrations from 10 to 60 g/l in pipes up to 11 m high under variable boundary drainage conditions. They measured only settlements and final densities, for comparison with a model designed to predict behaviour in a waste

containment area, as did Jung and Ryu (1979) who used dredged harbour material in 0.9 m columns.

None of the above studies employed pore pressure measurement and authors assumed that density states attained once settlement rates became very small could be used to construct consolidation curves. Variable compression behaviour within tests could therefore not be detected. Lacasse et al (1977) measured final densities and pore pressure in columns up to 2 m high with initial concentrations of 170 g/l, for settling periods up to 300 hours. Constant rate of deformation consolidation tests were then carried out and showed apparent overconsolidation, although the lack of earlier consolidation data prevented this from being identified as such. Kusuda et al (1980) measured pore pressures in 4 m columns but again could measure densities only at the end of each test. They used weighted markers to enable solids velocity measurement but differential movement of the soil particles relative to these might have contributed to the non Darcian flow behaviour observed for a waste sludge

Been (1980) measured pore pressures and densities using X-ray attenuation in 2 m columns and observed three zones almost identical to those found by Gaudin and Fuerstenau. He postulated that at concentrations above a critical value C_1 (around 200 g/l for the soil used) a suspension could never exist since pore pressures would be lower than vertical total pressures due to interparticle contact. Between this and a second, higher critical value C_2 an intermediate region existed in which some channelling occurred and compression ($e-\sigma'$) and permeability ($e-k$) behaviour were highly variable with particular test conditions. The upper concentration C_2 would eventually be attained at the surface of a soil consolidating under self weight alone but until this value was reached at the surface soil below, although at concentrations above C_2 , would not exhibit unique compression behaviour. Both C_1 and C_2 would depend on conditions at deposition although Been demonstrated apparent dependence of each value on the initial concentration C_i . Results showing segregation at concentrations below C_i cast doubt on the applicability to field behaviour of

any studies in which initial concentrations are below this value for the particular soil. Lin and Lohnes (1984), depositing a dredged clay very similar to that used by Been in 1.8 m columns with density measurement on extracted samples but no pore pressure measurement, found a similar critical concentration for their soil at 150 g/l. Sills and Thomas (1983) compared tests at depositional rates from 4 to 400 mm solids per day with those carried out by Been with instantaneous deposition of the same soil and found that at the same effective stresses much lower concentrations could exist following gradual deposition. These concentrations could be more than three times lower than the critical value suggested by Been and remain at these levels for long periods during consolidation generally without any significant particle size segregation occurring. Imai also observed segregation, increasing with suspension depth in columns up to 2 m high, and restricted detailed studies to 250 mm columns in which he observed suspension densities decreasing with time. He attributed this to the upward seepage force being increased by expulsion of water from the consolidating bed below, but since initial concentrations were as low as 45 g/l it is possible that segregation remained the cause. He also observed increasing concentration with time at the surface of the consolidating bed and for two highly plastic soils observed that the compression curves representing the consolidated state were not unique but depended on the initial concentration. A unique curve was obtained for kaolin. Although not recognised by Imai his compression data suggest strongly the existence of an upper bound unique compression curve for the two plastic soils to which other curves tended at higher stresses, although the non-unique behaviour might again have been partly due to segregation. Einsele et al (1974) observed that soil which had been sedimented at a slow rate was less compressible on loading than the same soil consolidated from a slurry. They also noted that compressibility decreased as salinity increased although curves for fresh and salt water soils merged at an effective stress around 0.5 kN/m^2 . This result was also observed by Migniot, but Owen, and Leonards and Altschaeffl (1964) found

compressibilities unchanged in salt and fresh water. Bishop and Vaughan (1972) concluded that a process which increased flocculation (such as salinity increase) caused significant changes only during the sedimentation phase.

2.3.3 Erodibility and strength tests

In conjunction with settling column tests several authors considered strength characteristics of the settled bed. Krone (1963) used a rotating cylinder viscometer and found power law dependence of shear strength on concentration and strength increase with shearing rate and salinity up to a value sufficient to obtain full flocculation. Similar results were found by Michaels and Bolger (1962b) and Migniot, who measured sensitivities as high as 10 after strong agitation, although only 15 minutes were required for full undisturbed strength regain. Owen found that the surface shear strength increased for smaller bed thicknesses, but this was probably only due to the increased consolidation rate for small sediment volumes. Thorn and Parsons (1980) and Hawley (1981) extended Owen's studies using flumes to deposit then re-erode sediment beds with current velocity increasing to provide increased surface shear stress on the beds. The most complete study is that by Einsele et al (1974), who used gradual deposition rates from 0.2 to 5 mm solids per day for 50 to 175 days to provide sediment beds of kaolin, bentonite and an illitic clay of similar plasticity to that used by Been. Bearing capacity was determined by applying surface loads and strength investigated generally using the shear vane, by tilting the tank, and by providing surface drag in a flume. Pore pressures were measured so that effective stresses were known. Undisturbed vane strengths increased at a decreasing rate with effective stress up to 3 kN/m^2 , but a constant ratio of remoulded undrained strength to vertical effective stress was found, equal to 0.03. Greater strength anisotropy resulted in a soil consolidated from a dense slurry than in soil consolidated following slow sedimentation. Comparison of shear strength obtained with the vane and the critical stress for erosion in the flume was of particular interest. If the soil was mixed as a slurry the two values were comparable,

whereas if the soil had been sedimented slowly to give the same critical erosive stress, the vane strength was around ten times higher. This could be largely due to the differences in strength anisotropy between the two soils but also indicates that very different mechanisms were involved in the two types of test.

2.4 Modelling and prediction of self weight consolidation

Consideration of results reported in the previous sections indicates that accurate prediction of the consolidation behaviour of a soil following sedimentation from a dilute initial state will be very much more difficult than for a soil subject to increased loading from a preloaded or dense state. In this section many of the assumptions made commonly are reviewed. The consolidation theories derived using these assumptions are described only briefly to place the various hypotheses and simplifications into a context in which the assumptions can be investigated experimentally.

The most general formulation for consolidation will consider continuity of volume to obtain an expression relating the relative velocity of the soil fabric phase (including bound water) to the free pore water, for an arbitrary element of constant cross sectional area. Choice of a appropriate (Lagrangian) vertical coordinate system will simplify the formulation by defining each element as containing a constant volume of solids and bound water, so that the vertical coordinate, z , may be related to the space coordinate, x , by

$$\frac{dz}{dx} = \frac{1}{1+e_m} \quad (2.4.1)$$

where the macrostructure void ratio, e_m , is defined as the volume of free pore water divided by the volume of solids plus bound water. Fabric phase continuity is then automatic and continuity of the macropore free water phase then gives

$$\frac{\partial e_m}{\partial t} = - \frac{\partial [n_m v_r]}{\partial z} \quad (2.4.2)$$

where n_m is the macrostructure porosity and v_r the relative velocity defined above. A constitutive relationship is required to describe the relative velocity as a function of other physical quantities such as the excess pore pressure, u_e , the void ratio, e_m , or the pore pressure gradient, $\partial u_e / \partial z$. Neglecting inertial effects, which are negligible even at very high relative velocities, the vertical forces including self weight are equated to give an expression for the excess pore pressure in terms of solid and fluid unit weights γ_s , γ_f , void ratio, e_m , effective stress, σ' , and intrinsic stress, σ'' , as described in section 2. This can be used together with the relative velocity relationship to replace $n_m v_r$ in equation 2.4.2 and gives a general equation governing one dimensional consolidation, of the form

$$\frac{\partial e_m}{\partial t} = - \frac{\partial}{\partial z} [f(e_m, \gamma_f, \gamma_s, \sigma', \sigma'', \frac{\partial \sigma'}{\partial z} \dots)] \quad (2.4.3)$$

where the only assumption made is that average flow is in one dimension only.

2.4.1 Simplifying assumptions and approximations

To yield forms of equation (2.4.3) which are soluble analytically or by numerical methods in particular situations a number of simplifications must be made. Possibilities are listed here in approximate ascending order with respect to their restriction on soil behaviour modelling capability or generality, although this order is highly subjective and depends on each situation to be modelled. Some are interrelated.

1. Soil skeleton is spatially homogeneous regarding material behaviour
2. Solids and pore fluid are fully saturated (and so incompressible)
3. $e_m = e$, the conventional void ratio (no bound water layers/micropores)
4. $\sigma = \sigma' + u$, intrinsic or electro-chemical forces are negligible
5. Secondary compression or skeleton creep effects are negligible
6. Behaviour independent of strain/loading rates or intrinsic time effects
7. Cementation, decomposition, metastability type effects do not occur
8. Darcy's law for pore fluid flow is valid, with permeability k
9. Monotonic compression and permeability relationships exist, $\sigma'(e)$ and $k(e)$
10. Some compound groupings containing $\frac{de}{d\sigma}$, k and e terms are constant

11. Self weight of the soil is negligible compared to applied loading

12. Permeability, k } are both constant throughout

13. Compressibility, $\frac{\partial e}{\partial \sigma}$, } any load or time increment.

None of the assumptions above is trivial although many have been assumed implicitly in existing consolidation theories.

Assumption 1 is invalid if previous or initial conditions cause variations in behaviour, for example where different fabric structures exist at the same void ratio or where different void ratios exist at the same effective stress. The second assumption is usually satisfactory in low stress ranges for submerged soils where gas generation is absent. Points 3 and 5 are interrelated. Presence of adsorbed water layers around clay particles or within aggregates will reduce the pore volume available for flow, increase the equilibrium void ratio and cause interparticle forces which give rise to deviations from conventional effective stress dependent behaviour. These effects cannot be quantified easily and for effective stress values much greater than zero are probably very small in most common soils. The time and chemistry dependent effects involved in 5,6 and 7 have been discussed earlier where influence of depositional conditions was also suggested. The existence of non Darcian behaviour is discussed below and simplification 9 requires most of the previous assumptions to be valid. Points 10, 12 and 13 are approximations which simplify the equations for ease of solution but the restriction to infinitesimal strains implied by 12 and 13 and the neglect of self weight (11) eliminate these last three possibilities for soft sediment models, unless incremental behaviour is considered.

2.4.2 Pore fluid flow relationship

In sediments in which the density is near to the suspension/bed transition density, relative flow behaviour between solid and liquid phases might be expected to deviate both from the hindered settling approximation that velocity is a function of concentration and from the stiff soil assumption that relative flow is proportional to hydraulic gradient, i (Darcy's Law). Observations by

Kusuda discussed in section 4 supported this hypothesis. Kos (1985) suggested that at such densities the increased shear on the solids from the flowing pore water would cause coarser structure with greater flow resistance. Hansbo (1960) concluded that at hydraulic gradients much less than 10 Darcy's Law might be modified to the form for the flow velocity, v

$$v = k i^n, \quad n \approx 1.5 \quad (2.4.4)$$

Hansbo (1960), Miller and Low (1963), Gairon and Swartzendruber (1975) and Olsen et al (1985) observed non zero hydraulic gradient intercepts at zero flow or measurable flows at zero gradients. These observations would support the existence of separate macropore/micropore flow postulated by Olsen (1962) and discussed already in section two. Olsen (1969) varied the magnitude of the hydraulic gradient intercept by superimposing electrical and chemical gradients on a kaolinite specimen and Mitchell (1976) described relationships for flow through soil including these and thermal gradient effects. Incomplete saturation could also cause these effects which might therefore be common in many natural soils. Mitchell and Younger (1967) pointed out that while field hydraulic gradients were seldom much greater than unity, in many laboratory tests they exceed 100 and low gradient deviations from Darcy's law are not detected. Pane et al (1983) concurred with this view, showing how measurement and interpretation errors and seepage effects could result in evaluation of nonexistent deviations and strongly recommending that very small gradients should be maintained in testing to yield valid results. Tavenas et al (1983a) showed that rapid pressure gradient changes could cause non Darcian behaviour to be observed. Dunn and Mitchell (1984) showed variations in permeability covering several orders of magnitude between different tests, supporting the view of Olsen and Daniel (1981) that permeability tests were not well defined or consistent. Olsen et al (1985) showed the definite advantages of the flow pump method (introduced by Olsen and preferred by Pane et al) over conventional constant or falling head techniques.

Most published work has assumed the validity of Darcy's Law, which can be written

$$nv_r = - \frac{k}{\gamma_w(1+e)} \frac{\partial u_e}{\partial z} = -ki \quad (2.4.5)$$

where n is the porosity, γ_w the unit weight of the pore fluid and k the hydraulic conductivity or permeability, assumed independent of the hydraulic gradient. This relationship has not been verified for soils in states wetter than the liquid limit, due to the difficulties in testing described above and most results reported simply consider the assumed dependence of k on the void ratio e , as deduced from particular tests. The effect of such a simplifying assumption is that at constant void ratio high variability may be observed (e.g. Been, 1980). The Kozeny-Carmen equation was derived for permeability of coarse grained soils:

$$k = \frac{\gamma_w}{\mu_w S_s^2} \cdot \frac{1}{koT^2} \cdot \frac{e^3}{1+e} \quad (2.4.6)$$

where μ_w is the pore fluid viscosity, S_s the specific surface of soil particles, ko is pore shape factor and T the flow path tortuosity. The first group is constant for a particular soil/water system and the second varies with void ratio and soil fabric. Although this relationship was derived for sands, Monte and Krizek (1976) assumed its validity for a kaolinite to evaluate $1/(koT^2)$ from permeabilities at different void ratios. Permeabilities deduced from consolidation test results were 100 times higher than those calculated directly in conventional tests and they concluded that two types of permeability existed, one corresponding to fluid passing through a soil matrix and the other as a fluid is squeezed from the matrix during consolidation. A modified form of equation 2.4.6 has been suggested

$$k = C \frac{e^n}{1+e} \quad (2.4.7)$$

by Samarasinghe et al (1982) who found values of n were typically between 4 and 5 while Carrier et al (1983) obtained values from 3.5 to 11 but commonly between 4 and 6 for a wide range of very soft soils at liquidity indices up to 6. Improved fits were gained by Carrier and Beckman (1984) using

$$k = C \frac{(e - \delta)^n}{1 + e} \quad (2.4.8)$$

where δ was related to the plastic limit and plasticity index but in practice was close to 0.5 for many soils and had little effect at high void ratios. The latter two papers found an approximate unique relationship between k and the liquidity index which was independent of soil type. Tavenas et al (1983b) argued that these and other models were based on remoulded soil testing and were inapplicable to initially intact soils, where a relationship of the form

$$\log\left(\frac{k}{k_i}\right) = -\frac{(e_i - e)}{C_k} \quad (2.4.9)$$

should be used where e_i and k_i are the initial values of e and k before further laboratory consolidation. Their preference of this form over others, including that of equation 2.4.7 above is disputable on the basis of their presented data alone, and limits strains to less than 20% for accurate modelling of consolidation. No data are presented for in situ void ratios higher than 2.5, but they did show dependence of k_i on, in addition to e_i , the clay fraction, the plasticity index and soil fabric and also suggested that permeability anisotropy in the natural state was negligible. Other authors have found more direct correlations between permeability and induced pore pressure (Barden and Berry, 1965), porosity (Bryant et al, 1975) and effective stress and void ratio together (Koppula and Morgenstern, 1982) although all may be related to the void ratio in specific situations. For very soft sediments and low hydraulic gradients in particular, a definite relationship describing pore fluid flow relative to the solid phase has yet to be established.

2.4.3 Compression relationships

Relationships between void ratio (or other volumetric parameter) and effective stress vary considerably between different soils, and have been observed to be strongly dependent on the method of testing. Leonards and Ramiah (1960) concluded furthermore that laboratory results depended on the initial water content before consolidation (an observation made also by Woo et al, 1977) and could deviate significantly from those observed in the field.

Crawford (1964) attributed this largely to unrealistic loading rates in the laboratory where rates of strain might be over one million times those occurring naturally in the field, causing an incorrect balance to be observed between hydrodynamic and structural viscosity or between primary and secondary consolidation. He recommended that the consolidation test be performed at a constant rate of compression and sufficiently slowly to prevent development of significant pore pressures, which is consistent with the requirements for accurate permeability measurement discussed in the previous section. To provide more consistent results a number of tests have been developed as improvements on the incremental loading test, in particular the constant rate of deformation (CRD) tests, the constant rate of loading (CRL) test and the continuous loading (CL) test. Each can be used to obtain soil constitutive relationships via analysis procedures which contain simplifying assumptions and relate to the specific boundary conditions (Znidarcic et al, 1984). The restricted flow test developed at Oxford University (Lee, 1977, 1979; Sills, Hoare and Baker, 1985) has the great advantage that tests may be performed quickly but at very low hydraulic gradients within the soil without complicated apparatus or feedback systems. The slurry consolidometer, in conjunction with one of the techniques above, is usually used to investigate behaviour at very high void ratios, where consolidation may be induced by seepage forces which complicate the analysis (Znidarcic and Schiffman, 1981) and these forces have been used to induce consolidation in seepage tests (Imai, 1979; Kirby, 1985) in which case properties may be calculated directly. All these methods yield only one compression curve per test so that variable behaviour due to rate or gradient effects, for example, cannot be detected. Scully et al (1984) determined compression and permeability relationships for phosphatic clay at initial void ratios up to 20 using incremental loading and CRD tests and compared compression results with those deduced from void ratio profiles at the supposed end of primary consolidation in centrifuge and settling column tests, for initial values up to 30. Wide variations obtained between tests of similar

and different types and apparent preconsolidation effects were observed for experiments begun at void ratios below a value around 23 representing the suspension/soil transition. Compression curves moved towards an upper bound virgin compression line which was less distinct than that seen in data presented by Imai (1981) and discussed earlier. Very similar variations between testing methods were observed by Lee (1979) who found that much lower void ratios were achieved at low stresses in incremental loading tests than in restricted flow tests. Scully et al suggest that compression curves (other than those from settling tests) are equivalent for void ratios less than 7, but even that appears speculative since at this value effective stresses in their tests range from 0.9 to about 2 kN/m² and the centrifuge curve represents in reality a void ratio-effective stress isochrone rather than a true consolidation path. The same limitation applies to the settling test curves and to all tests in settling columns reported elsewhere in which final void ratio profiles are used to derive compression data.

One-dimensional compression behaviour of soils in states denser than the liquid limit is usually described by relationships of the form

$$e = e_1 - C_c \log\left(\frac{\sigma'}{\sigma_1}\right) \quad (2.4.10)$$

where e_1 is the void ratio at an effective stress σ_1 and C_c the compression index. The compression index is assumed constant at these states and has been correlated with the liquid limit (e.g. Skempton, 1944, for remoulded soils), with the void ratio before load application (Leroueil et al, 1983) or with both (e.g. Salem and Krizek, 1976). A constant value of C_c is inappropriate for soils at high liquidity indices, although relationships have been proposed between C_c and current void ratio or density (e.g. Rendon-Herrero, 1980; Sridharan and Jayadeva, 1982). Such relationships effectively imply that equation 2.4.10 is not suitable at these states. Carrier and Beckman (1984) extended work by Carrier et al (1983) relating general soil behaviour parameters to index test results. They suggested that for many remoulded clay soils once initial void ratio effects have been overcome (and therefore for

sediments deposited from a slurry, where void ratio is maximised for a given effective stress) compression is described well by

$$e = \alpha \left(\frac{\sigma'}{p_{atm}} \right)^{-\beta} + \delta \quad (2.4.11)$$

where p_{atm} is the atmospheric pressure so that α , β and δ are dimensionless constants for a particular soil. This form of relationship appears suitable for void ratios up to 30 as well as for effective stresses as high as 10,000 kN/m² and points between. Following measurements by Olsen and Mesri (1970), who found liquidity index-effective stress curves well ordered by activity for clays sedimented from a slurry, they showed further that the compression behaviour of almost all the clay slurry sediments could be represented by

$$\frac{LI}{0.963 + .808/\text{activity}} = LI' = 0.953 \left(\frac{\sigma'}{p_{atm}} \right)^{-0.143} - 0.61 \quad (2.4.12)$$

where LI' is the modified liquidity index, defined to give an empirical best fit. Comparison of equations 11 and 12 suggests that β ($=0.143$) is constant for all clay sediments, while α is proportional directly to the plasticity index and inversely related to the activity and δ is related directly to the plastic limit and α .

2.4.4 Consolidation theory - simplifications

Large strain consolidation analyses are most often based on the formulation by Gibson, England and Hussey (1967). They required only assumptions 3,4 and 8 of section 1 to be valid in deriving general equations but simplified these by invoking in addition assumptions 1,2,5,6,7 and 9 to obtain

$$\left(\frac{\gamma_s}{\gamma_w} - 1 \right) \frac{d}{de} \left[\frac{k(e)}{1+e} \right] \frac{\partial e}{\partial z} + \frac{\partial}{\partial z} \left[\frac{k(e)}{\gamma_w(1+e)} \frac{d\sigma'}{de} \frac{\partial e}{\partial z} \right] + \frac{\partial e}{\partial t} = 0 \quad (2.4.13)$$

which is often termed 'the most general equation governing one dimensional finite strain consolidation', with all variables as defined earlier, and z measured against gravity. Similar equations have been formulated by Mikasa (1963), assuming uniform initial void ratio everywhere, in terms of strain and by Koppula and Morgenstern (1982) in terms of excess pore pressure. Solution of the non linear partial differential equation 13 for $e(z,t)$ requires

knowledge of the compression ($e-\sigma'$) and permeability ($k-e$) relationships and input of initial and boundary conditions. Analytical solutions to the general equation are not available and numerical methods are employed frequently which, although providing accurate and versatile capabilities, may require considerable computational effort if effects of different conditions are to be evaluated. Rewriting equation 2.4.13 as

$$-f(e)\frac{\partial e}{\partial z} + \frac{\partial}{\partial z}[g(e)\frac{\partial e}{\partial z}] = \frac{\partial e}{\partial t} \quad (2.4.14)$$

$$\text{where } f(e) = \left(\frac{\gamma_s}{\gamma_w} - 1\right) \frac{d}{de} \left[\frac{k}{1+e}\right] \quad (2.4.14a)$$

$$\text{and } g(e) = -\frac{k}{\gamma_w(1+e)} \frac{d\sigma'}{de} \quad (2.4.14b)$$

suggests that this effort may be reduced by assuming that the permeability and groupings such as $g(e)$ and $f(e)$ retain constant values throughout small elements during short timesteps and updating these values in successive timesteps, during each of which strains remain very small. This infinitesimal strain method allows variable deposition and soil inhomogeneity to be incorporated simply and has been used by Olson and Ladd (1979) and Yong et al (1983), although Sills and Lee (1979) show the disadvantageous effects of cumulative small errors.

Analytical solutions have been obtained to the governing equations under certain restrictive conditions and since the computational advantages allow more detailed inspection of the large strain (primary) consolidation process it is worth considering their respective merits and validity. Gibson et al obtained solutions for thin layers where self weight can be neglected so that $f(e)$ is zero by assuming that the finite strain coefficient of consolidation, C_F , is constant

$$C_F(e, e_i) = -\frac{k(e)}{\gamma_w} \frac{(1+e_i)}{(1+e)} \frac{d\sigma'}{de} = (1+e_i)^2 g(e) \quad (2.4.15)$$

where e_i is the uniform void ratio before any consolidation increment. Lee and Sills (1981) derived solutions under different initial and boundary deposition conditions for the linear soil model with

$$k = C_k(1+e) \quad , \quad C_k = \text{constant} \quad (2.4.16)$$

$$e = e_i - a_v \sigma' , \quad a_v = \text{constant} \quad (2.4.17)$$

in which case $f(e)$ is zero but self weight is not neglected and $g(e)$ is constant. It should be noted that some ambiguity has resulted from Lee and Sills entitling this constant " C_F , (as) defined by Gibson et al" whereas the two in fact differ by a factor $(1+e_i)^2$, seen in equation 2.4.15. Lee (1979) also considered the non linear soil model defined by

$$k = C_k(e - e_\infty)(1+e) \quad (2.4.18)$$

$$e = e_\infty + (e_i - e_\infty)\exp(-c\sigma') \quad (2.4.19)$$

where e_∞ is the void ratio at infinite stress, C_k and c are constants. Cargill (1984) showed that this compression relationship represented two dredged materials adequately, consolidating to effective stresses of 25 kN/m², from initial void ratios up to 18. In this case $f(e)$ and $g(e)$ are both constant so that equation 2.4.14 takes the linear form

$$-f \frac{\partial e}{\partial z} + g \frac{\partial^2 e}{\partial z^2} = \frac{\partial e}{\partial t} \quad (2.4.20)$$

Lee obtained analytical and approximate solutions for a uniform material consolidating under self weight alone for single and double drainage conditions, and for a normally consolidated material subjected to additional time dependent loading. Gibson et al (1981) discuss, using numerical solution techniques, predictions obtained for a particular field case using this soil model compared to predictions using conventional Terzaghi theory, which overestimates consolidation time but underestimates excess pore pressures. Cargill showed that the assumption of constant g is reasonable for void ratios greater than 2, but it may be deduced from his permeability data that $f(e)$ varies by several orders of magnitude in the same range, with greatest deviations from an average value at very high or low void ratios.

Schiffman et al (1984) discussed the limitations of large strain theory formulated under the assumptions leading to equation 2.4.13, including in particular the requirement for inclusion of creep effects. These and rate effects might be allowed for using an extension of the form proposed by Schiffman to

$$de = A(\sigma', t, \frac{\partial \sigma'}{\partial t}) d\sigma' + B(\sigma', t, \frac{\partial \sigma'}{\partial t}) dt, \quad \frac{\partial A}{\partial t} \neq \frac{\partial B}{\partial \sigma'}, \quad (2.4.21)$$

A number of models have been proposed which include time effects, generally derived from considerations of multi-element rheological systems in which compression and viscosity components exist. Early models were restricted to small strains and constant compressibility, using a linear viscosity law (Gibson and Lo, 1961; Lo, 1961). Barden (1965) analysed a simple system with non linear viscosity and compared it with other rheological models and experimental results (1968). He argued that such a model would predict the quasipreconsolidation effect under certain conditions of compression rate and that cementing might therefore not be the cause of such behaviour. The amount of secondary compression was shown to be strongly dependent on the sample thickness and pressure increment ratio in oedometer tests, indicating the errors inherent in arbitrarily dividing the continuous consolidation process into two components. De Josselin de Jong (1968) developed a mathematical model assuming an interconnected network of pores within the soil of different sizes (and hence, permeabilities). A simplification of this mechanism would assume that soil consisted of low permeability flocs or domains, responsible for secondary compression, interwoven by a network of macropores, drainage of which would be observed as primary consolidation since pore pressures measured externally would be those in the macropores. Berry and Poskitt (1972) formulated a finite strain, non linear soil theory to describe the behaviour of peat, in which macropores and micropores are known to exist, and Mesri and Rokhsar (1973) developed a similar theory to include the effects of a critical pressure. Garlanger (1972) observed that the parallel consolidation state isochrones reported by Bjerrum (1967) suggested simple extension of the compression relationship to

$$\frac{\partial e}{\partial t} = \frac{\partial e}{\partial \sigma'} \frac{\partial \sigma'}{\partial t} + \left(\frac{\partial e}{\partial t} \right)_{cr} \quad (2.4.22)$$

where the compressibility $\partial e / \partial \sigma'$ and creep rate $(\partial e / \partial t)_{cr}$ are functions of both void ratio and effective stress. His model accounted for the development of an apparent preconsolidation pressure for reloading after periods of creep, but

was based on data observed in a limited stress range. Berre and Iversen (1972), in a companion paper, found that the model predictions agreed qualitatively with results from oedometer tests on specimens of different heights. Kavazanjian and Mitchell (1977) developed a general model for soil behaviour in which the volumetric component was based on Bjerrum's data and the deviatoric component was also divided into two parts with instant strains based on normalised behaviour and creep strains based on a rate equation presented by Singh and Mitchell (1968).

2.4.5 Linking of sedimentation and consolidation

Been (1980) showed that equation 2.4.13 governing one-dimensional consolidation with finite strain reduced exactly to that developed by Kynch (1952) for hindered settling when the effective stress reduced to zero. He noted that Kynch's theory is consistent with the validity of Darcy's law, since in the case where the pore pressure is equal to the total stress, equation 2.4.5 reduces to the form for the hindered settling velocity, v_s

$$v_s = -\left(\frac{\gamma_s}{\gamma_w} - 1\right) \frac{k(e)}{1+e} \quad (2.4.23)$$

which varies only with the void ratio, as assumed by Kynch. Pane and Schiffman (1985) discussed how the general equation 2.4.13 could be used as a result to model the entire depositional process of sedimentation and concurrent consolidation. A major problem arises however in defining the transition point between settling and consolidation. Been found a distinct change in void ratio between the suspension and a value at the sediment surface below, which was not constant but varied with initial suspension density, and then decreased with time during consolidation to a presumed ultimate value, e_z . He found that this behaviour could be modelled using Lee's (1979) analytical solutions for a linear soil with an imaginary surcharge of consolidating soil. Although this technique has the appealing characteristic that it presents the time dependent void ratio change as a kind of overconsolidation effect (see section 2.2) it becomes largely empirical; the "imaginary surcharge mass" required was similar to the actual solids mass present and in one case twice as high.

Been also defined the ultimate surface void ratio e_z as the "zero effective stress point", "below which a unique virgin consolidation curve exists" to which all curves tend. Monte and Krizek (1976) defined and discussed the "fluid limit" e_o^* of a soil, at which no stress or shear strength could exist and above which the permeability measured directly in a permeability test would not be a function of void ratio. For deposition at void ratio $e_i \geq e_o^*$ the equilibrium soil fabric and behaviour would show no dependence on e_i . They found that e_o^* was close to five times the void ratio at the liquid limit, which agrees reasonably with the ultimate surface values e_z found by Been, ranging from 4.3 to 5.9 times the liquid limit value e_{LL} . Their conclusions about permeability however are at direct variance with the assumption, expressed by equation 2.4.23, that settling rate depends on the void ratio during settling. The stress free state may also be anomalous if the surface changes observed by Been occurred before e_o^* was reached. Umehara and Zen (1982) also observed that e_o^* was not unique but depended on e_i . Carrier et al (1983) suggested that $e_o^* = 7e_{LL}$ might apply in many soils and Ardaman and Associates (1983) proposed $e_o^* = 7e_{LL} - 7.6$ for phosphatic clays but this does not appear to be valid for other soils with lower liquid limits. The first relationship predicts Been's initial transition values adequately while the second approximates his ultimate values, e_z . Clearly, a difference exists between any ultimate void ratio e_z and the initial transition value obtained during settling, and the reason for the difference between these two has not been established satisfactorily to date.

Bloomquist and Townsend (1984) report results of centrifuge studies for clays of very high void ratio, where scaling relationships from prototype to model are $l_p:l_m = N$ for geometry and $t_p:t_m = N^x$ for time, where N is the acceleration factor (in gravities) and x a time scaling exponent. Modelling of models experiments suggested that x varies between 1.0 and 2.0, the theoretical values for settling and consolidation respectively. During hindered settling the exponent remained close to 1.6, but in a transition range of average void

ratio from 11.7 to 10.4 a gradual increase occurred up to $x=2.0$. A fluid limit in excess of 20 has been observed for a similar clay and casts doubt on some of the assumptions made using the same theory to predict prototype and model behaviour. Excellent agreement was, however, obtained otherwise between results from controlled field tests, centrifugal modelling and finite strain theoretical predictions. Good agreements of this type were also reported by Scully et al (1984) and Mikasa and Takada (1984), but for much lower initial void ratios.

Corresponding to the variable surface void ratio in this transition region been reported highly variable compression behaviour at any point in a sediment where $e > e_z$. Pane and Schiffman suggested that this might be due to inapplicability of the conventional effective stress principle at very high void ratios and proposed a more general form as

$$\sigma = \beta(e)\sigma' + u \quad (2.4.24)$$

where the interaction coefficient β is a monotonic function of the void ratio varying from zero for suspension settling up to unity for a mature soil with full particle to particle contact. The finite strain equation 2.4.13 then expands to

$$\left(\frac{\gamma_s}{\gamma_w} - 1\right) \frac{d}{de} \left[\frac{k}{1+e} \right] \frac{\partial e}{\partial z} + \frac{\partial}{\partial z} \left[\frac{k}{\gamma_w(1+e)} \beta \frac{d\sigma'}{de} \frac{\partial e}{\partial z} \right] + \frac{\partial}{\partial z} \left[\frac{k}{\gamma_w(1+e)} \frac{d\beta}{de} \sigma' \frac{\partial e}{\partial z} \right] + \frac{\partial e}{\partial t} = 0 \quad (2.4.25)$$

which reduces to Kynch's equation when $\beta=0$ or equation 2.4.13 when $\beta=1$. Values of β between 0 and 1 might occur with high relative soil/water velocities v_r , when flocculated elements are very unstable and drainage paths change rapidly, possibly with channelling. The deposition/ consolidation process is best analysed by applying the method of characteristics. Advantages of this modified, linked theory appear to be mainly computational; it is not clear from their paper what physical meaning β or σ' might have for $0 < \beta < 1$, although this method might be able to account for the presence of microstructural forces of attraction and repulsion. For a step change from $\beta=0$ to $\beta=1$ (as in their example case) there seems little advantage over using two theories separately

with compatible boundary conditions at the settlement/consolidation interface, as proposed by Tiller (1981).

A more fundamental approach to dealing with anomalous behaviour in a transition region is that proposed by Kos (1985). He considers non-Darcian flow behaviour represented by a permeability $k(e, v_r)$, although permeability is a misleading concept under these conditions and behaviour would be better described by $v_r(e, i)$ where the velocity-gradient relationship is no longer linear. The relative velocity is also presumed to affect the compression behaviour, so that

$$de = A(e, v_r) d\sigma' + B(e, \sigma') dv_r \quad (2.4.26)$$

where the equation has been reformulated here in terms consistent with the other work discussed. The first term will dominate in soils consolidating at low void ratios provided velocities are small and the second term will govern settling behaviour when $\sigma' = 0$ or will dominate in consolidation when effective stresses are small and velocities are high. The normal equations of continuity will link these two constitutive models for flow and compression and determine time and space dependence, resulting in a relationship of the form described generally at the beginning of this section by equation 2.4.3.

2.4.6 Summary of consolidation modelling

In this section an attempt has been made to place into perspective many of the different aspects of one dimensional volume compression behaviour. It has been shown that even the most generally accepted theories in common use may be severely limited in their range of application by the assumptions necessary to provide equations in tractable form. Some of these limitations may be overcome by specific models which are valid under particular conditions, such as those which consider creep and rate effects. Others are due to the application of assumptions, such as validity of Darcy's law or dependence of void ratio solely on effective stress, which have been verified only for limited ranges of soil states. A third class of assumptions are used in the absence of any practical method for determining true soil characteristics under all conditions, such as

the separation of solids and fluid phases in conventional density parameters to describe volume changes or fluid flow, where only a limited fraction of the fluid phase may be free to migrate.

All these limitations are magnified at very high void ratios and low effective stresses. Reduction of computing costs has encouraged the advent of extensive theoretical and empirical models for these soil states which require input parameters obtained from laboratory tests or field studies. Most laboratory tests in particular are still based on those designed for stiffer, denser soils and, as in field studies, important parameters such as void ratio and pore pressure are not measured continuously throughout samples and during the entire testing period, so that derived average parameters do not reflect the fundamental soil behaviour.

2.5 Strength testing in very soft soils

Very few reported studies exist of strength testing carried out concurrently with measurement of effective stresses and compression behaviour in normally consolidated soils at vertical effective stress levels below 5 kN/m². The only attempt at such a study appears to be that by Einsele et al (1974) discussed in section 3. Patterns of behaviour found were sufficiently variable to suggest that at these low stresses, strength behaviour is understood very poorly.

Shear strength in soils has been represented traditionally by a (Mohr) failure envelope limiting possible stress states on a shear stress-direct stress Mohr diagram (or on a modified diagram using stress invariants). This envelope may vary in position and shape with many factors dependent on particular testing methods and different envelopes may be required to describe peak strength, remoulded strength or residual strength at very large displacements. Some evidence (e.g. Olson, 1974) suggests that the envelope for normally consolidated, uncemented soils passes through the origin of these axes but is curved, so that a tangent to any point will intersect the shear stress

axis at a non-zero value. This curvature is greater in soils of high plasticity and at very low direct stresses ($<10 \text{ kN/m}^2$). Approximation of the Mohr envelope in a particular stress range by a straight line gives the classical failure criterion for shear stress on the plane of failure

$$\tau_f = c' + \sigma_v' \tan \phi' \quad (2.5.1)$$

where c' , the "apparent cohesion" and ϕ' , the angle of friction for effective stresses, are therefore both functions of σ_v' or void ratio if wider stress ranges are considered. Skempton (1964) however described why this approach is an oversimplification in soft clays, where fabric rather than void ratio will be important. Failure in soils at lightly overconsolidated states might be represented by an equivalent expression to 2.5.1 provided c' and ϕ' are obtained as functions of an equivalent effective stress for a normally consolidated soil state at similar void ratio. In soils of medium to low plasticity ϕ' is often nearly constant at all stresses so that c' is always very small; in the critical state models for example, c' is assumed zero and ϕ' constant for soils sheared to the constant volume (critical) state. Voight (1973) and Kanji (1974) compared correlations between ϕ' (or $\tan \phi'$) and plasticity index obtained for many soils by different authors, but although they claim good correlations for some soils the wide range of data shown by Kanji support the opinion stated by Kenney (1967) that good such correlations valid for all soil types do not exist. This may largely be explained by the envelope curvatures demonstrated by Olson, and also indicates that different values of ϕ' apply in different tests. At the very low shear strengths found in recent sediments, samples will not stand unsupported and conventional triaxial testing is impossible so that effective stress paths followed during shearing can not be measured easily. Consequently, strength testing on such soils has to be done either in situ or using field-type inserted instruments. Many factors influence the results of such tests, including in particular (see Crawford, 1963)

- method of testing and mode of failure (determined or free)

- orientation of failure zone, where anisotropy occurs
- boundary conditions, freedom of principal stress rotation.
- size of failure zone and amount of strain at assumed failure
- time to failure or rate of testing, including effects of structural and pore fluid viscosity, partial consolidation and undrained creep
- soil disturbance due to: stress relief with removal of overlying soil; simultaneous disturbance of exposed surface; insertion of testing instrument
- time between disturbing operation and testing.

Results of these tests are often described by the concept of an undrained strength, although this is notoriously unsatisfactory if used alone since any one value is dependent on all the factors above. It is usually defined as half the difference between the major and minor principal stresses at failure,

$$s_u = \frac{1}{2}(\sigma_1' - \sigma_3')_f = \frac{1}{2}(\sigma_1 - \sigma_3)_f \quad (2.5.2)$$

which ignores the value of the intermediate principal stress σ_2' . This definition is useful where σ_1' and σ_3' (or σ_1 and σ_3) are known, but as described by Wroth (1984) in detail, the undrained strength is often equated simply to the assumed shear stress at failure on a known plane, τ_f . If a failure envelope is tangential to the Mohr's circle at this stress point $\tau_f = r \cos \phi'$ and is less than the Mohr's circle radius, r , if ϕ' is greater than zero. Skempton and Sowa (1963) stated that

"If two identical specimens of saturated clay are subjected to different changes in total stress without alteration in water content, and if the strains consequential upon these stress changes cause little if any alteration in microstructure prior to failure, the undrained strengths of the two specimens will be identical."

This "principle" has often been paraphrased by the assumption of constant undrained strength at constant water content although it is the second part of the principle concerning microstructural changes that is probably most important and cannot be verified easily where different tests are used.

It is useful to normalise the undrained strength by some relevant effective stress to describe soil behaviour over a wide stress range and compare behaviour in different soils. Wroth suggests the most sensible normalising value would be p_f' , the mean effective stress at failure, but since neither this nor the initial value p_o' is usually known a satisfactory engineering compromise is to use the initial vertical effective stress, σ_{vo}' and investigate variations in s_u/σ_{vo}' . This ratio will vary with ϕ' for normally consolidated soils and will not be constant for a soil if the failure envelope is curved or if the testing factors are influential, which may explain scatter in correlations between s_u/σ_{vo}' and plasticity index (proposed by Skempton, 1954) or liquid limit (Hansbo, 1957) and discussed by Karlsson and Viberg (1967). Wroth invoked the Matsuoka failure criterion to include effects of the intermediate principal stress and compare tests with different applicable values of ϕ' but starting at the same initial states σ_{vo}' , and suggested that ϕ' for triaxial compression is approximately 8/9 times ϕ' for plane strain, assumed the same for all plane strain tests including direct simple shear (and the shear vane test).

Many authors have investigated variation in undrained strength with void ratio, water content or other measure of density, although few studies included soils at states wetter than the liquid limit. Data for these states have been compiled from many sources listed in table 2.1 and are shown in figure 2.2 in a plot of liquidity index against remoulded undrained shear strength. Also shown are results of viscometer tests undertaken by Owen (1970) on a settled bed and critical erosive stress data obtained from erosion flume tests. Several points are of interest. Logarithmic scales are used on both axes for ease of comparison and it is seen that almost all data are closely grouped, despite the wide range of soil types represented, about a best fit curve which is linear in this plot for liquidity indices greater than 0.4 and is described by

$$s_{ur} = s_u(\text{liquid limit}) / LI^3 \quad (2.5.3)$$

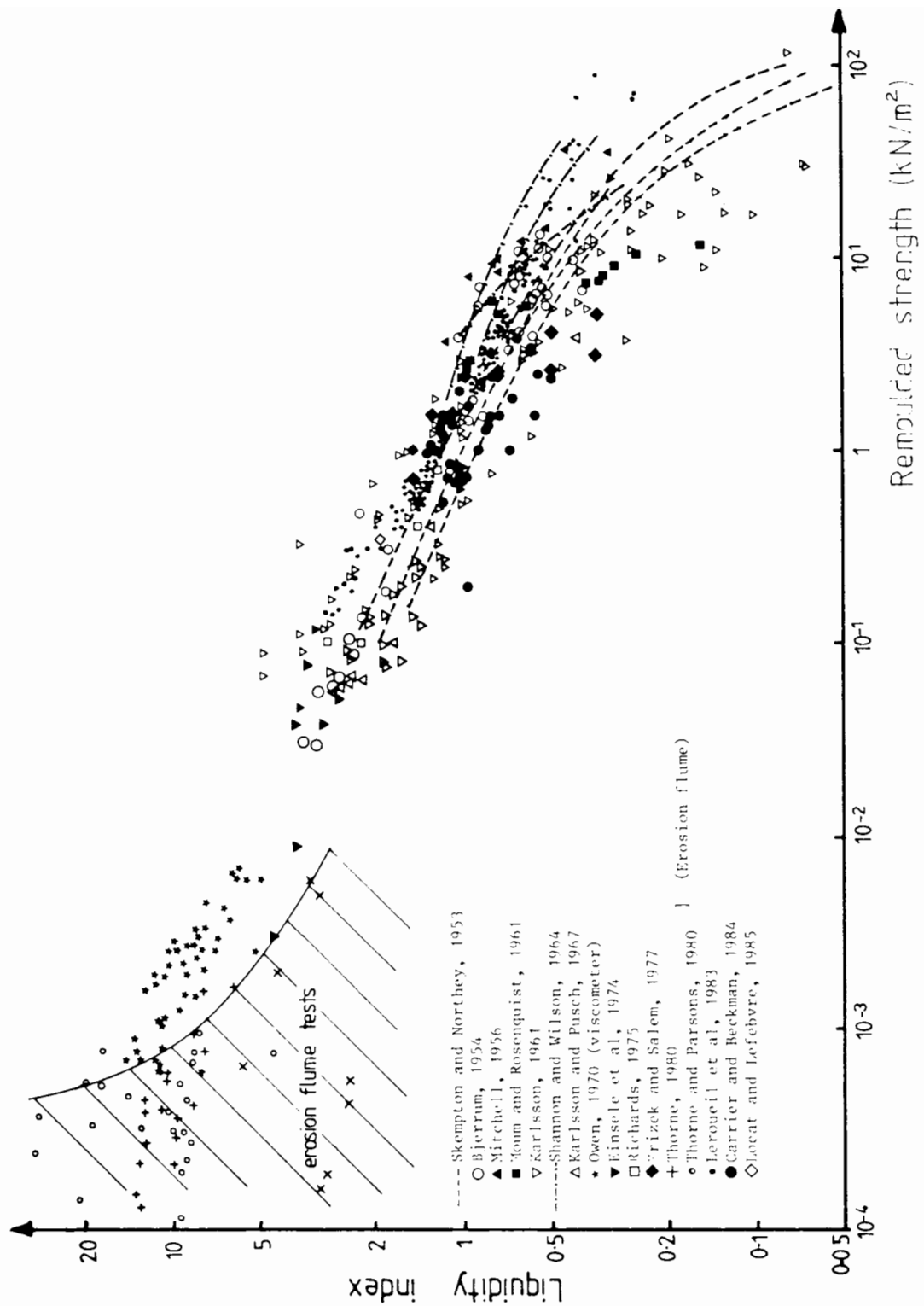


Figure 2.2 Variation in remoulded strength with liquidity index.
Dashed lines are curve fits reported by authors for data.

REFERENCE	SOIL TYPE	LL METHOD	RANGE OF LL PL		STRENGTH TEST
Skempton & Northey (1953)	English & Norwegian	Casagrande	30-97	14-65	vane
Bjerrum (1954)	Norwegian clays	Casagrande	20-60	5-38	vane
Mitchell (1956)*	various	Casagrande	-	-	-
Mourn & Rosenqvist (1961)	Montmorillonite & illite	-	52-310	-	-
Karlsson (1961)	Swedish & Liberian soils	Casagrande (& fall cone)	40-820	12-400	fall cone & vane
Shannon & Wilson (1964)*	Alaskan soils	-	-	-	-
Youssef et al (1965)	Egyptian soils	Casagrande	34-190	8-150	vane
Karlsson & Pusch (1967)	Swedish clays	Fall cone?	50-80	25-50	fall cone, vane, & UU triaxial
Owen (1970)	English, Avonmouth	-	91	57	Brookfield viscometer
Einsele et al (1974)	Kaolin, bentonite, & illite	-	48-137	13-83	vane, erosion flume
Richards (1975)	various marine	Casagrande	50-144	14-91	vane
Krizek & Salem (1977)	river dredgings	Casagrande	50-90	20-55	vane
Thorn (1980)	Avonmouth	-	91?	57?	erosion flume
Thorn & Parsons (1980)#	various	-	60-130?	30-100?	erosion flume
Leroueil et al (1983)	various, worldwide	Fall cone	-	-	fall cone
Carrier & Beckman (1984)	Phosphatic clays	Casagrande	155-222	110-165	vane
Locat & Lefebvre (1985)	Canadian	Fall cone	34-48	12-25	fall cone
* denotes data from Mitchell (1976)					
# LL & PI estimated on the basis of mineralogy and data from similar muds					

Table 2.1 Sources of data for figure 2.2.

The shear strength at the liquid limit appears to be variable in a range between 0.5 and 4 kN/m². It has been shown elsewhere (Youssef et al, 1965; Sherwood and Ryley, 1970) that if the Casagrande apparatus is used to determine the liquid limit the shear strength decreases with increasing liquid limit. If the fall cone is used for this determination, the vane shear strength is nearly constant and in addition scatter is reduced considerably. The nature of the cone test (standard penetration of a standard cone) implies that the liquid limit in this case is determined at a (nearly) constant strength. Karlsson (1961) shows vane strengths varying between 0.5 to 4.2 kN/m² at the Casagrande liquid limit but between 1.5 and 2.1 kN/m² at the fall cone limit for the same soils. The unnecessary errors associated with operator variability in particular (Karlsson, 1977; Sherwood and Ryley, 1970) support strongly a recommendation that the fall cone test should completely supersede the Casagrande apparatus method. Leroueil et al (1983) found that shear strengths for many soils could be represented in the range $0.5 < LI < 2.5$ by

$$s_{ur} \text{ (kN/m}^2\text{)} = (LI - 0.21)^{-2} \quad (2.5.4)$$

but all strengths were measured with the fall cone assuming a constant value of $s_{ur} = 1.6 \text{ kN/m}^2$ at the liquid limit and hence a constant cone calibration factor. Other methods might result in greater variation. Houlsby (1982) showed theoretically how the cone factor might vary with the ratio of soil-cone adhesion : undrained strength. Variation in this factor between soils for one cone type was reported by Karlsson and would account for small vane strength variations at constant cone penetration. Carrier and Beckman (1984) noted that shear strength increases as activity decreases at a given liquidity index; this effect was also observed in the data plotted in figure 2.2. The relationship expressed by equation 2.5.3 is perhaps to be preferred for these reasons. It is also able to describe the results from viscometer tests, where the Bingham yield value (extrapolated shear stress at zero shear rate) is assumed to be a measure of the undrained strength. The two models are shown in figure 2.3, where ranges of data obtained from the different types of tests are indicated.

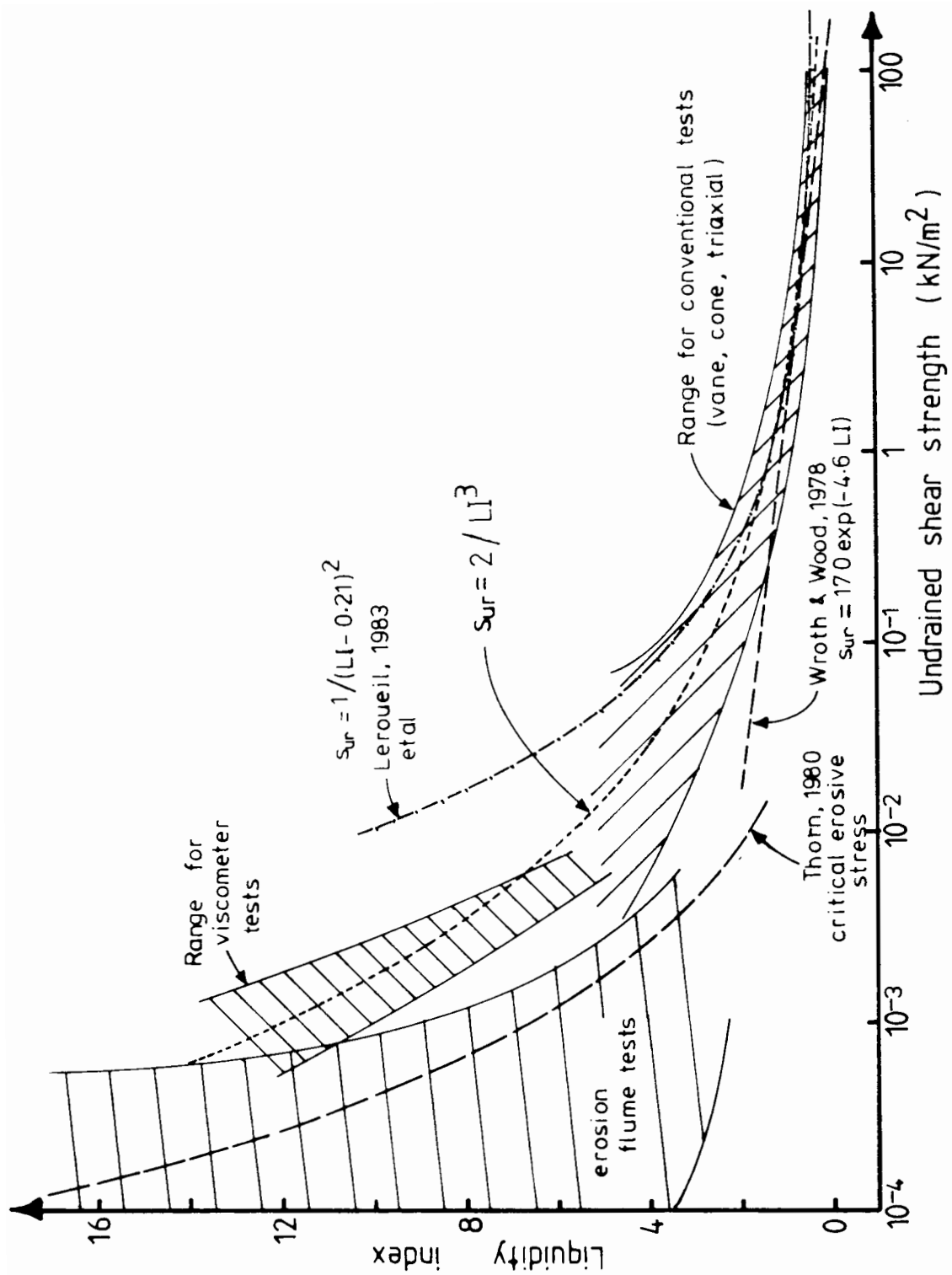


Figure 2.3 Some models for remoulded strength variation with liquidity index.

A linear liquidity index scale is used as is more common in geotechnical engineering. Comparison with figure 2.2 shows the advantages of the logarithmic scale at higher liquidity indices. It is interesting that at these higher values a linear $\log LI - \log s_{ur}$ relationship implies a nearly linear relationship between logarithm of water content or void ratio and $\log s_{ur}$; Butterfield (1979) suggested that a $\log(\text{void ratio}) - \log(\text{effective stress})$ plot is also linear in this range, with a negative slope equal to 0.143 suggested by Carrier and Beckman (see equation 2.4.12). Combination of these models suggests a relationship between remoulded, undrained shear strength and vertical effective stress in a normally consolidated soil

$$s_{ur} \approx c \sigma_v'^{0.43} \quad (2.5.5)$$

which might apply at very low strengths or high liquidity indices. The constant, c , would depend on plasticity index and activity for a particular soil, as suggested by Carrier & Beckman for their compression and strength relationships. Equation 2.5.5 may be contrasted with similar forms of relationship for stronger soils where s_u/σ_v' is often assumed constant, as discussed earlier. This assumption is made in critical state models and a relationship suggested by Wroth and Wood (1978) is plotted in figure 2.3. It is seen to describe data well in the range below the liquid limit for which it was intended, but clearly should not be extrapolated far above this range. A final point to note is that the critical erosive stress obtained in flume tests appears to be lower in most cases than the equivalent "strength" at the same liquidity index. Consideration of these differences is outside the scope of this study but may be due to the method of calculating a fluid imposed shear stress on the basis of an assumption about near bed flow. The high variation in these data when compared to that in the conventional strength data suggests that a detailed study of this behaviour carried out in conjunction with conventional strength tests could be of great benefit to those studying erodibility phenomena.

2.6 Implications of review for research requirements

In this chapter an attempt has been made to collate known behaviour of extremely soft sediments, accepted models for soil behaviour and some ideas and results derived in other areas which may help to explain the complex and variable behaviour observed in these sediments. It has been shown that many interpretations of observed sediment behaviour are contradictory and no clear pattern has emerged. This is due in part to the difficulty of obtaining accurate data and in part due to the application of assumptions which have not been validated at very low density states. All the questions following require further consideration.

1. Under what conditions do macrostructural parameters adequately represent soils states and behaviour governed by interparticle forces which cannot be measured directly?

- what volumetric parameters are appropriate to represent state? To describe compression or volume change?

- what stress or pressure parameters are appropriate?

2. What effects do initial depositional conditions (concentration, rate, mixing process) or boundary conditions (drainage, surcharge) have?

3. When does a settling suspension become a consolidating soil?

- when pore pressure falls below total stress?

- at a "fluid limit" e_0^* defined uniquely?

4. When can conventional theories be applied?

- what constitutive relationships (e.g. $e-\sigma'$, $k-e$) should be used?

- how should initial deposition effects on subsequent properties be included?

5. Are theories required which are formulated on different bases?

6. How is behaviour best modelled experimentally? Can long term effects be studied in small, short term tests (e.g. oedometer / slurry consolidometer / settling column)?

7. What measurements should be made and how? How should sampling be undertaken and what will its effects be?
8. Can recommendations be made to improve testing equipment, methods or procedures?
9. Can recommendations be made about ways of improving soil properties? For example, how might settling and consolidation be accelerated by:
- using different depositional processes?
 - altering properties with a dispersant/flocculant?
 - changing drainage conditions or imposing a hydraulic gradient?
 - applying surcharges?

Most of these questions are examined or addressed in this thesis.

- 3.1 Tall Settling Columns, 1-4 metres
 - 3.1.1 Pressure measurement
 - 3.1.2 Triaxial segment

- 3.2 Short Columns

- 3.3 Density Determination Using X-ray Attenuation
 - 3.3.1 Application of attenuation theory
 - 3.3.2 Technical problems and practical use of theory.
 - Sources of error
 - 3.3.3 Summary of the X-ray technique

- 3.4 Strength Testing - Interpretation and Requirements.

- 3.5 Fall Cone Testing
 - 3.5.1 A modified dynamic analysis
 - 3.5.2 Use of fall cones

- 3.6 Shear Vane Testing

- 3.7 Cavity Expansion Testing
 - 3.7.1 Theoretical analysis
 - 3.7.2 Testing apparatus
 - 3.7.3 Testing techniques
 - 3.7.4 Summary

Conclusions drawn in chapter two suggested the need for further investigation of the engineering behaviour at a fundamental level in extremely soft sediments settling and consolidating under self weight. Many conventional testing methods impose conditions which are not typical of those acting in the field. Settling columns have the advantage that compression can occur under self weight alone and were used in most experiments in this study.

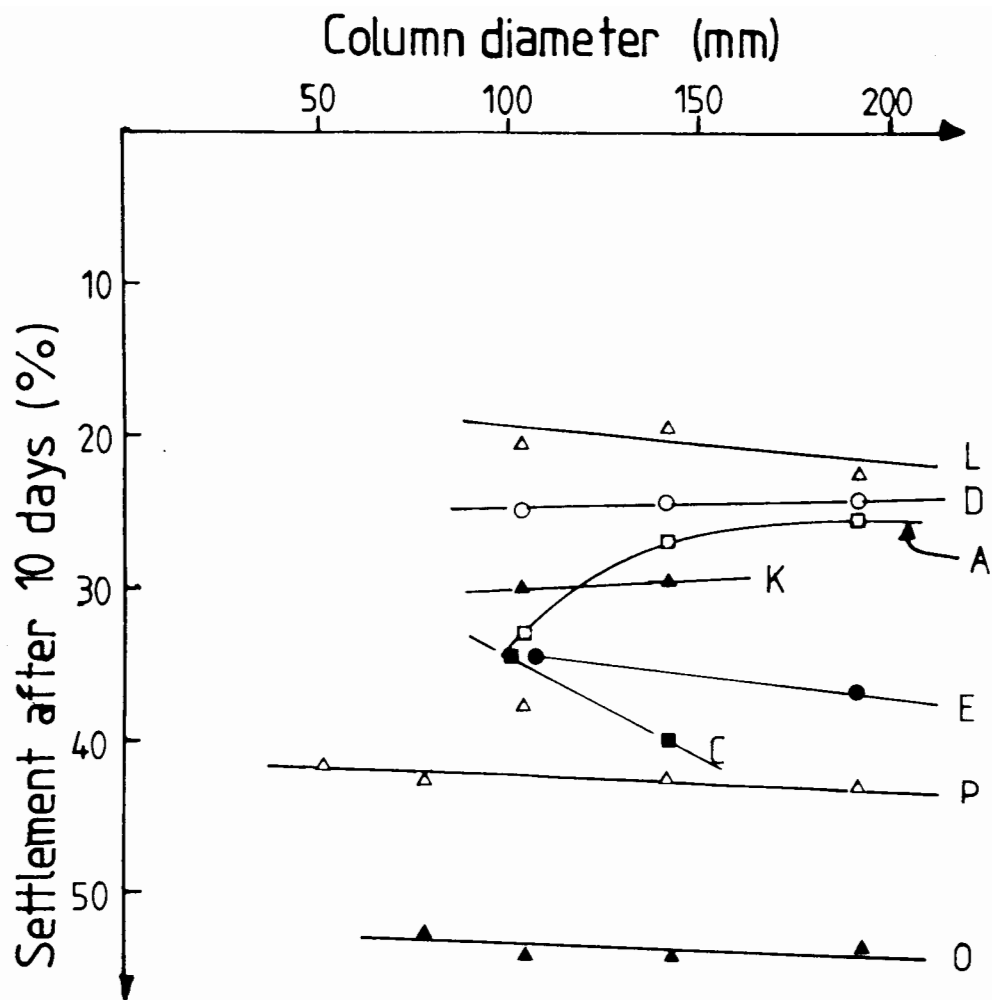
3.1 Tall Settling Columns, 1-4 metres

These were constructed from transparent acrylic tubes 102 mm ID and 114 mm OD supplied as 2 m lengths. A taller 3.5 m column consisted of two such segments joined and sealed. Been (1980) measured negligible decreases in total stress at the bases of similar columns during consolidation but initial slurry concentrations were generally lower than in this study and it was therefore necessary to consider the possible influence of column walls. Michaels and Bolger (1962) found the diameter effect to be negligible for flocculated kaolin suspensions at initial concentrations from 5 to 130 g/l in tubes up to 1200 mm high with diameters 48-65 mm. Settling rates in suspensions mixed with a blender were higher than those mixed by repeated tube inversion but a lesser effect occurred in low pH solutions. They concluded that settling rate is governed by flocculated aggregates which grow by collision but break down under shear forces caused by mixing, so that if aggregates become sufficiently large wall effects may extend a significant distance into the settling zone. Stevenson (1972) summarised factors governing floc formation and size from a theoretical viewpoint while Peirce and Williams (1966) observed two types of settlement curves. At low concentrations settlement rate depended on aggregate size and maximum settling rate occurred immediately after input. At higher concentrations structure developed associated with a floc network extending to the container walls and settlement rate increased for a period following deposition.

The most relevant data were reported by Sills and Thomas (1983) and other unpublished data were available from experiments carried out by Thomas at Oxford during 1981. Combwich silty clay mixtures in fresh and sea water were deposited at concentrations above those found in suspensions of the same soil by Been. Column diameters varied between 50 and 190 mm. Slurry was pumped in through a tube either held at the column base during filling then removed rapidly or raised with the slurry surface. Some columns were shaken after filling and in others a spreader cone or plate was attached to the end of input tube. Many of these data have been reanalysed as part of this study and results are shown in figure 3.1. Results are plotted for convenience in terms of the settlement after 10 days as a percentage of the initial height and each curve is drawn through points representing the same depositional technique. In most cases no discernable diameter effect occurs but curves A and C, for input at the column base, show variations outside the bounds of measurement errors suggesting that this technique should be avoided. No obvious explanation for this is available but it might be hypothesised that the gentle mixing during tube withdrawal following base input is sufficient to induce aggregate growth by collision but insufficient to shear the larger aggregates.

A mechanism has been observed visually which might accelerate settlement and act in opposition to any retardation due to side wall friction. Richardson and Zaki (1954) and Been (1980) reported that mass movement of fluid up small channels could be seen at the column walls in dense suspensions. Observations of small "volcanoes" across the settlement surface suggested that these channels were distributed throughout the suspension and not just at the walls. Fitch (1966) described how this mechanism might increase the downward flux of solids if flow in the channels occurred more freely than through the settling solids. Figure 3.1 suggests either that column diameter has little influence over this effect or that this and side wall friction cancel in combination.

These considerations led to the conclusion that the columns used by Been (102 mm ID) were also suitable for study of behaviour at higher concentrations



Input Method	Expt No.	Salinity	Density (Mg/m ³)	Height (mm)
Surface	D	Salt	1.17	760
	E	Tap	1.13	590
Base	A	Salt	1.14	815
	C	Tap	1.14	700
Spreader	P	Salt	1.13- 1.16	950
	L	Salt	1.18	640
	O	Tap	1.12	970
	K	Tap	1.12	510

Table 3.1

Figure 3.1 Effect of column diameter on settlement (extracted from unpublished data by R. Thomas, 1981).

provided the input process was controlled carefully. This allowed use of some of the existing equipment without design modifications.

Some columns were constructed from 100 mm segments (130 mm total height with 30 mm overlap) to allow testing at all heights following consolidation. Sealing joints against 2 m water heads for up to 2 years presented difficulties due to lack of uniformity in the acrylic tubing but a combination of silicon grease between flanges and pipe sealant at exposed cracks proved sufficient in most cases. Water absorption by the acrylic walls can be significant and was overcome by saturating segments before use.

3.1.1.1 Pressure measurement

All columns were instrumented with pore pressure transducers, mounted in housings attached to the column walls and separated from the soil by porous filters. Druck PDCR22 series differential transducers (0-5 psi FSD) were used throughout, but although these were the most suitable of a number commercially available it was felt generally that they did not perform entirely up to expectations, and a number of standard checking procedures had to be established before reliable results could be obtained. These and details of installation are described in separate reports (Elder, 1982, 1985). By following these procedures absolute pore pressures could be calculated to accuracy ± 1 mm water head (± 0.01 kN/m²) for periods up to several hundred hours after deposition of soil, or to an accuracy ± 0.005 kN/m² for short term pressure changes. Long term measurements were less reliable but accuracy to ± 0.1 kN/m² could usually be obtained. Rubber ports were placed at regular intervals in the walls of most columns. Hypodermic needles could be inserted through these to allow additional pressure measurement.

A porous base was used in later columns to provide double drainage facility. This consisted of a "vyon" filter (permeability $\approx 2 \times 10^{-3}$ mm/s) 2 mm thick over a grooved plate. Maximum head drop through this filter was less than 1 mm H₂O at typical flow rates. Total vertical stress was measured using a transducer protruding through the filter at the centre of the base and pore

pressure was measured beneath the porous disc at a point sealed from the drainage channels.

Been used a pressure transducer with its diaphragm at the column wall to measure total horizontal stress but observed considerable scatter. In an attempt to improve resolution, rubber membranes were stretched inside two of the 130 mm column segments to cover circular channels in the column walls. These were filled with water and de-aired and the pressure measured remotely. De-airing and sealing difficulties precluded use of this technique in more than one experiment and the original system is recommended for re-adoption in further work.

3.1.2 Triaxial segment

In two column experiments the base segments were replaced by an acrylic split former, constructed from the standard tubing, containing a 102 mm triaxial membrane, into which the soil consolidated. The standard base was adapted to attach directly into a 100 mm triaxial cell while continuing to allow total stress and pore pressure measurement and drainage. Once sedimentation and consolidation had proceeded to the required state, upper column segments could be removed and the entire lower segment placed in the triaxial cell for further testing, avoiding loss of lateral support at any stage.

3.2 Short columns

Experiments in tall columns lasted for periods up to 19 months. To enable extensive investigation of subsidiary parameters (such as strength testing rate and fall cone constants) thirty short columns were constructed, 102 mm ID x 200 mm high, with impermeable bases. Transducers were not fitted, but rubber ports were placed in the walls 20 mm and 60 mm above the base through which hypodermic needles could be inserted for pressure measurement. In some tests these columns were filled with soil which was allowed to consolidate under self weight alone and in other tests thin layers of slurry at low initial

concentrations were capped with filter papers and a layer of fine sand then subjected to surcharges using coarse sand or lead shot. This second technique allowed examination of consolidation behaviour under rapid rates of effective stress increase without the loss of measurement accuracy found at low stresses in conventional consolidation cells.

3.3 Density determination using X-ray attenuation

Soil bulk density was measured in the settling columns using the non-destructive X-ray attenuation technique developed at Oxford (Been, 1981), with a number of modifications made to improve versatility and reliability. The X-ray tube, collimator and detector are now mounted on a fixed horizontal arm with columns placed midway between source and detector collimation slits as recommended by Sirwan (1964). The arm is driven vertically via a lead screw using a stepping motor, giving displayed positional accuracy ± 0.5 mm and traverse rates continuously variable from 45 mm/min to 1580 mm/min. Vibration is minimised by mounting the motor on damping pads and reducing the X-ray collimated pathlength to 300 mm, which has the added advantage of increasing the detected intensity so that shorter count integration times can be used. A Harwell 6000 series rate counting system includes a digital counter which may be used to check analogue readings.

3.3.1 Application of attenuation theory for density calibration

An ideal, narrow, monoenergetic beam of radiation, initial intensity I_i , is attenuated in a homogeneous material according to (Kohl et al, 1961)

$$I = I_i \exp(-\mu_m \rho x) \quad (3.3.1)$$

where ρ is the bulk density and x the pathlength in the material. The mass absorption coefficient, μ_m , combines the effects of scattering and photoelectric absorption. Variation in μ_m with beam energy is shown in figure 3.2. For compounds or mixtures

$$\mu_m = N_A \alpha_e \left(\frac{\sum_i n_i z_i}{\sum_i n_i A_i} \right) = N_A \alpha_e (\bar{Z}/\bar{A}) \quad (3.3.2)$$

where N_A is Avogadro's number, α_e sums the scattering and absorptional effective cross sectional areas per electron and n_i is the number of atoms of

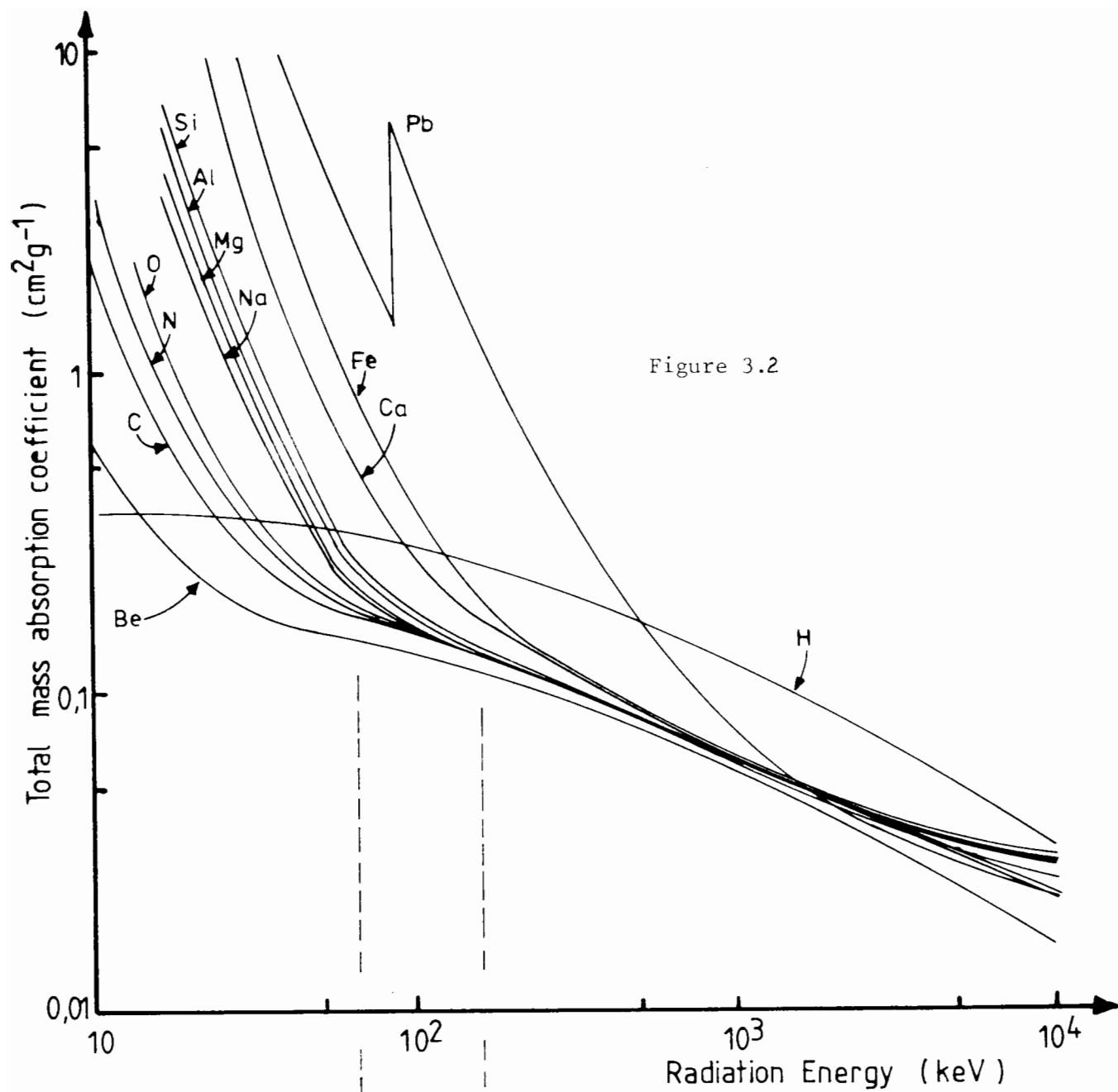


Figure 3.2

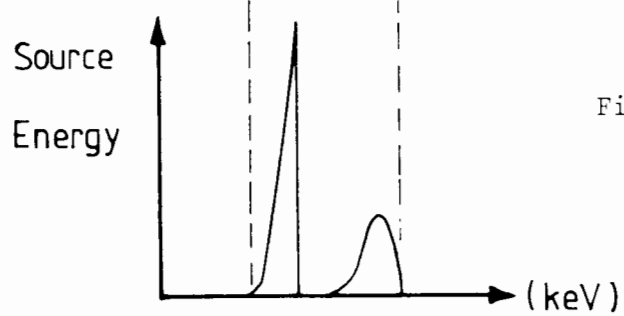


Figure 3.3

Figure 3.2 Variation in mass absorption coefficient for different elements.

Figure 3.3 Source energy distribution for Oxford X-ray system.

element i having atomic number z_i and atomic weight A_i . Maus et al (1973) list values of $\bar{Z}/\bar{A}$ for typical sediment constituents. These are within $\pm 1\%$ of 0.5 g^{-1} for most constituents but the value for water is 11% higher at 0.555 g^{-1} , since the value for hydrogen is nearly 1.0 g^{-1} . At radiation energies above 300 keV scattering dominates in light elements (Preiss, 1968) and α_e is independent of element type so that μ_m can be considered a function of water content alone. At lower energies, although an average value can be calculated, α_e becomes highly dependent on radiation energy. Assuming negligible fluctuations equation 3.3.1 may be extended by summing components from each element, compound or phase to give

$$I = I_i \exp\{-(\mu_m \rho x)_1 - (\mu_m \rho x)_2 - \dots\} \quad (3.3.3)$$

although Coumoulos (1967) showed that this summation is only a (close) approximation. Use of this equation with figure 3.2 to calculate densities directly is limited mainly by departures from the monoenergetic source assumption and by variations in dimensions and geometry of the specimen and column walls.

For reasons of economy and safety the X-ray source at Oxford produces a beam with continuous energy spectrum between 0 and 160 keV. After filtering by a 1.8 mm lead plate the incident beam has the energy distribution shown in figure 3.3 and comparison with figure 3.2 reveals that theoretical density calculation is virtually impossible. Two alternative methods were used. The first employed samples of known density and assumed identical geometry as standards for attenuated intensity calibration. These consisted initially of kaolin and bentonite mixtures in water, with densities between 1.0 and $1.5 \pm 0.001 \text{ Mg/m}^3$ determined by moisture content analysis. Later samples were prepared using the same soil as tested in the columns. The second method involved back calculation of the average attenuation equation constants from each attenuation profile, assuming a known mass of solids in each column. Applying equation 3.3.1 to water only (of density ρ_w) and then to a homogeneous

mixture of soil and water (of density ρ) gives, after elimination of the incident intensity,

$$\rho - \rho_w = -\frac{1}{k_x} \ln(N/N_w) \quad (3.3.4)$$

where N and N_w are now the count rates observed following attenuation in the soil/water mixture and water respectively, and $k_x = \mu_m x$ is the average attenuation constant, which includes the effects of material properties and sample geometry. N_w can be measured from the water overlying the soil in a column so that only k_x need be determined. The total vertical stress above hydrostatic at the base of the column may be written using a spatial vertical coordinate, z

$$\sigma_{ex}^* = \int_0^h (\rho - \rho_w) g \, dz \quad (3.3.5)$$

where h is the total height of soil and the superscript $*$ denotes a value which depends only on the mass of soil and is independent of external forces acting on the soil. Combining equations 4 and 5 and replacing the integral by incremental summation of short linear segments gives

$$\sigma_{ex}^* = -\frac{g}{2k_x} \left[\sum_{i=1}^{n-1} \{ \ln(N_i/N_{i+1}) \Delta z_i \} - 2h \ln N_w \right] \quad (3.3.6)$$

where the profile is defined by (z_i, N_i) for $i=1$ to n and $\Delta z_i = z_{i+1} - z_i$.

Calculation of σ_{ex}^* from initial mixture weighing, basal total stress measurement or some other means allows calculation of k_x for any particular profile and equation 3.3.4 can then be used to calculate densities at specific heights.

3.3.2 Technical problems and practical use of theory. Sources of error

During the first year of the experimental programme modifications were being made to the X-ray equipment. Following completion of this work some major malfunctions and problems occurred which were largely associated with the specialised demands placed on the equipment. Those malfunctions which occurred in the generating and detecting systems and were induced in the peripheral electronics are described in a separate report (Elder, 1985) which details the

fault diagnosis carried out. The report also recommends precautions or modifications necessary to prevent such faults in similar systems.

Analysis of error sources is more directly relevant. Deviations from the ideal conditions described in the previous section were of three types: variation in beam energy, variation in column geometry and attenuation and unrepresentative calibration samples.

1. X-ray beam

The energy distribution of the incident beam varied with tube current at constant applied potential although the maximum energy value remained the same. This caused the attenuation constant k_x (or μ_m) to vary with incident intensity. To yield maximum accuracy and repeatability a particular current was chosen and used in all experiments.

2. Column geometry and attenuation properties

The extrusion process used to manufacture acrylic tubing results in considerable inhomogeneity along the column length. Column curvature, misalignment from geometrical centrepoint and wall thickness variations at segment joints all contributed additional errors. The relative effects are evaluated by considering attenuation of an incident beam, of count rate N_i , according to

$$N = N_i \exp(-\mu_{mc} x_C \rho_C - \mu_m x \rho) \quad (3.3.7)$$

through the column walls (μ_{mc}, x_C, ρ_C) and soil (μ_m, x, ρ) respectively. Each effect was examined separately and results are summarised in table 3.1.

Table 3.1 Effects on attenuation of column variations

Reason for error	Variation in affected parameter			
Joints, layers of sealant	$\mu_{mc} x_C \rho_C$	$\pm$	27%	$\pm 35\%$
Inhomogeneity, wall thickness (6 ± 0.5 mm) internal diameter (102 ± 1 mm)	x_C	$\pm$	8%	
	x	$\pm$	1%	
Misalignment, along centreline (± 1 mm) transverse to beam (± 1 mm)	x	$\pm$	0.3%	$\pm 4.3\%$
	x	$\pm$	3%	

Consideration of maximum errors in average values of $\mu_{mc} x_c \rho_c = 0.18$ and $\mu_m x \rho = 1.02$ in a water filled column shows that the effect on each may be comparable:

$$-\ln(N/N_i) = (0.18 \pm 0.06) + (1.02 \pm 0.04) = 1.2 \pm 0.1 \quad (3.3.8)$$

Without correction, calculated densities could be in error by over 8%. A standard procedure was developed to eliminate these errors by comparing count rate profiles during each experiment with a dummy profile on the water filled column before soil deposition. Variations from the mean due to column variation could then be subtracted from the actual profile obtained. This technique is also described in the separate report.

3. Density calibration samples

In early experiments (and in the work reported by Been, 1980) densities were calculated from calibrations obtained using samples of assumed density. Discrepancies were observed in this study which suggested that this technique was unreliable. In some samples assumed densities, ρ' , may have been incorrect due to entrapped air or inhomogeneity in samples taken for moisture content analysis. Bacterial action and long term settlements also lead to sample inhomogeneity. Differences between x, x_c in columns and x', x'_c in samples had the effects described already. Most important however was the discovery from analysis of more than 20 column experiments that the integrated total stresses above hydrostatic using kaolin/bentonite samples to obtain soil densities were always about 10% higher than the values measured directly by transducers. No such discrepancy occurred if calibration samples consisted of the same soil used in the columns. Similar but smaller discrepancies were found when calibration samples mixed with fresh water were used to calculate densities in salt water soils. The only apparent explanation for these effects is that the material attenuation constants for kaolin/bentonite, for the soil used, for fresh water and for salt water were all different, as might be expected from description of the beam energy spectrum effects in section 2.3.1.

Using equation 3.3.3 to consider the effects of soil (μ_m, x_s, ρ_s) and water (μ_{mw}, x_w, ρ_w) attenuation components separately and assuming column homogeneity

$$N = N_0 \exp(-\mu_{mw} x_w \rho_w - \mu_{ms} x_s \rho_s) \quad (3.3.9)$$

where N_0 is the count rate which would be recorded for an empty column,

$$x_w = nx \quad \text{and} \quad x_s = (1-n)x \quad (3.3.10)$$

$$\text{and} \quad n = \frac{\rho_s - \rho}{\rho_s - \rho_w} \quad (3.3.11)$$

for full void saturation where n is the porosity and ρ the bulk density.

Combining equations 9,10,11, with $k_w = \mu_{mw} x$ and $k_s = \mu_{ms} x$, gives

$$\ln N = \ln N_0 + \frac{\rho_s \rho_w (k_s - k_w)}{\rho_s - \rho_w} - \frac{(k_s \rho_s - k_w \rho_w)}{\rho_s - \rho_w} \rho \quad (3.3.12)$$

$$\text{or} \quad \ln N = \ln N_0' - k' \rho \quad (3.3.13)$$

where N_0' and k' will be the apparent constants in the effective attenuation relationship which retains the same form as for a homogeneous, single phase material. A logarithmic plot of the attenuated count rate, N , varies linearly with bulk density between the separate lines for water and soil alone, intersecting each respectively at $\rho = \rho_w$ and $\rho = \rho_s$. The effective calibration lines for kaolin/bentonite samples and samples of Combwich soil (used in this research), all mixed in fresh water, were obtained and the actual value of N_0 was measured for an empty column segment. These are shown in figure 3.4. The true attenuation lines for kaolin/bentonite, Combwich soil and water are also constructed using the common value of N_0 and known values of ρ_w and ρ_s . Assuming the effective pathlength is approximately the internal column diameter, apparent values of the mass absorption coefficient, μ_m , for each mixture and actual values for each component are calculated:

component	water	kaolin/bentonite	Combwich	k/b + water	Combwich + water
μ_m (cm ² /g)	0.102	0.130	0.152	0.147	0.183

Comparison of these values and figure 3.2 is of interest. The coefficient for water is lower than all the other values since at low energies the value for hydrogen alone does not increase rapidly. The value for Combwich suggests that slightly heavier elements (such as calcium or iron) are present in greater quantities than in kaolin or bentonite. A final conclusion which may be drawn

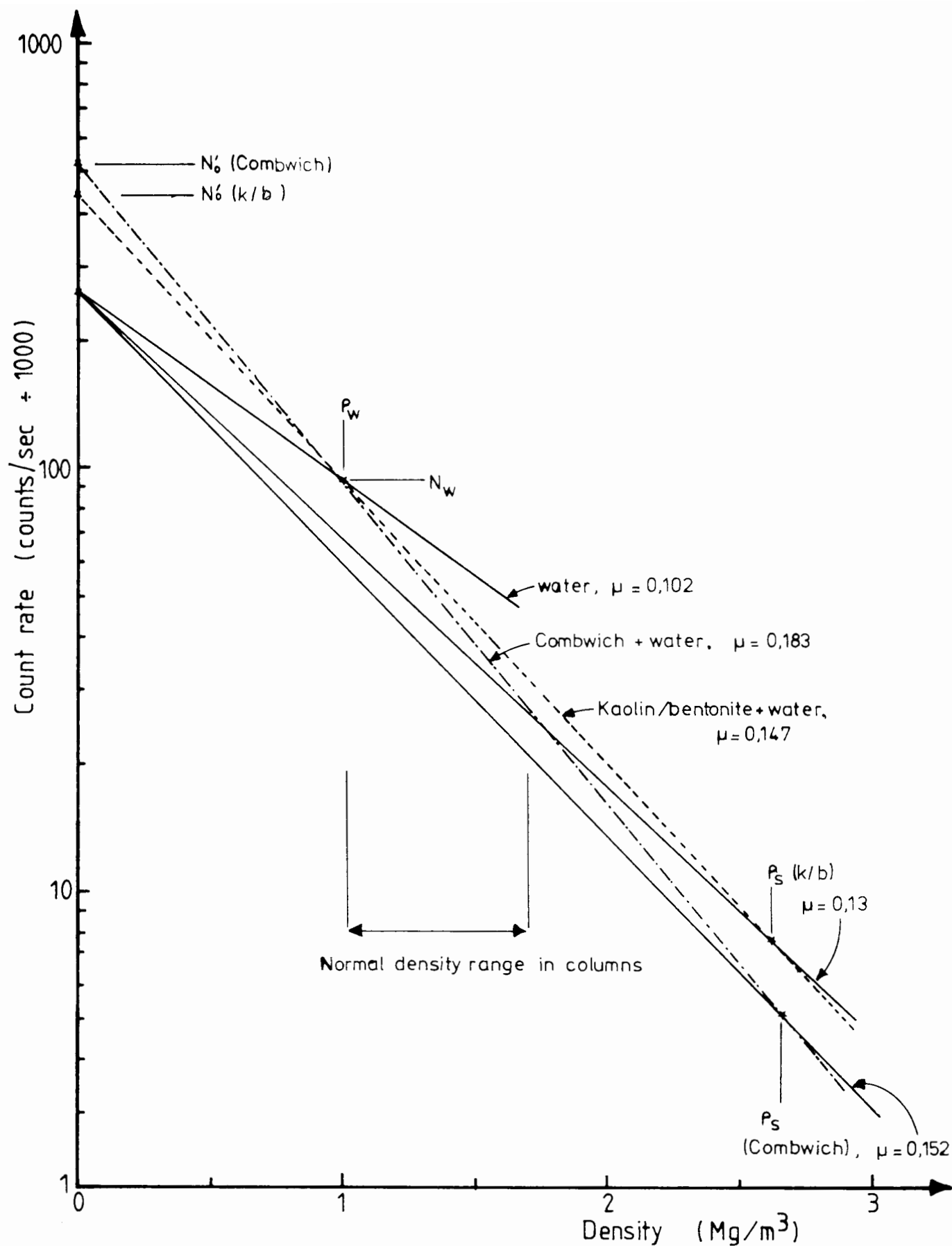


Figure 3.4 True and apparent attenuation lines for water, kaolin/bentonite, Combwich, and mixtures.

from this analysis and figure 3.4 is the advantage in using the water count rate, N'_w as a calibration parameter rather than N'_0 , the apparent incident count rate. Both procedures should give the same results but greater errors occur using N'_0 since additional calculations are required to obtain this artificial parameter.

3.3.3 Summary of the X-ray technique

Theoretical and practical considerations concerning the use of an X-ray attenuation technique to measure soil bulk density have been discussed here and are evaluated fully elsewhere (Elder, 1985). Three general conclusions may be drawn.

1. Samples of known density are not required for calibration providing the initial mean density or base total stress is known accurately. Use of calibration samples is, in fact, the least accurate method of density calculation.
2. Material attenuation properties are highly dependent on tube current and potential difference at the radiation energies used. Both should be set to standard values to obtain maximum accuracy and repeatability between tests.
3. Dimensional and geometric inhomogeneity in columns caused large attenuation changes. The procedure described should be used to eliminate these effects without need for separate analysis.

Use of the procedures recommended in this section to reduce errors allow the average bulk density at a horizontal column section to be calculated to within $\pm 0.005 \text{ Mg/m}^3$ at densities near that of water and to within $\pm 0.01 \text{ Mg/m}^3$ or better at densities near 1.5 Mg/m^3 . At this higher value of density, the void ratio (2.33) is accurate to within ± 0.07 or $\pm 3\%$. Spatial accuracy for a column traversed in less than 5 minutes is $\pm 0.5 \text{ mm}$ at any point. These values were considered very satisfactory.

3.4 Strength testing

To evaluate relative importance of the factors discussed in chapter 2 four different strength testing methods were used. Triaxial tests were conducted on samples using special segments at the bases of two columns, but of necessity at higher stresses than those in the columns. These were complemented by conventional CU tests on reconstituted samples. Shear vane and fall cone tests were carried out at 100 mm intervals down each column as segmented sections and overlying soil were removed. The fourth method involved expansion of a fluid filled cavity, nominally spherical, from near zero radius, at the tip of a hypodermic needle inserted through the column wall into the soil in its in situ state. Use of this technique has not been reported elsewhere and analyses of techniques or relationships between results from different techniques are not standardised even for stronger soils, so that a further part of the experimental programme involved making comparisons between these tests for a range of conditions. These techniques used in a settling column are shown schematically in figure 3.5.

Effects of stress relief and disturbance

Sample disturbance before testing was minimised by carrying out strength tests in situ in the columns, so that stress relief during unloading and local disturbance due to instrument insertion only need be considered. Few data have been reported regarding these effects at low stress states. Drnevich and Massarsch (1978) simulated such effects in a resonant column apparatus and found that the initial tangent shear modulus derived at very low strains varied with effective and total stress changes during swelling. Kirkpatrick and Khan, in the second of two companion reports (1984) measured undrained strengths in oedometer consolidated samples, which had been unloaded and trimmed. Using a 12.7 x 12.7 mm vane, they found strength decreases in illite samples around 32% one day after removal from the cell and 38% decrease after 7 days, while UU triaxial tests gave reductions of 20%, 40% and 45% after one hour, one day and seven days respectively. The short drainage paths in these tests (and removal

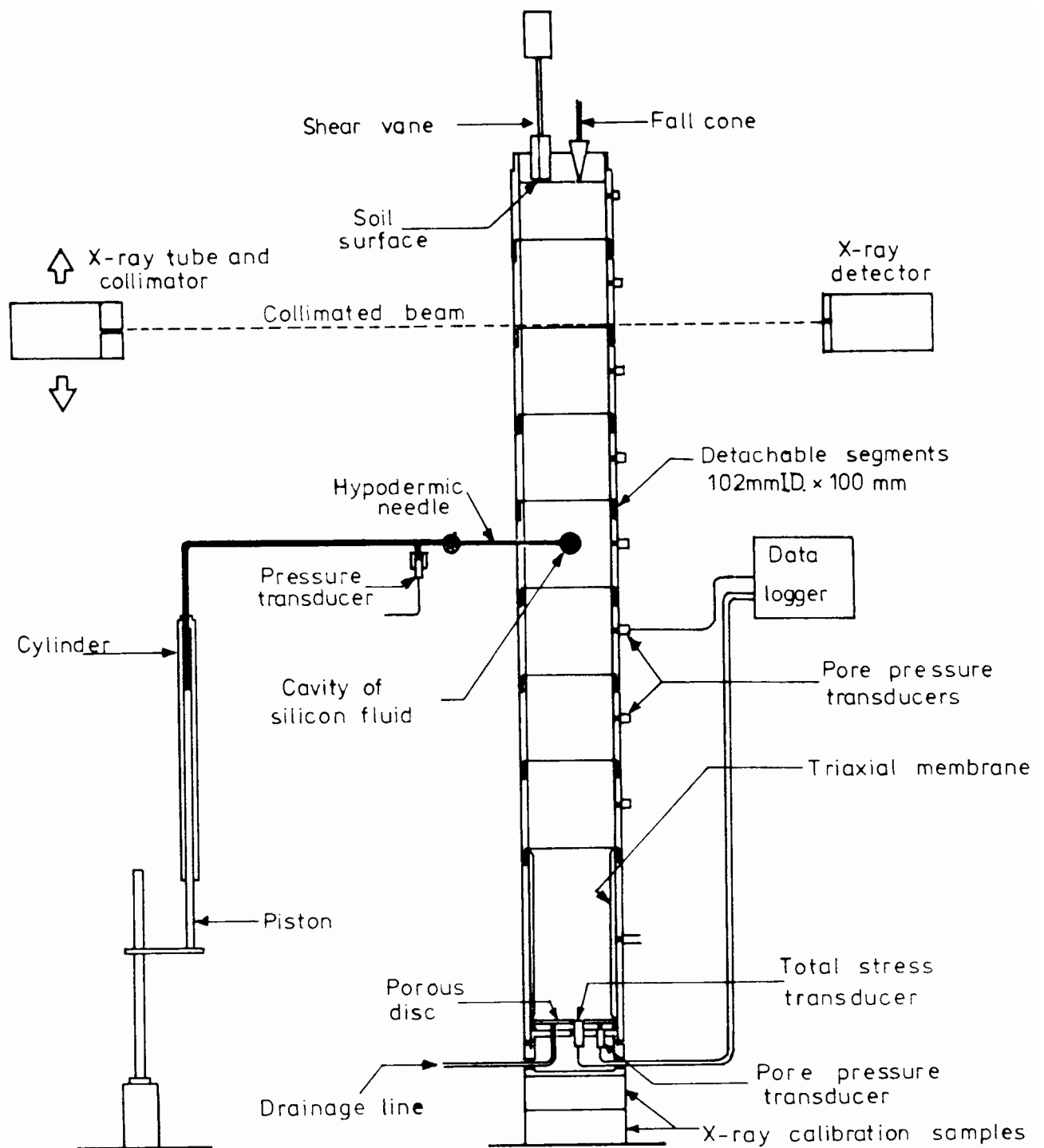


Figure 3.5 Schematic of test apparatus (not to scale).

of lateral restraints) suggest effective stress decreases would be much faster than in the settling columns. Mesri et al (1978) measured rates of swelling for oedometer samples of maximum height less than 25 mm. These rates were sufficiently slow at the high pore pressure gradients involved that negligible effective stress changes might be expected during the time of testing in settling columns, where gradients are very low.

Once soil in any column segment had been tested, samples were taken for density analysis and remaining soil removed to a level 5 mm above the next joint, except at the walls where a ring of soil was removed to expose the joint. The upper segment was removed and soil levelled to the top of the lower segment using a cheese wire and pal ette knife. It was estimated that the exposed surface (which was kept wet) had been disturbed to, at most, a depth less than 1 mm.

3.5 Fall cone testing

Use of the fall cone apparatus to determine the liquid limit of soils has been discussed already. Several analyses are possible to enable calculation of undrained strength, s_u , from known penetration, d , of a given cone of weight w and apex angle 2α . Simple dimensional analysis of the static condition at full penetration (Wroth and Wood, 1978; Houlsby, 1982) leads to an expression similar to that used by Hansbo (1957) for the load on the cone

$$\frac{s_u d^2}{P'} = k(\alpha, \frac{a}{s_u}) \quad (3.5.1)$$

$$P' = W - \frac{1}{3}\pi\gamma h^3 \tan^2\alpha = W - W_b \quad (3.5.2)$$

where a is the soil-cone adhesion, determined by the surface roughness of the cone and P' is the modified cone force excluding the hydrostatic uplift (buoyancy) given by the second term in equation 3.5.2. Since very light cones are required to measure low strengths the buoyancy force is not negligible compared to the cone weight. For example, a 30° 15 g cone embedded to a depth 20 mm in a soil of density 1.5 Mg/m^3 (void ratio ≈ 2.3) has about 6% of its weight carried by this force; a 60° 10 g cone at 15 mm depth in the soil has

its effective weight reduced by 18%. Determination of the cone factor k has often been carried out by comparison of cone penetrations with s_u determined from other strength tests, particularly the shear vane. In this way dependence on roughness (a/s_u) is implicit and values are obtained for different cone angles. Some of these values are summarised in table 3.2. Error ranges are estimated from authors' data. Numbers in brackets indicate ranges of penetration values from which constants were obtained, where given. An * indicates that the value was obtained from the value for the other apex angle using the $k_{60}:k_{30}$ ratio determined by Hansbo.

Reference	Calibration method	k_{30°	k_{60°
Hansbo, 1957 s_u field	a. Piston sampler	1.0 ± 0.1	0.24 ± 0.04
	vane, fall cone in lab	(5-16mm)	(*)
	b. Foil sampler	0.8 ± 0.08	0.20 ± 0.03
		(5-9mm)	(*)
	s_u remoulded with lab vane $k_{60}/k_{30} = 0.29-0.44$ (1-7mm)(2-18mm)	1.2 ± 0.2 (*)	0.30 ± 0.05 (4.5-11mm)
Karlsson, 1961	s_u with 3 lab vanes H:D = 2.1 $k_{60}/k_{30} = 0.29-0.44$	$0.70-0.86$	$0.25-0.35$
Wood, 1985	s_u with 3 bladed rhomboidal section vane $k_{60}/k_{30} = 0.33-0.38$	0.85 ± 0.05 (7-30mm)	0.29 ± 0.05 (4-19mm)

Table 3.2 Variations in cone factors reported by different authors.

It can be seen that a wide range of values have been reported for the cone factors for both 30° and 60° cones. Karlsson shows trends with plasticity index for each factor, while Wood observes this for his 60° cone results only, although with fewer data. The variations in k_{30} in table 3.2 imply variation in the fall cone strength at the liquid limit (30° , 80g, 20mm) between 1.4 and

2.7 kN/m² which seems rather a large range for such a test. Houlsby (1982) solved the equilibrium and yield equations, using the method of characteristics, to determine theoretical values of the cone factors under different conditions. He found that small variations in apex angle or tip bluntness had minimal effects but that failure to account for heave around the cone caused k_{30} to be about 9% too high. The major influence however was the surface roughness which caused k_{30} to decrease by almost 50% and k_{60} by over 30% when a/c_u increased from 0 (smooth) to 1 (rough).

Hansbo also carried out a dynamic analysis of the fall cone test, making a simplifying assumption for the total force on the cone, P , that

$$P = W - \frac{s_u z^2}{k} \quad (3.5.3)$$

where the second term is the equivalent static resistance that would be felt at a penetration z . He showed that this quasi-static approach provided excellent agreement with experimental observations of the cone motion during penetration. For low strengths it is again appropriate to replace P by P' (equation 3.5.2) so that the equation of motion becomes

$$m\ddot{z} = m\frac{dv}{dz} = W - \frac{1}{3}\pi\gamma z^3 \tan^2\alpha - s_u z^2/k \quad (3.5.4)$$

Integrating and evaluating the constant (= zero) when $v=0$ at $z=0$ gives an expression for the full penetration under dynamic conditions, d_d

$$P'' = W - W_b/4 = \frac{s_u}{k} \left(\frac{d_d}{\sqrt{3}}\right)^2 \quad (3.5.5)$$

This may be compared to equation 3.5.1 obtained from static analysis. In a soil of undrained strength s_u , a given cone which penetrates to depth d_d under dynamic conditions will penetrate to a depth

$$d_s = \frac{(W-W_b)}{3(W-W_b/4)} \approx d_d/\sqrt{3} \quad (3.5.6)$$

if static conditions occur (i.e. slow lowering). The buoyant force has a much smaller effect under dynamic conditions.

Houlsby's computed values of k_{30} for different roughnesses are used together with equations 3.5.1 and 3.5.5 to compute values of undrained strength at the liquid limit in table 3.3, for the static and dynamic analyses. Buoyancy effect is ignored.

a/s_u	0	0.2	0.4	0.6	0.8	1
Heave { static	1.89	1.58	1.36	1.22	1.09	1.00
ignored dynamic	5.67	4.74	4.08	3.66	3.27	3.00
Heave { static	1.75	1.45	1.24	1.10	1.00	0.92
included dynamic	5.24	4.34	3.73	3.29	2.99	2.75

Table 3.3 Undrained strength at liquid limit implied by different analyses (after Houlsby, 1982). Strengths in kN/m².

Values calculated using the dynamic analyses do not fall within the range of results observed in the previous section despite the good correlations with observed motion of the cones obtained by Hansbo. It may be worth noting that if a different cone constant should apply to the dynamic analysis from that appropriate to static conditions then the dynamic analysis might still be valid; comparison of equations 3.5.1 and 3.5.5 suggests that the dynamic constant would have to be lower by a factor of 2 to 3 to yield similar strengths to those observed at the liquid limit. It is possible that the initial assumptions in the dynamic analysis also require modification, in particular the approximation that constant a/s_u is mobilised along the cone surface.

3.5.1 A modified dynamic analysis

A simple example of a modification to the dynamic analysis is discussed in this section which predicts cone motion during penetration consistent with that observed by Hansbo but can also be used to predict shear strengths at the liquid limit (using Houlsby's static cone factors) which are nearer to those reported earlier.

Previous analysis assumed that s_u and the cone factor k remained constant during penetration so that the force exerted by the soil on the cone, excluding buoyancy, was given by $s_u z^2/k$ as in equations 3.5.3 and 3.5.4. It is assumed here that s_u/k is not constant, but instead varies linearly with the penetration, z , from some initial value to a final value s_{us}/k_s , the static

value, when full penetration is obtained at $z = d_d$. This variation may be written as

$$s_u/k = -\frac{s_u}{k_s} \left[1 + C \left(1 - \frac{z}{d_d} \right) \right] \quad (3.5.7)$$

where $C = \left[\left(\frac{s_u}{k} \right)_{z=0} - \left(\frac{s_u}{k_s} \right)_{z=d_d} \right] / \left(\frac{s_u}{k_s} \right)$ (3.5.8)

so that a positive value of C denotes a decrease in s_u/k during penetration. This might occur if s_u increased with strain rate; at initial cone entry into the soil strain rates are very high at all points adjacent to the cone whereas at larger penetrations highest strain rates occur near the tip and the average rate will have decreased, so that an average value of s_u throughout the yielding zone will decrease. A decrease in s_u/k during penetration might similarly be caused by an increase in k . Houlsby showed that $k(= 1/F)$ increases as the roughness coefficient a/c_u decreases, which might result at larger penetrations as soil near the cone surface becomes preferentially orientated to reduce the adhesion. Although both these mechanisms are speculative only they might be predicted from the considerations of fabric and effect of strain rates discussed in chapter 2. The linear model represents only a simple attempt to approximate these phenomena in the absence of experimental data. The equation of motion becomes

$$mv \frac{dv}{dz} = W - W_b - \frac{s_u}{k_s} z^2 \left[1 + C \left(1 - \frac{z}{d_d} \right) \right] \quad (3.5.9)$$

Integrating, with constant equal to zero from initial conditions, gives

$$\frac{1}{2}mv^2 = (W - W_b/4)z - \frac{s_u}{k_s} z^3 \left[\frac{1}{3}(1+C) - \frac{1}{4}C \frac{z}{d_d} \right] \quad (3.5.10)$$

The maximum penetration is obtained when $v=0$ and $z=d_d$ as

$$W - W_b/4 = -\frac{s_u}{k_s} d_d^2 \left(\frac{1}{3} + \frac{C}{12} \right) \quad (3.5.11)$$

which reduces to equation 3.5.5 when $C=0$ and s_u/k is constant at the static value. The static penetration, d_s , is related to the dynamic penetration by

$$d_s = d_d \cdot \left(\frac{1}{3} + \frac{C}{12} \right) \frac{(W - W_b)}{(W - W_b/4)} \quad (3.5.12)$$

It remains to investigate how reasonable variations in C (or s_u/k) will affect the calculated undrained strength s_{u0} at full penetration and then to check that cone motion for these values of C is consistent with experimental observations. It is assumed that s_{u0} rather than some higher value of s_u is appropriate for comparison with other data since the cone test causes soil deformations which are very rapid (up to 500 mm/s) compared to those in other strength tests. The same values of the static analysis cone factor k_s obtained by Houlsby and used to calculate strengths in table 3.3 are used to find undrained strengths at the fall cone liquid limit, for various values of C , in table 3.4. Hydrostatic uplift is ignored and the factors used are those including heave calculations. It is seen that values of $C \geq 2$ are sufficient to predict strengths close to those reported earlier in this chapter. At $C=2$ the value of s_u/k at initial cone entry is three times that at full penetration. The effects of these values on the cone motion are shown in figure 3.6, and it can be seen that the motion of a cone assuming $C=2$ is consistent with Hansbo's observations although $C=4$ leads to higher velocities than in Hansbo's tests. It may be concluded from this simple analysis that variation in s_u , surface adhesion, or other average values during penetration of the cone can account for high values of strength inferred from analyses where these effects are not considered. The linear variation of s_u/k assumed here is likely to be an oversimplification of behaviour but may be adequate as a first approximation if experimental data can be obtained supporting these variations. This behaviour is not considered further in this study.

3.5.2 Use of fall cones

The fall cone apparatus used in this study was modified slightly from the standard laboratory liquid limit apparatus by extending the support arm to allow greater horizontal and vertical travel. Measurements by Hansbo and theoretical analysis by Houlsby suggest that undrained penetration is completed in less than 0.1 seconds, whereas the standard apparatus allows a 5 second penetration time. During early tests it was observed that following an initial

a/s_{us}	Houlsby, 1982 k_s	Undrained strength at liquid limit (kN/m^2)					
		$C = 0$	1	2	3	4	5
0	0.890	5.24	4.19	3.49	2.99	2.62	2.33
.2	0.738	4.34	3.48	2.90	2.48	2.17	1.93
.4	0.634	3.73	2.99	2.49	2.13	1.87	1.66
.6	0.560	3.29	2.64	2.20	1.88	1.65	1.47
.8	0.508	2.99	2.39	1.99	1.71	1.50	1.33
1.0	0.467	2.75	2.20	1.83	1.57	1.37	1.22
d_s/d_d		0.577	0.65	0.71	0.76	0.82	0.87
z/d_d at $v = v_{max}$		0.577	0.533	0.500	0.476	0.459	0.446

Table 3.4 Variations in undrained strength (at the end of penetration) at liquid limit from modified dynamic analysis.

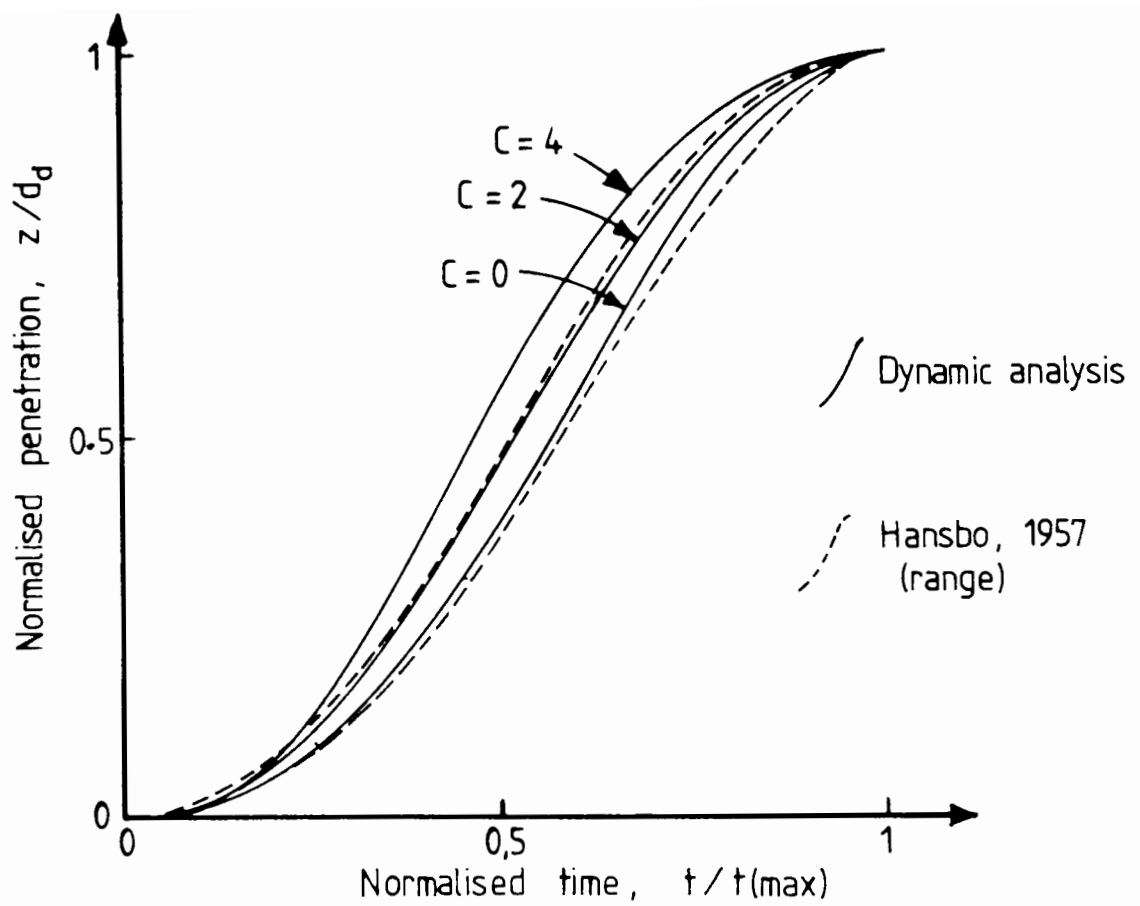


Figure 3.6 Cone motion for S_u/k decreasing during penetration.

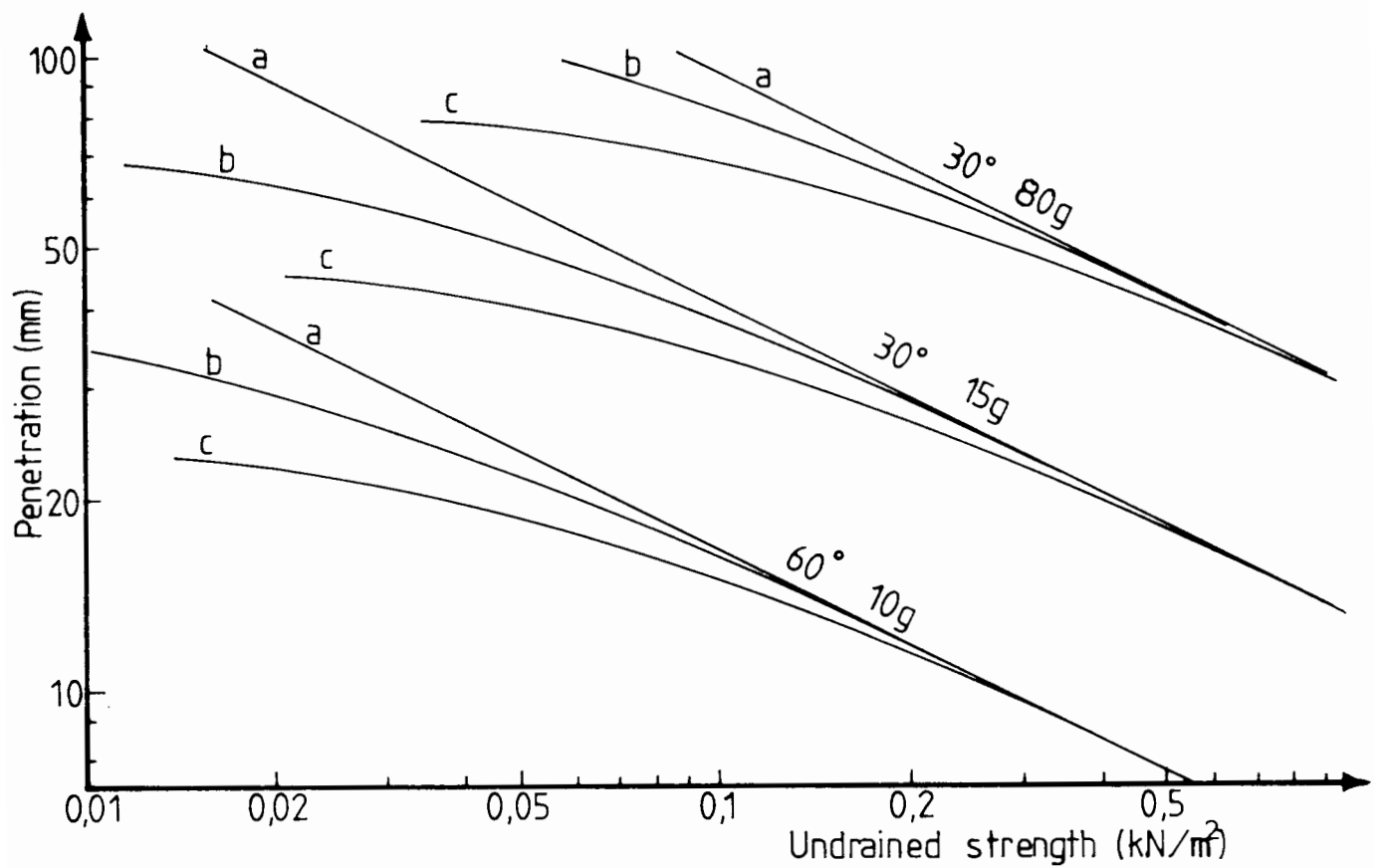


Figure 3.7 Calibration plots for cone strength from penetration for three cones, using different analyses, and assuming $S_{uLL} = 2.2 \text{ kN/m}^2$.

rapid penetration the cone remained stationary for about 1 second then penetrated to a greater depth (at a slow rate) until clamped after 5 seconds. These observations, although visual only, suggest that consolidation or undrained creep occurs in the soil around the cone following undrained penetration; the magnitude of this was estimated to be up to 5% of initial penetration. Since this effect was greatest at higher fluid contents, the electronic timer was reset to allow a free fall time of 0.5 seconds only. Scatter in penetration data was also reduced considerably once this alteration had been made. A positive recommendation is made on the basis of existing evidence that the liquid limit standard procedure should be altered similarly. An extensive study of cone motion throughout the full 5 second period for a wide range of clay soils would be useful in supporting this recommendation.

To allow strength measurement over the range expected four cone types were used, constructed from three materials for assessment of surface adhesion effects:

30° 80 g	} steel, 'perspex' acrylic, PTFE, machined and polished to apex angle $\pm 0.1^\circ$, mass ± 0.01 g, with bluntness limited according to BS1377.
30° 60 g	
30° 15 g	
60° 10 g	

Penetration was measured to ± 0.01 mm by dial gauge but final accuracy was about ± 0.1 mm due to errors in placement of the cone apex at the soil surface.

Between each test a cone was washed, dried and oiled lightly. Houlsby suggested a disturbed region around the cone extended to a radius at the soil surface similar to the penetration depth. At average penetrations this allowed five independent tests at any section in the 102 mm ID columns.

Equations 3.5.1 and 11 may be used to provide equations for the (static) undrained strength, s_u , as a function of penetration depth d , for the analyses described previously.

$$\text{Static analysis: } s_u = k_s \left(\frac{W - W_b}{d^2} \right) \quad (3.5.13)$$

$$\text{Dynamic analysis: } s_u = k' \left(\frac{W - W_b/4}{d^2} \right), \quad k' = k_s / \left(\frac{1}{3} + \frac{C}{12} \right) \quad (3.5.14)$$

where k' is assumed constant for a particular cone and soil. Since uncertainty exists regarding values of k_s and k' , values for this study are calculated by comparing cone penetrations and vane strengths for remoulded soils. Using this method k_s and k' will receive similar values (the same values if buoyant effect is ignored, $W_b=0$). It is realised that these values cannot both be correct; nevertheless it is worth investigating whether it is important which analysis is used providing k_s and k' are chosen in this way. To do this, an example is used where $s_{uLL}=2.2$ kN/m², giving $k_s=1.134$ and $k'=1.125$ assuming a liquid limit close to 80 (k_s and k' are not sensitive to this). If buoyancy is ignored, $k_s=k'=1.121$, which is in error by 1%. Figure 3.7 shows undrained strengths predicted from penetrations using these analyses for three cones. Values for the 60° cone assume $k_{60}/k_{30}=0.245$ as given by Hansbo. For the 30° 80 g standard cone, the static and dynamic analyses do not diverge significantly at all likely penetrations and even at $d=40$ mm the difference between strengths is less than 10%. With the 30° 15 g cone 10% deviations occur at around 25 mm penetration and for the 60° 10 g cone at around 13 mm. These values define reasonable upper limits on penetration due to the uncertainty regarding validity of respective analyses. Lower limits also apply where the maximum likely error in penetration measurement (± 0.1 mm) causes an unacceptable error in strength. Applying a $\pm 5\%$ criterion for strength accuracy gives lower penetration limits of 4 mm for all cones. The ranges of undrained strength which may be measured accurately by each cone are therefore as follows:

60° 10 g	$0.14 < s_u < 1.7$ kN/m ²
30° 15 g	$0.24 < s_u < 10.3$ kN/m ²
30° 80 g	$0.4 < s_u < 55$ kN/m ²

where errors up to 10% may result at the lower end of each range due to analysis uncertainty and errors up to 5% at the top of each range from measurement errors. The need for further studies of cone behaviour to validate one or other analysis is highlighted by the small range of suitability of the 60° cone.

3.6 Shear vane testing

The vane shear test is used widely in situ and in the laboratory to give a rapid and reliable estimate of the undrained strength. Many authors have attempted to analyse the test experimentally and theoretically and the degree of variability observed when results are compared to undrained strengths measured in other tests has caused the vane test to be subjected to considerable criticism (e.g. Schmertmann, 1975). A full report of these findings or further analysis are not attempted here but some factors particularly relevant to this investigation are discussed. Factors known to affect interpretation of results include:

1. The failure surface is predetermined, nominally a vertical cylinder although the precise shape may vary. Wilson (1963) observed that the initial surface was almost square at low rotation angles and Rush (1974) observed prefailure strains consistent with this. Roy (1975) reported that rotations in excess of 45° were required to develop a cylindrical failure surface. Skempton (1948) and Arman (1975) suggested the cylindrical diameter was greater than that of the vane blades by about 5%.
2. The shear stress distributions on the horizontal and vertical failure surfaces are not known accurately even for isotropic soil states. Menzies and Merrifield (1980) found that the shear stress on the vertical surface was approximately constant away from the ends but that in clays the shear stress on the horizontal surfaces was highly non linear and very small near the axis. Results agreed with those predicted by Donald et al (1977) from an elastic 3 dimensional finite element analysis and Wroth (1984) suggested a power law relationship to describe this variation by:

$$\tau = \tau_{\max} \left(\frac{r}{D/2} \right)^n \quad n \approx 5 \quad (3.6.1)$$

at radius r from the axis of a vane of diameter D and height H . Assuming $\tau_{\max}$ is fully mobilised on the vertical surface $\tau_{\max}$ may be obtained in terms of the total torque exerted, T , as

$$\tau_{\max} = 2T / \left[\pi D^2 \left(H + \frac{D}{n+3} \right) \right] \quad (3.6.2)$$

As shown by Wroth, for a H:D ratio of 2 and for $n=5$ the resistance measured is almost entirely due to the shear stress on the vertical plane which contributes 94% of the total torque. This is consistent with vane tests carried out on a sample in a triaxial cell by Law (1979) who showed that resisting torque was insensitive to changes in vertical effective stress but increased with increasing horizontal effective stress. Values of n in excess of 1 (triangular τ distribution) have not otherwise been used previously; $n=1$ (e.g. Aas, 1965) implies the vertical surface shear stress contributes only 89% of the total torque.

3. Anisotropy of strength. This has been evaluated using vanes with different H:D ratios or using diamond shaped blades (Aas, 1965; Richardson et al, 1975; Menzies and Mailey, 1976). In addition to the uncertainty regarding stress distribution on each plane, problems arise with progressive failure where peak strengths are mobilised at different rotations on different planes (Wiesel, 1973). Donald et al concluded that the strength on horizontal planes could not be estimated reliably by any current method under most conditions.

4. Rotation rate. Cadling and Odenstad (1950) found a 20% increase in strength for rotation rate increased from 6° to $60^\circ/\text{min}$ and other authors have found similar or greater variations (Flaate, 1966; Perlow and Richards, 1977; Karlsson, 1961) although Aas (1967) did not observe any rate effects for a similar range. Kirkpatrick and Khan (1981) reported that vane strength increased faster than triaxial strength with decreasing time to failure, which also caused a greater rotation angle to be attained at peak strength.

5. Increase in strength with a delay between insertion and testing was recorded by Rush (1974) with a 20% rise after about one day which then remained nearly constant. This effect is probably due to consolidation around the vane and strength regain following disturbance.

6. Principal stress rotation. The direct stress acting in the horizontal direction before shearing is the minor principal stress (in a normally consolidated soil) but lies between σ'_{1f} and σ'_{3f} at failure. Wroth (1984)

postulated that the radial and circumferential direct effective stresses might remain constant during shearing so that the maximum shear stress on the cylindrical surface at failure would be equal to the Mohr's circle radius, (figure 3.8a) i.e. the common definition of the undrained strength. This would require the actual failure plane to be inclined at an angle $\phi'/2$ to the cylindrical surface. An alternative mechanism might allow the circumferential direct stress to be greater than the radial direct stress at failure so that (σ'_{rf}, τ_f) corresponded to the point on the Mohr circle in contact with the failure envelope. The undrained strength, $s_u = (\sigma'_{1f} - \sigma'_{3f})/2$ would then be equal to $\tau_f \sec \phi'$ (figure 3.8b). If ϕ' varied with plasticity index as suggested by Voight (1973) and Kanji (1974) the ratio $s_u/\tau_{f(vane)}$ would also vary as shown in figure 3.9. The range of data obtained by Bjerrum (1972) and extended by Ladd (1975) from back analysis of failed embankments and probably encompassing all the factors described earlier is shown for comparison.

Use of the vane shear test

Despite the many sources of variability described above, the vane shear test remains the most widely used in soils which are too soft to sample for triaxial testing. It has until recently been the only instrument in widespread use for in situ sea-bed investigations and is therefore the source of most data relating undrained strength to liquidity index (figure 2.2). Under laboratory conditions, effects of various factors can be investigated as part of the testing programme. The vane shear test was therefore used in this series of experiments as a major link between the results from this study and those reported elsewhere.

A standard laboratory shear vane machine was adapted in several ways. Two vanes were constructed from "dural" sheet with rectangular blades and H:D ratios of 2:1, one 40 x 20 mm, the other 25.4 x 12.7 mm. Area ratios (vane cross-sectional area: circular cross-sectional area) were reduced below those for standard vanes by using finer blades and a smaller shaft diameter, to reduce insertion disturbance. The torque measuring spring was replaced by a

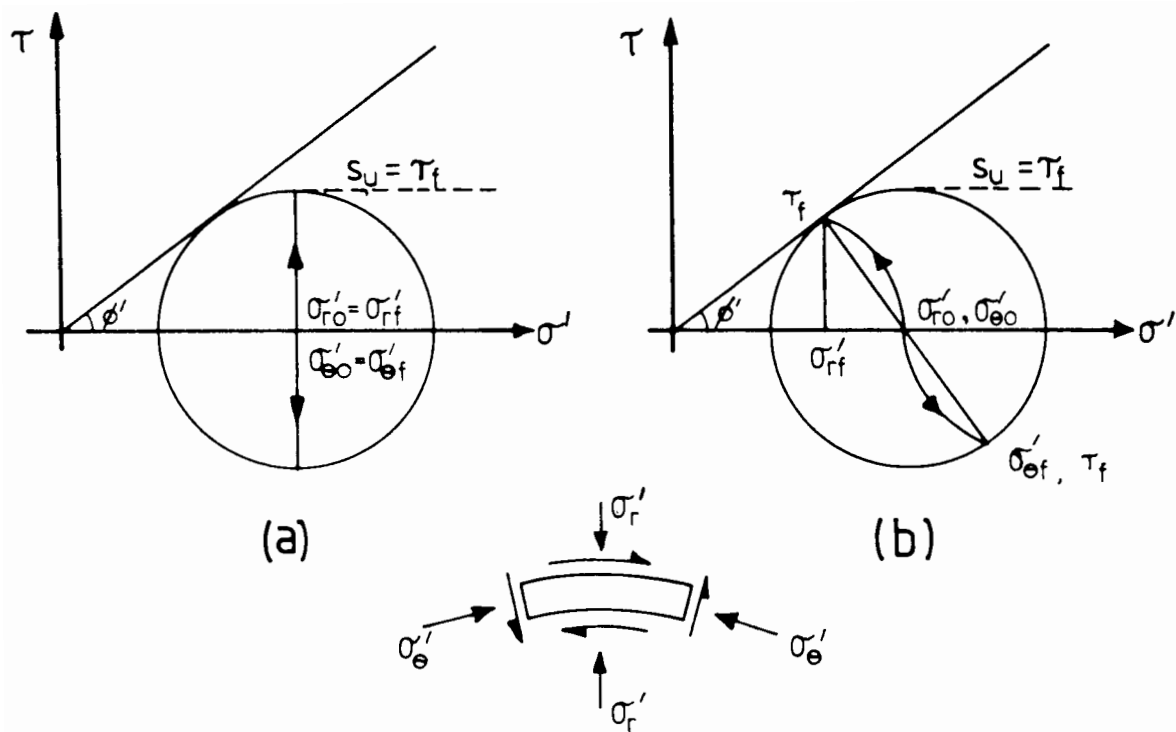


Figure 3.8 Stresses acting on vertical planes during vane shear test.

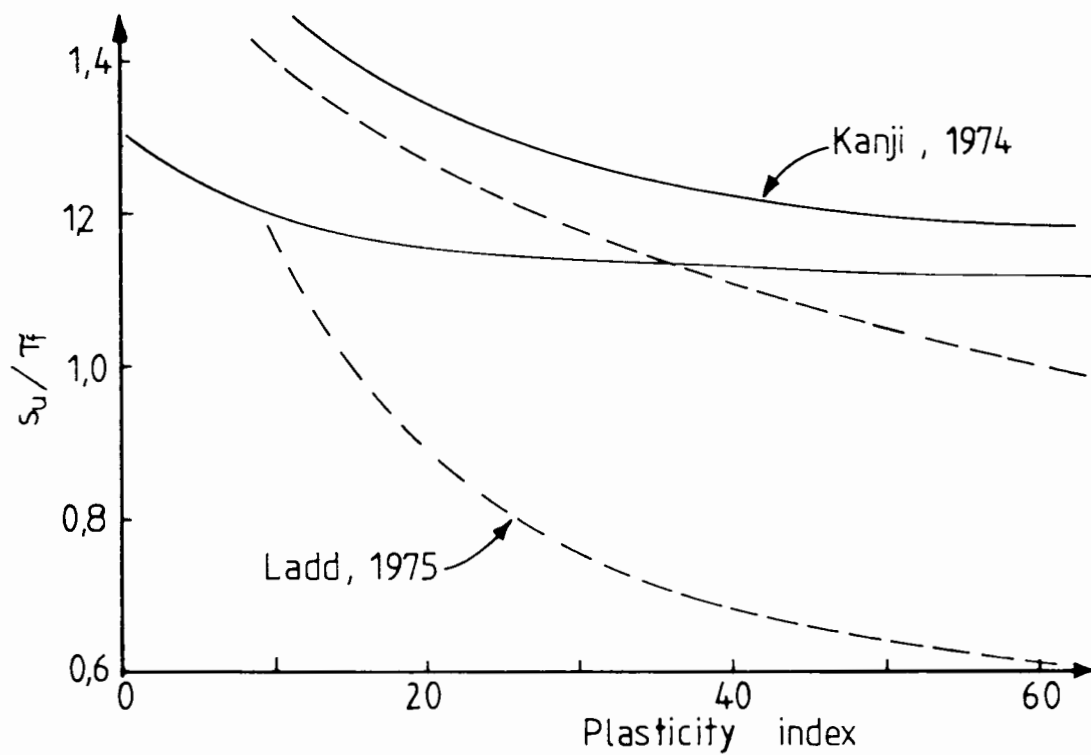


Figure 3.9 Variation in ratio of undrained strength to maximum vane shear stress on plane of failure (mechanism b).

rigid transducer consisting of a hollow brass shaft machined over a necked middle region to 3.00 ± 0.01 mm OD with 0.15 ± 0.01 mm wall thickness. A "Kyowa" (KFC-2-D2-23) four arm balanced strain gauge bridge measured torque transmitted through the transducer independent of axial stress, and a dummy gauge for temperature compensation was mounted nearby. The output from this system (mV) was directly proportional to the applied torque, and calibration was carried out prior to testing in which different load/unloading ranges, hysteresis, direction of loading, rate of loading, backlash on reversal, zero torque variation with angular rotation and repeatability were all evaluated. The available rotation was limited to 280° in either direction by the wires connected to the strain gauge although 'rotation remoulding' could be achieved by reversing rotation direction or by turning the vane by hand. Assuming validity of equation 3.6.2 for the undrained strength

$$s_u = \tau_{\max} = 2T / \left(\pi D^2 \left(H + \frac{D}{n+3} \right) \right)$$

values of s_u accurate to ± 0.001 kN/m² or better could be obtained with this system. Accurate rotation rate control was achieved by replacing the DC drive motor by a stepping motor which, after gearing, required 91 steps per degree vane rotation. Full preset control was possible at rates continuous between 0 and $600^\circ/\text{minute}$. Since rate was independent of load the rotation was calculated directly as a linear function of the time using a digital step counter or from chart records. Continuous torque-rotation data were recorded for all tests. Monney (1971) reported that tests in marine soils were usually at rates from 6° to $60^\circ/\text{min}$; rates from 4° to $100^\circ/\text{min}$ were investigated in this study.

Once fall cone testing was complete at any section the vane was inserted immediately until covered by a depth equal to the blade height. Stevenson (1973) suggested boundary confinement effects were negligible if the container radius R_c exceeded three times the vane radius R and even for the larger vane this was easily satisfied ($R_c/R > 5$). Pore pressure changes at the column wall were measured continuously during testing and found to be small. Test

procedures enabled estimation of sensitivity, thixotropic regain behaviour and torque relaxation after completion of rotation.

Undrained shear modulus

Since shear strains around the vane are not measured it is not possible to calculate the undrained shear modulus, G , directly without making some simplifying assumptions. Cadling and Odenstad (1950) assumed a homogeneous, isotropic, infinite soil with linear elastic stress-strain properties and considered an infinitely long vane (and cylindrical shear surface of maximum stress) to obtain an expression for the shear modulus at an angle of rotation θ

$$G = \tau / 2\theta \quad (3.6.3)$$

They obtained a coefficient of 3 in the similar equation for a spherical surface and estimated that a value of 2.25 might apply for a finite vane. In a coaxial container of finite radius R_c , the coefficient should be modified to $2/(1 - R^2/R_c^2)$ for an infinite vane (Stevenson, 1973) which for the columns used in this study represents a difference of only 4% from the value for an infinite soil. In light of the results reported already showing the minimal influence of the shear at the end surfaces, the shear stress may be related directly to the torque using

$$T \approx T_v = \frac{1}{2} \pi D^2 H \tau \quad (3.6.4)$$

where T_v is the torque due to shear on the vertical cylindrical surface alone. Combining (3.6.3) and (3.6.4) allows an average or secant modulus to be calculated directly from the torque-rotation curve as

$$G = \left(\frac{1}{\pi D^2 H} \right) \frac{T}{\theta} \approx \frac{1}{6.3 D^3} \frac{T}{\theta} \quad \text{if } \frac{H}{D} = 2 \quad (3.6.5)$$

Madhav and Krishna (1977) made the same assumptions about the soil properties and considered normal pressures varying with distance from the axis of smooth vane blades. For undrained conditions where $G=E/3$ their result may be written

$$G = \frac{1}{3 I_\theta D^3} \frac{T}{\theta} \approx \frac{1}{5.1 D^3} \frac{T}{\theta}, \text{ this study} \quad (3.6.6)$$

where I_θ is a function of H/D , H_0/D (H_0 = soil depth above vane) and ν (Poisson's ratio), and was calculated by mesh and strip analysis. For $H/D=2$, $H_0/D=2$ and $\nu=0.5$, a value of $I_\theta \approx 1.7$ is obtained giving the second value in

(3.6.6) , which is seen to be similar to that in equation (3.6.5) despite the different method of analysis.

The ratio of undrained shear modulus : undrained strength is often useful and can be estimated from the elastic analyses at any rotation as

$$\frac{G}{\tau} = \frac{1}{2\theta} , \quad \text{Cadling \& Odenstad} \quad (3.6.7a)$$

$$= \frac{1}{1.6\theta} \quad \text{Madhav \& Krishna} \quad (3.6.7b)$$

A more general soil model might assume viscoelastic properties. Stevenson showed that the viscoelastic shear modulus form:

$$G(t) = G_1 t^{-n} \quad (3.6.8)$$

was valid for some marine sediments, where G_1 and n could be obtained from vane test results. Equating the time averaged shear modulus G obtained from equation (3.6.5) for a period of rotation up to angle θ at torque T , to the value obtained by integration of equation 8 gives

$$\left(\frac{1}{\pi D^2 H}\right) \frac{T}{\theta} = \bar{G}(t) = \frac{G_1}{1-n} t^{-n} \quad (3.6.9)$$

A plot of $\log \bar{G}$ v $\log t$ for constant angle θ from different tests at different rates enables G_1 and n to be calculated. Stevenson found that these plots were linear (justifying use of equation 3.6.8) and that n was nearly constant for different rotation rates in the same soil, while G_1 decreased with angle of rotation during shear, indicating sediment non-linearity. An extrapolated value of $G_1(\theta=0)$ correlated well with liquidity index, as did the value of the exponent n .

These analyses suggest that despite uncertainty regarding basic interpretation of the vane test, if good quality torque-rotation data are obtained some reasonable estimates may be made of soil behavioural properties.

3.7 Cavity Expansion testing

The fall cone and shear vane techniques are straightforward and relatively simple to interpret but both suffer from the major disadvantage the testing cannot be carried out until overlying soil has been removed. Even if this process causes no direct disturbance to soil below, effective stress changes

during swelling may still cause measurements to differ from those if the soil had been tested in its natural state. Briaud (1980) suggested that development of an in situ technique for expansion of a spherical cavity in soil might provide useful information about stress-strain and strength properties. Apparatus to do this under controlled conditions has not been developed or used previously although the problem has been considered theoretically. Spherical (or hemispherical) cavity analysis is used to model aspects of pile and cone penetration, where expansion occurs from zero initial radius, and cylindrical analysis is used to interpret the pressuremeter test with expansion from a finite radius, determined by borehole size.

3.7.1 Theoretical analysis

Spherical expansion analyses are formulated in spherical co-ordinates r, θ , where symmetry applies and body forces (self weight of soil) are assumed negligible. The equations of equilibrium on a soil element at a distance r from the centre of the cavity reduce to

$$\frac{\partial \sigma_r}{\partial r} + \frac{2}{r}(\sigma_r - \sigma_\theta) = 0 \quad (3.7.1)$$

where σ_r is the radial stress and σ_θ the circumferential stress. A homogeneous and isotropic soil mass is assumed. The cavity is surrounded by a plastic region extending from the current cavity radius R_c (initial value R_{c0}) to a radial distance R_p , beyond which linear elastic behaviour occurs. Solution of the problem requires use of a relationship describing the yield behaviour and for soils the Mohr-Coulomb criterion is suitable

$$\sigma_r - \sigma_\theta = (\sigma_r + \sigma_\theta) \sin \phi + 2c \cos \phi \quad (3.7.2)$$

Bishop, Hill and Mott (1945) solved the case of a frictionless ($\phi=0$) incompressible medium where equation 3.7.2 reduces to the Tresca criterion and Gibson (1950) analysed the cohesionless case ($c=0$) for a purely frictional material, also assuming incompressible conditions. Hill (1950) extended the analysis by Bishop et al to allow for the effects of elastic volume changes in the plastic region. Ladanyi (1963) presented an analysis in which

experimentally determined stress-strain curves could be used to obtain semi-empirical solutions by numerical integration, assuming similitude between triaxial and expansion tests so that behaviour during undrained shear could be described by the deviatoric stress, $q(\epsilon_1)$ and the excess pore pressure $\Delta u(\epsilon_1)$, both as functions of axial strain ϵ_1 .

The most general analysis is that by Vesic (1972) who considers the conditions at an ultimate cavity pressure, p_u , only and uses the Mohr Coulomb criterion. The total volume changes of the elastic and plastic zones are assumed equal to the change in volume of the cavity and the result may be written as

$$p_u \approx (p_0 + c \cot \phi) \frac{3(1+\sin \phi)}{3 - \sin \phi} \left[\frac{G/s_u}{1 + \epsilon_v G/s_u} \right]^{\frac{4 \sin \phi}{3(1+\sin \phi)}} \quad (3.7.3)$$

where the expression is a close approximation for $\epsilon_v < 0.15$ and $0 < \phi < 45^\circ$ and p_0 is the initial uniform stress in the soil, s_u the undrained strength and G the elastic shear modulus. The average volumetric strain ϵ_v has to be determined in some way as a function of stress conditions for some volume change-stress relationship, using an iterative procedure to obtain the suitable value at ultimate conditions. This analysis is not valid for $\phi=0$, where the Bishop et al solution applies.

Davis & Mullenger (1983) used a rate-type constitutive law for the soil employing an elliptical critical state type yield surface and obtained an analytic solution for the radial total stress σ_r under undrained conditions, rewritten here as

$$\sigma_r = p_0 - \sqrt{6} M \int_{\infty}^r \tanh \left[-\sqrt{\frac{2}{3}} \frac{G}{M} \ln \left(1 - \frac{R_c^3 - R_{c0}^3}{R'^3} \right) \right] \frac{dr'}{R'} \quad (3.7.4)$$

where M is the critical state failure stress, $M = \frac{1}{2} M_0 p_0$ and R_{c0} the initial cavity radius. The cavity pressure is obtained as σ_r ($r = R_c$) and numerical integration is required to obtain solutions, from which the undrained strength ($M/2$) may be obtained if G is known.

In this study, undrained (incompressible) conditions are assumed. The undrained soil moduli are related by $G = E/3$ and the cohesion, c , above is

equivalent to the undrained strength s_u . Results are analysed using the simple analysis presented below which follows that by Bishop, Hill and Mott but retains several terms assumed negligible in the original analysis, in order to assess their importance for very soft soils. The earlier assumptions of isotropy and homogeneity apply and the initial total stress state is assumed to be p_0 everywhere in the soil ($r > R_{c0}$). The effect is considered of an increase in cavity pressure from p_0 by an amount Δp , causing a radial displacement $u(r)$ and stress changes $\Delta\sigma_r$, $\Delta\sigma_\theta$ at a radial distance r . Radial and circumferential strains are given by $\epsilon_r = \frac{du}{dr}$ and $\epsilon_\theta = \frac{u}{r}$. The elastic constitutive equations, valid in the elastic region, are (for $\nu = 1/2$)

$$E\epsilon_r = \Delta\sigma_r - \Delta\sigma_\theta, \quad E\epsilon_\theta = -\frac{1}{2}\Delta\sigma_r + \frac{1}{2}\Delta\sigma_\theta \quad (3.7.5)$$

so that $\epsilon_r = -2\epsilon_\theta$ and, integrating for u with $u=0$ at $r=\infty$,

$$u = A/r^2 \quad (3.7.6)$$

Equilibrium requires that

$$\frac{d\sigma_r}{dr} = \frac{2}{r}(\sigma_\theta - \sigma_r) = \frac{2}{r}(\Delta\sigma_\theta - \Delta\sigma_r) \quad (3.7.7)$$

so using (3.7.5) and (3.7.6) and integrating with $\sigma_r = p_0$ at $r=\infty$

$$\Delta\sigma_r = \sigma_r - p_0 = \frac{4EA}{3r^3} \quad (3.7.8)$$

At the current cavity surface, $r=R_c$, $\Delta\sigma_r = \Delta p$, so that $A = 3R_c^3 \Delta p / 4E$.

Substituting back into (3.7.8) and (3.7.6) gives equations for $\Delta\sigma_r$ and u and using (3.7.5) and (3.7.6) to find $\Delta\sigma_\theta$ gives equations for displacements and stress changes in an elastic region surrounding the expanding cavity

$$\frac{u}{R_c} = -\frac{3\Delta p}{4E}\left(-\frac{R_c}{r}\right)^2, \quad \frac{\Delta\sigma_r}{\Delta p} = \left(\frac{R_c}{r}\right)^3, \quad \frac{\Delta\sigma_\theta}{\Delta p} = -\frac{1}{2}\left(\frac{R_c}{r}\right)^3 \quad (3.7.9)$$

Yield will occur (initially at $r=R_c$) when

$$\Delta\sigma_r - \Delta\sigma_\theta = 2c \quad (\text{assume } c \equiv s_u) \quad (3.7.10)$$

so using equations (3.7.9) shows that this occurs when

$$\Delta p = 4c/3, \quad R_c = R_{c0} / \left(1 - \frac{c}{E}\right) \quad (3.7.11)$$

and following this a plastic region will surround the cavity to a radius R_p , with an elastic region outside. In the plastic region, equilibrium (3.7.7) and (3.7.11) require that

$$\sigma_r = p - 4c \ln(r/R_c) \quad (3.7.12)$$

$$\sigma_\theta = p - 2c - 4c \ln(r/R_c) \quad (3.7.13)$$

after evaluating constants from $\sigma_r = p$ at $r = R_c$. The elastic displacement just outside $r = R_p$ may be obtained using (3.7.9a) as

$$u_{Rp} = \frac{3R_p}{4E} (\Delta p - 4c \ln(\frac{R_p}{R_c})) \quad (3.7.14)$$

and at $r = R_p$ equation (3.7.12) may be used with (3.7.11) to give

$$\Delta \sigma_r = \Delta p - 4c \ln(\frac{R_p}{R_c}) = 4c/3 \quad (3.7.15)$$

giving the displacement at the distance of an element just yielding

$$u_{Rp} = R_p c/E \quad (3.7.16)$$

Equation (3.7.15) may be used to determine the cavity pressure if R_p/R_c can be found as a function of the cavity radius R_c alone. This may be done by equating the volume of the plastic region $R_c \leq r \leq R_p$ to its original volume $R_{co} \leq r \leq R_p - u_{Rp}$, giving

$$\frac{R_p^3}{R_c^3} = \frac{R_p^3 (1 - R_{co}^3/R_c^3)}{3u_{Rp} (1 - u_{Rp}/R_p + u_{Rp}^2/3R_p^2)} \quad (3.7.17)$$

Using (3.7.16) for u_{Rp}/R_p and substituting for R_p/R_c in (3.7.15) gives the relationship between the cavity pressure, p , and the cavity radius, R_c

$$p = p_0 + \frac{4}{3} c \left\{ 1 + \ln \left[\frac{E}{3c} \frac{(1 - R_{co}^3/R_c^3)}{(1 - c/E + c^2/3E^2)} \right] \right\} \quad (3.7.18)$$

which is valid only after yield has occurred at the cavity wall. As $R_c \rightarrow \infty$ a limiting value of p will be reached, which effectively will be observed (within 1% of Δp_L) when $R_c \geq 4R_{co}$. This value will be

$$p_L = p_0 + \frac{4}{3} c \left\{ 1 + \ln \left[\frac{E}{3c} (1 - c/E + c^2/3E^2) \right] \right\} \quad (3.7.19)$$

It can be seen that for stiffer soils or metals where $E/c \gg 1$ this reduces to the form of Bishop et al

$$p_L = p_0 + \frac{4}{3} c \left\{ 1 + \ln \frac{E}{3c} \right\} \quad (3.7.20)$$

The difference between limiting pressures Δp_L using (3.7.19) and (3.7.20) is about 1% if $E/c = 20$, about 3% if $E/c = 10$ and about 20% if $E/c = 4$. Since values of E/c might be below 10 in the soft soils in this study, equation (3.7.19) should be used. To obtain maximum information from a particular

expansion test, equation (3.7.18) suggests plotting Δp v $\ln(1 - R_{Co}^3/R_C^3)$. The slope will then be $4c/3$, giving the undrained strength, and the intercept (as $R_C \gg R_{Co}$) will be $4c/3\{1 + \ln[\frac{E}{3c}/(1-c/E + c^2/3E^2)]\}$. This method will allow the validity of this analysis to be tested; a non-constant slope will indicate a non-constant value of E/c . The tangent to the curve at the intercept will then give the undrained strength. Both c and E/c might be compared to values from other tests.

Summary of undrained spherical cavity expansion theory

If a spherical cavity of radius R_{Co} exists in an isotropic soil mass with initial stress p_o everywhere, the cavity will begin to expand when the cavity pressure p exceeds p_o . Soil behaviour will initially be elastic everywhere until the yield criterion (equation 2.7.10) is satisfied at the cavity surface. This will occur when

$$\begin{aligned} R_C = R_{Co}/(1-c/E) &= 1.5 R_{Co} \quad \text{if } E/c = 3 \\ &< 1.1 R_{Co} \quad \text{if } E/c > 11 \end{aligned} \quad (3.7.21)$$

and the cavity pressure when first yield occurs will be

$$p = p_o + \frac{4}{3}c \quad (3.7.22)$$

As the cavity is expanded further it will be surrounded by a plastic soil region to a radius R_p , in which the radial total stress, σ_r , will be

$$p_o + \frac{4}{3}c \leq \sigma_r \leq p \quad (3.7.23)$$

and an elastic region beyond this where $\sigma_r < p_o + \frac{4}{3}c$. At large cavity radii, the cavity pressure will tend to the limiting value given by equation 3.7.19.

For $E/c > 10$ the total increase in cavity pressure above p_o will be approximately

$$\Delta p = p_L - p_o \approx \frac{4}{3}c\{1 + \ln\frac{E}{c}\} \quad (3.7.20)$$

and will be attained within 1% when $R_C \geq 4R_{Co}$.

Theoretical cavity pressure:volume and pressure:radius curves are shown in figure 3.10 for a typical example where the initial cavity radius is 1 mm, the undrained strength c is 0.2 kN/m² and $E/c = 6$. It can be seen that yield is

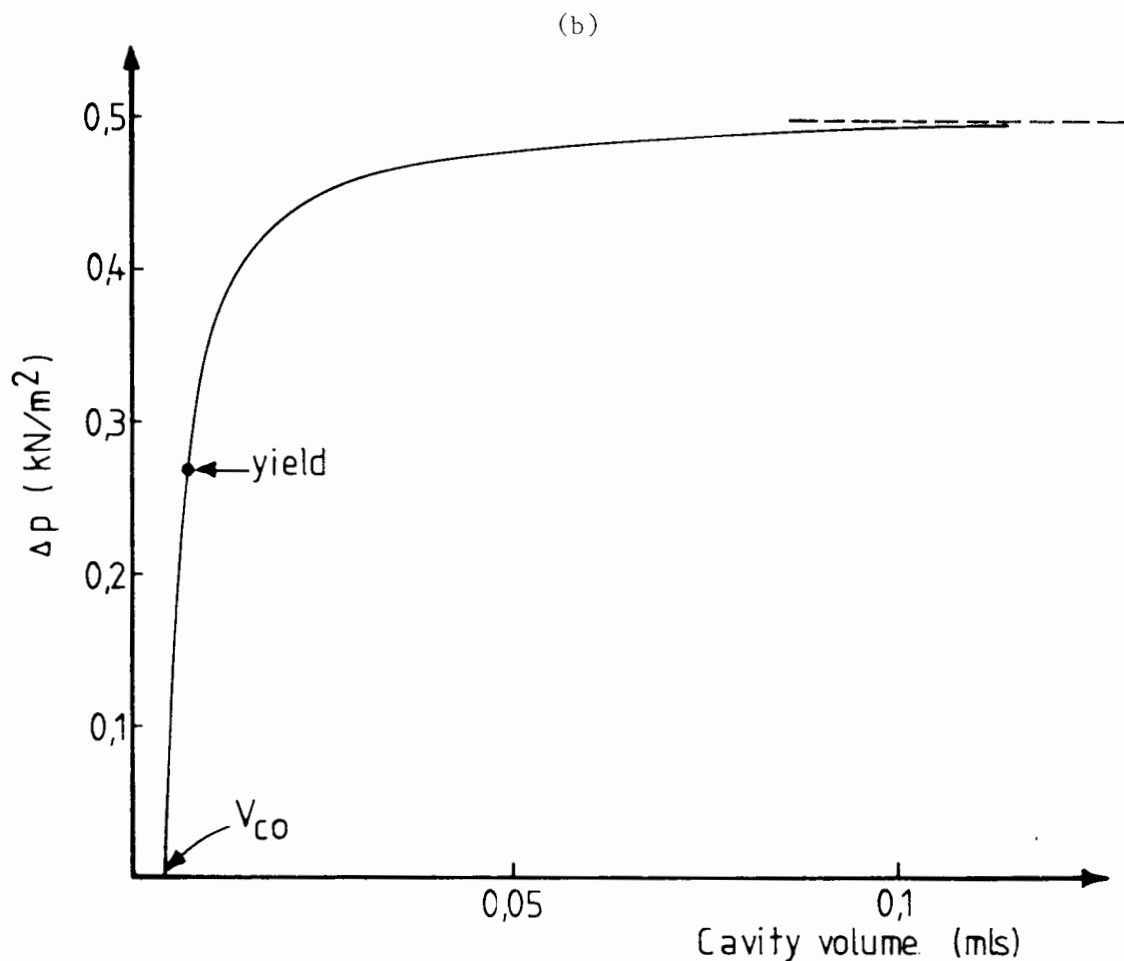
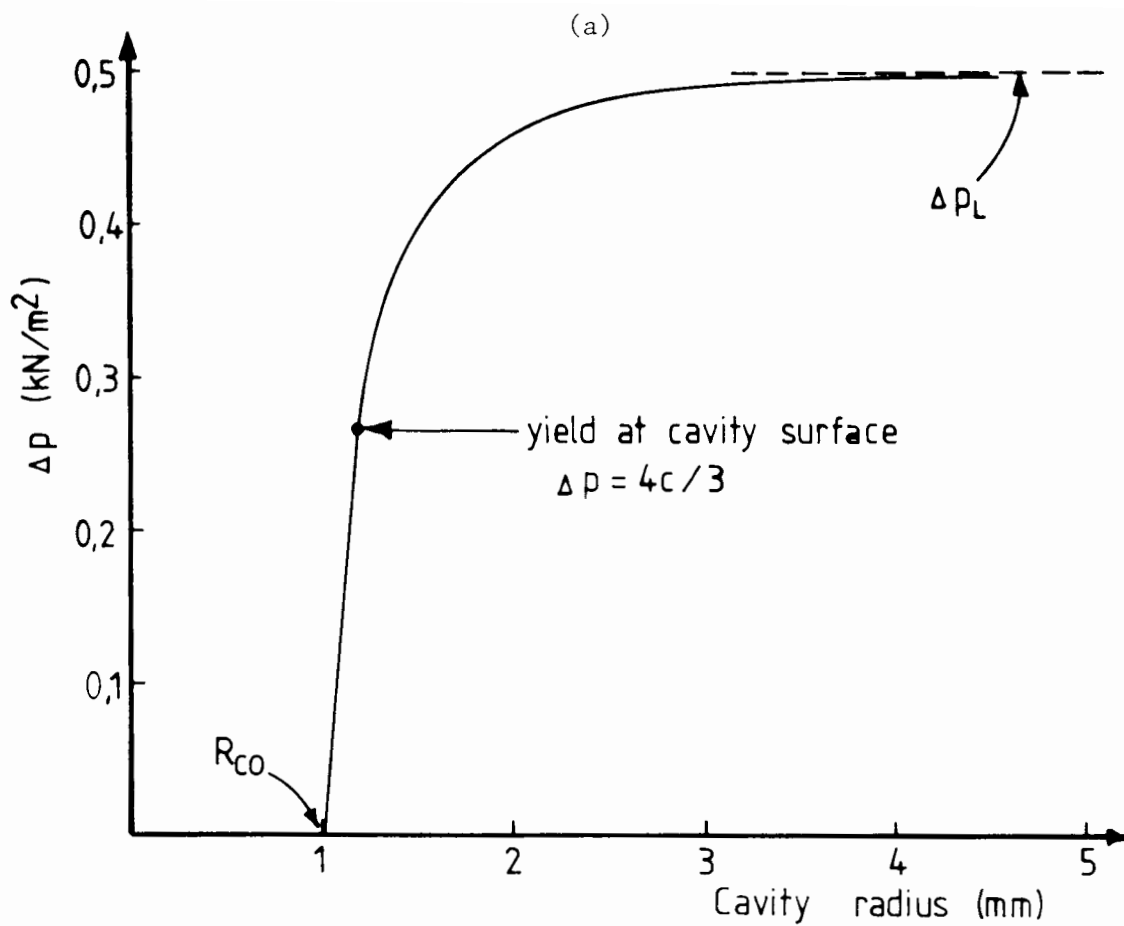


Figure 3. 10 Theoretical curves for expansion of spherical cavity for initial radius 1mm, with $E/c=6$, $c=0.2 \text{ kN/m}^2$.

not defined well on either curve and that the limiting pressure is attained with relatively small volume increase.

3.7.2 Testing apparatus

A system was designed which enabled a fluid filled cavity to be expanded at the centre line of a soil column, at the tip of a hypodermic needle inserted through a rubber port at the column wall shortly before testing. Several particular requirements were determined. A system was required which could increase the cavity volume at a controlled and preset rate, from which volume changes could be calculated. The cavity pressure had to be measured remotely to the same accuracy obtained for pore pressure measurement in the column, ideally $\pm 0.005 \text{ kN/m}^2$. Most important, the cavity had to be constrained to remain intact within the two phase soil medium (rather than diffusing into the pores or mixing with soil particles).

The spherical cavity

The system was initially designed and constructed using water as the cavity fluid, contained inside a spherical membrane of initial diameter equal to that of the hypodermic needle. This proved unsatisfactory due to unavailability of a membrane material with all the desired properties of elastic isotropy, high flexibility and strength, low hysteresis and creep characteristics and able to be expanded from 1 mm initial diameter up to 10 mm maximum. These membrane contained cavities were used in preliminary soil experiments only. An alternative system was developed using silicon fluid as the cavity medium and relying on its insolubility and viscosity to retain a coherent intact shape as the cavity expanded. This fluid has a density slightly above that of water so that in low density soils buoyancy effects were not important. In all column tests, when the soil was subsampled the cavities were found intact and fluid filled (although the volumes could not be measured accurately). This was taken as sufficient evidence that, at least for small cavity volumes, the fluid filled cavity method was suitable.

Volume control and pressure measurement

The cavity fluid was contained in an acrylic cylinder 1 m long x 7.0 mm ID. A 6.7 mm diameter piston moved inside the cylinder and was connected to the moving nut on a lead screw of 1 mm thread pitch, which was rotated using a stepping motor. With this system, continuously variable rate control over expansion was attained from 0 to 10 mls/min with volume changes calculated from motor drive input accurate to $\pm 0.2 \text{ mm}^3$, measured from digital output and from chart record timebase. Thick walled "zero volume change" nylon tubing connected the cylinder to the hypodermic needle.

Pressure in the system was measured using a transducer mounted in the pressure line near the needle. A number of calibration procedures were required to obtain correction factors for pressure losses through the tubing and needle, which were applied to measured pressures to obtain actual cavity pressures. These corrections depended mainly on expansion rates and regular procedures for calibration and testing were established to ensure repeatability. Accuracy of calculated cavity pressures was at worst $\pm 0.02 \text{ kN/m}^2$ for flow rates of 2 mls/min and proportionately lower for lower flow rates, since the main source of error was the pressure loss between the cavity and the transducer, which varied linearly with flow rate. Calibrations to quantify these effects are described in a separate report (Elder, 1985).

3.7.3 Testing technique

Each settling column had up to five self sealing rubber ports. Following calibration at the flow rate to be used in the test the needle was positioned horizontally level with the port and aligned radially and fluid expelled to form a drop at the needle tip; this prevented air from entering the needle. A sleeve guide was used to assist smooth insertion through the port to the side of the column, where fluid was expelled through the needle to balance a 2 mm retraction due to the increased pressure on insertion. Excess pressures were allowed to dissipate and the needle was then pushed slowly to the centre of the column, and driving pressures allowed to dissipate fully. The cavity was

then expanded at a constant rate of volume increase with continuous recording of cavity pressure and pore pressures at adjacent transducers in the column wall, both during and after expansion until transient pressures had stabilised. The needle was then withdrawn for testing at other locations.

3.7.4 Summary

Since this study represents the first reported use of such a technique, some particular advantages and disadvantages of the test, as developed here, are summarised.

Advantages

- testing at in situ stress level
- little soil disturbance
- path for preferential drainage to surface not created (existing excess pore pressure gradient in the horizontal direction is likely to be small)
- pore pressures and total stresses may be estimated subject to assumptions
- cavity shape might be used to measure anisotropy (not this study)
- after completion of test soil consolidation continues unaffected outside localised area

Disadvantages

- no previous results available; assumptions and analysis not validated
- high pressure measurement accuracy required (differential pressures much less than hydrostatic pressures)
- cavity has spherical symmetry but columns have axial symmetry
- tests volume rate rather than strain rate controlled
- effect of soil anisotropy unknown
- considerable development time required

Only the most important of the points above could be considered in this study. Considerable further development remains to be done before the spherical expansion technique can be used other than in the specific laboratory conditions for which it was designed.

4.1 Introduction

4.2 Soil description and preparation

4.3 Experimental programme

4.4 Results of short column experiments

4.4.1 Fall cone calibrations

4.4.2 Spherical cavity expansion testing

4.5 Results of tall column experiments

4.5.1 Experiment 5

4.5.2 Experiments 6,7 and 9

4.5.3 Experiments 8,10 and 11

4.5.4 Experiment SW2

4.6 Incremental surcharge experiments, 11 to 15

4.1 Introduction

In this chapter the soil used in experiments is described and compared to other soils on which detailed studies have been reported previously. The preparation techniques used before the soil is deposited in columns are outlined. The experimental programme is summarised in section 3 and reasons for choice of initial parameters are presented. Other tests are described which are complementary to those in settling columns.

The remainder of the chapter presents the direct experimental results obtained in all settling column experiments, including settlements, density and pore pressure profiles, and strength tests. Experiments are grouped for comparison where initial parameters are similar. The cavity expansion testing procedure in particular is described in detail. High variability occurs between these tests and some aspects of results are very difficult to explain, particularly pressure variations or fluctuations measured at the tip of the probe before undrained expansion begins. Consistent trends are nevertheless observed for pressure changes occurring during the undrained expansion phase and undrained strengths are calculated from these values. Fall cone and shear vane tests yield consistent results and appear to be suitable for strength evaluation in these very soft soils. Detailed analyses of consolidation and strength behaviour are carried out in chapters 5 and 6 respectively.

4.2 Soil description and preparation

A natural estuarine soil, collected from the upper 50 cm of intertidal deposit from the Parrett river at Combswich, Somerset and designated "Combswich" soil or mud, was used in all experiments. Direct comparison is possible with previous experiments at Oxford reported by Been(1980) and Sills and Thomas (1983) who used the same soil. Little change is observed between batches of soil collected from this site over a period of 8 years. The average specific gravity of solids is 2.66 ± 0.01 and organic contents are less than 5% and typically around 2%. All natural particles pass a 425 μm sieve and > 99% pass

a 63 μm sieve. The clay fraction ($< 2 \mu\text{m}$) is close to 40% ($\pm 5\%$), classifying the soil as a silty clay, and is mainly illitic with some kaolinite and a smaller amount of chlorite. The silt fraction consists of 35-40% quartz with lesser amounts of calcite, feldspar, illite and kaolinite in approximate decreasing order (Sills et al, 1985). The salinity of water at Combwich varies between fresh (4 ppt) and saline (35 ppt salt, by mass) and is typically less than 12 ppt.

Soil for experiments was sieved wet to remove the $> 63 \mu\text{m}$ fraction using tap water (4 ppt, density 1.003 Mg/m^3) in all but one experiment, SW2, where natural ocean water (35 ppt, density 1.024 Mg/m^3) was used for sieving and mixing. Atterberg consistency limits were determined under different conditions and are summarised in table 4.1. Values reported for some other natural clays which have been subjected to detailed investigation are also shown and although wide ranges have been reported, the particular samples described appear similar to Combwich on this basis.

Soil Type	LL	PL	PI
Combwich unsieved, Combwich water	66-73	28-30	38-40
Combwich sieved in tap water (6 batches)	62-63	29-32	30-34
Combwich sieved in salt water	58	28	30
Drammen clay (Bjerrum, 1972)	61	32	29
Maine organic clay (Ladd & Edgers, (1972))	65	31	34
Bangkok clay (" ")	65	24	41

Table 4.1 Atterberg limits for soils *

The reason for the decrease in limits between unsieved and sieved material is not known but is probably due to the nature of the material collected on the 63 μm sieve which is black and possibly organic. Differences between limits for fresh and saline conditions are small and all fresh water sieved values fall within a narrow range, suggesting that the sieving process does not markedly affect soil properties but standardises behaviour of different batches. Soil was stored underwater between sieving and use.

Before input into columns, soil was mixed to the required density (ascertained using a hand held PAAR digital density meter, accurate to $\pm 0.001 \text{ Mg/m}^3$) using de-aired water, then circulated by continuous pumping and stirring for 30 minutes. Deposition was by pouring (short columns) or continuous pumping with the outlet pipe held just below the surface of the rising slurry. Pore pressure and total stress transducer readings were logged continuously from 30 minutes before deposition to obtain accurate zeroes.

4.3 Experimental programme

Details of experiments carried out in settling columns are shown in table 4.2. Experiments 1 to 11 were designed to simulate settling and consolidation behaviour over a wide range of initial densities ($1.02 - 1.25 \text{ Mg/m}^3$, $5.7 \leq e_i \leq 100$) and for different masses of solids ($0.4 - 5.7 \text{ kg}$, representing solid material heights, h_s , $19-260 \text{ mm}$). In chapter 2 the existence of a critical value e_1 for the depositional void ratio e_i was discussed. Been (1980) suggested that at $e_i > e_1$ soil could exist initially as a suspension in which no effective stresses would be measured. At $e_i < e_1$ effective stresses would always be greater than zero and a suspension would never exist. For Combwich soil the critical value e_1 was around 13. If subsequent consolidation and strength properties are to be examined, initial deposition in the suspension range is undesirable since segregation may occur and the consolidating bed will be non uniform. In all but experiments 1a, 1b and 9 initial void ratios were below e_1 . Experiment 9, in a higher column (3.5 m) existed just inside the suspension phase initially and was intended to approximate the natural condition where a deep sediment layer forms below a dense suspension. Comparisons of consolidation behaviour under basal drainage conditions varying from impermeable to zero head could be made using experiments 1a and 1b, 10 and 11, and at different times in experiments 3 and 5. Experiment SW2 was begun by R. Thomas who collected data for an initial 100 hour period; thereafter measurements were continued as part of this study. Experiments were continued

Expt. Number	Column Type I-Intact S-Segments P-PWPS	Initial Height (mm) h_i	Initial Density (Mg/m ³) P_i	Initial Void Ratio e_i	Soil Mass (kg) M_s	Duration (720hrs= 1 month) (hrs)	Drainage S-Single D-Double	X-Ray Profiles	Tests S-Cavity C-Cone V-Vane T-Triaxial	Comments
1a	I	1842	1.022	86.2	0.46	1,180	S	1	-	Drainage comparisons
1b	I	1842	1.020	96.5	0.41	1,180	D	1	-	
2	S	957	1.18	8.36	2.22	2,580	S	-	S, C, V	Surcharge applied
3	I	1478	1.174	8.69	3.32	2,570	Varied	3	S	Drainage effects
4	S	889	1.235	6.14	2.70	1,700	S	4	(T)	Surcharges applied
5	S, P	1748	1.210	7.00	4.75	518	S, D, S	4	S, C, V, T	
6	S, P	1748	1.251	5.68	5.69	5,670	D	4	S, C, V	
7	S, P	1197	1.251	5.68	3.89	11,000	D	4	S, C, V	
8	I, P	1500	1.16	9.55	3.09	745	S	14	-	
9	I	3510	1.094	17.2	4.19	7,350	S	1	S	
10	S, P	1553	1.147	10.5	2.93	5,050	S	7	S, C, V	Oedometer sample
11	S, P	1553	1.153	10.0	3.06	15,000	S, D, S	9	C, V	Surcharges applied
SW2	I	1711	1.141	13.0	2.66	12,000	S	7	-	Salt water Surcharge applied
12	I	43	1.207	7.12	0.115	134	S	0.2 kN/m ²	Surface surcharges applied incrementally to obtain rapid consolidation testing	
13	I	39	1.212	6.93	0.107	90	S	0.4 kN/m ²		
14	I	36	1.206	7.16	0.097	84	S	0.8, 2.5 kN/m ²		
15	I	41	1.119	13.3	0.062	75	S	0.8, 4.6, 6.7 kN/m ²		
101-104 105-108 109-112 113-116	I	200	1.34	3.9	0.88	1 10 100 1,100	S	-	S, C, V	Investigation of time effects and secondary variations within each type of strength test. Four experiments in each group; eighteen in each series.
201-204 205-208 209-212 213-214 216-217 218	I	200	1.41	3.0	1.07	1 10 100 2,300 6,500 530	S	-	S, C, V	
301-304 305-308 309-312 323 324-325 326-328	I	200	1.25	5.7	0.65	1 10 100 2,800 6,200 6,200	S	-	S, C, V	
										Surface surcharges applied after 5,300 hours.

Table 4.2 Details of settling column experiments.

for periods up to 2 years; measured excess pore pressures in some tests had not exceeded 90% dissipation after 8 months and settlements continued up to the completion of testing in all cases.

The excessive timescale for tests forced by this behaviour caused two further series of experiments to be undertaken. Experiments 12 to 15 had very low initial slurry heights. Immediately after deposition wet filter papers were placed on the slurry surface. Fine sand was sprayed evenly on the papers followed by several layers of coarser sand or lead shot with the water level held always above the sand surfaces. Measurement of the surcharge mass and slurry surface height (accurate to ± 0.1 mm averaged around the column wall) enabled rapid consolidation testing (using load increments) at stress levels down to 0.2 kN/m^2 . This is considerably below the values attainable in a conventional oedometer due to piston friction limitations, and the small sample heights (≈ 40 mm) ensured sample homogeneity at the completion of each increment (i.e. negligible self weight). Compression ($e-\sigma'$) behaviour could thus be measured without resort to the assumptions required for, for example, interpretation of the slurry consolidometer test.

Three series of experiments, with 18 short (200 mm) columns in each series, were carried out to investigate time variation of near surface (low effective stress) properties and to enable detailed study and comparison of those factors most important in strength test behaviour, including particularly testing rates. In each series all tests were begun under identical conditions. After each of the time periods indicated in table 4.2, up to 4 samples were tested at the various conditions, to determine trends both with aging or consolidation and with testing methods.

Other tests

1. An oedometer consolidation test was performed on a sample taken near the base of the column in experiment 10 after testing and soil unloading.

2. Surcharge loads of sand and lead shot were applied to the surfaces of soil in experiments 2,4,11,SW2,324 and 325 to investigate behaviour under increased loading following initial consolidation.
3. 102 mm diameter triaxial samples prepared at the base of columns 4 and 5 were tested in the triaxial cell. Sample 4T failed by slumping when the lateral restraint was removed before shearing.
4. Isotropically consolidated CU triaxial testing was carried out on 7 samples (T1-T7) at consolidation pressures from 35 to 400 kN/m². Three 38 mm tube samples were taken from columns 7,10 and 218 but all failed by slumping under self weight before testing could be carried out.
5. A limited series of fall cone tests was undertaken on remoulded samples to evaluate cone factors (see chapter 2) for different cones.
6. In situ hand vane tests were carried out at Combwich for comparison with laboratory tests.

4.4 Results of short column test, series 100, 200, 300

In each series of short column (200 mm) tests, all columns were filled with soil which had been remoulded using a standard laboratory mixer. At particular times (usually 1,10, 100 hours) after remoulding and deposition, columns were tested in groups of four, so that soil properties were identical between columns within a group. Cone tests were carried out at the soil surface and shear vane and cavity expansion tests below the surface, at different testing rates. The testing programme for these columns is summarised in table 4.3. Vane rotation rates of 4, 15, 60 and 100°/min were used, encompassing most rates in common laboratory useage. Suitable expansion rates for spherical cavity tests were not known since experimental use of this technique has not been reported elsewhere. A forty-fold variation from 0.05 ml/min (0.8 mm³/sec) to 2.0 ml/min (33 mm³/sec) was investigated as earlier tests indicated that this was the range in which accurate measurements could be made of cavity volume and pressure simultaneously. In this range times

Series no.	Time to testing (hrs)	Vane rate (°/min) Cavity expansion rate (ml./min)	100 2	60 0.5	15 0.2	4 0.05
100 1.345 Mg/m ³ e _i = 3.86 LI = 3.56	1-4 10 100 1100		101 105 109 113	102 106 110 114	103 107 111 115	104 108 112 116
200 1.414 Mg/m ³ e _i = 3.04 LI = 2.61	1-4 10 100 530 2300 6500	(218)	201 205 209 213 216	202 206 210 214 217	203 207 211	204 208 212
300 1.249 Mg/m ³ e _i = 5.75 LI = 5.76	1-3 10 100 2800 6200 6200	(323) 0.2 kN/m ² 0.8 kN/m ² (Surface loads applied at 5300 hrs)	301 305 309 326 328	302 306 310 327 324 325	307 311	312

Table 4.3 Details of short column experiments.

Experiment nos. given in right hand columns.

required to achieve a limiting pressure were comparable to those at peak resistance in the fastest vane tests.

Figure 4.1 shows normalised surface settlements for each series of tests. The normalised variable soil height/solids height is useful since it enables experiments with similar initial void ratios to be compared easily. Although the logarithmic timescale exaggerates the effect, it can be seen that settlements are very small initially but accelerate slightly after about 100 hours. Initial soil heights were only 200 mm, but settlements continue to occur after periods close to one year (≈ 9000 hours). Density profiles obtained using the hand held density meter are shown in figure 4.2. Densities at the soil surface (zero effective stress) increase with time suggesting that some form of time dependent creep is occurring in conjunction with any self weight consolidation. Two experiments in series 300 were loaded with surface surcharges σ_s after 5350 hours. The maximum effective stress due to self weight in these columns was 0.48 kN/m^2 and was probably attained in the non-loaded columns after 6100 hours. The surcharges are seen to have a relatively small effect compared to the total density changes undergone previously and the surface density under a 0.78 kN/m^2 surcharge does not attain a value as high as that at the base after the full period under 0.48 kN/m^2 self weight alone.

Fall cone, shear vane and cavity expansion tests were carried out in each short column. Figures 4.3, 4.4 and 4.5 describe torque-angular rotation data obtained from shear vane tests at different testing times and rates in these columns. Full analysis and comparison is left to chapter 6 but the trend for peak resistance to increase with time while residual strength remains fairly constant is seen in all test results. Both values increase, as expected, with rotation rate. Most tests involved initial rotation to 140° , followed by a relaxation period with vane stationary for 1 minute, then further rotation intervals 140° – 160° , 160° – 180° and 180° – 210° with successive relaxation periods of 10 and 100 minutes between each. This sequence enabled thixotropic regain

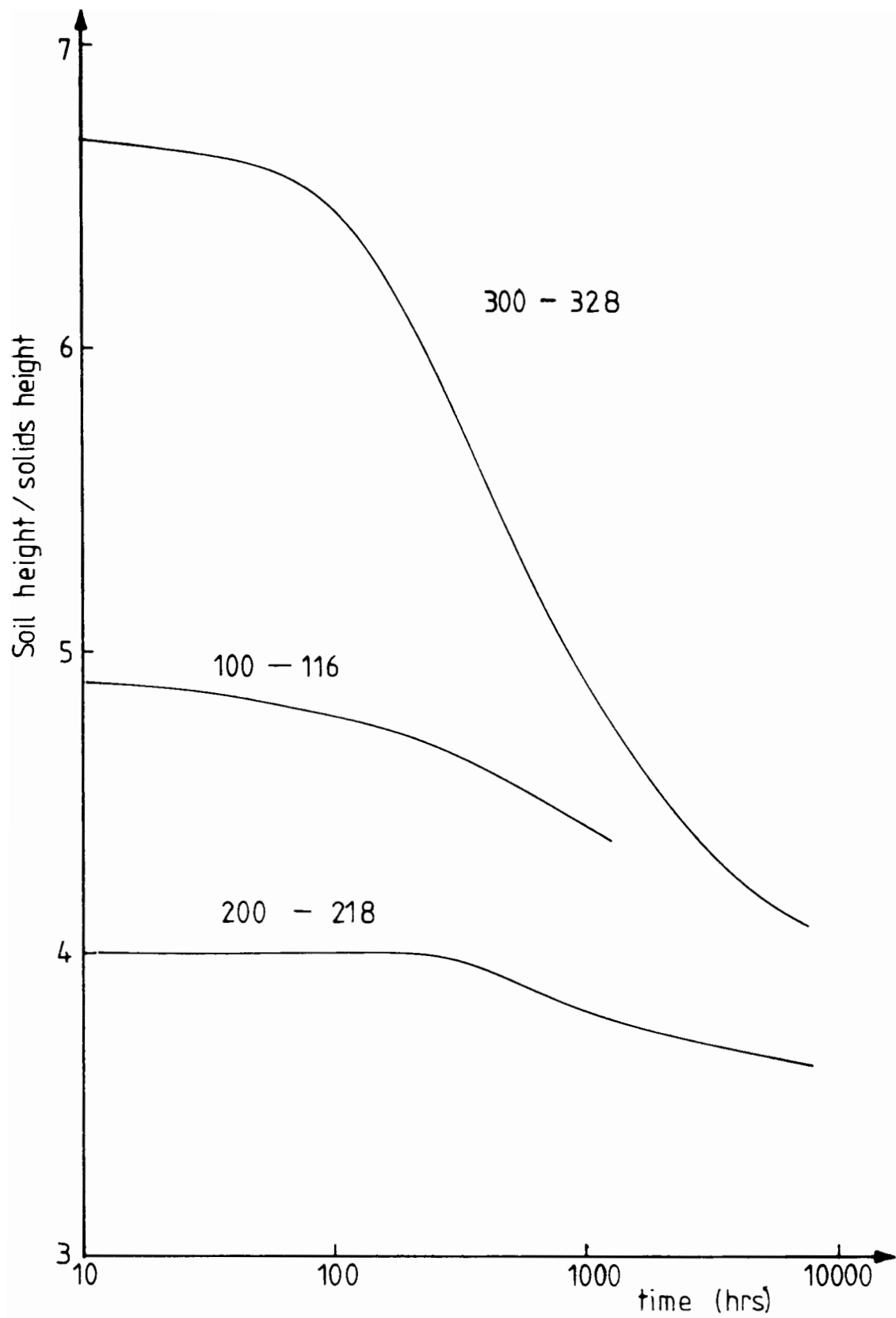


Figure 4.1 Normalised surface settlements, expts. 100, 200, 300 series.

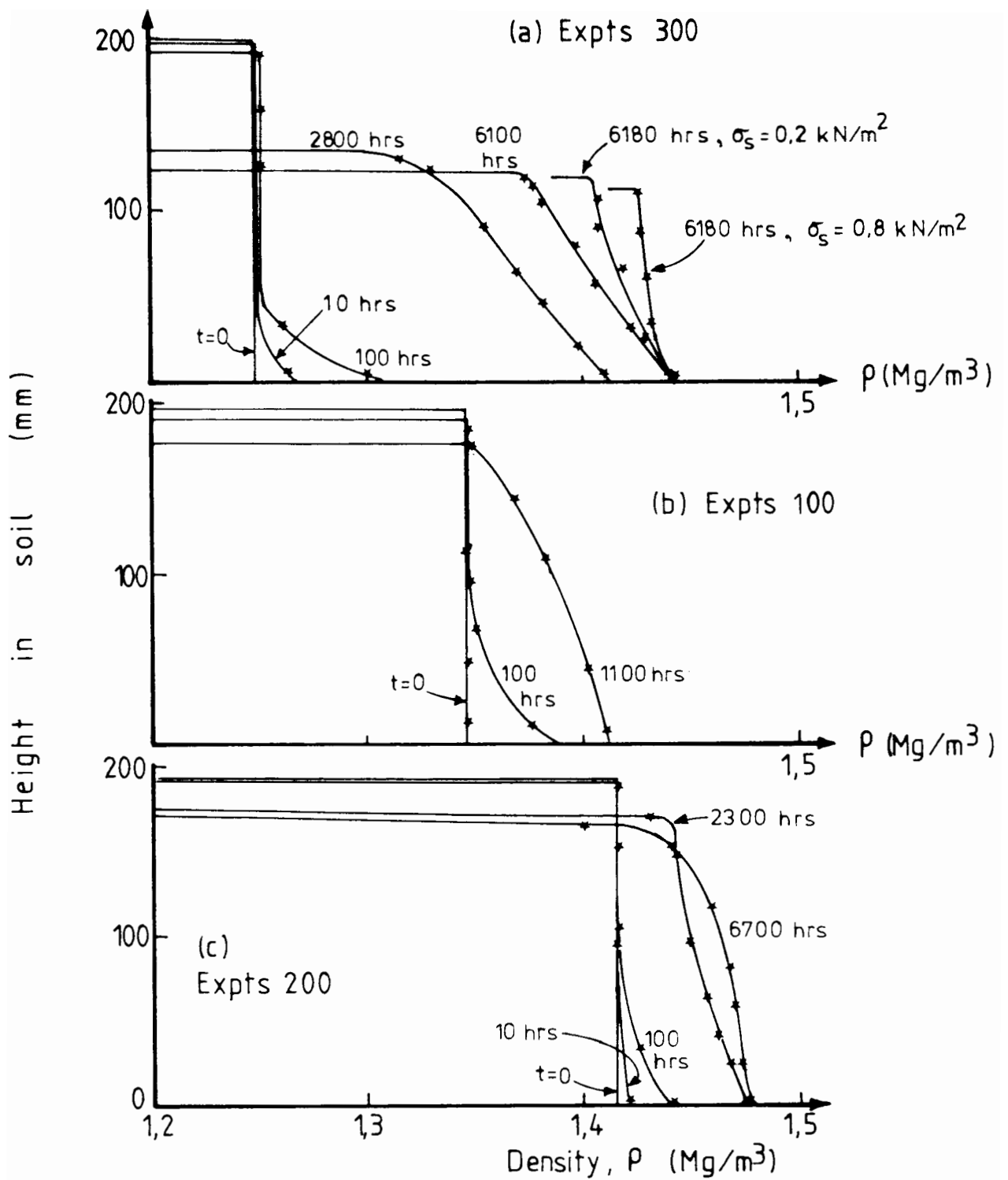


Figure 4.2 Density profiles for short column experiments.

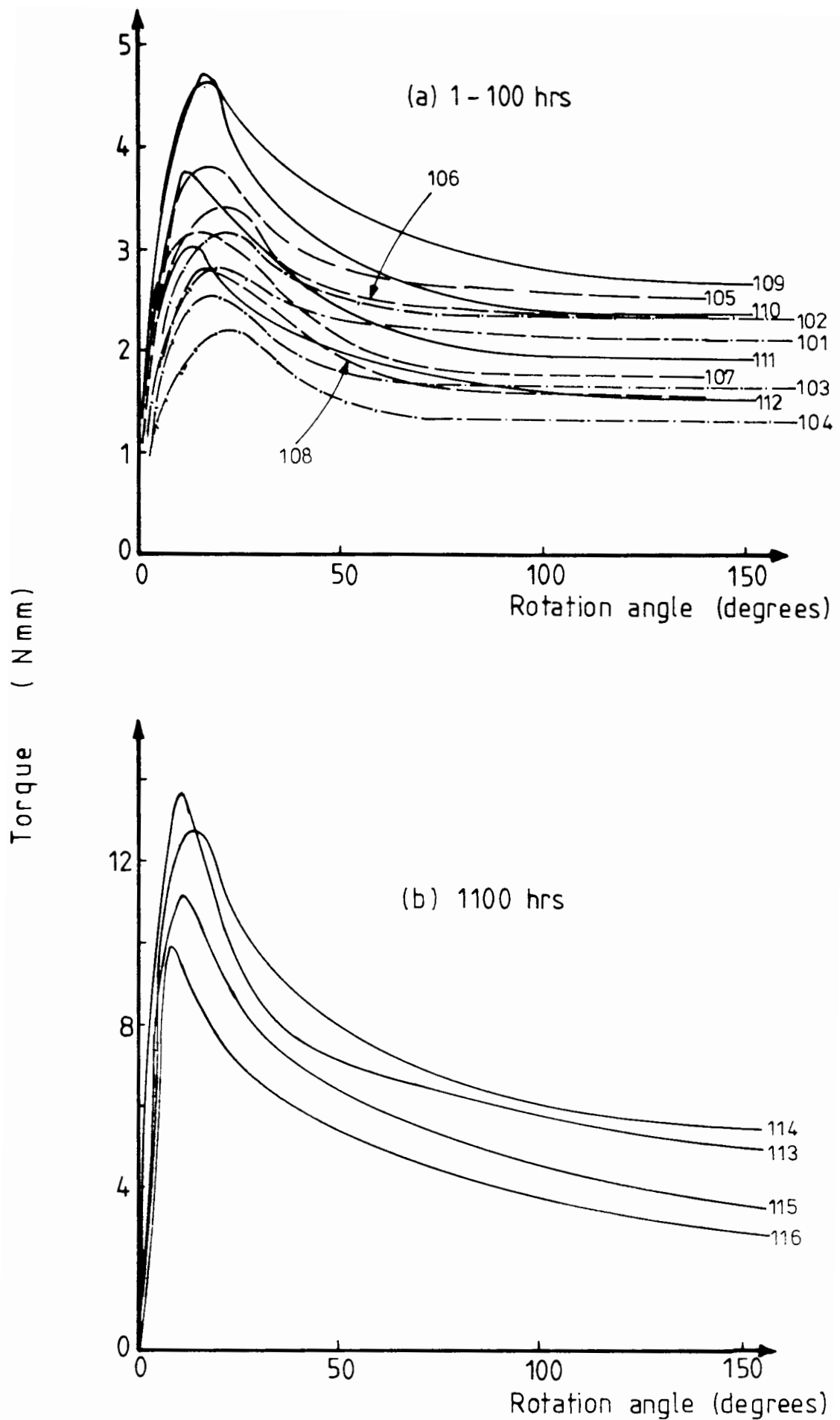


Figure 4.3 Shear vane test results, expts 100.

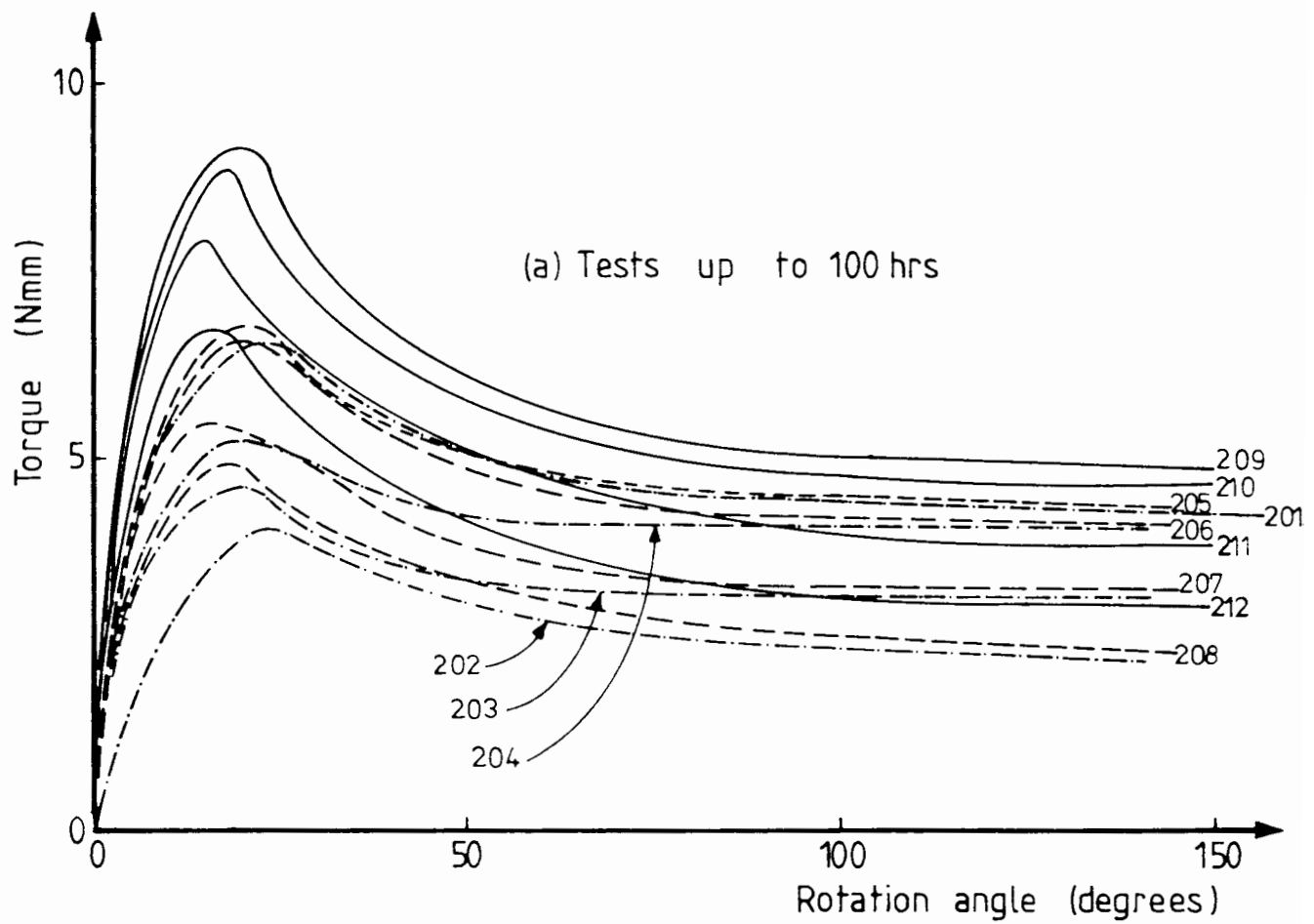
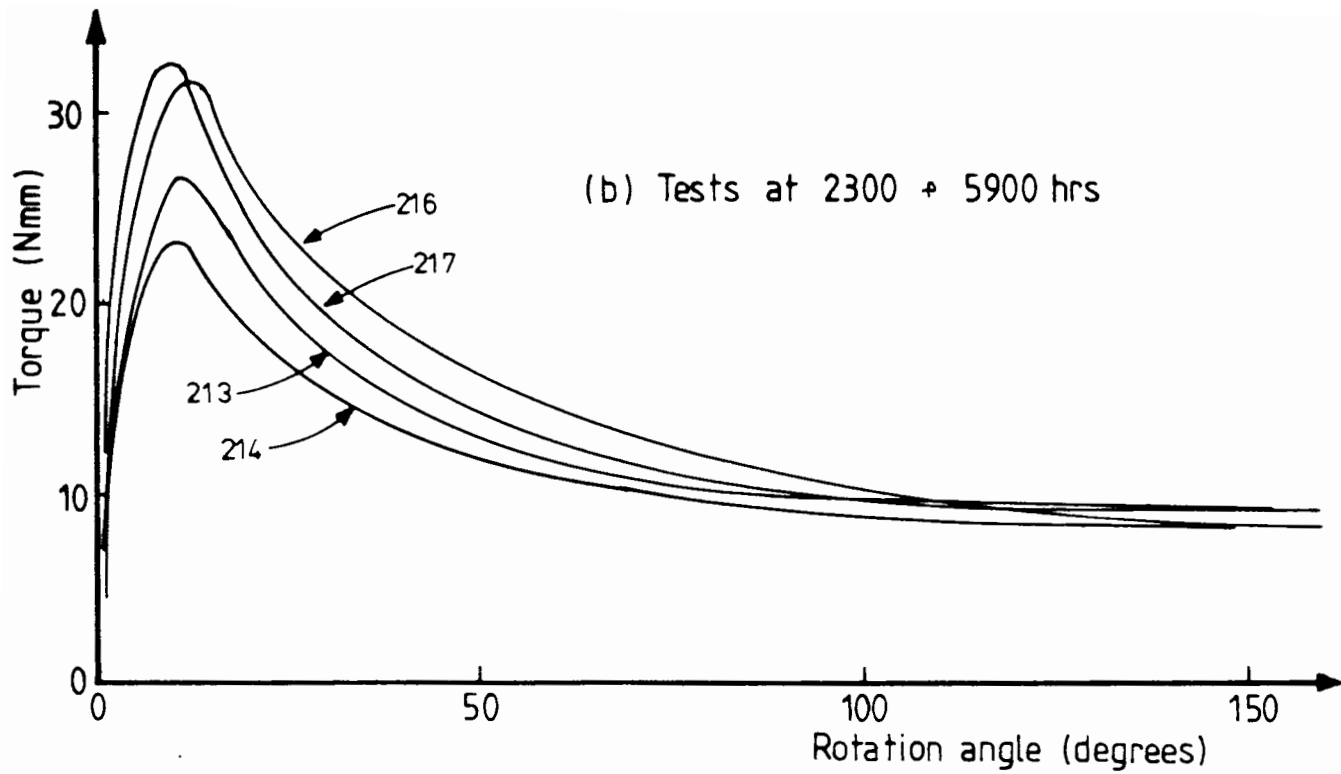


Figure 4.4 Shear vane test results, expts 200.

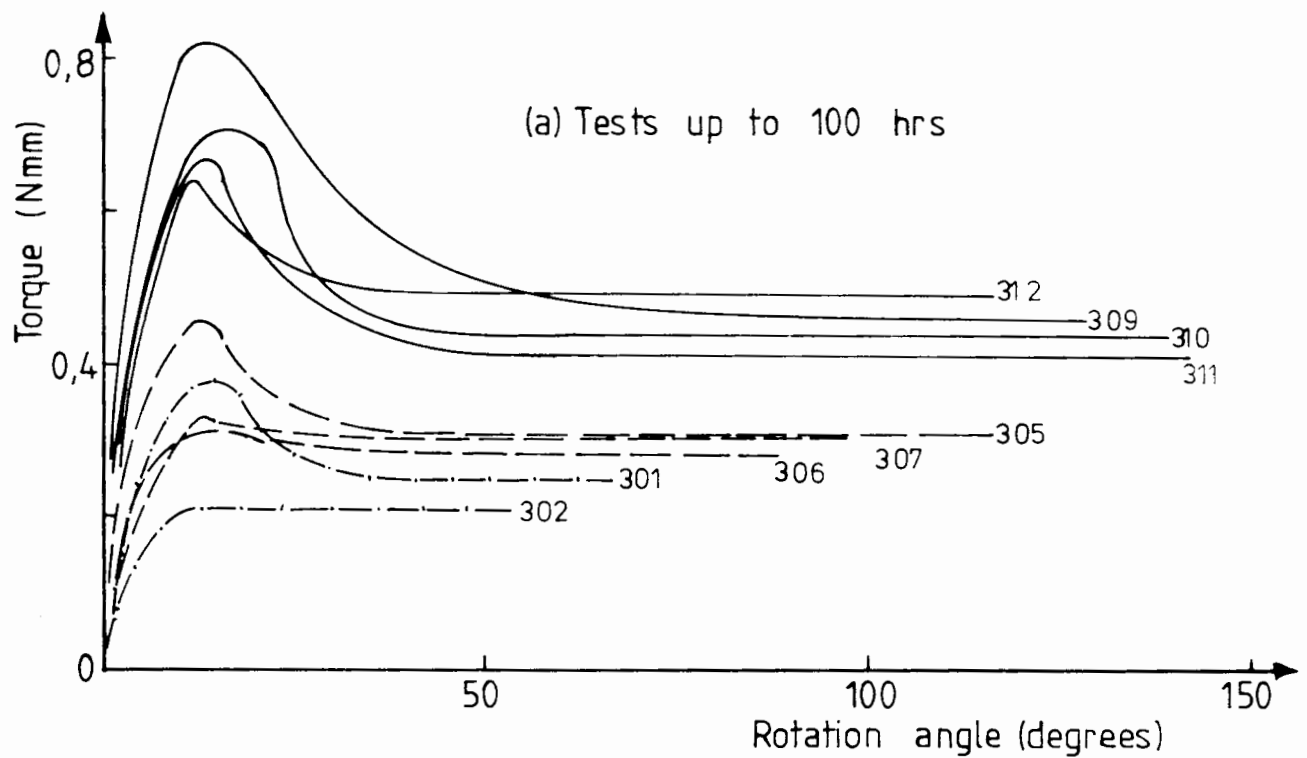
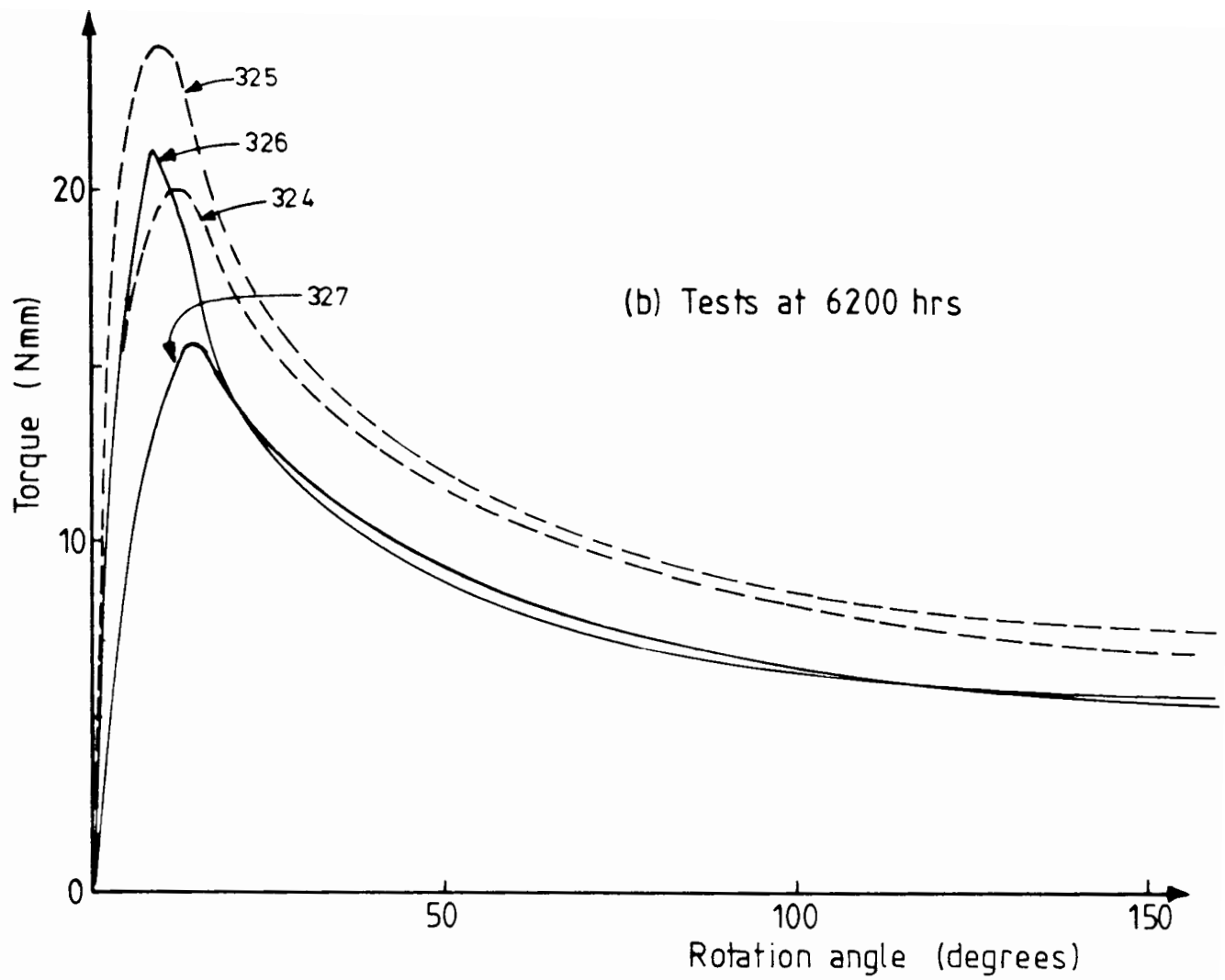


Figure 4.5 Shear vane test results, expts 300.

of peak strength to be estimated for the different regain times. Torque-rotation curves followed during these later rotations are not shown, but results are described in chapter 6.

4.4.1 Fall cone calibrations

Penetrations achieved by different fall cones are compared in figure 4.6. The lower set of data compare 60° 10 g and 30° 15 g PTFE cones and a reasonable best fit line gives the ratio of the cone factors for 60° and 30° cones $k_{60}/k_{30} = 0.32$. The range of data obtained by Hansbo (1957) from similar tests is shown; the different slope may be due to errors at the small penetrations used by Hansbo or due to other effects such as soil type and surface roughness. The upper data compare similar cones (60°, 10 g) with different soil:cone adhesion properties, obtained in one case using a different material (acrylic) and the other case by not oiling the cone surface before each test. The effect of a dry surface appears to be negligible, while some scatter results with the acrylic comparison but from the limited data presented average penetrations appear to be the same.

In figure 4.7 penetrations of fall cones at the soil surface are plotted against vane peak shear strengths obtained in the same column at the same density. Best fit lines describing the relationship $s_u = kW/d^2$ are fitted for different vane rotation rates and result in cone factors $k_{60} \approx 0.38$ for vane tests at 100° or 60°/min and 0.285 for tests at 4°/min. These may be compared with values reported by Hansbo, Wood and Karlsson (see chapter 3) which varied from 0.2 to 0.35. Hansbo and Wood do not report vane rotation rates but Karlsson used various rates and determined the rate at which minimum resistance resulted, using this value for cone factor calibration. The results shown in figure 4.7 may provide some explanation for the discrepancy between experimental vane strengths at given cone penetration and the strengths determined at the same penetration from a theoretical dynamic analysis by Houlsby (1982). Theoretical strengths were considerably higher, as shown by table 3.4 in the previous chapter. Fall cone penetrations are achieved in

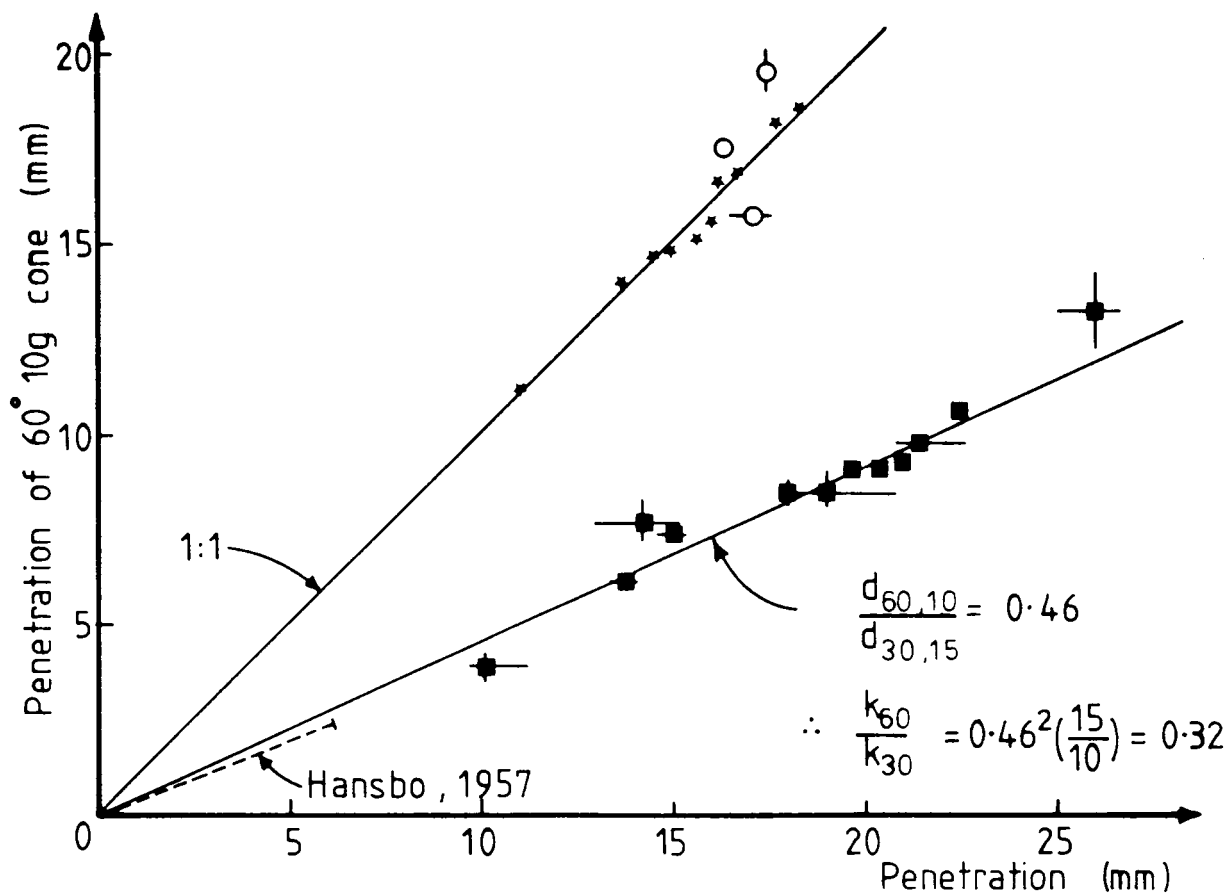


Figure 4.6 Comparison of cone penetrations for different cones.

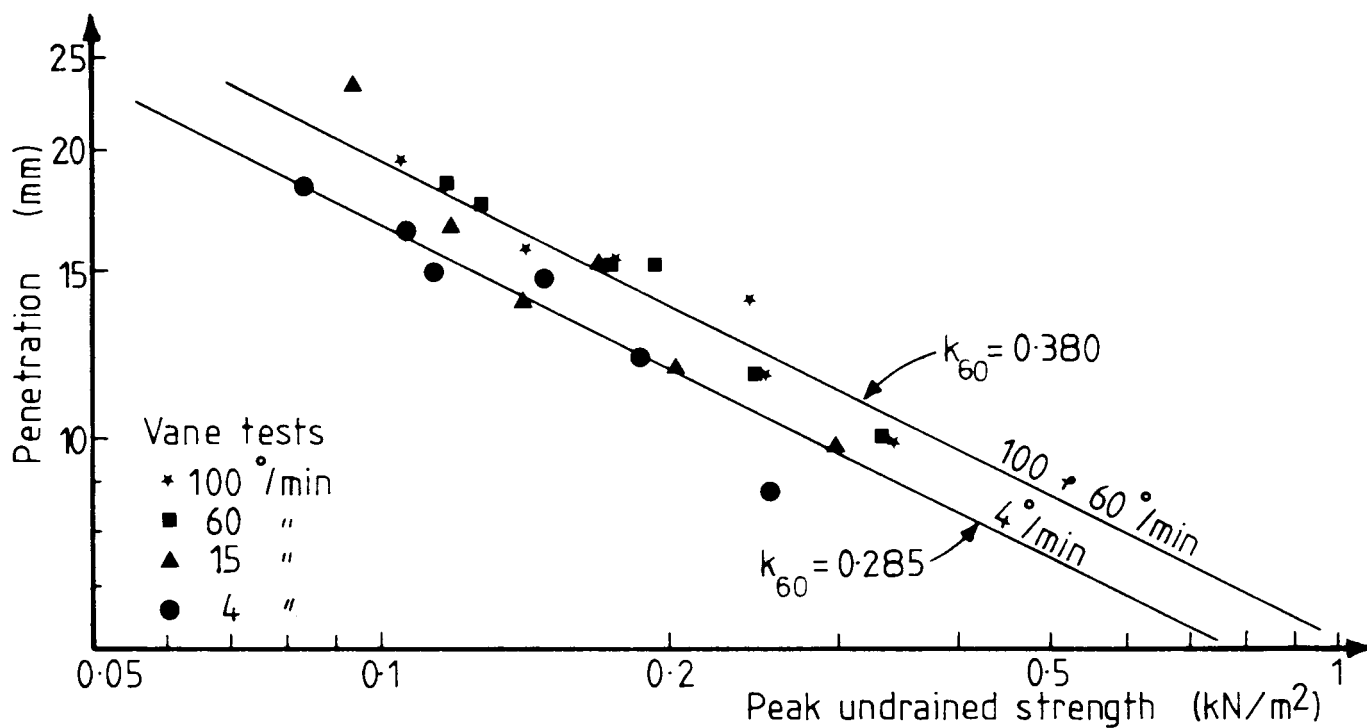


Figure 4.7 Calibration for 60° 10g fall cone using shear vane strength results from expts 100, 200 series, tests up to 100 hours.

around 0.1 seconds whereas even at the fastest vane rotation rates used here ($100^\circ/\text{min}$) failure at peak resistance required about 10 seconds. This suggests that the fall cone is not measuring the same strengths as the shear vane unless very high rotation rates ($> 1000^\circ/\text{min}$?) are used. At these rates, calculated cone factors from data of the form shown in figure 4.7 would be much higher so that deduced fall cone shear strengths would be higher for given penetrations. The method, using vane strengths at lower rotation rates to obtain cone factors therefore contains, in addition to the direct calibration, a correction for the different rate effects inherent in each test. This hypothesis should be investigated in a future study using vane rotation rates up to $1000^\circ/\text{min}$ or more and comparing strengths with theoretical fall cone strengths.

It should also be noted from figure 4.7 that the cone buoyancy effect, indicated by departure from linear fits to data, does not appear to be significant. Comparison with figure 3.10 shows that if the simple static analysis were valid noticeable curvature would occur at these penetrations, supporting the suggestion that the dynamic analysis applies (with an appropriate cone factor including the rate correction described above). The difference between the dynamic analysis with buoyancy, and that represented by equation 3.5.1, is negligible in the likely range of penetrations if appropriate cone factors are chosen from figure 4.7. In the remainder of this thesis equation 3.5.1 is used to determine strengths, and cone factors $k_{60} = 0.38$ and $k_{30} = 0.38/0.32 = 1.10$ are applied.

4.4.2 Spherical cavity expansion testing

Figure 4.8 shows the entire test history of a particular cavity expansion test carried out in soil in column 115, 1100 hours after remoulding. The pressure at the tip of the needle is calculated from that measured at the external transducer applying the corrections described in chapter 3. The initial pressure, p_i , recorded as the needle is pushed through the rubber seal in the column wall, is high but immediately falls to a value below the hydrostatic pressure as the rubber seal returns to its original position and

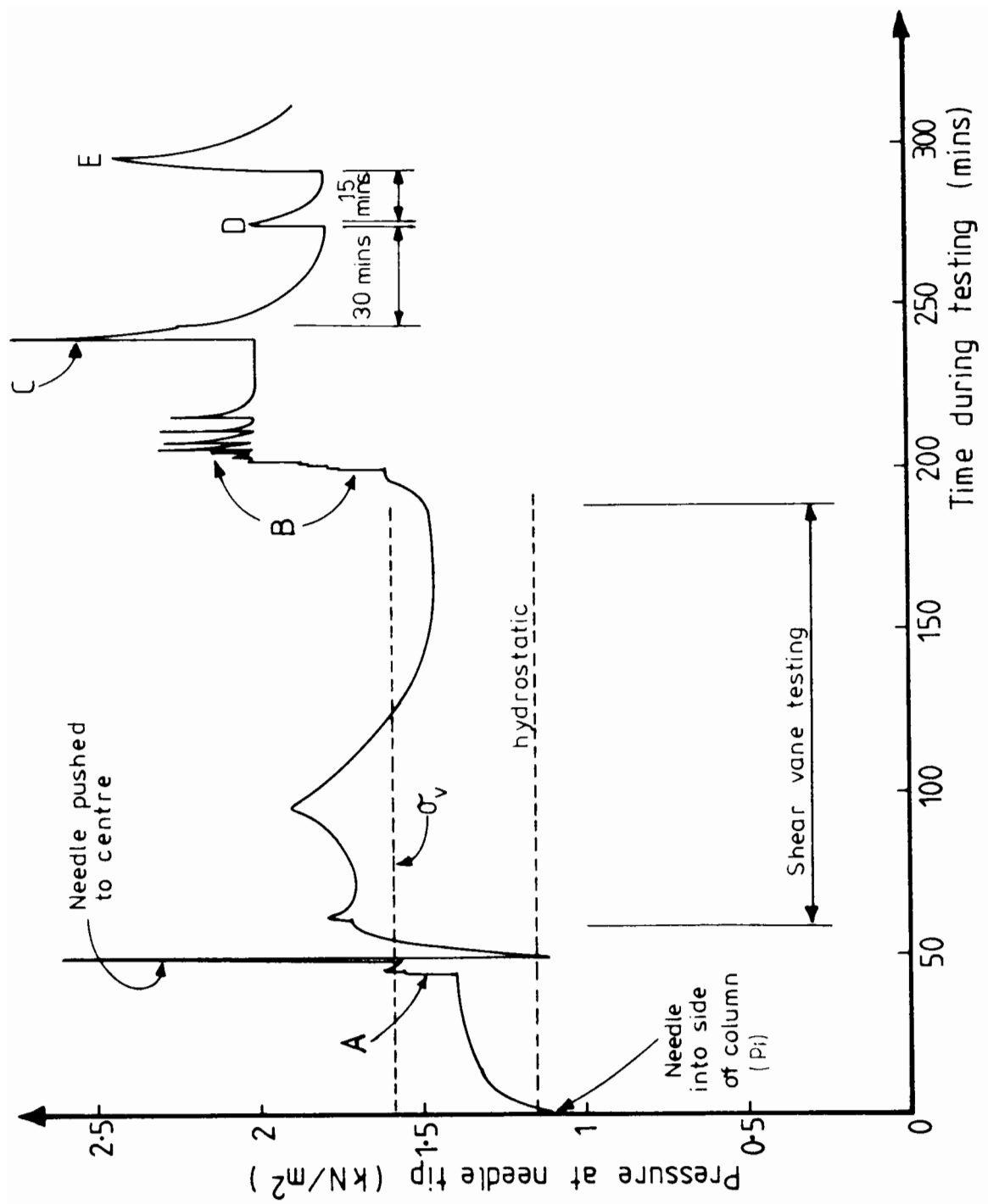
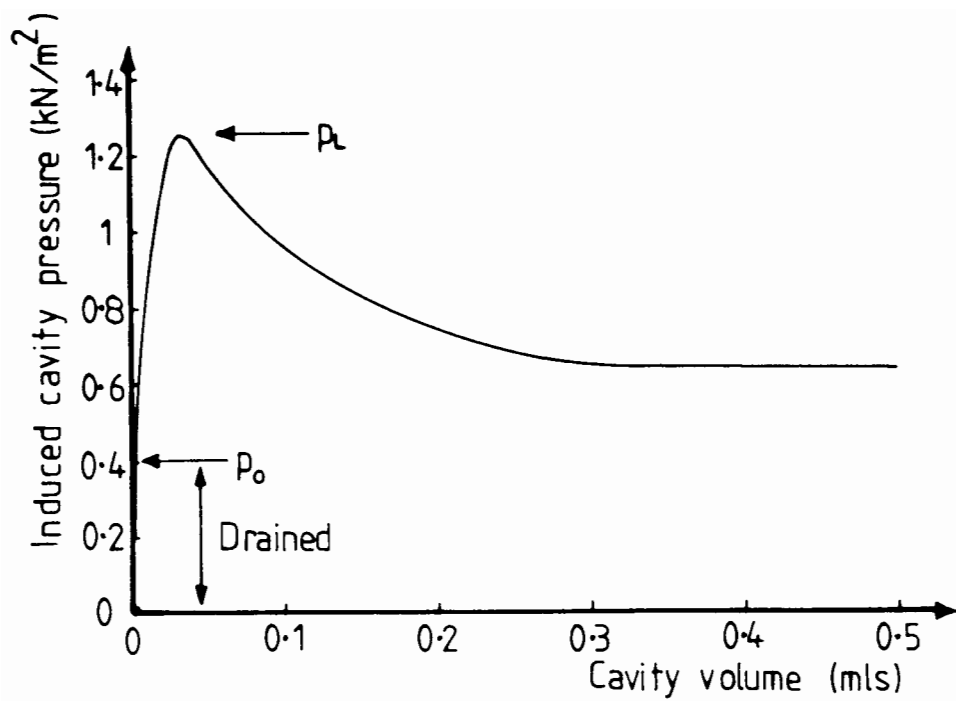
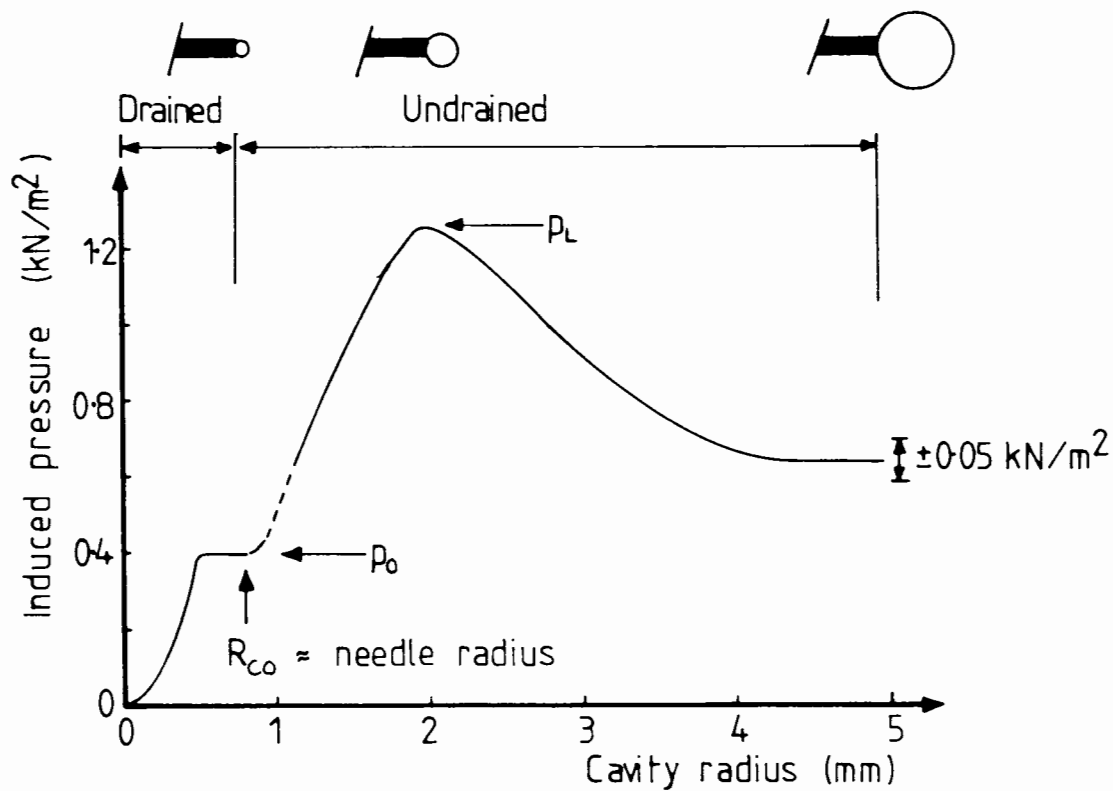


Figure 4.8 Typical cavity expansion test history, expt 115 (1100 hours).



(a) Cavity pressure - cavity volume.



(b) Cavity pressure - cavity radius.

Figure 4.9 Pressure curves for cavity expansion, expt. 115.

causes some local suction. Dissipation of the excess pressure following first insertion is therefore recorded as a pressure increase, and is nearly complete after 44 minutes. The change in pressure on insertion is sufficient to cause the silicon fluid interface to retract slightly into the needle as the apparatus expands. Prior calibration showed this retraction to be about 1.5 mm (3.5 mm^3). At step A this volume is regained by advancing the volume controlling piston in small increments. The needle is then inserted to the centre of the column at a steady rate during approximately one minute and pressures allowed to dissipate as before. It can be seen that the pressure achieved after 60 minutes (total time) is close to and greater than total vertical stress. Transient pressure fluctuations in the system during 20 minutes following handling (e.g. insertion) were, however, always of similar order to the excess total stress above hydrostatic and often higher, due to ambient temperature variations near the apparatus which were assumed to cause contractions and expansions of the fluid filled tubing. Reliable and steady values were not obtained until the room was sealed with lights turned off as light draughts caused by laboratory air conditioning units were reflected in the recorded pressures. Shear vane testing was carried out in the soil from 60 to 188 minutes, accounting for the pressure fluctuations during this time.

Following cessation of vane testing the tip pressure stabilised at 1.6 kN/m^2 , again close to the total vertical stress, although the timescale required in this plot somewhat disguises the stable value. At this stage, the fluid is level with the end of the needle and surface tension effects with high interface curvatures would allow relatively high pressure differences across the interface so that again this pressure value might not be representative of any pressure or stress acting in the soil. To obtain a reference cavity pressure before beginning undrained expansion, the cavity was expanded in 0.1 mm^3 increments, with full dissipation after each, up to a cavity volume of 2 mm^3 (step B), which occurs with a (spherical) cavity radius approximately equal to the needle radius (0.8 mm). The increase in pressure with cavity radius is

shown in figure 4.9b where it is seen that during this initial, drained phase a constant pressure was reached. The value of this pressure varied between tests in a manner which was apparently random but the value was often close to the vertical total stress, σ_v . In conventional (cylindrical) pressuremeter testing it is usually assumed that the cavity radius will not increase until the cavity pressure exceeds the existing total stress in the soil. It seems reasonable to assume that under fully drained conditions (as in step B) the cavity pressure at very small volumes will be close to the mean total stress in the soil, p_0 , provided soil is nearly isotropic. Since a preformed cavity strictly does not exist in the current situation the initial cavity radius and volume could be taken as zero, requiring infinite strains in the soil during expansion and suggesting that an ultimate pressure would be attained immediately. It is more likely however that a small zone ahead of the needle would be disturbed by the needle insertion particularly as the needle was observed to retract about 1 mm as the rubber seal of the wall relaxed following insertion. In this case the initial drained expansion during step B would merely cause the fluid filled cavity to expand in this disturbed zone.

Step C (in figure 4.8) involved increasing the cavity volume by injecting silicon fluid at a preset rate and measuring pressure changes. This sequence is shown in figure 4.9a where pressure~cavity volume behaviour is followed and in 4.9b, showing pressure changes as a function of the cavity radius. The pressure rises to a limiting value p_L at small cavity volumes. Figures 4.9 should be compared to figures 3.10a and 3.10b where examples of theoretical pressure curves are given. The peak pressure on the experimental curves is obtained at approximately twice the cavity radius, which is consistent with the theoretical value of $\Delta p > 90\%$ of Δp_L at similar radius. The experimental pressures in most tests decreased following the initial peak. Figure 4.10 shows most of the general forms of pressure~volume curve recorded in different tests. Consistent patterns were not detected within the series of tests undertaken. Several explanations seem possible for the anomalous post~peak

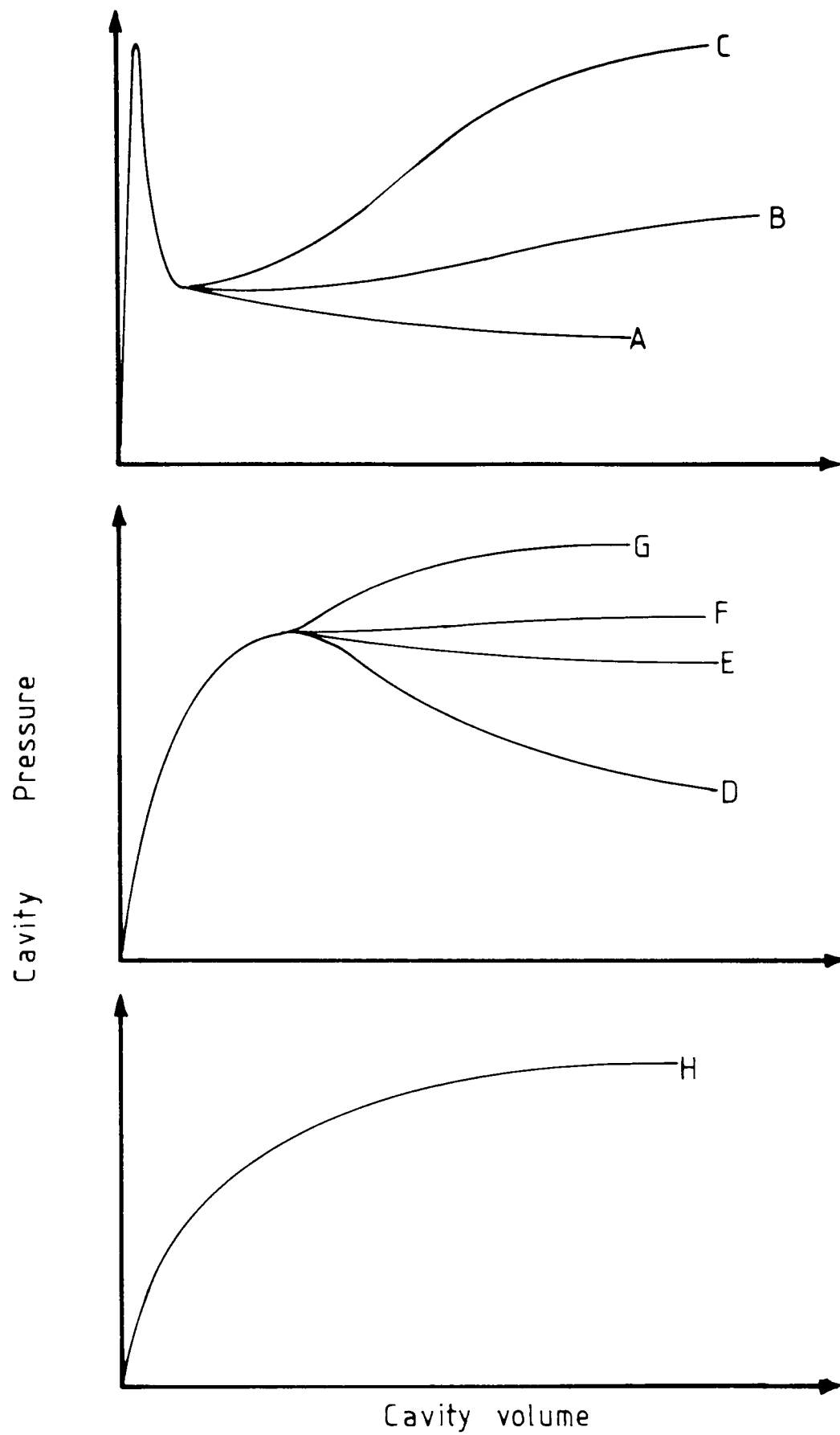


Figure 4.10 Types of pressure - volume curves recorded in cavity expansion tests.

behaviour. At very slow testing rates, drained behaviour (pore pressure dissipation) might occur. Since the cavities are expanded at constant volumetric rather than radial rate, radial displacement rates in the soil decrease progressively as cavity volume increases so that pore pressure dissipation might occur at similar or faster rate than cavity pressure increase. An alternative explanation is that fluid may be able to escape from the cavity, possibly along the needle. A third possibility may be that anisotropy of stresses and fabric in the soil cause cavity behaviour to deviate from the ideal isotropic conditions postulated in chapter 3. The cavity itself might not expand spherically under these conditions and eventually the cavity fluid might disperse over some plane of failure, a mechanism assumed in hydraulic fracture tests. Each of these three mechanisms could cause the post peak pressure drop observed in curve types A,B,C,D,E. Any explanation for cases where the pressure subsequently increased again would be excessively speculative and is not attempted here. Curve type H is closest in shape to that predicted theoretically in chapter 3 but was observed in very few tests. It was not possible to evaluate these explanations with the apparatus used in these experiments although intact, fluid filled cavities of the approximate spherical dimensions expected were found in all experiments when soil was removed from the columns.

Although it was not possible to determine a consistent pattern in the value of the steady pressure, p_o (defined in figure 4.9), following drained expansion to a cavity radius equal to the needle radius, the pressure difference $p_L - p_o$ did appear to follow consistent trends with soil density and age. This difference was therefore used to determine the undrained strength in spherical cavity expansion tests, s_{us} , according to the theory presented in chapter 3. It is noted that if p_o as measured above does represent the mean total stress in the soil, the coefficient of earth pressure at rest, K_o , could be calculated if pore pressures are known. Values of p_o in these experiments varied from slightly above hydrostatic to approximately two times the excess

total vertical stress so that K_0 might vary between 0.4 and 1.7. Although K_0 values have not been reported elsewhere for such soft soils it seems unlikely that such wide variations would occur between similar soil states. Further investigations are needed to determine the reasons for variation in p_0 . Large soil samples consolidated isotropically and symmetrically around an existing "cavity" in the form of a large fluid filled membrane (radius > 10 mm) will probably be required.

Following expansion C in figure 4.8 to 0.5 ml total volume (5 mm radius) pressures were allowed to dissipate for 30 minutes before expansion D to 0.6 ml volume. A final expansion (E) to 1.0 ml volume was carried out after 15 minutes further dissipation. In view of the uncertainty regarding earlier cavity behaviour, results of these later tests are not analysed in this study.

4.5 Results of tall column experiments

In this section surface settlement, continuous (X-ray) density profiles, pore pressure and stress profiles and basic data from testing at the end of each experiment are presented individually. Comparisons are made where initial densities or solids masses are similar.

Figures 4.11 and 4.12 show settlements with time and final density profiles for experiments 1a and 1b, deposited at initial density, $\rho_i = 1.02 \text{ Mg/m}^3$ as fine suspensions of solids. Experiment 1a allowed surface drainage only but 1b drained from the base through a tube to the free water surface. Density profiles after 940 hours are almost identical, allowing for a slight difference in solids amount present in each. Settlements are also similar at all times. The analytical large strain consolidation solutions obtained for the linear soil model by Lee (1979) and modified in Been & Sills (1981) were extended to predict settlements under doubly drained conditions. The analysis is not presented here, but input parameters given in figure 4.12 correspond to those defined by Been & Sills and were obtained from experiment 1a by fitting a theoretical curve to settlement data as indicated in figure 4.12. Predicted

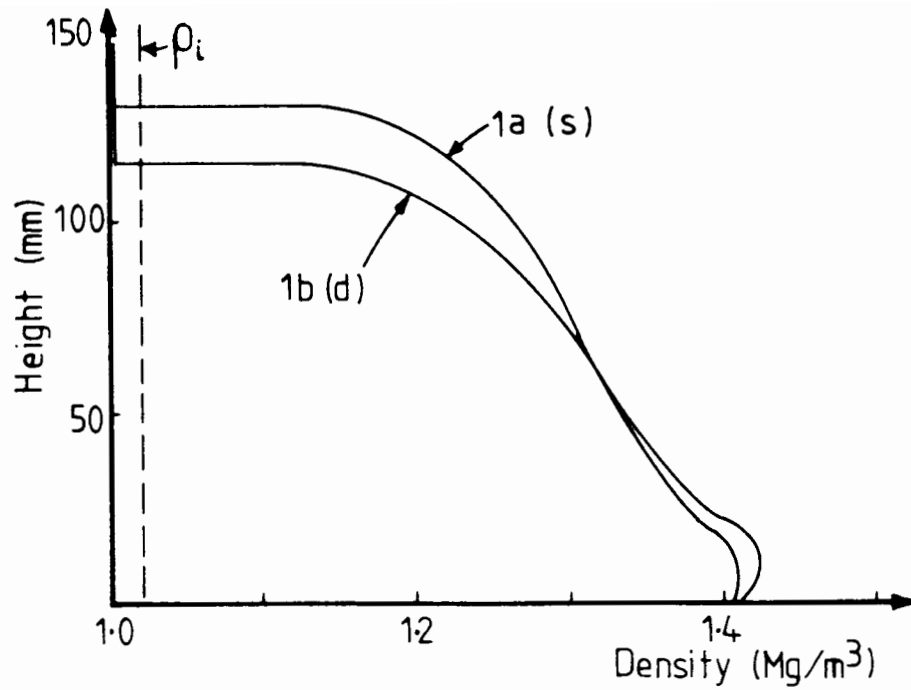


Figure 4.11 Density profiles at completion of consolidation (940 hrs) for expts. 1a and 1b, $\rho_i = 1.02 \text{ Mg/m}^3$.

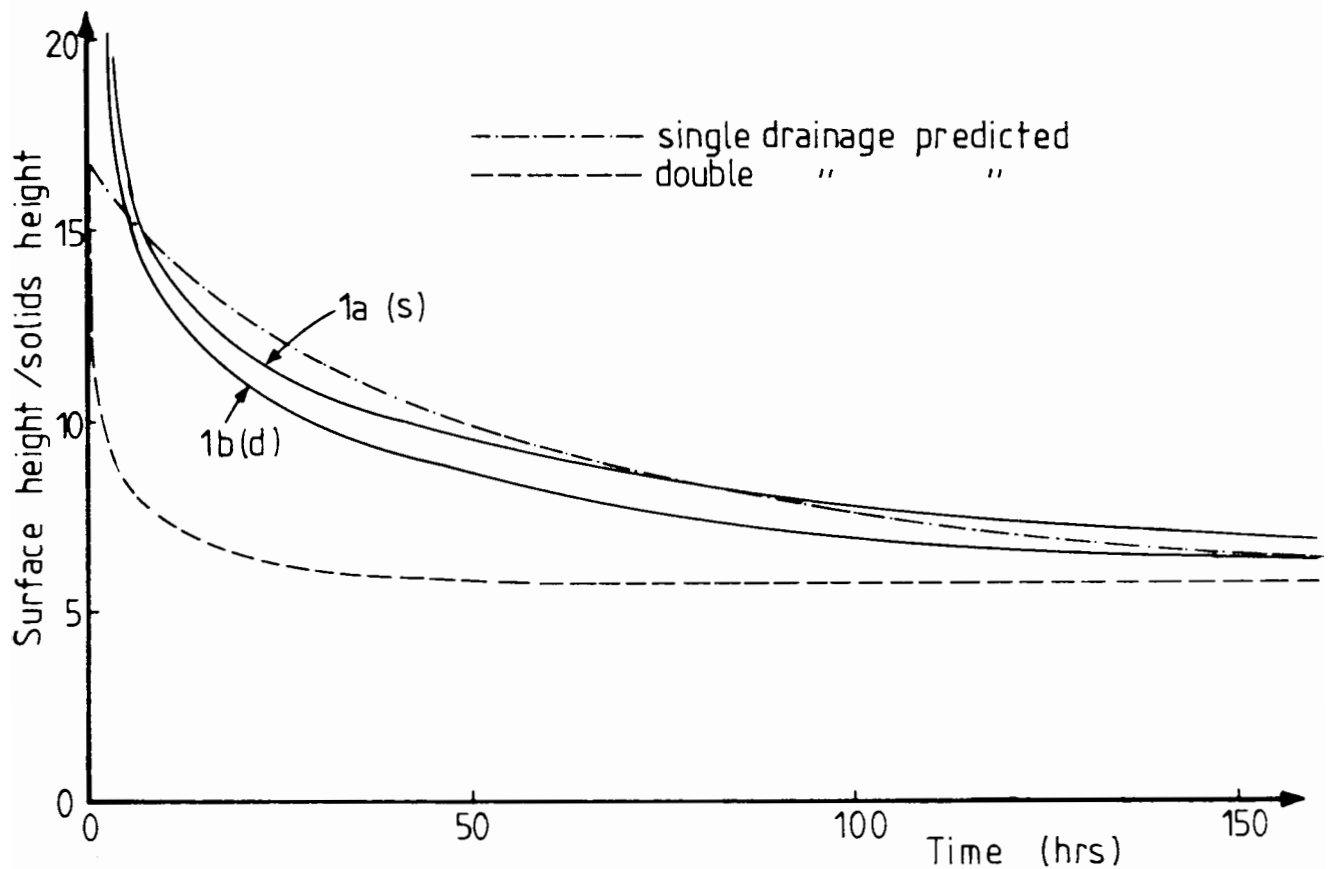


Figure 4.12 Comparison of observed and predicted settlements under single and double drainage conditions using modified linear theory ($e_{so} = 15.8$, $e_{sf} = 6.2$, $\beta = 1.39 \text{ cm}^{-1}$, $C_f = 1.7 \times 10^{-4} \text{ cm}^2/\text{s}$). Parameters defined by Been and Sills, (1981).

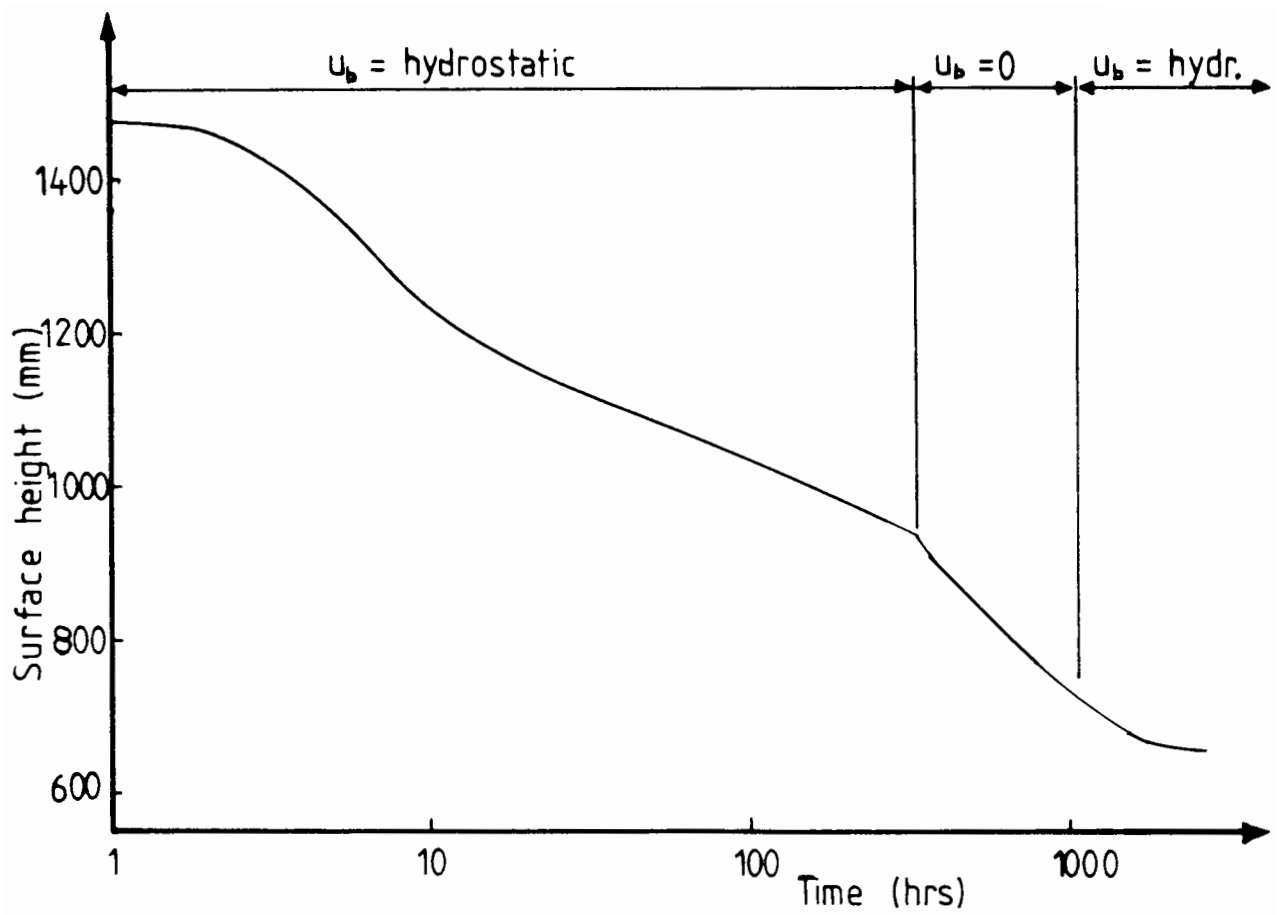


Figure 4.13 Surface settlement expt 3, $P_i = 1.174 \text{ Mg/m}^3$.

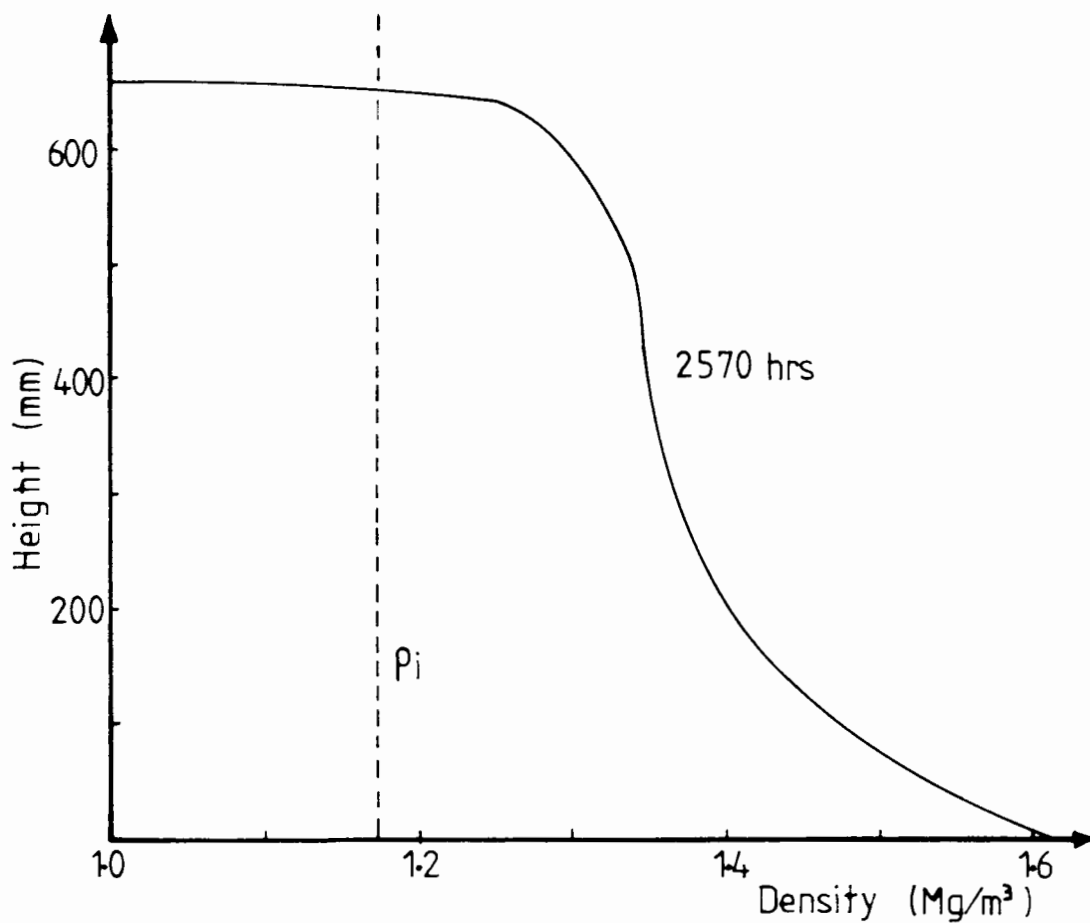


Figure 4.14 Density profile at completion of consolidation, expt 3.

settlements for double drainage are much faster than those observed; the actual advantage gained by the provision of base drainage appears to be minimal.

Figures 4.13 and 4.14 show settlements and final densities for experiment 3, $\rho_i = 1.174 \text{ Mg/m}^3$, in which a suspension was never present. Base drainage was provided to the free water surface during the first 330 hours of consolidation. At this time the drainage tube was lowered below the column base so that the pore pressure at the base was zero. The settlement rate increased immediately, but decreased only slightly when the tube was replaced at the free surface after 1080 hours suggesting that the high hydraulic gradient at the base had only a small effect at this stage. Consolidation continued until 1800 hours but slowed markedly thereafter. A reason for the reduced effect of the free base drainage may be the relatively thin layer of high density near the base; although local hydraulic gradients were high the decreased permeabilities would reduce the rate of flow.

Experiment 4 ($\rho_i = 1.235 \text{ Mg/m}^3$) was allowed to consolidate under self weight for 331 hours then subjected to surface loads in three successive increments up to maximum $\sigma_s = 14 \text{ kN/m}^2$.

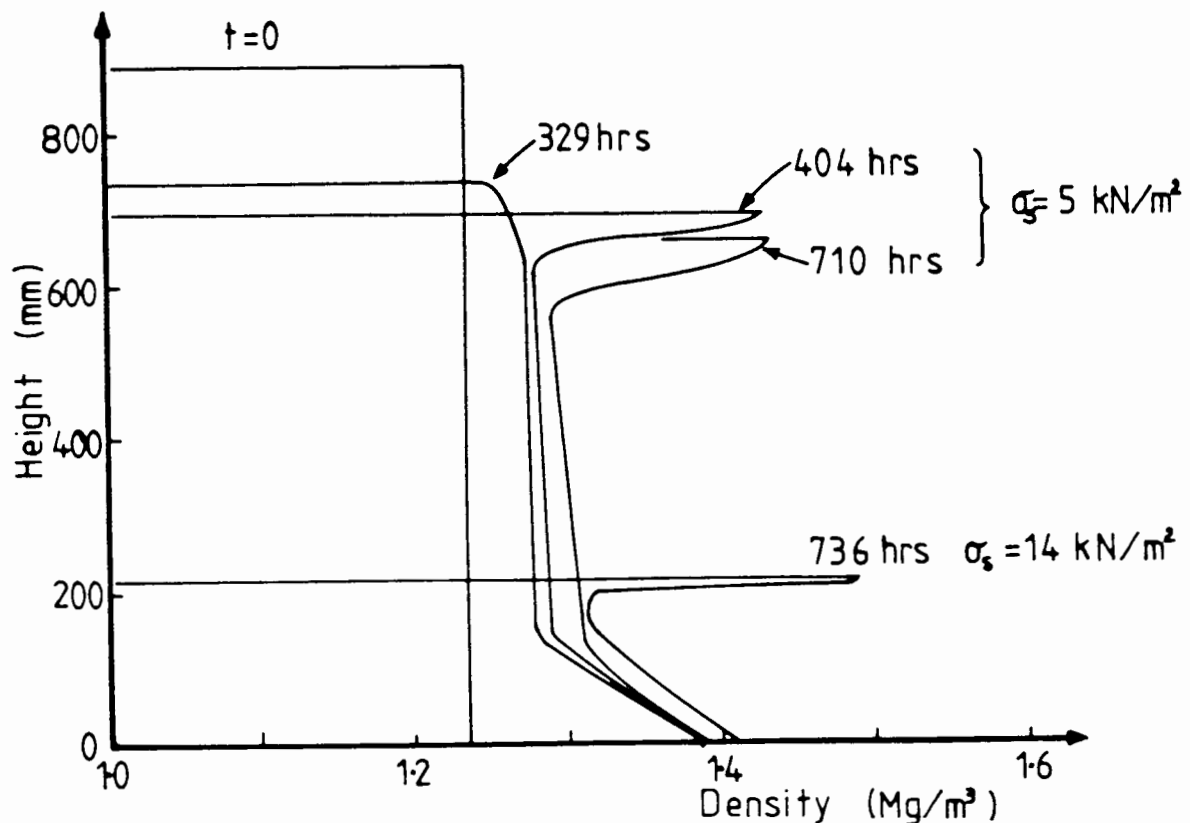


Figure 4.15 Density profiles expt. 4, $\rho_i = 1.235 \text{ Mg/m}^3$. Single drainage.

Density profiles at the end of each increment are shown in figure 4.15 for the soil only; the surcharge layers (up to 1 m high) are excluded. As in experiments 3 high density layers form near the drainage boundary, in this case at the surface, when total stresses are high. These inhibit consolidation lower in the soil, where densities remain close to those before surcharge application. It should be noted that before the third loading increment at 717 hrs soil was removed to a height 228 mm above the base so that the area under the density profile is no longer equal to that under the other profiles.

4.5.1 Experiment 5, $\rho_1 = 1.21 \text{ Mg/m}^3$

This was the first experiment in which segmentable sections were used enabling full testing at the end of consolidation. Figure 4.16 shows surface settlement during three phases of drainage, which from 6 to 170 hours was permitted to hydrostatic pressure. Comparison with experiment KB8 (Been, 1980) suggests that little advantage was eventually gained from the double drainage period since the settlement plots merge after about 500 hours. Dense boundary layers are again seen near the base in figure 4.17a and the pore pressure profiles at corresponding times (4.17b) confirm that throughout most of the column drainage remained in the upward vertical direction despite the high gradients near the base. Samples were taken for particle size analysis at four heights during unloading and the distributions at each height are seen to be identical in figure 4.18, confirming that no particle size segregation had occurred. Moisture content samples were taken at several positions in horizontal cross sections following vane tests at each height and results are shown in figure 4.19, where the height axis is intersected by the average void ratio at each section. Higher void ratios at the column centre suggest some differential swelling has occurred between removal of overlying soil and sampling. Although void ratios of samples taken within 10 mm of the surface at the centre and sides differ by up to 10%, changes in soil surface height were not observed to be greater than 1 mm suggesting that swelling was probably confined to the region nearest the surface. Excess pores pressures in expt.5

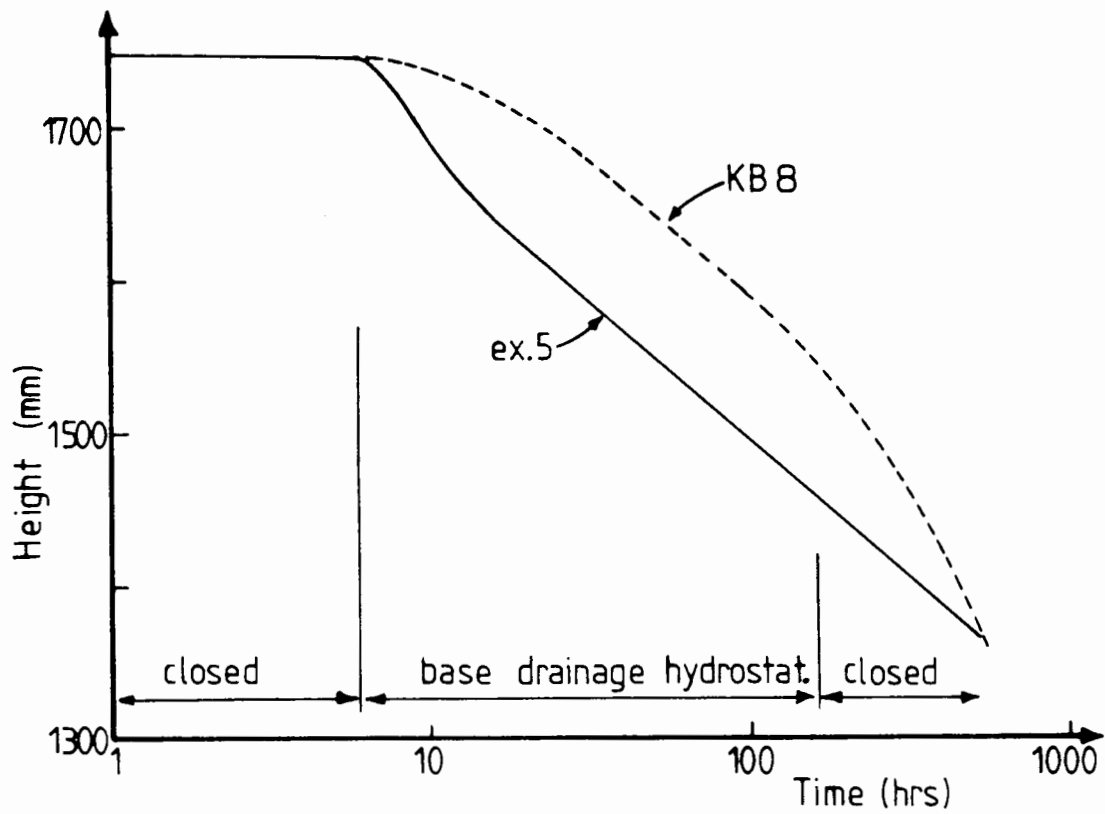


Figure 4.16 Surface settlement, expt 5, $\rho_i = 1.21 \text{ Mg/m}^3$, under variable drainage conditions. Also shown, KB8 ($\rho_i = 1.22 \text{ Mg/m}^3$) surface drainage only.

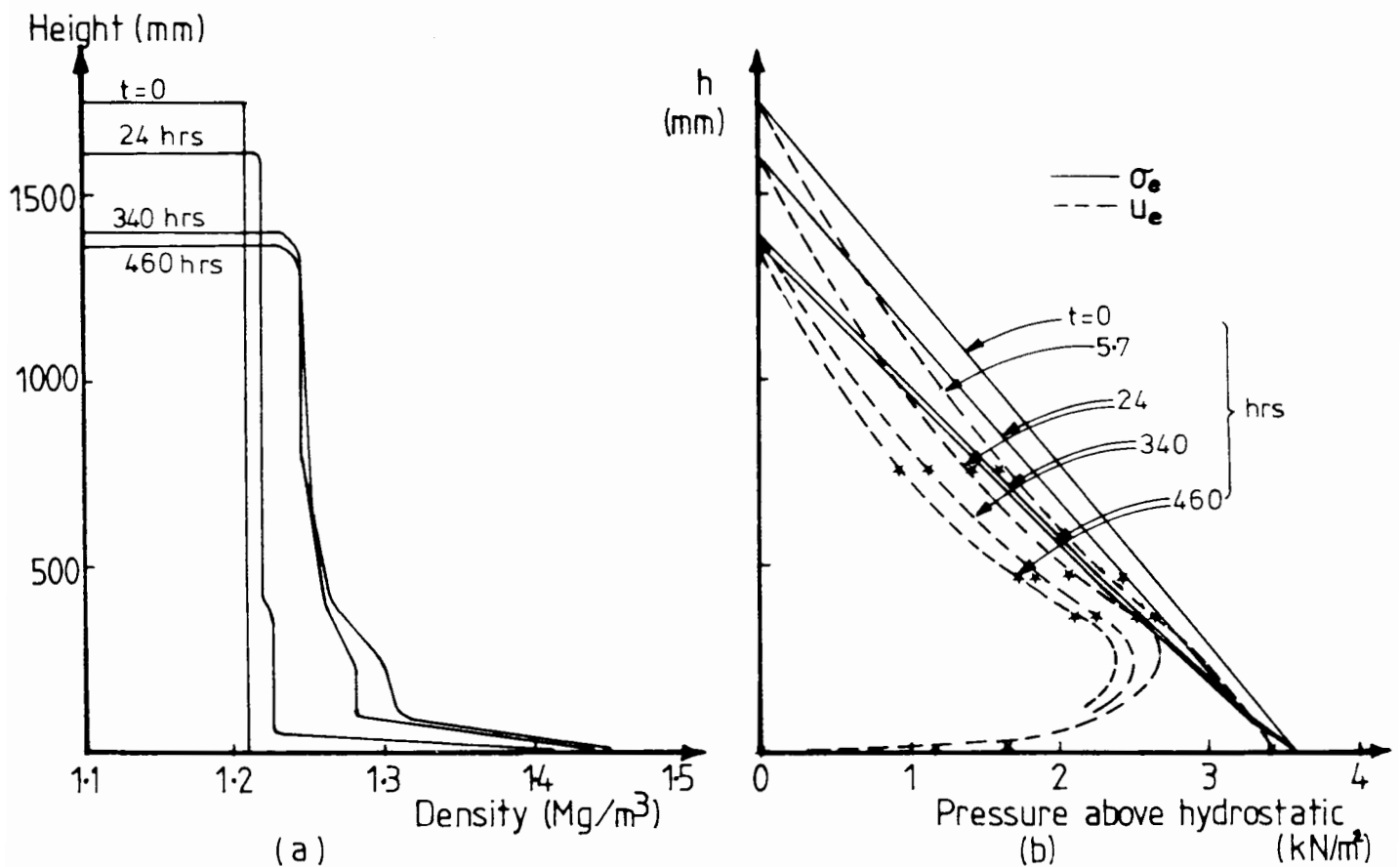


Figure 4.17 Expt 5. Profiles of (a) density and (b) total stress and pwp at different times during consolidation.

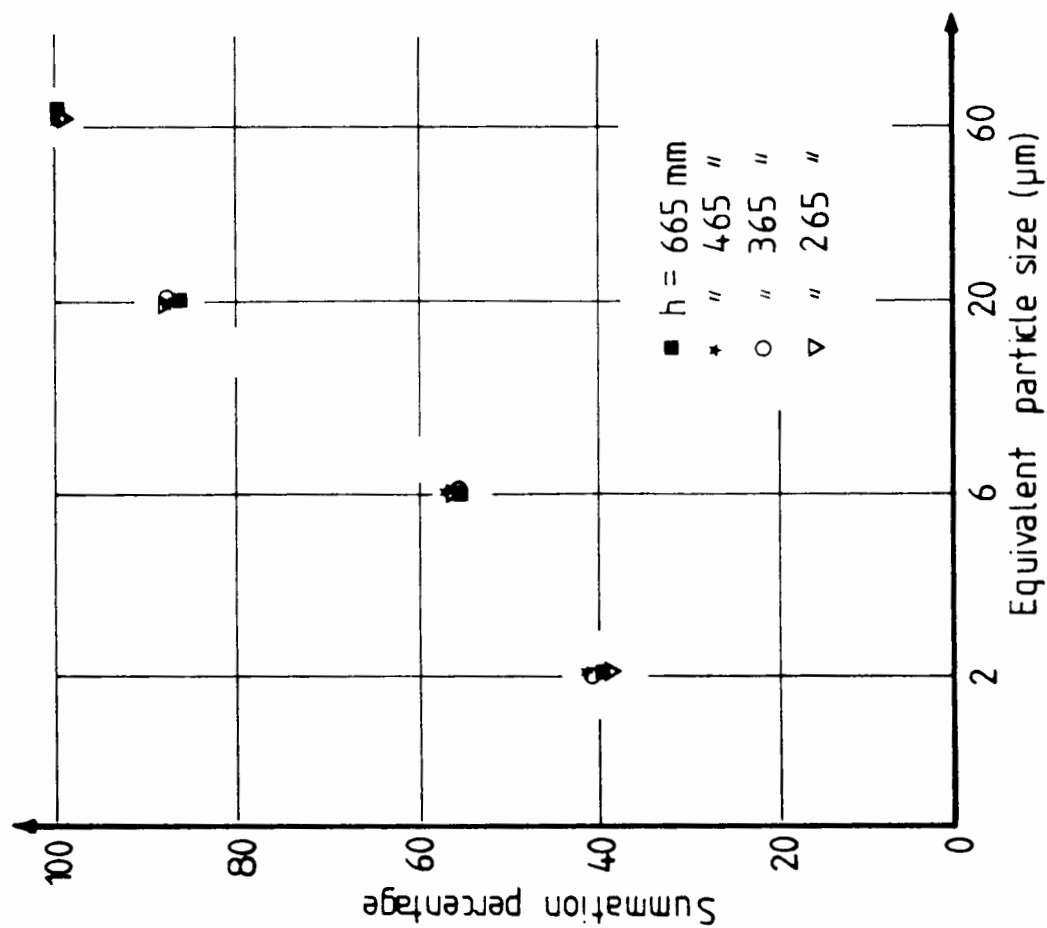


Figure 4.18 Particle size distributions at different positions in column after testing. Expt 5.

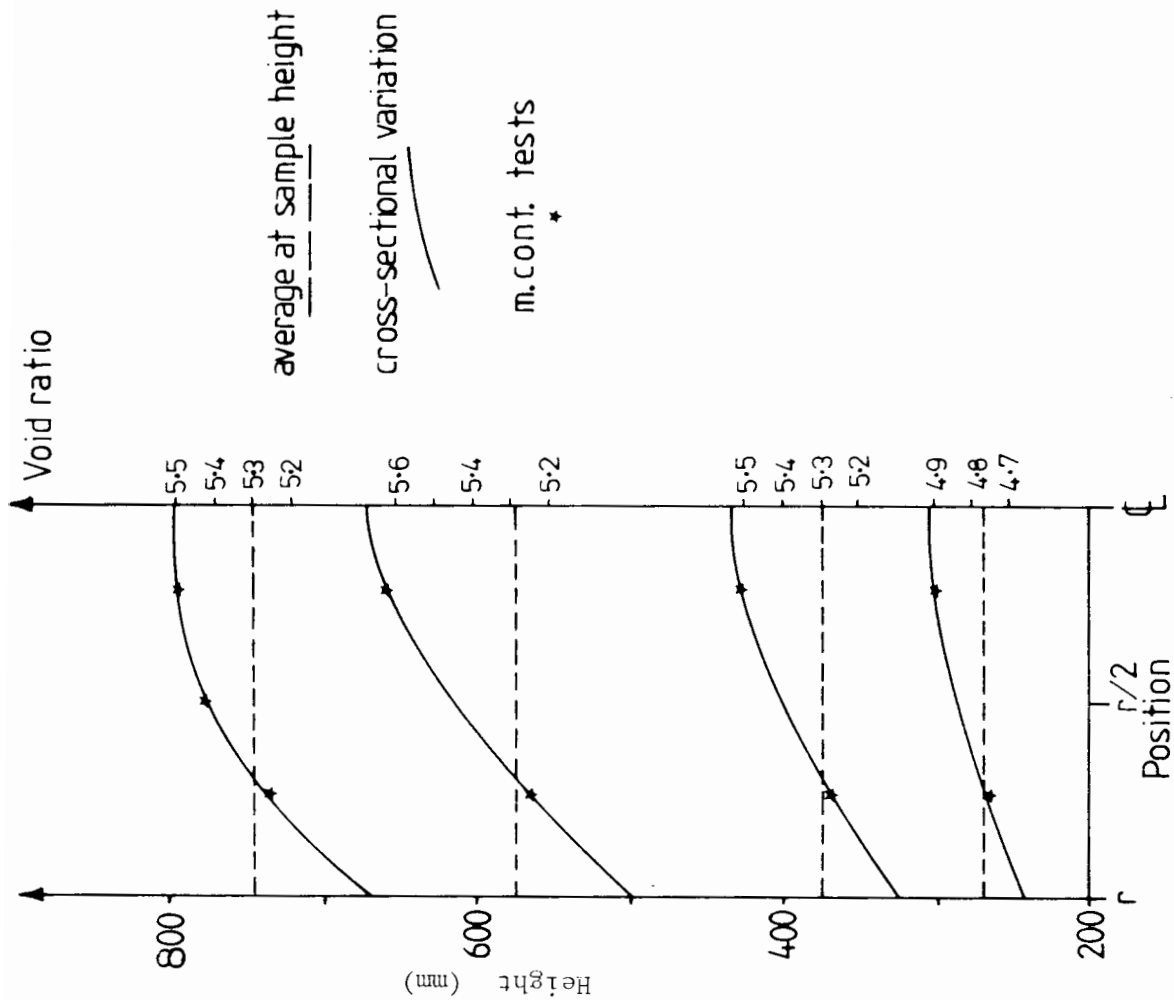
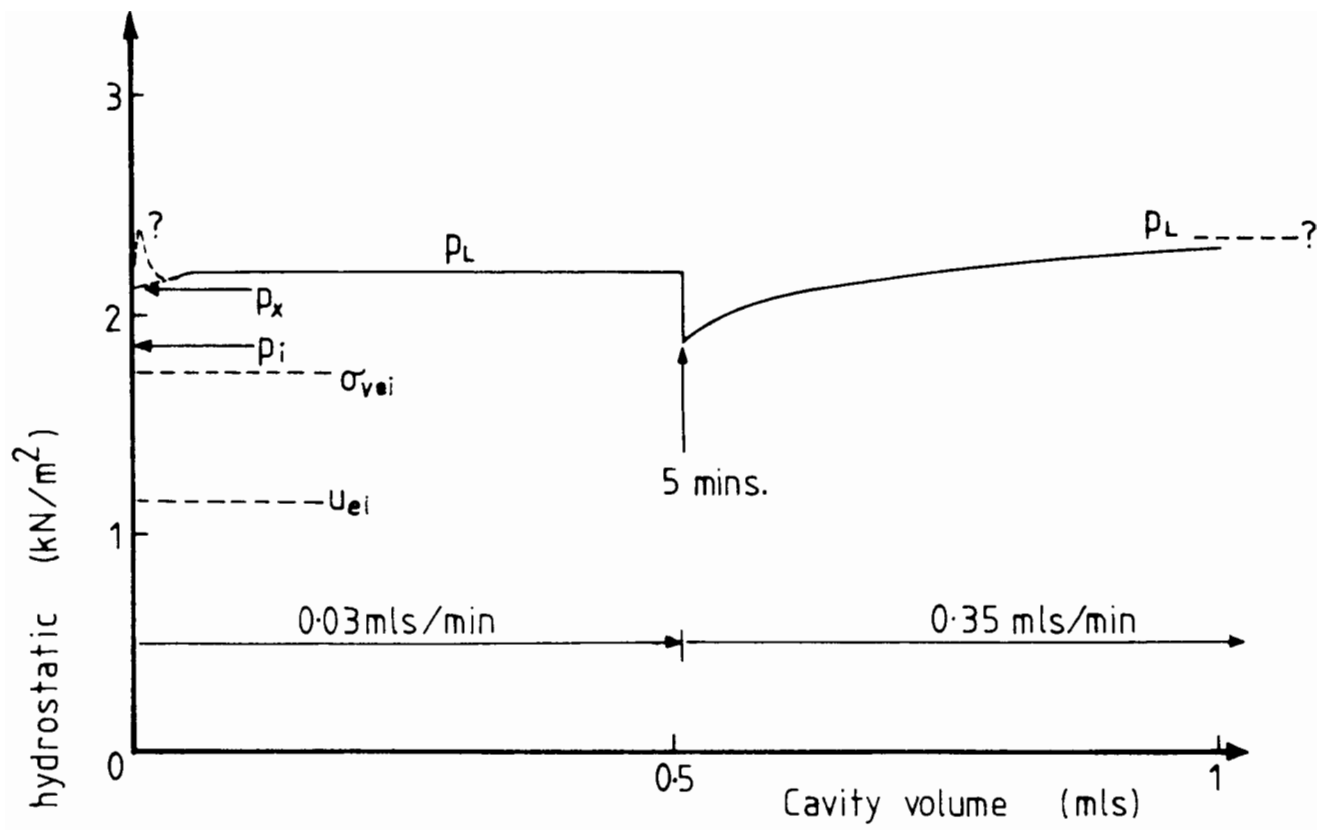


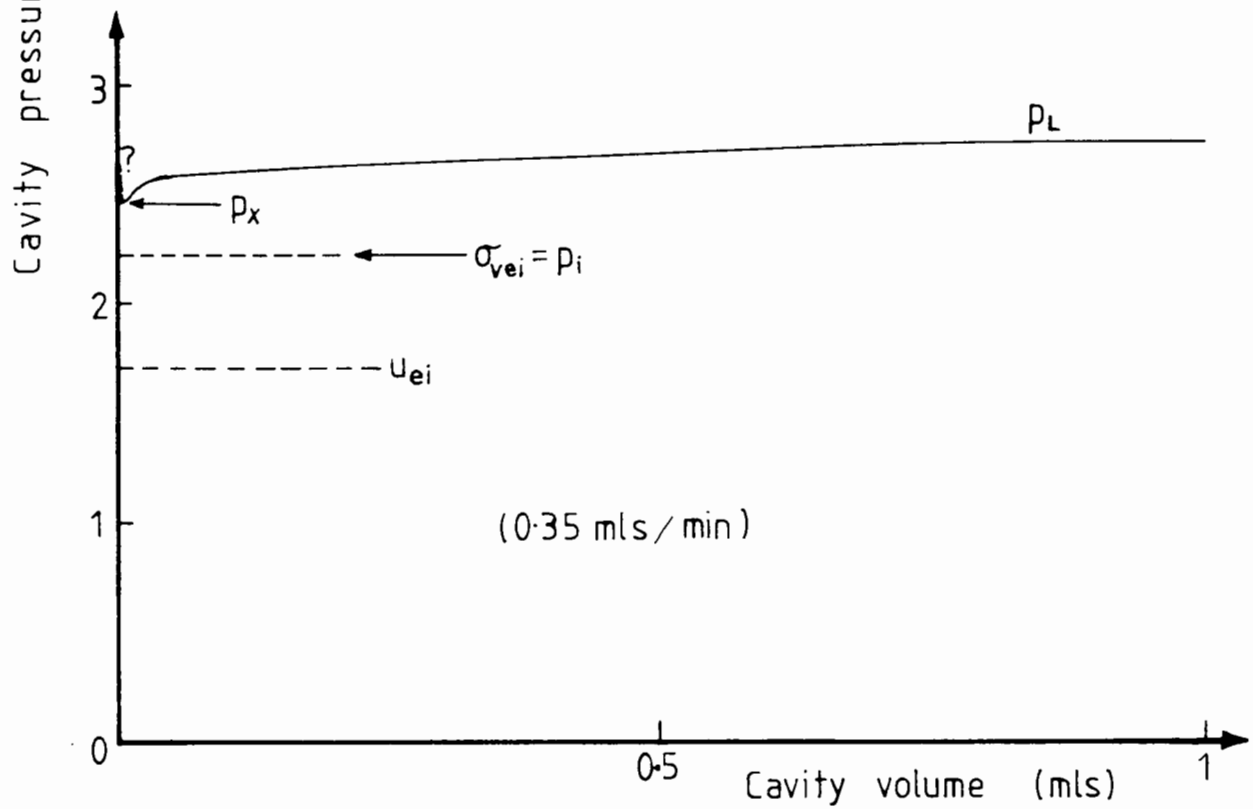
Figure 4.19 Cross sectional density variation after unloading. Expt 5.

were still high at the time of unloading and densities close to the initial density; in later experiments where pore pressures were almost fully dissipated at the time of testing relatively little cross-sectional density variation was observed.

Results of cavity expansion tests carried out between 360 and 430 hours are shown in figures 4.20a to d, where cavity pressures (above hydrostatic) are plotted against cavity volume. Existing pore pressure (u_{ei}) and total stress (σ_{vei}) levels are also shown and the initial cavity pressure measured after insertion and dissipation is in all tests close to σ_{vei} . Due in part to the larger hydrostatic pressure which has to be subtracted from measured pressures, expansion results are rather more difficult to interpret than those for the short column tests discussed in the previous section. Each test showed a sudden pressure increase as expansion began which dropped immediately and then increased more slowly to a limiting value. The initial peak occurred at a cavity radius close to the needle radius and was possibly due to the fluid inertia. The limiting pressures occurring at greater cavity volumes are used for undrained strength determination although the justification for this choice is somewhat arbitrary. In each test it would also be possible to estimate a pressure corresponding to expansion initiation and these values are indicated, but are subjective only as actual values are obscured by the initial pressure response. In the test at $h=665$ mm (figure 4.20a) pressure was recorded after expansion was ceased at 0.51 ml volume, and had decreased to a fairly stable value equal to the initial insertion pressure after 5 minutes. This behaviour is slightly surprising in the confined columns and might suggest that a horizontal failure plane existed so that the dissipated cavity pressure corresponded to the total vertical stress. Again this hypothesis was not borne out by examination of the cavity during unloading, when it appeared to be spherical and intact. During the expansion test, pore pressure changes in the soil were recorded at the column walls by transducers at the cavity heights. These are shown in figure 4.21, normalised by the current cavity pressure. In



(a) $h = 665\text{mm}$.



(b) $h = 465\text{mm}$.

Figure 4.20 Pressure changes in spherical cavities, expt 5.

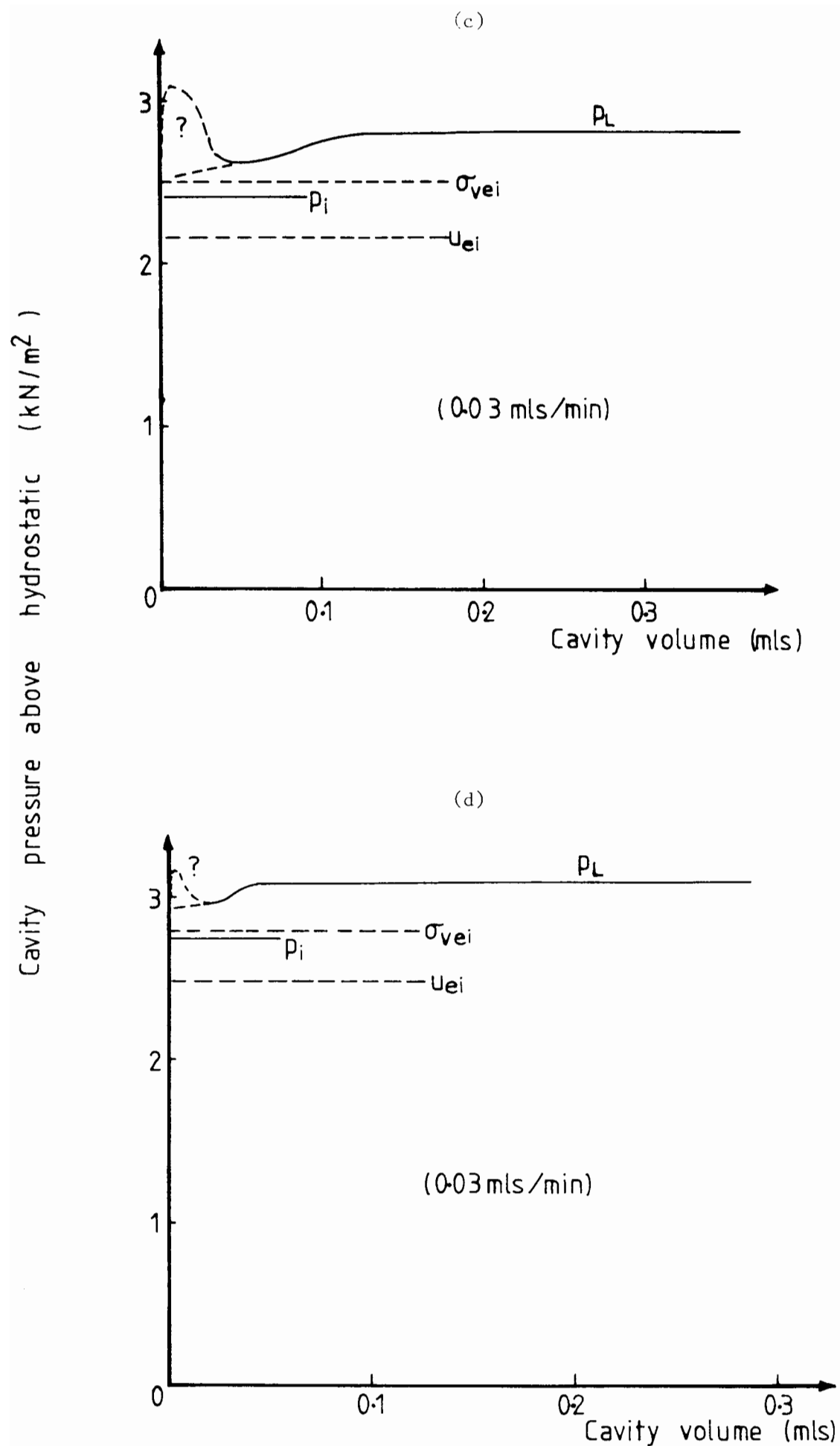


Figure 4.20 Pressure changes in spherical cavities, expt. 5.

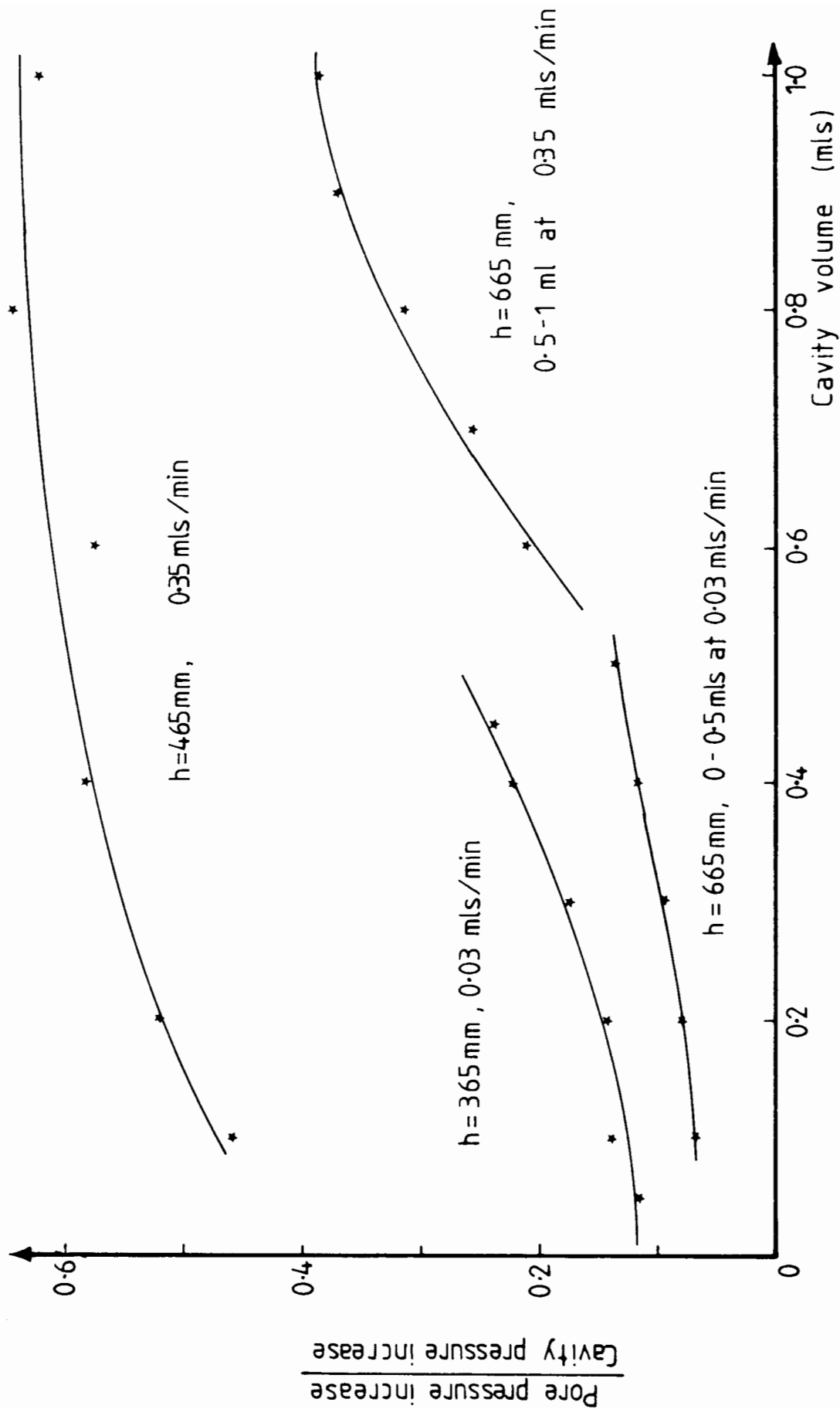


Figure 4.21 Changes in pore pressure at wall of column adjacent to expanding cavity, expt.5.

all tests pore pressure continue to increase after limiting cavity pressures have been reached. The analysis in chapter 3 predicts this behaviour as the plastic zone around the cavity propagates outwards through the soil. Data are not sufficiently accurate for determination of the moment of yield, when the plastic zone reaches the column wall, from changes in the pore pressure response. Higher normalised pore pressures occur for the faster expansion rate used at $h = 465$ mm. This suggests that some drainage occurs at larger radii at the lower rates since predicted pore pressure distributions are independent of expansion rate.

Results of shear vane tests are shown in figures 4.22 and peak and residual values are used to obtain the shear strength profiles with the soil in figure 4.23b. "Static" shear strengths, obtained from resisting torques shortly after rotation was ceased in each test, are also plotted although the rapid decay in torque exerted on the stationary vane leads to considerable scatter. Static values are non-zero and slightly below residual values indicating the importance of rotation rate but also that measured resistances are not due solely to soil viscosity. Values of the shear modulus:undrained strength ratio G/s_u are calculated from the vane test results at peak resistance as proposed in chapter 3

$$G/s_u = \frac{1}{2\theta} \quad (\theta \text{ in radians}) \quad (3.6.7a)$$

and are used with equation 3.7.19 to calculate cavity expansion undrained strengths which are also plotted in figure 4.23b. The corresponding effective stress and liquidity index profiles are shown in figure 4.23a. The shape of the effective stress profile is due to the incomplete consolidation at the time of testing, with a marked increase near the base, at which drainage had been permitted. The vane residual strength increases slowly with depth, in line with the decrease in liquidity index, but the peak strength increases quite rapidly. Cavity expansion strengths are greater than vane strengths at both expansion rates.

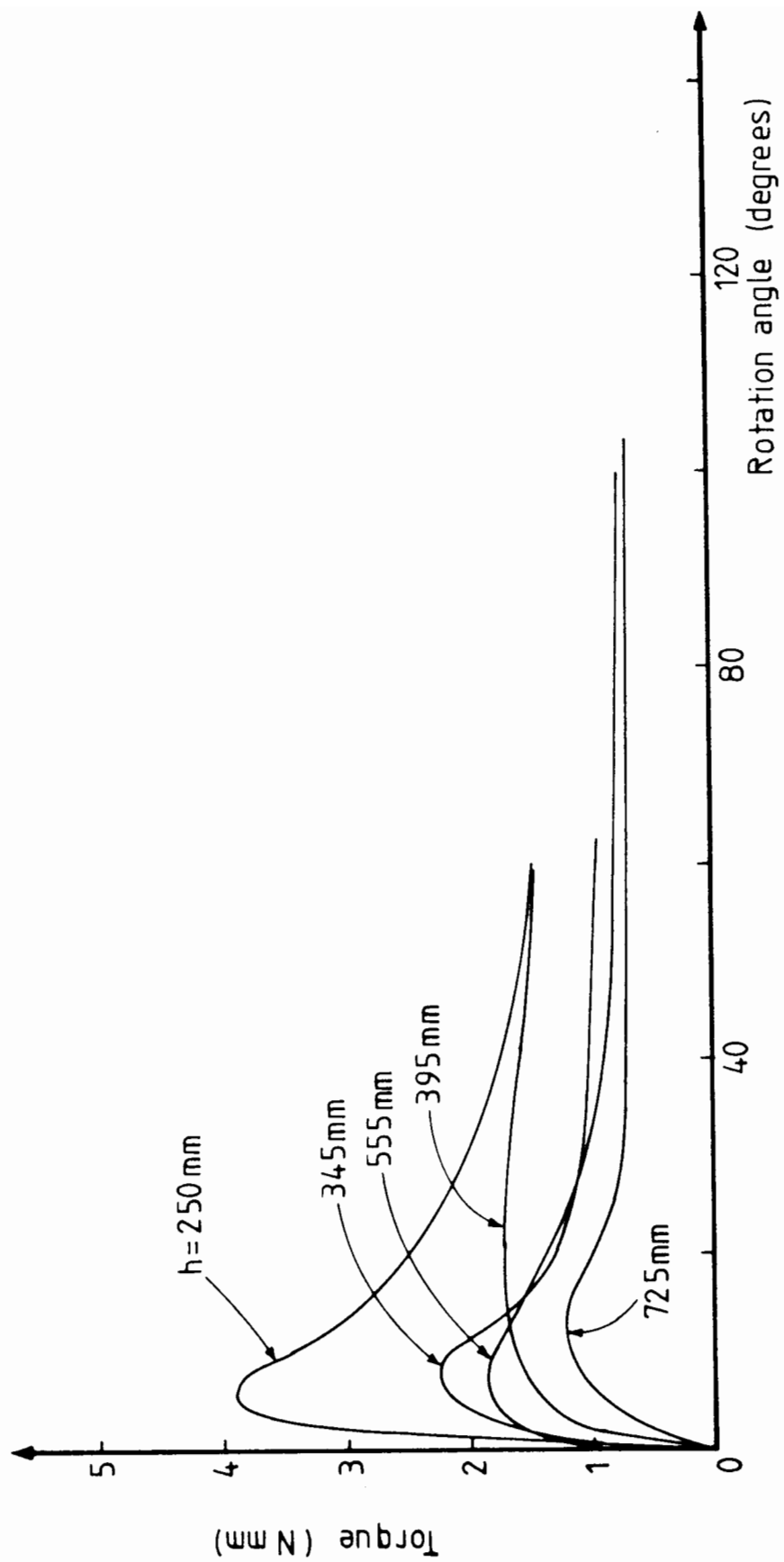


Figure 4.22 Vane tests at 60°/min in expt 5 after 520 hours.

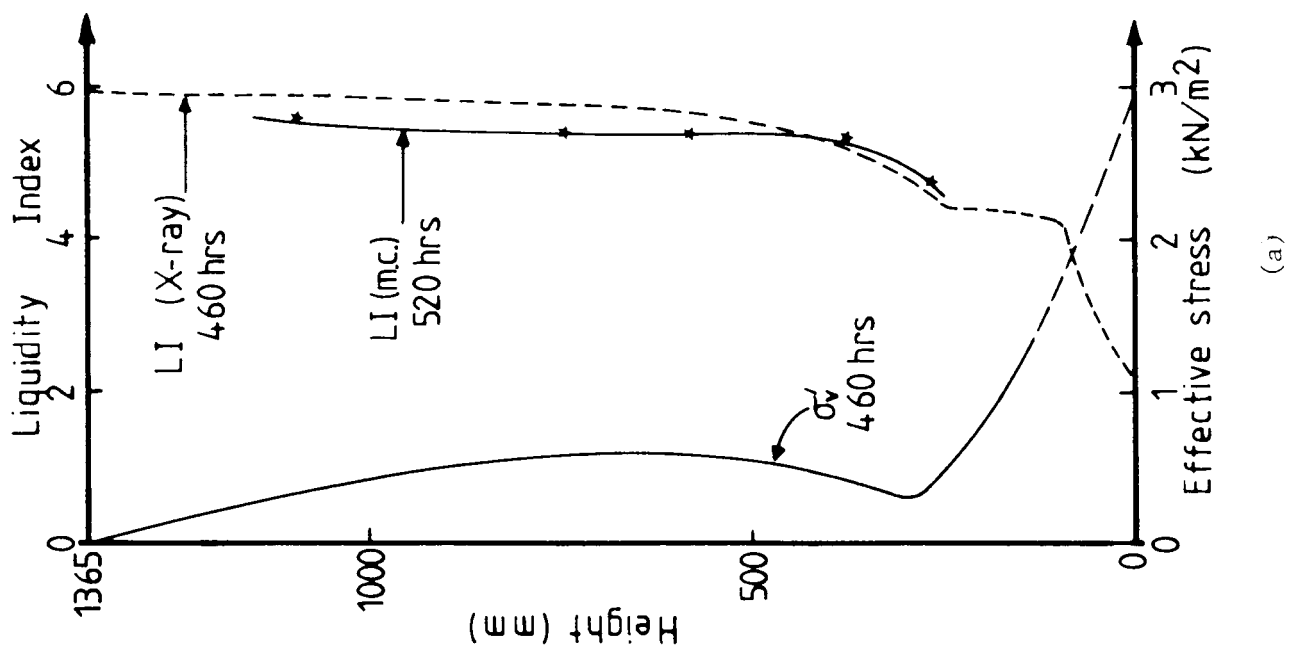
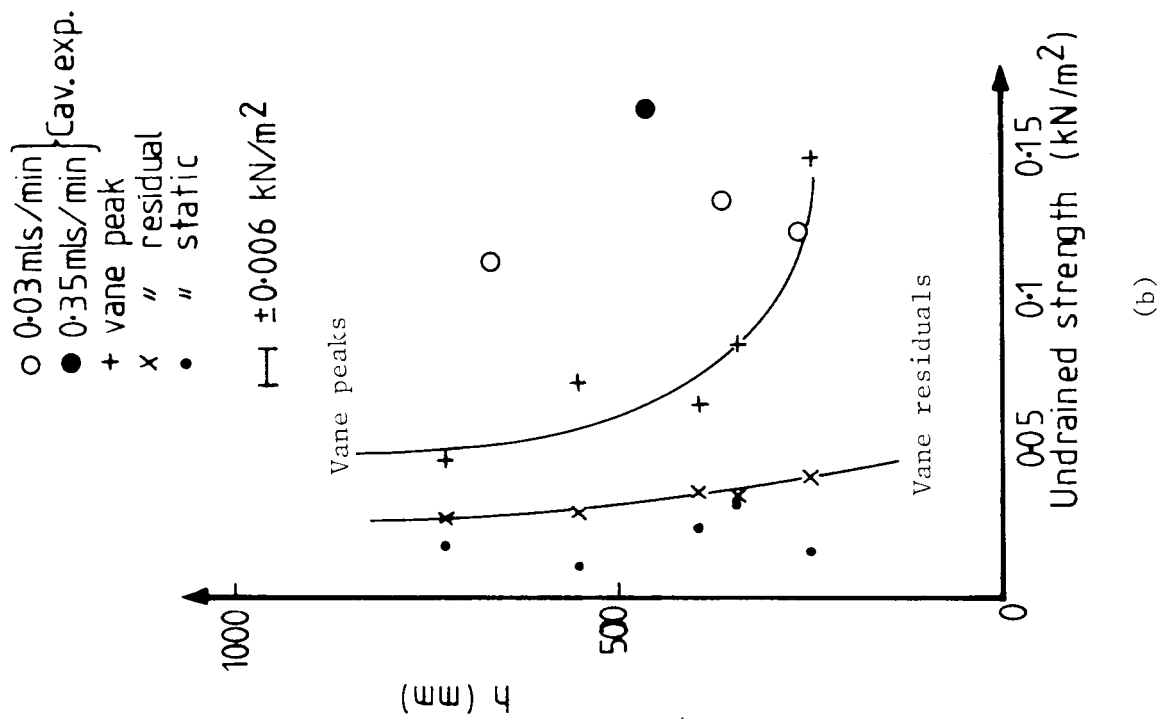


Figure 4.23 (a) Stress, liquidity index and (b) strength profiles with depth, expt 5.



4.5.2 Experiments 6, 7, and 9

Figures 4.24 and 4.25 show settlements and density profiles for three experiments with the same initial density (1.25 Mg/m^3) but different initial heights (experiment 6, 1748 mm; experiment 7, 1195 mm; experiment series 300, 200 mm) and two experiments with similar solids mass but different initial density (experiment 7, 3.9 kg, 1.25 Mg/m^3 and experiment 9, 4.1 kg, 1.09 Mg/m^3). Only experiment 9 began as a suspension. Experiments 6 and 7 were doubly drained although it has already been shown that this does not markedly accelerate consolidation. Comparison of experiments 7 and 9 in figure 4.25 suggest that segregation of particle sizes was significant in experiment 9, causing high densities near the base while densities near the surface remained much lower than at similar time in experiment 7. Profiles for experiments 6 and 7 are similar in shape but densities are higher in experiment 7 due in part to the shorter drainage paths. Pore pressure profiles for experiments 6 and 7 (figures 4.26, 4.27) confirm that dissipation of excess pressures was considerably faster in the shorter column and again it is seen that drainage upwards was dominant. Dissipation in experiment 7 was virtually complete after 4300 hours but further density increase occurred uniformly throughout the soil height up to 11,000 hours at near constant effective stress. Series 300 experiments, even after only 2800 hours, attained higher densities than either experiment 6 or 7. Pore pressures were measured in the short columns using an inserted probe; dissipation was almost complete after 2800 hours and the uniform density increases thereafter and up to 6200 hours must be attributed to creep at constant effective stress.

Figures 4.28 show results of cavity expansion tests, including pore pressure changes, in experiment 6 after 5000 hours. All tests were at expansion rate 2 ml/minute . Interpretation is again subject to the uncertainty discussed for tests in experiment 5. The initial stable excess cavity pressure p_i registered on insertion is in all tests closer to the existing pore pressure, u_{ei} than to the total stress obtained by density profile integration. The possibility exists that the initial pressure registered is in fact the pore

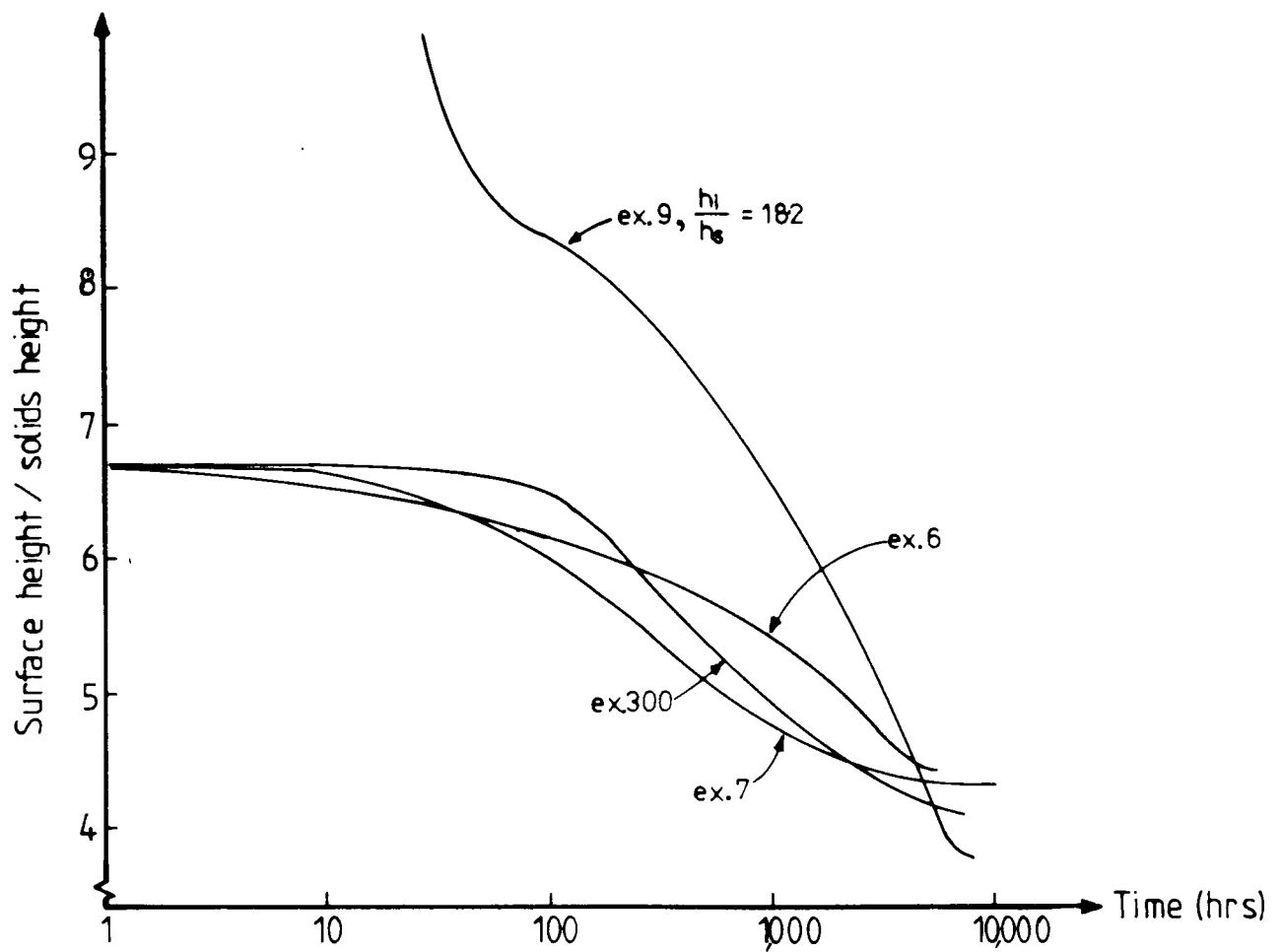


Figure 4.24 Normalised surface settlements with time, expts. 6, 7, 9, 300.

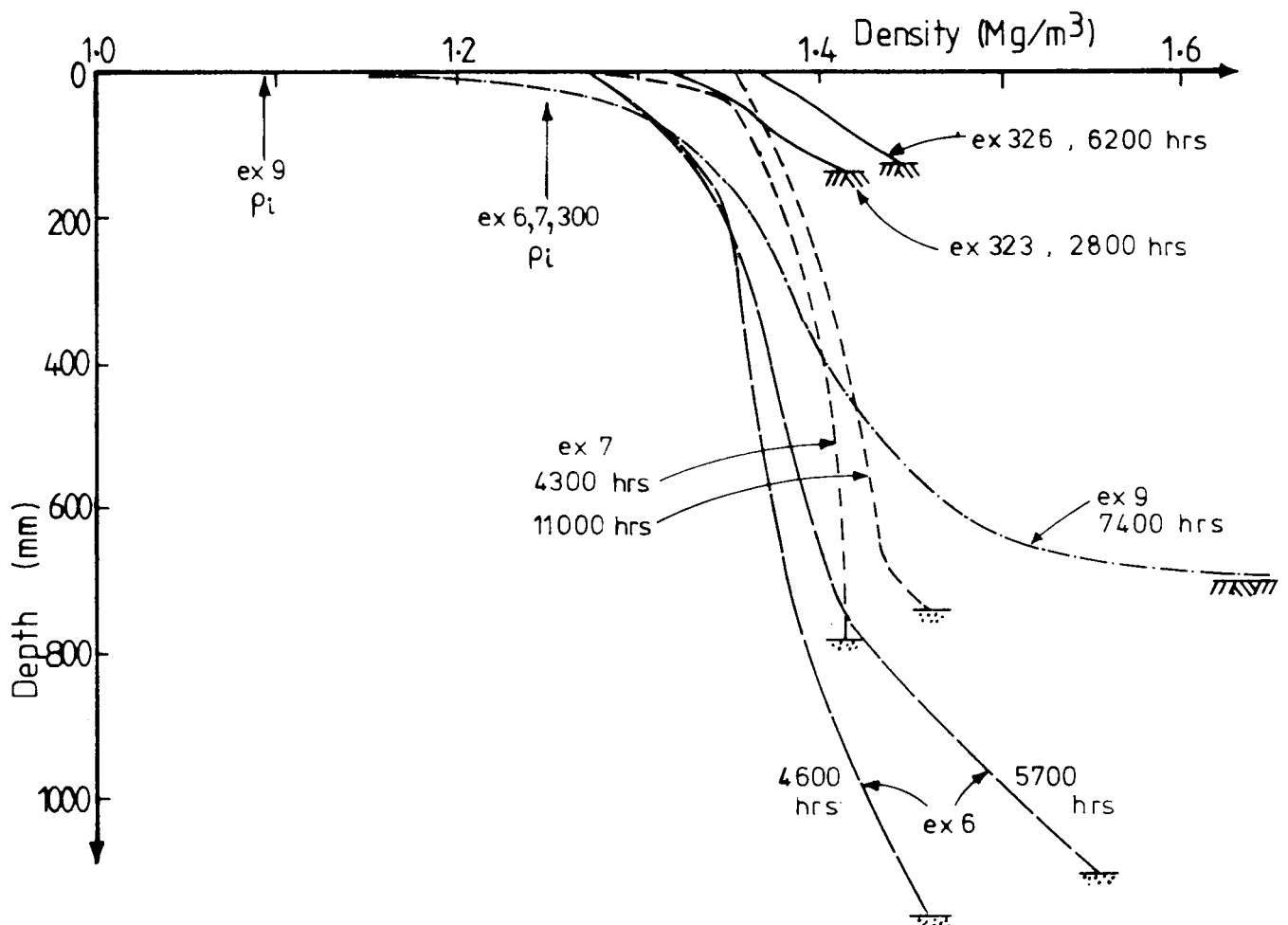


Figure 4.25 Density profiles with depth for expts with same initial density (6, 7, 323, 326) or similar solids mass (7, 9).

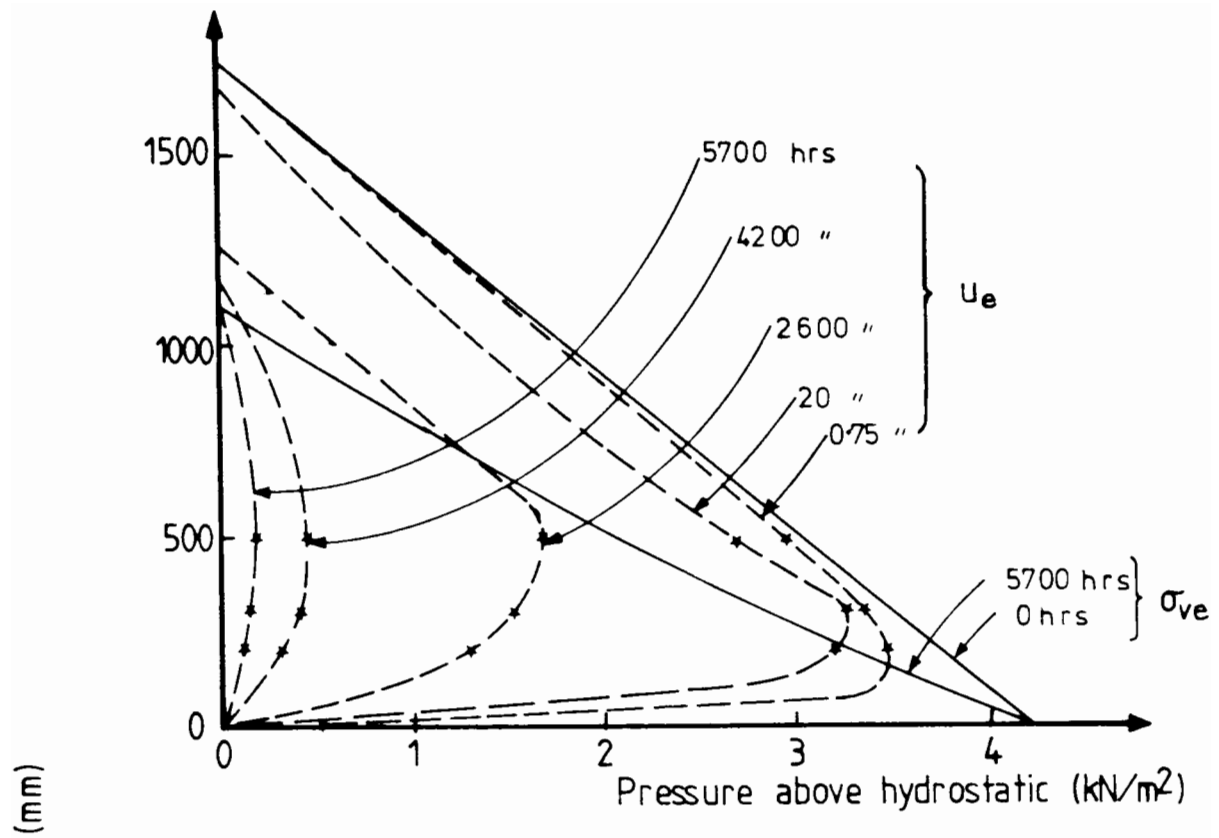


Figure 4.26 Pore pressure and stress profiles, expt 6.

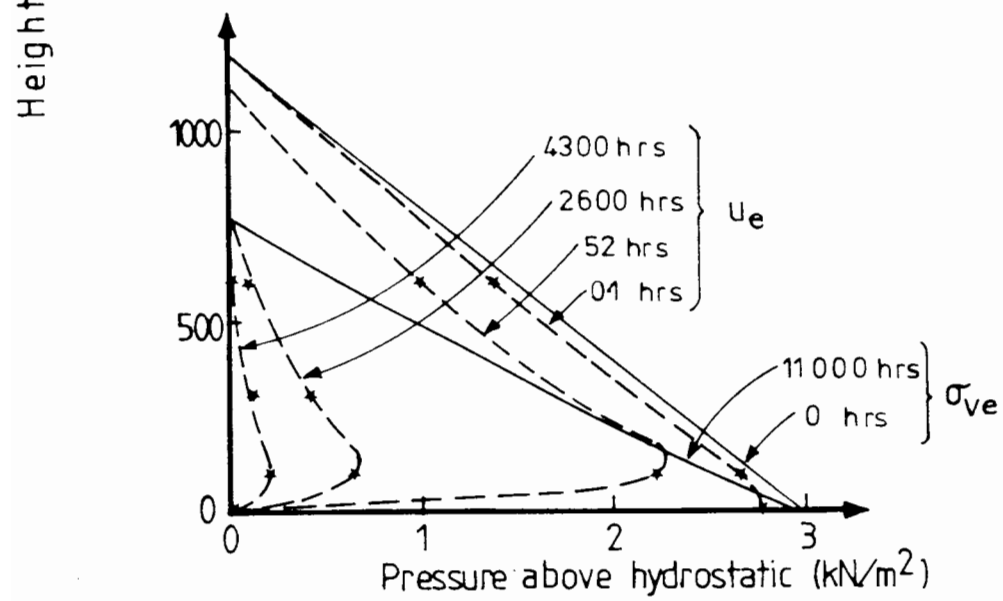


Figure 4.27 Pore pressure and stress profiles, expt 7

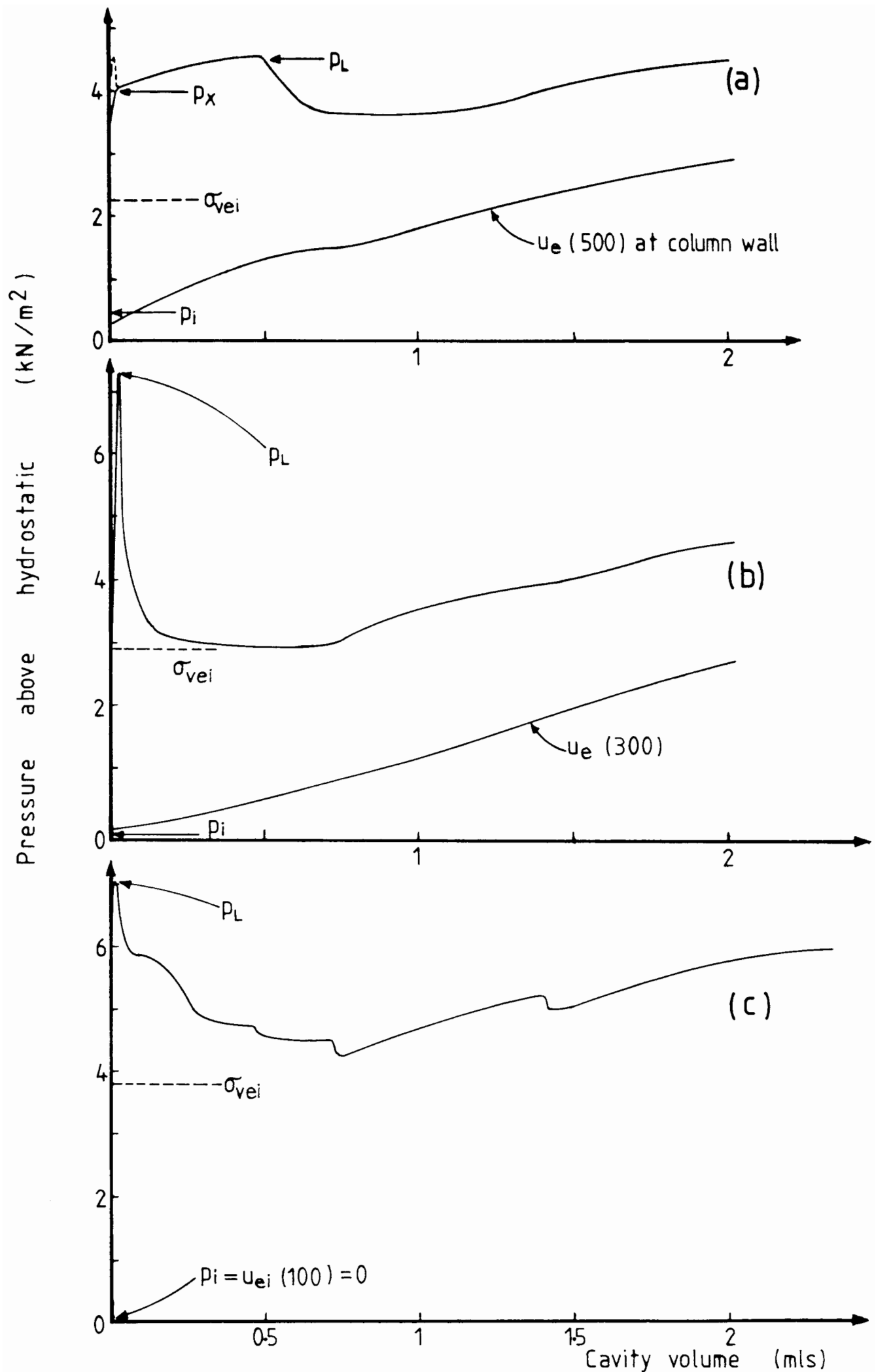


Figure 4.28 Pressure changes during cavity expansion tests in expt. 6.

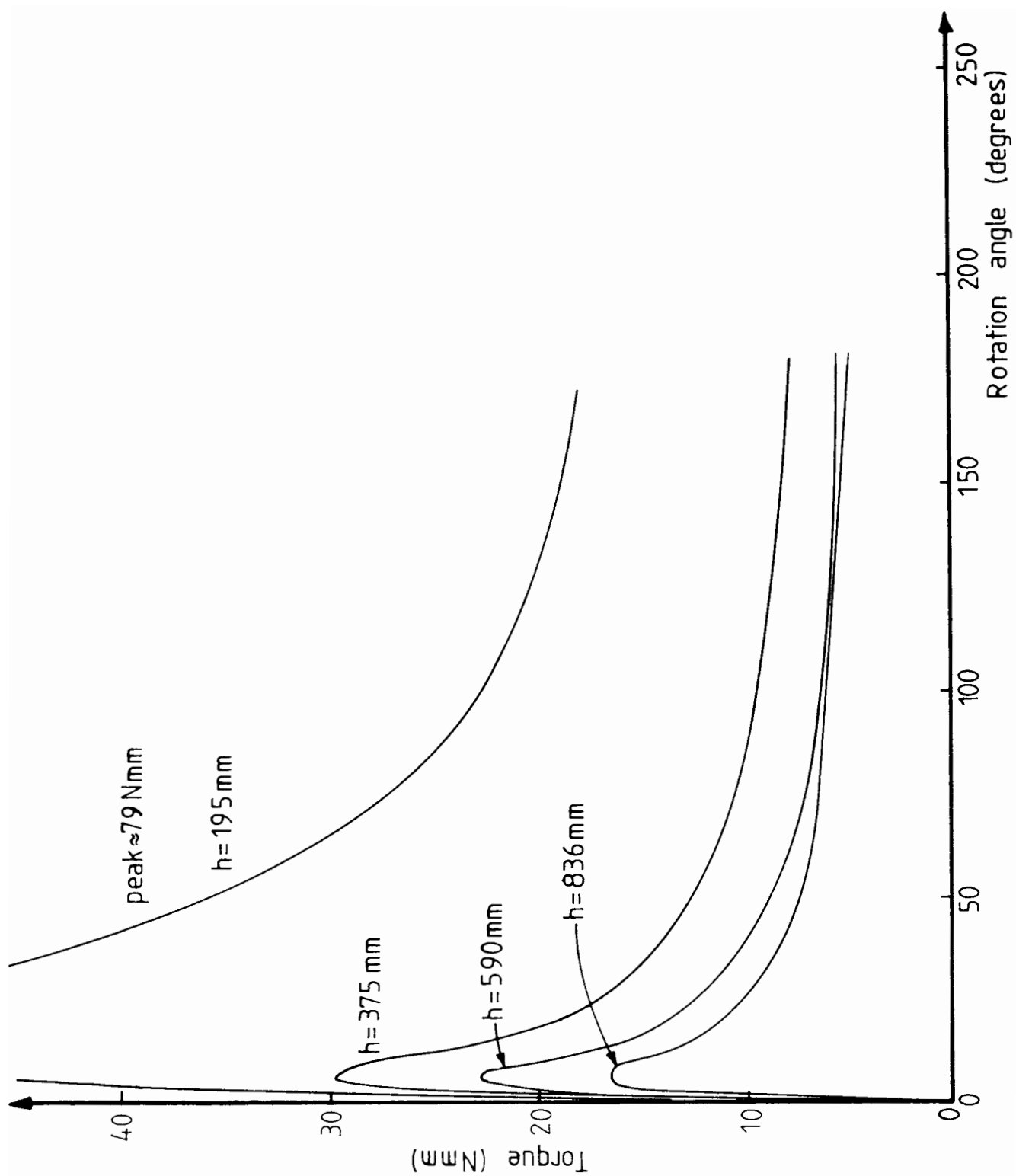


Figure 4.29 Vane tests at $60^\circ/\text{min}$ in expt 6 after 5,670 hours.

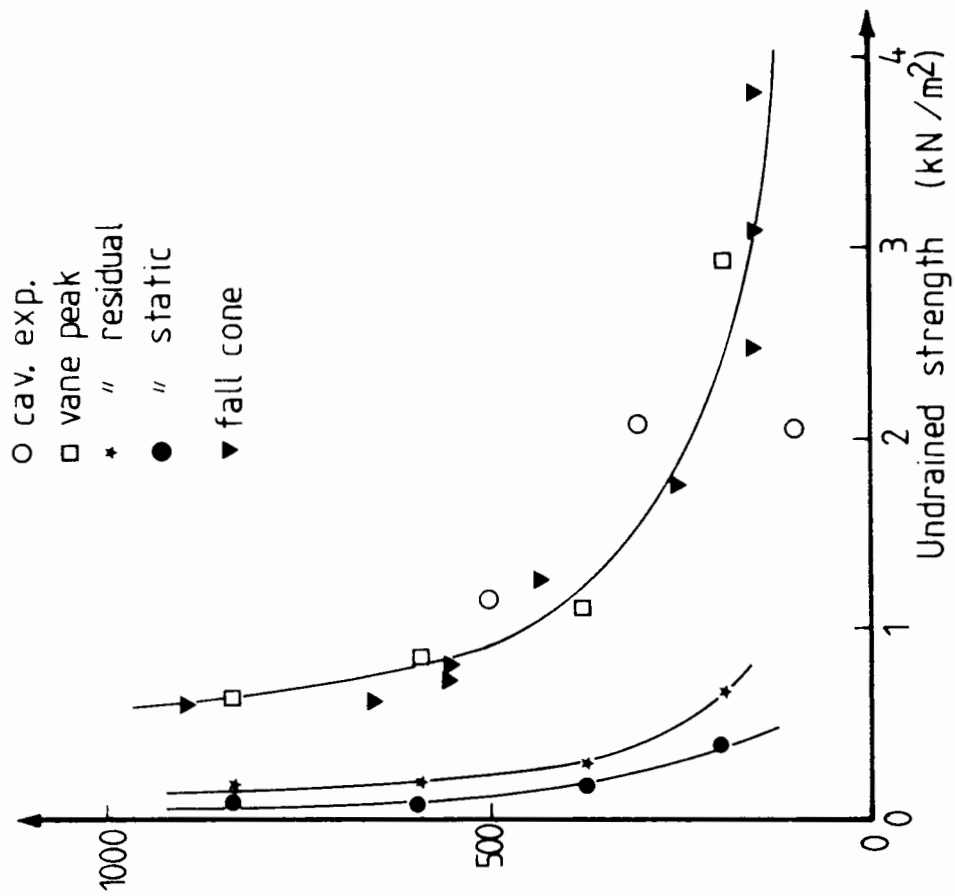
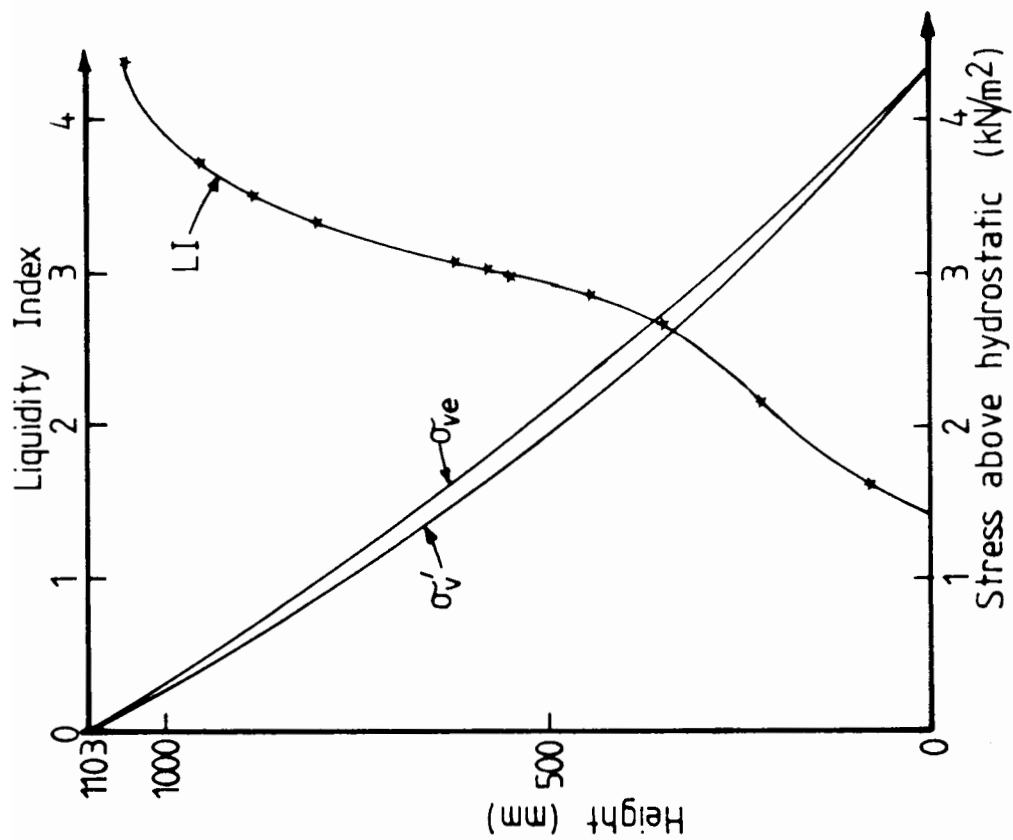
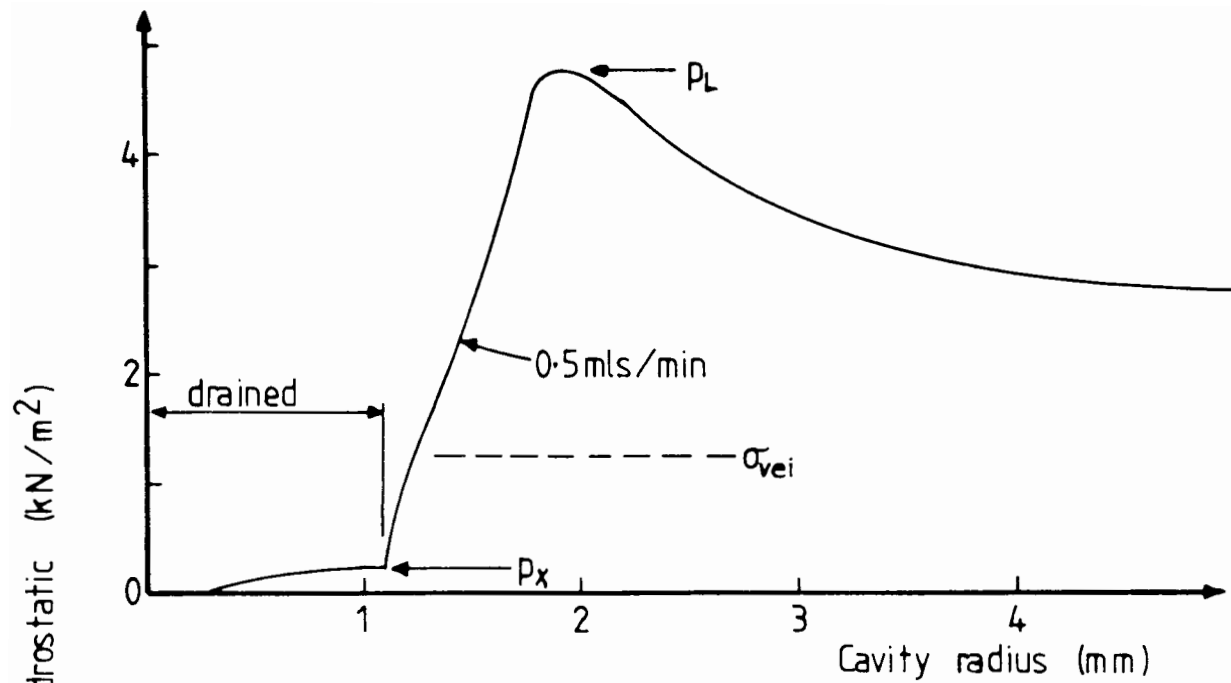


Figure 4.30 Test results for expt 6 after 5,700 hours consolidation.

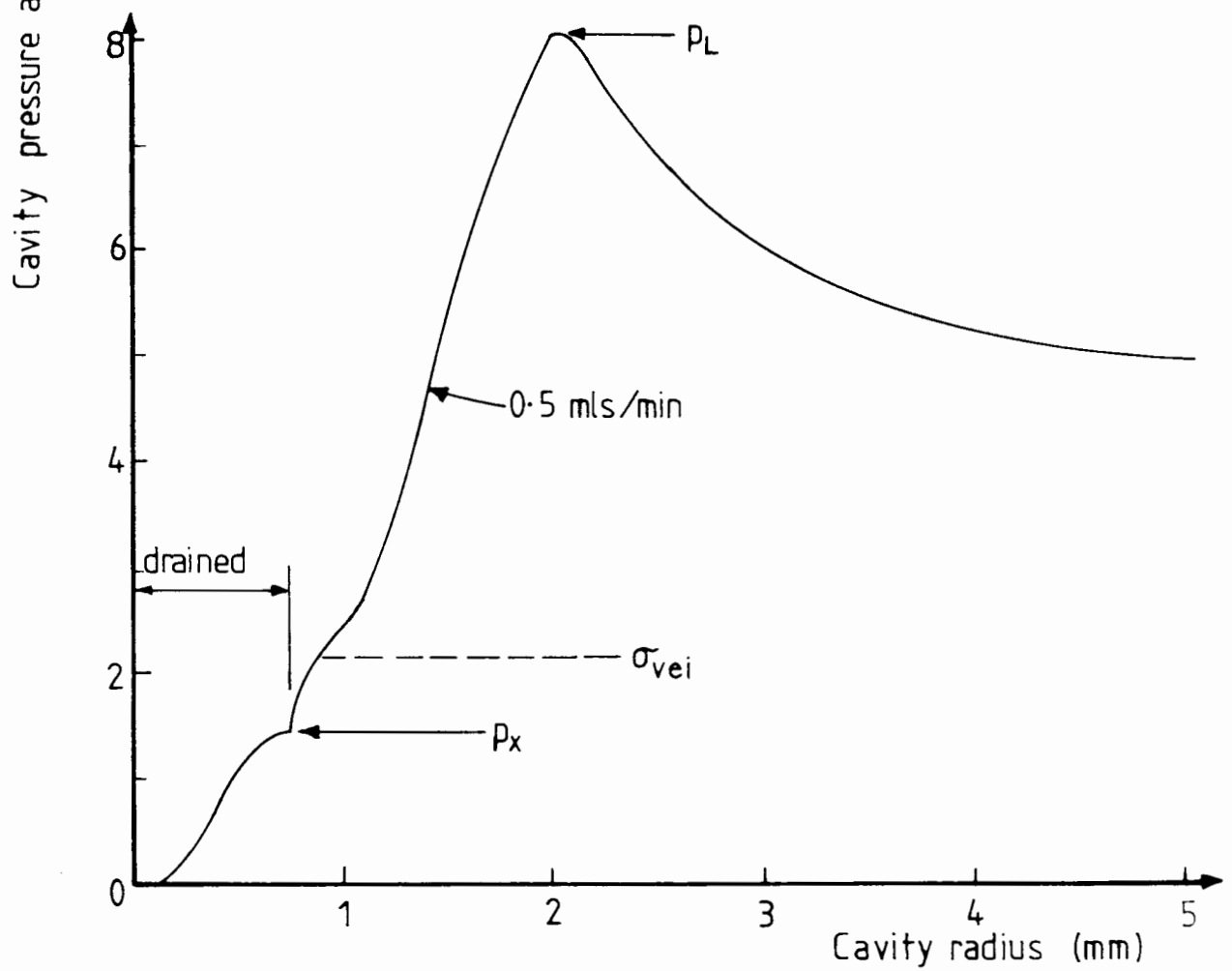
pressure but this is at variance with data from earlier tests where p_i was considerably higher than either the existing pore pressure or the total vertical stress. As in experiment 5 a pressure p_x required to initiate expansion might be deduced and is indicated, but presence of the early peak pressure obscures this value. The limiting pressure indicated in the test at $h = 500$ mm occurred at a reasonable cavity volume (0.45 ml) but volumes of 2.0 ml or greater in tests at 300 mm and 100 mm and the irregular pressure drops recorded suggest that some form of failure had occurred already. In the two latter tests the initial peaks were used to estimate undrained strengths, with shear modulus and undrained strength ratios obtained from peak vane resistances (figure 4.29). These strengths are similar to vane and fall cone strengths, plotted with depth in figure 4.30. Effective stresses before unloading and liquidity indices from moisture content samples taken during unloading are also shown.

Cavity expansion tests in experiment 7 after 11,000 hours included an initial drained expansion phase as described earlier for tests in short columns. To show the results more clearly pressure changes are plotted against cavity radius in figures 4.31. In both cases the pressure attained in the drained phases was below σ_v , although a point of inflection in the pressure:radius curve close to $p = \sigma_v$ may indicate a threshold pressure for expansion. Vane test results are shown in figure 4.32 and undrained cavity, cone and vane strength profiles in figure 4.33 along with effective stress and liquidity index profiles at 11,000 hours. The cavity expansion rate used (0.5 ml/min) was slower than that in experiment 6 tests and appears to give more consistent results, as concluded also from the short column tests.

Experiment 9, initial height 3.51 m, was begun in a settling column outside the laboratory, which was segmented and moved indoors when the surface fell below 1.5 m. Pore pressures were therefore not recorded, but the settlement profile (figure 4.24) suggests that at the time of testing (7400 hours) equilibrium had probably been attained, and results of experiments 6 and



(a) $h = 400$ mm



(b) $h = 200$ mm

Figure 4.31 Pressure changes during cavity expansion tests in expt. 7.

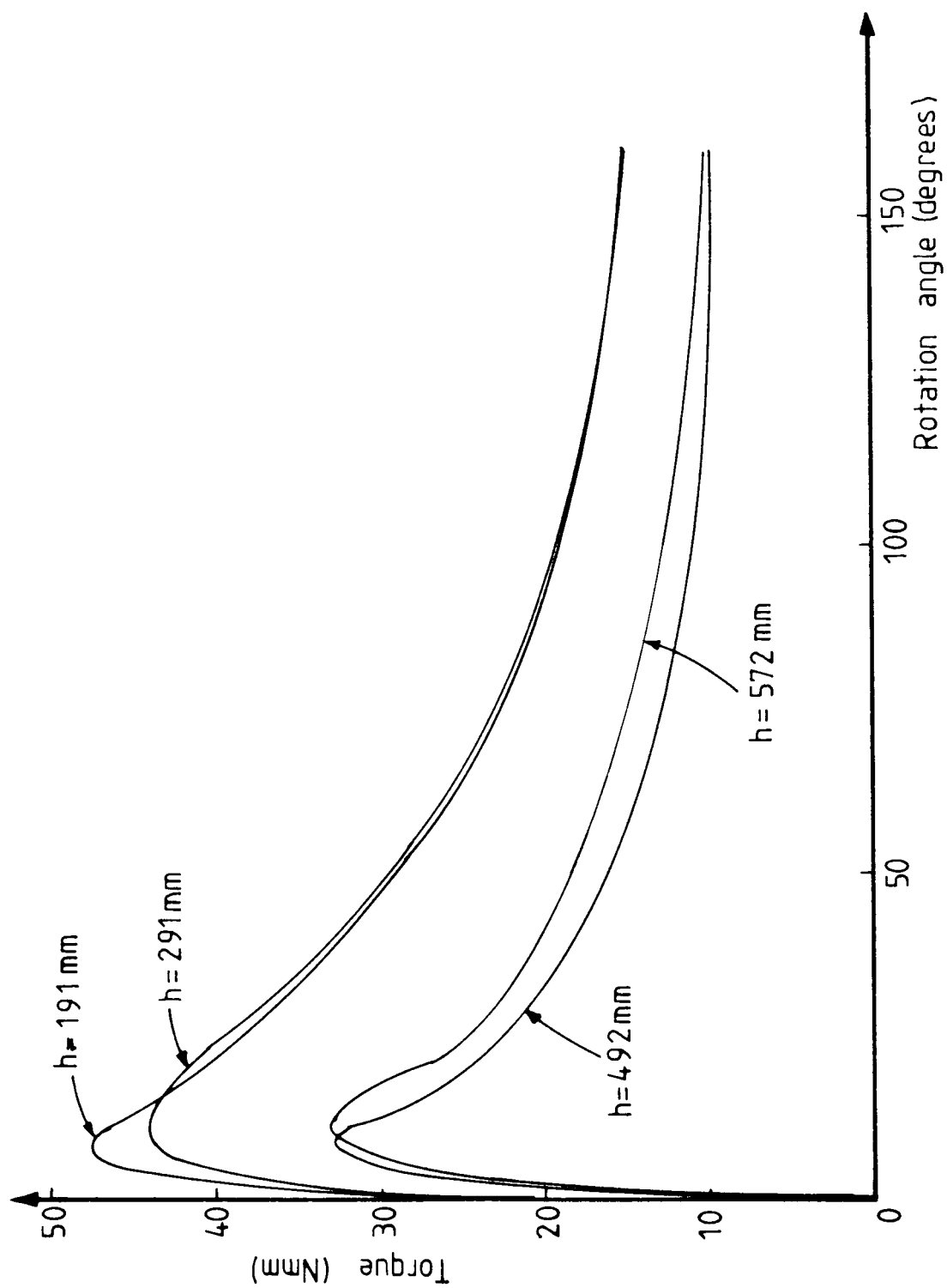


Figure 4.32 Vane tests at $60^\circ/\text{min}$, expt 7.

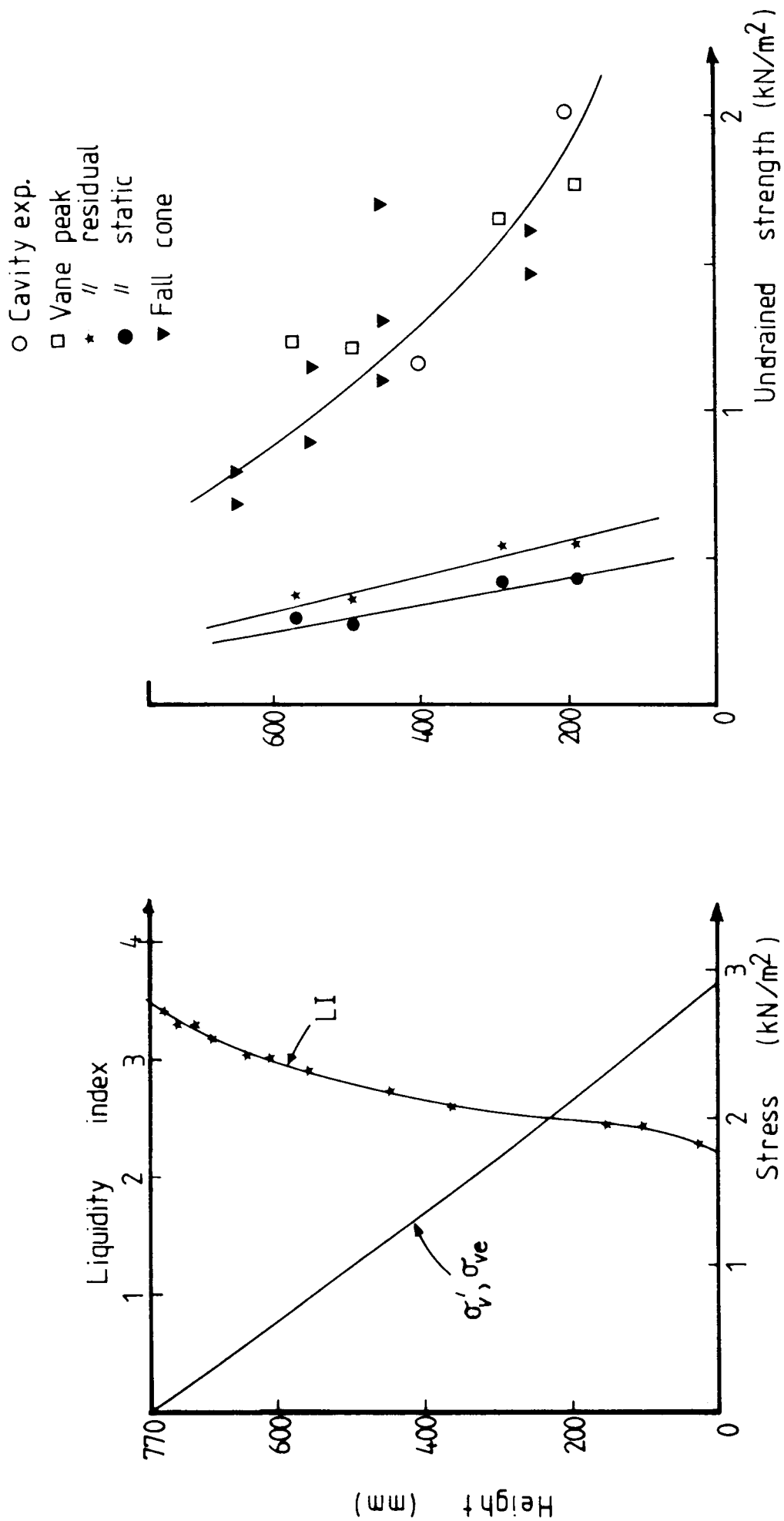


Figure 4.33 Stress, liquidity index and strength profiles after 11,000 hrs in expt 7.

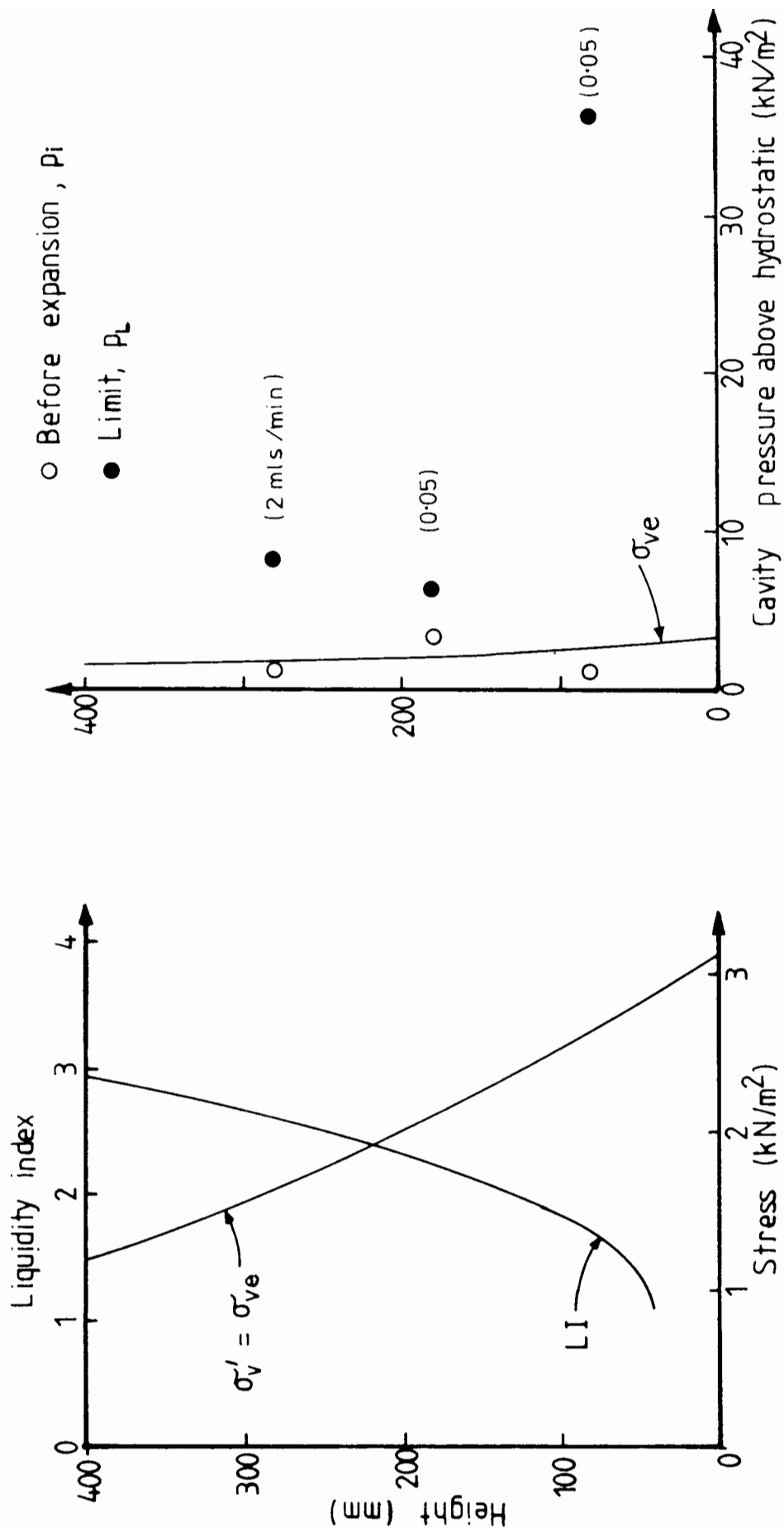


Figure 4.34 Stress, liquidity index and cavity pressure profiles for expt 9 at 7,400 hours.

7 support the assumption that excess pore pressures had dissipated fully at the time of testing. Total stress and liquidity indices at this time are shown in figure 4.34a and cavity pressures (after insertion and limiting) for three tests in figure 4.34b. Undrained strengths s_{us} cannot be calculated readily since G/s_u values are not known. The likelihood of particle size segregation in this experiment means that values obtained in other tests are unlikely to apply, at least near the base, but higher in the soil densities are similar to those in experiments 6 and 7 (figure 4.25). Using $G/s_u = 5$ suggests that s_{us} is approximately $(p_L - p_o)/3.6$, giving s_{us} values of 2.0 kN/m² at $h = 280$ mm (expansion rate 2 ml/min) and 0.8 kN/m² at $h = 180$ mm (0.05 ml/min).

4.5.3 Experiments 8, 10 and 11

Figure 4.35 shows settlements for two experiments with the same initial height and similar density (experiment 10, 1553 mm, 1.147 Mg/m³; experiment 11, 1553 mm, 1.153 Mg/m³) and a third experiment with slightly higher density and lower height (experiment 8, 1500mm, 1.16 Mg/m³). These densities were all just above the critical density found by Been & Sills (1981) which appeared to define when a suspension phase would occur. The absence of the suspension phase is indicated by the slow settling rates shortly after deposition. Settlements for all three experiments are similar although experiment 8 settles rather faster during the first 50 hours. The reason for this is not clear but may be due to slightly different soil properties as the soil for 8 was collected from Combech two years earlier than that for experiment 10 and 11 and had a higher silt fraction. Experiments 8 and 10 were singly drained throughout while experiment 11 was permitted to drain from the base (at hydrostatic pressure) between 330 and 950 hours. The modified Lee model discussed in chapter 2 (Been & Sills, 1981) was used to predict settlements during this period which are shown as a dotted line; the actual settlement rate appeared to remain unchanged. Density profiles recorded when each soil height was closest to 750 mm are compared in figure 4.36 and are almost identical

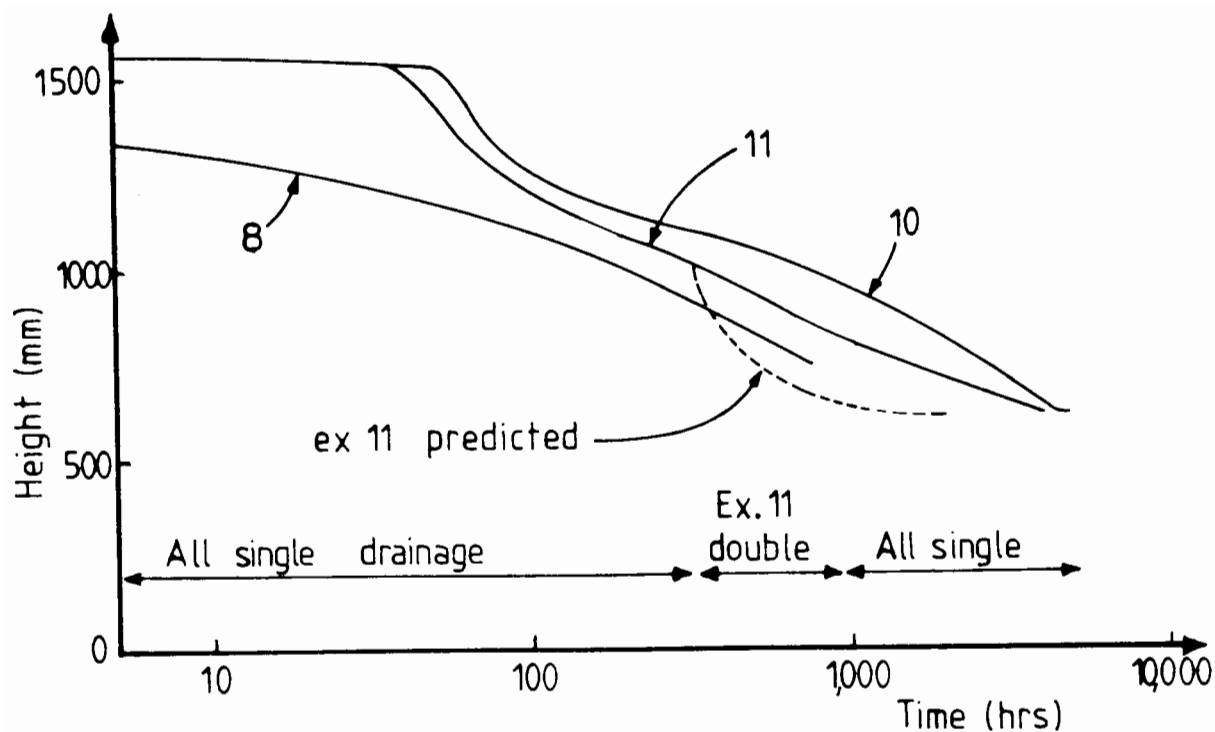


Figure 4.35 Surface settlements, expts 8, 10, 11.

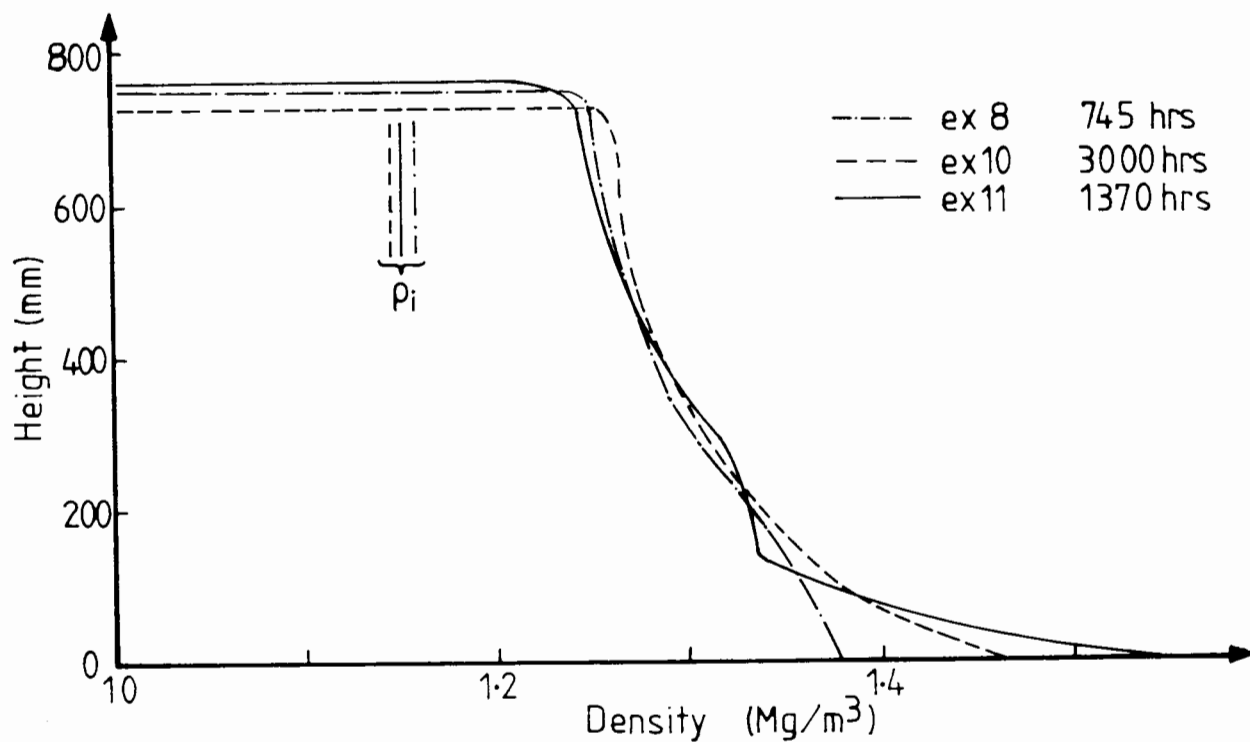


Figure 4.36 Comparison of density profiles, expts 8, 10, 11 (similar initial densities and containing similar masses of solids) at times when surface heights are similar.

although the double drainage in experiment 11 has created a thin, high density layer near the base.

Figures 4.37, 38, 39 show selected density and excess pore pressure profiles for each experiment during consolidation. Those for experiment 8 (figure 4.37) followed the expected form found by Been (1980). The 150 hour profile for experiment 10 shows a density inversion towards the base which was also observed by Been at a similar initial density. The reason for this is not known, but may be due to segregation in this region. A similar effect occurs in experiment 11. An alternative explanation (Sills et al, 1985) is that an open framework is established initially, which collapses when some threshold effective stress is exceeded. The more rapid effective stress increase near the base would cause this region to collapse earlier than that above. All three experiments also exhibited significant density increase at the soil surface, occurring at a rate more rapid than in experiments at higher initial density. Experiment 11 also included two total horizontal stress (σ_h) measuring segments (described in chapter 2) at heights 100 mm and 400 mm.

At $h = 400$ mm, σ_h remained close to the excess pore pressure throughout consolidation but darkening of water inside the rubber membrane suggested a leak may have occurred. At $h = 100$ mm the horizontal effective stress σ'_h varied between zero and about $0.3\sigma'_v$. The scatter in these results was considerable. Increased soil/wall adhesion in the 100 mm segments enclosed by the membranes, and the reduced internal diameter (by 1 or 2 mm) may account in part for the density anomalies. The original system used by Been, with a transducer mounted in the column wall, is to be preferred in future experiments.

Additional surface loads were applied to soil in experiment 11 in three increments after 4060 hours. Density profiles are also shown in figure 4.39 (for the soil only). Dense layers near the surface have inhibited drainage and the partial restriction from the membrane segment at 400 mm may have prevented the full load from being transferred to soil below this height.

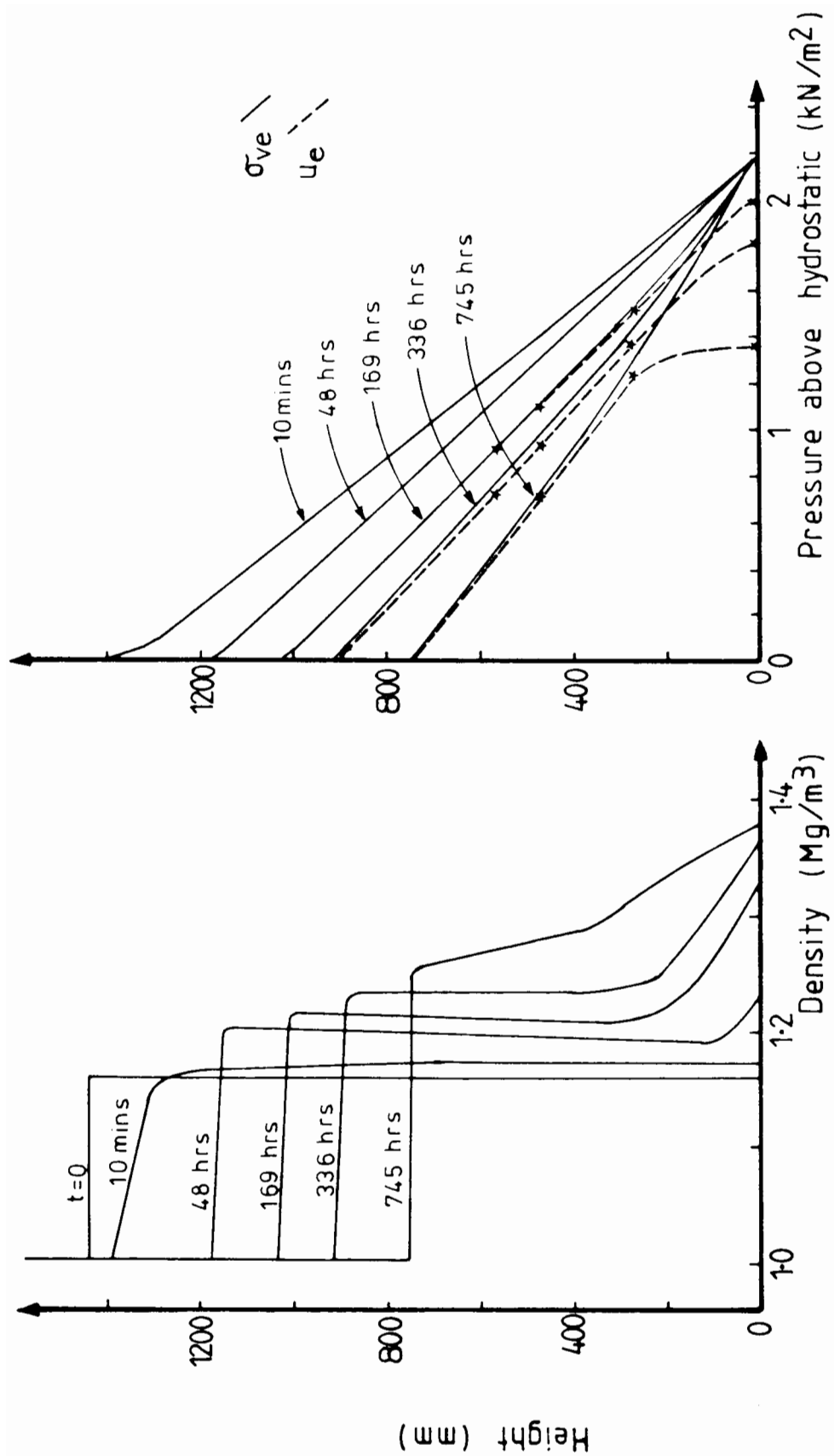


Figure 4.37 Density and pore pressure profiles, expt 8.

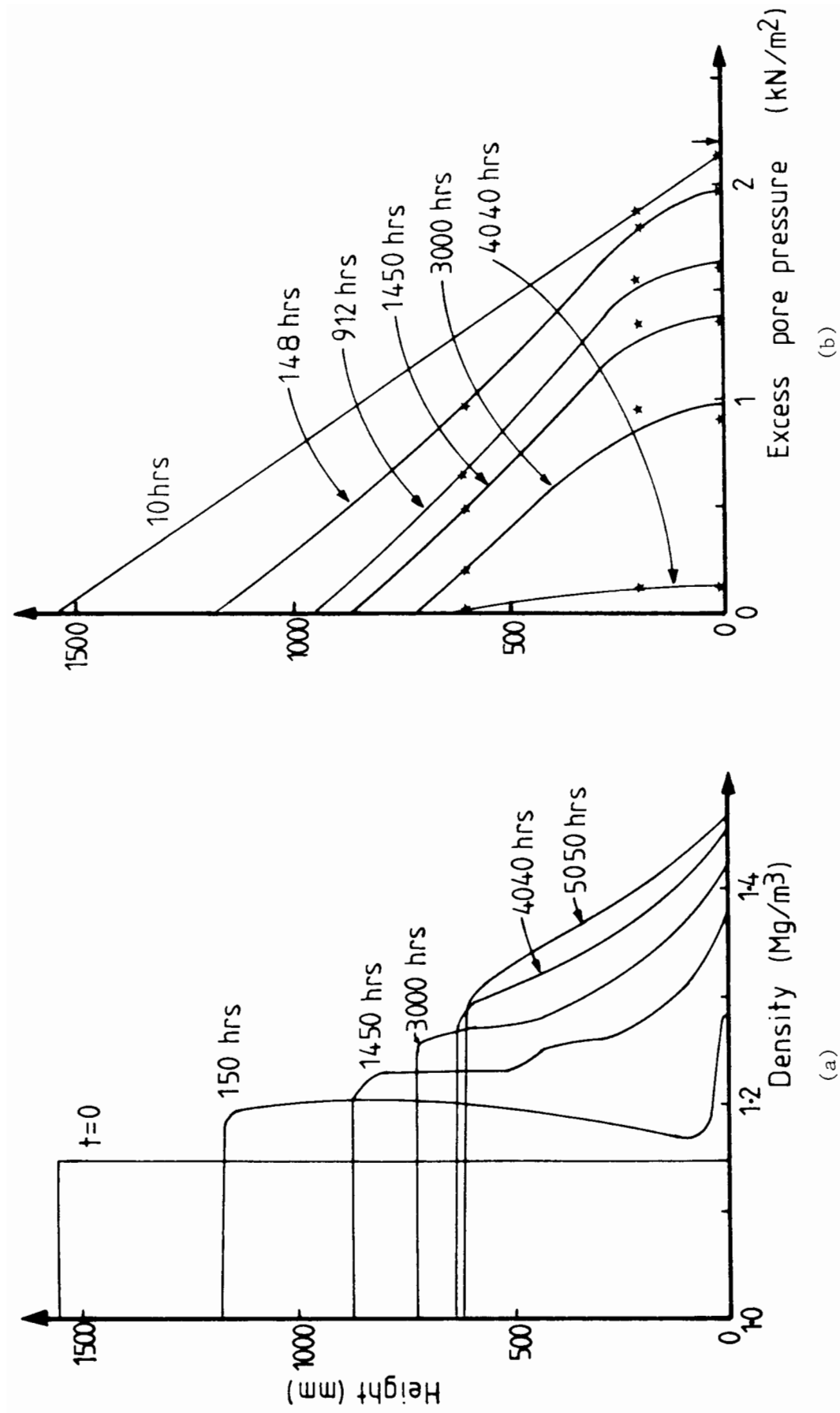


Figure 4.38 (a) Density and (b) pore pressure profiles during consolidation, expt 10.

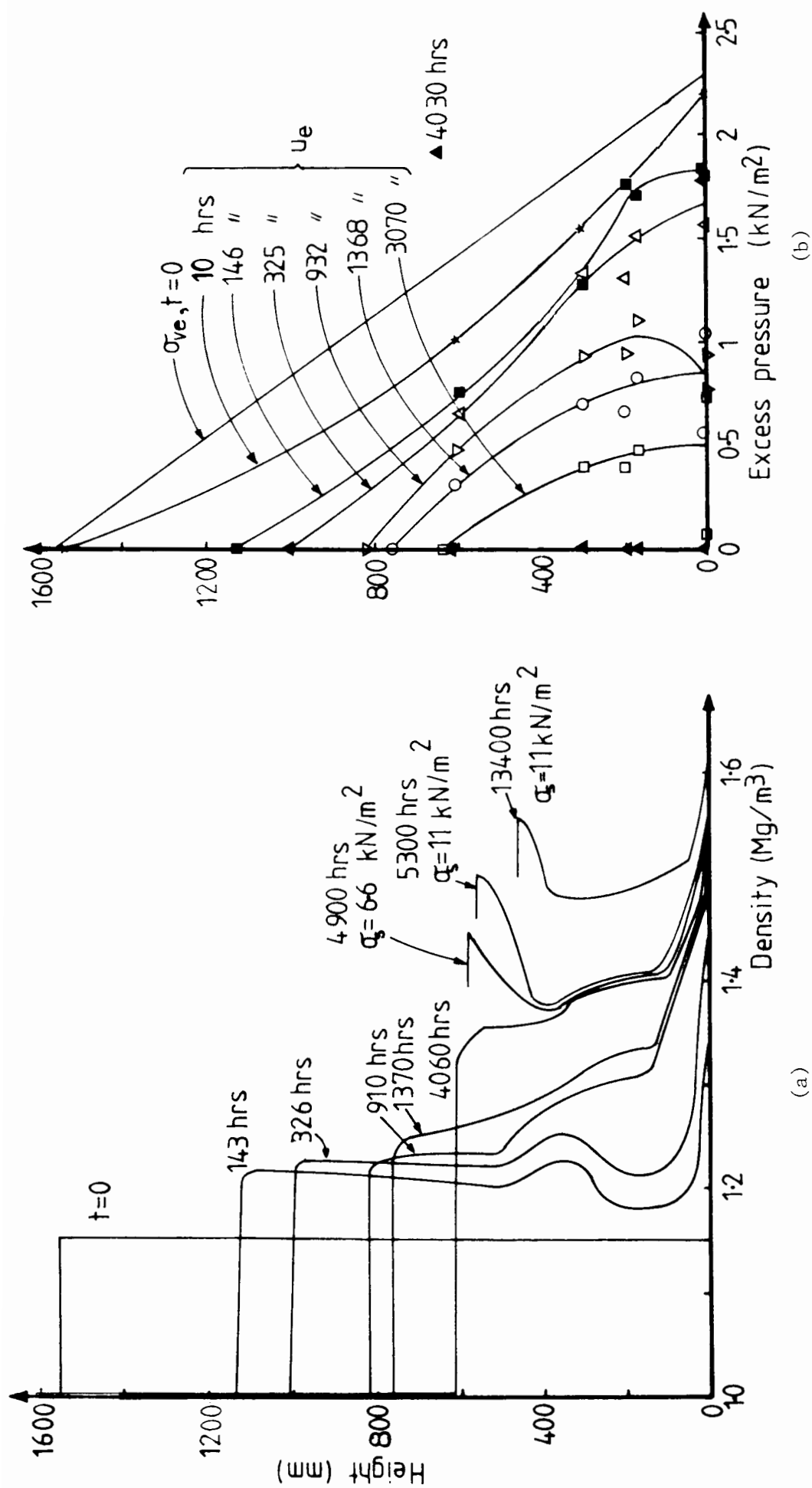


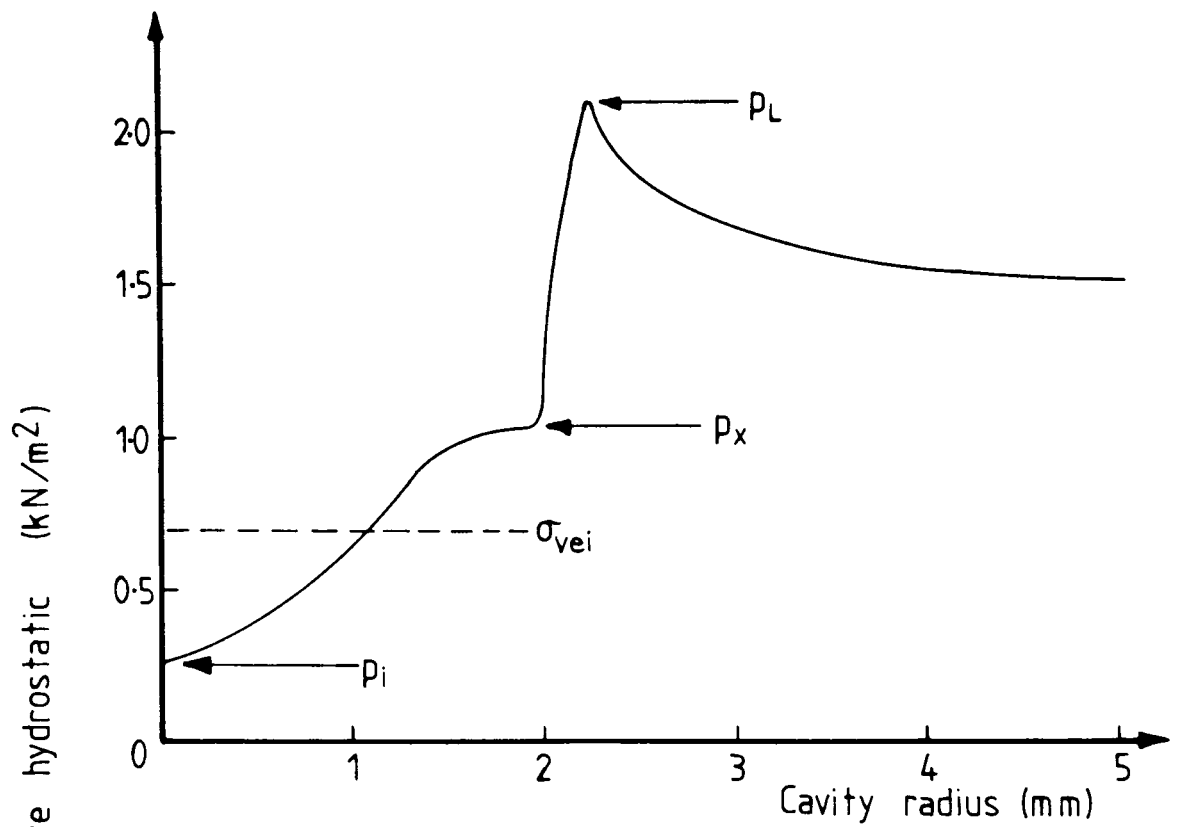
Figure 4.39 (a) Density and (b) pore pressure profiles during consolidating phases, expt 11.

Cavity expansion tests in experiment 10 after 5000 hours (figure 4.40) show a clearly defined change in pressure-radius behaviour at a radius approximately that of the needle. Although these values are higher than the total vertical stress, they are assumed to represent p_0 , the initial mean total stress and are used with p_L to calculate undrained strength with G/s_u values from vane tests (figure 4.41). Liquidity index, stress and strength profiles are shown in figure 4.42. Close agreement is obtained between the fall cone and peak vane strengths although it can be seen from figure 4.35 that the total soil height had decreased at this time to below 40% of the initial height, and anisotropy might have been expected to cause these tests to give different results.

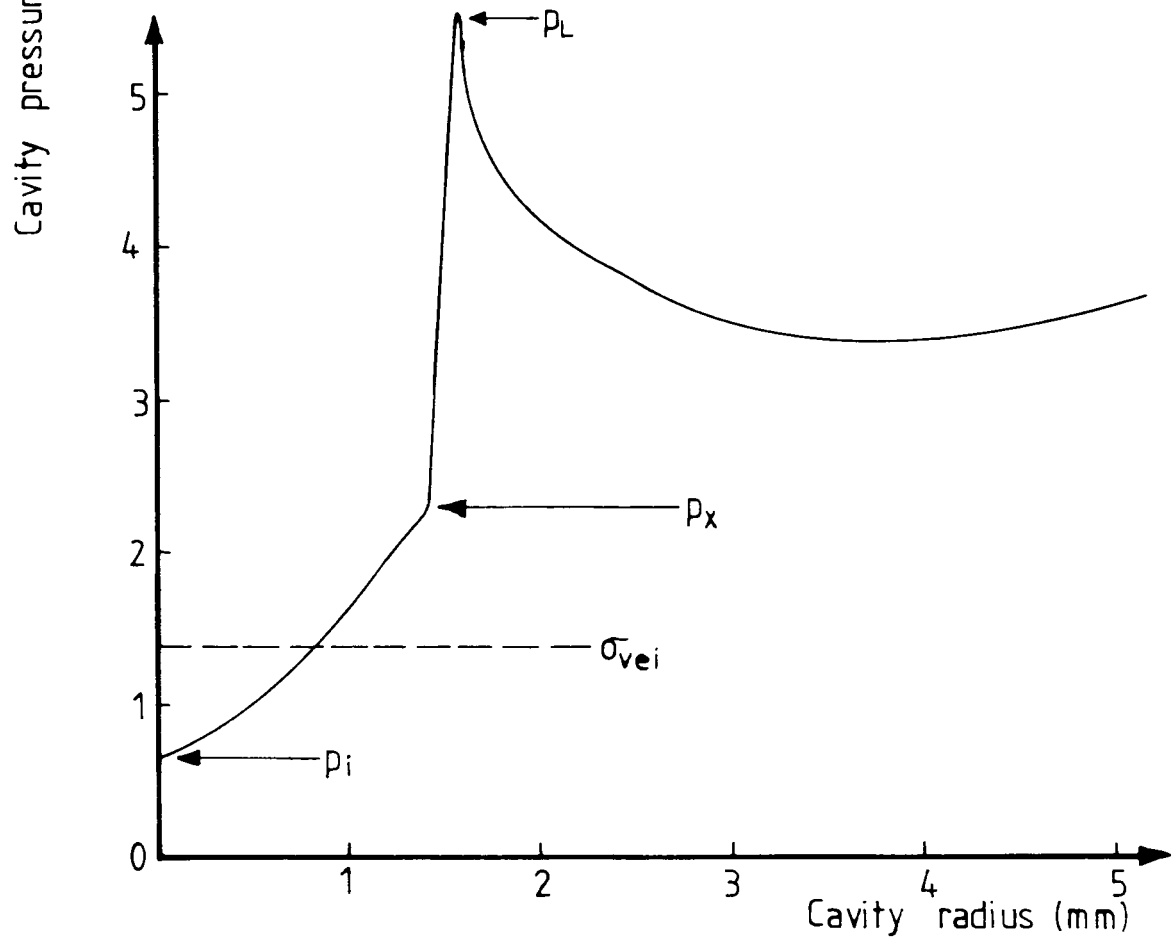
Due to the similarity of cone and vane test results in experiment 10, a more frequent and rapid sequence of vane testing alone was preferred in experiment 11, carried out after the surface surcharge had been removed at 13,500 hours. Results are shown in figure 4.43. The cavity expansion technique was not used because of the high expected degree of anisotropy, but two fall cone tests were carried out at the soil surface immediately after surcharge removal. Vane tests were continued to very large rotations (up to 2000°) following the initial resistance minima which occurred between 200° and 500° . Further decreases did not occur at high rotations and in some tests slight increases were observed.

4.5.4 Experiment SW2

Soil for this experiment was sieved and mixed in natural ocean water, salinity 35 ppt by mass, to initial density 1.141 Mg/m^3 . This is not directly comparable with fresh water soils at similar densities due to the inclusion of dissolved salts and instead the void ratio ($e_i = 13$) should be used when making comparisons. Experiment 8, 10 and 11 contained similar solids masses although void ratios and initial heights were lower. Settlements, density and pore pressure profiles for experiment SW2 are shown in figure 4.45. The settlement curve is unusual and the rapid early settlement is more typical of suspension



(a) $h = 402\text{mm}$.



(b) $h = 202\text{mm}$.

Figure 4.40 Cavity expansion tests at 0.5 mls/min after 5,000 hours, expt. 10.

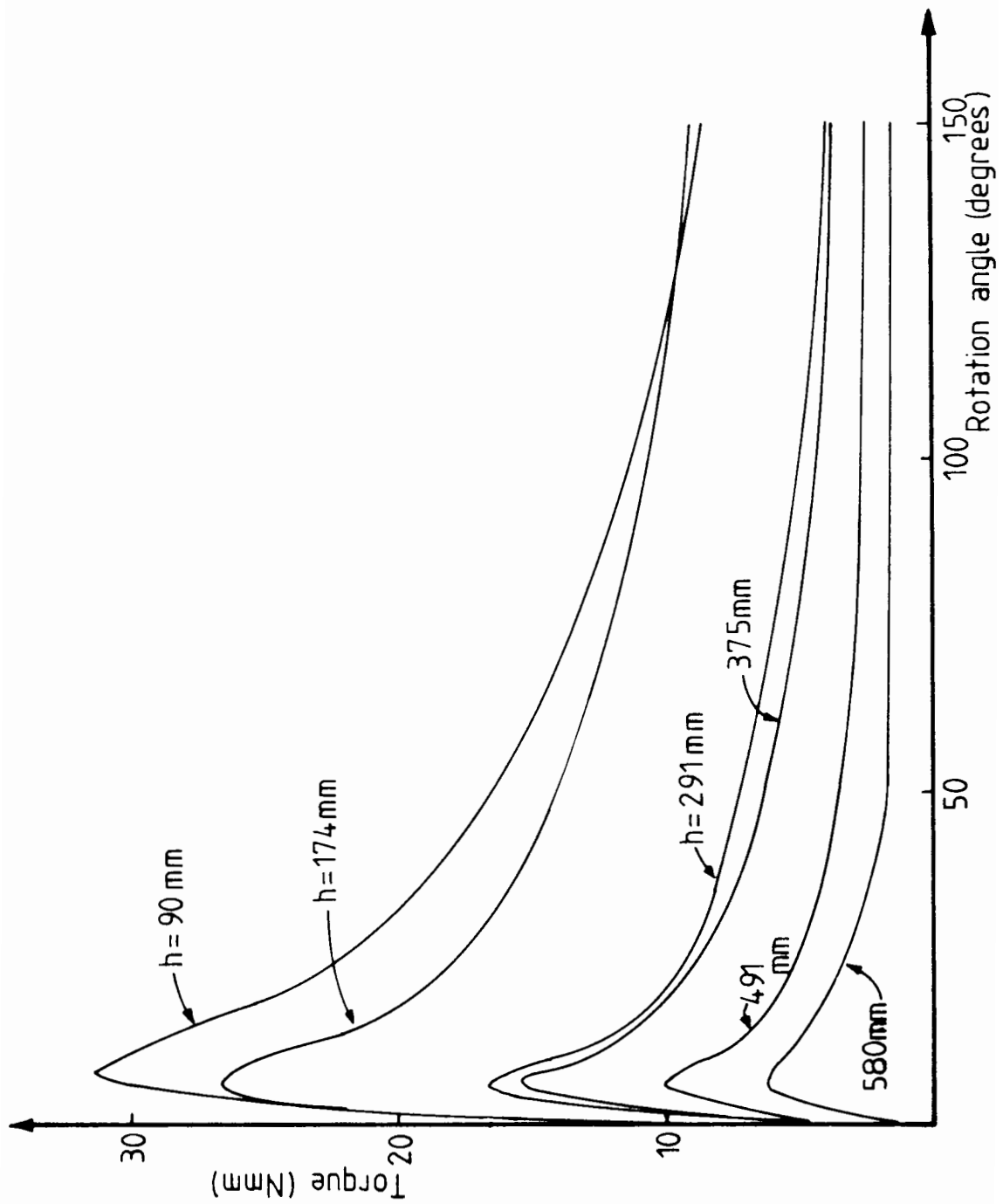


Figure 4.41 Vane tests at $60^\circ/\text{min}$ in expt 10 after 5,000 hours.

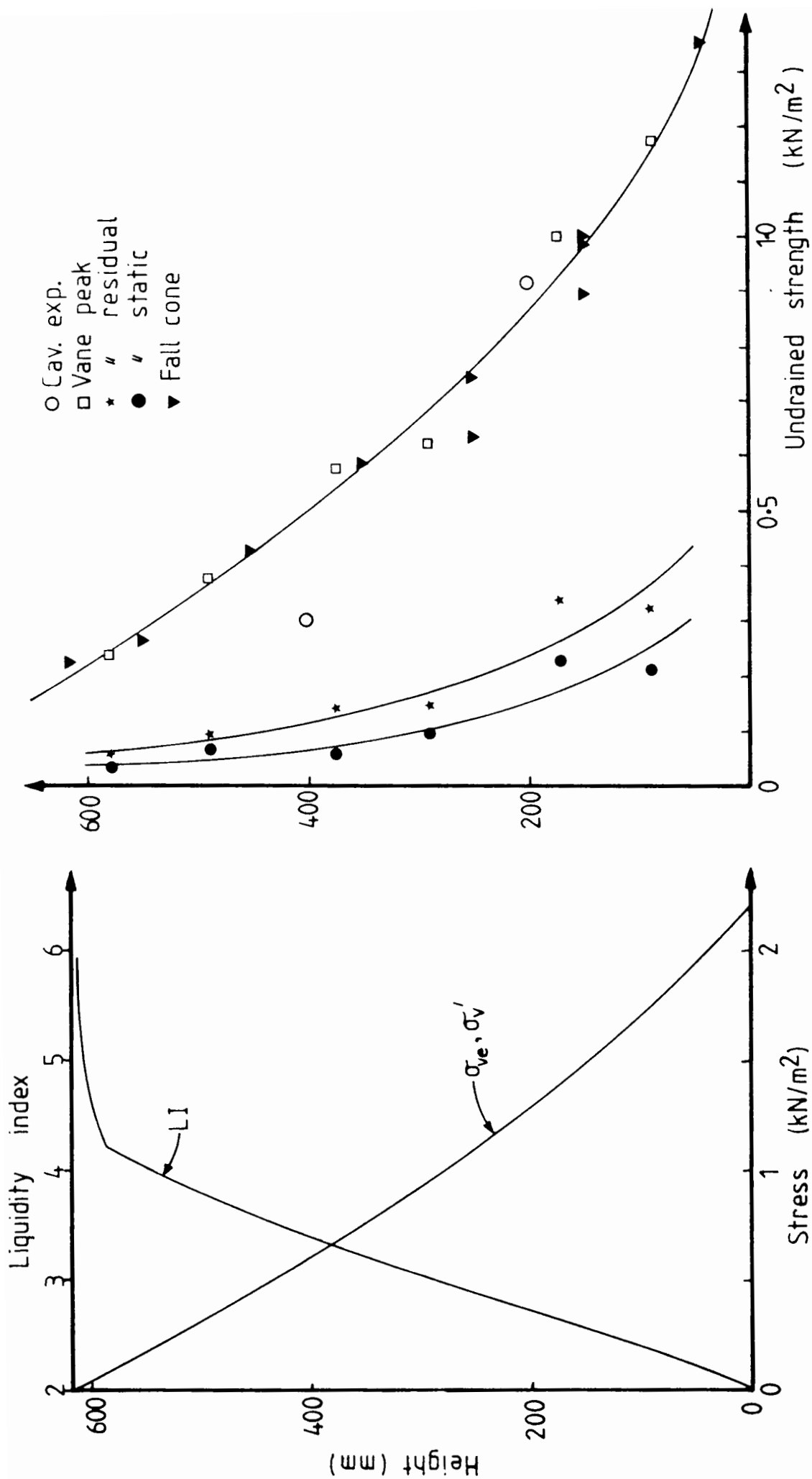


Figure 4.42 Stress, liquidity index and strength profiles for expt 10 after 5,000 hours.

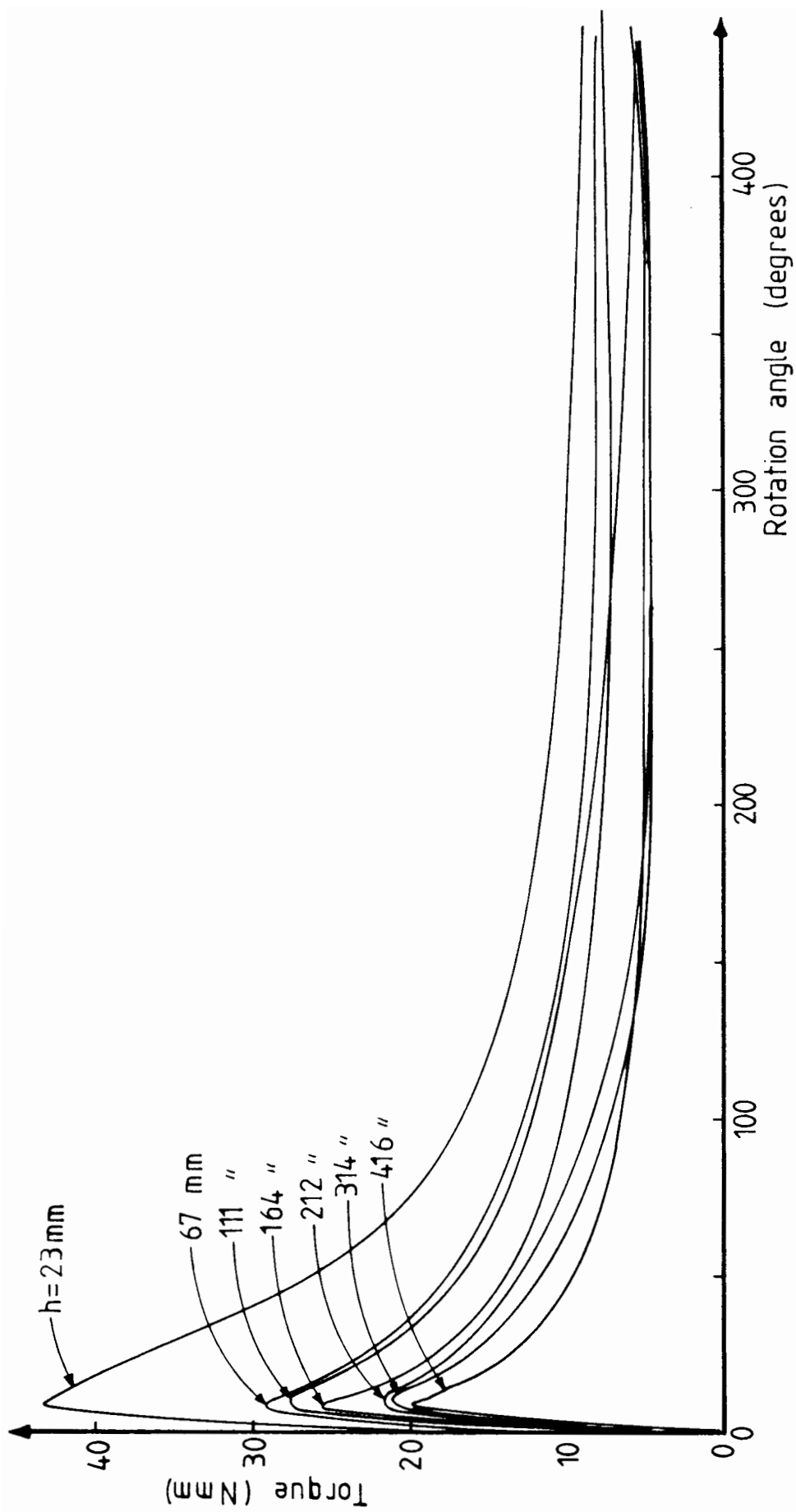


Figure 4.43 Vane tests after 13,500 hours in expt 11. Rotation rates $60^\circ/\text{min}$. Vane $25.4 \times 12.7\text{mm}$.

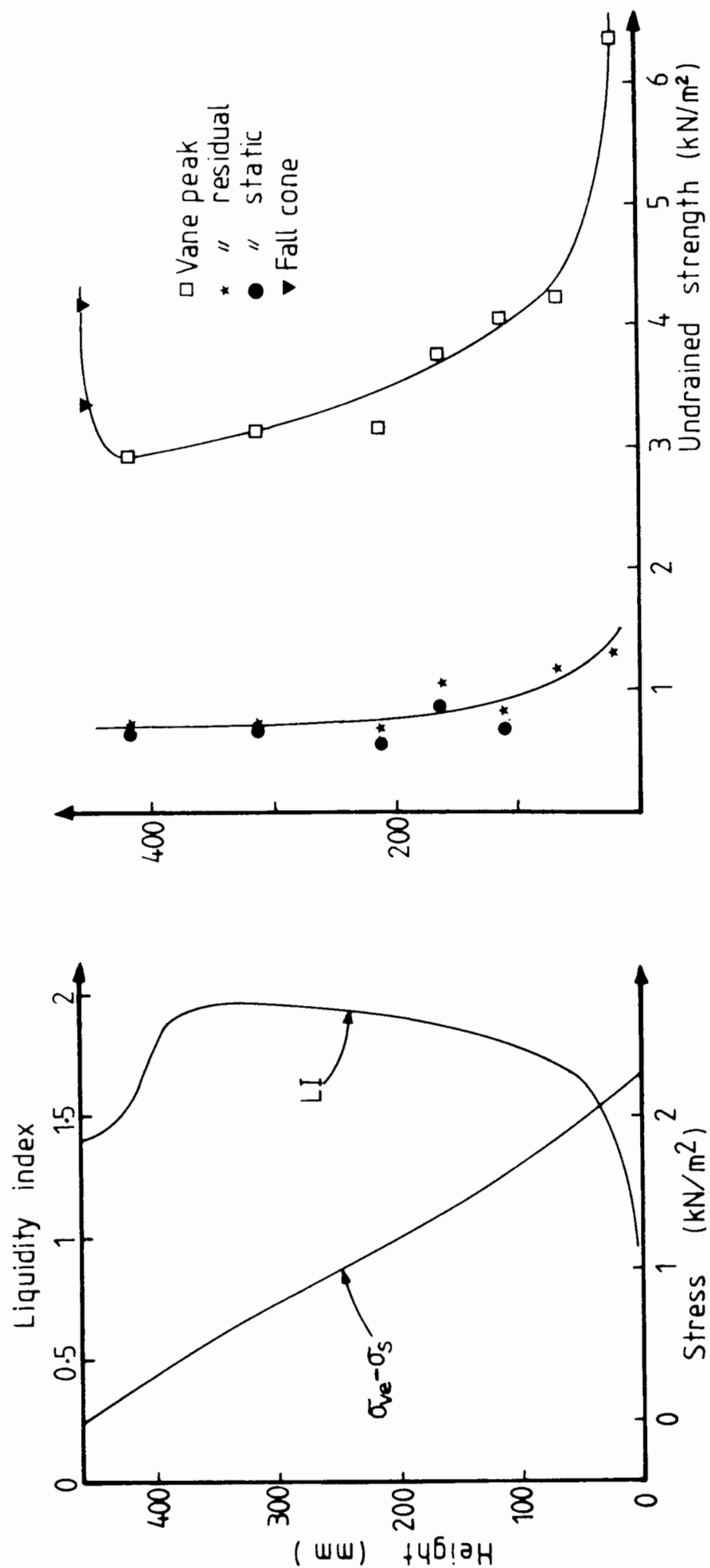


Figure 4.44 Stress, liquidity index and strength profiles after 13,500 hrs in expt 11.

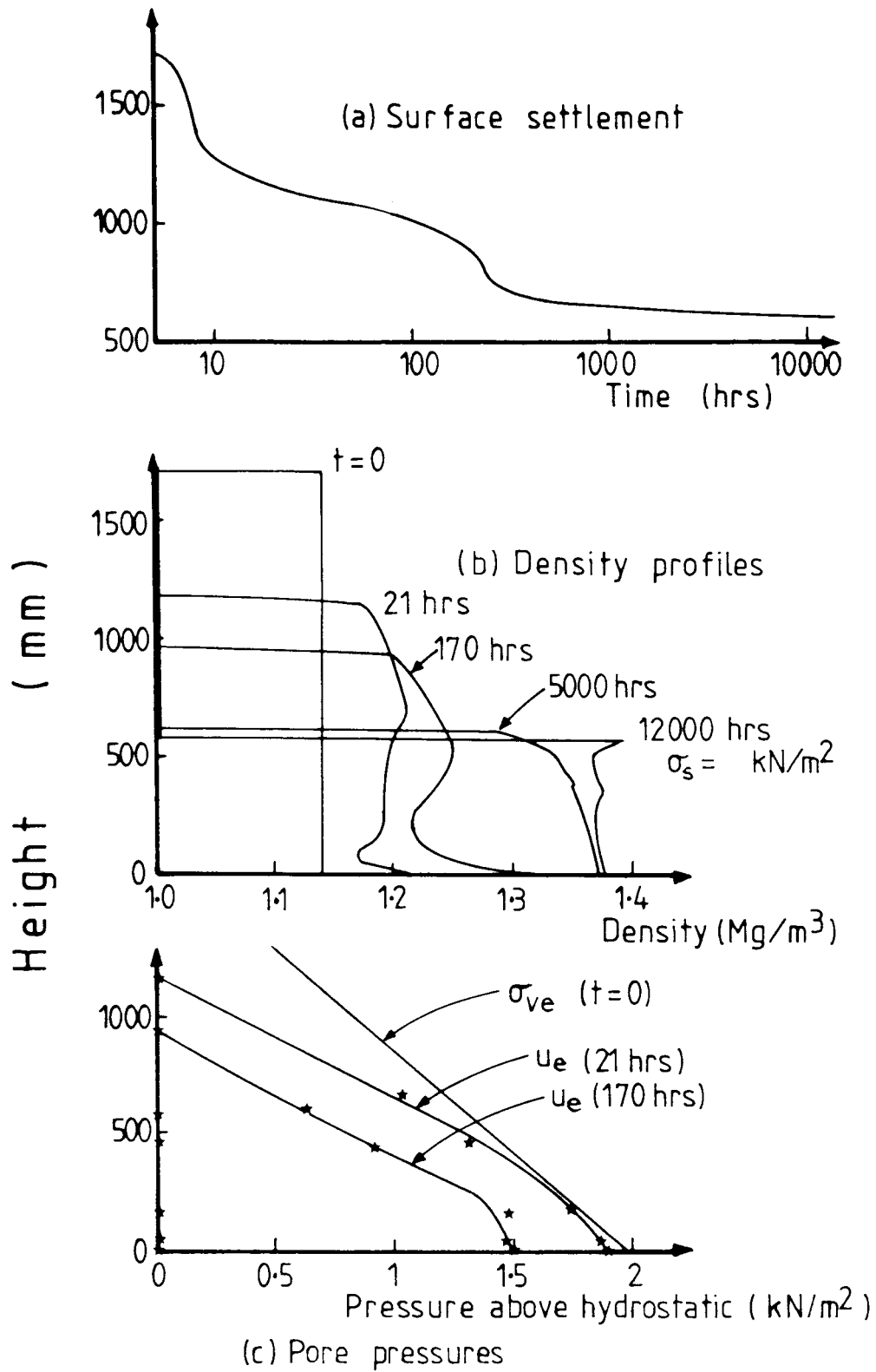


Figure 4.45 Settlement, density and pressure profiles, expt SW2 (salt water), initial density 1.141 Mg/m^3 .

settling, but intermediate density profiles not shown here confirmed that a suspension never existed and during this time pore pressures were smaller than vertical total stresses. Settlements after 300 hours are much smaller than any of experiments 8, 10 or 11. Density profiles follow similar patterns to those in experiments 8, 10 and 11 and again show the density inversion near the base. A surface load was applied after 5000 hours and results in smaller density increases than similar loads applied in experiment 11.

4.6 Incremental surcharge experiments 11 to 15

Surface surcharges of sand and lead shot were applied to thin layers of soil at high initial specific volumes to determine consolidation characteristics under rapid compression. Results are shown in figure 4.46 where the normalised soil height at the end of each increment may be assumed equal to the uniform specific volume throughout each sample since self weight stresses are much less than surcharge stresses. Square root time plots are used in order not to disguise either initial or longer term settlement behaviour. Two different types of curve can be seen. In the first type, settlement progresses immediately following load application while in the other type rates are initially very slow. It is possible that the creep (at constant effective stress), discussed earlier in relation to surface density changes in tall columns, is also significant in these tests. Slow initial settlements might indicate that little primary (stress induced) consolidation is occurring, and most of the subsequent volume compression is due to creep alone. The implications for consolidation behaviour are discussed in the next chapter.

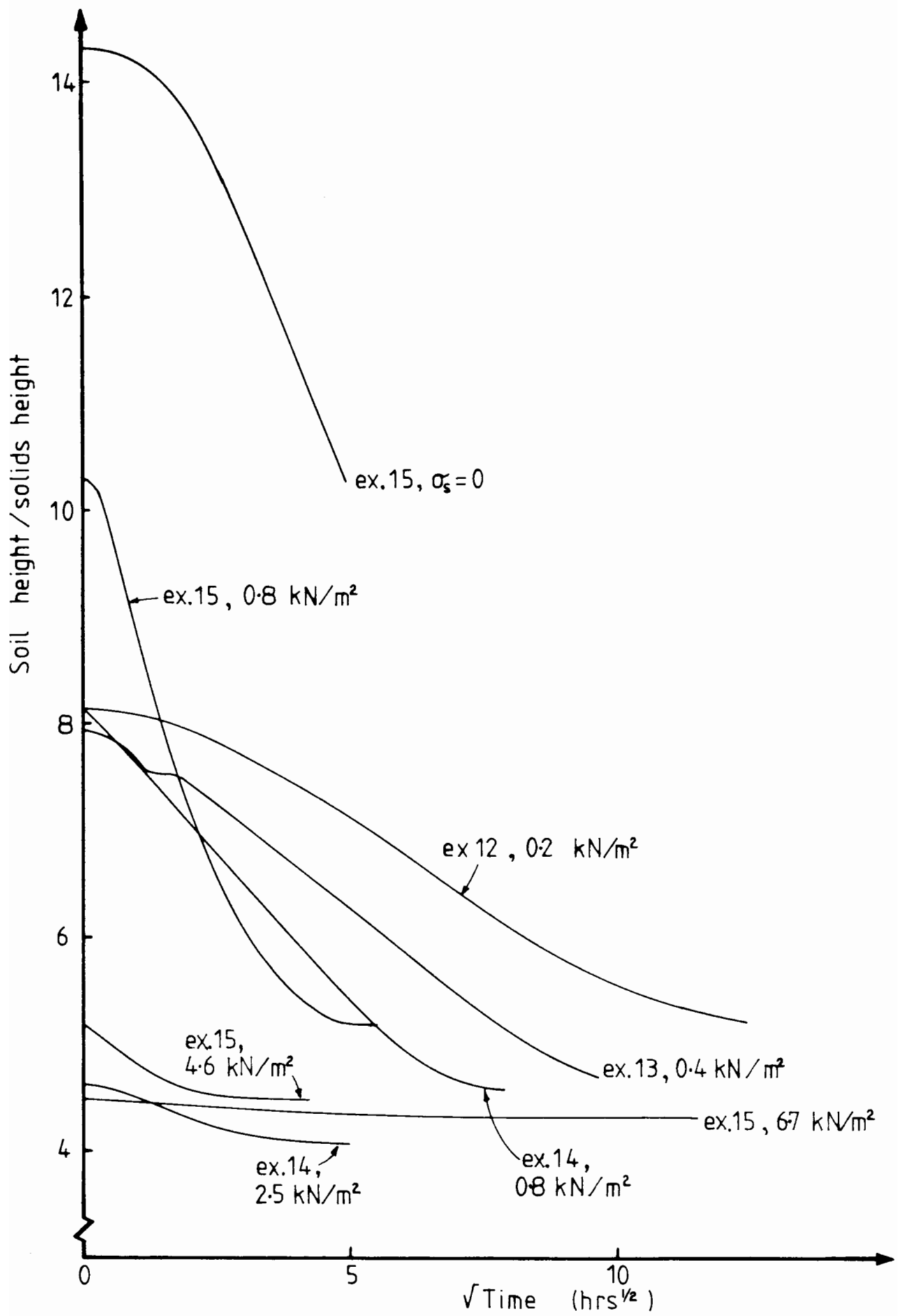


Figure 4.46 Expts 12, 13, 14, 15. Normalised settlements.

5.1 Introduction

5.2 Appropriate plotting forms for compression data

5.3 Variation in void ratio at zero effective stress, e_0^*

5.4 Compression behaviour

5.4.1 Soils deposited near the transition density

5.4.2 Soils deposited with effective stresses present initially

5.5 Consolidation under rapid loading - instant compression curve

5.6 Pore water flow

5.7 Discussion

5.1 Introduction

In this chapter one dimensional consolidation behaviour is analysed for soils compressing under self weight stresses alone and under very low applied stresses. Compression paths, represented by density-effective stress data, are presented for different soil elements within a single experiment and shown not to be unique. Further variation occurs between experiments depending on depositional conditions of density and height. Density changes occur at constant effective stress (as deduced from external measurements). This time dependent compression (creep) is shown to be of magnitude similar to or often greater than the 'primary' compression occurring due to effective stress change alone, and is typified by the large density changes able to occur at the surface of consolidating sediment where the vertical effective stress is zero. The resulting difficulties in predicting consolidation behaviour at these low stress levels are enhanced by non-Darcian and non-unique pore water/soil relative flow relationships.

5.2 Appropriate plotting forms for compression data

Compression data for soils are most often plotted in the form void ratio (or specific volume, $v = 1+e$) - $\log(\text{effective stress})$. This is suitable for soil states dryer than the liquid limit where compressive strains are small but considerable curvature results in relationships at low density states or for soils sedimented and consolidating from a slurry. A logarithmic void ratio axis is sometimes used to linearise these plots. Butterfield (1979) considered natural volumetric strains for an increment of compression from initial specific volume and mean effective stress state (v_o, p_o') to final state (v, p'), to suggest that a more appropriate plotting form is $\log(\text{specific volume}) - \log(\text{mean effective stress})$. Using this form he showed that many data plot linearly and can therefore be represented by the relationship

$$\ln\left(\frac{v}{v_0}\right) = -C \ln\left(\frac{p'}{p_0'}\right) \quad (5.2.1)$$

where the first term is the change in natural volumetric strain during loading from (p_0', v_0) . He suggested that the second term $\ln(p'/p_0')$ might be called the natural stress although it has a dimensionless form. This approach using natural strains is extended here to a logical form suitable for one-dimensional sedimentation and consolidation.

The natural volumetric strain for a material compressed to volume V can be defined as

$$\epsilon_n = \int_{v_0}^v \frac{dv'}{v'} \quad (5.2.2)$$

where v_0 is the unstressed volume. To replace the actual volume, V , by an appropriate soil density parameter it is suggested further that a natural compression law would predict zero volume at infinite stress. In this case the void ratio is a suitable parameter since the void volume is able to reduce to nearly zero for infinitely stressed clays, whereas the specific volume tends to a value of unity. (In other respects the specific volume is an easier parameter to use since it is directly proportional to actual volume and therefore, in one-dimensional compression, the soil height. This advantage is less important in deep, soft soils where self weight causes the specific volume to vary with depth.) The natural volumetric strain for soil (voids) may then be written from equation 5.2.2

$$\epsilon_n = \int_{e_0^*}^e \frac{de'}{e'} = \ln e - \ln e_0^* \quad (5.2.3)$$

$$\text{so that} \quad \log e = \log e_0^* + \epsilon_n/2.3 \quad (5.2.4)$$

The parameter e_0^* represents the void ratio at which effective stresses just begin to act, and was discussed in chapter 2, where it was suggested that it might define the state at the transition from a suspension to a consolidating, structured phase. It is clearly of great importance to determine whether a unique value of e_0^* might exist for a particular soil or how it will vary with different factors.

Butterfield suggested that $\log(v) - \log(p')$ data plot linearly. In the present study it was not possible to determine σ_h' to a high degree of accuracy

so that p' was not known, and the vertical effective stress σ'_v is used instead. Initial results are plotted in the standard $e - \log \sigma'_v$ form for ease of interpretation. From this point onwards the vertical stress is assumed where σ' is used without subscript.

5.3 Variation in void ratio at zero effective stress, e^*_0

Figure 5.1 shows void ratio changes at the surface of consolidating soils, subject to different initial and boundary conditions, plotted against time on a logarithmic scale. Data have also been analysed from experiments KB10-15 carried out at Oxford by Been (1980) and RTC12 (Sills & Thomas, 1983), which have not previously been compiled in this form. In most experiments the surface density is clearly defined by the continuous profile. Where it is not, the intersection point of the density profile extended from its slope lower in the soil with the surface height is used. This method seems justified since results will be consistent and representative of consolidating soil beneath. Been and Sills (1981) showed that a slurry of Combwich soil deposited rapidly exists initially as a fluid supported suspension if the initial void ratio, e_i , is greater than 12. This suspension, in which effective stresses are zero (pore pressure equal to total vertical stress as integrated from density profiles), settles rapidly to form a consolidating layer, in which effective stresses exist. This behaviour is indicated clearly by fresh water experiments in figure 5.1. For initial void ratios below about 12, there is no suspension phase present and the surface density increases slowly with time. For $e_i > 12$ the void ratio increases initially at the surface, probably due to segregation of coarser particles which fall through the suspension to the base. When the surface of the falling suspension meets the surface of the consolidating layer beneath, an apparent density step occurs to a void ratio which depends on the suspension density but is in no case lower than 12, i.e. the value which determined whether a suspension would exist. Following this a gradual decrease in surface void ratio occurs on a much longer time scale at a similar rate to

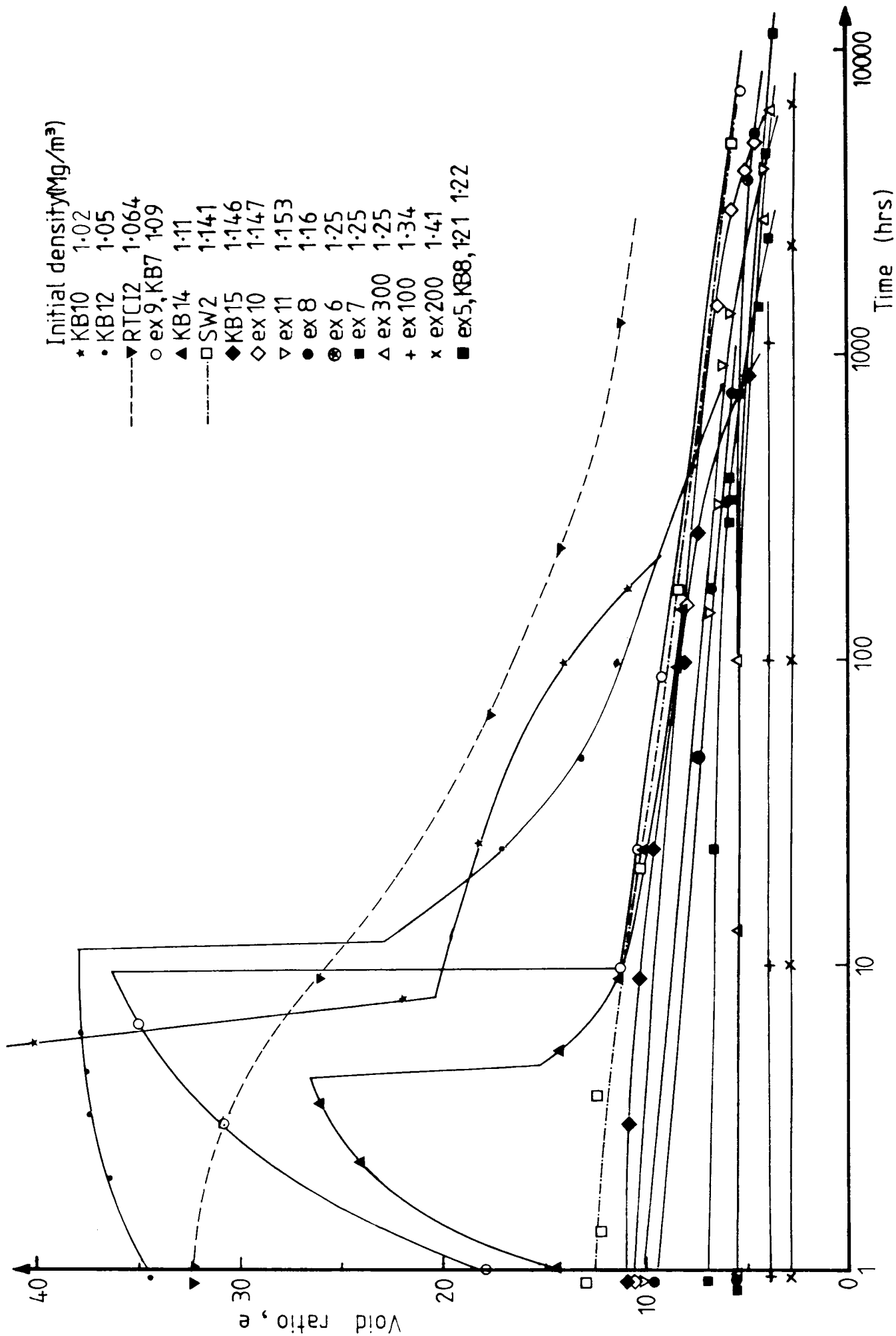


Figure 5.1 Void ratio changes at surface of mud during periods of settlement and consolidation.

that in sediments which never experienced a suspension phase. This void ratio around 12 is a critical value for fresh water slurries of this soil deposited rapidly at uniform density. Below this value the slurry behaves immediately as a consolidating soil while above this value no definite structure is present and the system is highly unstable; segregation and large deformations occur. It therefore provides a good initial estimate for the value of e_0^* , the zero effective stress void ratio defined earlier. It is worth noting that this value ($e_0^* = 12$) is approximately seven times the void ratio at the liquid limit ($e_{LL} = 1.7$ for fresh water Combwich soil); the same factor of seven was suggested by Carrier et al (1983) for other soils. The salt water experiment, SW2, has an initial void ratio of 13 which is slightly higher than the transition value for fresh water soil, but the subsequent decrease indicates that it belongs to the consolidating phase. The transition value for salt water soils may therefore be slightly higher, possibly due to formation of larger or more structured flocs.

It is interesting to compare the surface void ratios of the consolidating soil following a suspension phase with those where a suspension never existed. The suspension has the effect of allowing the consolidating layer to accumulate slowly, so that structural strength is able to develop before significant load increase occurs. Consequently the surface void ratios are able to exist at values higher than the transition value for rapid deposition or accumulation (approximately 20 for KB12, $e_i = 34$ and 23 for KB10, $e_i \approx 96$). These high values may in part be due to the loss of coarser particles from surface layers but also suggest that slower accumulation rates will allow less dense structures to be developed and maintained. This is borne out by the curve for RTC12. In this experiment soil was deposited steadily at a solids rate 7 g/hour for 105 hours, resulting in a final solids mass of 0.72 kg, similar to that in short column experiments 300 (0.65 kg). At the end of deposition the surface void ratio was 33, but there was no evidence of the presence of a suspension phase or subsequently of segregation. The surface void ratio

decreases very slowly, showing no sudden change, and even after 500 to 1000 hours is at a value which for a rapidly deposited slurry would be associated with a suspension. In experiment KB10 the surface void ratio after the completion of the suspension phase (i.e. deposition for about 8 hours) is close to that in RTC12, but the subsequent rate of decrease is much higher.

Figure 5.1 indicates clearly that the depositional characteristics, including initial density, height, rate and time of deposition, are of major importance in determining the subsequent compression behaviour and eventual consolidation states of a sediment. A single value of e_0^* is not appropriate since its initial value is dependent on the characteristics above, and in addition it then decreases with time. Even after 10,000 hours stable values had not been reached although results in chapter 4 suggest that pore pressures in the soil below were almost fully dissipated at this time. This provides strong evidence for the important influence of creep in soil compression at these states.

5.4 Compression behaviour

Results obtained by other workers and discussed in chapter 2 suggested that unique compression behaviour was often not observed in soils at very high void ratio states. Been (1980) suggested that for Combwich soil at void ratios higher than about 6, a unique compression relationship ($e - \sigma'$) did not exist while below this value $e - \sigma'$ curves for all soil elements converged to a unique consolidation line for this soil as consolidation times increased. In this section compression behaviour observed in experiments is described and trends evaluated and compared.

Density and pore pressure profiles were presented for experiments in chapter 4. The total vertical stress may be calculated at any section in the soil by numerical calculation of the integral of $\rho g dz$ between the section and the soil surface. The effective stress is then given by the difference between the total stress and the pore pressure at that section. Void ratio is

calculated directly from the density using equation 2.1.1. It is most useful to obtain the consolidation path followed by a particular soil element, rather than by elements passing a particular point in the column, and this is done by calculating (e, σ') points at constant values of the excess total stress above hydrostatic. Consolidation paths followed by elements under different conditions are presented in the next two sections.

5.4.1 Soils deposited near the transition density

Experiments 10 ($e_i = 10.5$), 11 ($e_i = 10$) and SW2 ($e_i = 13$) were all begun by rapid deposition of slurry at void ratios slightly below the transition region discussed in the previous section. All might therefore be expected to show an extreme of consolidation behaviour for soils which never experienced a suspension phase. Figure 5.2 shows consolidation paths followed by different soil elements in experiment 10. Each element is defined by a constant normalised material co-ordinate, η , using the soil surface as a datum, where

$$\eta = \frac{\text{mass of solids above the element}}{\text{total mass of solids present}}$$

(N.B. In contrast to this definition Been (1980) used the column base as the co-ordinate datum).

Solid lines follow elements of constant η and dotted lines are isochrones connecting all points in the soil at stated times. The first point of interest is that during the first 24 hours no compression occurs, but effective stresses increase in the soil. (This behaviour was indicated in chapter 4, figure 4.38 by pore pressures decreasing below the total stress.) During this period soil structure must be developing by a form of flocculation with interfloc contact; permeabilities are correspondingly reduced so that settlements are retarded. More important for subsequent behaviour is that effective stresses do not increase to the same value so that variable stress history between soil elements is created immediately. Following this initial period of structure development compression occurs in the soil and consolidation paths in figure 5.2 move downwards but remain separate and nearly parallel for different elements. Large void ratio decreases occur with very small changes in

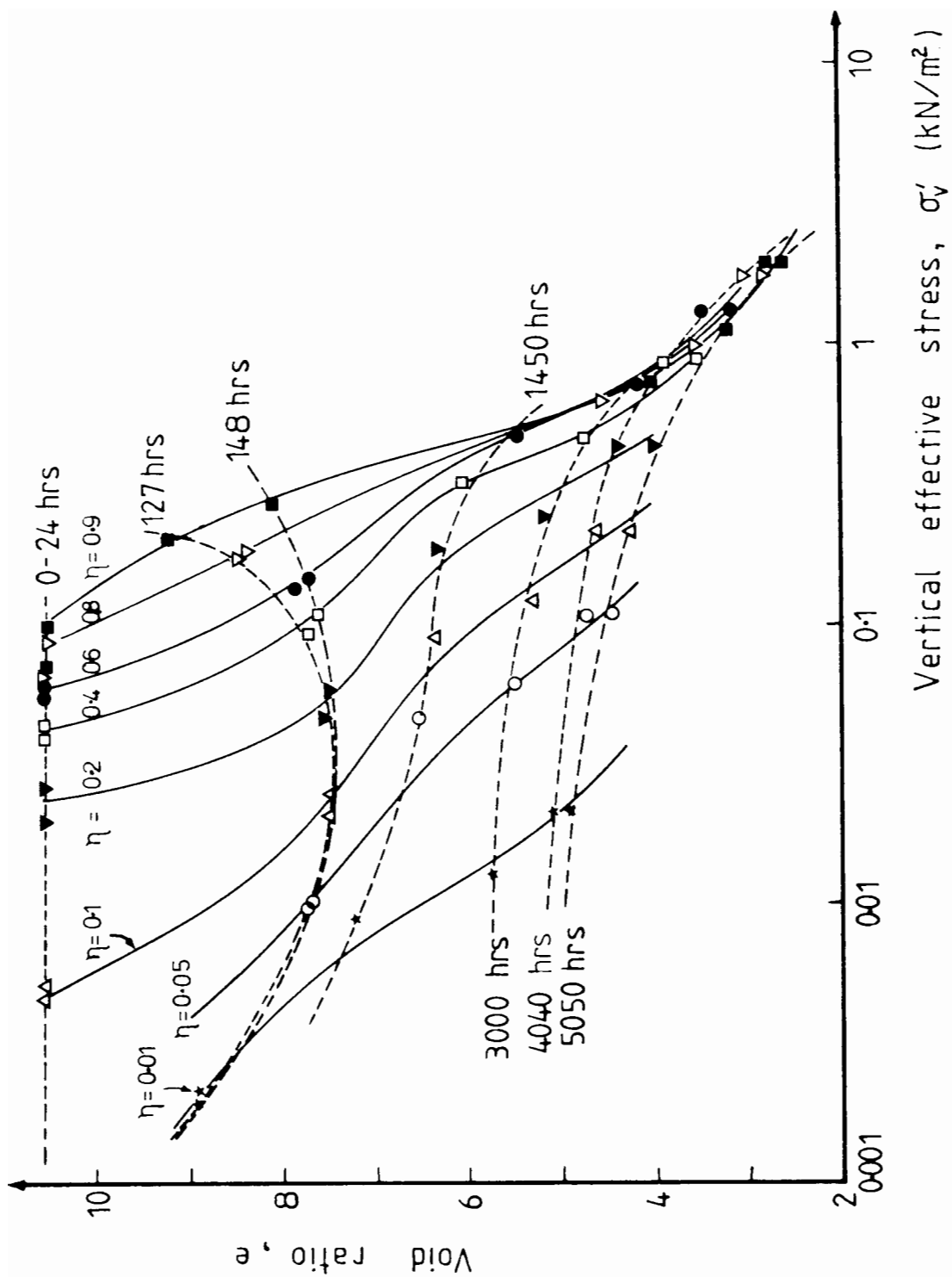


Figure 5.2 Consolidation curves for soil elements in expt. 10, $e_i = 10.5$.

effective stress. Any particular effective stress level may be associated with a wide range of void ratios, suggesting that time dependent compression, or creep, is occurring. There is no indication of a unique consolidation curve being attained even after very long consolidation times; after 4040 hours pore pressures were almost fully dissipated (figure 3.38). The low density region seen near the base in figure 3.38 after short consolidation times is reflected in figure 5.2 where higher void ratios occur with higher effective stresses at these times.

Figure 5.3 shows consolidation paths for elements in experiment 11, similar to experiment 10 except that base drainage was permitted between 330 and 900 hours. Compression is slightly faster than in experiment 10 but overall behaviour is very similar. After 1370 hours in experiment 11, the element at $\eta = 0.8$ has void ratio $e = 3.6$ at effective stress $\sigma' = 1.2 \text{ kN/m}^2$ and the element at $\eta = 0.1$ has $e = 5.6$ at $\sigma' = 0.06 \text{ kN/m}^2$. In experiment 10, corresponding values at 1450 hours are $e = 4.5$, $\sigma' = 0.64 \text{ kN/m}^2$ at $\eta = 0.8$ and $e = 6.3$, $\sigma' = 0.09 \text{ kN/m}^2$ at $\eta = 0.1$. After 4040 hours, when effective stresses are almost equal for corresponding elements in experiments 10 and 11, void ratios are very similar near the base of each soil although near the surface they have remained slightly higher in experiment 10.

Figure 5.4 shows consolidation paths for elements in experiment SW2, begun at higher void ratio (13) than experiments 10 and 11, although it is not known how close this value is to the transition value for salt water soil. Significant compression has already occurred by the time effective stresses can be measured sufficiently accurately to be shown in figure 5.4. The pattern therefore appears to be one of large reductions in void ratio at very low stresses, apparently mainly as creep compression, followed by compression which is more stress dependent. Consolidation paths remain separated for different elements.

Consolidation paths for fresh water (experiment 10) and salt water (experiment SW2) experiments are compared in figure 5.5. Elements are chosen

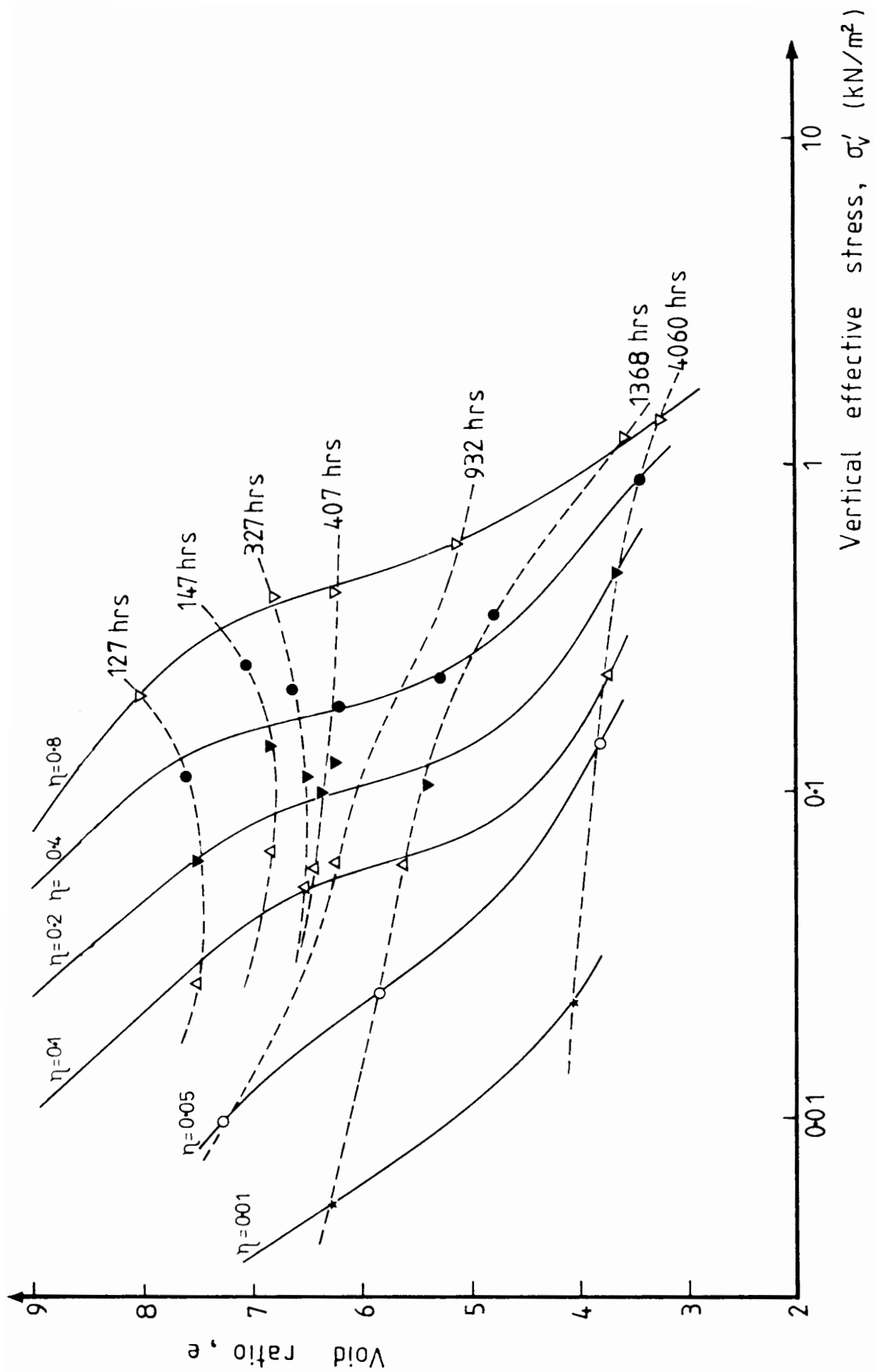


Figure 5.3 Consolidation curves for soil elements in expt. 11, $e_i = 10$.

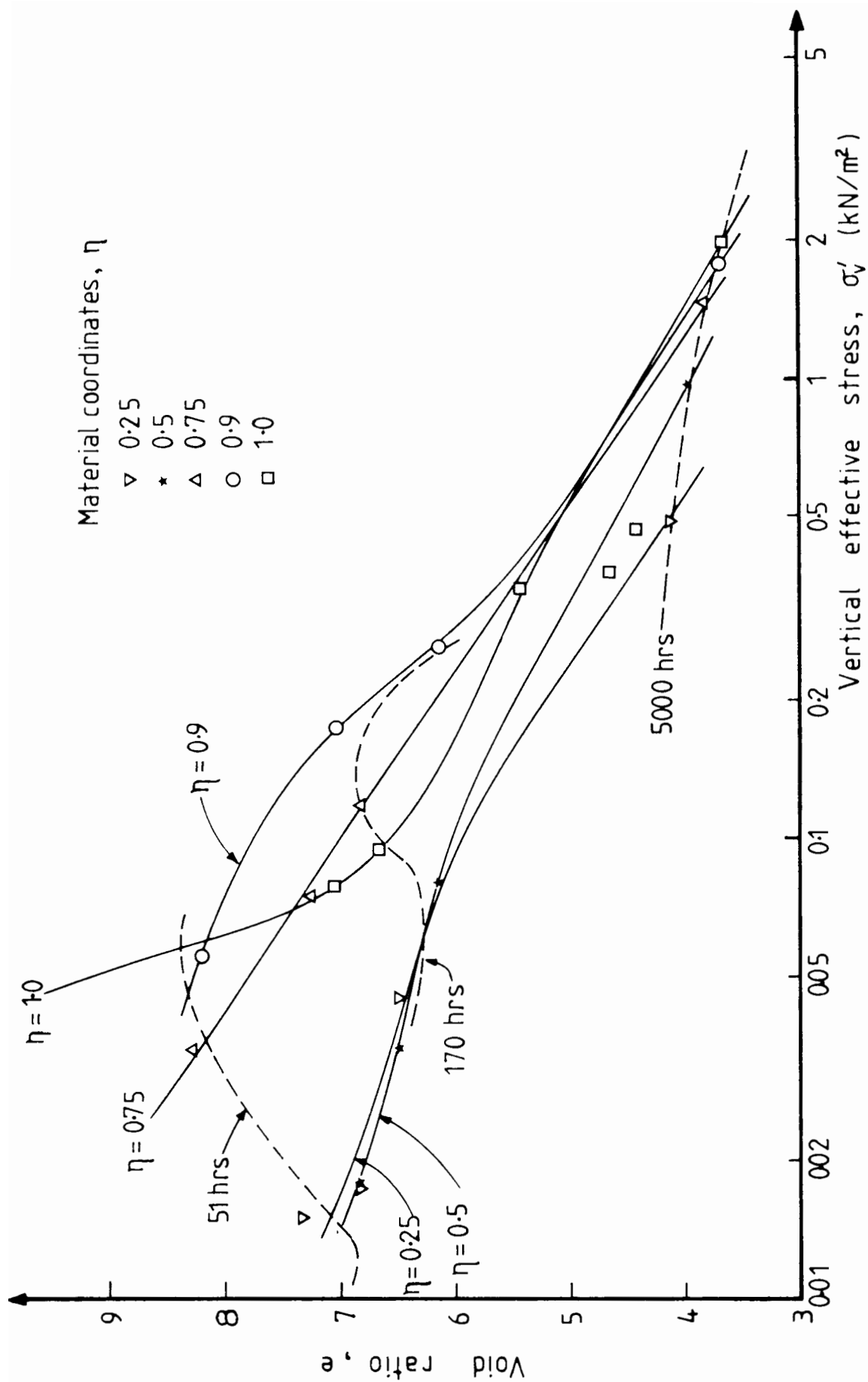


Figure 5.4 Consolidation curves for soil elements in SW2. $\rho_i = 1.141 \text{ Mg/m}^3$.

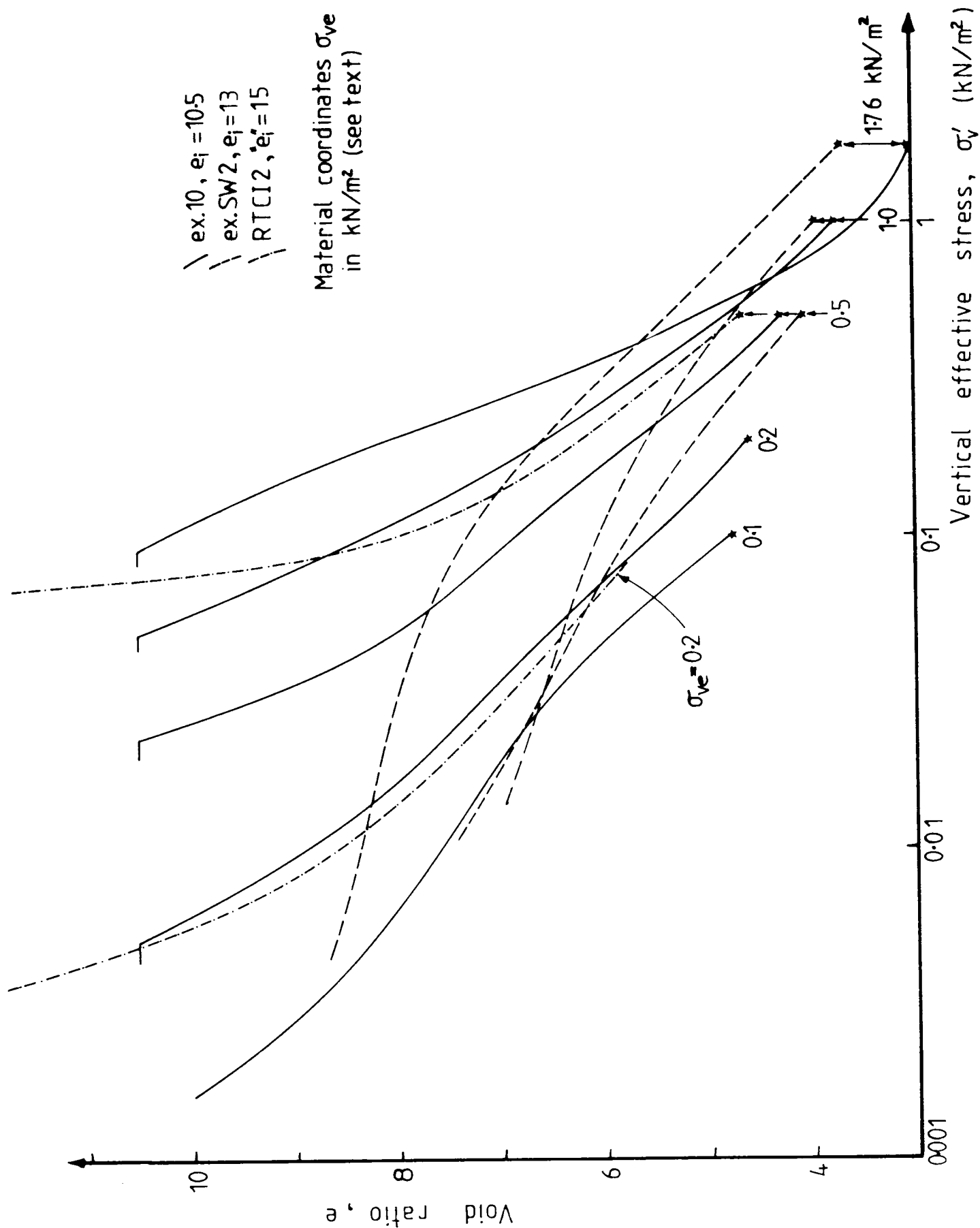


Figure 5.5 Consolidation paths followed by equivalent elements in soils under different conditions.

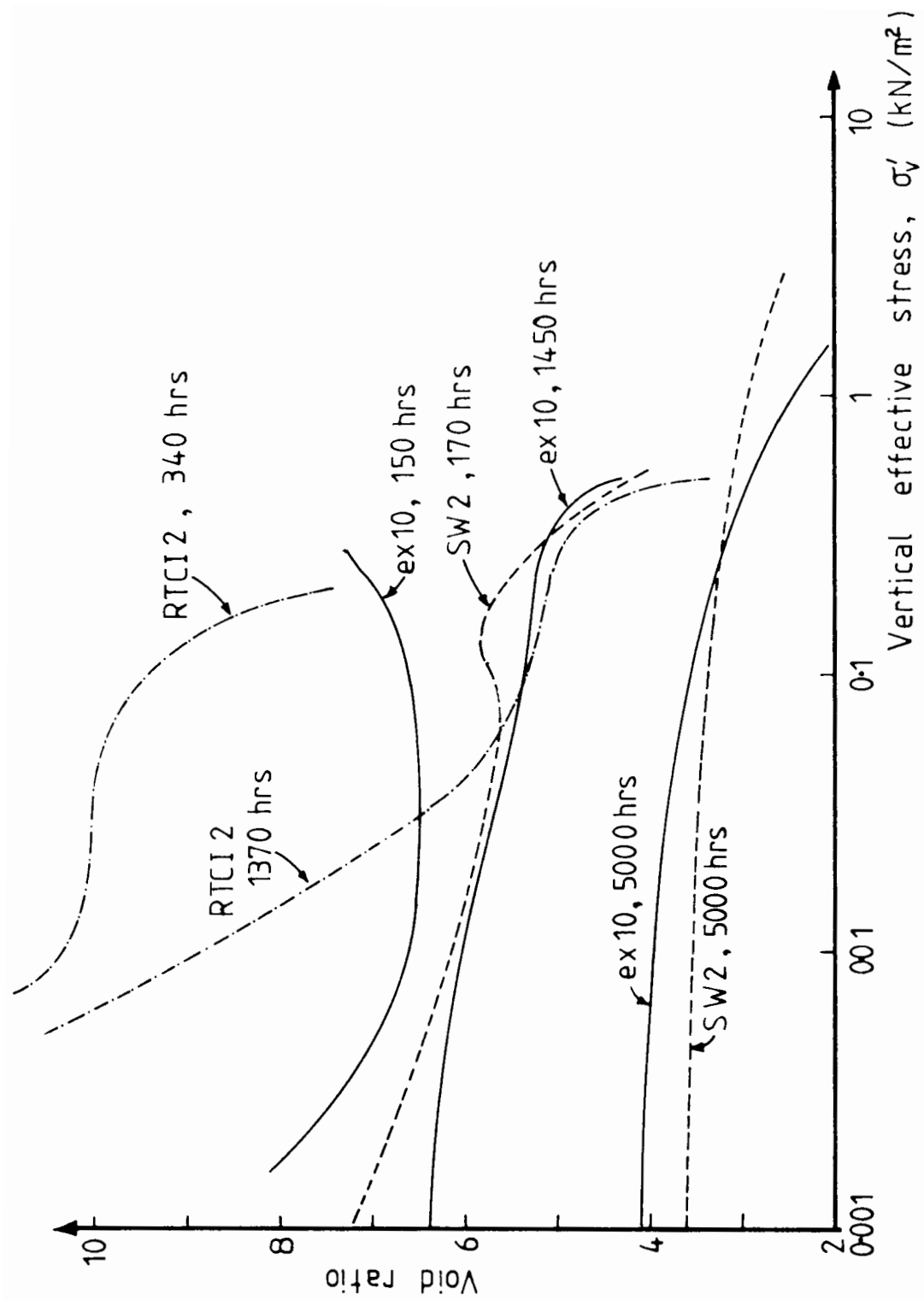


Figure 5.6 Consolidation states achieved in soils under different conditions.

which have the same actual (not normalised) material co-ordinate below the surface, i.e. the excess total stresses or masses of solids above compared elements are equal so that eventual effective stresses will be the same. A major difference in behaviour can be seen at earlier times during consolidation, but paths appear to be quite similar in shape after longer periods. The conclusion, surprising perhaps, is that the initial structure developed upon deposition in salt water is less stable than that in fresh water and significant effective stresses are unable to be maintained until the void ratio has decreased to around 8 or 9. Thereafter behaviour is similar to that for fresh water soil. The different initial void ratios in the two experiments may account for some of the behavioural differences and further investigation of salt water behaviour is required. In particular it will be necessary to establish for salt water suspensions the suspension/structured phase transition void ratio indicated for fresh water soils in figure 5.1.

The consolidation stress history variations indicated in figures 5.2 to 5.5 suggest that depositional characteristics are of great importance. Consolidation of a sediment deposited at a slow, uniform rate might be expected to exhibit further variable behaviour. To investigate this, consolidation paths for the example used already, RTC12 (7 g/hr for 105 hours), are also shown in figure 5.5. The same material co-ordinates are chosen as for experiments 10 and SW2. Density profiles have been presented elsewhere (Sills & Thomas, 1983) and show that the average void ratio after 114 hours (9 hours after completion of deposition) was about 15 and up to 26 at the surface, although a suspension phase did not exist. Consolidation paths are very similar in shape to those observed in experiment 10. Elements nearer the base of the soil are able to achieve and sustain higher effective stresses at given void ratios, and compression occurs at nearly constant stress. A major difference between experiments 10 and RTC12 however was indicated in figure 5.1 by the very slow surface density increase in the experiment begun by gradual deposition. This can also be seen in figure 5.6 where consolidation states

attained by all points in different soils are shown at particular times. The highly variable consolidation behaviour is illustrated extremely well.

The three soils shown all begin with deposition at or near suspension densities, so that initial deposition characteristics might be expected to have insignificant effect on consolidation behaviour once compression has occurred to lower void ratios. Consolidation states are compared first at times 150 hours (experiment 10) and 170 hours (SW2) after deposition. The similar curve shapes reflect the similarly shaped density profiles seen at these times in figures 3.38 and 3.45, including the inverted density gradients near the base that are characteristic of experiments begun at these densities. The salt water soil however has attained denser states throughout despite deposition at a less dense state than the fresh water soil. (Thin, dense layers formed near the base of each soil at these times as shown in figures 3.38 and 3.45 are not shown in this figure as they are not representative of the general soil behaviour). In contrast to both these soils is the state attained in experiment RTCI2 after 340 hours (230 hours after the end of deposition). Only near the base is the void ratio similar to those achieved in experiment 10 at the shorter time. Higher in the soil states exist at void ratios similar to those at deposition in the two experiments deposited rapidly. Experiments 10 and RTCI2 are also compared after 1400 hours; at effective stresses above about 0.1 kN/m^2 consolidation states are very similar but higher void ratios are maintained in RTCI2 nearer the surface. The soil height in experiment 10 at this time is 875 mm while it is only 250 mm in RTCI2; despite the far shorter drainage path compression has not occurred at a significantly faster rate in RTCI2 up to this time. The element at which 0.1 kN/m^2 effective stress occurs is about 110 mm below the soil surface in experiment 10 at this time and about 130 mm below the surface of RTCI2. This surface layer is of particular concern where, for example, erodibility is important so that the void ratio variations in this layer may be of considerable engineering significance. A third comparison is made between the salt and fresh water soils after 5000 hours of

consolidation at which time measured pore pressures were essentially hydrostatic in each. The void ratio variation is considerably greater in the fresh water soil, particularly at higher stresses. The "flat" consolidation state seen for elements in SW2 is typical of soils where creep, rather than stress dependent compression, has governed volume change so that void ratio states attained are relatively stress independent.

In summary of this section, the general variability of consolidation paths and states is emphasised. Figure 5.6 shows that in soils subject to effective stress levels around 0.05 kN/m² different structures may develop under different conditions allowing void ratios to vary from 4 up to 11 during consolidation.

5.4.2 Soils deposited with effective stresses present initially

Experiment 5 was deposited at void ratio $e_i = 7$, well below the transition value, so that effective stresses would be expected to act immediately. This is borne out by the consolidation paths plotted for different elements in figure 5.7. Near the column base (before base drainage was permitted) effective stresses up to about 1 kN/m² are present with very little change in void ratio. Subsequent consolidation paths follow the patterns observed already, with slow compression occurring while effective stresses remain fairly constant. The base drain was opened to the free water surface after 5.9 hours and explains the more rapid consolidation near the base. Immediately following the X-ray density profile taken at 460 hours the column was segmented for strength testing. It is worth comparing results with those for a similar experiment, KB8, for which density and pore pressure profiles are reported by Been (1980). Three consolidation paths are calculated and also shown in figure 5.7 and the trends recorded at earlier times in experiment 5 of this study are continued.

Three other experiments (experiment 6, $h_i = 1748$ mm; experiment 7, $h_i = 1197$ mm; experiments 300, $h_i = 200$ mm) were begun at void ratio $e_i = 5.7$. Consolidation states observed at different times are compared in figure 5.8. The profile for experiment 6 at 20 hours shows that stresses up to 3 kN/m² were

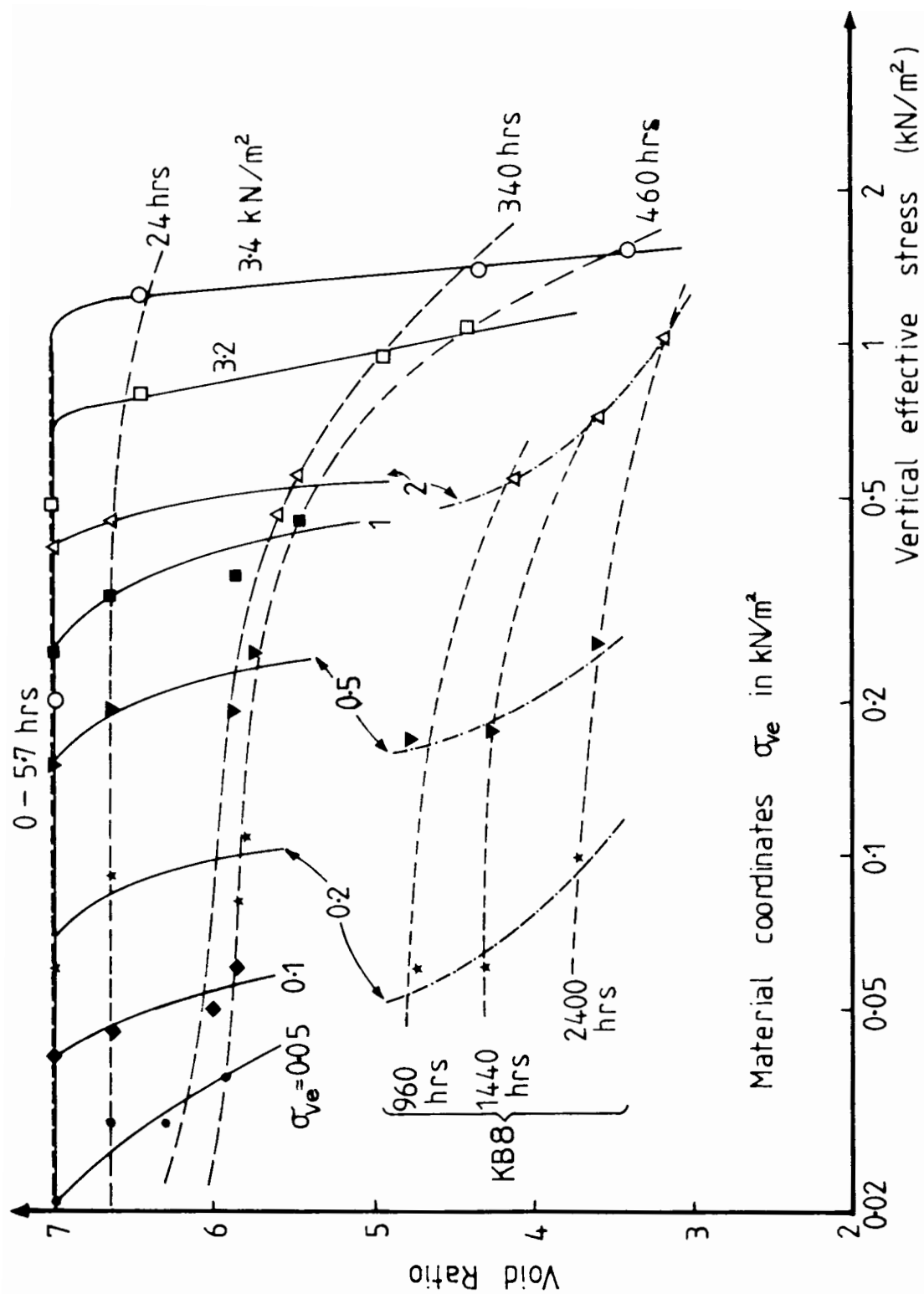


Figure 5.7 Consolidation paths and states for elements in expt. 5, $e_i = 7$. Also paths for KB8 (original data from Been, 1980) $e_i = 6.6$

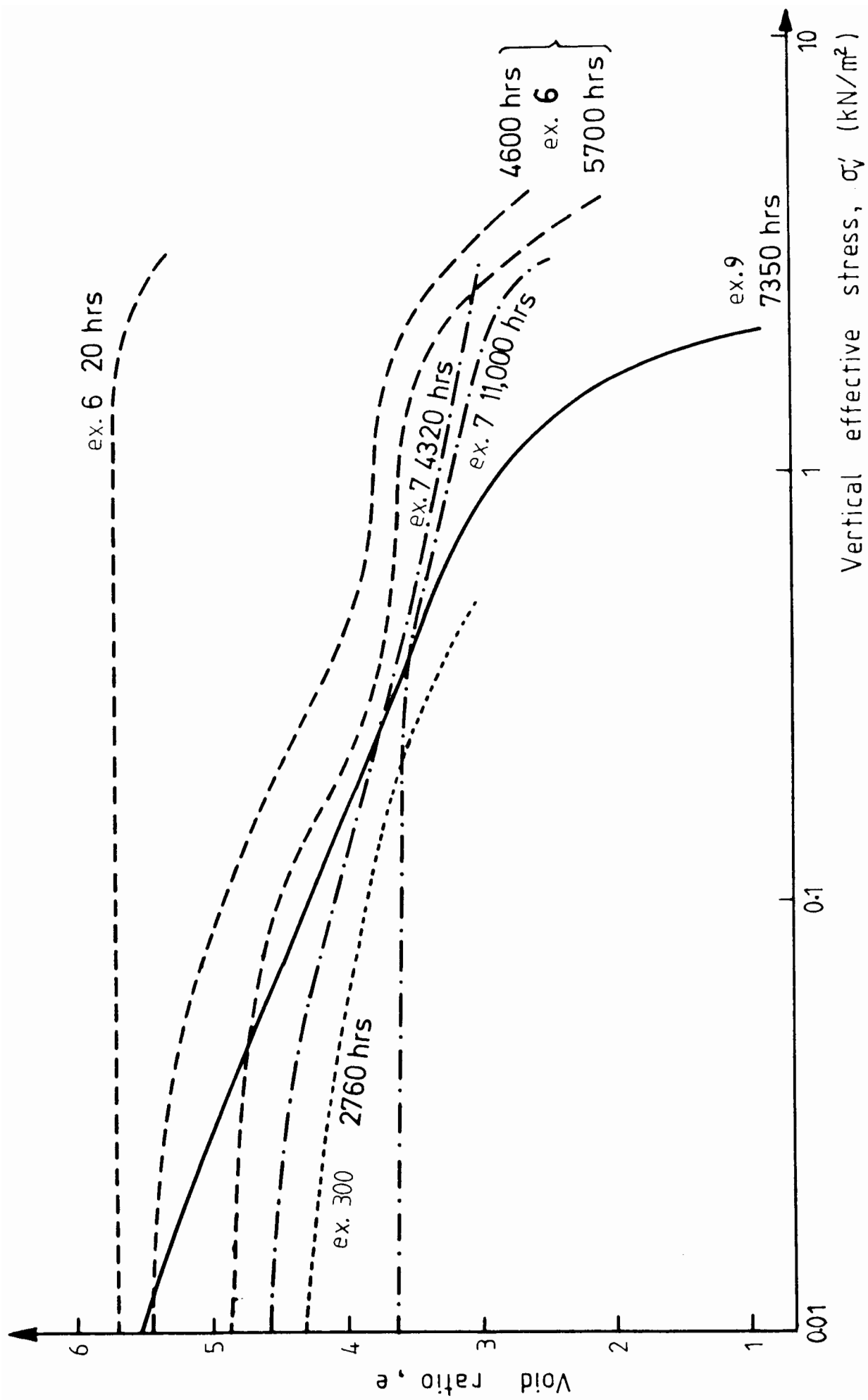


Figure 5.8 Stress states in soil at various times after deposition showing effects of initial conditions and stress history. Lines connect all points in soil at stated time.

sustained with very little immediate compression. It can be seen that shorter soil heights result in greater compression at any particular stress level and time. At an effective stress 0.5 kN/m^2 for example, experiments 300 have a void ratio of 3 at the soil base after 2760 hours, when pore pressures have fully dissipated. The void ratio in experiment 7 at the same stress is 3.5 after 4320 hours and in experiment 6 at 4600 hours it is close to 4, even though double drainage was permitted in this experiment. Only after much longer periods (up to 11,000 hours) does the void ratio near the surface in experiment 7 fall below that after 2760 hours in experiments 300. This pattern is different to that discussed in the previous section where surface densities in the soil deposited slowly remained very low for long periods, despite the shorter overall height. The implication is that the depositional rate and initial soil height both strongly influence the subsequent consolidation behaviour, the first by determining initial soil structure and the latter probably by controlling the drainage path length. Comparison of figures 5.6 and 5.8 suggests that deposition rate is the more important factor (although initial void ratios are not comparable) since greater subsequent variability occurs in consolidation behaviour.

The consolidation state in experiment 9 after 7350 hours is also shown in figure 5.8. The likelihood of significant segregation in this experiment, due to the initial height (3.5 m) and lower initial density (1.09 Mg/m^3), was discussed in chapter 4 and the effects can be seen in the much steeper curve representing the consolidation state. Coarser particles near the base cause very low void ratios to be achieved while the predominance of finer material towards the surface allows void ratios to remain high.

5.5 Consolidation under rapid loading - instant compression curve

The existence of elements in a soil which have approximately the same void ratio but different effective stresses is behaviour typical of overconsolidated states. This pattern was seen in two different ways in figures 5.2 to 5.8. The first occurs when soil is deposited in a thick layer

at a density higher than the critical transition density. Effective stresses develop immediately, to about 0.1 kN/m² at $e_i = 10.5$ (expt 10, fig 5.2), to about 1 kN/m² at $e_i = 7$ (expt 5, fig 5.7) and to about 3 kN/m² at $e_i = 5.7$ (expt 6, fig 5.8). Soil in these states is essentially remoulded so that structure is probably isotropic, although this cannot be verified from the test data available. Shortly after deposition, elements are able to sustain effective stress increase with minimal volume change, as though returning along "swelling" lines from overconsolidated states. The upper limits on effective stress immediately attainable for input at any void ratio might therefore be defined by an instant compression curve which is valid for primary consolidation under loading sufficiently rapid that creep effects are negligible. As observed shortly after deposition, elements are able to move along the overconsolidation or swelling lines very quickly, by achieving effective stress increase with negligible volume change. Subsequent compression however will depend on the starting point (e, σ') following the initial stress increase. If the rate of stress increase is sufficiently fast, elements will move to the instant compression curve and proceed to consolidate, initially at least, to $e-\sigma'$ states defined by this curve. Changes from the isotropic structure present initially however will probably cause elements to move from this curve to a similar curve for anisotropic structure. Elements which initially do not reach the stresses required by this curve at particular void ratio, or for which the rate of stress increase is very slow, will show behaviour dominated by time dependent compression (creep) and will therefore exhibit volume changes at nearly constant effective stress.

The second type of pattern results when creep compression occurs much faster than primary (stress dependent) consolidation. This behaviour is well documented for stiffer soils and was described in a logical manner for in situ clays by Bjerrum (1967). The difference between normal assumptions about creep compression and the results observed here is that in these experiments the creep component appears comparable to, or even dominates,

the primary compression. Conventional analyses assume that while creep may always be present, the creep rate is sufficiently slow that it may be assumed negligible until primary compression is nearly completed. The consequence for subsequent loading of this second type of overconsolidation behaviour is that a unique consolidation curve will only be recorded if rates of stress increase are sufficiently fast to return $e-\sigma'$ states to the primary consolidation curve. Evidence for the behaviour discussed hypothetically above is presented in this section, obtained from the following sources:

1. A restricted flow (small pore pressure gradient) oedometer consolidation test was performed on a sample taken at height 20 mm in experiment 10 during unloading (5050 hours). This corresponded to the element at normalised material co-ordinate 0.9 in figure 5.2.
2. Surcharges were applied at the surfaces of experiments 2,4,11 and SW2 following periods of self weight consolidation. Although complete consolidation was not achieved throughout the soil in most cases, the volume change very near the surface was sufficiently rapid to define $e-\sigma'$ points for these loadings.
3. Experiments 12, 13, 14 and 15 consisted of various consolidation increments for samples of very low initial height. Settlement results were discussed in chapter 4.
4. Soil in a 210 mm segment at the base of experiment 5 was removed after unloading (520 Hours) and loaded with surface surcharges then consolidated isotropically in a triaxial cell up to 75 kN/m² before shearing.

Results of these tests are shown in figures 5.9 and 5.10. Figure 5.9 shows the end states of each consolidation increment for experiments 12 to 15. Stresses are calculated at sample midheights (represented by the first shown point for each experiment before load application) but self weight stresses are sufficiently small compared to surface loadings that void ratio uniformity can be assumed. Interpretation of consolidation paths between increment endpoints is subject to speculation. The shapes of settlement curves for several increments in figure 4.46 (expt.12 and the second increment in expt.14)

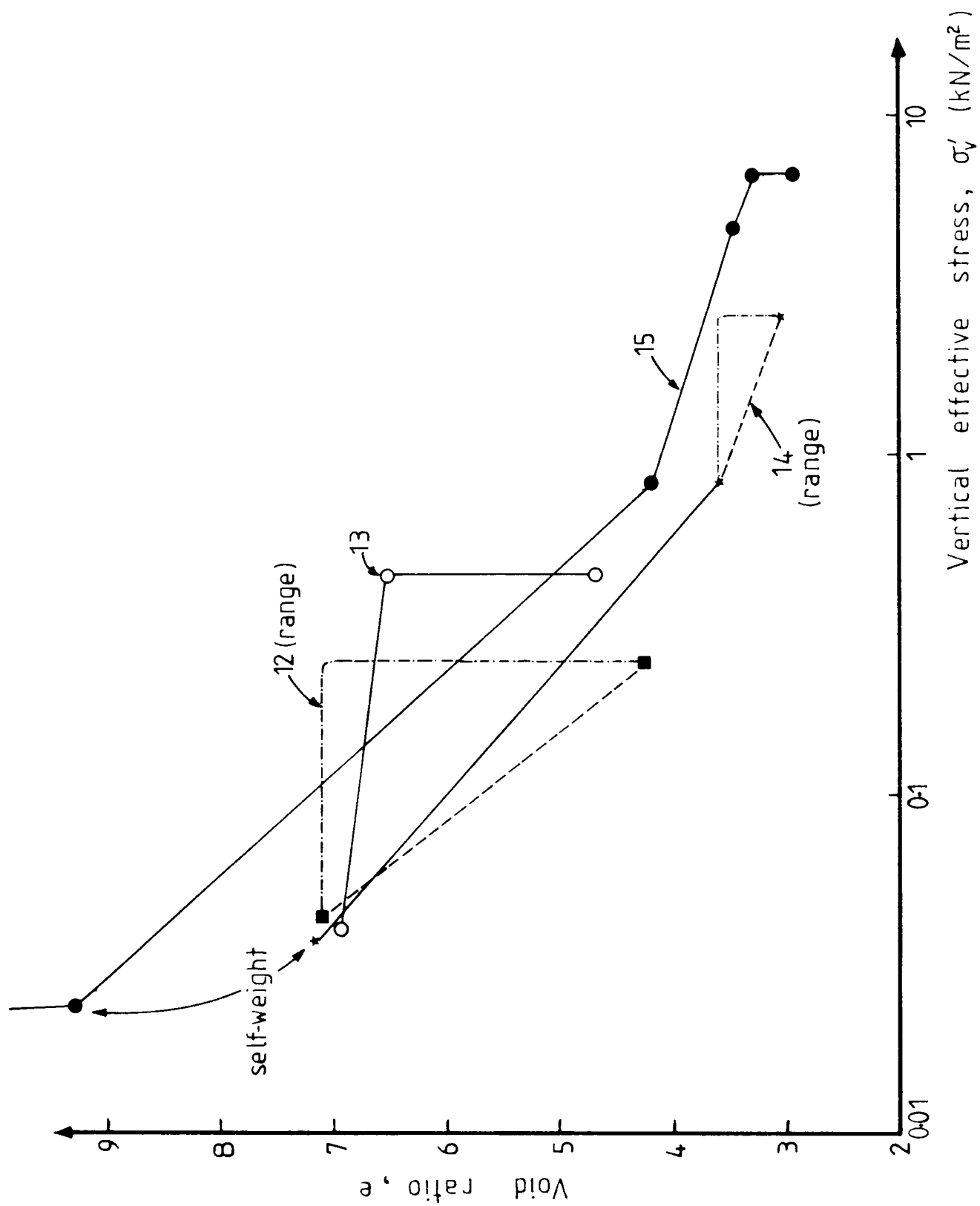


Figure 5.9 Consolidation behaviour in expts. 12 to 15.

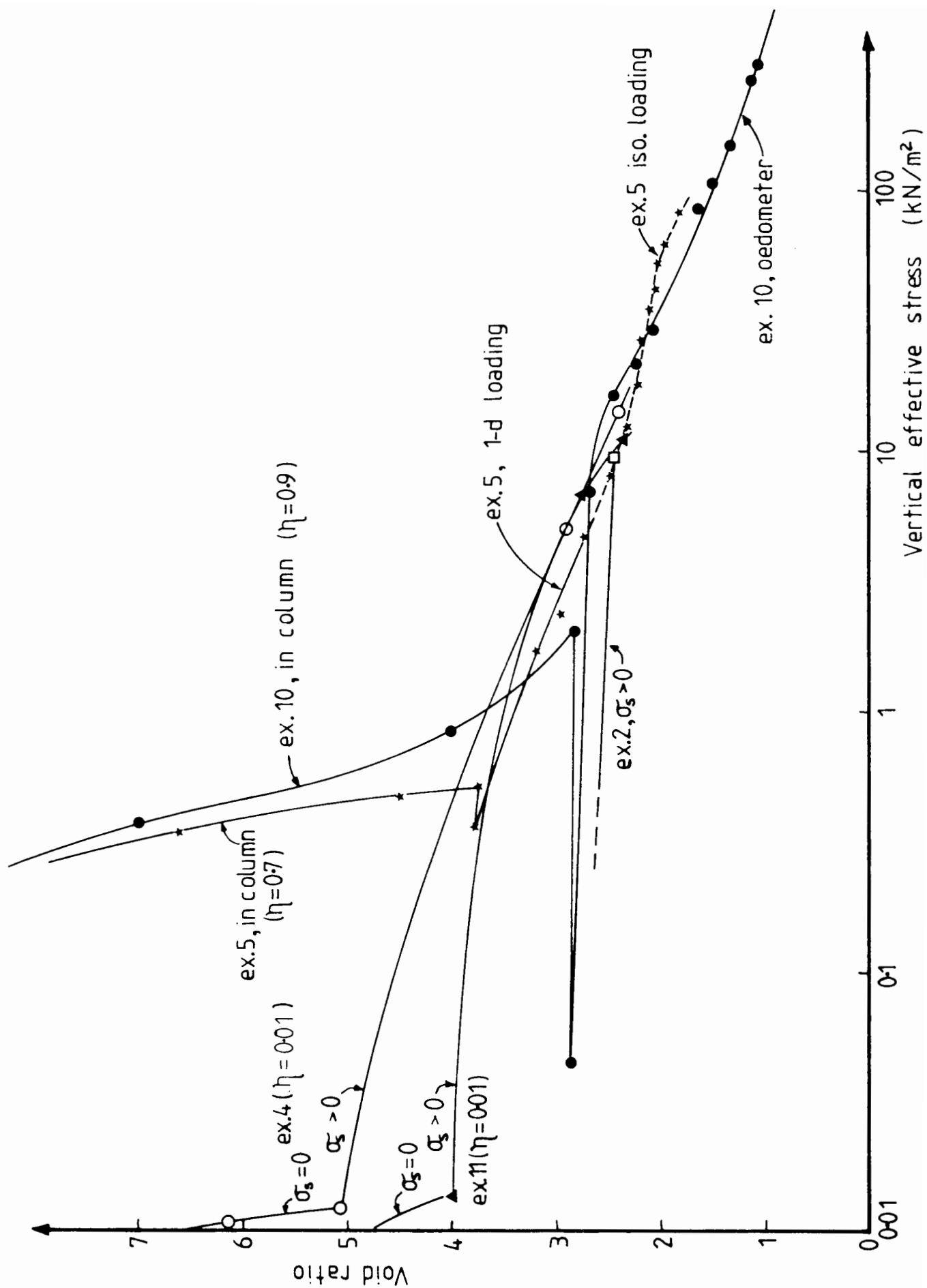


Figure 5.10 Consolidation of soil in columns subjected to increased loading.

suggested that samples existed in the overconsolidated states defined in the previous section. Subsequent behaviour in these two increments was therefore interpreted as stress increase at nearly constant void ratio, followed by creep compression at constant effective stress.

Figure 5.10 shows paths for surcharge load increments followed by elements previously consolidated under self weight alone in columns. In all cases stiff response is exhibited on reloading and consolidation paths tend towards one curve. This is represented best by results of the restricted flow (RF) oedometer test on the sample from $\eta = 0.9$ in experiment 10. Minimal swelling occurred during sample preparation. The start of the RF test is indicated by the point $e = 2.85$, $\sigma' = .045 \text{ kN/m}^2$, corresponding to the mean self weight stress in the 20 mm sample. An apparent preconsolidation pressure is indicated by the sharp curvature at an effective stress about 12 kN/m^2 and subsequent behaviour follows a typical curve for a normally consolidated soil. Consolidation states for elements near the loaded surfaces in experiments 2, 4 and 11 lie close to this line. Points for loading of the base sample from experiment 5 lie below this line, but the sample thickness (210 mm) may have prevented primary consolidation from occurring sufficiently quickly and some creep compression may have occurred. The sample was subsequently loaded under isotropic applied stresses and again relatively stiff response is observed, as the consolidation state moves towards an isotropic instant compression curve beyond the one-dimensional curve. This may have been reached at higher stress increments as the consolidation path moves downwards at stresses higher than 40 kN/m^2 . It should be noted that for this isotropic loading the vertical effective stress was almost equal to the mean effective stress, (a small deviatoric stress component was maintained); if K_0 is less than 1 the mean effective stress in one-dimensionally consolidated samples will be less than σ' . The curves in figure 5.10 therefore do not fully indicate the difference between the isotropic and one-dimensional states.

Evidence for existence of "upper bound" instant compression curves is compiled in figure 5.11. All points from figures 5.9 and 5.10 which appeared to approach such curves are shown. Points defining the maximum stresses obtained at deposition before significant volume change occurs are also shown. These maximum depositional stress points define an instant compression curve for isotropic structure at high void ratios which is constructed using the isotropic curve suggested by experiment 5 triaxial increments, although these points represent stress isotropy whereas the higher points represent structural isotropy only. Points at early stages of consolidation for elements near the base in RTC12, begun by gradual deposition, are shown and appear to lie close to the instant compression curve for one-dimensional compression. This suggests that creep is less significant in early stages of consolidation following gradual deposition, presumably due to the higher structural strength discussed earlier. Results shown for a restricted flow consolidation test carried out on soil from the same area by Lee (1979) are consistent with those from this study. The logarithmic void ratio axis used in figure 5.11 is seen to produce a linear compression curve which can then be described by

$$e = C\sigma_v'^{-n} \quad (5.5.1)$$

Over the range of void ratio (1 to 14) and effective stress (0.01 to 300 kN/m²) shown, appropriate values of C and n are 5.7 and 0.29 respectively. The points representing consolidation following gradual deposition suggest this curve is valid even at void ratios higher than the transition value, about 12, defining whether effective stresses will exist for a soil deposited rapidly. It will be useful in later studies to determine whether this curve might be able to represent the short term consolidation behaviour at still higher void ratios in very thin layers sedimented gradually from a suspension, such as those layers deposited in natural fluvial or marine environments. It may be noted finally that use of void ratio appears preferable where very large volume changes occur, due to the linearity of the fit obtained in figure 5.11. Significant curvature would result if logarithm of specific volume were used instead.

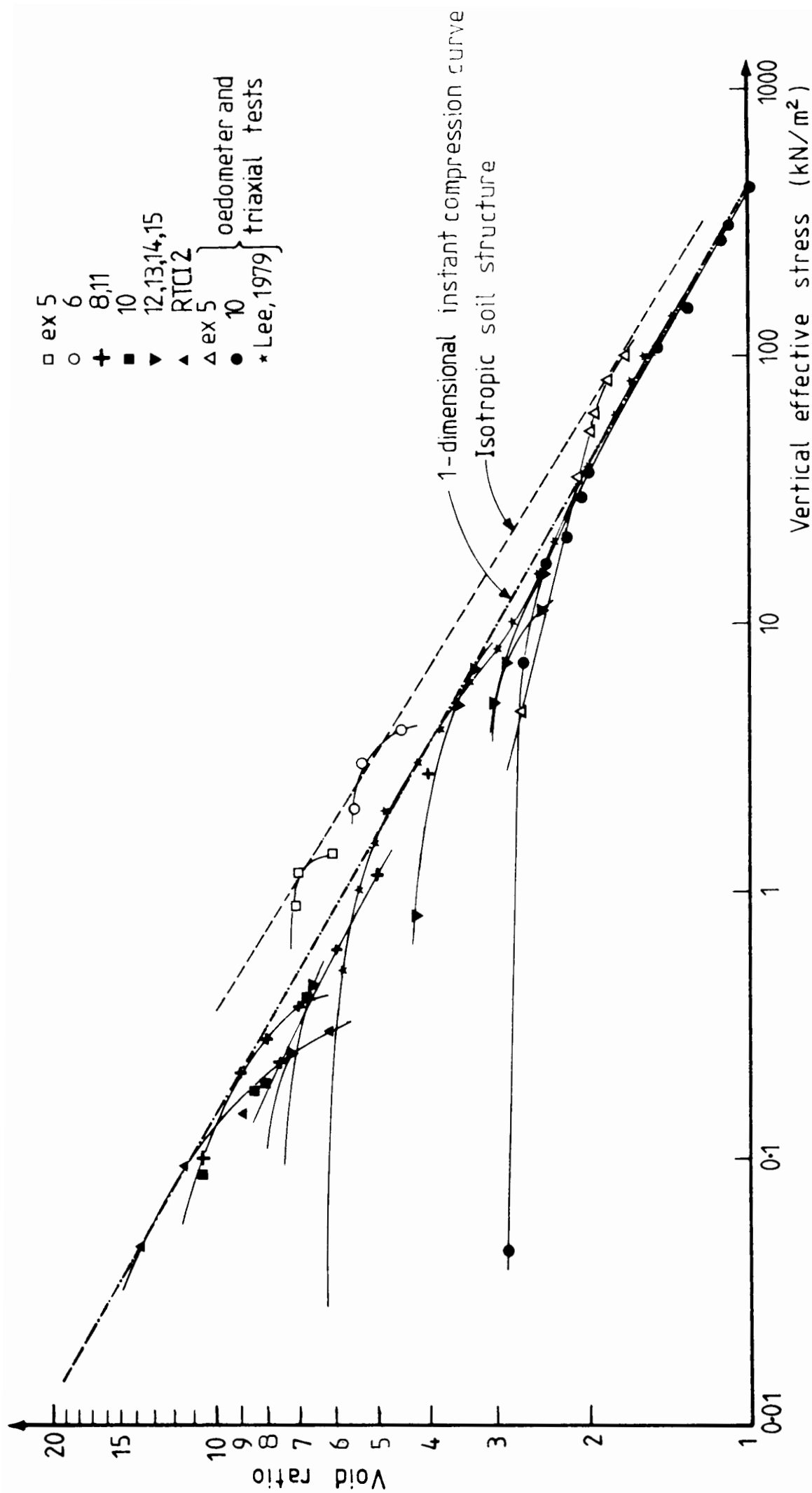


Figure 5.11 Instant compression lines for Combwich soil.

5.6 Pore water flow

In chapter 2 it was suggested that pore water flow relative to the soil structure might not follow Darcy's law at high void ratios. The general form of Darcy's law can be written

$$nv_r = -k \frac{\partial h_e}{\partial x} = -ki \quad (5.6.1)$$

where n is the soil porosity, v_r the average velocity of pore fluid relative to soil particles and i is the hydraulic gradient for the spatial (Eulerian) co-ordinate system x . Continuity of the fluids and solids phases, if both are assumed to move separately, requires that the net flow of solids and fluid across a fixed horizontal plane at height x in a given time is zero, so that

$$(1-n) v_s + nv_f = 0 \quad (5.6.2)$$

in a soil column where base drainage is not occurring, so that

$$nv_r = n(v_f - v_s) = -v_s \quad (5.6.3)$$

The solids velocity for any element in the soil may be calculated from density profiles, by calculating the distance moved by an element of given material co-ordinate in the time between successive profiles. Darcy's law may therefore be investigated by plotting the solids velocity, v_s , against the hydraulic gradient measured from excess pore water pressure profiles. This is done for experiments 10 and 11 in figure 5.12. Behaviour is highly non linear for all void ratios greater than 4. At higher void ratios (6 to 9) the relative velocity does not appear to be a function of the hydraulic gradient at all although examination of figures 5.2 and 5.3 shows that significant effective stresses have been developed. Hydraulic gradients however remain close to the critical values i_{cr} defined by the gradients of excess total stress, σ_e

$$i_{cr} = \frac{1}{\gamma_w} \frac{\partial \sigma_e}{\partial x} = -(\gamma - \gamma_w) \quad (5.6.4)$$

Using equation 2.1.1 this may be rewritten in terms of void ratio alone

$$i_{cr} = -\left(\frac{\gamma_s}{\gamma_w} - 1\right)/(1+e) = -1.65/(1+e) \quad (5.6.5)$$

where γ , γ_w and γ_s are the soil, water and solids unit weights respectively, and the constant is obtained for $\gamma_w/g = 1.003 \text{ Mg/m}^3$ and $\gamma_s/g = 2.66 \text{ Mg/m}^3$.

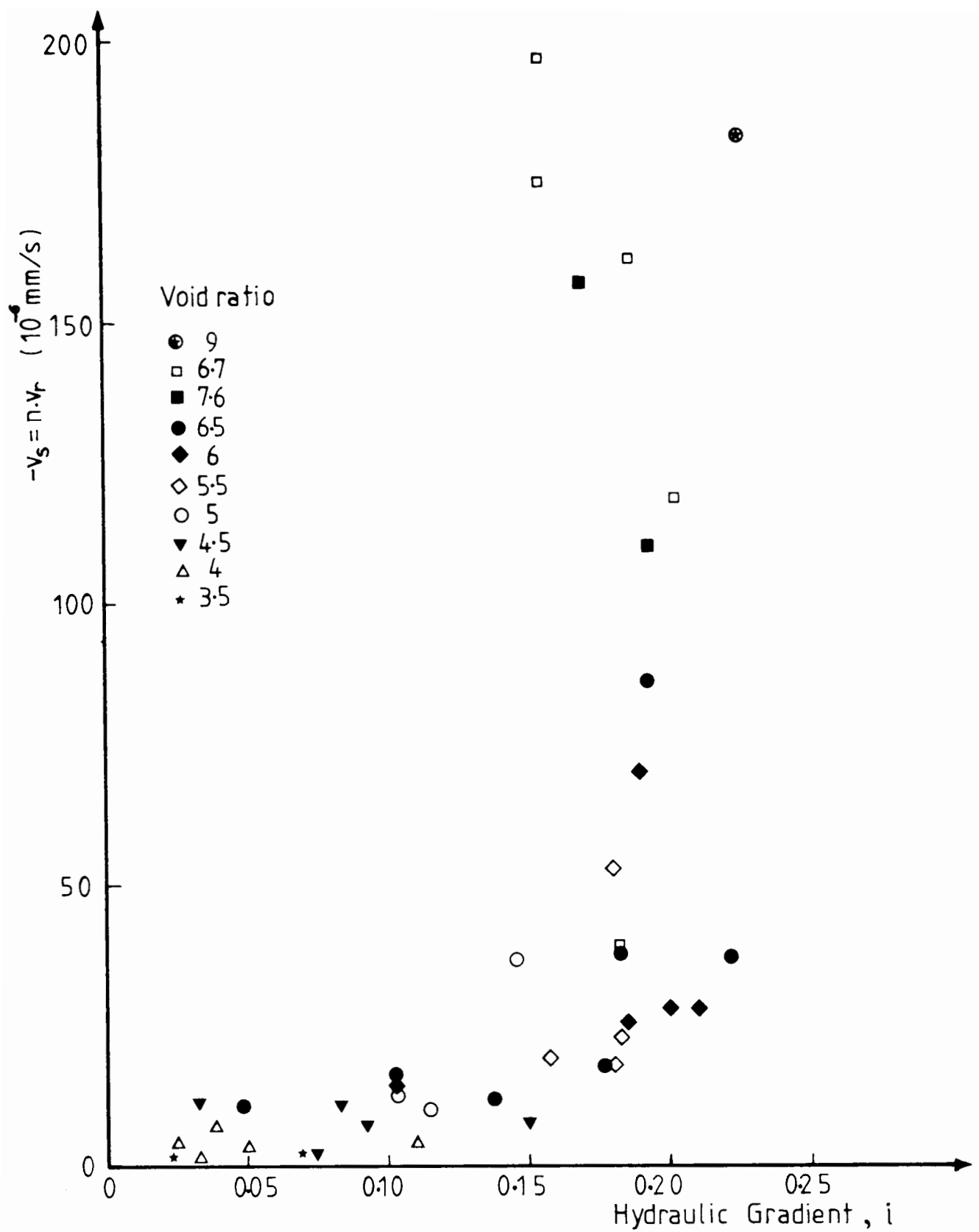


Figure 5.12 Pore water/soil relative flow for expts. 10 and 11.

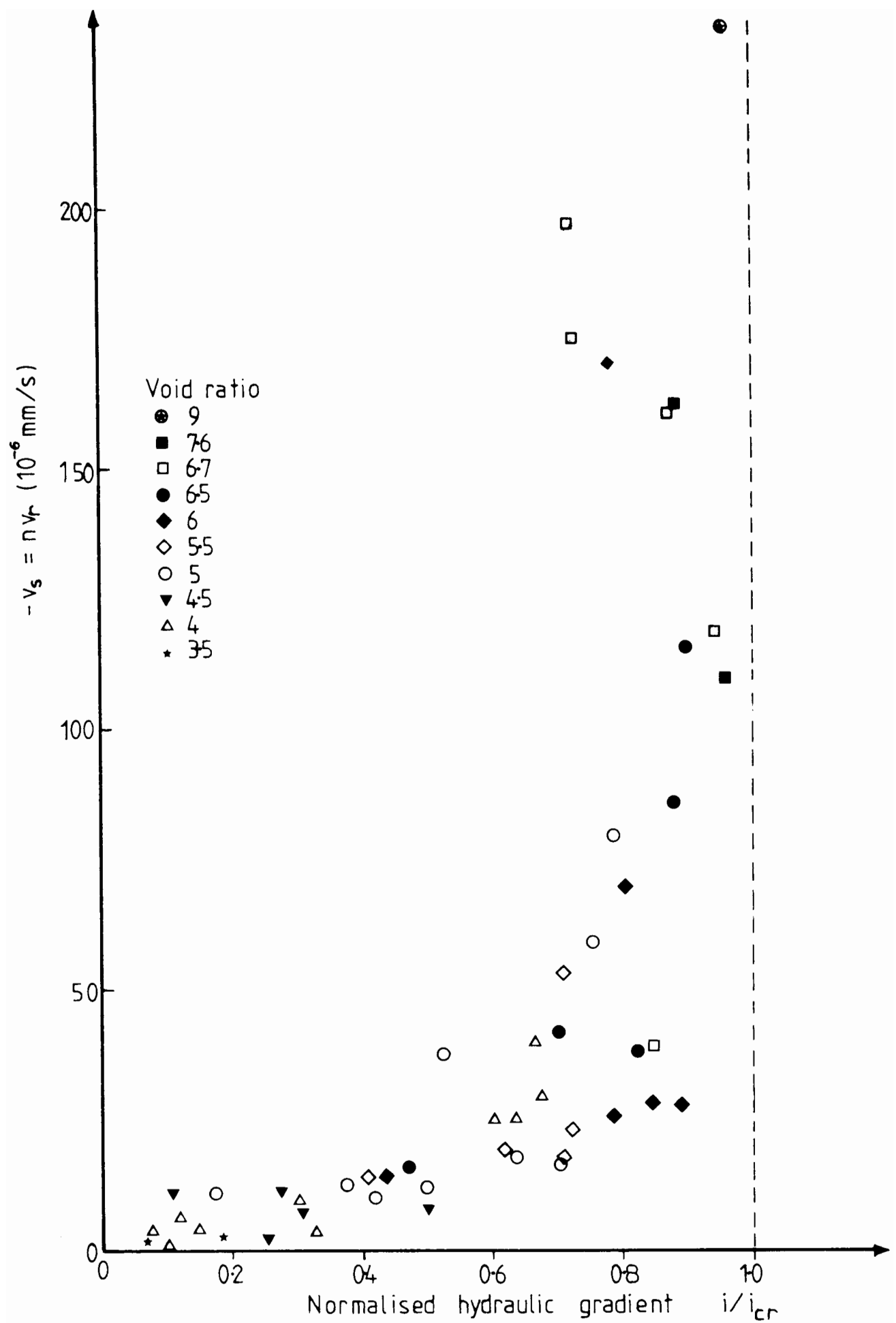


Figure 5.13 Flow relationship data for expts 10, 11, 5 and KB8.

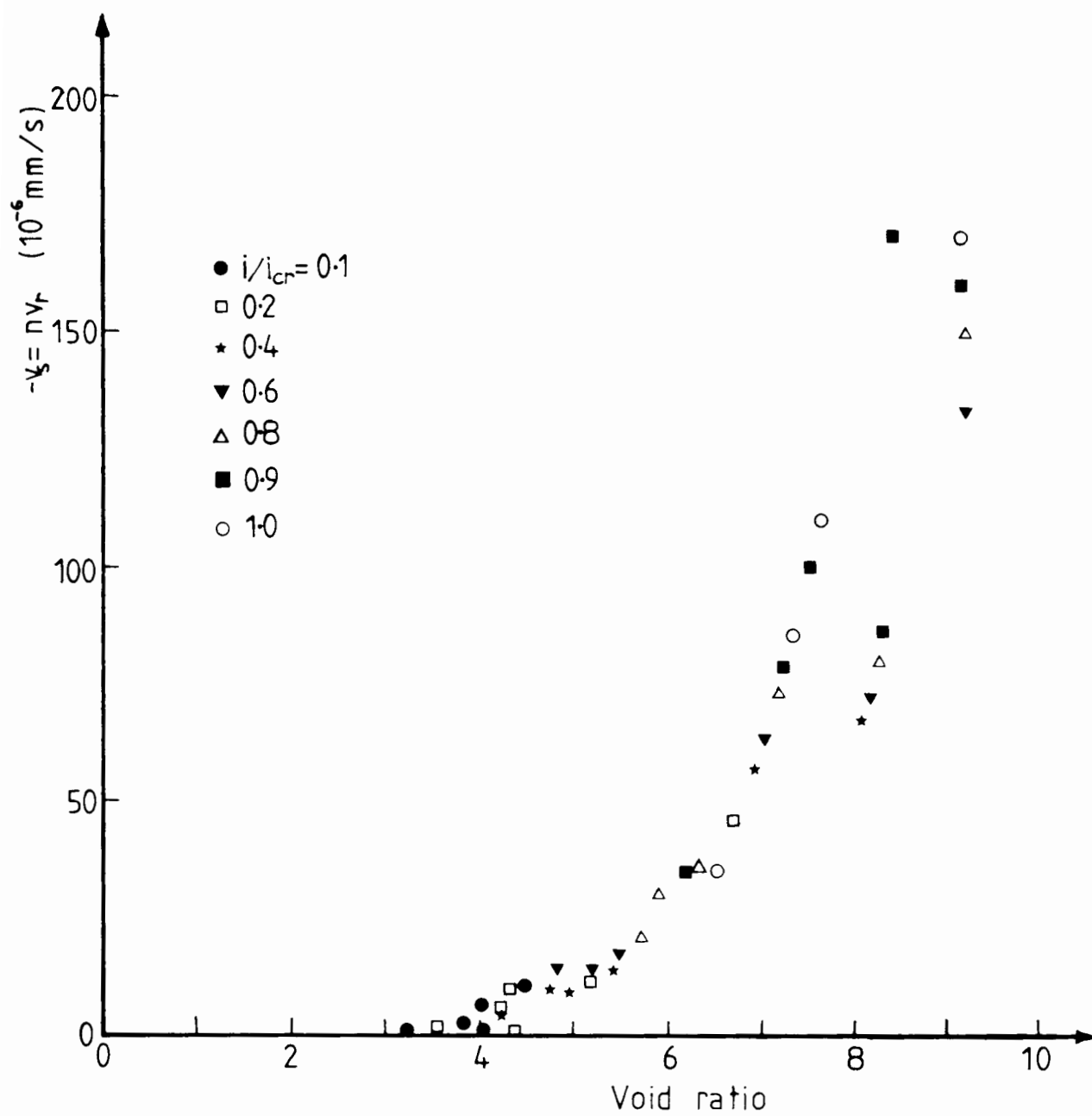


Figure 5.14 Relative flow velocity as a function of void ratio for expts. 10 and 11.

The data of figure 5.12 are replotted in figure 5.13 using a normalised hydraulic gradient defined as i/i_{cr} , which incorporates void ratio from equation 5.6.5. Data for experiment 5 and KB8 (from figure 5.7) are also included. Data for particular void ratios follow more consistent trends (increase in relative velocity with increase in normalised hydraulic gradient) than the data plotted in the form of figure 5.12. Darcy's law is clearly not satisfied however, even if the normalised hydraulic gradient were used to replace i , although data at each void ratio follow reasonably linear forms up to $i/i_{cr} = 0.7$. There appears however to be some value in plotting pore fluid flow data using the normalised hydraulic gradient for self weight consolidation where $i \approx i_{cr}$.

In hindered settling studies the relative velocity is often assumed to be a function of the concentration, and in figure 5.14 nv_r is plotted as a function of void ratio. Data are grouped more closely together than in either figure 5.12 or figure 5.13 and do not vary significantly with different hydraulic gradients. This suggests that although effective stresses are significant in this soil at these void ratios and influence compression behaviour, the pore water expulsion rate, and therefore the overall rate of compression, are dependent largely on the void ratio as would be expected in hindered settling. Conventional hindered settling theories are unable to allow for non zero effective stress, but correspondingly conventional consolidation theories all assume the validity of Darcy's law. Behaviour exhibited in these experiments appears to fall into neither category and major reformulation of compression theory will be required to enable accurate compression modelling based on this revealed behaviour.

5.7 Discussion

The effects of various initial and boundary conditions on the self weight consolidation of Combwich soil have been analysed. The consolidation process is considerably more complex than that assumed by most existing models. In

thick sediment layers, the most important feature revealed is that the volumetric state is not defined solely by the effective stress; large void ratio changes may occur at nearly constant effective stress if the rate of stress increase is sufficiently slow, in a creep-type process. The general behaviour might be explained by gradual collapse of the soil structure with stress increase. Rather than particle or floc spacing decreasing and causing interparticle forces to increase, the flocs break down or rearrange so that a smaller stress increase results, and much of the load continues to be carried by the pore fluid. An alternative although similar explanation which would also account for sustained high measured pore pressures during periods of large compression is the macropore/micropore consolidation postulated by Olsen (1962) and developed by de Josselin de Jong (1968), and discussed in chapter 2. In very high void ratio sediments an extension of this idea suggests that 'micropores' within flocs could allow the flocs themselves to consolidate under stress increases. The different interparticle forces acting at the clay particle level would cause this process to differ considerably from that involved in macroconsolidation, but the effect would be that drainage from the flocs into the macropores (between flocs) would mean pore pressures in the macropores would not reflect the overall compression behaviour. Since only macropore water pressures can be detected by the transducers at the column walls the observed effect would be volume change at near constant effective stress as seen in figures 5.2, 5.3 and 5.7. As micropore volumes decrease the measured effective stress change would appear to play an increasingly important role. Processes of this kind may help to explain the differences between behaviour observed for equivalent elements in sediments deposited in fresh and salt water at similar initial void ratios (figure 5.5). Initial compression in the salt water is more rapid and occurs at lower stresses and might correspond to a less stable microstructure. Eventually macrostructure consolidation begins to dominate and consolidation curves for the two types of soil merge.

The depositional condition most important in determining subsequent behaviour is the rate of input. Where initial void ratios in soil deposited rapidly are above the critical transition value (about 12 for Combwich mud, defined by figure 5.1) a suspension phase exists and the structured phase forms from the base of the column by gradual deposition of the settling sediment, at a rate depending on the initial concentration. Some segregation is expected during this type of deposition and a more natural process is probably the gradual, uniform deposition of suspended sediment occurring in experiment RTCI2, reported earlier by Sills & Thomas (1983) and Elder & Sills (1984) and analysed in this chapter. The slow deposition rate allows the developing structured layer to be stable in a less dense state than is possible following rapid deposition. It might be speculated that the main difference between the extremes of rapid and slow deposition is the length of time that sediment 'flocs' rest on the surface of the consolidating soil before being loaded by further sediment. If this time is sufficiently long (or rate of subsequent loading is sufficiently slow) some strength develops and the open structure can be maintained. The different behaviour observed following gradual deposition therefore provides some indirect evidence for time dependent strength increase or cementation in the soil.

Soil at zero effective stress (at the soil surface) undergoes void ratio changes with time (figure 5.1) and these changes are of comparable magnitude to the void ratio changes lower in the soil where effective stresses are present. The implication that time dependent compression is relatively independent of effective stress level means that the macrostructure/microstructure compression model, or any consolidation model which permits secondary compression only at non-zero effective stress, is unable to describe behaviour completely. The good correlation between the rate of pore fluid flow and void ratio independent of hydraulic gradient (figure 5.14) is consistent with the independence of compression behaviour from effective stress level. Data suggest that at void ratios as low as 4 compression might best be modelled using hindered settling

theory similar to that developed by Kynch (1952), using only the requirements of volume continuity and some relationship between settling velocity and void ratio (or concentration). As shown by Been (1980), the large strain formulation of Gibson, England and Hussey (see chapter 2, equation 2.4.13) reduces to Kynch's formulation when the effective stress is zero. Results here suggest that this formulation is not valid for non-zero effective stresses at very high void ratios since Darcy's law does not apply. Modifications of either theoretical approach, such as those proposed by Tory & Shannon (1965) for settling in compression or by Pane & Schiffman (1985) for effective stresses "not fully developed" are empirical and fundamental theory modification is required. This might proceed along the lines suggested by Kos (1985) and discussed at the end of chapter 2, but may also require consideration of microstructural forces of attraction and repulsion at the interparticle level.

The demonstration of a unique instant compression curve for one-dimensional consolidation (figure 5.11) which is rarely attained in soil layers consolidating under self weight is significant in determining the variable compression behaviour. Soil behaves initially as though overconsolidated following deposition and if the rate of effective stress increase is slow, creep compression dominates subsequent volume changes so that the instant compression curve is never reached. Subsequent soil states will never show characteristics of normal consolidation under further loading. Observation of such behaviour in these experiments casts doubts on the relevance of many consolidation tests to field situations, where rates of effective stress increase or pore pressure dissipation may be extremely slow. Even tests in settling columns may not be useful unless densities and pore pressures can be measured regularly at different depths in the soil, so that consolidation paths can be observed for different elements.

6.1 Introduction

6.2 Remoulded and peak strength variations

6.3 Undrained shear modulus, G and ratio G/s_u from vane tests

6.4 Viscoelastic soil behaviour during shear

6.5 Spherical cavity expansion results

6.6 Strength dependence on effective stress. Normalised soil behaviour

6.7 Triaxial test results

6.8 Discussion of strength results

6.1 Introduction

Results of strength tests carried out in settling columns were presented in chapter 4. In this chapter relationships with other soil properties are established and the vane, cavity expansion and fall cone tests are evaluated and compared. The importance of specific test factors such as rate and time to failure are discussed. Aging is shown to cause considerable increases in soil sensitivity. Variations in shear moduli indicate viscoelastic soil behaviour during shearing. Normalised strength behaviour is evaluated in terms of measured effective stresses and the equivalent effective stresses defined at the same void ratio by the instant compression curve proposed in chapter 5. Trends are typical of those for overconsolidated soils although soils in the columns have never experienced stress decreases. Undrained strengths obtained from triaxial tests at much higher stresses are compared with those obtained in columns.

6.2 Remoulded and peak strength variations

In chapter 2 it was shown that remoulded strengths for many different soils are almost equal at the same liquidity index. Variations in residual vane strength with liquidity index for short column experiments 100, 200 and 300 are shown in figure 6.1. In columns tested at times up to 100 hours after deposition, the residual strength measured increases consistently with rotation rate at all liquidity indices. The static resistances, calculated immediately after cessation of testing, are variable due to the difficulty of estimating an instant value before relaxation occurs, but they generally plot just below the curve for tests at 4°/min. Residual strengths after longer consolidation times are rather higher. This suggests that the rotation remoulding was insufficient at the total rotation angles achieved ($\approx 210^\circ$) to reattain the fully remoulded state. Increased structural anisotropy following longer consolidation periods

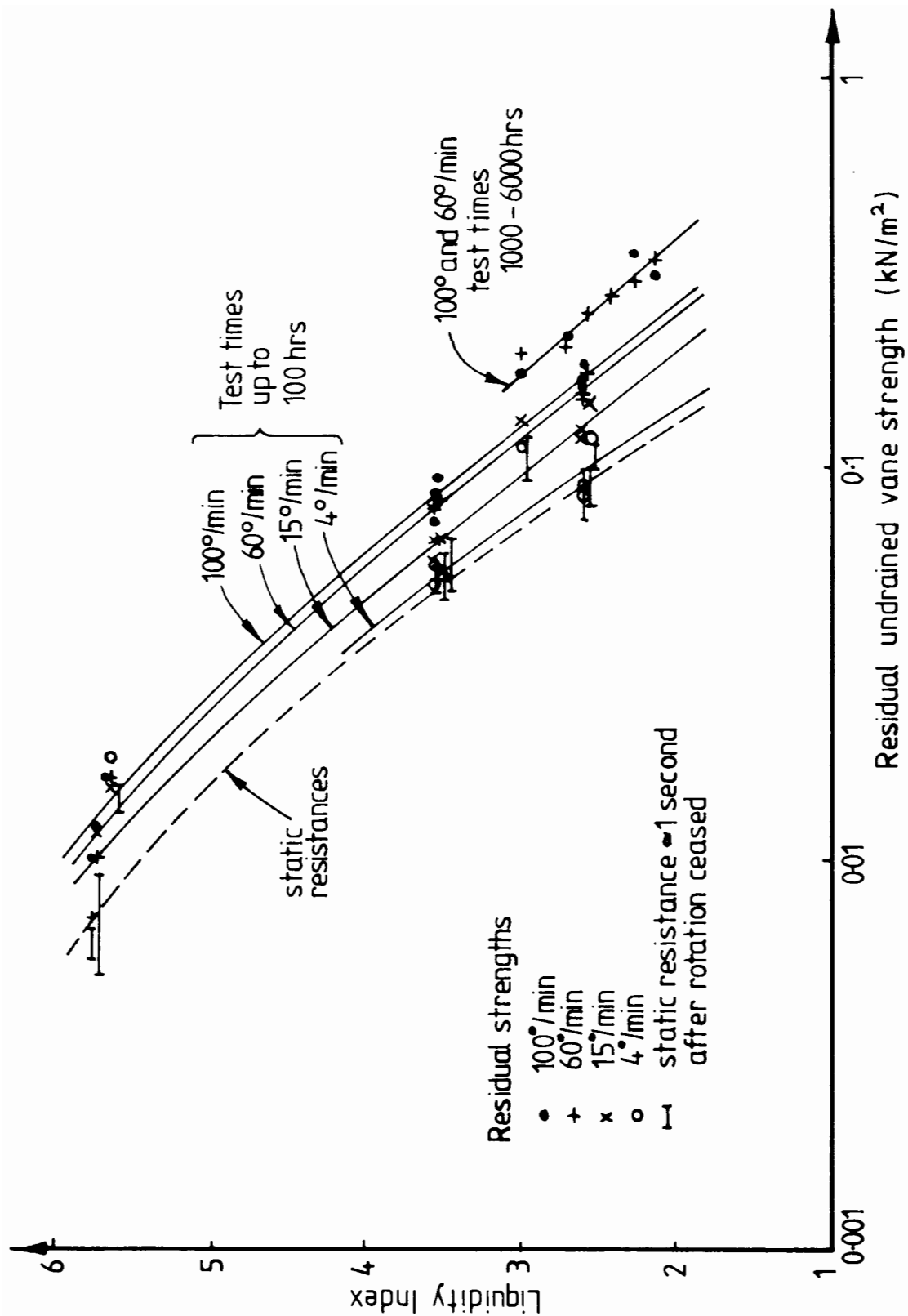


Figure 6.1 Residual undrained vane strength as function of liquidity index, showing effects of rotation rate and aging. Expts 100, 200, 300 series.

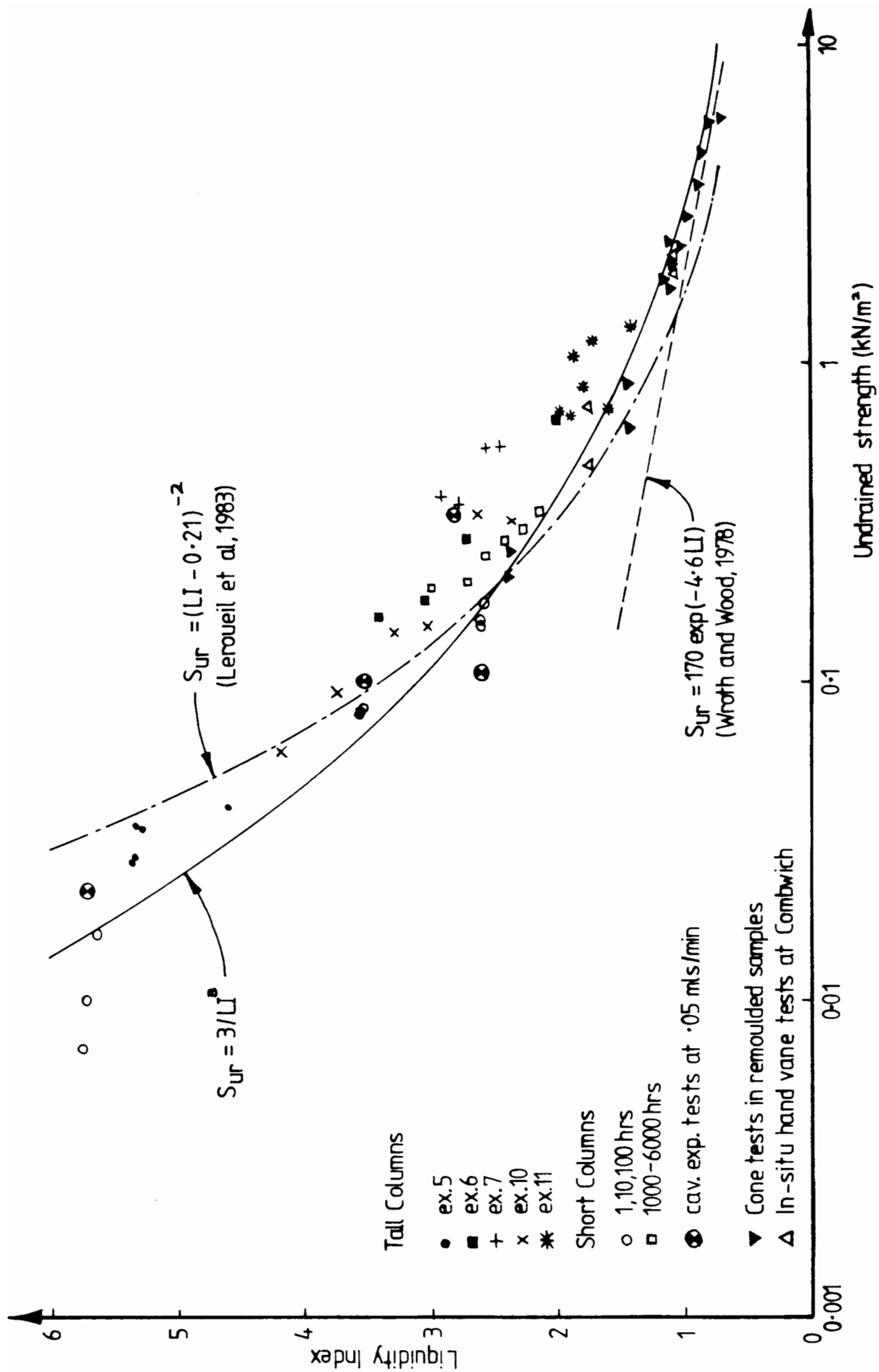


Figure 6.2 Residual and remoulded shear strength for all tests. Vane tests at 60°/min, fall cone tests in remoulded samples, cavity expansion tests at 0.005 mls/min.

may have contributed to this. Figure 6.2 presents residual/remoulded strength data for all experiments. Vane tests in tall columns were at 60°/min and only tests at this rate in short columns are represented. Tests in columns contribute data at liquidity indices down to 1.4, for soil deposited at liquidity indices initially much higher (2.6 to 11). A series of fall cone tests was carried out in smaller remoulded samples at liquidity indices from 0.7 to 2.4, and four hand vane tests were carried out in situ in Combwich mud at liquidity indices 1.06 and 1.74, so that overlap occurred between each group of tests. The in situ test results are consistent with those in the reconstituted laboratory soil. Fall cone data are also consistent with minimum residual vane strength data and confirm that the cone factors selected in chapter 4 (figure 4.7), on the basis of vane strengths up to 0.35 kN/m² only, are appropriate in a wider range. Cavity expansion tests at 0.05 ml/minute yield strengths close to those from cone and vane tests; reasons for choice of this particular expansion rate are discussed in a later section. In chapter two, a relationship was proposed which fitted a wide range of remoulded strength data

$$s_{ur} = s_{uLL}/LI^3$$

This provides a reasonable fit to data in figure 6.2 using $s_{uLL} = 3 \text{ kN/m}^2$ at the liquid limit. A value closer to 2 kN/m² might be more appropriate but would underpredict strengths at higher liquidity indices. The relationship proposed by Leroueil et al (1983) overpredicts strengths at high liquidity indices while that proposed by Wroth & Wood (1978) is appropriate below the liquid limit only, the range for which it was derived.

Peak strength data from vane and fall cone tests are shown in figure 6.3, again using only vane tests at 60°/min. The curve for remoulded data from figure 6.2 is included as a reference. A very clear trend can be seen for peak soil strengths to increase with time according to the approximate isochrones fitted for different times following deposition. It should be noted however that these are only approximations since the compression and aging processes

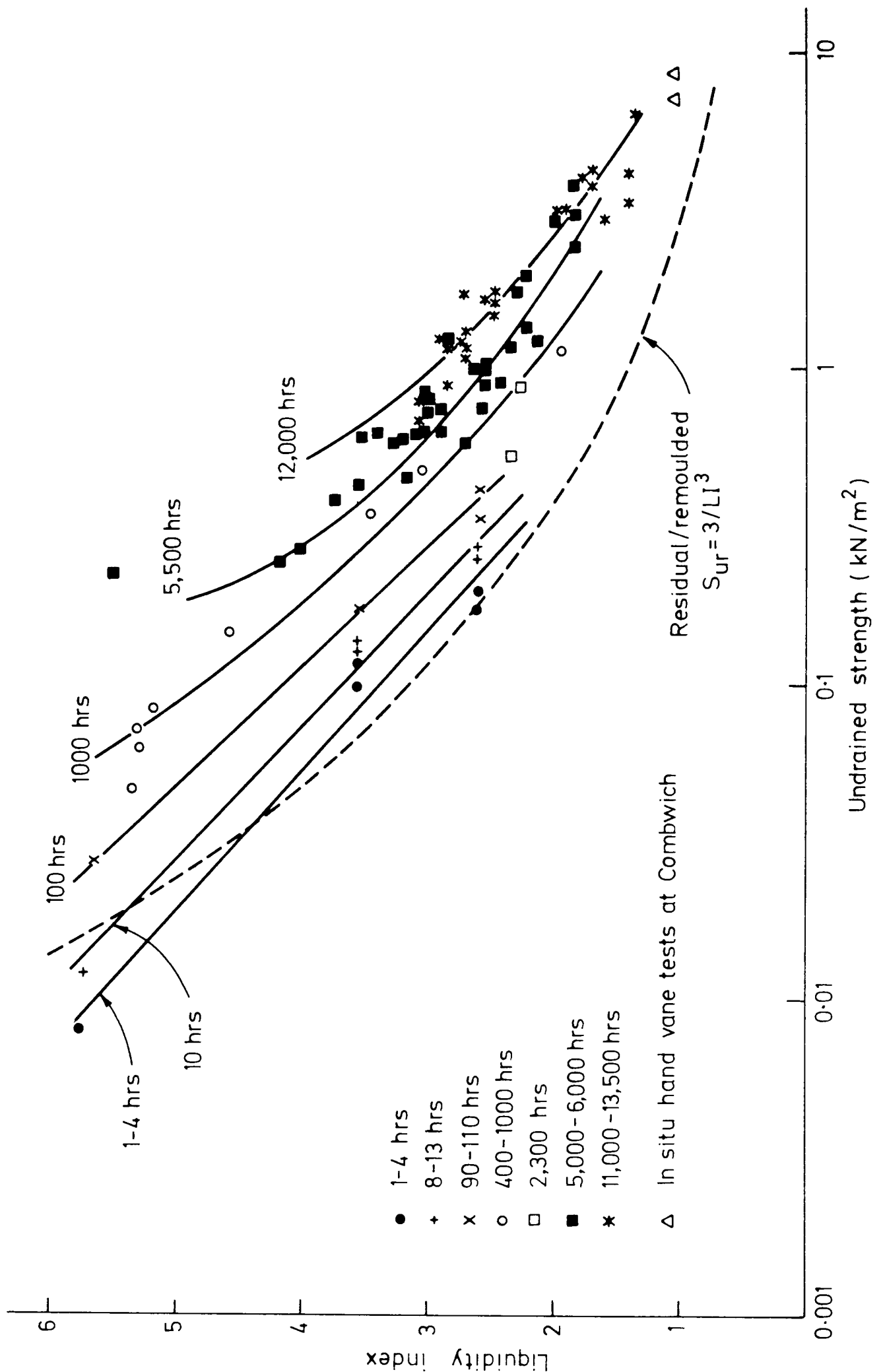


Figure 6.3 Peak shear strength increase with aging as a function of liquidity index. Data from vane tests at 60°/min and fall cone tests (averages of up to 20 cone tests represented by each data point).

occur simultaneously and the actual time the soil has existed near the liquidity index stated is less than the total time since deposition. A point of some importance is that peak strengths after very short consolidation periods lie close to the remoulded curve (or to residual/remoulded strengths in figure 6.2). The two may be assumed equal initially - in other words the sedimented and remoulded strengths are equal immediately following deposition.

Some of the trends observed in figures 6.1 to 6.3 are analysed separately in figures 6.4 and 6.5. Strengths measured in experiments 100 to 300 are normalised for an assumed value of unity in each case at 60°/minute rotation rate. For all tests, regardless of the duration of the consolidation period or actual strength, the peak vane strength decreases with increasing time to failure (figure 6.4). Normalised strengths are compared for different rotation rates in figures 6.5a,b. Considerable scatter is evident at each rate, particularly at 100°/minute, but mean strengths are approximated closely by the fits shown (linear for these axes). Rotation rate has a very similar effect on residual or peak normalised strengths, independent of the actual value of the strength or of the consolidation time. For the range of rotation rates shown, a tenfold increase in rotation rate causes an 11% increase in peak strength and a 15% increase in residual strength. Conversely a tenfold increase in time to failure causes a 10% decrease in peak strength. These effects are somewhat less than those reported in chapter 3 from work elsewhere and might be magnified in stiffer soils.

Figure 6.6 depicts the time dependent increase in soil sensitivity, defined as the peak divided by the residual strength for vane tests. Despite the wide range of liquidity indices represented by data in figure 6.6 (see figure 6.3) a reasonable approximation may be obtained to all points by the curve shown. Values for experiments 100, 200 and 300 are the average of sensitivities for tests at 4°, 15°, 60° and 100°/min, among which no separate trends existed. Thixotropic regains are also included for periods 1, 10 and 100 minutes. These were evaluated during vane testing by holding the vane

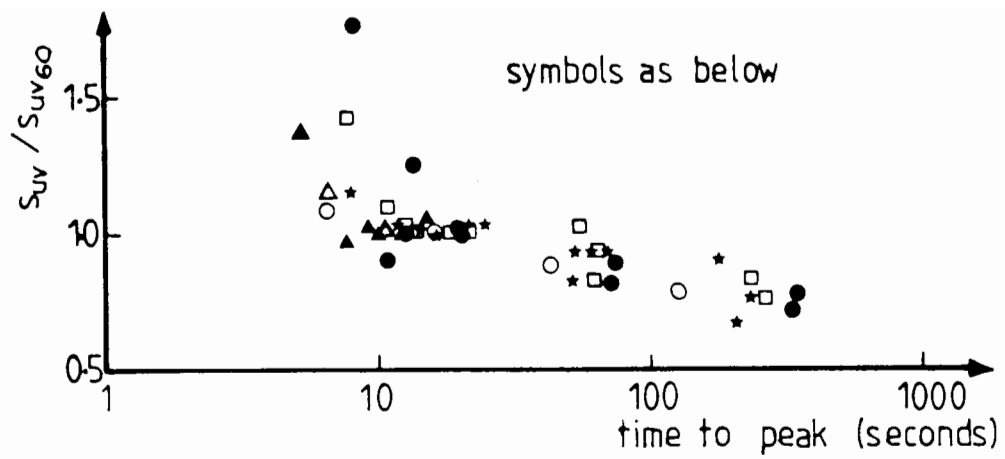
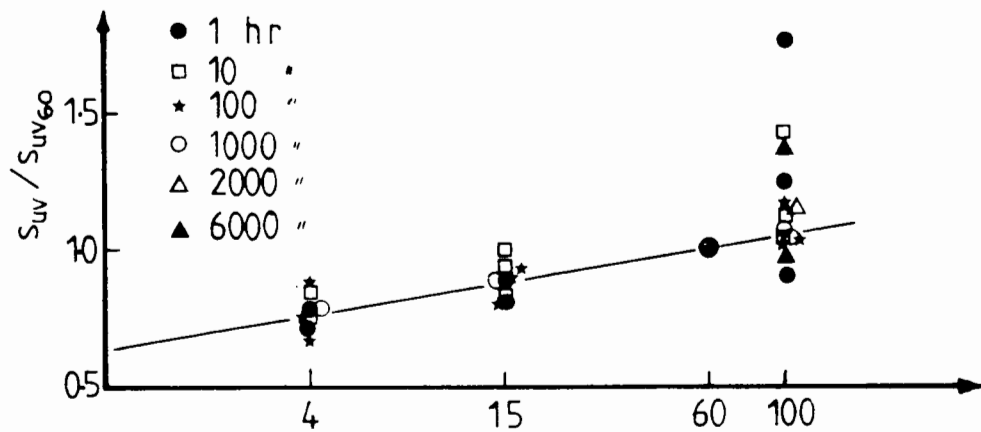
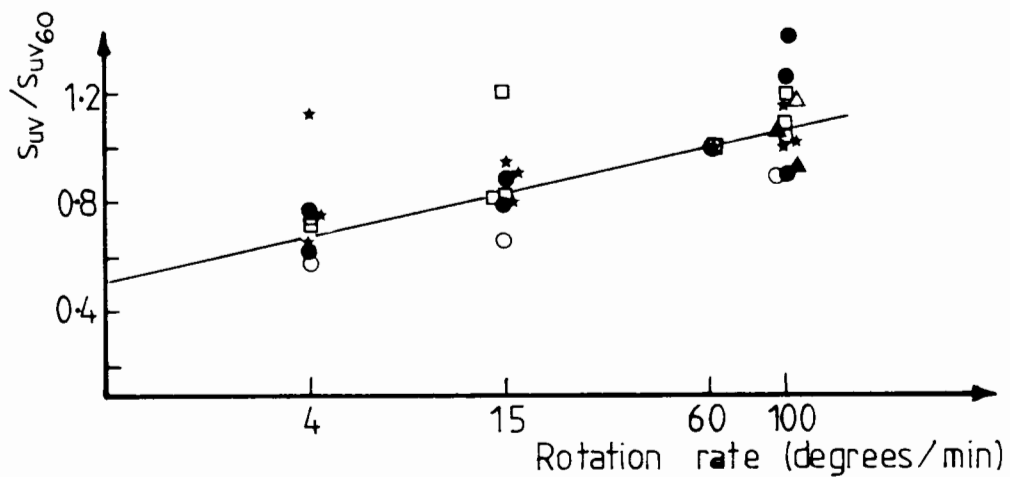


Figure 6.4 Variation in vane peak undrained strength with time to failure.



(a) Peak strengths.



(b) Residual strengths

Figure 6.5 Effect of rotation rate on undrained strength. Strengths normalised to fraction of strength at 60°/min. Expts 100, 200, 300.

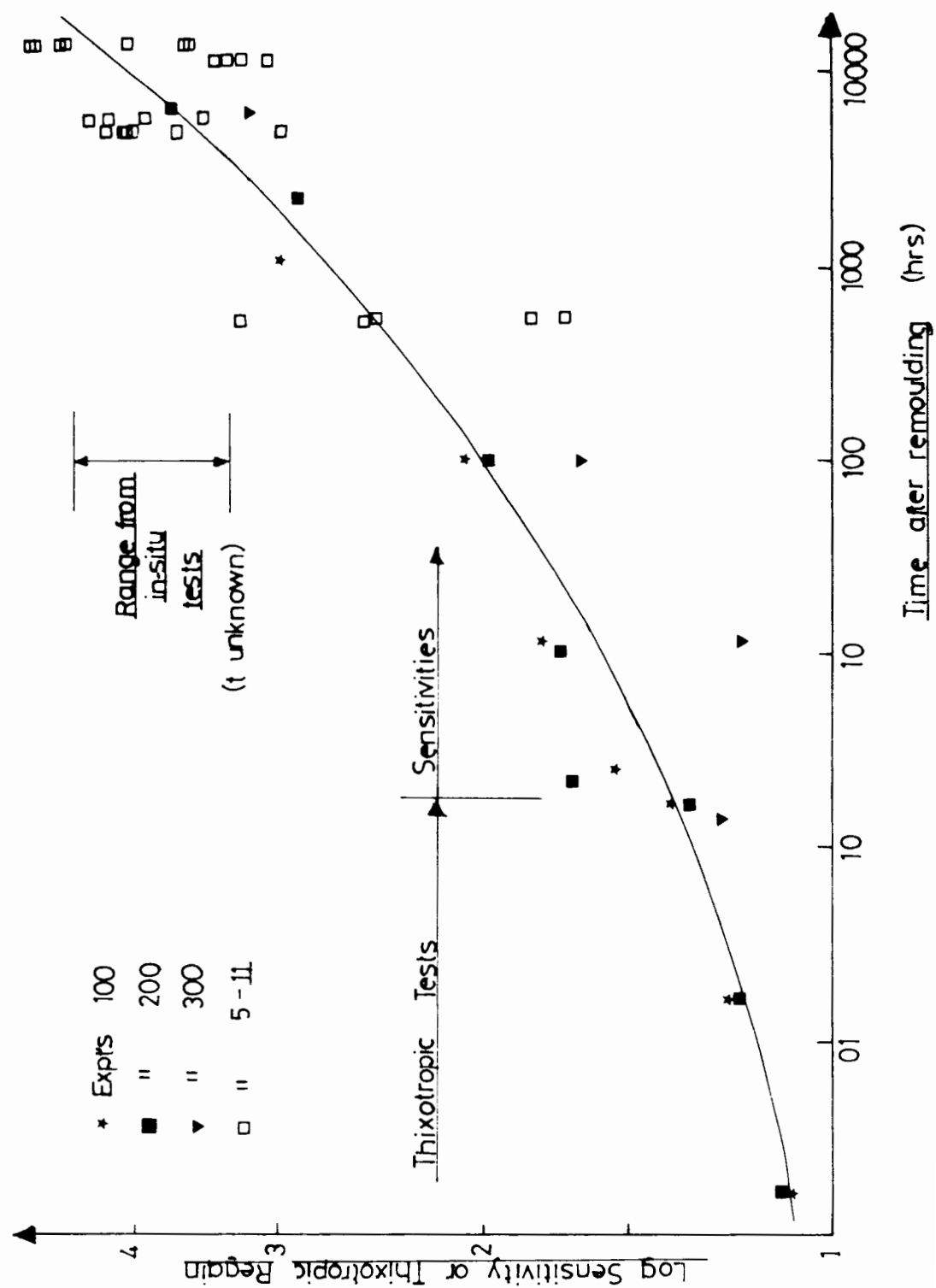


Figure 6.6 Increase in sensitivity with time.

stationary for each time period respectively once the residual strength condition had been attained. The regain is defined as the peak strength during subsequent rotation divided by the residual strength. As might be expected the trend exhibited by sensitivities is consistent with that for thixotropic regain. In situ sensitivities in Combwich mud ranged from 3.3 to 4.5, suggesting that mud had lain essentially undisturbed for about one year. The independence of data from liquidity index supports the suggestion made above that sedimented and remoulded strengths are initially equal.

Definite evidence of strength anisotropy could not be gathered from the testing methods used in this study. In none of figures 6.1 to 6.6 however were there significant unexplained trends, although the scatter observed in most figures may be attributable in part to different degrees of anisotropy.

6.3 Undrained shear modulus, G and ratio G/s_u from vane tests

Evaluation of the undrained shear modulus, G , from strength test results is subject to simplifying approximations discussed in chapter 3 and departure from the idealised behaviour may result in considerable quantitative error. Analyses presented in chapter 3 allow the shear modulus to be calculated from vane test results and the ratio E/s_u and s_u , and hence E and G to be calculated from spherical cavity expansion results. Due to the small initial cavity volumes, limiting pressures were attained in cavity tests very rapidly. Pre-peak data were therefore suspect and cavity expansion test data could not be used to derive soil moduli. The quality of vane test data however was in almost all cases very good, and these data are used in this section.

Secant moduli were calculated from values on the torque-rotation curves presented in chapter 4, according to equation 3.6.5

$$G = \frac{1}{6.3D^3} \frac{T}{\theta} \quad (H/D = 2) \quad (3.6.5)$$

To investigate the effect of calculating the secant modulus at different stages up to peak resistance, figure 6.7 compares normalised values for vane tests at 60°/min in experiment series 2. This experiment series contained the most

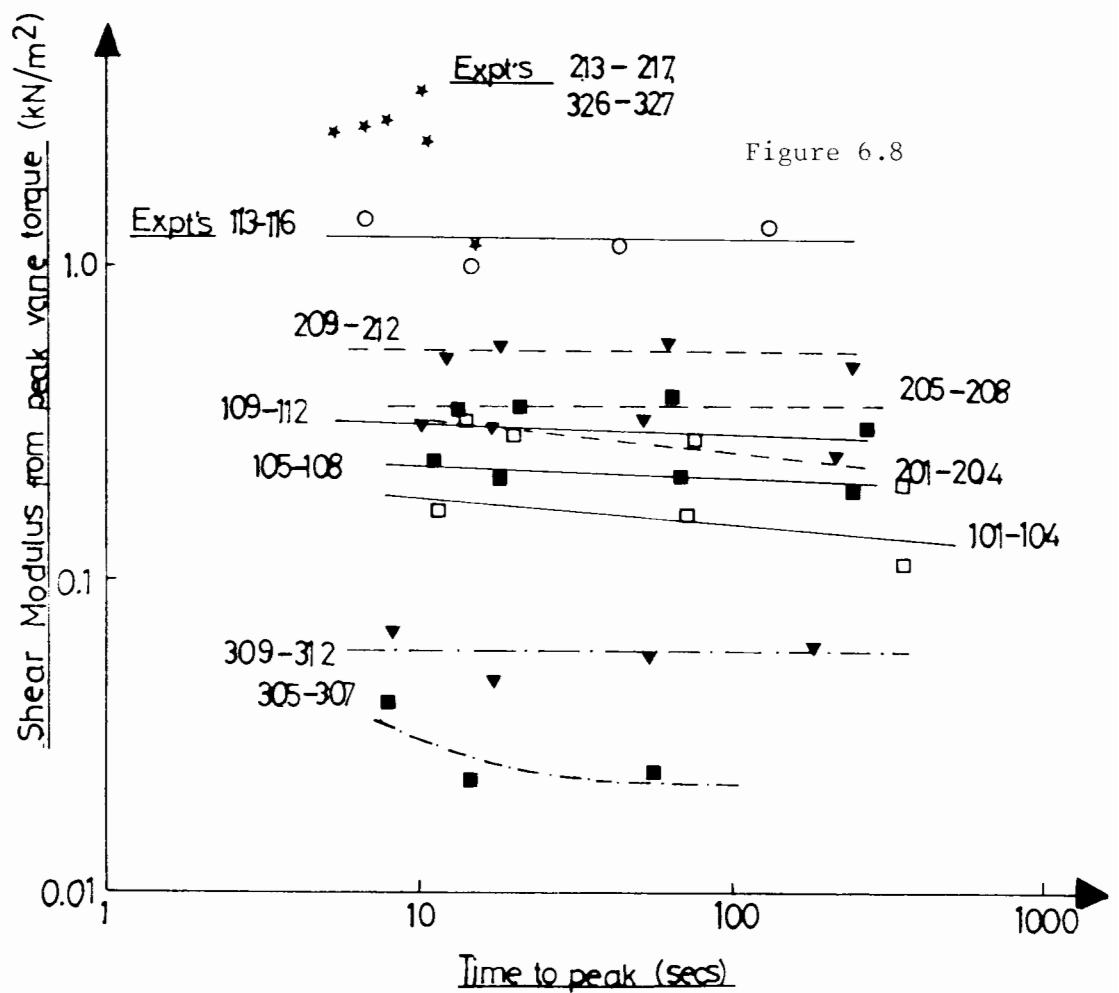
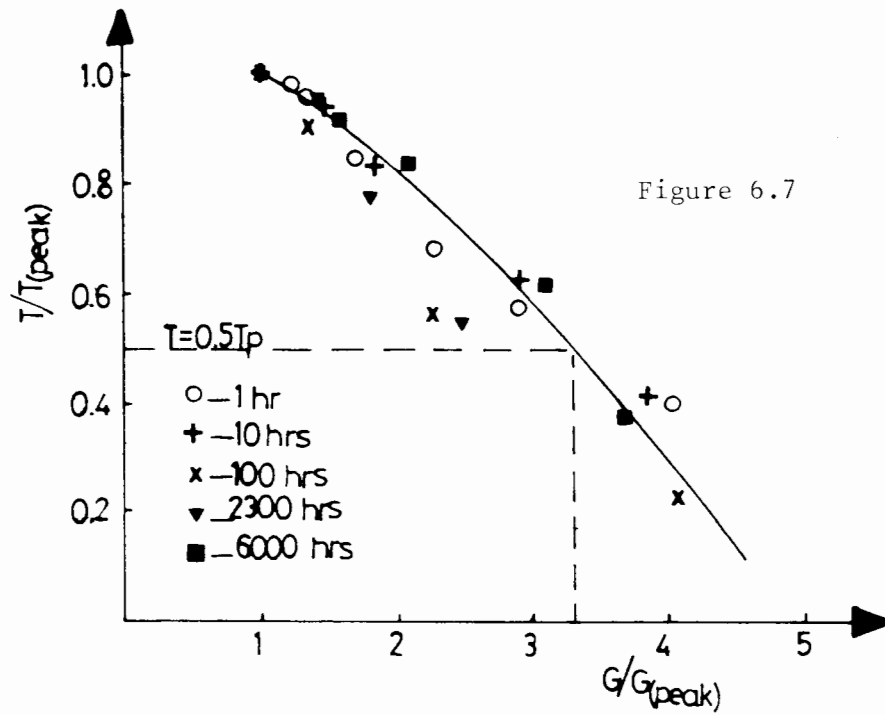


Figure 6.7 Variation in secant vane shear modulus, G , up to peak torque, T_p for vane tests at $60^\circ/\text{min}$, expts 200.

Figure 6.8 Effect of time to peak on shear modulus at peak resistance for vane tests in expts 100, 200, 300.

comprehensive series of tests at different times following deposition and up to 6000 hours. The shear modulus-torque relationship is nearly the same for all tests if both values are normalised by their values at peak resistance. The secant modulus G_{50} at 50% of maximum resistance is slightly more than three times the modulus at peak resistance and the initial tangent modulus (extrapolated at $T/T(\text{peak}) = 0$) about five times that at peak although this is more difficult to estimate. The secant modulus at peak resistance was used for comparisons in this section for several reasons. Previous reports (see chapter 3) suggested that considerable rotations are often required to develop a cylindrical failure surface; rotations at peak in this study were not high (6° – 20°) so that pre-peak strain distributions might not satisfy the assumptions used to derive equation 3.6.5. The peak on the torque rotation curve was sharply defined in all tests and significant flattening of the torque-rotation did not occur prior to peak. The stiff pre-peak behaviour observed in torque-rotation curves meant that torque values at particular rotations were difficult to measure accurately. Figure 6.8 shows the negligible effect of time to peak on the shear modulus calculated at peak resistance for tests at different rotation rates in short columns. Lines connect tests in identical samples. The same data, as well as those for tests in tall columns, are plotted in figure 6.9 which shows variation in shear modulus with liquidity index. Shear moduli at peak resistance in short columns closely follow the trends seen in figure 6.3 for the peak undrained shear strengths, increasing with decreasing liquidity index and with increasing consolidation time. Markedly different behaviour is shown by data from tall column tests where all points can be approximated by a single curve despite consolidation times varying from 560 to 13,500 hours. This appears inexplicable initially but some explanation may be provided by the patterns of consolidation behaviour discussed in chapter 5. Liquidity indices at deposition were higher in most tall column experiments so that eventual compression to a given liquidity index level was greater and "stress history" effects diminished. In addition to this effect, compression

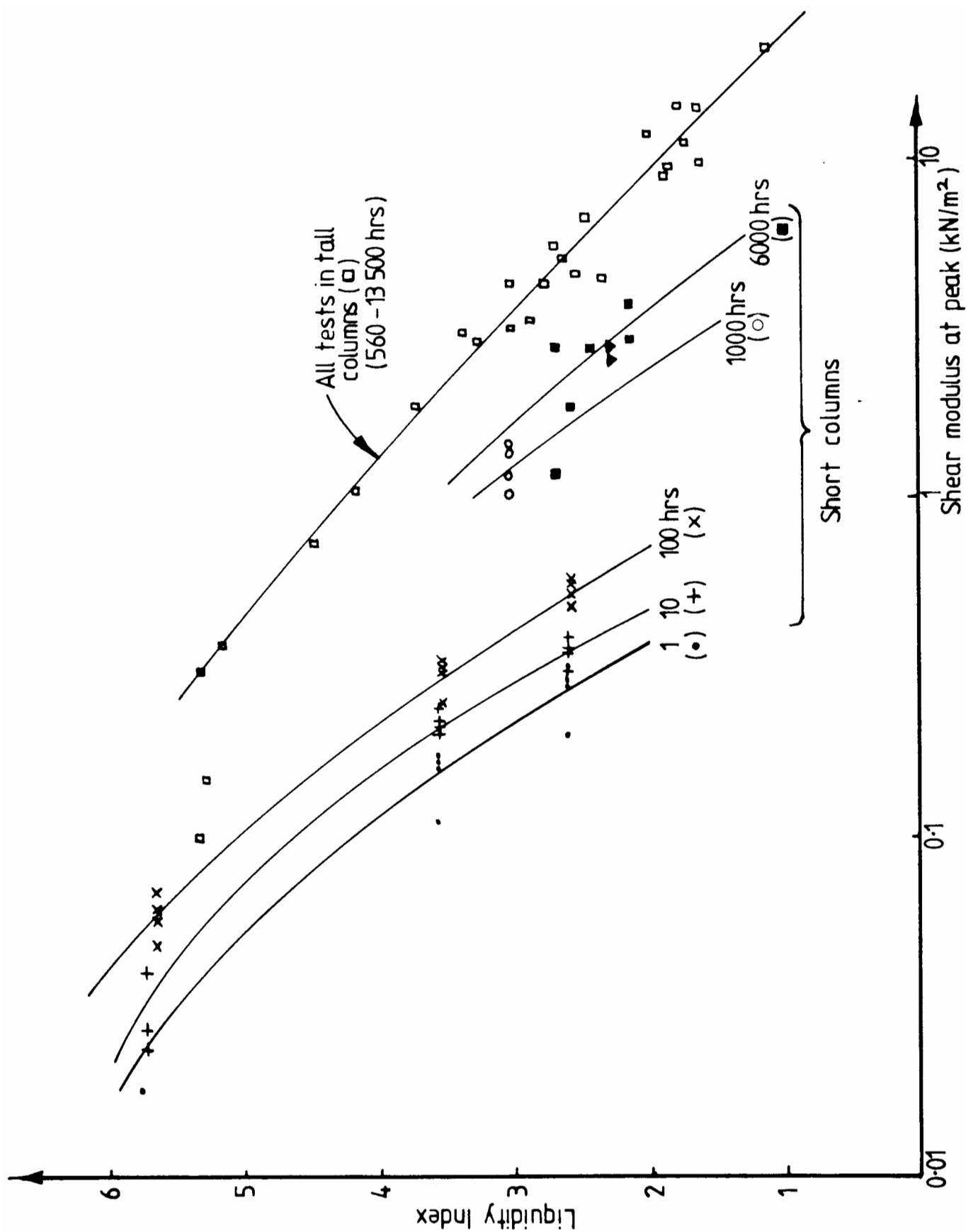


Figure 6.9 Variation in shear modulus at peak torque with liquidity index.

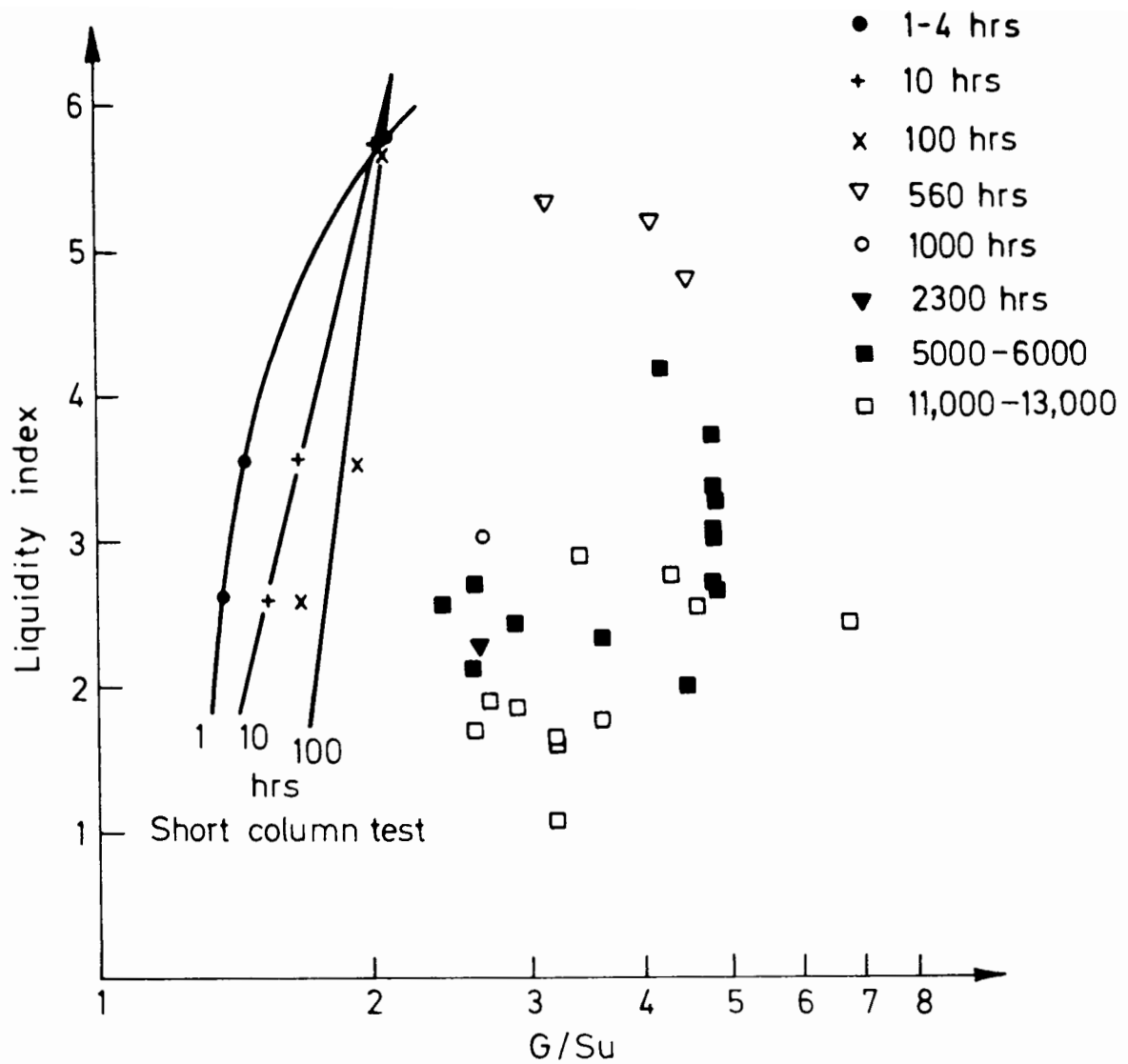


Figure 6.10

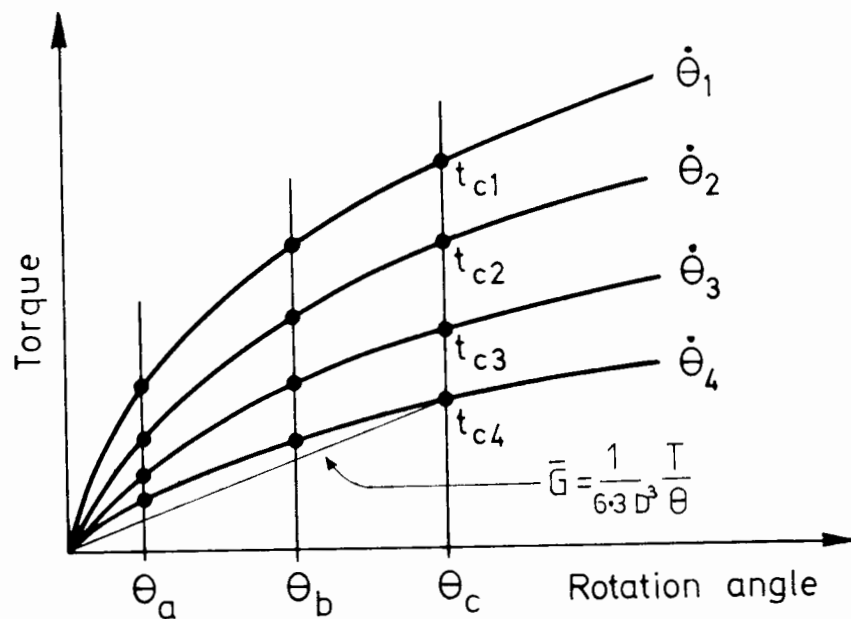


Figure 6.11

Figure 6.10 Variation in ratio of shear modulus: undrained strength at peak vane resistance for all tests.

Figure 6.11 Schematic construction for calculation of $(\bar{G}, t)$ at particular angle θ from tests at different rates in identical samples.

in tall columns occurred more slowly than in short columns of similar initial density. At comparable liquidity indices and times following deposition, effective stresses were generally higher in the tall columns (shown by figure 4.25 in chapter 4). The existence of the instant compression line proposed in chapter 5 suggested that most soil states behaved as though overconsolidated; the difference between data for tall and short columns seen in figure 6.9, then, might be due to differences in equivalent overconsolidation ratios. Further evaluation of this effect is left to section 6.6.

Variation in the ratio of shear modulus (at peak resistance) to undrained strength, calculated using equation 3.6.7a

$$G/s_u = 1/2\theta \quad (\theta \text{ in radians}) \quad (3.6.7a)$$

is shown in figure 6.10. Values do not follow consistent trends with liquidity index and after very short consolidation times (< 100 hours) G/s_u decreases with decreasing liquidity index. This is contrary to expectations for normally consolidated soils and supports the existence of equivalent overconsolidation.

6.4 Viscoelastic soil behaviour during shear

In the previous section only the shear modulus at peak resistance was considered. A method using shear vane results to determine parameters which might define viscoelastic soil shear behaviour was presented by Stevenson (1973) and discussed in chapter 3. The shear modulus $G(t)$ at a particular time t during rotation prior to peak resistance is assumed to be represented by

$$G(t) = G_1 t^{-n} \quad (3.6.9)$$

The construction used to calculate $\bar{G}$ - t data is shown in figure 6.11. In figures 6.12 to 6.14 secant moduli $\bar{G}$ are plotted against time required to achieve particular rotation angles. Lines at constant angles of rotation provide good fits (on double logarithmic plots) to tests in identical soils at different rates of vane rotation. Existence of these linear fits supports the use of the power law form. Values of n and G_1 are obtained for each line as the slope and time averaged modulus at $t = 1$ second respectively. (Values of n

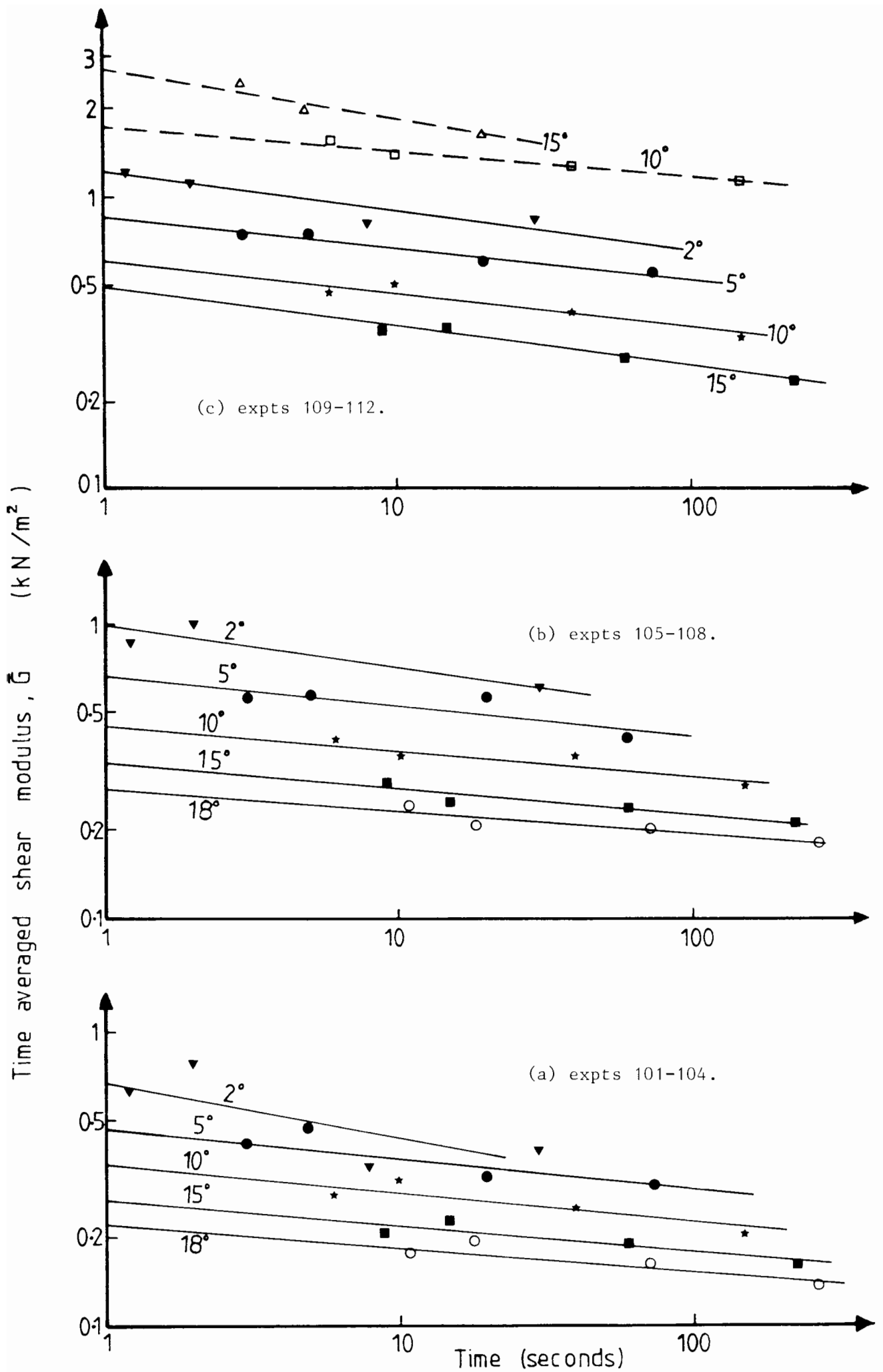


Figure 6.12 Variation in time averaged shear modulus at different angles, expts 100.

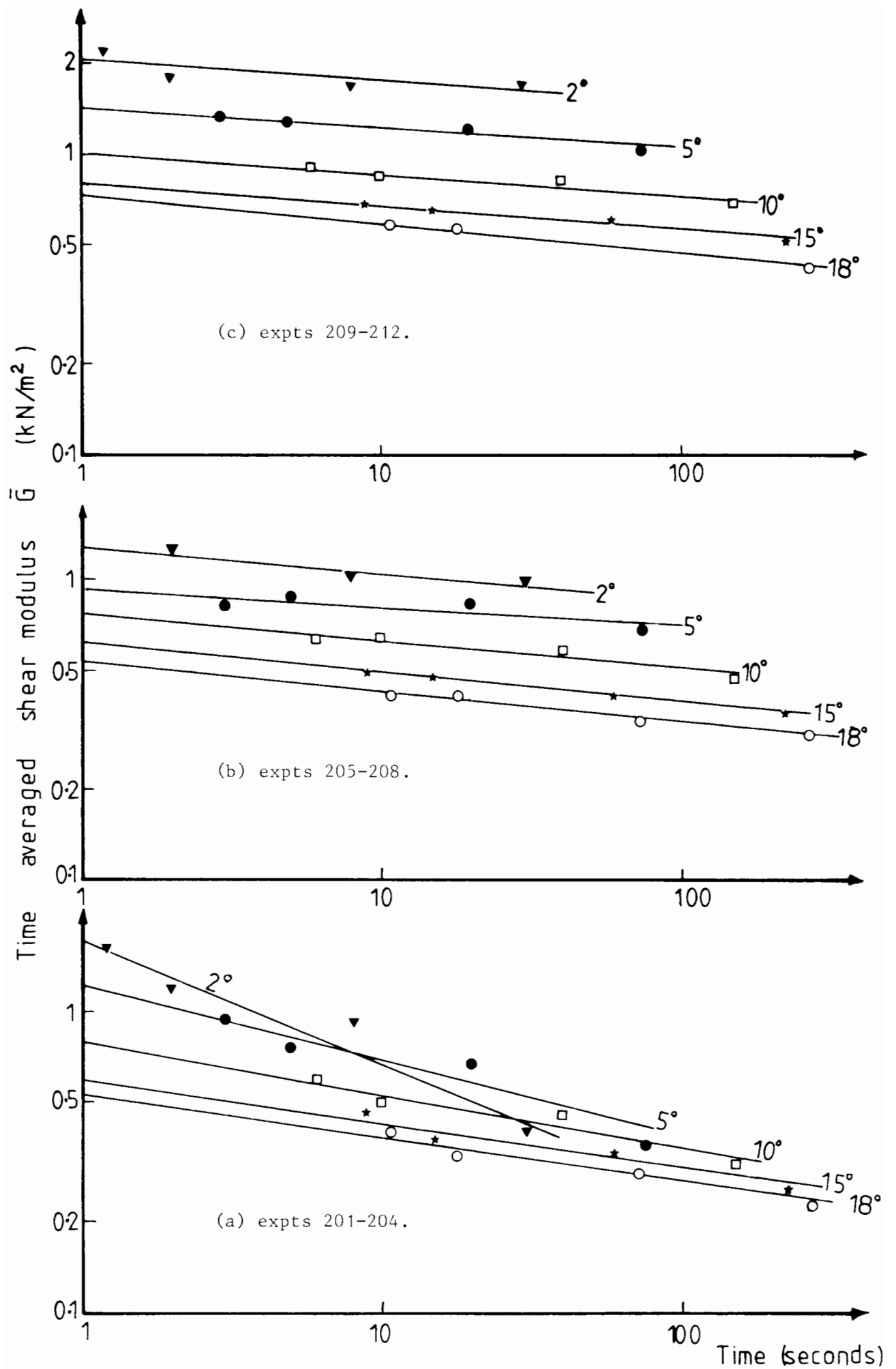


Figure 6.13 Variation in time averaged shear modulus at different angles, expts 200.

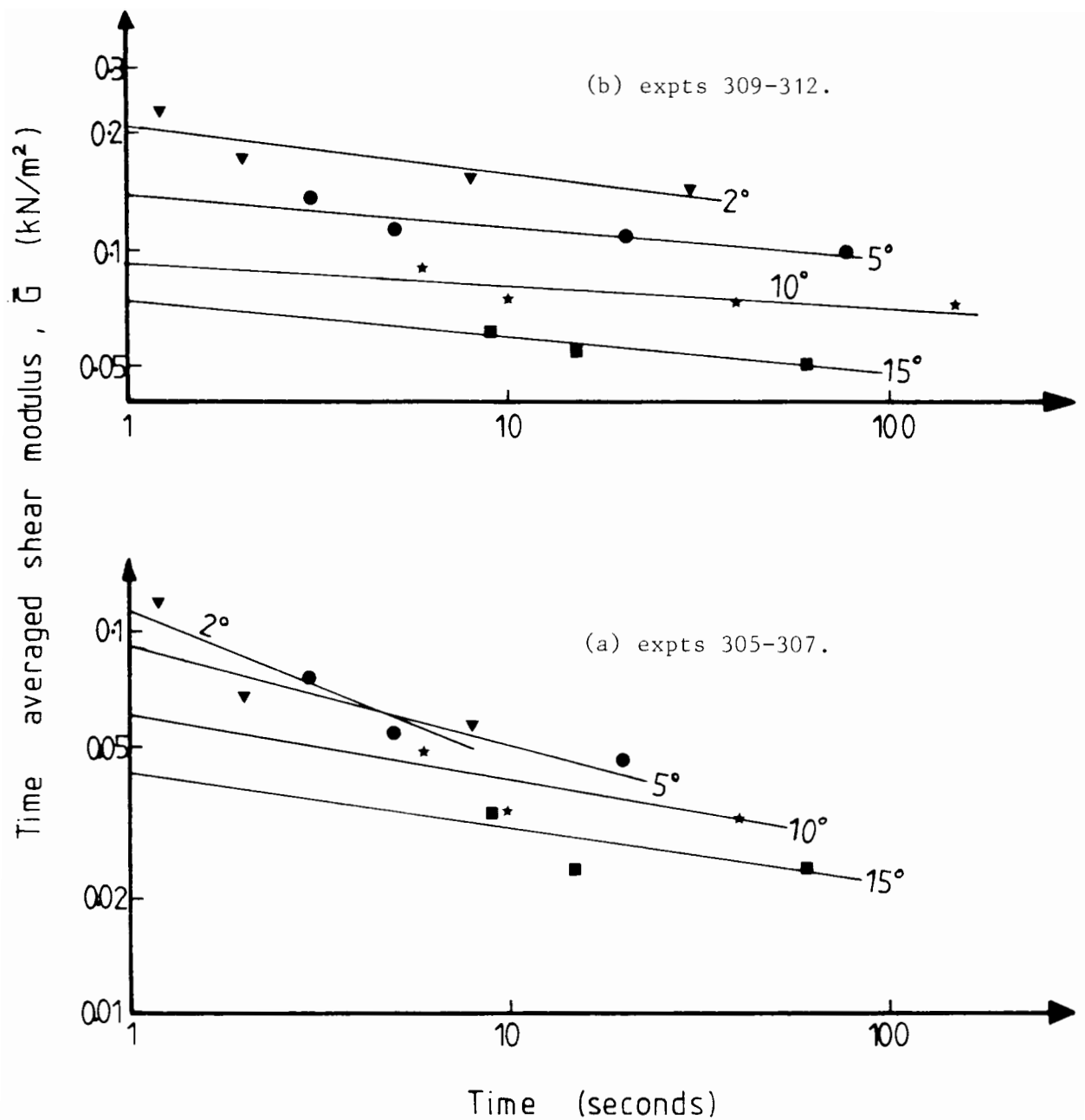


Figure 6.14 Variation in time averaged shear modulus at different angles, expts 300.

are independent of the timescale used but G_1 values are not. Times before failure are consistently reported in seconds in this study). Variations in n and G_1 with rotation angle are shown in figure 6.15. For many tests the exponent n is around 0.1. G_1 decreases with increasing angle of rotation and for different soil states but slopes of different lines are similar and suggest that G_1 might be approximated by

$$\log\left[\frac{G_1}{G_1(\theta=0)}\right] = -k\theta \quad (6.3.1)$$

where $k = 0.025 \text{ degree}^{-1}$ would be suitable for all soil states. Values of $G_1(0)$ at zero rotation are extrapolated from each curve and plotted against liquidity index in figure 6.16, together with the exponent n . Soil aging is again seen to be important. Although G_1 and the actual shear modulus, G , or the time averaged secant modulus, $\bar{G}$, are not equivalent since equation 3.6.8 models an infinite shear modulus at zero rotation, the variations in $G_1(\theta = 0)$ with changes in liquidity index are similar to those observed for the peak shear modulus in figure 6.9. The peak shear modulus curve for tests one hour after deposition is shown for comparison in figure 6.16. The exponent n is almost independent of liquidity index and time following deposition and might therefore be considered a vane test parameter rather than a soil parameter.

In this section a simple viscoelastic model has been shown to approximate vane test results adequately over a wide range of soil states. Two parameters must be specified to determine behaviour under any vane testing conditions, but one (n) appears constant at about 0.1 for all soil states. One vane test in a particular soil is therefore sufficient to allow prediction of results for tests at other rotation rates, using equations 3.6.9 and 6.3.1.

6.5 Spherical cavity expansion results

Variations in undrained strengths (calculated as described in chapter 4) with liquidity index and expansion rate are shown in figure 6.17 for experiments 100, 200 and 300. Significant strength increases occur with increasing expansion rates from 0.05 to 2 ml/minute. Different figures

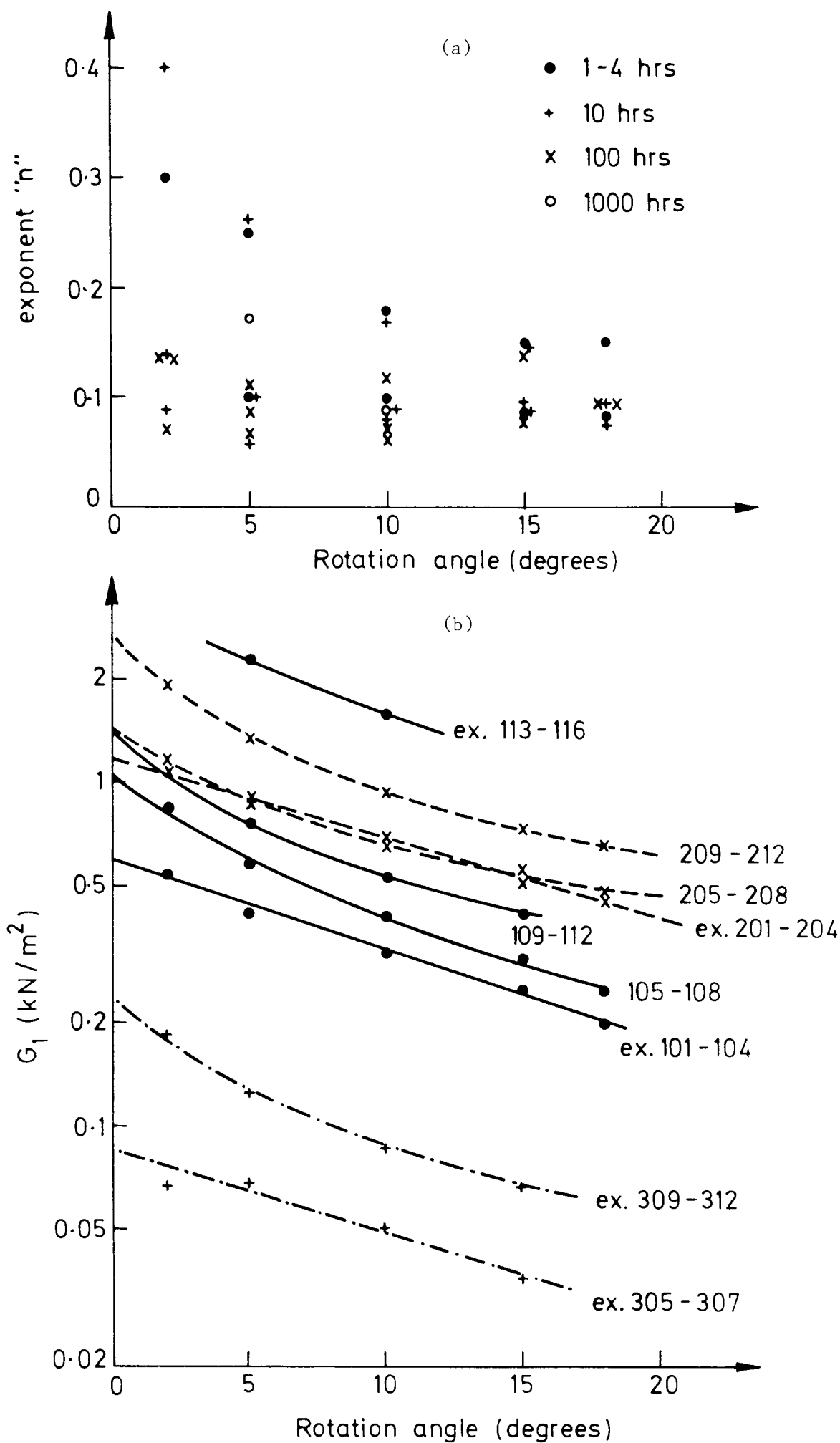


Figure 6.15 Variations in n and G_1 with rotation angle, $G(t) = G_1 t^{-n}$.

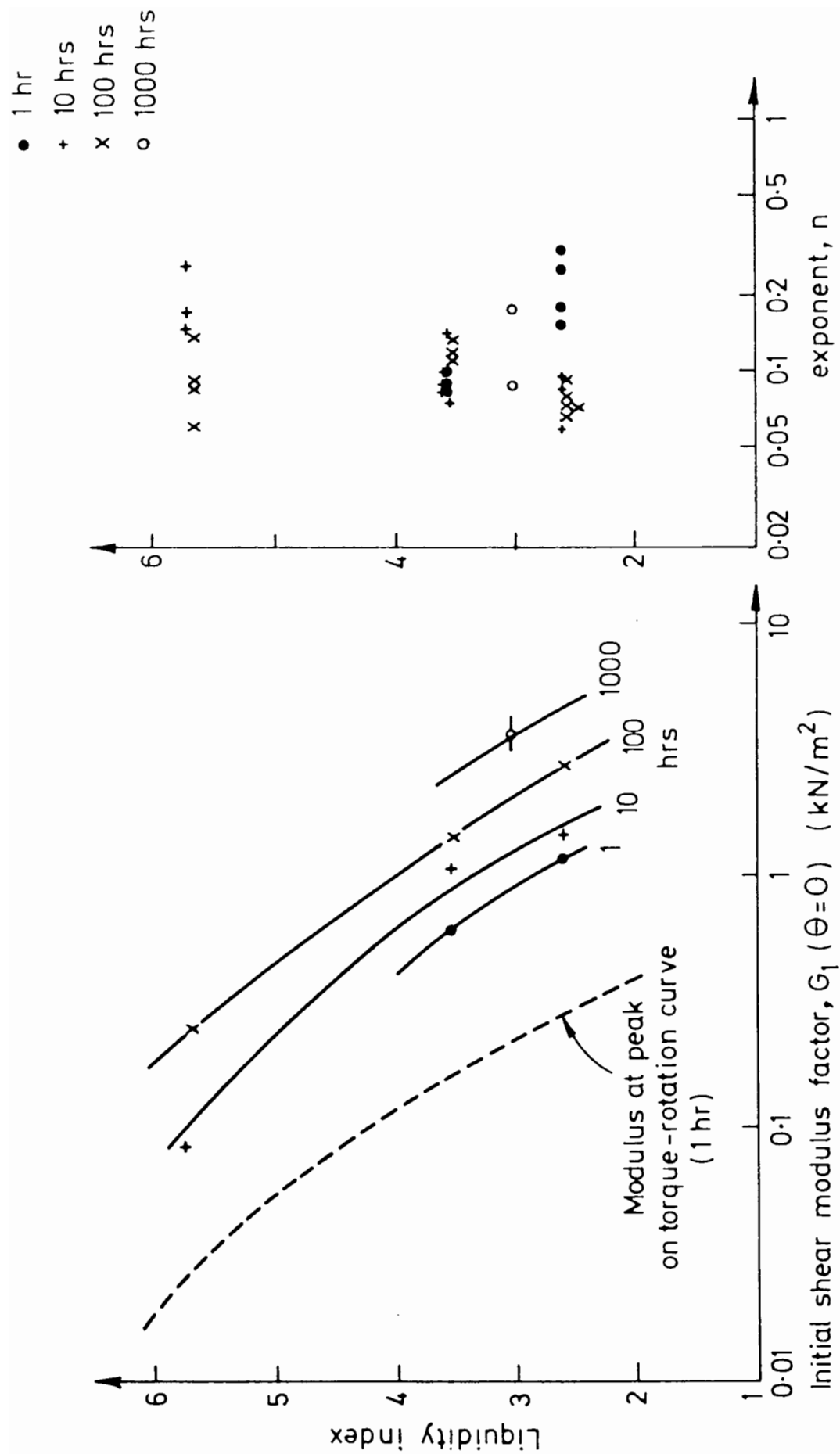


Figure 6.16 Variation in $G_1(0)$ and n with liquidity index and aging.

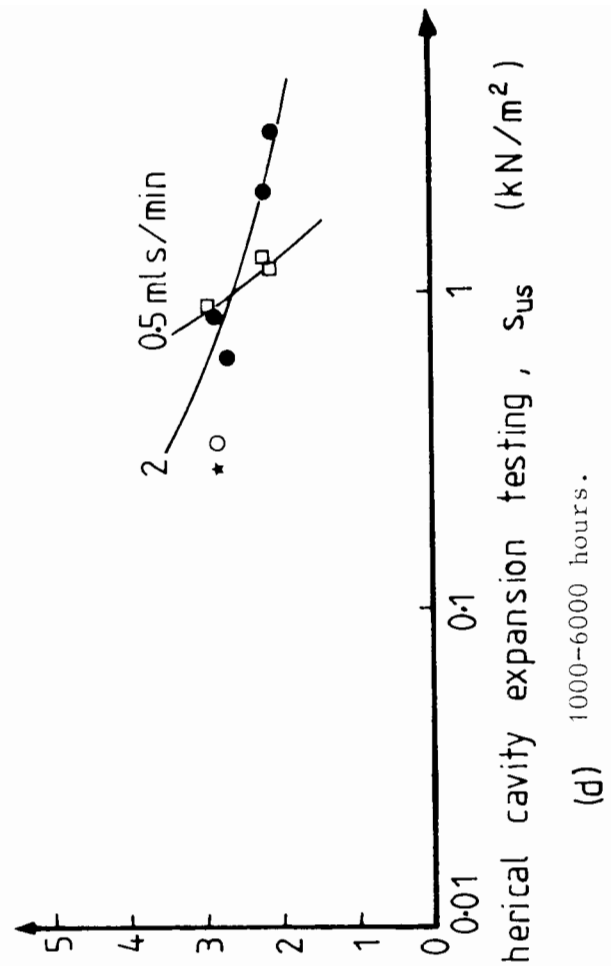
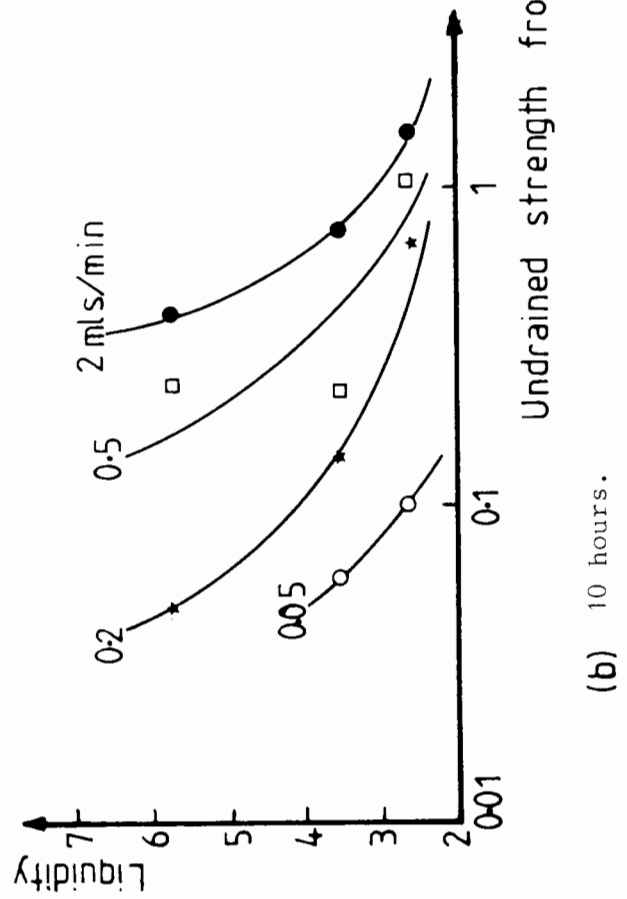
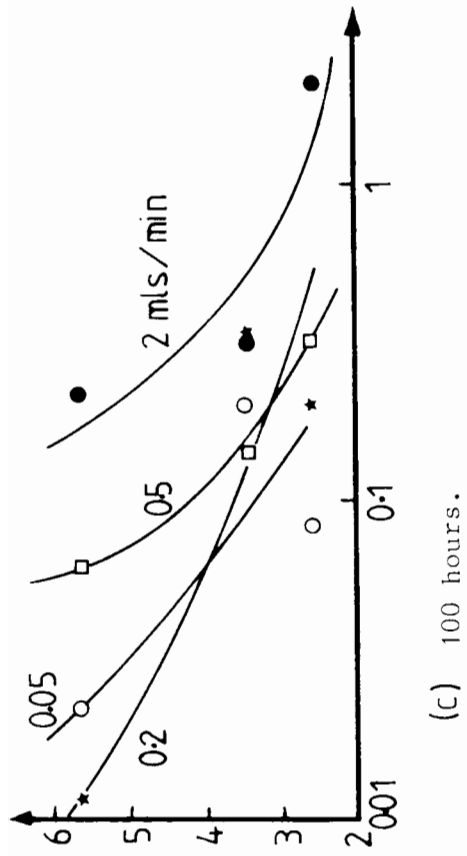
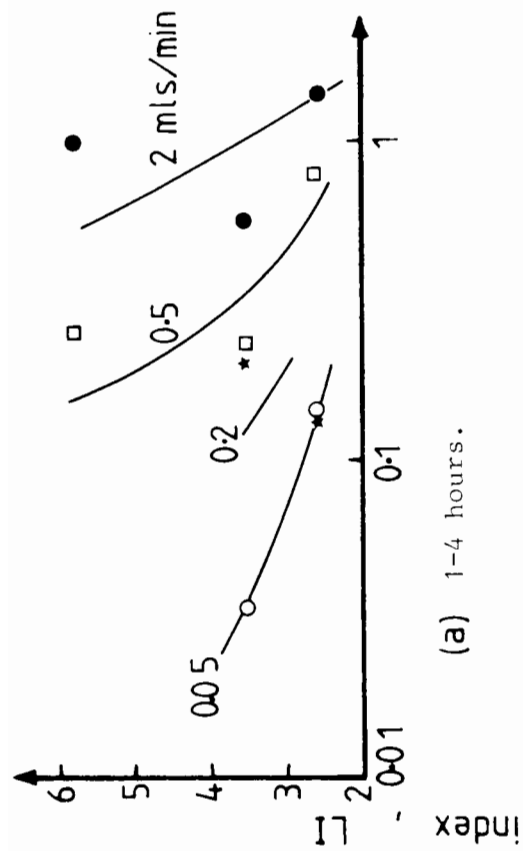


Figure 6.17 Variations in undrained strength from cavity expansion tests with liquidity index and expansion rate, expts 100, 200, 300.

represent different consolidation periods; considerable scatter is evident but curves for constant expansion rates are similar in all figures and consistent time dependent strength increases are not evident at constant liquidity index. This suggests that the cavity strength is representative of the residual rather than peak soil strength. The undrained expansion analysis presented in chapter 3 explains this observation; at limiting pressure a plastic zone extends around the cavity to a distance much greater than the cavity radius. Throughout most of this zone, and in particular adjacent to the cavity, soil will be in the residual state even if the shear stress at each soil element has earlier passed through a peak. All data from figure 6.17 are combined in figure 6.18 and tests results from tall column experiments included. Trends are consistent with the variation in residual strength measured in vane and fall cone tests, figures 6.1 and 6.2, represented by the approximation $s_{ur} = 3/LI^3$. Cavity strengths are generally higher than cone and vane strengths although cavity tests at 0.05 ml/minute expansion rate give similar results.

Possible explanation for the higher cavity strengths is indicated in figure 6.19, showing cavity strength variation with time to peak pressure for experiments 100, 200, 300. Strengths are normalised for a value of unity at 2 mls/minute expansion rate. Rather more scatter results than in the similar figure for vane test results (figure 6.4) and is probably due to the theoretical prediction that limiting pressure occurs at infinite cavity volume whereas in these tests peak pressures occurred at small cavity volumes. Reasons for this were discussed in chapter 4. Peak pressures are attained in most cases within ten seconds while times to peak resistance in vane tests were up to 300 seconds at slower rotation rates and around 10-20 seconds at 60°/minute, the rate used in tall column tests.

Effects of expansion rate are evaluated separately in figures 6.20a and b, for experiments 100, 200 and 300 only. The increase in normalised strength is up to tenfold for a tenfold increase in expansion rate and is most marked for tests carried out shortly after deposition (figure 6.20a). The

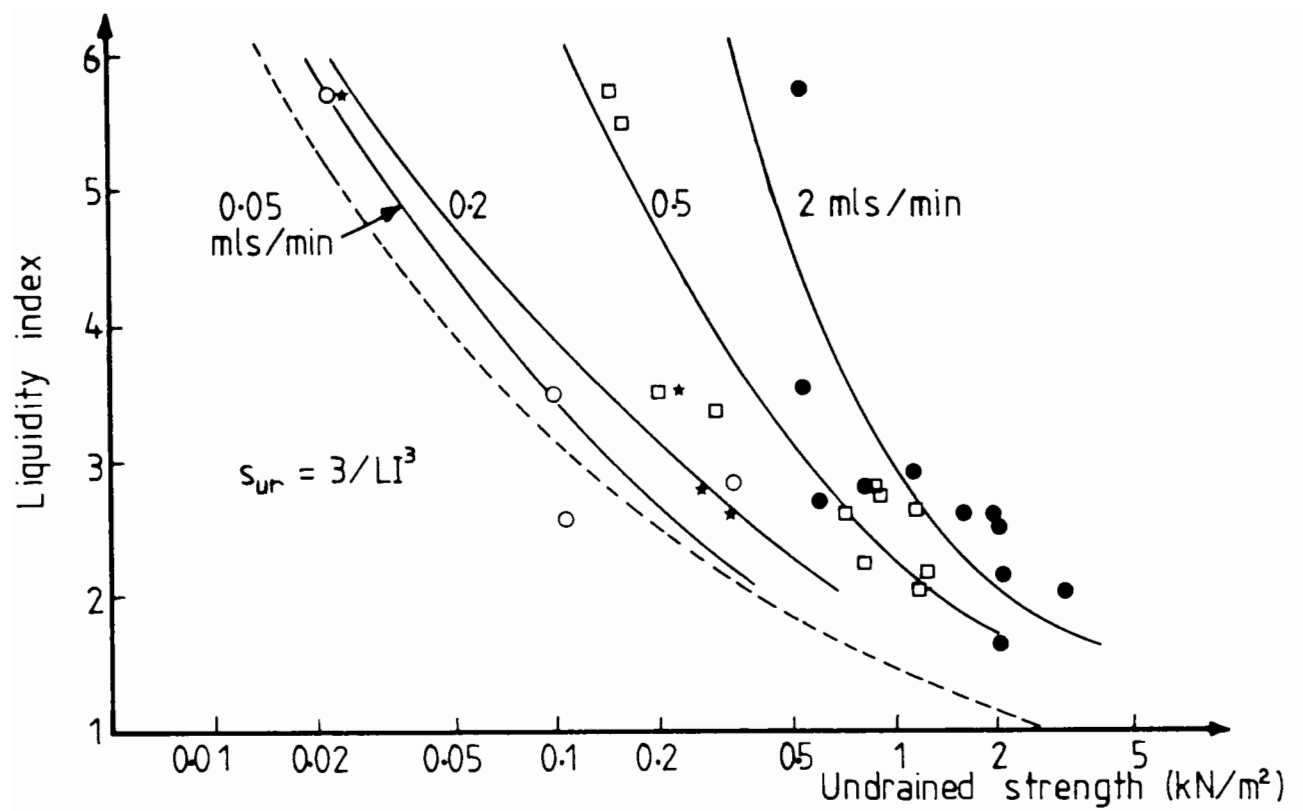


Figure 6.18 Variation in undrained shear strength with liquidity index for all cavity expansion tests showing effects of expansion rate. Strength averaged from tests at 1-1,000 hours, and 2000, 6000 hours at each LI level.

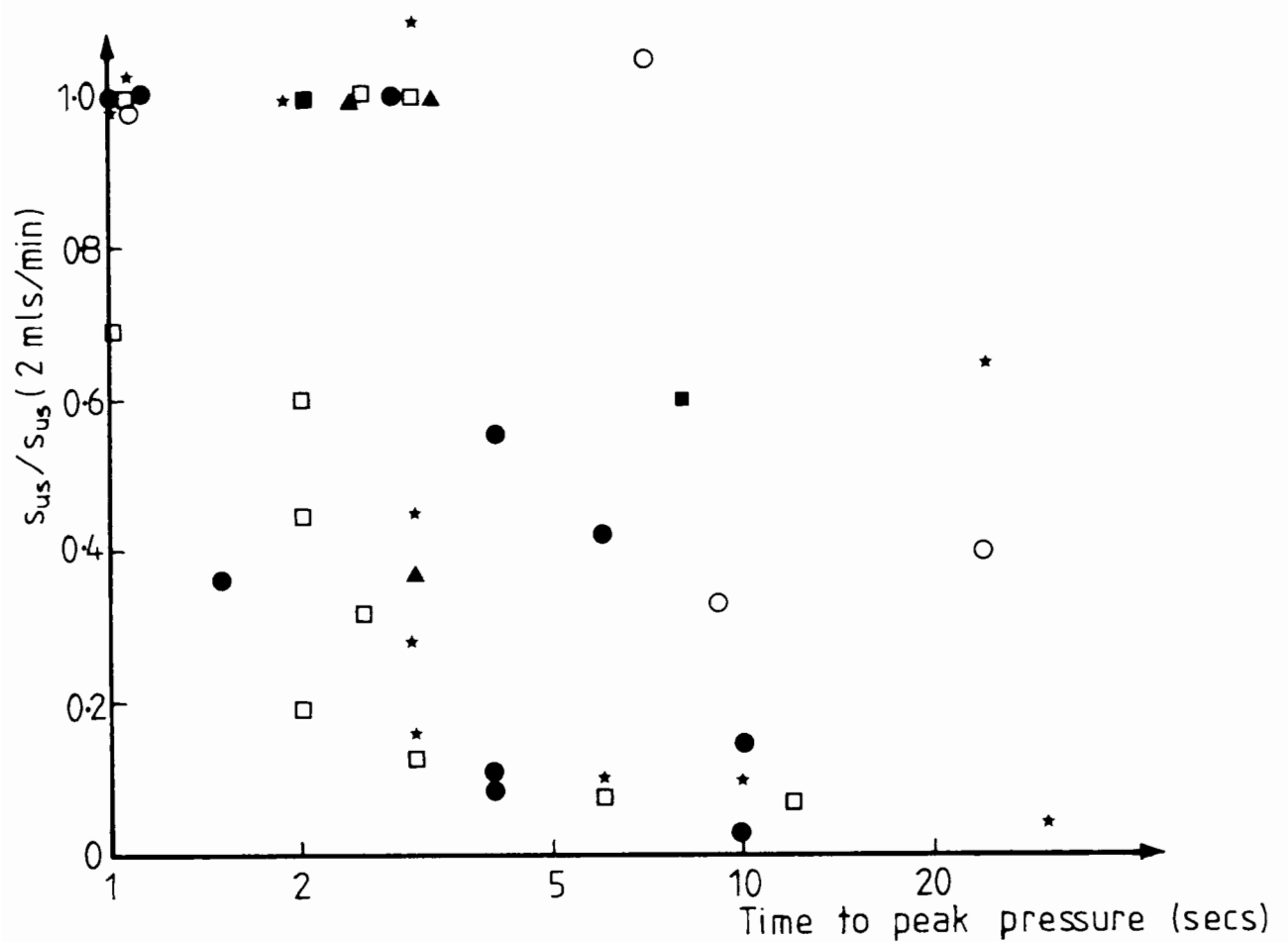


Figure 6.19 Effect of time to peak cavity pressure on strength as ratio of strength at 2 mls/min, expts 100, 200, 300.

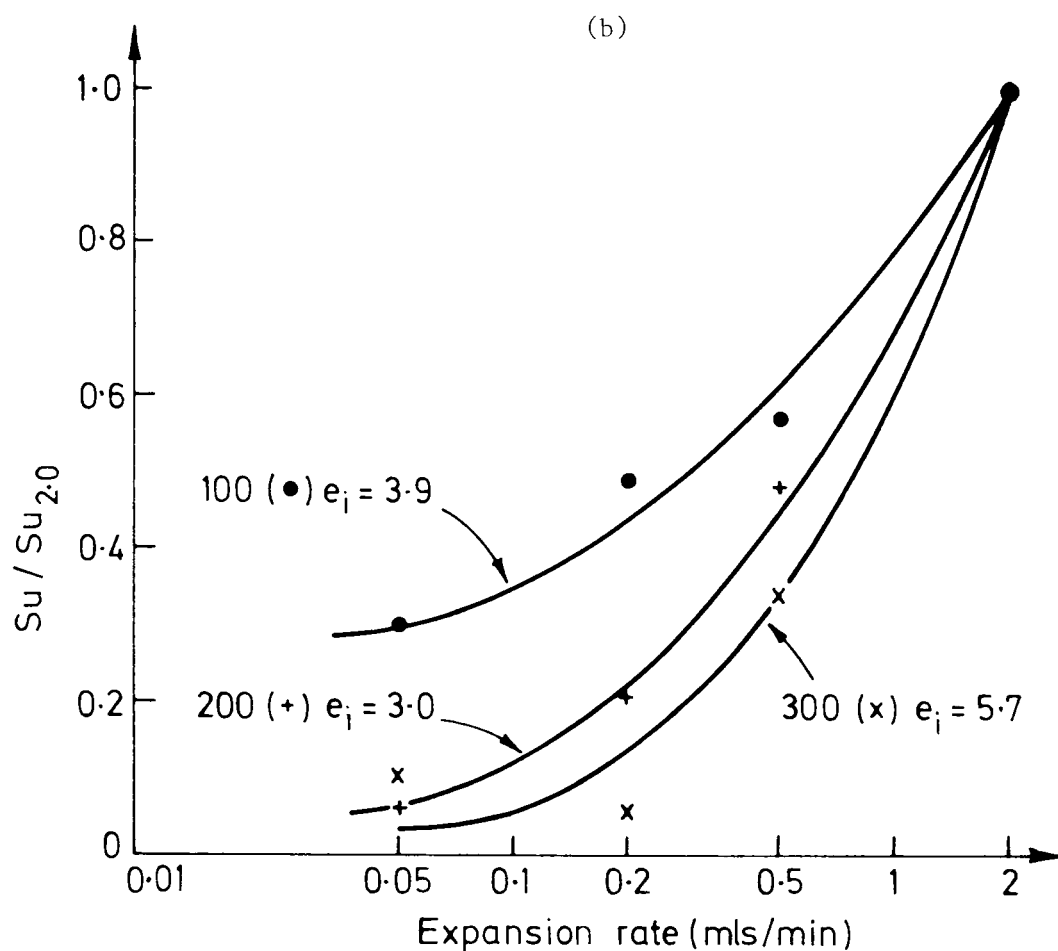
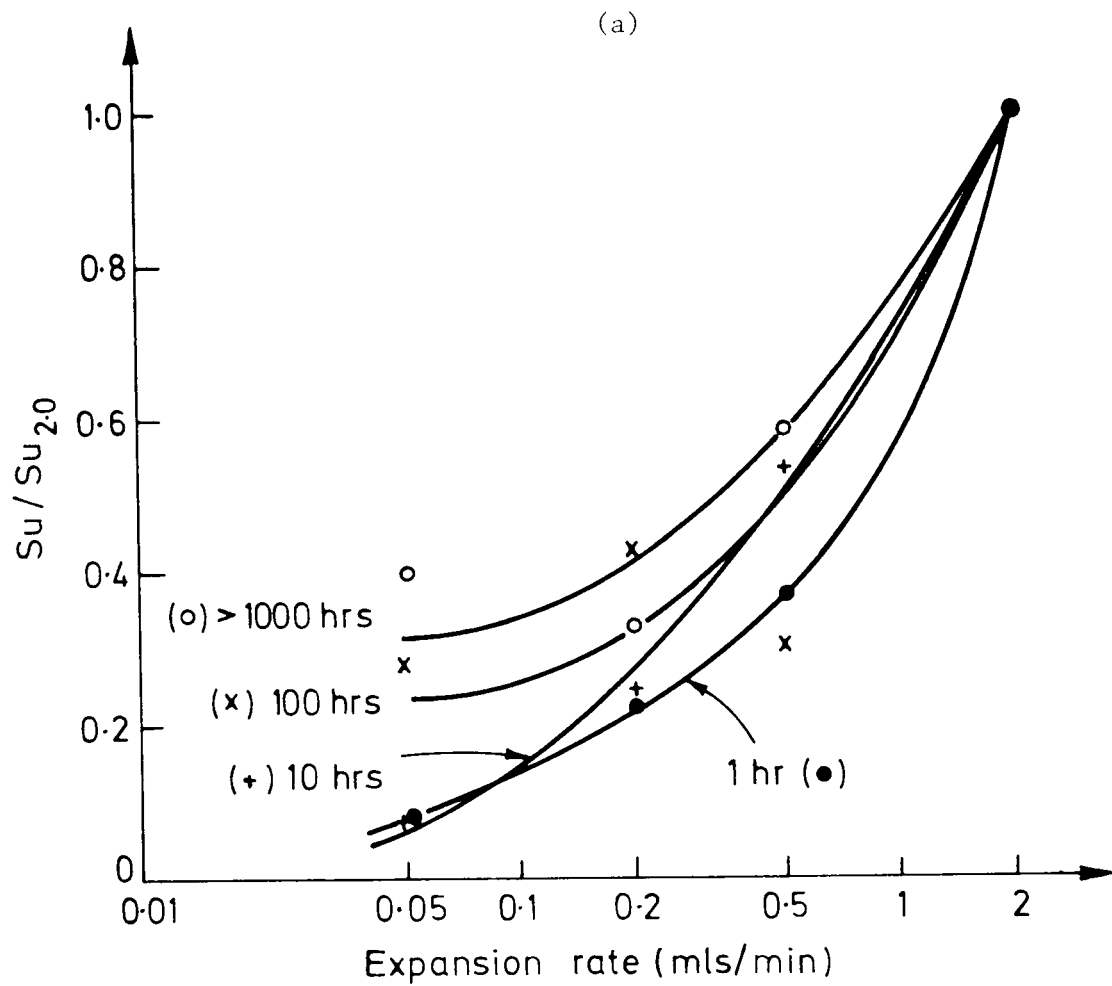


Figure 6.20 Effect of expansion rate on undrained strength from cavity expansion tests, strengths as ratio of strength at 2.0 mls/min rate
 (a) at different times after deposition
 (b) for different series of tests (different initial densities).

interchanged hierarchy of normalised strengths ($s_u(\text{ex } 100) > s_u(\text{ex } 200) > s_u(\text{ex } 300)$) from that of initial void ratios ($e_i(\text{ex } 300) = 5.7$, $e_i(\text{ex } 100) = 3.9$, $e_i(\text{ex } 200) = 3.0$) suggests that the effect of expansion rate is relatively independent of liquidity index in the soil. Rate effects are of much greater significance in cavity expansion tests than in vane tests, where a tenfold rate increase caused only a 10% increase in residual strength. Effects of consolidation around the cavity and structural viscosity are combined in the effects shown here and would be difficult to separate without a complex testing programme.

In figure 6.21 undrained strengths are compared for cavity expansion and vane tests in the same samples, experiments 100, 200 and 300. Residual vane strengths obtained at 60°/min are used. Circular symbols representing cavity tests at 0.05 ml/min lie close to the line for cavity strength : vane strength equality at strengths from 0.02 kN/m² to 0.3 kN/m². Curves for cavity tests at faster rates are nearly parallel to this line (in double logarithmic space) indicating that tests at faster rates give strengths which are approximately constant multiples of the equivalent vane strengths. Cavity and fall cone strengths are compared in figure 6.22. The fall cone has been shown already to measure the soil peak strength rather than the residual strength; nevertheless reasonable correlation exists between results from the two tests. Expansion test results are dominated by rate effects and for the range of expansion rates employed strengths vary by a factor up to 20, whereas soil sensitivities are less than 5. This explains the correlation in figure 6.22; an expansion rate increase from 0.05 to 0.5 ml/min is sufficient to cause a cavity strength increase to be registered which is greater than the difference between vane residual and fall cone strengths.

6.6 Strength dependence on effective stress. Normalised soil behaviour

In normally consolidated soils good correlations may usually be obtained between undrained strength and the in situ effective stress. Although the mean

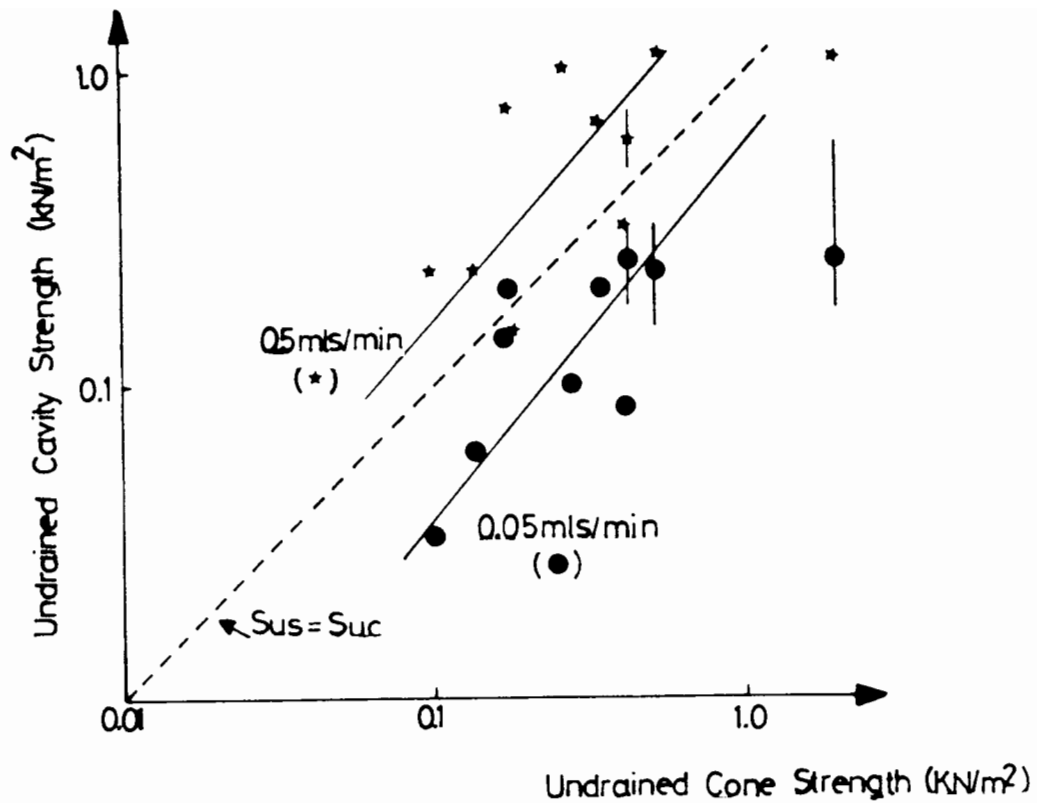


Figure 6.22 Comparison of undrained strengths from fall cone and cavity expansion tests for expt series 100, 200, 300.

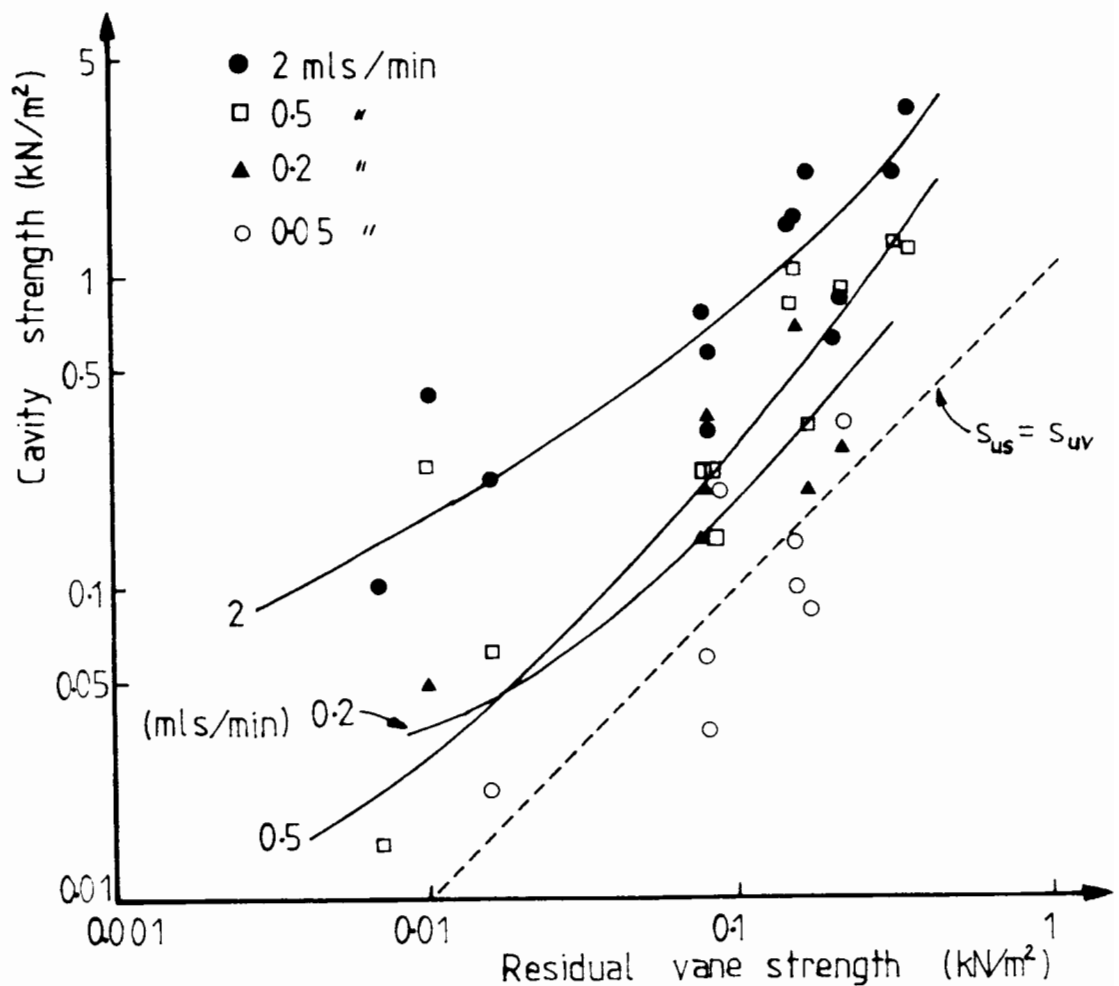


Figure 6.21 Comparisons of undrained strengths from vane and cavity expansion tests for expt series 100, 200, 300.

effective stress, p' , is most suitable for use in such correlations it was not possible to calculate the horizontal effective stress, σ'_h , to a sufficient degree of accuracy for p' to be calculated and instead vertical effective stresses are considered.

Variation in residual undrained strength s_{ur} with the vertical effective stress measured before unloading and testing, σ'_{vo} is shown in figure 6.23. There is clearly no unique relationship and even within experiments scatter about mean lines is considerable. The indication is that soil is not behaving as though normally consolidated. The conclusions drawn from analysis of consolidation behaviour in chapter 5 suggest that creep compression may have caused soil states to move below the curve for instant compression (normal consolidation) in void ratio-effective stress space. On subsequent loading or shear testing the soil would effectively behave as though overconsolidated. If this explanation is valid it should be more appropriate to relate the remoulded or residual strength to the equivalent vertical effective stress, σ'_{ve} , on the instant compression curve at the same void ratio. The equation of this curve for the soil used was found in chapter 5 (figure 5.11) to be

$$e = 5.7 \sigma'_{ve}{}^{-0.29} \quad (\sigma'_{ve} \text{ in kN/m}^2) \quad (6.6.1)$$

All data in figure 6.23 have been replotted in figure 6.24, showing s_{ur} as a function of σ'_{ve} . A logarithmic scale is used on both axes in anticipation of a non linear relationship between s_{ur} and σ'_{ve} but a remarkably good fit is obtained to all data by the fitted line defining a constant $s_{ur}:\sigma'_{ve}$ ratio

$$s_{ur} = 0.03 \sigma'_{ve} \quad (6.6.2)$$

The value of the constant however is much lower than typical values for normally consolidated soils, given for example by Skempton's (1954) equation

$$s_{ur}/\sigma'_v = 0.11 + 0.0037 PI \quad (6.6.3)$$

where PI is the plasticity index. For the soil in these tests $PI = 32$, suggesting a ratio $s_{ur}/\sigma'_v = 0.23$. The value of the constant found (0.03) shows that stress dependent behaviour is very different at the high liquidity indices investigated in this study. Comparison of figures 6.23 and 6.24 shows

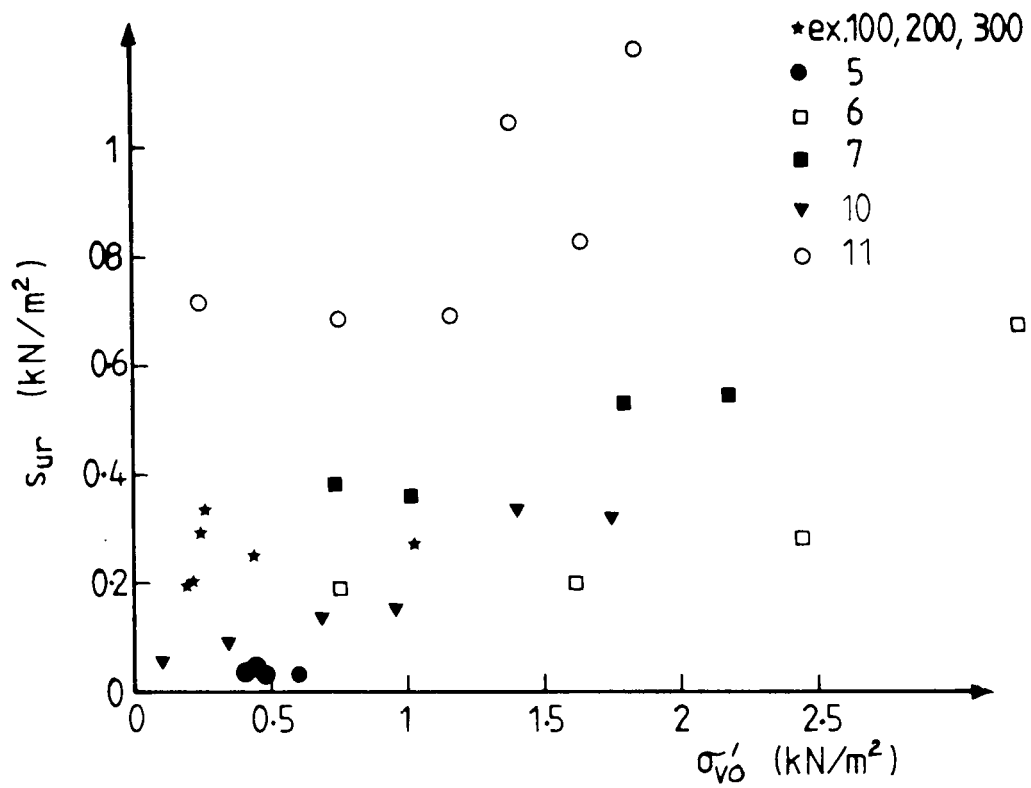


Figure 6.23 Variation in residual undrained strength s_{ur} with vertical effective stress before testing, σ'_{v0} .

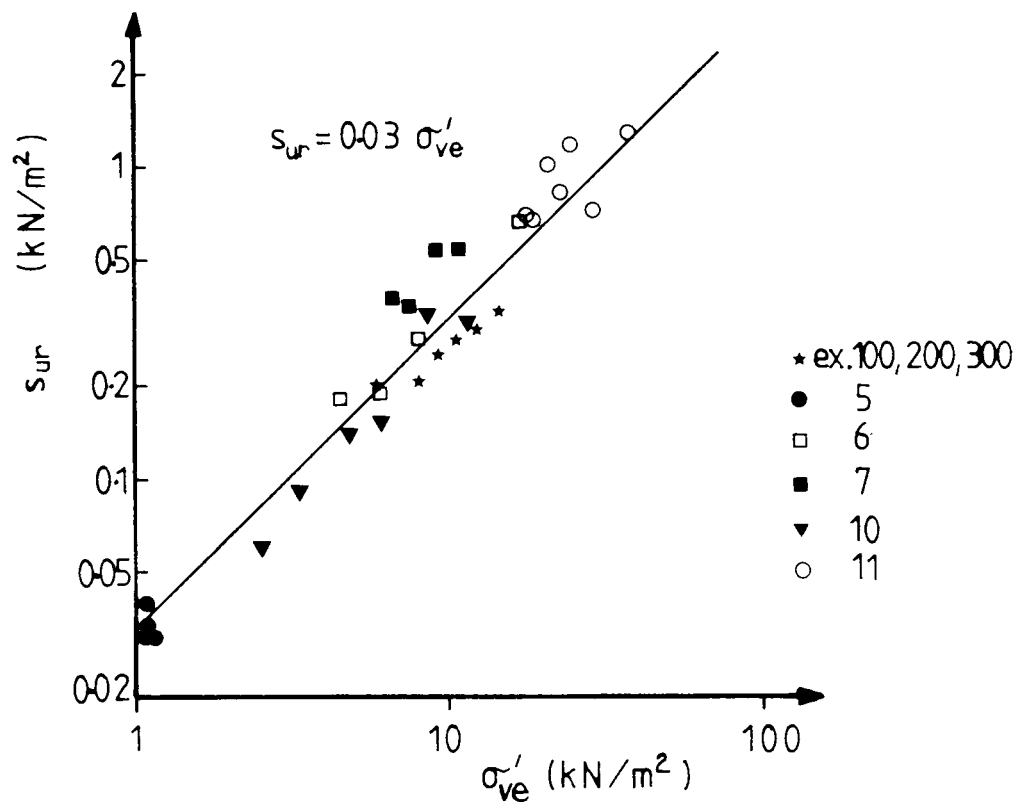


Figure 6.24 Residual undrained strength s_{ur} variation with equivalent effective stress, σ'_{ve} .

that great caution should also be used when making such correlations if any creep compression is likely to have occurred in a soil assumed to be normally consolidated. Calculation of s_{ur}/σ'_{vo} for different experiments from figure 6.23 yields values which vary from 0.07 (experiment 5) to 0.75 (experiment 11). The existence of peak strengths higher than residual values has already been shown to depend on an aging process during consolidation. If the effect of this aging is simply to cause an increase in soil frictional strength reasonable correlation should be obtained between the shear strength and some measure of the overconsolidation ratio. A logical extension of the discussion above, and of results from chapter 5, suggests that the equivalent overconsolidation ratio might be defined by

$$OCR = \sigma'_{ve}/\sigma'_{vo} \quad (6.6.4)$$

i.e. the ratio of the equivalent vertical effective stress on the instant compression curve at the same void ratio to the existing in situ stress. The peak undrained strength s_{up} should correspondingly be normalised using σ'_{ve} . Peak strength data are presented in this form in figure 6.25. Even with a logarithmic OCR scale scatter is considerable and no evident trend can be seen. The highest s_{up}/σ'_{ve} values are for the experiments in which the consolidation time was longest. Peak strength behaviour appears therefore to be dominated by time dependent cementing processes rather than frictional processes.

Variation of the undrained shear modulus to shear strength ratio G/s_u was examined in section 6.3, where poor correlation with liquidity index was observed. In figure 6.26 however, good correlation is obtained between G/s_u and the overconsolidation ratio defined by equation 6.6.4. Similar variations were shown for several clays by Ladd and Edgers (1972) where G/s_u was calculated from direct simple shear test results at 1/3 and 2/3 of the maximum shear stress. Overconsolidation ratios were defined by the maximum consolidation stress (the same for each test) before swellback to different stresses. Values of G/s_u obtained in this study are very much lower however; values in figure 6.26 range from 1 to 7 while Ladd and Edgers report values

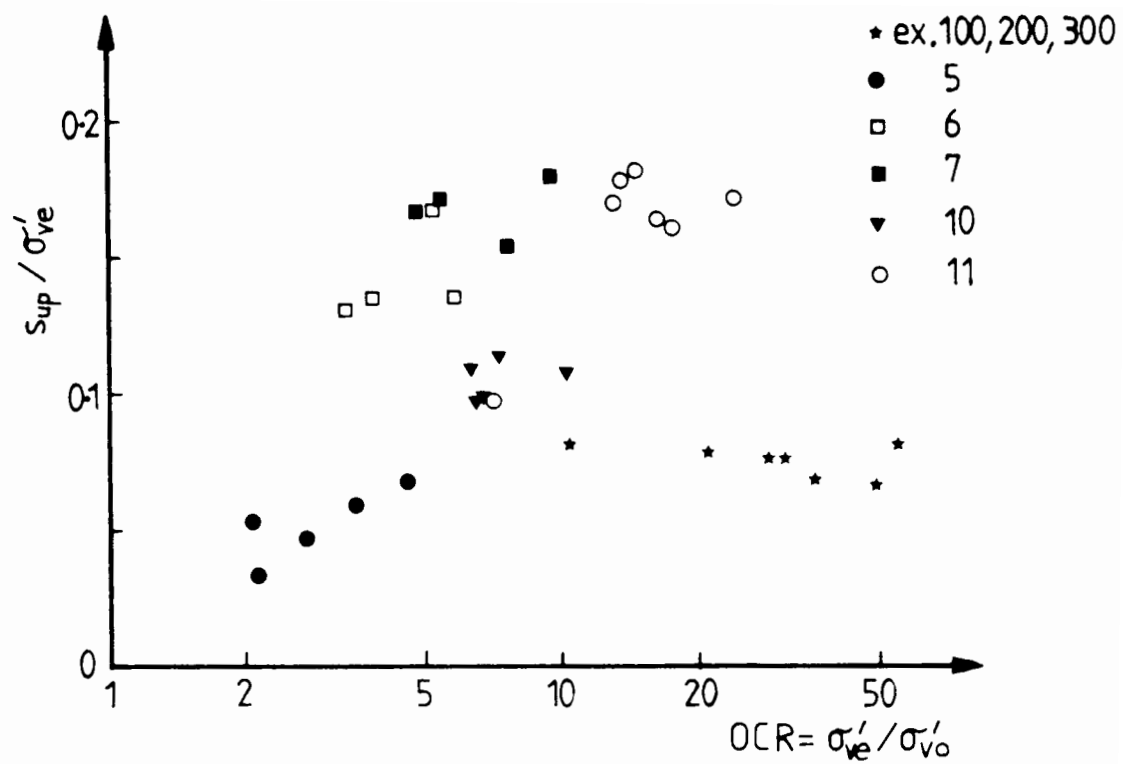


Figure 6.25 Normalised peak strength variation with equivalent OCR.

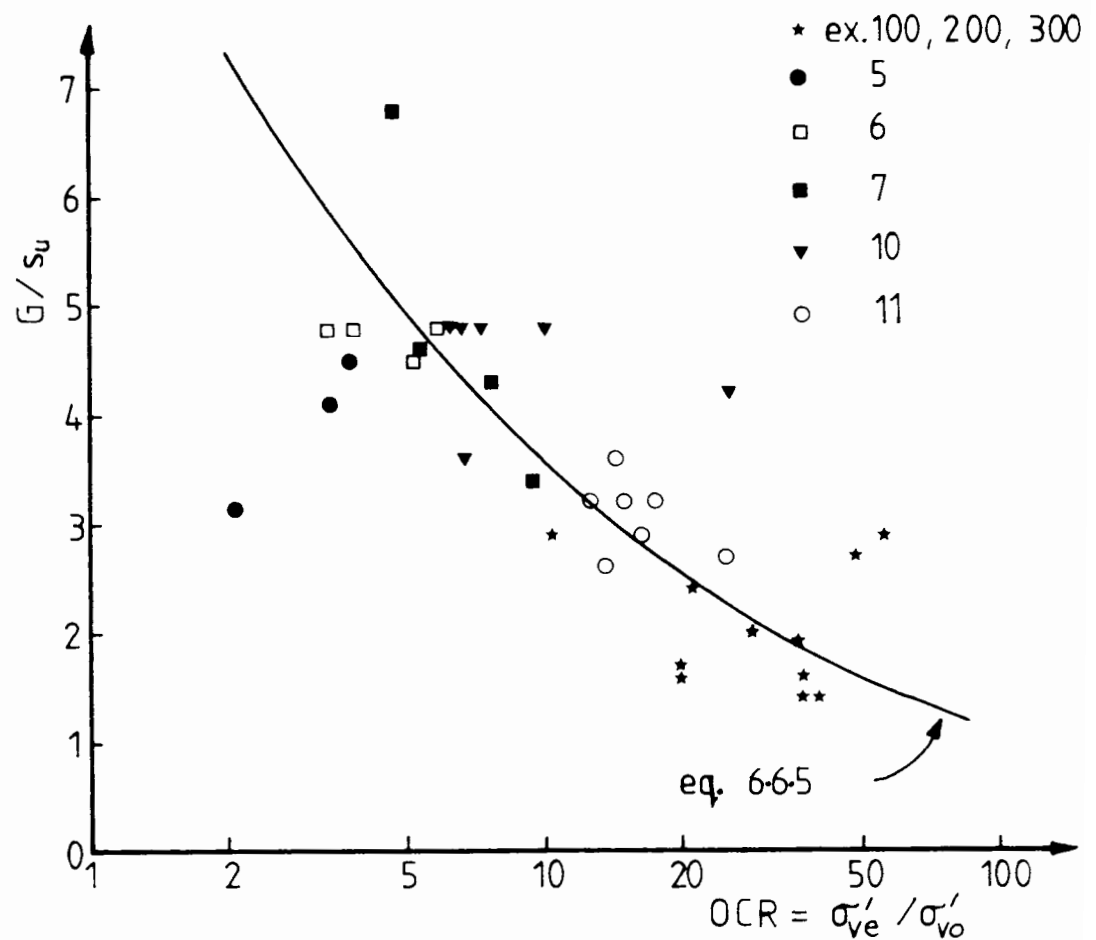


Figure 6.26 Variation in ratio of shear modulus to undrained strength with equivalent overconsolidation ratio. Liquidity indices for data shown range from 1.1 to 5.8.

from 20 to 300. Wroth et al (1979) suggested that G/s_u might vary according to the relationship

$$G/s_u = (G/s_u)_{nc} [1 + C \ln(OCR)] (OCR)^{-\Lambda} \quad (6.6.5)$$

where $(G/s_u)_{nc}$ is the value of G/s_u on the normal consolidation curve, C is a soil constant and Λ is given by

$$\Lambda = (\lambda - \kappa)/\lambda \quad (6.6.6)$$

where $-\lambda$ and $-\kappa$ are the slopes of the normal consolidation and swelling lines respectively for data which plot linearly in $e-\ln p'$ consolidation space.

Equation 6.6.5 has been fitted to the data in figure 6.26 by the curve plotted and gives values of $(G/s_u)_{nc} = 9.7$, $C = 0.27$ and $\Lambda = 0.65$. Both C and Λ are consistent with values expected for stiffer soils while $(G/s_u)_{nc}$ remains low.

It is important to note two points. The G/s_u values in this section have been calculated from peak resistances, and the overconsolidation values defined using the equivalent effective stress σ_{ve}' at the same void ratio. Expression 6.6.5 was derived for an overconsolidation ratio using the previous maximum effective stress, i.e. taking into account void ratio variation along the swelling curve. Nevertheless the fit obtained in figure 6.26 suggests that the general method is suitable. Consolidation data at these void ratios do not plot linearly in $e-\ln p'$ space so that consideration of λ and κ values is not particularly useful.

6.7 Triaxial test results

Consolidation behaviour for soil at the column base during experiment 5 and subjected to one-dimensional and isotropic reloading phases was discussed in the previous chapter. The effective stress path followed during consolidation and undrained deviatoric phases is plotted in figure 6.27 together with the stress path during shearing for a 38 mm sample (T7) consolidated isotropically to 41 kN/m². The critical state line defined by stress path end points has slope $M = 1.44$ ($\phi' = 36^\circ$), which is rather high for a soil of this type having medium plasticity. The stress path followed by an

"undisturbed" sample JG02 from depth 2.4 m at Combwich (Graham, 1982) is shown for comparison. Handling of this sample was minimised and the quality of sampling, trimming and testing believed to be high. The sample was consolidated at a constant stress ratio $\sigma_3'/\sigma_1' = 0.48$, which resulted in very small lateral strains so that the sample remained close to the K_0 condition. The maximum deviatoric stress during shearing is slightly below, but close to, the CSL for the two samples consolidated isotropically in this study. Ratios of the undrained strength to the vertical effective stress before shearing are 0.45 (T7) and 0.35 (experiment 5) for the samples where $\sigma_{v_o}' = p_o'$, and 0.34 for JG02. These values are all considerably higher than the value of 0.03 suggested for the ratio s_{ur}/σ_{ve}' in the previous section. This value occurred in soils at effective stresses up to 3 kN/m² which had experienced considerable aging and indicates the very different strength characteristics exhibited by soils at states wetter than the liquid limit.

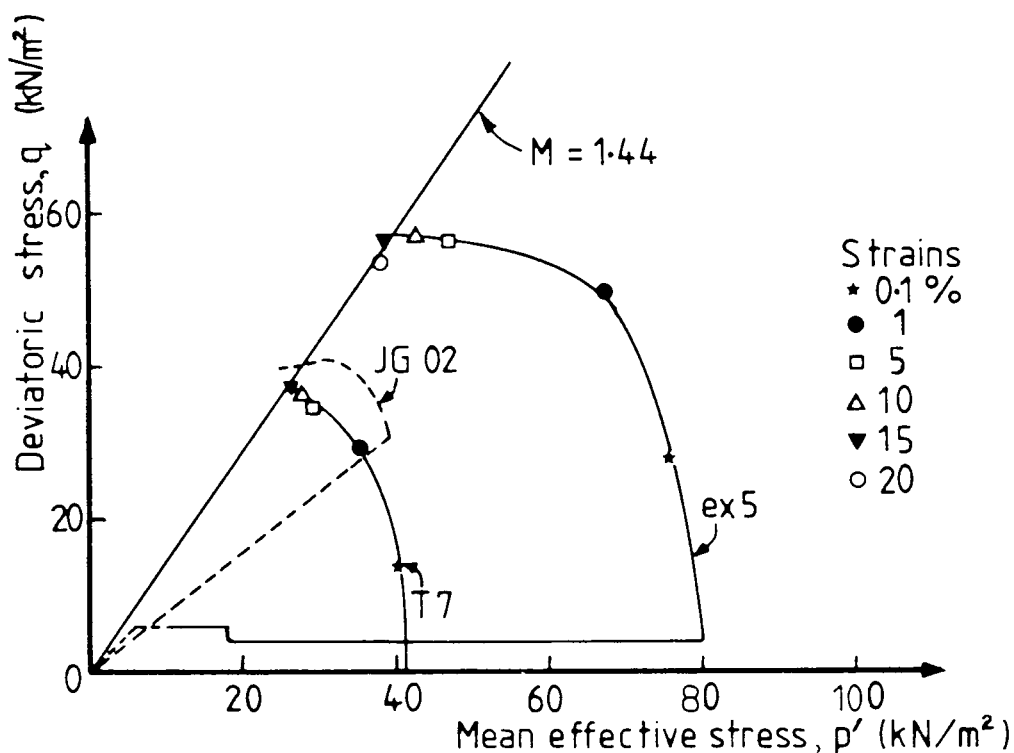


Figure 6.27 Stress paths in triaxial tests. Sample from expt 5 below 212mm after 500 hours, reconstituted sample no. T7 and "undisturbed" sample JG02 tested by Graham J. in 1982.

7.1 Summary of results and conclusions

7.1.1 Experimental techniques

7.1.2 Compression behaviour

7.1.3 Strength behaviour

7.2 Implications and recommendations

7.3 Contributions of this thesis

7.4 Requirements for future work

6.8 Discussion

In this chapter strength behaviour has been examined in soils at liquidity indices ranging from 0.7 to 6 following compression under low stresses or self weight alone. Residual or remoulded strength correlates well with the liquidity index but peak or in situ strengths increase considerably with aging over relatively short time periods. Sensitivities up to 5 were recorded 1 year after deposition. Testing rate has a significant effect which is independent of the liquidity index level. Strengths measured using the shear vane and fall cone are almost identical over the entire testing range. Cavity expansion strengths are consistent with these but dominated by rate effects and considerable further development and testing with this technique is required although first results appear very promising. Both shear strength and shear modulus behaviour support the hypothesis presented in chapter 5 that the effect of creep compression is to cause soil to act as though overconsolidated under subsequent testing. The instant compression curve found in chapter 5 may be used to obtain values of the equivalent effective stress, σ_{ve}' , for normal consolidation. Undrained strengths vary nearly linearly with σ_{ve}' at all liquidity indices obtained in settling columns. The constant $s_u/\sigma_{ve}' = 0.03$ is equal to that obtained by Einsele et al (1974) for the ratio s_u/σ_{vo}' from a smaller number of tests on different soils which had been consolidated under self weight stresses up to 3 kN/m², following slow uniform deposition. This similarity is consistent with the hypothesis proposed in chapter 5, that consolidation following gradual deposition follows the instant compression line more closely than consolidation following rapid deposition, so that σ_{vo}' is close to σ_{ve}' .

Leroueil et al (1985) suggested that four distinct soil states occurred; the intact (natural), the destructured (large deformations), the remoulded and the resedimented (self weight consolidation) states. In particular, they distinguish between the sedimented and remoulded strengths and caution against confusion of the two. Their conclusions were based on the results of

sedimentation/consolidation tests on a clay of low plasticity and activity, reported by Locat and Lefebvre (1985), who suggested that sensitivity was a soil characteristic rather than a variable. Each soil column was built up by ten successive additions of a dispersed soil-water mixture at void ratio approximately 27. Results discussed in chapter 5 suggest that this method would probably have permitted significant segregation during each settling increment so that an inhomogeneous, layered sediment could have resulted. The (fall cone) strengths measured in this sediment might then not have been comparable to those measured in the same soil following full remoulding. An additional explanation for their measurement of sedimented strengths higher than remoulded strengths is the time elapsed during a consolidation phase up to vertical effective stress 50 kN/m^2 , which appears to have been about 300 hours. Samples were then unloaded with swelling permitted, so that in addition to the aging during loading, strengths were measured for overconsolidated states and so were not necessarily representative of the sedimented state at the same liquidity index.

Results evaluated in this chapter, when considered with this example, show that great care should be exercised when assigning strengths to different soil states. The remoulded and resedimented strengths are similar if aging or preconsolidation effects have not occurred; aging causes the soil sensitivity (and peak strength) to increase but the remoulded or residual strength is close to the resedimented strength in the same soil following rapid consolidation with minimal aging. Residual strength behaviour following creep compression is best described in terms of equivalent overconsolidation, but peak strengths are more dependent on aging. Anisotropy, which could not be evaluated here, may account for greater scatter observed in peak strength data.

7.1 Summary of results and conclusions

7.1.1 Experimental techniques

Settling columns have been used to model self weight consolidation of sediment layers deposited at void ratios from 3 to 14 and initial thicknesses from 200 mm to 3.5 m. The highest void ratios corresponded to soil states slightly denser than those found in suspensions so that subsequent compression was nearly representative of natural soils consolidating following rapid sedimentation from dense suspensions. The lowest initial void ratios occurred with significant effective stresses and strength existing in the soil immediately following deposition and states were comparable to those found in dense slurries of dredged or pumped waste. Effective stresses up to 4 kN/m² were attained at the base of the sediment columns but periods up to one year were required to achieve full pore pressure dissipation. Although the settling columns provide conditions relevant to those in the field, the long timescale of experiments means that laboratory modelling of sediment layers of this thickness is inconvenient and space consuming. Well instrumented long term field studies will be required to extend the range of data obtained in this study.

The X-ray density measuring apparatus developed at Oxford has been modified and used to obtain repeatable and accurate profiles of soil density and analysis has been presented which ensures consistency of results between profiles. Some discrepancies observed when using this system have been explained and systematic methods devised to circumvent the possible resulting errors. The horizontal stress remains very difficult to measure accurately without causing local soil disturbance due to the curvature and small diameter of the columns.

Soil strengths have been measured in the columns using the standard fall cone and shear vane techniques with equipment modified to improve resolution.

Results from these two tests are consistent at all soil states observed and both techniques appear suitable at very low soil densities. A new test has been developed involving expansion of a fluid filled near spherical cavity which allows testing at in situ stress levels with minimal soil disturbance. Some aspects of test results are difficult to explain but it is possible to make some reasonable assumptions about soil behaviour around the expanding cavity and calculate undrained strengths which follow trends observed in fall cone and shear vane tests, although magnitudes of strengths are very dependent on expansion rates. A survey of literature reporting strength tests in soils where other soil characteristics (e.g. liquid and plastic limits) are known suggests that strength values obtained from viscometer tests (extrapolated values of shear stress at zero shear rate) are consistent with fall cone and vane strengths, but that the critical fluid shear stress for bed erosion is up to several orders of magnitude lower than the remoulded undrained strength.

7.1.2 Compression behaviour

Several distinct regions of behaviour have been identified during the settling and consolidation process. If the deposited slurry has a void ratio greater than about 12 the soil will exist initially as a suspension with pore pressures equal to the total vertical stress and with no effective stresses present. A denser layer, in which effective stresses are greater than zero, will accumulate upwards from the base but will not necessarily behave in the same manner as material deposited at the same density. Variations may occur due to segregation, re-orientation and flocculation which have been allowed to occur freely during the suspension phase and due to other time dependent processes of volume change which have differed between the two situations. If soil is deposited at void ratio below 12, near immediate increases in effective stress occur without significant compression. Volume reduction in either case at later stages is controlled by the superposition of two effects; the decrease in void ratio due to effective stress increase as pore pressures dissipate, and the decrease with time alone (creep) at nearly constant effective stress. At

any instant, the relative importance of these two effects will determine the instantaneous direction in which a soil element will move in $e-\sigma'$ consolidation space. A large sediment depth, even at high void ratios, may cause creep compression to dominate at almost all stages of a test, due to the very slow dissipation of pore pressure. During periods where creep compression dominates, Darcy's law is not applicable to pore fluid flow and the settlement velocity or rate of pore fluid expulsion appears to be almost independent of the hydraulic gradient but uniquely dependent on the void ratio (or other measure of soil density). At lower void ratios, and hydraulic gradients less than about 60% of the critical hydraulic gradient for instability, stress compression becomes more important and Darcy's law is approximately satisfied.

The variable compression behaviour observed in experiments can be explained, with allowance for creep compression, by the existence of an instant compression curve, along which soil elements will consolidate if the rate of loading is sufficiently rapid. This curve has been deduced, for the Combwich soil used, from tests where the rate of effective stress increase was sufficiently rapid that negligible creep compression occurred, and is linear in $\log e - \log \sigma'$ space. Soil states represented by points below this line act as though overconsolidated. This provides an explanation for the rapid effective stress increases observed following deposition below the transitional void ratio, where soil elements consolidate as though moving back along a "swelling" line towards a preconsolidation point defined by the equivalent stress σ'_{vei} on the instant compression curve at the depositional void ratio. This overconsolidation or swelling line then defines soil states before any compression occurs. The instant compression line and a typical overconsolidation line are shown schematically in figure 7.1. The effect of creep compression is to cause consolidation paths for soil elements to move below these lines and attain states of greater equivalent overconsolidation at stresses σ'_{v2} and σ'_{v3} . If subsequent rapid loading is applied, stress dependent compression occurs and elements move along new overconsolidation lines to the

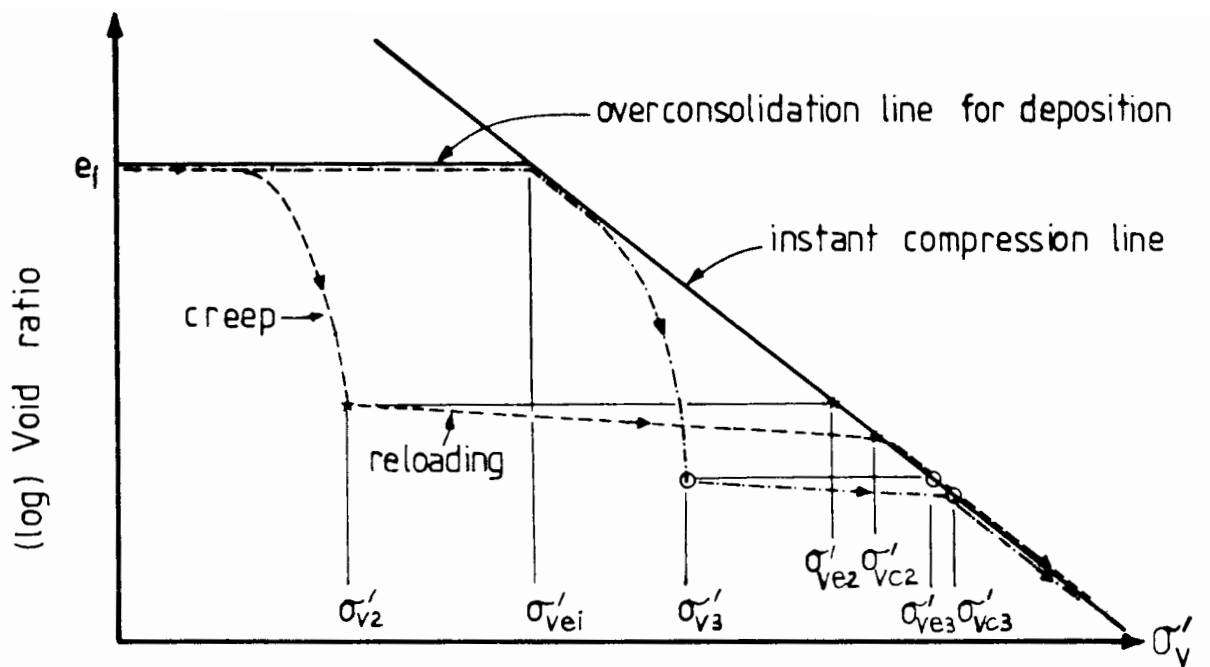


Figure 7.1 Consolidation paths followed by soil elements during deposition, self weight consolidation with creep compression, and on subsequent rapid loading.

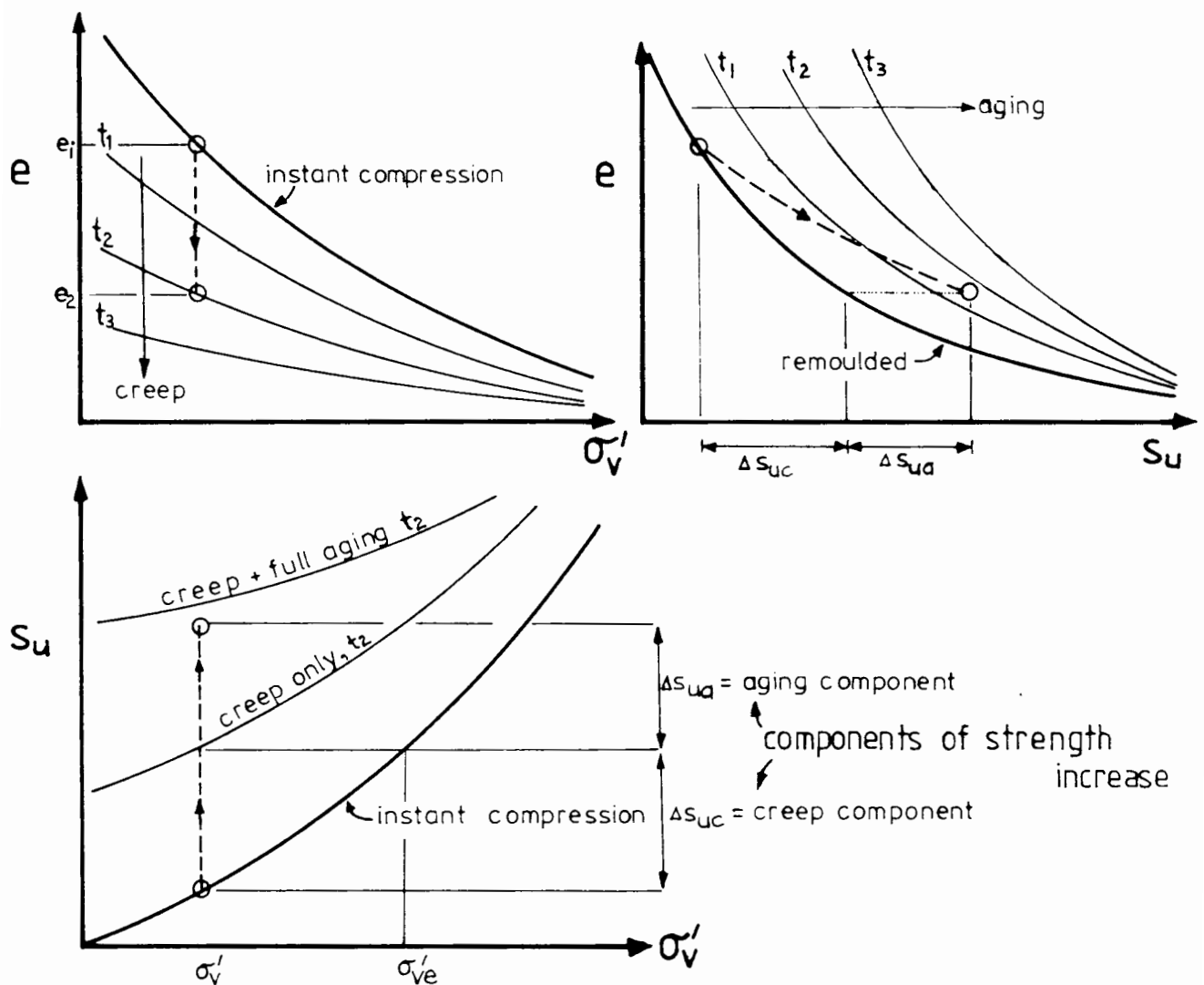


Figure 7.2 Components of strength increase in soil subject to creep compression and age strengthening.

instant compression curve. Soil elements will have new apparent preconsolidation stresses, σ'_{VC2} and σ'_{VC3} . The equivalent overconsolidation ratio is best defined as $\sigma'_{ve2}/\sigma'_{v2}$, since σ'_{VC2} can not be specified without separate testing.

Soil sedimented at a slow uniform rate undergoes less subsequent creep compression and consolidation paths remain closer to the instant compression curve. A stronger structure (possibly due to larger flocs) is able to develop at very high void ratios so that stress dependent compression dominates, even at void ratios which could only exist in a suspension in soil deposited rapidly. A relatively stable, low density structure can exist even after long periods of consolidation. On the basis of one experiment where soil was mixed in salt water (35 ppt) it appears that compression behaviour is similar in salt and fresh (3 ppt) water soils, although creep compression at higher void ratios appears to be of greater magnitude in salt water soils.

Allowing base drainage in some experiments does not greatly accelerate consolidation since a localised, rapid effective stress increase increases densities and decreases permeabilities.

7.1.3 Strength behaviour

Two distinct aspects of soil strength exist in a consolidating sediment. The first is the remoulded strength, s_{ur} , which is relevant immediately following vigorous mixing and deposition and is equal to the residual strength obtained from vane tests at large rotation angles. The second type is the peak strength, s_{up} , which increases from the remoulded or residual strength with aging and is probably caused by microstructural cementing. The peak strength is equal to the remoulded strength if no aging occurs and the two are approximately equal if compression is sufficiently rapid and stress dependent alone, although anisotropic effects probably account for the small amount of scatter in peak strength behaviour observed in this study. Good correlation is obtained between the remoulded strength and the liquidity index. Independence of this correlation from other experimental parameters shows that the type of

compression experienced, whether creep or stress dependent, does not influence the remoulded strength which depends on the volumetric state alone. Remoulded strength varies linearly with the equivalent instant compression stress σ_{ve}' in the range tested but the ratio $s_{ur}/\sigma_{ve}' = 0.03$ found in these tests is much lower than that for denser soils and indicates that further work is required to bridge the gap between very soft and stiffer soils.

The modes of strength increase with decreasing void ratio and with aging are shown schematically in figure 7.2. At a given void ratio the remoulded strength has a unique value but the peak strength varies between the remoulded strength (if compression has been very rapid) and a value attained only if all aging has occurred at the current void ratio. At a particular effective stress the strength may be considered to have three components. The first is due to the effective stress σ_v' and the second due to the increase in equivalent effective stress from σ_v' to σ_{ve}' due to creep compression; the sum of these two components is the remoulded strength. The third component is due to aging alone, and is equal to the difference between peak and remoulded strengths. Figure 7.2 shows how sensitivity is not a unique soil property and is not dependent on the total compression time alone, but instead depends on both the type and rate of compression, as well as aging.

Rate effects on strength in the soil states considered are similar to those in denser soils except for strengths derived from cavity expansion tests. The high rates of strain occurring with expansion from very small initial cavity diameters may cause the much greater strength changes observed with cavity volume expansion rate.

7.2 Implications and recommendations

Results discussed in the previous sections suggest that several cases should be treated separately when modelling soil settling and compression. If the modelled soil is deposited uniformly at void ratio below the critical transition value e_0^* so that a suspension phase does not exist, effective

stresses will be present immediately and upper regions of the soil will act as though overconsolidated. If the initial void ratio is greater than e_0^* and deposition is rapid, segregation may occur and the consolidating bed will not be homogeneous. If on the other hand deposition is very slow, the bed will form at a higher value of e_0^* , and subsequent compression may be smaller in magnitude and primarily stress dependent, so that allowance for creep compression in the model should be reduced. The revelation that considerable variations in e_0^* can occur under different conditions and with time is of significance to consolidation models where a unique compression relationship is often assumed, defining a constant value of e_0^* at zero effective stress. Results presented here suggest that the instant compression relationship suggested in chapter 5 for Combwich soil

$$e = 5.7 \sigma_v'^{-0.29}$$

might be valid at void ratios up to 15 or higher if deposition is sufficiently slow.

These results have some particular implications for standard consolidation tests. Any compression curve obtained for such a test will be relevant to the particular loading rate only and a better description of behaviour would be obtained from a series of tests at different loading or compression rates. Several advantages of the restricted flow consolidation test developed at Oxford, in which the flow rate is controlled using filters in the drainage line, are apparent. The slow drainage ensures that high pore pressure gradients do not occur through the sample even at high testing rates, so that gradients and flow velocities may remain close to those which occur in the field. The recommended series of tests may be carried out rapidly by using different combinations of the total applied stress and the flow restriction; each test may be completed in any period from less than an hour up to typical field consolidation times, and yields full compression data throughout the entire range of stress applied.

Some recommendations may be made regarding methods for accelerating rates of compression. The provision of base drainage may not be particularly useful since high density layers will form locally and restrict drainage. Surface surcharges should be applied as early as possible during consolidation to achieve maximum benefit since creep compression in a sediment which is not subject to a surcharge may eventually result in similar densities being attained to those due to a surcharge. Surcharges applied at later times will have to overcome the equivalent preconsolidation stresses resulting from creep compression and will therefore be less effective. Although flocculated suspensions will settle faster initially, results obtained here and by Been (1980) (and discussed in greater detail by Elder & Sills, 1985) suggest that once the consolidating phase has been reached, considerably higher densities are attained in a dispersed clay, at the expense of leaving the finer particles in the supernatant liquid. The addition of flocculants will result in stronger floc structures similar to those occurring in sediments deposited very slowly, and may be counter-productive for the consolidating phase by causing densities to remain low for long periods.

A relationship between strength and density, which considerations in chapter one suggested might be of use, has been obtained between the remoulded strength and liquidity index, but peak strength, more relevant to many field situations, is time dependent. Caution must therefore be exercised when using concepts such as the "nautical depth" (a depth within the suspension above whose altitude vessels can safely sail) defined as the depth at which the soil density is 1.2 Mg/m^3 , since at a constant density strength may increase with time alone. Use of a value related to the critical transition value, e_o^* , obtained for particular conditions of sedimentation, might be more appropriate, since at this void ratio effective stresses, and therefore strengths, are zero.

A final point of interest is that apparent overconsolidation behaviour has been observed in soils which have never experienced higher stresses and is attributed to the large amount of creep compression which can occur at high

void ratios. Creep compression (over much longer time periods) may also be the cause of apparent overconsolidation exhibited by marine sediments for which there is no geological evidence of previous loading and unloading.

7.3 Contributions of this thesis

The analysis of results in this study should lead to a much improved understanding of the mechanisms causing compression at very high void ratios. Consistent patterns of soil behaviour have been established and explained at void ratios up to 14 (liquidity indices up to 15), and account for the considerable variability in behaviour observed previously at very low stress levels. Results show that conventional geotechnical concepts such as effective stress and shear strength may be applied in soil states which are outside normal ranges of soils engineering data. Correlations established particularly between void ratio and effective stress for instant compression and between pore water/solids relative velocity and void ratio, together with the evidence for the major role of creep compression, may provide the basis for fundamental reformulations of consolidation/ compression theory at very high void ratios.

Modifications and improvements made to existing equipment and techniques and development of a new technique have allowed evaluation and validation of strength testing methods at liquidity indices up to 6. This range is considerably more extensive than any reported elsewhere and provides good overlap with ranges of test data from viscometer and erodibility tests. Data will be useful for future comparisons of these techniques.

Some major drawbacks in use of current laboratory testing procedures for very high void ratio soils have been identified, especially in consolidation and permeability tests where applied loadings and boundary conditions are often not relevant to conditions in the field.

This thesis represents a significant step towards bridging the gap between conventional soils engineering and sedimentology.

7.4 Recommendations for future work

The major aim for future work should be to obtain sufficient data, for a wide variety of depositional conditions, to enable formulation of a model which can describe the various aspects of behaviour reported in this study. Oedometer consolidation tests (or slurry consolidometer tests) at different loading rates will provide data for comparison with settling column and surcharged slurry tests, and should enable creep compression to be quantified as a function of void ratio, effective stress, time or other parameters. The model would be required to predict the different compression paths followed by different soil elements in the same soil mass, as observed in chapter 5 of this study. An extension of an earlier series of experiments begun by slow deposition of sediment should cover a range of rates and total soil masses in a consistent manner. A programme of 9 experiments might involve deposition of solids at 5 g/hr for 10, 100, 1000 hours, at 50 g/hr for 1, 10, 100 hours and at 500 g/hr for 0.1, 1 and 10 hour depositional periods. Some shear strength testing should be undertaken to confirm the suggestion made in chapter 5 that slow deposition rates allow significant strength development at high void ratios. Further experiments using soil sedimented in salt water will be required to confirm similarities and differences between fresh and saline environments, and should also include strength tests. Repetition of experiments in this study for other soils, preferably having different plasticity and activity (both higher and lower), will be of interest although Combwich soil is typical of many common clay soils and sediments, as shown in chapter 4.

Investigation of near surface properties and geotechnical behaviour of very thin sedimented beds will provide a further link between soils engineering and sedimentology. Mechanisms governing the variation of the surface void ratio, e_o^* , under different conditions, and subsequent decreases with time, will provide important input for a soil compression model and may involve microstructural examination. Accurate pore pressure measurements and improved

long term measurement reliability will be required in such a study, and horizontal stress measurement will also be useful for evaluation of anisotropy, both in thin and deeper beds.

A programme of field measurements has already begun and equipment has been designed and constructed for installation in a large settling pond. Four total stress pads are to be placed on the bed of the pond following a programme of dredging, two measuring vertical total stress and two measuring horizontal total stress. These and two pore pressure sensors are to be placed symmetrically at a 1 m radius around a central base and similar measurement of the hydrostatic pressure using a flexible tube floating in the pond will allow calculation of excess stresses above hydrostatic. Regular density profiles are to be taken using a γ -ray backscatter densimeter constructed at Oxford.

The final area for future work lies in further evaluation of strength testing methods. The recommendation has been made already that the fall cone standard time of fall should be reduced to about 0.5 seconds to minimise effects of subsequent creep or consolidation. Further experimental investigation of the cone test is required to explain discrepancies between observed strengths and those predicted theoretically and might include measurement of surface adhesion and the cone motion with time. A series of vane tests which attain peak resistances in around 0.1 seconds, equivalent to cone penetration times, will be useful. The spherical cavity expansion technique requires considerable further development to explain some of the anomalous behaviour observed in this study. An appropriate investigation might involve isotropic consolidation of soil symmetrically around a fluid filled membrane of constant volume and radius not less than 10 mm, to avoid the high strain rates occurring at very small radii. Expansions at constant rate of volume increase and constant rate of increase of radial strain should be compared and measurement of pore pressures at different radii from the cavity would assist interpretation of expansion data and allow full consolidation analysis during the period after completion of each expansion.

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Geotech - Geotechnique.

ASTM - American Society for Testing and Materials.

ICSMFE - International Conference for Soil Mechanics and Foundation Engineering.

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