A Laboratory Study of the Marchetti Dilatometer

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SUMMARY

The Marchetti dilatometer is an in-situ testing device the results of which have been interpreted chiefly through empirical correlations based on the results of field tests. The interpretation has mainly involved the use of two readings, P_0 and P_1, though recently a third reading P_2, has been introduced. Dilatometer tests were carried out in nine clay beds. The stress history and stress state of each clay bed was systematically varied in the test programme to allow their individual effects on the dilatometer readings to be assessed.

The study revealed that in clay the dilatometer P_0 reading is controlled by the undrained strength and the horizontal stress and was independent of the degree of overconsolidation. Through use of this result it was shown that the applicability of many of the empirical correlations presently used to evaluate the dilatometer readings was restricted to deposits with one-dimensional stress histories. It was also found that $P_0 - P_2$ was dependent on the undrained strength and that the P_2 reading was close to the total horizontal stress of the sample though this last result does not appear to be supported by the few field results that are available. The results have highlighted the redundancy of the P_1 reading for property evaluation. It was found that a timed sequence of P_0, P_1 and P_2 readings could be used to detect the variation of the consolidation properties between clay beds.

In addition to the tests in clay, the results of 31 dilatometer tests carried out in a calibration chamber for sand have also been analysed. An inter-relationship between P_0, P_1 and P_D was found suggesting that they were all measuring the same soil response; that of the pressure required to open a cavity in the sand. The P_0 reading was shown to be dependent on the horizontal stress and the state parameter, though this dependence could not be exploited to back calculate the horizontal stress. Overconsolidation of the sand specimens had no significant influence on the readings.

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List of Symbols

a	Effective radius of probe
a_D	Cross-sectional area of dilatometer
A	First Dilatometer reading
A'	Corrected A reading
A_s	Cross-sectional area of sample
B	Second Dilatometer reading
B'	Corrected B reading
$\underline{B}$	Pore pressure parameter
c_h	Horizontal coefficient of consolidation
c_v	Vertical coefficient of consolidation
c_{vs}	Vertical coefficient of swelling
C	Third dilatometer reading
C'	Corrected C reading
D	State parameter
d_v	Diameter of vane
e	Void ratio
e_{cs}	Reference void ratio for critical state line
e_{is}	Reference void ratio for isotropic consolidation line
$e_{\min}$	Minimum void ratio
$e_{\max}$	Maximum void ratio
$e_{1\text{-}d}$	Reference void ratio for one-dimensional compression
E	Young's modulus
E_D	Dilatometer modulus
EOCR	Extended overconsolidation ratio
G	Soil shear modulus
G_s	Specific gravity of solids
G_{ur}	Unload-reload shear modulus
h_v	Height of vane
i	Hydraulic gradient
I_D	Dilatometer material index
I_r	Rigidity index of the soil = G/s_u
k	Permeability
k_v	Vertical permeability
K	Stress ratio
K_d	Dilatometer prediction of stress ratio
K_D	Dilatometer horizontal stress index
K_s	Undrained bulk modulus

K_0	Stress ratio at rest
K_{0nc}	Stress ratio during one-dimensional normal compression
m	Exponent in Eqn 2.19
m_v	Coefficient of volume compressibility
M	Constrained modulus
M_θ	Rotational stiffness constant
N_D	Dilatometer factor
OCR	Overconsolidation ratio in terms vertical stress
p	Mean stress
p'	Mean effective stress
p'_c	Mean effective stress at the intersection of the current yield locus with the p' axis
$p'_{\max}$	Maximum mean stress
p'_T	Mean effective stress at testing
p'_{1-d}	Mean effective stress during one-dimensional consolidation
P_D	Dilatometer thrust
P_i	Cone-pressuremeter lift-off pressure
P_L	Cone-pressuremeter limit pressure
P_r	Perimeter ratio
P_0	Corrected A' reading
P_1	Corrected B' reading
P_2	Corrected C' reading
q	Deviator stress
q_c	End bearing capacity of cone
q_D	End bearing capacity of dilatometer
Q	A constant depending on the compressibility and mineralogy of the sand
r	Radius to point of interest
r_a	Radius of loaded area
R	Overconsolidation ratio in terms of mean stress
R_d	Relative density
R_D	Effective radius of dilatometer
R_{ep}	Radius of the elastic-plastic boundary
R_M	Marchetti's modulus factor
s	Shear stress
s'	Mean stress = $(\sigma'_1 + \sigma'_2)/2$
s_r	Length of rod embedded in clay
s_u	Undrained strength

t	Time
t_b	Thickness of vane blade
t_{50}	Time for 50% reduction in $P_2 - u_o$
t_f	Time to failure
t_r	Diameter of vane rod
t_{flex}	Time to point of contraflexure
T	Time factor
T^*	Time factor incorporating rigidity index
T_q	Torque
u_0	Initial water pressure
U	Degree of consolidation
V	Volume of mesh
w	Water content
$w(r)$	Movement normal to the surface at radius r
Z_M	Control unit reading when system is vented
α	Proportion of strength mobilised around rod
γ	Bulk unit weight of soil
γ_w	Unit weight of water
γ_{oct}	Octahedral shear strain
ΔA	Stiffness correction to A
ΔB	Stiffness correction to B
Δe	Change in void ratio
$\Delta e_{q/p}$	Change reference void ratio associated with change in the stress ratio
Δp	Change in mean total stress
$\Delta p'$	Change in mean effective stress
ΔV	Change in the volume of the mesh
ε_{ij}	Strain
ε_r	Radial strain
ε_v	Volumetric strain
ε_z	Vertical strain
η	Stress ratio
θ	Angle of rotation
κ	Slope of unloading line in e: lnp' plane
λ	Slope of normal compression line in e: lnp' plane
Λ	Plastic volumetric strain ratio
μ	Plastic deformation parameter
M	Critical state strength parameter
ν	Poisson's ratio of soil

σ_H	Total horizontal stress
σ'_H	Horizontal effective stress
σ_i	Principal stress
σ_V	Total vertical stress
σ'_V	Vertical effective stress
ϕ	Friction angle
ϕ_{ps}	Plane strain friction angle
ϕ_{cs}	Critical state friction angle
ψ	State parameter

CHAPTER 1

INTRODUCTION

1.1 IN-SITU TESTING

In contrast to most other areas of engineering, the solution of geotechnical engineering problems generally requires a subsurface exploration programme to obtain the design parameters. The quality and relevance of the soil parameters used in the analysis heavily influence the overall accuracy of any predictions made. This, together with the considerable variation in geological conditions and design and construction problems, has encouraged the continuous development and use of a number of laboratory and in-situ soil tests. Traditionally in Britain, more than in the rest of Europe, great emphasis has been placed on the fundamental solution of geotechnical problems through the use of the results from laboratory tests. This is partly due to the type of soil deposits generally found in Britain, which permit relatively straightforward sampling but which are highly variable, and partly due to a peculiarity in the development of the geotechnical engineering profession here.

In the rest of Europe, in-situ testing methods have played an important role for many years; in Holland the cone penetration test, in France the Menard pressuremeter and in Scandinavian countries the weight penetrometer, are of central importance in the analysis of many geotechnical problems. The readings taken with these devices are analysed using empirical correlations; an approach which is alien to the British tradition and which is less attractive in the variable deposits found here than, for example, in the uniform clay deposits of Holland. For this reason, and the general conservatism of the British engineering profession, the adoption of new in-situ devices here has been a slow process.

In general the tests, whether laboratory or in-situ, promoted as the ones best suited to obtain

particular pieces of geotechnical information, have changed over time as their individual limitations and strengths have become apparent. New tests have appeared in response to the desire to overcome the perceived weaknesses in existing tests. A recognition of the difficulties associated with laboratory tests, such as sample disturbance and the proper simulation of the in-situ stress system, has led to increased interest in in-situ devices. Laboratory tests have many advantages, such as the ability to measure local small strain behaviour and to subject specimens to elaborate stress paths with control over the drainage conditions; but the requirements of the analyses employed to tackle large modern geotechnical projects i.e. the soil conditions and the particular parameters that are sought, now favour the increased use of in-situ tests. In-situ testing techniques are able to test large volumes of soil cost effectively and avoid some of the problems inherent in laboratory testing. They are able to provide complete profiles of soil properties and accurate local determinations.

The general approach used in site investigations is to measure the site variation of pertinent properties through use of a profiling type test, with high quality testing at a few locations of interest using either laboratory tests on samples or one of the more sophisticated in-situ techniques. In-situ devices can broadly be divided into these two basic groups: profiling tools and direct tests.

Profiling tools are the quick, economical, penetration-type tests used primarily for stratigraphic logging during the exploratory phase of a site investigation, but which can sometimes provide approximate values for various geotechnical parameters. These methods measure quantities from which it is difficult to estimate soil parameters directly, and it is therefore necessary to resort to empirical correlations for their interpretation. An example of this type of tool is the cone penetrometer. Direct tests aim to preserve in-situ conditions and test the soil in a way amenable to theoretical analysis. They are used to provide accurate assessments of specific soil parameters in areas identified by the profiling methods as being of special interest. The direct tests are usually more specialised and are consequently more

expensive to perform than the profiling tests. The self-boring pressuremeter is an example of this type of test.

Of the in-situ testing devices presently available, the Marchetti dilatometer is a relatively recent addition. It works using simple mechanical-electrical principles in contrast to many of the newer devices which use electronic sensors. It has been regarded as a compromise between the traditional profiling tools and the more sophisticated direct tests, because it attempts to minimise insertion disturbance through its blade-like shape and then measure a soil pressure and modulus, which some of its proponents claim are relevant to the in-situ horizontal stress and soil modulus. The evaluation of the results is empirical, but unlike many other tests which rely on this form of interpretation it produces extremely reproducible readings with the minimum of standardisation of test procedures. It is used essentially for stratigraphic profile determination but it has also been shown to provide good estimates for many soil parameters, and in particular it has been advanced as a test for the measurement of the horizontal stress; a parameter for which a quick and reliable method of determination has yet to be found. At present only the slow and expensive self-boring pressuremeter has been recognised as a device which can, in principle, measure the in-situ horizontal stress. Even so, uncertainties accompany its use in sands. According to Fahey & Randolph (1984) even the small disturbance caused to interlocking sand grains by the entry of a self-boring pressuremeter would be sufficient to significantly alter the horizontal stress.

A knowledge of the magnitude of the horizontal stress is vital for the satisfactory solution of many geotechnical problems. It is needed in the design of the support systems in excavations and the prediction of their lateral deformations. The calculation of the design capacity of friction piles requires a knowledge of the horizontal stress. It is necessary in the use of yield envelopes which demand a complete knowledge of the initial stress. Constitutive models also use the mean stress level to control the magnitude of soil moduli. In other in-situ tests the horizontal stress exerts a controlling influence on the interpretation, e.g. in the cone

penetration test in sands q_c depends on the friction angle and the effective horizontal stress. The measurement of relevant soil parameters in laboratory tests requires the proper simulation of the in-situ stress conditions e.g. in triaxial stress path testing.

The vertical effective stress can be determined with relative accuracy and it is useful to express the horizontal effective stress as a proportion of it as in Equation. 1.1.

$$K = \frac{\sigma'_H}{\sigma'_V} \qquad \text{(Eqn 1.1)}$$

Where

σ'_H = Effective horizontal stress,

σ'_V = Effective vertical stress,and

K = Stress ratio.

K_0, the stress ratio at rest, is relevant for soils which have been normally consolidated or overconsolidated one-dimensionally. As discussed in Chapter 5, relatively new deposits which have experienced simple stress histories have K_0 profiles which can be predicted roughly from simple empirical equations. However, as a consequence of soil aging and weathering it is usual for deposits, including those which have experienced simple stress histories, to have stress ratio profiles which differ to some extent from the predictable. Site measurements of the stress ratio therefore provide useful information about the geological condition of a deposit as well as an important parameter in itself.

Laboratory methods for the measurement of the stress ratio exist, but they are hindered by

the sampling process, as they rely on the sample being obtained and tested without any change of effective stress and with minimal disturbance, Benoit & Lutenegger (1992). In-situ methods require the insertion of a device which will alter to some degree the existing horizontal stress, though the intention with the self-boring pressuremeter is to reduce this disturbance to zero. In contrast the Marchetti dilatometer has the aim of producing a consistent and repeatable level of disturbance, which can be empirically accounted for in the interpretation. The empirical charts upon which the interpretation of the dilatometer readings is based, have proved to be moderately successful, except in the analysis of tests conducted in deposits outside the geological regions used to construct the correlations.

It is in the nature of empirical correlations that when the connection between in-situ measurement and in-situ property is poorly understood, the applicability of a correlation to a particular soil can only be judged on the basis of precedent. For example a soil which is known to be sensitive with regards to strength, compared to those used to build the correlation, possesses an important difference in respect to interpretation only if the in-situ reading is influenced by the soil strength. A knowledge of the primary factors which affect the readings (e.g. strength, horizontal stress, etc.) is therefore of great value in assessing the importance of secondary factors (e.g. cementation, overconsolidation, etc.) which generally act to modify the primary factors.

Much effort has been expended on widening the range of soils field tested with the Marchetti dilatometer. This has been done without an understanding of the parameters which are important to the dilatometer readings, and in consequence the results from various deposits have merely added to the scatter of data upon which the original correlations are based. In some cases new correlations have been proposed. With a better understanding it may have been possible to explain the apparently inconsistent results of some of these tests in terms of the idiosyncrasies of the particular soils.

Recently, improved theoretical understanding of the dilatometer in clay, Yu et al. (1992), Finno (1993), has revealed the sensitivity of the readings to variations in particular soil parameters. The horizontal stress has been cited in both studies as being of prime importance; though the complex dilatometer geometry has necessitated the employment of a number of simplifications in each study, the significance of which is difficult to gauge. Research using instrumented dilatometers by Lutenegger (1988) and Campanella & Robertson (1991), has also provided important insights into sand and clay behaviour during penetration and membrane expansion. The dilatometer has been tested extensively in calibration chambers for sand by Bellotti et al. (1979), Bellotti et al. (1982) and Baldi et al. (1986); and their results have proved to be particularly useful.

The objective of this research project was to acquire a better understanding of the relationship between the Marchetti dilatometer readings and soil parameters, concentrating in particular on the effect of the horizontal stress. A laboratory study was thought to be the most appropriate way to achieve this because the accuracy of field tests (and consequently the empirical charts presently used to interpret the dilatometer readings) is restricted by the precision with which the soil properties and in-situ stresses are known. The quality of the empirical interpretation of results in natural deposits is also impaired by the indeterminate effects of secondary soil properties such as mineralogy, cementation, sensitivity, aging, anisotropy and fabric which can influence the soil parameters strength, stiffness etc. measured in the reference tests but may or may not influence the readings taken with the dilatometer.

These problems can be overcome by using a laboratory calibration chamber wherein soil specimens can be prepared and tested with a high degree of control. Calibration chambers also have the advantage over field tests that they allow the independent examination of the effects of individual soil parameters. This is important because of the strong interdependence of soil parameters in the field (e.g. the link between s_u, OCR and K_0 in clays) which can conceal the parameter actually affecting the readings taken.

A set of results for dilatometer tests conducted in the Oxford University calibration chamber for sand by Mr R. Hitchman under the supervision of Prof. G.T.Houlsby, was available at the start of the present laboratory study of the Marchetti dilatometer. These results are presented and analysed in Chapter 6. The remainder of the project was concerned with the design and construction of a calibration chamber for clay and its use for the testing of the Marchetti dilatometer. The philosophy behind the preparation and testing of the clay beds is the subject of the next section.

1.2 CALIBRATION CHAMBER FOR CLAY - TESTING PHILOSOPHY

Although the properties of a natural soil deposit vary with its age and fabric, it is primarily the strength, stress level and stress history which determine the soil's behaviour. It is chiefly relationships between these parameters and the soil's physical behaviour which provide the most successful basis of understanding. The use of laboratory soils reduces the variables to these principal parameters, removing the unmeasurable influence of minor properties and thereby improving comparison with the measured soil behaviour.

The long time needed for sample preparation restricted the number of tests that could be conducted. The stress states and histories were chosen to extract as much information as possible from the limited number of tests that were feasible. The philosophy behind the approach has been described by Houlsby (1988): in the standard dilatometer test two measurements are taken, namely the P_0 and P_1 readings. The P_0 reading could broadly be expected to be related to the horizontal stress and the difference, P_1-P_0, to the soil stiffness. As both of these parameters are affected by overconsolidation it was surmised that the dilatometer might have the potential to estimate the *OCR* of soil deposits. Houlsby listed three basic differences between normally consolidated and overconsolidated elements of soil at the same level in the ground:

- The mean stress level
- The stress ratio
- The maximum past mean stress level

The test programme was devised therefore to examine these three quantities. By taking the clay sample through the stress histories given in Table 1.1, and illustrated in Figure 1.1, the effect of the three variables listed above could be investigated separately. It was originally intended that Test 9 should follow the overconsolidated path OEFE (Figure 1.1) but towards the end of the test programme it became apparent that a test at a third mean stress level was required (originally it had been planned to test at two mean stress levels only). As there was only time for nine tests in total, Test OEFE was replaced by test OCDX. This was preferred to a test which involved a cycle of overconsolidation on the $K = 2.0$ line but with a greater amount of unloading than OEFE, because it would then only differ in stress level from Test OCDC and would therefore permit a comparison of stress level effects.

Tests 1 & 3, and 4 & 6, examine the effect of a cycle of overconsolidation on the measured soil response at two different stress ratios.

Tests 1 & 2; 3 & 9; 4 & 5 and 7 & 8, examine the effect of stress level at three different stress ratios.

Tests 1, 4 & 7 and 2, 5 & 8, examine the effect of stress ratio at two different stress levels.

The stress paths the samples follow between stress states are not the straight ones suggested in Figure 1.1. Section 3.5 discusses the actual stress paths experienced by the clay beds during their preparation.

The analysis of the dilatometer readings has involved their comparison with a number of soil

parameters: horizontal stress, vertical stress, mean stress, stress ratio, overconsolidation (in terms of mean stress), undrained strength, rigidity index and the coefficient of consolidation. Standard plots relating the dilatometer readings to some of the above parameters have already been widely adopted. At the beginning of this study it was hoped that it would be possible to develop other, more theoretically sound, relationships, that would mitigate some of the dilatometer's present failings.

The following chapters describe the Marchetti dilatometer, the current interpretation of the readings and the present state of understanding (Chapter 2); the design and construction of the calibration chamber for clay (Chapter 3); the supplementary tests conducted in the clay bed to establish its properties (Chapter 4); the analysis and interpretation of the dilatometer tests in clay (Chapter 5); and the analysis and interpretation of the dilatometer tests in sand (Chapter 6). In Chapter 7 the main conclusions are summarized.

CHAPTER 2

THE MARCHETTI DILATOMETER

2.1 APPARATUS

Excluding the equipment needed to insert the blade, the apparatus comprises five main items, (Figure 2.1):

- Dilatometer blade
- Control unit with a pressure readout system
- Pneumatic-electrical cable
- Calibration device
- Pressure source

The dilatometer consists of a stainless steel blade with a 16° angled tip and a thin flat circular expandable steel membrane on one side. The membrane is centrally positioned and flush with the face of the blade. The dimensions and geometry of the blade are shown in Figure 2.2.

A pneumatic-electrical cable transmits gas pressure and provides electrical continuity between the control unit and the blade. The standard control unit is fitted with a 40 bar (4000 kPa) capacity manually read Bourdon gauge which is suitable for most soils.

Figure 2.3 shows a cross-section through the dilatometer blade and membrane. The instrument works using a simple mechanical-electrical system. Using a micrometer valve on the control unit, gas pressure is slowly increased in the blade, causing the membrane to deflect outwards. The gas pressure is recorded manually at two preset points during the membrane expansion: when the membrane and the feeler lose contact, and when a deflection

of 1.1 mm is reached. The movement of the membrane off the feeler breaks an electrical circuit which stops an audio signal on the control unit. The pressure at this point is termed the "lift off" pressure. At a deflection of 1.1 mm the spring loaded steel cylinder makes contact with the overlying sensing disc and the electrical circuit is re-established reactivating the audio signal.

Pressure corrections for membrane stiffness are found using the calibration device (pressure gauge and syringe activated vacuum-pressure source). CPT driving rods can be used to transfer the thrust from the insertion equipment and to carry the pneumatic-electrical cable from the control unit to the dilatometer blade. A ground cable provides electrical continuity between the pushrod system and the control unit.

2.2 DEVELOPMENT OF THE MARCHETTI DILATOMETER

The dilatometer was initially developed by Professor Silvano Marchetti in 1974, as a device to evaluate a post-installation undrained soil modulus, E, in an investigation of laterally loaded driven piles. The dilatometer subjects the soil to a stress path similar to that induced by driven piles, and it was therefore expected that there would exist a close correlation between E and the dilatometer modulus E_D (Section 2.4.2). The configuration of the device was chosen to satisfy the following requirements, Schmertmann (1983):

- It had to be capable of being used to depths appropriate to the design of piles so a penetrometer type test was chosen. This also gave it an advantage of speed and an ability to take near continuous profiles of readings.

- It had to be robust enough to withstand driving forces but flexible enough to detect the influence of soil deformability. It was concluded that two separate members were necessary.

The first one to carry driving forces, and the second to engage the soil in some movement to test its stiffness. The blade was constructed out of solid stainless steel, and the dimensions chosen to make the body as structurally rugged as possible.

- It had to duplicate closely the load sequence induced in the soil by driven piles subsequently subjected to lateral loads. A circular membrane was chosen (for ease of mechanical construction and test interpretation), which could be expanded horizontally after the penetration stage. In order to keep the stresses in the nearly elastic range, the membrane displacement required to measure the soil deformability was kept small.

- It had to be simple to construct and reliable. A non-electronic system was chosen to keep the cost down and allow simple operation and field maintenance.

Problems with the independent determination of a unique post-installation soil modulus led Marchetti to abandon his initial aims and investigate correlations with more verifiable soil properties. The blade geometry was revised to streamline its shape and give it a sharper curved cutting edge in 1977 and it has essentially retained this shape up to the present time.

2.3 CONDUCTING THE TEST

Schmertmann (1986a), gives a detailed suggested procedure for performing the dilatometer test. Only a summary of the main recommendations is given here.

After the dilatometer blade is pushed vertically into the ground to the desired test depth any static load on the rods is removed and the membrane is expanded into the soil using gas pressure. The electrical buzzer on the control unit sounds until the membrane has just lifted off the plane of the blade. The operator records this lift off pressure and continues to expand

the membrane. When a central deflection of 1.1 mm is reached the buzzer is reactivated and the pressure is again noted by the operator. The two pressures are the A and B readings respectively. The operator vents the air pressure and the blade is advanced to another depth and the test repeated. A test depth increment of 0.2 m and a penetration rate of 20 mm/s are typically used in the dilatometer test.

The first reading should be obtained within 15 to 30 seconds of reaching the test depth. The rate of increase in gas pressure should be such that the second reading is obtained within 15 to 30 seconds of the first. The dilatometer records total pressures, so when testing in relatively high permeability deposits such as silt or silty fine sand, where significant pore pressures can be generated during penetration, a consistent membrane strain-rate is important.

Thus one dilatometer sounding furnishes two profiles; one of the A pressure reading and another of the B pressure reading. The A and B readings are corrected for membrane stiffness and the offset in the measuring gauge, Z_M, to give the pressures P_0 and P_1 respectively. (ΔA and ΔB membrane stiffness corrections should be determined before and after each sounding).

Recently a modification to the test procedure has been proposed, in which controlled deflation of the membrane is carried out. A record is made of the pressure when the membrane re-establishes contact with its seating. This pressure has been termed the C reading.

Continuous measurement of the end thrust necessary to advance the blade is presently recommended for the determination of some geotechnical parameters for sands and silty sands, and has been found to be useful for stratigraphic profiling in all soils.

2.4 DILATOMETER INDICES

The aim of the dilatometer test is to produce as low a degree of penetration disturbance as practicable and then correlate the membrane measurements to the in-situ soil parameters and properties existing prior to the insertion of the blade. Usually this has been done by use of the dilatometer indices. The readings A and B are first corrected for membrane stiffness to give the soil contact pressures P_0 and P_1, at the start and end of expansion. At each test level the effective vertical stress σ_V', the pore water pressure u_0, prior to insertion of the dilatometer, and the P_0 and P_1 pressures, are combined to give the three dilatometer indices K_D, I_D and E_D.

2.4.1 Correction of Control Unit Readings

Membrane stiffness corrections, ΔA and ΔB, are found before and after each sounding. ΔA is equal to the external pressure which must be applied to the membrane to overcome its natural outward deflection and force it into contact with its seating. ΔB is the internal pressure which, in air, gives a membrane displacement of 1.1 mm. These are determined using the vacuum pressure calibration device. Typically ΔA is about 10 kPa and ΔB is about 60 kPa. A serious drawback reported by Lacasse & Lunne (1982a) is that these values can represent a significant proportion of the readings in soft clays at shallow depth.

The control unit readings, A and B, are corrected using this calibration information and the offset in the gauge zero, Z_M. The soil contact pressure at 1.1 mm membrane displacement P_1, is given by:

$$P_1 = B - Z_M - \Delta B \qquad \text{(Eqn 2.1)}$$

The function of the feeler, Figure 2.3, is to improve the definition of the instant at which the electrical circuit is interrupted. Marchetti & Crapps (1981) give the nominal elevation of the feeler as 0.05 mm above the sensing disc and this must be accounted for when evaluating P_0. The soil contact pressure at zero membrane displacement is found from the A and B readings using the linear extrapolation illustrated in Figure 2.4. P_0 is then given by:

$$P_0 = \frac{1.1}{1.05}(A - Z_M + \Delta A) - \frac{0.05}{1.05}(B - Z_M - \Delta B) \qquad \text{(Eqn 2.2)}$$

where

A = First dilatometer reading

B = Second dilatometer reading

ΔA = Stiffness correction to A

ΔB = Stiffness correction to B

Z_M = Control unit reading when system is vented

P_0 = Corrected first reading

The extrapolation used in the determination of P_0, Figure 2.4, assumes a linear stress strain response in the soil during expansion of the membrane. In most soils, because of the low strains involved, this appears to be a reasonable approximation. For soils that exhibit a marked non-linear response the extrapolation can sometimes lead to a negative P_0 reading. There is ambiguity in the literature about the precise displacements the P_0 and P_1 readings refer to. The extrapolation detailed above is rarely mentioned as it is embedded in a computer programme which is often used to reduce the dilatometer data.

By analogy with Equation 2.2 the C reading is corrected to give the P_2 pressure using the

following:

$$P_2 = \frac{1.1}{1.05}(C - Z_M + \Delta A) - \frac{0.05}{1.05}(B - Z_M - \Delta B) \qquad \text{(Eqn 2.3)}$$

The equations used to determine P_0, P_1 and P_2 for the particular dilatometer used in this study are discussed in Chapter 5.

2.4.2 Dilatometer Modulus, E_D

Having determined P_0 and P_1, the difference can be used to find a modulus of elasticity for the soil. If the space surrounding the dilatometer is assumed to be formed by two elastic half spaces in contact along the plane of symmetry of the blade, the membrane expansion can be modelled as the loading of a circular area subject to the condition of zero displacement external to this area. Equation 2.4 shows the mathematical solution to this problem:

$$w(r) = \frac{4}{\pi}\left(\frac{1-\nu^2}{E}\right)(P_1 - P_0)\, r_a \sqrt{1 - \left(\frac{r}{r_a}\right)^2} \qquad \text{(Eqn 2.4)}$$

where:

$w(r)$ = movement normal to the surface at radius r

r = radius to point of interest

r_a = radius of loaded area

E = Young's modulus of soil

ν = Poisson's ratio of soil

The ratio $E/(1-\nu^2)$ is defined as the dilatometer modulus E_D. For r_a = 30 mm, r = 0 mm

and $w(0)$ = 1.1 mm, Equation 2.4 becomes:

$$E_D = 34.7\ (P_1 - P_0) \qquad \text{(Eqn 2.5)}$$

E_D is drained in sands, undrained in clays and partially drained in silts and sand-clay mixtures.

The theory assumes that the stress distribution applied by the membrane to the soil is uniform. This may be a reasonable assumption given the high flexibility of the membrane compared with any soil. In any case the effect of the differences between theory and reality is probably, to an unknown degree, taken into account in the empirical nature of the correlations between E_D and soil parameters. The dilatometer modulus forms the basis for evaluating the in-situ one-dimensional drained modulus.

2.4.3 Material Index, I_D

The material index is the ratio of a modulus $(P_1 - P_0)$, to a measure of the horizontal effective confining stress, $(P_0 - u_0)$.

$$I_D = \frac{P_1 - P_0}{P_0 - u_0} \qquad \text{(Eqn 2.6)}$$

The index is used predominantly as an approximate indication of soil type and as a qualifier for limiting the applicability of empirical correlations.

2.4.4 Horizontal Stress Index, K_D

The horizontal stress index is analogous to the coefficient of earth pressure at rest, K_0. It is the ratio of a measure of the lateral earth pressure to the vertical effective stress.

$$K_D = \frac{P_0 - u_0}{\sigma_V'} \qquad \text{(Eqn 2.7)}$$

It increases in proportion to the existing lateral stress but it is also sensitive to other material properties. Marchetti (1980) argues that, when inserted, the wedge shape of the dilatometer produces uniform strain in a large volume of soil facing the blade. The value of P_0 is thus influenced not only by the disturbed soil near the blade but also by the soil properties outside this zone. Soil overconsolidation, aging and cementation have been found to increase K_D. As well as in the calculation of K_0, it is used by Marchetti (1980) in the calculation of the overconsolidation ratio and the undrained strength.

2.5 INTERPRETATION OF RESULTS

2.5.1 Introduction

Marchetti (1980) presented a set of empirical correlations between the indices defined in Section 2.4 and the following soil parameters: coefficient of earth pressure at rest, K_0, overconsolidation ratio *OCR*, constrained modulus *M*, and shear strength s_u. An indication of the soil type and soil density were also given. These initial correlations were based on laboratory data from ten well documented sites in Italy.

A number of researchers, in particular Jamiolkowski et al. (1985), Powell & Uglow (1988a), Lacasse & Lunne (1988) and Larsson & Eskilson (1989), have carried out detailed studies of Marchetti's correlations. Their experience has shown that the correlations provide a good indication of soil type and reasonable values of s_u, K_0 and OCR for soft to medium, uncemented, insensitive clays. It has, however, in a number of cases, proved difficult to extrapolate to soil types and regions outside those used to establish the initial correlations.

Only two sand sites were used in the construction of the original correlations, and it is in general the correlations that relate to sands that have performed poorly, and which have been subject to the greatest revision. The following sections attempt to summarise the development, and in some cases replacement, of Marchetti's original correlations, and give details of new correlations with soil properties not covered in Marchetti (1980).

2.5.2 Soil Identification and Density

Marchetti's (1980) data indicated that the Material Index I_D, was related to the dominant grain size fraction. Marchetti & Crapps (1981) produced the chart shown in Figure 2.5 relating a material description to the I_D and E_D values.

Campanella & Robertson (1983), Davidson & Boghrat (1983a) and Jamiolkowski et al. (1985) report reasonably accurate predictions of soil type using the chart with identifications moderately on the fine side.

Powell & Uglow (1988b), in dilatometer tests in a variety of UK clays found that use of the chart produced mixed success in identifying soil types and their densities. The dilatometer was able to detect changes in soil type and unit weight but the precise values were inaccurate. They attributed this to the very high overconsolidation ratios and the old age of some of the

deposits. It has been noted by Bechai et al. (1986) that the drainage paths available in fissured clays leads to them be identified by the dilatometer as silts. Some of the clays investigated by Powell & Uglow were fissured and this could account for their poor results.

2.5.3 Horizontal Stress

The dilatometer is most often referred to as a device for the measurement of K_0. Marchetti (1980) uses the dilatometer horizontal stress index, K_D, to evaluate K_0. His correlation (Equation 2.8) and field results are shown in Figure 2.6. The correlation is based on predominately uncemented clays. It is not relevant for clays which have experienced aging and cementation, for which unrealistically high K_0 values are predicted.

$$K_0 = \left(\frac{K_D}{1.5}\right)^{0.47} - 0.6 \qquad \text{(Eqn 2.8)}$$

2.5.3.1 Clays

The findings of Lacasse & Lunne (1982a) and Campanella & Robertson (1983) generally give support to the use of Equation 2.8. However Jamiolkowski et al. (1988) report that the correlation tends to overestimate K_0 in very stiff clays (I_D < 1.2) and they recommend that use of the correlation should be restricted to soft, and medium to stiff uncemented clays. Also Powell & Uglow (1986) found that this correlation performed badly in highly overconsolidated clays. A number of researchers have suggested revisions of the Marchetti (1980) correlation. Their modifications are shown in Figure 2.7

Lacasse & Lunne (1988) carried out dilatometer tests at a number of sites in Norway and the UK and one site in Brazil. They indicate a range of possible K_0 values for a particular K_D

reading. They associate the upper extent of the range for a particular value of K_D, with high plasticity clays and the lower extent with low plasticity clays.

Powell & Uglow (1988b) evaluated the performance of the dilatometer at a number of well-documented UK test sites ranging from soft normally consolidated clays to heavily overconsolidated stiff clays. They found that there was a distinct difference between the results of tests in 'young' clays (≤ 70,000 years) and 'old' clays (≥ 60 million years). Their correlation for the estimation of K_0 in 'young' UK clays is given by Equation 2.9.

$$K_0 = 0.34\ K_D^{0.55} \qquad \text{(Eqn 2.9)}$$

Equation 2.9 is not relevant for cemented and fissured clays which have substantially higher K_0 values than use of this equation would predict.

Larsson & Eskilson (1989) carried out a detailed study of the evaluation of the dilatometer readings in Swedish soils. Their correlation, Equation 2.10, is only recommended for the evaluation of K_0 in Scandinavian soils.

$$K_0 = 0.24\ K_D^{0.84} \qquad \text{(Eqn 2.10)}$$

In all these suggested modifications the basic premise that there is a direct relationship between K_0 and K_D, has not been challenged.

2.5.3.2 Sands

Marchetti (1980) speculated on the basis of a limited number of results, that his correlation (Equation 2.8) would also be applicable to cohesionless deposits. Calibration chamber tests and field experience soon indicated that there was a need to take some account of the density in any correlation as Equation 2.8 was found to overestimate K_0 in dense sands and underestimate it in loose sands.

Schmertmann (1983) used the ratio q_c / σ'_V, where q_c is the end bearing stress of the cone penetrometer, to provide additional information about the density and thereby make allowance for its effect. A CPT sounding parallel to the dilatometer sounding is needed in this method; although if the thrust required to advance the dilatometer blade is measured an estimate of the end bearing capacity of the dilatometer, q_D, can be made and used in place of the q_c value. Using the theory of Durgunoglu & Mitchell (1975), Schmertmann established a relationship between K_0, K_D and ϕ. This was combined with a second empirical relationship between q_c, K_0 and ϕ (Equation 2.11) which he had obtained from the analysis of dilatometer and cone-penetrometer tests carried out in a calibration chamber. An iterative procedure, the essential features of which are summarised in Figure 2.8, was used to solve for the friction angle and the stress ratio.

$$K_0 = \frac{40+23K_D-86K_D(1-\sin\phi)+152(1-\sin\phi)-717(1-\sin\phi)^2}{192-717(1-\sin\phi)} \qquad \text{(Eqn 2.11)}$$

Marchetti (1985) eliminated ϕ from Schmertmann's equations and produced the chart shown in Figure 2.9. The chart also shows field lines derived from the results of 25 pairs of q_c and

K_D profiles in a natural sand deposit which are probably more realistic than the calibration chamber derived lines.

Similarly Baldi et al. (1986) have devised an empirical equation (Equation 2.12) from the results of calibration chamber and field tests which uses the cone resistance at the depth where K_D has been measured.

$$K_0 = 0.376 + 0.095\ K_D - 0.0046\ \frac{q_c}{\sigma'_V} \qquad \text{(Eqn 2.12)}$$

Both Marchetti (1985) and Baldi et al. (1986) have attempted to account for the aging effects not present in laboratory tests by empirically incorporating the results from dilatometer tests conducted in natural sand deposits, into the equations they derived from calibration chamber data.

Jamiolkowski & Robertson (1988) have used the 'state parameter' ψ, to relate the horizontal stress to the dilatometer K_D reading. The state parameter is defined as the difference between the current void ratio and the void ratio on the critical state line at the same mean stress level. The perceived advantage the state parameter has over the relative density is that it combines the influence of void ratio and stress level. Their data for normally consolidated and overconsolidated Hokksund and Ticino sand fit the following expression:

$$\frac{K_D}{K_0} = 1.35\ e^{-8.08\psi} \qquad \text{(Eqn 2.13)}$$

The right hand side is termed the amplification factor. Its practical application is limited though, because the state parameter requires an initial value of K_0 to be estimated. Jamiolkowski & Robertson (1988) used a second relationship between the cone resistance q_c and ψ to overcome this problem:

$$\frac{q_c - p}{p'} = 30.8\ e^{-8.75\psi} \qquad \text{(Eqn 2.14)}$$

where

p' = Mean effective stress.

Eliminating the state parameter from 2.13 and 2.14 yields the following:

$$\frac{K_D}{K_0} = 0.0571 \left[\frac{q_c - p}{p'}\right]^{0.92} \qquad \text{(Eqn 2.15)}$$

The requirement to know p and p' is bypassed through an approximation:

$$\frac{q_c - p}{p'} \approx \frac{q_c}{p'} \approx \frac{q_c}{\sigma'_V}\left(\frac{3}{1+2K_0}\right) \qquad \text{(Eqn 2.16)}$$

K_0 can be found from the measured value of q_c/σ'_V using Equation 2.15 with the approximation of Equation 2.16. The chart shown in Figure 2.10 gives a graphical representation of these equations together with the relationship proposed by Baldi et al. (1986)

Equation 2.12.

All the above mentioned methods use a similar approach for the determination of K_0 in cohesionless deposits. They attempt to assess the density through a penetration thrust reading and use this to evaluate the increase in the horizontal stress caused by the insertion of the dilatometer. In all the methods the calculated K_0 is sensitive to small changes in K_D. This ill-conditioning is credited by Jamiolkowski & Robertson (1988) to the dependence of P_0 on the penetration thrust.

2.5.4 Overconsolidation Ratio

The overconsolidation ratio is usually defined as the ratio of the maximum past vertical effective stress to the present vertical stress. It is needed in the selection of the correct soil parameters for design. Marchetti (1980) presented a correlation (Equation 2.17) relating the *OCR* to the horizontal stress index for uncemented cohesive deposits (I_D < 1.2).

$$OCR = (0.5\ K_D)^{1.56} \qquad \text{(Eqn 2.17)}$$

The dependence of the *OCR* on K_D alone, implies that there is a relationship between K_0 and *OCR*. If Equation 2.17 is combined with Equation 2.8 the following relationship is found.

$$K_0 = 1.14\ OCR^{0.3} - 0.6 \qquad \text{(Eqn 2.18)}$$

Marchetti emphasised that Equation 2.17 was not applicable to deposits which had experienced anything other than simple unloading. Overconsolidation (in the sense that the

yield pressure is greater than the existing pressure), can be present in soils which have not undergone any form of geological unloading. This can be due to the effects of aging, desiccation or cementation. The OCR predicted for these soils using 2.17 has been termed by Marchetti (1979) the extended overconsolidation ratio (EOCR) as it is different from that which would be deduced from geological data but essentially provides the same useful information.

In cohesionless soils prestressing has the effect of increasing the stress ratio and the stiffness. Marchetti (1982) shows, using the results of calibration chamber tests, how the dilatometer is insensitive to the increase in stiffness associated with prestressing and is therefore unable to provide direct information about the *OCR* in sands.

2.5.5 Strength

2.5.5.1 Undrained Strength

Marchetti (1980) proposed a method for determining the undrained strength from dilatometer results which used the relationship between K_D and *OCR* and the relationship of Ladd et al. (1977), Equation 2.19, between the normalized strengths of normally consolidated and overconsolidated clays, and the overconsolidation ratio.

$$(s_u/\sigma_V') = (s_u/\sigma_V')_{nc} OCR^{m} \qquad \text{(Eqn 2.19)}$$

For the simple shear strength, values of 0.22 and 0.8 for $(s_u/\sigma_v')_{nc}$ and m respectively are appropriate for many clays. Through substitution of Equation 2.17 into Equation 2.19 a relationship between K_D and s_u (Equation 2.20) is obtained which according to Marchetti (1980), yields a conservative prediction of the undrained strength.

$$s_u = 0.22\ \sigma'_V\ (0.5\ K_D)^{1.25} \qquad \text{(Eqn 2.20)}$$

Lacasse & Lunne (1988) and Jamiolkowski et al. (1988), found that the above expression was reliable in soft to stiff unfissured clays. Equation 2.19 (Ladd et al. (1977)) is inappropriate for cemented clays but curiously Marchetti (1980) found his expression (Equation 2.20) was suitable for the determination of strength in these materials.

2.5.5.2 Friction Angle

Jamiolkowski & Robertson (1988) categorise the existing methods available for the determination of the friction angle, ϕ, from the results of penetration tests:

- Empirical correlations
- Bearing capacity theories
- Cavity expansion theories.

Of the above, the bearing capacity theories have proved to be the most useful for determining the friction angle using the dilatometer. Empirical correlations are limited in their ability to take proper account of the variables involved; and cavity expansion theories require a knowledge of additional soil parameters.

Schmertmann (1982) presents a method for the determination of the friction angle in sands from the Marchetti dilatometer test. It is based on the assumption that the penetration of the dilatometer causes a plane-strain bearing capacity failure. He uses the work of Durgunoglu and Mitchell (1975), who developed a theory for the bearing capacity of a rigid wedge, to find the friction angle from the stress ratio, the P_0 reading and force required to advance the dilatometer blade. A computer is needed to carry out the iterative procedure required to solve

the equations.

Jamiolkowski & Robertson (1988) give an example of an evaluation of the plane strain friction angle from the results of dilatometer tests in calibration chamber using Schmertmann's method. Their results indicate that the angle can be determined to within 5% in the laboratory if the horizontal stress is known.

2.5.6 Stiffness

2.5.6.1 One-dimensional Constrained Modulus, M

The local constrained drained modulus M $(=1/m_v)$ in sands and clays has been correlated by Marchetti (1980), to E_D, with I_D and K_D acting as qualifiers which determine the precise relationship. The loading direction and conditions are far removed from those used to find the reference moduli and in consequence the relationship is purely empirical.

2.5.6.1.1 Clays

For soil deposits with I_D less than 0.6 (clays) the following equations are given by Marchetti (1980).

$$R_M = 0.14 + 2.36 \log K_D \qquad \text{(Eqn 2.21)}$$

where

$$M = R_M E_D \qquad \text{(Eqn 2.22)}$$

In clays E_D is the undrained modulus of prestrained soil and frequent misgivings have been expressed about linking it to the undisturbed drained modulus. However, in field tests the

results from the application of Equation 2.21 have, in general, been good. Lacasse & Lunne (1988) recommended the continued use of the Marchetti correlation and Powell & Uglow (1988b) found site specific relationships between E_D and M with the R_M value approximately equal to 1.0.

2.5.6.1.2 Sands

Because of the difficulty of retrieving samples, the standard penetration test and the cone penetration test have traditionally been used to establish the deformation characteristics of sand deposits. However, it has been recognized for some time that penetration resistance tests are inherently incapable of sensing the in-situ compressibility of granular soils, Leonards & Frost (1988). Schmertmann (1986b) expresses the opinion that dilatometer evaluations of the soil modulus are likely to be more accurate than those obtained from the CPT, because the dilatometer attempts to measure a modulus while the CPT infers one from the cone end bearing capacity. Baldi et al. (1988) list the following as influencing the deformation characteristics of a soil:

- Stress and strain history
- Mean effective stress
- Induced level of shear strain
- Effective stress path
- Aging
- Fabric (particle arrangement and orientation)

As many of the above factors are either destroyed or altered through disturbance caused by dilatometer insertion it is difficult to understand why there should be a correlation between the original soil moduli and the dilatometer readings. Doubts are also expressed by Baldi et al. (1988) who, after scrutinising the results of a large number of dilatometer tests in a

calibration chamber, conclude that the stress history, which heavily influences soil stiffness, has little effect on the dilatometer modulus.

To obtain the vertical tangent constrained modulus, E_D must be multiplied by a correction factor R_M which is presented by Marchetti (1980) as a function of K_D. He argues that K_D is sensitive to *OCR*, aging and relative density, and is therefore able to correct E_D for the factors which control the soil moduli and which are obliterated during the insertion process. Results from the calibration chamber studies of Baldi et al. (1986) provide some support for Marchetti's claim. They found that the tangent Young's modulus, measured during the consolidation stage of calibration chamber tests, increased from three to six times as a result of the prestressing of the specimen. This had little effect on E_D but resulted in an increase in K_D by a factor of between 1.05 and 2.66. The sensitivity of K_D was found to decrease with increasing relative density suggesting that it was the change in density associated with prestressing, (which is greater for looser samples), which was being detected by K_D. However Baldi et al. concluded that the sensitivity of K_D to prestressing was relatively poor and that Marchetti's procedure for the determination of the constrained modulus would lead to a pronounced underestimate in overconsolidated sand.

However, Schmertmann (1986) presented convincing comparisons between dilatometer predicted settlements and actual recorded settlements. Also Lacasse & Lunne (1986) report that dilatometer derived constrained moduli from tests conducted in a deposit of Drammen sand compare well with the results of screw plate and cone penetrometer tests and those back-calculated from the observed settlement of a silo. It could be that the results of dilatometer tests in natural sand deposits yield more appropriate soil moduli than calibration chambers would suggest.

2.5.6.2 Shear Modulus

A value for the shear modulus can be derived from the dilatometer M value by assuming a value for the Poisson's ratio.

Finno (1993) has shown numerically that the soil which ends up in contact with the membrane experiences loading and unloading during the installation of the dilatometer. His predictions are based on an idealisation of soil behaviour and may not be accurate in detail, but it is highly probable that the soil under test has undergone some form of cyclic loading. Depending on the relative magnitudes of the unloading prior to the dilatometer test and the reloading involved in the membrane expansion to take the P_0 reading, the difference (P_1-P_0) could be expected to be a function of the undrained strength or the unload-reload modulus, G_{ur}. Clarke & Wroth (1988) set out the following mechanisms with their possible relationships:

1. No unloading during installation, soil at failure.

$$(P_1-P_0) = f(s_u)$$

2. Unloaded during installation but not reloaded to the maximum previous pressure by membrane expansion.

$$(P_1-P_0) = f(G_{ur})$$

3. Unloaded during installation and reloaded beyond maximum previous pressure by membrane expansion (only for perfectly plastic materials).

$$(P_1-P_0) = f(G_{ur})$$

By comparing the unload-reload modulus obtained from self-boring pressuremeter tests with dilatometer P_0 and P_1 values obtained at the same sites, they found that there was no general correlation between (P_1-P_0) and G_{ur}. This suggests that there was either no unloading during installation or if there was, there was also a reloading as predicted by Finno (1993) which brings the soil back to failure prior to the membrane expansion.

2.5.7 Coefficient of Consolidation

Davidson & Boghrat (1983a) used a piezoblade (a dilatometer blade fitted with a porous cap and pressure transducer in place of the membrane) to investigate the pore pressures generated during penetration and their subsequent dissipation. They presented a plot relating the percentage dissipation of excess pore pressure one minute after penetration against I_D. The percentage dissipation increased with increasing I_D, confirming the ability of I_D to give an indication of the bulk drainage properties of soil deposits.

Campanella & Robertson (1991) compared the rate of dissipation of pore pressures generated around a piezocone and a dilatometer equipped with a pore pressure transducer. They found that the rate of dissipation was slower around the dilatometer than around the cone. They also found that the P_2 reading (corrected C reading) was similar to the pore pressure on the face of the membrane in soft clays, and that by conducting repeated expansion-deflation tests P_0; P_1; P_2, they could follow the dissipation of pore pressure with time. The P_2 pressures tend with time to the equilibrium pore pressure.

Methods have been suggested for the estimation of the coefficient of consolidation in soft

clays by Lutenegger & Kabir (1988), Robertson et al. (1988) and Schmertmann (1988) from repeated P_0; P_1; P_2 readings; and by Marchetti & Totani (1989) from repeated P_0; P_2 readings. Campanella & Robertson (1991) conducted both types of repeated sequence in a single deposit and found they gave similar results but because they found it easier to perform, they favoured the repeated P_0; P_2 sequence.

2.5.8 Other Soil Properties

Marchetti (1982) and Robertson & Campanella (1986) have proposed methods for the determination of the liquefaction potential of saturated sands based on the results of calibration chamber dilatometer tests. The assessment uses K_D which is sensitive to the same soil properties as the liquefaction potential: the relative density, the in-situ stresses, stress history, aging and cementation.

The dilatometer has also been used to assess the effectiveness of ground improvement measures Schmertmann et al. (1986), Campanella & Robertson (1983), Lacasse & Lunne (1986); to estimate the limiting skin friction for piles driven in clay Marchetti et al. (1986); and to evaluate *p-y* curves for laterally loaded driven piles, Robertson et al. (1989).

There are numerous cases in the literature on the dilatometer of Marchetti's equations being applied to soil deposits which were specifically stated as being outside their area of applicability. Not surprisingly the dilatometer correlations have fared badly in these cases.

2.6 NUMERICAL AND ANALYTICAL APPROACHES

The insertion of an in-situ device imposes a complex loading on the soil. The interpretation

of in-situ tests firstly has to account for the resulting soil disturbance and secondly has to allow determination of soil parameters from the readings. In most in-situ tests, perhaps excepting the self-boring pressuremeter, at least one of these processes presents a major difficulty. The self-boring pressuremeter is not without problems but it produces minimal installation disturbance, and under a few simplifying assumptions a closed-form solution is available to evaluate certain soil parameters from the results. In the CPT the measurements are taken during the penetration process, and the determination of the friction angle, for instance, from the force readings is fraught with conceptual and practical difficulties. In the dilatometer test the determination of some soil parameters from the readings is relatively straightforward, because the dilatometer P_0 reading is a pressure which is directly comparable with the horizontal stress, and P_1-P_0 provides a stiffness to relate to the soil moduli. However, making an allowance for the disturbance is an empirical procedure at present and is therefore subject to severe errors. In consequence a number of researchers have attempted to relate theoretically the soil disturbance in the dilatometer test to fundamental soil parameters. So far the three-dimensional geometry of the blade has thwarted attempts to obtain an unequivocal theoretical solution, but by employing varying levels of simplification a number of researchers, using numerical methods, have made some important advances in the understanding of the test. Details of the two most significant follow.

Yu et al. (1992) present a theoretical treatment of the dilatometer test in cohesive soil. The soil has been idealised as an elastic-perfectly plastic material that obeys the Tresca yield criterion. By exploiting the dilatometer's two axes of symmetry the installation of the dilatometer has been modelled two-dimensionally in the horizontal plane using the finite element method. The contact pressure on the base of a 47 mm wide rigid strip after 7 mm of movement into the soil was taken as the dilatometer P_0 reading. The P_1 reading was not investigated as the expansion of the circular membrane is explicitly a three-dimensional problem. Their results indicate a dependence of the P_0 reading, on the undrained strength,

the rigidity index of the soil, and the total horizontal stress, Figure 2.11. This is a reasonable result because cavity expansion pressures depend on the strength and stiffness to strength ratio of a soil, and since the expansion of cavities is predominantly in the lateral direction for in-situ tests one would expect the horizontal stress to have some influence also.

When a finite element mesh with rigid boundaries is used to calculate the stresses on an intrusion there is an overprediction in the calculated stresses, which is dependent on the relative volumes of the intrusion and the mesh, and the bulk modulus of the mesh elements. An approximate correction can be applied to the stresses to account for this. Details of the correction and its application to the results of Yu et al. are described in Chapter 5.

Finno (1993) presents a three-dimensional numerical analysis of dilatometer penetration into a cohesive soil using the strain path method, Baligh (1985). In the strain path method it is assumed that probe penetration produces a deformation field which is independent of the strength of the surrounding soil. Analytical expressions for the required stream functions are found which can then be differentiated to find the velocity field, and from this the soil strains. The complex geometry of the dilatometer required the numerical definition of the stream function using a grid of spherical point sources. The velocities were determined by superposition of the velocity components of each component of the stream function. The total stresses, and hence the stresses acting on the dilatometer blade after penetration, are calculated from equilibrium. Finno (1993) used an anisotropic bounding surface model developed by Banerjee & Yousif (1986) to evaluate the effective stresses from the computed soil strains and total stresses. As with the analysis of Yu et al. (1990) the method is unable to evaluate the P_1 reading. The soil model uses the isotropic compression index λ, the isotropic recompression index κ, the critical state friction angle ϕ_{cs}, Poisson's ratio ν and a parameter μ which determines the amount of plastic strain which takes place inside the bounding surface.

It is necessary to assume that the soil has no shearing resistance to establish the strain field and therefore the method does not provide an exact solution but one which is indicative of the actual condition. Figure 2.12 shows the octahedral shear strain pattern around the dilatometer in a horizontal plane at the level of the centre of the membrane. Octahedral shear strains (Equation 2.23) have been used because they capture the three dimensional nature of the soil distortion. The octahedral strains at the centre of the blade are nearly the same as those that would be expected from plane strain (an infinitely wide blade) penetration, while the strains at the blade edges approximate to an axisymmetric (a cylindrical blade) response.

$$\gamma_{oct}=\frac{1}{3}\sqrt{(\varepsilon_{11}-\varepsilon_{22})^2+(\varepsilon_{22}-\varepsilon_{33})^2+(\varepsilon_{33}-\varepsilon_{11})^2+6(\varepsilon_{12}^2+\varepsilon_{23}^2+\varepsilon_{31}^2)} \qquad \text{(Eqn 2.23)}$$

In the strain path method the boundary of the dilatometer blade represents a singularity and the calculated strains become irregular as it is approached. Finno has therefore calculated the stresses associated with the numerically stable strains closest to the blade. Figure 2.13 shows the predictions of total and effective stresses normal to the centre of the dilatometer membrane for the soil parameters one would expect for a lightly overconsolidated medium plastic clay. The stresses have been normalised by the vertical stress. The normalised horizontal effective stress decreases from 0.65 (the initial condition) far in front of the blade to 0.35 50 mm in front of the tip. The stresses in the soil rise as it passes the cutting edge to a maximum at the point of constant blade cross-section. The stresses reduce thereafter to a relatively constant level across the membrane. The element of soil which is in contact with the dilatometer membrane when penetration is stopped to perform the test, has therefore undergone some form of stress relief. The change in total stress is predominately due to the generation of pore pressures.

A parametric study was used to investigate the sensitivity of the P_0 reading to variations in

four of the model material properties, ϕ_{cs}, λ, κ, ν and the initial state defined by K_0, *OCR* and the void ratio, e. It was concluded that the *OCR* and K_0 have the greatest influence on the P_0 reading and that strength and deformation parameters have less effect. However, at a constant vertical stress, variations in the *OCR* also affect the ultimate strength of the material. It is probable that if the author had taken account of the variation in the strength associated with different *OCR* values through normalisation of the P_0 reading by the strength, then the P_0 reading's apparent strong dependence on *OCR* would have been reduced. Similarly, increasing the stress ratio at constant vertical stress as was done in this study, has the effect of increasing the mean stress and therefore strength. Part of the link found in the parametric study between K_0 and P_0 can therefore probably be accounted for by the hidden linking of K_0 and the strength.

2.7 COMPARISON WITH OTHER DEVICES

Clark & Wroth (1988) compared the results from the dilatometer test and the self-boring pressuremeter in silty clays, till and heavily overconsolidated clay at eight sites. They compared the predictions of undrained strength, horizontal stress and shear modulus made with each device, and found that the dilatometer, using Marchetti's correlations, yielded soil parameters which did not agree with those obtained using the self-boring pressuremeter. They found site specific correlations between the results from the two instruments and suggested that the dilatometer could be used as a profiling tool between boreholes in which high quality tests had been conducted such as the self-boring pressuremeter.

Powell & Uglow (1986) compared the limit pressures, P_L, derived from Menard

pressuremeters, self-boring pressuremeters and push-in pressuremeters, to the dilatometer P_1 pressures obtained at three heavily overconsolidated clay sites. The limit pressures are from three different types of device and in particular the Menard pressuremeter calculates its limit pressure in a different way to the other two devices. However, a site-specific relationship between P_1 and the P_L readings was found to exist with nearly all the data enclosed by the limits $P_1 = P_L$ and $P_1 = 1.4\ P_L$.

Lutenegger & Blanchard (1990) compared the results of the cone-pressuremeter with those of the dilatometer at a number of clay sites. In the cone-pressuremeter two readings relating to the pressuremeter part are taken: the cavity pressure immediately after installation, P_i and the cavity pressure after approximately 30% cavity strain, P_L. These were compared with the dilatometer P_0 and P_1 pressures. Figure 2.14 shows a plot of P_i against P_0. It can be seen that there is a loose correlation between the two. Also Figure 2.15, which shows a plot of P_L against P_1, reveals a similar loose correlation but with P_1 slightly lower than the limit pressure.

2.8 RESEARCH DILATOMETERS

A research dilatometer was developed at the University of British Columbia to provide an improved understanding of the relationship between the readings and the soil behaviour during penetration and membrane expansion. It allowed the passive measurement of the following:

- pore water pressure at the centre of the membrane
- deflection at the centre of the membrane
- gas pressure in the blade during membrane expansion

- penetration force immediately behind the blade
- verticality of the blade

Typical results obtained using the instrumented dilatometer of the total pressure and pore pressure versus membrane deflection are shown in Figure 2.16, Campanella & Robertson (1991). It should be noted that the results of this instrumented dilatometer show a 1.0 mm displacement between P_0 and P_1, instead of the 1.1 mm which is found in the standard dilatometer. It is not known whether this is a peculiarity of their device or a simplification in the plots.

The results indicate that in soft clay (Figure 2.16a) large pore pressures are generated and that effective stresses during penetration and membrane expansion are small. The penetration pore pressures strongly control the two dilatometer readings. The effective stress adjacent to the membrane does not change during the expansion and unloading phase of the test as the increase in applied stress is matched by an equal increase in pore pressure. The authors point out that this response implies that the material adjacent to the expanding membrane is at failure. The closing pressure P_2 is similar to the penetration pore pressure.

In stiff clays (Figure 2.16b) the pore pressures immediately after penetration at the membrane are very small or negative. The effective stresses on the membrane are large and increase when the membrane is expanded into the soil. The P_2 reading appears not to be related to the penetration or hydrostatic water pressures. Similar findings for clays with $OCR > 2.5$ were reported by Lutenegger (1988).

For the tests in sand (Figure 2.16c) no excess pore pressures are generated during both the penetration and expansion phase of the test. The pressure on the membrane when it returns to the lift off position corresponds to the static pore water pressure. This is attributed by

Campanella & Robertson (1991) to arching of the sand as the membrane deflates. Controlled deflation of the membrane thus allows the determination of the groundwater table in sands. The expansion curve is similar in form to that for a cone-pressuremeter with a stiffer response if an unload-reload loop is carried out (3 - 5 times stiffer than the simple expansion A to B) suggesting that the primary expansion has both elastic and plastic components.

Robertson et al. (1988) found that the pore pressures measured with the research dilatometer in both soft and stiff clays were very similar to those measured behind the friction sleeve in the piezocone.

Lutenegger (1988) using a piezoblade (a dilatometer fitted with a pore water pressure measuring device in place of the membrane) obtained similar results to Campanella & Robertson (1991). Figure 2.17 shows a comparison between P_0 and the pore water pressure 15 seconds after piezoblade penetration into a number of soft clays. It can be seen that in most cases P_0 is essentially measuring the penetration pore water pressure. Results from piezoblade tests in overconsolidated stiff clays, Figure 2.18, show that there is a significant and variable component of effective stress in the P_0 reading.

2.9 CONCLUSION

The aim of the dilatometer test is to produce as low a degree of penetration disturbance as practicable and then correlate the membrane measurements, through the dilatometer indices, to the in-situ soil parameters existing prior to blade insertion.

The indices originally proposed have largely been found wanting when used to analyse new deposits, and generally confidence in the results of the dilatometer is restricted to site specific

correlations. Even though the dilatometer appears at present to have limited potential for acquiring accurate soil parameters, it has repeatedly shown its ability for good site characterization.

The instrumented dilatometer and piezoblade have opened the way to a more analytical approach to using and understanding the dilatometer readings. In clays dilatometer penetration and membrane expansion take place in undrained conditions. In soft clays large excess pore pressures are generated which dominate the membrane pressure readings. Cavity expansion theories predict that in soft clays there exists a limit pressure. It seems probable that penetration of the dilatometer blade is sufficient to induce pressures equivalent to the limit pressure. Stress relief phenomena and pore pressure dissipation result in lift off pressures, P_0, less than this pressure. The expansion of the membrane re-establishes this limit pressure, and P_1, therefore, is probably related to some form of cavity expansion. In stiff and overconsolidated clays P_0 from the dilatometer is greater than the pore water pressure measured with a piezoblade indicating blade insertion causes an increase in effective stress.

A quantitative description of the above processes is essential if the site specific nature of the present correlations is to be overcome. Until the penetration and membrane expansion mechanisms are understood more fully, back analysis to determine the original soil parameters will remain essentially empirical in nature, with the limitations that this entails.

The two measurements made with the dilatometer will at most provide two fundamental soil properties. The correlation of the dilatometer readings to the range of properties discussed in Section 2.5 merely indicates the interdependence of these soil properties.

CHAPTER 3

CALIBRATION CHAMBER FOR CLAY

3.1 INTRODUCTION

It is possible to reduce greatly the uncertainty involved in the interpretation of in-situ test results by verifying, in a large calibration chamber, any empirical or theoretical relationship between a measured quantity and soil parameters. Indeed, the high degree of control over soil specimen history, uniformity and boundary conditions attainable in calibration chambers allows some confidence to be attached to relationships derived entirely from their use. However, because of aging, leaching, weathering and fabric effects, natural soil deposits behave differently to laboratory produced samples, and it is advisable to take care when applying such relationships to the results of field tests. The finite size of the soil specimen which is practical, and the nature of the boundary conditions in the laboratory also lead to uncertainties, but calibration chambers still provide an excellent means of studying the fundamentals behind soil-probe interaction, and provide reasonable data points to compare with those obtained in the field.

A number of calibration chambers for sand have been constructed and successfully used over recent years: Chapman (1974); Parkin et al. (1980); Bellotti et al. (1982); and Schnaid (1987). The chambers are all similar in design and operation, and have provided much insight into the capabilities and limitations of various in-situ test methods. However, their use is restricted to sands, and major design alterations are required to accommodate the large movements and tractions that accompany the preparation of a clay specimen from a slurry.

The Oxford University calibration chamber for clay was designed and commissioned by the

author in ten months. This chapter describes its development, and the procedures involved during the following 29 months, in the preparation and testing of 9 kaolin beds.

3.2 EXISTING CALIBRATION CHAMBERS FOR CLAY

A few research groups have built and used flexible-walled calibration chambers suitable for cohesive soils. Their number include the following: Steenfelt et al. (1981); Huang et al. (1988); and Anderson et al. (1989).

The chamber of Steenfelt et al. (1981) was developed to study instrumented model piles jacked into clay. It could be used to produce samples 600 mm high and 250 mm in diameter and essentially differs only in scale from the chamber of Anderson et al. (1989) which is described later.

Huang et al. (1988) developed a calibration chamber system and testing procedure to perform miniature pressuremeter tests in clay soils. They used a two stage approach for sample preparation. A slurry was first one-dimensionally consolidated in a two-part rigid-sided cylindrical container to form a clay cake 200 mm in diameter and 360 mm high. The lower part of the container could be broken into two halves and was lined with a rubber membrane. This enabled the specimen to be transferred to stage two of the process without the need for extrusion or trimming; i.e. with the minimum of mechanical disturbance. After removal from the consolidometer the clay specimen, still inside the membrane, was placed in a calibration chamber equipped with side and base membranes. It was then consolidated to a vertical effective stress higher than that experienced during the first stage. The calibration chamber was double-walled allowing a zero average lateral strain condition to be applied by balancing the pressure inside and outside the inner wall whilst preventing the movement of water out of the space between the sample and inner wall. The second stage of consolidation was thus,

in a sense, one-dimensional also.

Anderson et al. (1989) also adopted a two stage approach to clay sample preparation but on a much larger scale. His apparatus could be used to produce clay beds 785 mm in diameter and approximately 900 mm high. The chief differences between the clay bed preparation procedures of Anderson et al. and Huang et al. are as follows:

• No system was available to continue the first stage one-dimensional consolidation during the second stage, though stress-controlled anisotropic consolidation was possible.

• The rigid body consolidometer, in which the K_0 consolidation of slurry was carried out, was seated inside the second stage chamber and was removable. By withdrawing the consolidometer an isotropic stress system could be applied using the surrounding triaxial chamber without the need to move the clay bed. This was necessary because of the weight of the clay bed after the initial preparation stage.

• The second stage of consolidation was carried out before complete dissipation (around 50 %) of the pore water pressures generated through the first stage.

3.3 DESIGN CONSIDERATIONS

The investigation of in-situ devices, if it is to have any relevance outside the laboratory apparatus, must either be performed in a sample of soil large enough to eliminate boundary effects or in one where the boundaries can be controlled. For most soils it is impractical to produce samples of sufficient size to satisfy the first condition, and it is in any case desirable to have control over the boundary conditions so that their influence can be explored. With a finite soil specimen, the type of condition applied at the boundary will affect the response

measured by a probe. The dimensions of the specimen compared to those of the probe also influence the result. The larger the specimen relative to the probe the closer the peripheral conditions approach to the ideal conditions of zero stress change and zero deflection. Therefore, unless a stiffness can be applied to the outer surface to simulate the effect of a surrounding soil, the dimensions must be chosen so as to minimise the deflection at the boundaries, but at the same time keep the specimen a manageable size.

The distortions produced by probe insertion in clay are high in zones where yield occurs but lower in the surrounding elastic region. By ensuring the clay at the sample boundary remains in an elastic state the boundary deflections can be minimised. If the dilatometer geometry is approximated to a cylinder with the same cross-sectional area and the penetration is modelled as a cylindrical cavity expansion, the interface of the highly distorted plastic region with the relatively intact elastic region can be approximately located. For an undrained cylindrical cavity expansion, Yu (1990):

$$\frac{R_{ep}}{a} = \sqrt{I_r} \qquad \text{(Eqn 3.1)}$$

where

R_{ep}= radius of the elastic-plastic boundary

a = 'effective radius' of the probe

I_r = rigidity index of the soil = G/s_u

The rigidity index is affected by overconsolidation, stress level and strain level, but because of difficulties in measuring the shear modulus the effects of each of these factors are still uncertain. The results from tests on a number of clays, summarised by Wroth et al. (1979), suggest that a rigidity index of 500 is an upper limit for normal engineering stress levels. Using this value with Equation 3.1 gave the radius of the elastic-plastic boundary as 0.46 m

suggesting a 1.0 m diameter clay bed was required to prevent large distortions reaching the boundaries.

The depth of the clay bed was selected in a more empirical fashion. It was limited by the increasing consolidation time with bed height, but had to be sufficient to give a central region free from the influences of the top and bottom boundaries. A one metre deep bed was chosen in line with the height successfully used in a number of calibration chambers for sand. For ease of deairing, pumping and also the production of a uniform sample, it was necessary to start with a slurry having a water content approximately double the liquid limit. This results in an initial slurry height approximately twice the final clay bed height.

Having decided upon the clay bed dimensions the final step was to engineer an efficient method of preparation. The difficulties associated with a single stage preparation technique quickly became apparent. Consolidating directly into a membrane in the same manner as Huang et al. (1988) presents problems. Side friction forces develop as the clay consolidates which must be resisted locally if membrane tearing is to be avoided. Huang et al. achieved this by introducing sheets of sand paper between the chamber sides and rubber. This solution was thought impractical for the large scale apparatus planned. Protecting the membrane with a 'Chinese lantern' type shield between the clay and membrane was also considered and then discounted. The preferential drainage paths it may have allowed could have induced non-homogeneity through radial drainage, Nageswaran & Houlsby (1982).

It was decided to continue with proven methods of preparation and a two stage approach based on the apparatus of Anderson et al. (1989) was adopted i.e. one-dimensional consolidation in a rigid-walled tube followed by the removal of the tube without the need to move the clay, and then reconsolidation in a triaxial chamber.

Apart from the first stage of one-dimensional consolidation, sample preparation was to be

solely stress controlled. Many of the sand calibration chambers in existence have a double wall construction consisting of two concentric cylinders. By balancing the pressures on either side of the inner cylinder its position can be kept fixed, allowing either the sample volume to be kept constant or the change in volume to be measured. However, it was envisaged that in neither of the second stage of clay bed preparation or the testing stage would strain control or accurate volume measurement be required so the design of the triaxial chamber was based on a single-walled cylinder.

3.4 DESIGN AND OPERATION DETAILS

3.4.1 Introduction

The chamber system as designed allows the manufacture and testing of a one metre diameter by one metre high clay bed. The basic preparation and testing steps are illustrated in Figures 3.1a) to 3.1d). More detail is given in the following sections.

The clay is first consolidated one-dimensionally from a slurry in a tube using a rigid piston to ensure uniform straining. The assembly permits drainage at both ends, Figures 3.1a) & b). This tube is then removed allowing the sample to be stressed and further consolidated in a flexible walled chamber, Figure 3.1c). The mean stress is approximately doubled during reconsolidation to minimise the effects of unloading. After the stress state required has been reached the sample is ready to be tested with the dilatometer, Figure 3.1d).

Speswhite kaolin, the basic properties of which are given in Table 3.1, was selected for the test programme. It is obtained from the English China Clay Company as a dry powder. This material was chosen for two main reasons:

• For a clay it is relatively permeable and therefore consolidates rapidly allowing the large

samples required for the test programme to be prepared reasonably quickly (approximately ten weeks).

• The geotechnical properties of the material have been thoroughly explored and it has been found that the framework of critical state soil mechanics provides a useful guide to its behaviour. According to this model the isotropic normal compression line, one-dimensional normal compression line and critical state line have parallel projections on the e : $\ln(p')$ plane. Figure 3.2 illustrates the significance of some of the constants included in Table 3.1.

The values assigned to the constants in Table 3.1 have been taken principally from the work of Al-Tabbaa (1987). Other researchers have investigated the mechanical properties of this clay: Houlsby (1981), Steenfelt et al. (1981), Clegg (1981), Martins (1983), Airey (1984), and Elmes (1985); and a range of values for each property has been found. This is not unexpected as the details of the tests used by each researcher differ, and it is also probable that different batches of the same clay have different properties. Al-Tabbaa's values have been chosen because they represent the most recent and comprehensive study into the properties of speswhite kaolin and are in broad agreement with values cited by the other researchers.

In particular the value for K_0 entered in Table 3.1 of 0.69 was obtained by Al-Tabbaa (1987) using a load cell oedometer able to measure directly the horizontal stress, and this value was therefore preferred over the more often cited value of 0.61 which is found by substituting a friction angle of 23° for Speswhite kaolin into the Jaky formula, Muir Wood (1990):

$$K_0 = 1 - \sin(\phi) \qquad \text{(Eqn 3.2)}$$

The properties in Table 3.1 were successfully used to formulate and control the preparation

procedures of each of the nine clay beds. These are described in detail in the following sections.

3.4.2 Clay Slurry Preparation

A slurry was formed by mixing the dry Speswhite kaolin powder with water in a ribbon blade mixer. Air bubbles were removed by applying a vacuum during mixing. The powder was mixed at a water content of around 120 % to form a pumpable slurry and to prevent the formation of fabric features during the filling stage. Checks on the completed mixes indicated that the water content of each batch was within ± 2 % of the target. The water content of 120 % is approximately twice the liquid limit which is the value recommended by Sheeran & Krizek (1971) for the preparation of uniform clay beds using slurry consolidation. Anderson et al. (1989) used a water content 1.5 times the liquid limit to reduce the volume changes required with no apparent detrimental effect on the uniformity of the final sample.

Between 14 and 16 batches were required to provide the necessary amount of slurry, each taking two hours to mix, with a half hour to pump and then prepare the following batch. Slurry preparation was spread over two to three days.

3.4.3 First-Stage Consolidation

The first-stage consolidation apparatus comprises a 2.5 metre long GRP tube with an internal bore of 986 mm, a heavy duty hydraulic jack, a solid piston attached to the jack plunger and a reaction frame connected by four lengths of studding to the triaxial chamber. Figures 3.3a) and b) are two computer generated images of the first-stage consolidation apparatus. For clarity the clay bed has been omitted from the section through the apparatus shown in Figure 3.3b). A conventional section with annotation is shown in Figure 3.4.

The GRP tube was obtained from Johnston Pipes Limited. It was manufactured using 50 g centrifugal casting which gave the internal bore a high degree of circularity. The tube walls are 20 mm thick and consist of a mixture of sand and resin reinforced with glass, sandwiched between 4 mm layers of resin and glass fibre at the outer and inner surfaces. The tube has a high circumferential stiffness and strength but would deflect significantly from its cast shape when placed on its side under the action of its own weight. This was felt to be an advantage as it could accommodate any non-circularity in the piston and still resist high internal pressures. The manufacturing process left a smooth resin-rich coat on the bore, but to reduce possible side friction effects to a minimum, the internal GRP tube walls were lightly smeared with water pump grease. Excessive friction could have resulted in significant variations in the water content down the sample because of the loss in axial stress. The GRP tube sits on the base membrane attachment ring and is sealed against leaks with silicone plastic.

Figure 3.5 shows the detail at the base membrane. Silicone grease is used to create a low friction contact between the base membrane and the protective rubber sheet above. A flexible mesh and two 1.0 mm thick Vyon filters rest on the rubber sheet. According to the manufacturer the filters are able to retain particles 27 microns in size. This appeared to be sufficiently fine to prevent leaching of the kaolin particles. The base membrane acts as a flexible stress-controlled boundary during the second stage of consolidation so radial cuts were made in the filters to ensure that they were able to change shape with the membrane. A coiled lead connects the drainage port in the base membrane to the base of the chamber. This drainage line was shut and the tube was filled with water to a depth of 0.1 m prior to the slurry filling stage.

Slurry was pumped from the mixer through a 30 mm internal diameter flexible tube and directed to a position under the water level in the GRP pipe. A 2.5 mm thick Vyon filter and separator mesh were placed on the slurry surface after the pipe was filled with the required height of slurry. The number of mixes required for each test bed varied according the final

stress state. The reaction frame and piston were raised above the tube and connected with the studding. The piston was then lowered using the nuts on the studding until it reached the water sitting on top of the slurry. At this level the studding nuts were tightened and the jack was connected to the hydraulic system described in Section 3.4.4. With both the top and bottom drainage lines open the free water was driven out using the jack. A jack pressure equivalent to a vertical stress of 25 kPa on the slurry was used initially. This was low enough not to endanger the competence of the seals at the bottom of the GRP tube and between the piston and tube, but sufficient to give relatively rapid consolidation. The consolidation of the clay was monitored by measuring from the top of the piston to the top of the GRP tube. Detail A of Figure 3.4 shows the piston seal. The solid o-ring acted to align the piston centrally in the tube and the hollow rubber tubing provided the seal. The hollow tubing had to be replaced after five consolidations but otherwise the seal proved to be very effective.

Double vertical drainage was accomplished using ports in the piston and a connection through the base membrane to the chamber base. These were connected by tube to a constant head device which worked on a water overflow principle. The constant head device was kept 150 mm above the level of the piston during the consolidation process. Excess water was carried from the constant head device in a tube to a drain. Before the piston movement reached its full stroke of 330 mm the drainage lines were closed, the jack pressure was dropped to zero and another extension piece was added. Loading and extension piece replacement were continued until the sample reached a vertical height of around 1.05 metres. At this height the jack pressures were lowered and the sample was allowed to expand to change the effective stress in the sample to an approximately isotropic state (Section 3.5). Figure 3.6 shows a settlement-time plot for test OCD.

The one-dimensional consolidation apparatus was designed for a maximum vertical pressure of 600 kPa which is in keeping with a maximum mean stress of 800 kPa for the triaxial stage,

given the increase in stress required to eradicate the effects of the unloading between the first and second stages of clay bed preparation.

3.4.4 Sample Extrusion

The clay bed is extruded from the GRP tube using the assembly shown in Figure 3.7.

The drainage lines are first shut to ensure that the water content of the sample is preserved whilst it is transferred between the first and second stages of consolidation. A jack with a 1.02 m stroke is screwed into the top of the piston after the reaction frame, heavy duty jack, and extension pieces have been removed. This reacts against a cross beam attached to the tube at four points and forces the tube to move relative to the piston and clay sample. The four attachments consist of pairs of friction-plates bolted together through the GRP tube wall and pinned to the cross beam. The available stroke of the jack required that the final 75 mm of extrusion was carried out by raising the tube with an overhead crane and relying on the weight of the clay bed to resist the vertical pull. The efficiency of the piston seal was such that it was necessary to open the top drainage line to break the suction between the top of the clay bed and the underside of the piston during this process. The rate of extrusion was typically about 10 mm/sec. The jack is driven using the hydraulic system shown in Figure 3.8.

The electrically driven hydraulic pump is used for the relatively quick operation of sample extrusion. Oil pressures are controlled using the oil dump valve. The first stage sample consolidation, (Section 3.4.3), is spread over a number of weeks. Using a motor driven system for this length of time is inefficient so an air-oil intensifier is used to generate the required oil pressure from the compressed air supply. An air regulator is used to control the oil pressure.

The pressures required to initiate extrusion were equivalent to frictional forces of between 4 and 11 kPa at the tube-clay boundary. It is thought that the variation of the resistance was mainly dependent on the action of the grease and not on the strength of the clay, as there was no specific correlation between the frictional resistance and the vertical stresses applied during the one-dimensional consolidation. In all the samples the force required to continue extrusion was less than that required to initiate it. After 200 mm of tube movement relative to the clay the frictional resistance ranged between 1 and 3 kPa.

3.4.5 Second-Stage Consolidation

The second-stage consolidation apparatus is shown in Figure 3.9. The extruded sample was trimmed to 15 mm below the top of the upper lateral membrane attachment ring using a blade mounted on a rigid beam. This was done to allow space for a gauze and new 1 mm thick Vyon filter, and to ensure that the sample was lifted clear of the chamber base during the second-stage of consolidation. The trimming typically involved removing a layer of clay 20 mm thick. The lateral membranes were filled with water from the mains. The outer surface of the inner ring of the chamber lid was smeared with sufficient silicon rubber sealant before the lid was bolted to the top flange to ensure the gap between the two rings was filled; see Detail A of Figure 3.9. The water pressures at this location are very low and no leakage problems were encountered in any of the tests. The ring connected to the underside of the chamber lid as well as providing a means of correctly positioning the lid could easily be adapted in the future to secure a membrane and make the top boundary a flexible one.

The base and lateral membranes were pressurized using the system shown in Figure 3.9. An air regulator and air-water interface connected to each membrane permitted independent control of the base and lateral pressures making axisymmetric reconsolidation possible. The water pressure in the base membrane was first raised to a level sufficient to bring the clay bed into contact with the underside of the top lid. One of the three port-holes in the top lid was

left open to observe this operation. The port-hole was then plugged and the pressures in the two membranes were raised together to between 100 and 200 kPa depending on the previous maximum stress experienced by the clay bed. The lid was heavily reinforced with stiffeners to ensure that the top boundary was rigid enough to avoid non-uniform deformation of the clay bed under high vertical loads. The details of the application of deviator stresses to the clay bed are included in Section 3.5.

The lateral and base membranes were made from three millimetre thick shotblast rubber with vulcanised scarf joints to facilitate their sealing. They were connected independently to the cylinder and base which would have permitted relatively easy access and repair if they had ever been damaged. Detail B of Figure 3.9 shows the attachment of the membranes at the base. In both attachments the membrane is sandwiched between a flange and a clamping ring. This metal-rubber-metal type seal was found to be useful as the few leaks that were encountered were due to loose clamping rings, and could be stemmed by tightening the bolts through the flange in the vicinity of the leak.

Although the preparation of each clay bed was designed to restrict the boundary strains, the confidence that could be placed in the parameters that were used in the calculations, was limited. It was therefore important to have some measure of the base membrane position so that over-stretching could be avoided and so that the dilatometer was not pushed too far during the test. This was done by monitoring the volume of water passing into the membranes and being expelled from the clay. The water expelled from the clay bed was piped to a constant head device 150 mm above the level of the top of the bed to give an equilibrium hydrostatic pore water pressure distribution throughout the sample at the end of consolidation. The overflow from the device was collected in a bucket. It was possible to monitor separately the lateral and vertical volume changes by recording the water level movement in the relevant air-water interface. The correlation between the volume of water discharged from the sample and the volume of water entering the membranes, was sometimes

poor, though this could usually be accounted for by losses of the expelled water from leaks.

The pressures applied to the clay bed were calculated from the air pressure (measured using a digital pressure indicator) and the water level in each air-water interface. The manual gauges were sufficiently accurate to use in the setting of the water pressures.

During this stage of the preparation it was important to have an idea of the pore water pressures so that the degree of consolidation and the effective stresses could be estimated. These were also needed to gauge the magnitude of the deviator stress increment that could be applied, without risking failure, during the preparation of non-isotropic clay beds. Early on in the project it was decided to avoid using pore pressure transducers because of their poor reliability and general inconvenience, and instead to estimate the pore water pressures through measurement of the rate of flow of water out of the clay bed. If the flow is assumed to be one-dimensional the rate of flow out of one end of the sample, Q, is given by the following expression:

$$Q = k A_s i \qquad \text{(Eqn 3.3)}$$

where

k = Permeability of clay

A_s = Cross-sectional area of bed

i = Hydraulic gradient at the end of the sample.

Al-Tabbaa (1987) investigated the variation of the permeability of Speswhite kaolin with void ratio. Her results for both overconsolidated and normally consolidated clay can be represented by the following expression:

$$k_v = 0.5\ e^{3.25} \cdot 10^{-9} \ \text{m/sec} \qquad \text{(Eqn 3.4)}$$

where

k_v = Vertical permeability

e = Void ratio

The equation forms a lower bound to the work of other researchers and therefore, if anything, the pore water pressures will be predicted as being higher than they actually are. Using a prediction of the void ratio the vertical permeability can be calculated, and from this, using Equation 3.4, the hydraulic gradient at the ends of the clay bed. If the isochrone shape is assumed to be parabolic, it can be shown mathematically that the central excess pore pressures is half of that found if the hydraulic gradient at the end of the sample is extrapolated to the centre of the bed.

3.4.6 Driving Mechanism

An existing driving assembly was modified slightly to allow it to be used to insert the dilatometer into the calibration chamber, Figure 3.10. The design and operation of the equipment are described in detail by Schnaid (1990) and only a summary description is given here.

A 3 kW DC motor is geared to rotate the shaft of a ball screw at a preset speed using a chain and sprocket arrangement. A robust carriage, threaded onto the screw and restrained from lateral motion by two precision linear guides, transfers the torque from the screw to a downward thrust on the dilatometer driving rods. The axial forces generated in the screw are transmitted by the top bearing into the base plate through a stiff frame. Reaction forces are resisted by four supports welded onto the chamber lid and connected to the base plate using four pins. The driving assembly gives the dilatometer a linear speed of 20 ± 5 mm/sec and is able to provide a downward load of up to 100 kN.

3.4.7 Instrumentation

The dilatometer was equipped with two load cells to measure the total axial thrust and the thrust immediately behind the blade during insertion, Figure 3.11. Two RDP E307-2 Digital Transducer Indicators were used to supply excitation voltages and to amplify the return signals. A Calvin tensioned-cable position-transducer powered by a five volt DC supply, was used to measure the displacement of the carriage during the driving procedure. These were monitored during the test using the data acquisition system described in Section 3.4.8. It was found that there was no significant variation in the load cell calibration constants during the testing programme.

3.4.8 Data Acquisition

The data acquisition system described by Schnaid (1990) was used to monitor the instrumentation. In brief, the outputs from the two load cells and the potentiometer were read by a Minate 7010 Analogue Scanner which was attached to a Solartron 7060 Systems Voltmeter. A GPIB (General Purpose Interface Bus) board provided communication between the voltmeter and an IBM-XT computer. A program written in Hercules BASIC by Schnaid (1990) was used with minor modifications, to scan and store to hard disk the output signals from the instrumentation.

3.5 SUMMARY OF SAMPLE PREPARATION

Each clay bed was prepared in two separate stages: a one-dimensional phase and a triaxial phase.

In the one-dimensional phase a slurry was consolidated to form a clay bed strong enough to

support itself under its own weight. The quantity of slurry required for each test was dependent upon the final stress level (higher stress levels result in denser samples containing more solids), and the expected losses from trimming.

The grease on the side of the GRP tube as well as limiting the friction, served to discourage radial drainage. However, because of the discontinuity at the interface of the tube and clay it was thought likely that some radial drainage would take place. Radial drainage leads to non-uniform samples Nageswaran & Houlsby (1982) but the uniformity can be improved by limiting the magnitude of the pore pressures according to Atkinson et al. (1985). In oedometer consolidation tests it is standard practice to apply the load incrementally with each increment in load being equal to the existing load. High pore water pressures are generated during using this procedure. To limit the pore pressures it is necessary to limit the rate of loading by applying only small increments in load. This method of loading was adopted during the one-dimensional consolidation in the GRP tube as can be seen in Figure 3.6 which shows the preparation of sample OCD.

Limit equilibrium calculations suggested that at the end of the initial consolidation the clay bed had to possess an undrained strength of 15 kPa if it was to be stable and capable of resisting the extrusion forces. From Table 3.1 it can be seen that a vertical effective stress of 75 kPa is required throughout the sample to give this strength. Using this system, therefore, it is essential to bring the vertical effective stress in the sample to at least this value before transferring it to the triaxial phase of consolidation.

In opposition to the above requirement the need to eradicate the effects of unloading (during extrusion) on the final properties of the sample, exercises a restraint over the maximum vertical stress that can be applied during the first stage of consolidation. The recognised way of eliminating a specimen's 'unloading memory' is to reload it to a higher stress than it has previously experienced, Kirkpatrick & Khan (1984). However, the preparation technique

described here, involves changes in both mean stress and stress ratio and it is therefore more meaningful and convenient to prescribe a second-stage volumetric strain than to increase the mean stress (say) by a factor of 2. By setting the second-stage volumetric strain to 7% the ratio p'_T / p'_{1-d} was fixed for each sample; where p'_T is the mean effective stress at the testing stage and p'_{1-d} is the maximum mean effective stress reached in the sample during the first-stage of consolidation. Assuming K_0 = 0.69 (Table 3.1) gave the vertical stress that should be applied during the one-dimensional phase of consolidation to ensure that the second stage strains were sufficient to eliminate the unloading effects of extrusion. This method of analysis was suited to the preparation technique because it also limited the second stage volumeteric strains to acceptable levels; though, because different stress paths were used in the preparation of each sample the volumetric strains expressed themselves in different proportions of horizontal and vertical strain. Theoretically the shear strains associated with a particular stress path could have been calculated using the shape of the yield envelope, but the calculation is inherently inaccurate and was not performed.

The expected volume change for a given increment in mean effective stress can be found using the slope of the consolidation line, λ, Table 3.1. To predict the volume change associated with a stress path which is not a continuation of the sample's previous path, an assumption has to be made regarding the shape of the yield locus. Once this has been made, or if experimental data is available, a value can be assigned to $e_{q/p}$ which is the reference void ratio when p' = 1 kPa for consolidation at a particular stress ratio. Using Equation 3.5 it is possible to predict the change in void ratio resulting from a change in the mean stress and stress ratio.

$$\Delta e = \Delta e_{q/p} + \lambda \ln\left(\frac{p' + \Delta p'}{p'}\right) \qquad \text{(Eqn 3.5)}$$

where

Δe = Change in void ratio

$\Delta e_{q/p}$ = Change reference void ratio associated with change in the stress ratio

p' = Mean effective stress

$\Delta p'$ = Change in mean effective stress

Table 3.1 gives the experimental reference void ratios for consolidation paths at three different stress ratios. These are in moderate agreement with the idealised yield locus of modified Cam-clay. Theoretical reference void ratios for stress ratios, K, of 0.6 and 2.0 were found by assuming the shape of the yield locus was in full agreement with modified cam-clay theory. These are also entered in Table 3.1. The results of Al-Tabbaa (1987) show that if a soil is consolidated under anisotropic stress conditions an anisotropic yield locus will result. The effect of this on the shear strains is significant but for the simple purpose of calculating volume changes the modified Cam-clay yield locus is a reasonable approximation.

The following notes refer to Figure 3.12, which shows the effective stress path in $p' : q : e$ space the sample was taken through to bring it to stress state B. The stress path is also shown in the $p' : q$ plane in Figure 3.13. This path is idealized as OAB in Figure 1.1 and Table 1.1. For clarity the positions of the critical state line and one-dimensional compression line in stress space have been taken from the theoretical shape of the yield envelope in modified Cam-clay theory. Their experimental positions suggest a slightly different shape for the yield envelope.

1-2 This is the one-dimensional first-stage of consolidation. The consolidation along this path is described by Equation 3.6.

$$e = e_{1\text{-}d} - \lambda \ln(p') \qquad \text{(Eqn 3.6)}$$

During consolidation inside the rigid tube it was necessary to remove the piston load and insert an extension piece each time the jack reached the end of its stroke. Throughout this changeover period the top and bottom drainage lines were shut. The total stress unloading accompanying the insertion of extension pieces does not affect the effective stress path because drainage is prevented during the changeover period. Second stage deformation requirements and the position of the yield locus dictate the precise value of the mean stress at **2.**

2-3 The piston load on the sample was lowered to attempt to bring the sample to an isotropic stress state. The behaviour of Speswhite kaolin during one-dimensional unloading was found by Al-Tabbaa (1987) to follow closely the formula of Schmidt (1967), Equation 3.7.

$$\frac{K_0}{K_{0nc}} = OCR^{1.2 \sin\phi} \qquad \text{(Eqn 3.7)}$$

where

K_{0nc} = Stress ratio during one-dimensional normal compression

K_0 = Stress ratio

OCR = Overconsolidation ratio in terms of vertical stress

ϕ = Friction angle of the material

This equation was used to estimate the vertical drained unloading required at the end of the one-dimensional compression stage to bring the sample to an isotropic stress state. It was thought desirable to bring the sample to this state because it would then, in theory, undergo no further effective stress changes during the extrusion giving greater control over the

preparation process. It also appreciably lowered the effective stress level within the bed. This reduced the tensions required in the pore water to sustain the effective stresses and decreased the possibility of air being drawn into the sample as a result of cavitation of the pore water. The change in void ratio during the unloading to a mean stress p' was calculated from Equation 3.8.

$$\Delta e = \kappa \ln\left(\frac{p'_{max}}{p'}\right) \qquad \text{(Eqn 3.8)}$$

where

p'_{max} = Maximum mean stress

3-4 This part of the stress path assumes that the unloading has not reduced the deviator stress to zero. This is the undrained (constant volume) unloading path associated with sample extrusion. If an ideal elastic response is assumed for the soil (which is reasonable given that the reduction in deviator stress is likely to be small) the path follows a constant p', constant e line.

4-5 A large isotropic pressure increment is applied to the sample in the triaxial chamber to bring the mean stress to its final value. The path is shown as hysteretic and joining the isotropic consolidation line at a mean stress higher than intersection of the current yield locus with the p' axis. This is a qualitative representation of what took place as it has yet to be quantitatively described.

5-B When the soil was fully consolidated at point **5** a deviatoric component of stress was applied to the clay bed. The deviator increment size was chosen to limit the stress ratio

experienced by the sample to the stress ratio of the final stress state (point **B** in this case). This was principally done to avoid failing the sample but also provided a coherent means of deciding the appropriate increment of deviator stress that should be applied at a particular stress state. The application of the deviator increment caused the soil to yield and generated pore pressures and in order that the stress ratio in the clay bed could be limited it was necessary to estimate the effective stress path. For a constant λ, which is generally known fairly accurately, the undrained effective stress path is dependent on the equation for the yield locus of the material and κ, the slope of the unloading path. For the particular total stress path used in this preparation technique the estimate of the final stress ratio reached decreases with increasing κ. It was found by Al-Tabbaa (1987) that the value of κ was highly dependent on the degree of overconsolidation with very low values generally associated with small degrees of overconsolidation. A conservative estimate of the stress path (one which predicted a higher final stress ratio) was made by taking κ equal to zero. When this is done the undrained stress path and yield locus coincide. They are given by the following expression:

$$\left(p' - \frac{p_c'}{2}\right)^2 + \frac{q^2}{M^2} = \left(\frac{p_c'}{2}\right)^2 \qquad \text{(Eqn 3.9)}$$

where

p_c' = Mean effective stress at the intersection of the current yield locus with the p' axis.

After the application of each deviator stress increment the pore water pressures were allowed to dissipate. Their dissipation was monitored using the method described in Section 3.4.5. The single stress path shown in Figure 3.12 and 3.13 implies that the pore water pressures completely dissipate between the application of increments of deviator stress. The presence

of residual excess pore pressures would have resulted in different stress paths for each level in the clay bed. Time constraints placed restrictions on the degree of consolidation which could be attained so there was in general some excess pore pressure when the increments of stress were applied. Six deviatoric stress increments are shown in Figures 3.12 and 3.13.

The preparation of other normally consolidated samples was carried out similarly. Isotropic final stress states were approached along the isotropic normal compression line. Anisotropic states were reached by applying deviatoric stress increments at the final mean stress level taking care to limit the pore water pressures generated.

In the preparation of the overconsolidated clay beds the above procedures were used to bring the sample to a normally consolidated state. Unloading was then carried out in increments at constant stress ratio. As with the procedure described above the pore pressures were allowed to re-establish their equilibrium between increments. The time required for this was comparatively short because the response to unloading was much stiffer than the response to loading. For sample OABA the unloading was complete within two days. In the preparation of clay bed OCDX the unloading in terms of mean stress was approximately double the unloading of beds OABA and OCDC and it took six days to complete. This was partly due to the increased unloading, but was also affected by the increasing coefficient of volume compressibility with *OCR*, Section 4.3.

It should be noted that the above preparation technique is not ideal; it, for instance, leads to overconsolidation in terms of vertical stress when normally consolidated samples are prepared with K=2.

Each of the clay beds experienced some radial straining. Figure 3.14 shows the average of four vertical profiles of the side of sample OE. These were measured during the investigation of the clay bed. The shape of the profile is typical of those recorded in the other clay beds.

The radial strain at the ends of the bed is less than the central radial strain suggesting individual portions of the sample have undergone differing amounts of shear strain. This is probably a consequence of dissimilar stress paths experienced by the elements of the clay bed during the consolidation process. The final form of the clay bed is a complex function of the shape of the yield locus, the shape of the plastic potential and the stress paths of individual elements. The variations in shear strain within the sample probably occurred in the isotropic stage as well as (where this was carried out) in the stage involving the application of deviatoric stress increments. In modified Cam-clay theory increments of isotropic stress applied to clay at an isotropic stress state only generate volumetric strains. However Al-Tabbaa (1987) shows that anisotropic consolidation can produce sample states in which shear strains can be generated by isotropic increments applied to isotropically stressed soil.

The variation in shear strain within the sample is associated with the application of increments of stress to the clay bed before the excess pore pressures from a previous increment have completely dissipated. Care was taken to reduce the importance of this effect by only applying stress increments after the pore pressures were judged to have dissipated sufficiently. Because of the association of shear strains with isotropic stress states and increments, it was also important to allow the complete consolidation of the clay bed during the one-dimensional compression stage. This was necessary to ensure that the mean stress increase throughout the sample during the isotropic compression stage was uniform. It is also possible that the restraining effects of the upper and lower boundaries contributed to the lower radial strains at the ends of the sample. For clay bed OE the average radial strain is calculated as 7.5 %. Table 3.2 gives the average radial strains for eight of the nine samples. Typically the radial strain increased with increasing K.

Accurate measurements of the second stage vertical consolidation over the area of the sample were not made but it was observed that they were generally uniform with radius. The final heights of the clay beds indicated that the vertical strain ranged from 0.5 % in sample OEF

to 9.7 % in sample OAB. Table 3.2 also summarises the second stage vertical strains and the volumetric strains implied by these. In all the clay beds the second-stage volumetric strain was greater than the intended 7 %. This was due to the simplicity of the analysis in comparison with the relatively complex second-stage stress path. The homogeneities of the clay beds which resulted from the preparation procedure described in this section are discussed in Section 4.2.

3.6 DILATOMETER TESTING

The cap to the boss which seals the central hole in the top lid (Figure 3.9) was removed immediately prior to the dilatometer test. The vyon filter was cut away to expose the clay and allow free movement of the dilatometer. The dilatometer was calibrated and then pushed into the clay bed at 20 mm/sec using the driving apparatus, Figure 3.10. The standard test, described in detail in Section 2.3, together with one C reading, was performed every 100 mm in the top 600 mm of the clay bed. The readings were collected manually. The two load cells and the displacement potentiometer were monitored continuously during this part of the testing.

As discussed in Section 2.5.7, work by Campanella et al. (1985) has indicated that, in clay, the closing pressure (the pressure when the membrane returns to the lift-off position) is approximately equal to the pore water pressure generated through installation. Powell & Uglow (1988) showed how closing pressure readings, taken over a period at one level, decay with time. They suggested that it may be possible to interpret this variation with time as a pore water pressure dissipation and thereby assess the coefficient of consolidation for the soil. There are two ways of taking the closing pressure reading: with and without an intermediate B reading. Both methods were tried in each sample. The tests were carried out below the central portion of the sample at the 600 mm and 800 mm depth levels as they were

considered to be of secondary importance to the standard tests conducted in the central and upper portion of the bed. Each test was conducted over 24 hours. After the completion of the testing the dilatometer was withdrawn and calibrated. No significant variations were found in the calibrations before and after testing in each of the nine clay beds.

CHAPTER 4

CLAY-BED PROPERTY EVALUATION

4.1 INTRODUCTION

A site investigation was carried out in each clay bed after the completion of the dilatometer testing, to establish the homogeneity, consolidation characteristics, strength and shear stiffness. Typically the chamber was dismantled and the site investigation carried out immediately after the removal of the dilatometer, but in samples OA, OAB and OABA hand vane soundings were conducted in the sample through three ports in the calibration chamber lid when it was still subjected to stress.

To eliminate, as far as possible, the absorbtion by the clay of free water upon destressing, the drainage lines to the chamber were closed and water standing in the ring surrounding the boss was removed. The pressures in the base and side membranes were equalised for the anisotropic samples at the level of the lower of the two stresses, and then reduced to zero isotropically. The total stress path, the ideal elastic unloading effective stress path and a qualitative prediction of the actual effective stress path experienced by sample OAB during the release of the deviator stress, are shown in Figure 4.1. Actual undrained unloading effective stress paths generally involve some mean stress relief and this is discussed in Section 4.4.2.

The membranes were emptied of water and the lid and chamber were removed leaving the clay bed standing on the chamber base. Samples of clay were immediately taken from the exposed top and side for water content evaluation. They were taken in a single vertical plane at 100 mm spacings across the top of the sample and at 100 mm depth intervals down the

side. The clay bed dimensions and side curvature were recorded. Shear vane tests were conducted with the centre of the blade at a depth of 100 mm. A 100 mm deep disc of clay was then cut from the top of the sample exposing the 100 mm level. Water content samples were again taken and vane tests were conducted in the next level 100 mm below. This procedure was continued down to the base of the clay bed. Where samples were required for triaxial tests, 200 mm long stainless steel tubes were pushed into the clay from the level above. Oedometer samples were taken by pushing a cutting ring into the clay and excavating around it. The results of the site investigations are discussed in detail in the following sections.

4.2 WATER CONTENTS

The water content of clay is strongly linked to its strength and other properties, and so a knowledge of it is helpful in assessing the clay's in-situ properties. A comprehensive set of samples was therefore taken from each clay bed for water content evaluation. It was assumed that each bed was symmetric about its central vertical axis and clay samples were taken on a grid pattern of 100 mm squares down one half. The reasonableness of assuming clay bed symmetry is discussed later. A representation of the sample positions within a typical clay bed is shown in Figure 4.2. The large number of samples taken for water content analysis made it impractical to follow precisely the method for moisture content determination set out in BS 1377 (1975). It was found that the use of open tins with approximately 60 g samples was reliable if the weighing was carried out quickly. The use of a cooling desiccator was also found to be unnecessary if the samples were taken from the oven and immediately weighed. It was estimated that a worst combination of weighing errors would allow predictions of the water content to be made with accuracies of ± 0.1 %.

For saturated clay a variation in its water content reflects a variation in its effective stress

level or stress history or both, prior to sampling. (Section 4.5 contains details of tests conducted on samples taken from each clay bed which show that an acceptable degree of saturation was achieved). Profiles of water content therefore identify differences between areas of the bed in terms of their former stress path and stress state. Figures 4.3a to 4.11a show half-profile water content contour plots for each clay bed. Typically each plot is made up of water content values at 66 points; 6 columns of 11 points, at radial spacings and depth intervals of 100 mm, as shown in Figure 4.2. The plots were obtained using the contouring programme 2CAN written by Prof. G. T. Houlsby.

In interpreting the contour plots the primary consideration was whether the sample state , as characterised by the water contents found in the clay bed investigation, was the same as the state of the sample during the dilatometer test. Changes from the water content distribution of the undisturbed bed can be divided into those that result from insertion of the dilatometer, and those that result from removal of the dilatometer and the release of the boundary stresses.

The insertion of the dilatometer into the clay bed is comparable to the driving of a pile into cohesive soil. The driving of a pile has been modelled by Wroth et al. (1979a) as the creation of a long cylindrical cavity by radial soil movement. During the driving process large pore pressures are generated and the soil close to the pile is remoulded. Once installed the rigidity of the pile constrains the soil in contact with it in the axial and circumferential directions and only consolidation in the radial direction can take place. When the excess pore pressures have dissipated the soil close to the pile is left with a substantially increased lateral effective stress. It can be supposed that roughly the same basic processes of cavity creation and radial soil consolidation take place with the dilatometer and its connecting rods. The drainage lines are open during the dilatometer penetration so water is free to move out of and into the clay bed. Figures 4.3a to 4.11a show clearly the consolidated, lower water content area surrounding the position where the dilatometer penetrated. Outside this area and beneath the level to which the dilatometer was driven the clay beds retain a reasonably uniform

distribution of water content.

Removal of the dilatometer and then reduction of the boundary stresses causes a stress redistribution within the clay bed. Free water is taken up where it is available. It can been seen in the water content contour plots for clay beds OCDX, OA, OABA, OE and OEF, Figures 4.6a, 4.7a, 4.9a, 4.10a and 4.11a respectively, that the water available at the top lid opening (top left) has been absorbed by the surrounding clay. When the clay surface is in contact with the atmosphere there is also the possibility of air moving into the soil if the air entry value for the clay-air interface is exceeded. This is dependent on the differential pressure between the atmosphere and the pore water and hence on the magnitude of the stress level from which the sample has been unloaded. Air moving into the soil from the periphery would modify the water content distribution and relieve the capillary tensions. Kirkpatrick & Khan (1984b) found that the importance of this effect increases with time and suggested that it was probably dependent on the swelling coefficient for the clay.

Figure 4.12 shows a contour plot of the water contents found on a plan section through clay bed OCDX at the 500 mm depth level. The plot shows that there is a degree of asymmetry in the variation of the water content with radius. In Figure 4.13 the water contents used to produce Figure 4.12 have been averaged to find a typical variation of water content with radius. Each point is the average of 16 water content measurements except for the point at zero radius which is a single measurement. The upper and lower limits to the water content at each radius are also indicated. The range in water content at a particular radius was ± 1.0 % which is far greater than the 0.1% conceivable from weighing errors. The contour plots in Figures 4.3a to 4.11a should therefore be considered as examples of water content profiles rather than standard profiles.

Figure 4.13 shows that excepting the centrally disturbed zone there is a trend of decreasing water content with increasing radius. It is believed that this is a feature of the preparation

procedure as it exists to some degree in all the clay beds, and is not the result of water moving after the release of the boundary stresses. The evidence for this is provided by the general water content symmetry shown between the upper and lower halves in most of the clay beds. It is reasonable to suppose that any change in the water content would increase with time as it is dependent on the flow of water. In the top and bottom half of each clay bed, the samples for water content analysis were taken at significantly different times after the release of the boundary stresses (each level took approximately 45 minutes to examine) but the variation in water content between positions which are symmetric about the mid-height is negligible.

It is concluded that the effect of the release of the boundary stresses on the water content was probably negligible, and it is reasonable to assume that the measured water contents outside the centrally disturbed zone are representative of those that existed in the sample prior to testing.

The water content distribution resulting from the method of preparation employed here showed that reasonably homogeneous sample were produced. However there was a detectable trend for the water content to increase towards the centre of the sample. The mechanism behind the development of this in one-dimensionally consolidated samples has been explained and modelled by Nageswaran & Houlsby (1982). If radial drainage takes place in a sample undergoing vertical one-dimensional consolidation using a rigid top platen, a variation across any horizontal plane of the proportion of the total stress which is effective, is set up. The regions of higher effective stress are stiffer and therefore take a disproportionate amount of the load. In consequence after equilibrium has been reached between the load applied through the platen and the resistance offered by the clay bed, there is a variation of the effective stress distribution throughout the soil body and hence a variation in the water content. An initial variation in the water content of the clay, without any radial drainage, would produce a similar effect. The variations in water content set up during the first one-dimensional stage

of consolidation would be only partly erased during the triaxial phase of consolidation where stresses are applied through flexible boundaries. Given the form of the water content distribution it is most likely that it was caused by some of the drainage taking place in the radial direction. This would have been facilitated by the discontinuity between the GRP tube wall and the sample during the first stage of consolidation, and by creases in the lateral membrane (caused by the reduction in the radius of the clay bed) during the second stage.

4.3 OEDOMETER TESTS

4.3.1 Test Procedure

Two samples were obtained for oedometer testing from each clay bed; one from each of the levels at which repeated dilatometer readings were carried out. The samples were taken by pushing an oedometer cutting ring into the clay bed and then carefully excavating around it. The trimmings were collected for water content evaluation. The rings were then mounted in oedometer cells with filter papers and porous stones at either end. The oedometer cells were placed in loading frames with seating loads equivalent to between 25 and 50 kPa. The seating loads were increased to give a stress of approximately 100 kPa in later tests because of fears that the unloading was too large and was destroying too much of the sample's preconsolidation structure. However little difference was observed in the compression behaviour as a result of this.

The samples were left for a day to swell under the seating load and were then loaded in stages to stresses in excess of their vertical preconsolidation pressures. In each stage the load was approximately doubled and was left for 24 hours. Unloading was also carried out in stages with the load being halved each time. During the first ten minutes of each loading stage the displacements given by a dial gauge and the corresponding time were recorded at frequent intervals. Compliance movements were subtracted from the measured displacements.

A typical plot of sample displacement against the square root of time during a loading stage, is shown in Figure 4.14. The coefficient of consolidation was obtained from similar plots for each loading stage, using the graphical construction of the root-time method, Craig (1990). This was also used to find the coefficient of swelling for each unloading stage.

Before the oedometer cell was disassembled the stress on the samples was reduced to approximately 20 kPa which was as low as was compatible with keeping the creep, which was observed to increase with increasing unloading, to acceptable levels. At this stress level the coefficient of swelling was small which helped to preserve the appropriate water content during the dismantling process. Even so, the oedometer was taken apart as quickly as possible and two to three millimetres thickness of clay was cut away from the upper and lower surfaces of the clay disc to eliminate any water absorbed from the estimation of the final water content.

A void ratio versus vertical effective stress plot can be determined for each oedometer test using the initial height of the sample and the final water content. The movement of the samples was monitored during the swelling stage under the seating load, but there was generally poor agreement between the expansions measured in the two samples from each clay bed; even when slight differences in the loads were taken into account. In consequence, different initial heights and different initial void ratios were calculated for each test resulting in appreciably different void ratio-vertical stress plots for supposedly similar clay samples. However, it was found that the agreement between the positions of the two plots for each clay bed could be improved if the initial water content from the trimmings was used. By comparing the void ratio after swelling, with the void ratio, calculated from the trimmings, the initial height of the sample could be calculated. To calculate the void ratio after swelling the initial height of the sample is required, so it was necessary to follow through an iterative procedure. A significantly improved agreement between the initial heights of the samples from each bed was achieved using this method.

4.3.2 Results

Figures 4.15 shows a pair of e : ln σ_v' curves obtained from oedometer tests on samples taken from clay bed OC. It can be seen that there is good agreement between the position and slope of the two curves. However, there is no distinct change in slope at the sample's vertical preconsolidation pressure, only a gradual steepening as the pressure increases. In common with standard oedometer tests, the samples were subject to disturbance and unloading during the setting up procedure which would have the effect of making the yield point less marked. There are two additional factors in these tests which combine to make the yield point even less distinct: the preparation stress path of the clay bed is different from that experienced by the oedometer samples and the sample was only subjected to its maximum vertical stress for a relatively short time. In general these two factors are not present in tests on natural clays where a conspicuous preconsolidation pressure is often found.

It can be seen in Figure 4.16 that when the samples are stressed significantly beyond their preconsolidation pressures the slopes and positions of all the loading e : ln σ_v' curves approach a common line. This line is given by the following:

$$e = e_{1\text{-}d} - \lambda \ln p' \qquad \text{(Eqn 4.1)}$$

with $e_{1\text{-}d}$ = 2.099 and λ = 0.174. The mean effective stress, p', is equal to σ_v' (1 + 2 K_0)/3 with K_0 = 0.69. Al-Tabbaa (1987) found an equivalent line for a similar range of pressures with $e_{1\text{-}d}$ = 2.130 and λ = 0.187. In modified Cam-clay the unloading lines are taken to be straight in the e : lnp' plane. Figure 4.17 shows the unloading portions

of all the oedometer curves in this plane. Because of the one-dimensional nature of the movements in the oedometer, the stress ratio changed continuously during unloading and so to calculate the appropriate mean stress it was necessary to estimate the stress ratio at each point. This was done using Equation 3.7. It can be seen that, except for the first part of unloading, the curves are consistent with the modified Cam-clay model, as they can be characterised by a single slope equivalent to $\kappa = 0.05$. For a small region of unloading a lower value of κ, equal to 0.03, appears to be more appropriate.

The coefficient of consolidation was derived for each loading increment and the coefficient of swelling for each unloading increment, using the root-time-method, Craig (1990). This method is based on two empirical equations (4.2 and 4.3) which closely represent the relationship between the degree of consolidation, U, and the time factor T $(=c_v t/d^2)$.

for $U < 60$ %

$$T = \frac{\pi}{4} U^2 \qquad \text{(Eqn 4.2)}$$

for $U > 60$ %

$$T = -0.933 \log(1-U) - 0.085 \qquad \text{(Eqn 4.3)}$$

For a degree of consolidation of 90 %, the square root of the time factor given by Equation 4.3 is 15 % greater than that given by Equation 4.2. Use of this result is made by plotting the displacement (which for a constant stiffness is related to the degree of consolidation) against the square root of time. This allows the graphical construction shown in Figure 4.14, to be used to calculate the time for 90 % consolidation, t_{90}. The time factor for a degree of consolidation 90 %, is given by Equation 4.3 as 0.848 and hence the coefficient of consolidation or swelling, can be determined.

The coefficient of consolidation is defined as follows:

$$c_v = \frac{k}{\gamma_w m_v} \qquad \text{(Eqn 4.4)}$$

where

k = Permeability

γ_w = Unit weight of water

m_v = Coefficient of volume compressibility (= volumetric strain / change in stress)

The coefficient of swelling is defined similarly. It is possible using the root-time method to explore the relationship between void ratio and permeability. A value for m_v can be found by considering the change in vertical stress and the resulting change in height during each loading and unloading stage. The displacement versus the square root of time plots, are used for this as they allow the changes in height due to primary and secondary consolidation to be separated. It is only the change in height connected with primary consolidation which is relevant to the calculation of m_v. With c_v and m_v for each stage the permeability was calculated from Equation 4.4. Figure 4.18 shows the permeability calculated in this way against the average void ratio of the loading stage. There is good deal of scatter but there is a clear relationship between the two properties which appears to be independent of whether the soil is being loaded or unloaded. Also shown on the plot is the relationship between the permeability and void ratio for speswhite kaolin found by Al-Tabbaa (1987) using a combination of falling head and consolidation tests.

Figure 4.19 shows a plot of m_v against the overconsolidation ratio. In this plot the overconsolidation ratio is equal to σ'_{vmax}/σ'_v but with a slightly different definition: the maximum previous vertical stress, σ'_{vmax}, retains its value when it is exceeded during loading to allow examination of the gradual, rather than sharp, change in behaviour; and the existing

vertical stress is defined as the logarithmic mean of the initial and final increment stresses. A distinction has been made between the unloading and loading values of m_v. The plot shows that m_v during loading is relatively constant with overconsolidation ratio and does not increase when the preconsolidation pressure is exceeded. It is believed that this is due to the counteraction of two effects: there is a tendency with soil for the stiffness to increase with increasing pressure and a converse tendency for the stiffness to decrease when the soil yields. It appears that in these tests the two effects have cancelled each other out to give a stable value for m_v. For the unloading stages these two effects are added. The pressure and therefore stiffness decrease during unloading and there is also some hysteretic yielding of the soil which again causes the stiffness to decrease. As a result of this m_v shows a rapid increase with increasing degree of overconsolidation.

The overall effect of the variation of permeability with void ratio and the variation of m_v with degree of overconsolidation, on the variation of c_v with overconsolidation is presented in Figure 4.20 for the loading stages and Figure 4.21 for the unloading stages. The same definition of overconsolidation ratio applies to these plots as was described earlier for Figure 4.19. In Figure 4.20 it can be seen that the coefficient of consolidation for loading remains relatively constant throughout the reloading and virgin loading of the samples. In contrast the unloading coefficient of consolidation shows a rapid drop as the degree of overconsolidation of the sample increases. This is directly attributable to a corresponding increase in the coefficient of volume compressibility.

4.4 SHEAR VANE TESTS

The undrained strength distribution throughout each test bed was measured using a hand vane.

The hand vane consists of four rectangular blades set at right angles at the end of a central rod to make a cruciform section. The rod is connected to a spring which is mounted inside a metal casing. An arm on the top of the casing indicates the rotation of the rod relative to the casing and thus the torque being applied to the rod. When inserted into soil and rotated the maximum torque that the vane can sustain is related to the strength of the soil. Every 100 mm down the height of each clay bed vane tests were conducted at distances of 100, 200, 300 and 400 mm from the centre of the clay bed along three radii spaced at 120°. Thus twelve readings were obtained at each of the, typically, nine levels in each clay bed.

The mechanism of failure in the vane test is relatively complex and in consequence the results are sensitive to the test procedure. A procedure was therefore devised which it was felt would yield consistent and relevant results. A discussion of this and the reasons behind its adoption follows.

4.4.1 Test Procedure

The size of the vane tip was limited to one 30 mm in height and 20 mm in diameter, by the maximum torque that could be registered by the hand vane apparatus and the maximum expected clay strength. The bulk of analysis and literature on the vane test deals with field vanes which typically have tips 130 mm in height and 65 mm in diameter. In order that the conclusions arrived at in the literature regarding the comparison of the results of the vane test with the results of other types of test, could broadly be applied to the hand vane results, a test procedure was adopted in which an attempt was made to eliminate, as far as possible, the important differences between the two different sizes of tests. These differences can be divided into disturbance, viscous, consolidation and rod friction effects.

4.4.1.1 Disturbance Effects.

Roy & Leblanc (1988) showed that the effect of the disturbance, caused by the insertion of the vane, on the measured undrained strength varies with the properties of the clay and is most severe in sensitive clays. For a particular clay the disturbance (inferred from the change in measured undrained strength) was related to the thickness of the edge of the blade and the blade cross-sectional area. In reducing the size of the vane to allow it to be used in hand operations it is not possible to reduce the blade thickness in proportion to the overall reduction in scale because the blades would then be too flexible. In consequence the area ratio of the hand vane (that is the ratio of the horizontal areas of the vane and the cylinder of soil being sheared) is 21% compared to 8% for a typical field vane and the maximum of 12% recommended in BS 1377 (1975). The area ratio is strongly related to insertion induced disturbance and a reduction in the measured strength which was greater than the 10% typically found in field vanes, Chandler (1988), was therefore expected.

4.4.1.2 Viscous Effects.

The predominant deformation mechanism in the vane test is one of simple shear around the vertical periphery of the vane, Chandler (1988). Errors which may arise from the use of different diameter vanes due to strain rate effects can be reduced by conducting the tests at rotation rates which equalise the vane tip velocities. The rotation rate for the field shear vane is generally 6°/min. This is comparable to a rotation rate for the smaller hand vane tip of 20°/min. The hand vane requires casing rotation relative to the tip to generate torque and the soil also has a rotational stiffness of its own. Tests conducted by Rush (1974) in kaolin, suggested an average build up of shear stress with rotation according to the following equation:

$$\theta = M_{\theta}\, s \qquad \text{(Eqn 4.5)}$$

where

θ = Vane tip rotation in degrees

M_θ = A rotational stiffness constant

s = The soil shear stress

For speswhite kaolin M_θ was roughly 0.125 m^2/kN. Applying the same equation to the hand vane body gives a value for M_θ of 1.89 m^2/kN. Combining the stiffness of the soil and the vane apparatus with the required tip rotation rate of 20°/min gives a vane casing rotation rate of 325°/min. For simplicity, in all the vane tests conducted the casing was rotated to keep pace with the second hand of a clock i.e. at 360°/min. It should be noted that this approach only produced a reasonably consistent rotation rate for the vane tip in the initial elastic portion of the stress-rotation curve. Beyond yield the stiffness reduced significantly causing the rate of rotation for the vane tip to rise. At failure, where the stiffness of the soil is reduced to zero, the rotation rate for the vane tip was 360°/min. Despite these inconsistencies, within the limitations of a stress-controlled test, this approach provides a reasonably stable rate of rotation for much of the stress-rotation curve.

4.4.1.3 Consolidation Effects.

Torstensson (1977) investigated how the length of the delay between vane insertion and the shearing of the soil affected the measured strength. He found that for some time after insertion there was a gradual increase in strength until an increase of about 20 % above the initial strength value was reached and thereafter it remained constant with time. This phenomenon was attributed to the dissipation of pore pressures set up by the insertion of the vane. The time taken before a constant strength was recorded was related to the time taken for these to dissipate. Movement of water out of the disturbed soil surrounding the vane left it with a lower water content and therefore an increased undrained strength.

In the interpretation of the shear vane it is assumed that failure of the soil is achieved before there is any significant dissipation of the pore pressures. The rate of rotation should be high enough to ensure an undrained and not partially drained strength is obtained, but slow enough to avoid too great a viscous increase in strength. Chandler (1988) gives details of an approximate theory, supported by experimental evidence, which allows the determination of the rate of rotation required to ensure that undrained conditions exist up to the time of failure. The theory leads to the following equation:

$$T = \frac{c_v \, t_f}{d_v^2} \qquad \text{(Eqn 4.6)}$$

where

T = Time factor

c_v = Coefficient of consolidation of the soil

d_v = Diameter of the vane

t_f = Time to failure

With a coefficient of consolidation of 0.8 mm²/sec (Figure 4.20) and time to failure of 30 seconds (this assumes failure after 10° of rotation) the time factor at failure is 0.066. Using the theoretical relationship cited by Chandler (1988) this corresponds to a degree of consolidation of around 10 % which can be regarded as undrained. The rate of vane tip rotation of 20°/min is fast enough, therefore, to ensure that undrained conditions prevail at the time of failure.

4.4.1.4 Rod Friction Effects

To eliminate soil friction on the rod through which torque is applied to the field vane, the rod

is generally enclosed in a sleeve. The hand vane is used without a sleeve as the depth of penetration is usually small and a sufficiently short length of rod is subject to clay friction to make the loss of torque negligible. However, in this investigation hand vane tests were conducted with the centre of the vane tip 100 mm below the clay surface leaving 86 mm of rod exposed to friction from the clay. The possible importance of the rod friction effect was revealed by presuming that the failure surface was at the interface of the rod and clay, and that the shear strength of the soil was fully mobilised on this surface. When this was done it was found that potentially 21 % of the registered torque could originate from this source, which would lead to a significant overestimation of the soil strength.

Experiments conducted by the author indicated that when a rod was inserted into a clay bed and then immediately rotated, a shear resistance was mobilised equal to roughly 20 % of the strength measured using the vane in the overconsolidated samples and between 0 and 7 % in the normally consolidated samples. Again there is an assumption in these amounts that a cylindrical failure mechanism was operating. The justification for this assumption is provided by Karube et al. (1988) who found experimentally that approximately the same shear resistance was mobilized in the vane test as in the rotation of a rough cylinder of the same diameter. In order to eliminate the effects of installation the experiments were conducted on pre-placed vanes and cylinders. It can be presumed from this result that the mechanism of failure in the rotation of a vane and a cylinder is similar and dependent on the soil strength across the failure surface. In the cylinder this would be at or near the interface between the cylinder and the clay.

Installation disturbance, which one could expect to be different for the vane and the rod influences the effective stress level and hence strength along the potential failure surface. Hence, though the mechanisms of failure in the vane and cylinder are similar the strengths derived from analysis of the peak torque are different. It was noted that the rod friction dropped slightly after failure and then gradually increased with time as the rod was rotated.

This effect was attributed to the consolidation of the disturbed clay at the failure surface.

It was felt that it was reasonable to use the rod friction obtained from these tests to correct the hand vane tests. It was recognised that the peak strengths in both cases were unlikely to occur at the same angle of rotation, and also that the presence of the vane tip would result in a difference in the form of the disturbance around the rod between the tests with and without the vane tip. An allowance for rod friction equal to 0.23 s_u and 0.18 s_u was made in the interpretation of the vane tests in the beds OABA and OCDX respectively (Section 4.4.3). No measurements of rod friction were made in bed OCDC, but to be consistent with the other overconsolidated beds a correction was made equal to 0.2 s_u. It was considered that it was unnecessary to make an allowance for friction in the normally consolidated tests.

4.4.2 Stress Relief Effects

The release, under undrained conditions, of the boundary stresses acting on the clay bed would, if the clay behaved elastically, set up capillary tensions equal to the mean effective stress, p'. The water content and soil micro-structure would remain unchanged and one could expect there to be agreement between the results of strength tests conducted in the stressed and unstressed clay. In reality only a proportion of the capillary tensions required to preserve the mean effective stress are produced. This is due to the deviation from ideal elastic behaviour of the soil on unloading, Figure 4.1. As discussed in Section 4.2 if the clay is open to the atmosphere there is also the possibility of air moving into the soil altering the water content and reducing the capillary tensions further. These processes are complex and in consequence their effect on the undrained strength has to be allowed for empirically.

In clay beds OA, OAB and OABA vane tests were conducted before and after the release of the boundary stresses to assess the importance of stress relief effects. The tests on the

stressed samples were carried out through three port holes in the lid of the chamber using the hand vane fitted with a 1.0 m unsleeved rod. The rod friction was corrected for by carrying out one test with the rod alone. Figure 4.22 shows the results of the three rod tests conducted in beds OA, OAB and OABA in order to measure and allow for rod friction effects in the vane soundings. The figure shows a plot of the measured torque against the depth of rod penetration. The rod was advanced 100 mm and then rotated until the peak torque was recorded, and then advanced again increasing the overall length of rod subject to soil friction by 100 mm each time. The readings were taken at roughly 30 second intervals which was the time between penetration and failure for a typical vane test. For a constant mobilized strength around the perimeter of the rod, one would expect a linear relationship between the depth of rod penetration and torque. However a linear relationship was achieved only after three to four readings. It is believed that this was a result of variations in the pore pressure distribution along the rod due to differences in the times at which each layer was penetrated. If for simplicity a uniform average strength profile is taken to exist, the slopes of the lines in Figure 4.22 indicate that strengths existed around the rod equivalent to 40 %, 44 % and 54 % of the average undrained strengths for clay beds OA, OAB and OABA respectively.

Correcting for the rod friction instead of eliminating it inevitably led to greater scatter in the measured vane strengths but the general trend of strength loss after release of the boundary stresses is clearly apparent in Figure 4.23, which shows the vane strengths of the stressed sample plotted against the equivalent strengths in the unstressed sample. The calculation of these strengths is described in the next section. The vane soundings in the stressed sample were conducted on a pitch circle diameter of 500 mm and so the equivalent strengths were calculated by averaging the hand vane results at the same elevation at 200 and 300 mm from the centre of the clay bed. The difference between the two strengths is greatest for clay bed OAB which was unloaded from the highest mean stress level.

Kirkpatrick & Khan (1984b) have examined the influence of stress relief on the vane strength

of clays and relate the strength loss to the sample age. Their results using kaolin suggest that for the clay bed investigation, where the sample age was typically between one and seven hours, one should expect between a 30 % and 50 % loss in strength. The results shown in Figure 4.23 give an average strength loss of 12%. The discrepancy can be attributed to the presence of disturbance effects in the tests by Kirkpatrick & Khan (1984b) where samples were taken from the stress relieved clay using thin-walled tubes. This leads to the internal migration over time of water from remoulded clay in the disturbed zone at the edge of the sample. Arman & McManis (1977) showed how the decrease of undrained strength with age for tube samples was absent from block samples and ascribed this to the lower soil disturbance involved in taking block samples. Hight et al. (1985) also did not find an influence of storage time on the ambient stress level within samples of a low plasticity clay.

4.4.3 Undrained Strength Distributions

Three principal assumptions are made in the interpretation of the vane shear results:

- The soil disturbance resulting from the insertion of the vane is negligible.

- The cylindrical path swept out by the edges of the vane blades as they rotate coincides with the failure surface.

- The same shear strength is simultaneously mobilized over the entire failure surface.

It is difficult to quantify the effect of soil disturbance on the results of the vane shear test. Roy & Leblanc (1988) have correlated the disturbance due to vane insertion, via the measured undrained strength, to the perimeter ratio which is defined as follows:

$$P_r = \frac{4\ t_b}{\pi\ d_v} \qquad \text{(Eqn 4.7)}$$

where

P_r = Perimeter ratio

t_b = Edge blade thickness

d_v = Diameter of vane

The perimeter ratio for the vane tip used in this study was 10.4 % which compares unfavourably with the perimeter ratio of 2 % for the standard vane. If the results of Roy & Leblanc for a silty clay are extrapolated they suggest a 20 to 25 % loss in strength, compared with the standard vane, for the perimeter ratio of the hand vane. The results are for a natural clay under field conditions but they indicate the likelihood of a significant underestimation of strength from the vane tests conducted in this study.

Roy & Lablanc (1988) identify the failure surface from the results of X-ray photographs as a thin disturbed layer outside the surface circumscribed by the blade ends. The position of the observed failure surface does not diverge sufficiently from that assumed in the standard analysis to justify a more sophisticated analysis.

The interpretation of the vane is very sensitive to the assumptions made regarding the stress-strain properties of the clay. Theoretical and experimental studies Wroth (1984) confirm that the assumption of a uniform stress distribution on the vertical failure plane is reasonable. However the distribution on the horizontal planes is highly non-uniform with stress peaks at the blade edges. Wroth, in an analysis which uses a polynomial distribution of stress on the horizontal planes which is characteristic of the distributions found experimentally, derives an equation for the undrained strength which yields a value 10 % higher than that from the conventional equation. The strength obtained using either analysis is dominated by the

strength on the vertical planes. The data were not available to support the use of a more complex failure mechanism than the conventional one, in which the peak strength is assumed to be mobilised instantaneously over the entire failure plane. The standard analysis in which a correction was made for the rod friction, Equation 4.8, was therefore used to interpret the hand vane results.

$$T_q = \pi s_u \left(\frac{d_v^3}{6} + \frac{d_v^2 h_v}{2} + \frac{\alpha t_r^2 s_r}{2} \right) \qquad \text{(Eqn 4.8)}$$

where

T_q = Torque

d_v = Diameter of the vane (19.1 mm)

h_v = Height of vane (28.6 mm)

t_r = Diameter of vane rod (6.3 mm)

s_r = Length of rod embedded in clay

α = Proportion of strength mobilised around rod (roughly 0.2 for overconsolidated clay and 0 for normally consolidated clay)

For every level in the clay bed, the means of the three hand vane readings on each of the 100, 200, 300 and 400 mm radius circles, were found. These were used with Equation 4.8, to find average values for the undrained strength. The distributions of these average undrained strengths throughout each clay bed are shown in the contour plots in Figures 4.3b to 4.11b. The contour interval was chosen for each plot, as approximately ten percent of the average undrained strength. The plots provide evidence, in addition to that from the water content plots, that the clay beds were reasonably homogeneous. However, it should be noted that each point making up the profiles of undrained strength is the average of three readings, and so some differences should be expected between the profiles of water content (which are

made up of individual readings) and those of undrained strength.

In Chapter 5 the dilatometer reading at each level in the clay bed is compared with the undrained strength. The undrained strength in this case was calculated from nine readings, three each at 200, 300 and 400 mm from the centre of the clay bed. In Figure 4.24 these strengths are shown plotted against the average of the three water content readings from the same level at 200, 300 and 400 mm from the centre of the clay bed. A clear dependence of the undrained strength on the water content is shown with the trend of the strengths from the overconsolidated beds being slightly higher than that from the normally consolidated beds. Skempton & Sowa (1963) in a series of triaxial tests, found that though stress relief during undrained unloading was present for normally consolidated samples, overconsolidated samples showed near elastic behaviour with little loss in mean effective stress. The amount of stress relief influences the amount of strength loss and this may provide an explanation of why the overconsolidated clay beds appeared to have slightly higher strengths for a given water content than the normally consolidated clay beds; though the generation of lower pore pressures during insertion into the overconsolidated beds is another possibility.

Also shown in Figure 4.24 are the undrained strengths from the vane tests conducted in the still stressed clay beds. Each strength is the average of two readings and is plotted against the average of the water contents at radii of 200 and 300 mm. These tests show slightly higher strengths than those obtained from the unstressed samples. The investigation of the clay beds involved little prior disturbance in the vicinity of the vane tests and presumably therefore little effect on the measured vane strength. There were no variations in strength throughout the body of clay beds which could not be explained by the water content variations; and as it was concluded in Section 4.2 that the water content distribution found in the investigation is that of the stressed sample, one can attribute strength difference between the stressed and unstressed clay bed to the effects of stress relief alone.

4.5 TRIAXIAL TESTS

The non-linear behaviour of soil makes it impossible to define a shear modulus which is relevant to every situation. For instance the soil behaviour is idealised to incorporate a constant shear modulus in the cavity expansion analysis of the pressuremeter test, and in the analysis of the displacement under a footing. The appropriate shear modulus in each case is different, because the strain level, stress path and idealisations involved, are different. Also, even when these important conditions are simulated in the laboratory it is difficult to reproduce the same behaviour because of the extreme sensitivity of the shear modulus to the method of measurement employed. However, the significance of the shear modulus in influencing soil behaviour is such, that even if a precise and entirely relevant value cannot be measured, it is desirable to have some assessment of it.

In investigating whether the dilatometer readings can be related to the shear modulus it is important, therefore, to measure as far as possible, an appropriate modulus. There is some evidence (Chapter 2) that the forces against the dilatometer blade after penetration are the result of an unloading or reloading mechanism. Fortunately an unload-reload shear modulus can be determined in the triaxial test relatively easily and according to Davidson (1980), more consistently than the initial loading modulus.

Approximately 50 consolidated undrained triaxial tests, in which an unloading and reloading loop was performed, were carried out on samples taken from eight of the nine clay beds. A smaller number of unconsolidated undrained tests were also carried out. No triaxial tests were performed on samples taken from bed OCDX because of a failure of the triaxial apparatus.

4.5.1 Test Apparatus and Procedure

Samples for triaxial testing were taken from the central portion of each clay bed on a pitch

circle diameter of 500 mm. From the water content distribution plots shown in Figures 4.3a to 4.11a, it can be seen that this position is a compromise between avoiding the central disturbed area, and taking samples which are representative of the soil that existed prior to the dilatometer test. The sampling was carried out using 200 mm lengths of 38 mm i.d. stainless steel tubing. The tube walls were seamless but were 1.6 mm thick, and so to reduce the disturbance to the samples, a 2.5° cutting edge was machined on one end of each tube. To aid sampling and extrusion the tubes were coated inside and outside with silicone oil prior to being pushed vertically into the clay bed. The tubes were excavated from the clay bed as the investigation progressed and, to prevent the samples drying, the ends of the tubes were immediately sealed with wax. The tubes were then wrapped in cellophane, placed inside resealable polythene bags, and stored in a cool basement until they were tested.

When they were required for testing the samples were extruded using a solid rod, just smaller in diameter than the internal diameter of the sampling tubes. The ends of each sample were cut off and the remainder was placed in a former and trimmed to a length of 76 mm with a cutting wire. The former helped to ensure that the end faces were perpendicular to the vertical axis of the sample. The trimmings were used to establish the water content of each sample. The effectiveness of the measures taken to stop the loss of water during storage can be judged from Figure 4.25 which shows a plot of the water content of the trimmings from each triaxial sample against the water content of the clay bed at the same radius and elevation from which the sample originated. It can be seen that for nearly all the samples that there is close agreement between the two water contents. The moderate scatter can be attributed to the asymmetry of the clay beds about their central axes as shown in Figure 4.12.

The electronic instrumentation and apparatus used in the triaxial tests are shown in Figure 4.26. The instrumentation comprises one load transducer, one LVDT and three pressure transducers. The load transducer was fully submersible and was situated inside the triaxial cell to eliminate errors from ram friction. The LVDT was used to monitor the movement of

the ram relative to the body of the cell. One of the pressure transducers was used to monitor the cell water pressure and the other two were connected to the top and base drainage lines. When consolidation was carried out the volume of pore water expelled from each sample was measured using a burette. Deaired water was used in the drainage lines. The cell pressure and back pressure in the drainage lines, were controlled by regulating the compressed air supplying two air-water interfaces. To avoid air coming into contact with the deaired water the air and water in the interface connected to the drainage lines were separated by an expandable rubber bag. The triaxial cell was seated on the platen of a Tritest 50 kN load frame. A rate control mechanism on the machine enabled the operator to change the speed and direction of platen movement and hence the loading on the sample.

The data logging system used for the triaxial tests was very similar to the system which was used to monitor and record the instrumentation in the dilatometer tests, (Section 3.4.8).

The instrumentation was connected to a power supply box using six-core shielded cables. These carried the voltage supply to, and the voltage output back, from each transducer. A Solartron 7010 Minate Scanning Unit was attached to the transducer outputs at the back of the power supply box. Each transducer output was connected, using a twin core shielded cable, to an individual channel connector on the front of the scanning unit. A Solartron 7060 digital voltmeter was used to read the output from each transducer as it was scanned. This was linked to an IBM XT personal computer fitted with an IEEE interface communications board. The computer, as well as providing a means of control, was used to convert the digitised voltages from the transducers into pressures or displacements and also to combine them to obtain parameters such as the stress ratio and axial stress. These were displayed on the computer screen so that they could be used as a guide to the active control during the preparation and testing of each sample.

The controlling software was a modified version of a programme originally written by Mr D.

G. Boden. The programme was used primarily to scan the channels and write the data to disk, but it could also be used, without writing to disk, to scan selected channels at a high rate. This secondary mode was used when the triaxial tests were being set up. The channels were scanned every seven seconds during a typical triaxial test.

Two types of triaxial test were conducted: unconsolidated undrained tests and consolidated undrained tests. In both types of test the same initial setting up procedures were used. Air was removed from the porous stones by placing them in boiling water for ten minutes and the drainage lines were flushed to dislodge any air bubbles. A meniscus of water was formed on the pedestal at the base of the triaxial cell. The samples were then carefully positioned on the pedestal with porous stones and filter papers at either end. A rubber membrane was slipped over the sample and secured to the pedestal and cap with o-rings. The cell was filled with water through its base and the lid was bolted down. The cell pressure was raised to approximately 120 kPa, and the drainage lines were flushed again with a back pressure of 100 kPa. At this point the procedures that were subsequently followed in the two types of triaxial test diverged.

In the unconsolidated undrained tests the drainage lines were immediately closed and the cell pressure was raised to 100 kPa above the sample's previous mean effective stress level in the clay bed. The change in pore pressure accompanying the application of the cell pressure generally yielded a value for the pore pressure parameter $\underline{B}$ of between 1.00 and 0.97, which is indicative of saturated behaviour. In many of the cases where a poorer response was observed, the water content of the sample was less than the water content of the clay bed at the same depth and radius by more than two percent, suggesting that partial drying and desaturation had taken place during storage. Overall the measured pore pressure response demonstrated that the clay beds were saturated. The base of the loading ram was brought into contact with the cap of the sample by raising the platen by hand winding whilst observing the output display for the load on the computer screen. Great care was taken to avoid pre-loading

the samples. The speed control for the platen was set at 5 mm/min and the sample was sheared until the load began to drop off. With the drainage connections to the sample still closed the apparatus was dismantled and a specimen was taken from the centre of the sample to determine the water content.

In the consolidated undrained tests the drainage lines were left open for 24 hours with a back pressure of 100 kPa and the cell pressure at 100 kPa above the sample's previous mean effective stress level. With a value for c_v of 0.8 mm²/s it was calculated that 90 % of the reconsolidation would be achieved within one hour. The back pressure helped to improve the saturation of the system by encouraging the absorption by the deaired water of any concealed air bubbles. The volume of water expelled from the sample during reconsolidation was measured using a burette connected in series with the drainage lines. The drainage connections to the samples were closed just before the samples were loaded. During the shearing of each sample an unload-reload loop was carried out to establish the elastic properties. The samples were loaded at a slower rate than was used in the unconsolidated undrained tests to allow greater equalisation between the water pressures in the zone of shearing and in the drainage lines connecting to the pressure transducers. The samples were sheared at 0.3 mm/min until the stress ratio η equalled 0.6 whereupon the direction of loading was reversed. The rate of loading was also dropped to 0.1 mm/min to allow higher definition of the unload-reload loop using the same scanning rate. The samples were unloaded to η = 0.2 and were then reloaded to failure. The rate of loading during the reloading was kept at 0.1 mm/min until the slower stress changes associated with yielding of the soil appeared. At this point the rate was raised to 0.3 mm/min where it was maintained until failure of the soil. The logic behind unloading between the 0.6 and 0.2 stress ratios is discussed in Section 4.5.2.2.

4.5.2 Results

The samples suffer stress relief and disturbance during their removal from the clay bed. The sampling procedure minimised the disturbance but only the hydrostatic component of the stress system could be re-established through reconsolidation. It was found by Kirkpatrick & Khan (1984a), who investigated the reaction of clays to stress relief, that when the deviatoric component of the stress state is not re-established, the soil's properties are significantly affected. The stress-strain behaviour is appreciably different and the failure stress is underestimated. The observed behaviour of the isotropically consolidated samples is, therefore, not entirely relevant to the anisotropically consolidated clay beds but in the absence of a better indication of the soils stress-strain properties, it was still believed to be worthwhile.

There was only a loose correlation between the volume of water expelled from the samples during reconsolidation and the volume change inferred from the change in water content. The volume losses calculated from the change in water content were more consistent between samples taken from the same clay bed than the burette measurements, and were therefore used to calculate the post-consolidation dimensions of each sample. In the calculation of these dimensions it was assumed for simplicity, that the radial and vertical strains were equal.

In order to determine the change in height of the sample it was necessary to account for the compliance stiffness of the triaxial apparatus. This was done by recording the displacements that accompanied the loading of a metal cylinder in place of a clay sample. For ram loads in excess of 0.01 kN (equivalent to 10 kPa on the sample) the correction to the displacement was linearly related to the ram load.

During the undrained testing the volume of the sample remained constant and it was possible, by assuming the sample deformed as a right-cylinder, to estimate the cross-sectional area at

each stage of the test. However, it was observed that the samples generally deformed into a barrel shape because of friction at their ends, and there was therefore a small error involved in using the estimated cross-sectional area, to calculate the deviator stress.

4.5.2.1 Undrained strength

Figure 4.27 shows the stress paths in the p' : q plane of three consolidated undrained triaxial tests. Each test is a representative example of those conducted on samples taken from the three normally consolidated clay beds OA, OC and OE. It can be seen that the unload-reload loops for each stress path are approximately positioned between the 0.6 and 0.2 stress ratios. The same trend in undrained strength is apparent in these tests as was found in the vane tests: the samples from clay bed OC have lower strengths than the samples from bed OE which in turn have lower strengths than the samples from bed OA. This progression in strengths is a result of progressively larger yield loci for each bed as indicated by their progressively lower water contents. All three beds have experienced approximately the same maximum mean effective stress of 200 kPa, but because their deviator stresses in combination with this are different, the shape and size of the yield loci are also different. In consequence when samples OA and OE are tested from an isotropic stress state the initial portions of their stress paths (before the yield loci are reached) are near vertical because of the overconsolidated elastic nature of the behaviour. The samples fail at a stress ratio, q/p', of 0.8 suggesting a friction angle of roughly 21° for the speswhite kaolin.

It is worthwhile noting that modified Cam-clay would predict higher strengths for samples from OE than from OA which is the reverse of what was observed. A possible explanation for this is given in Figure 4.28, which shows the yield loci associated with each stress state according to modified Cam-clay and the yield loci suggested by the stress paths of Figure 4.27. Except for stress state OC there is a discrepancy between the two sets of yield loci

typified by the marked difference between the pressures p_c', which characterise their size. This is not surprising as modified Cam-clay was based predominately on the observed behaviour of samples with isotropic consolidation histories.

The results and discussion above also apply to the triaxial tests carried out on samples taken from the normally consolidated clay beds OAB, OCD and OEF.

Figure 4.29 shows the stress paths of consolidated undrained triaxial tests conducted on samples taken from the overconsolidated clay beds OABA and OCDC. As with the initial portions of the stress paths of samples OA and OE in Figure 4.27, the two stress paths are near vertical due to the predominantly elastic behaviour beneath the yield locus. The difference between failure on the critical state line and failure on the Hvorslev surface is very small for these samples because their overconsolidation ratios place their states very near to the boundary between "wet" and "dry" behaviour. The samples therefore fail at approximately the same stress ratio as the normally consolidated tests.

In general unconsolidated undrained triaxial tests give consistently lower strengths than consolidated undrained triaxial tests conducted on similar samples. This is due to a loss of effective stress during sampling and storage in the unconsolidated tests, and a reduction in the water content in the consolidated tests. There is a considerable degree of scatter, but when the difference in water content is taken into account, as has been done in Figure 4.30, the strengths of the reconsolidated tests are only slightly greater than the unconsolidated tests. This suggests that the loss of effective stress during storage is only of marginal importance.

The reduction in water content of the reconsolidated samples is illustrated by Figure 4.31, which shows the water content of the reconsolidated samples against the water content of the clay bed in the position from which the sample originated. The reduction in water content

is only significant for the normally consolidated tests. This was because the normally consolidated samples were reconsolidated to stresses around the level of their yield stresses where the volumetric response is very sensitive to variations in mean stress.

4.5.2.2 Shear Modulus

The determination of the stiffness of a sample from external measurements of displacement is subject to a number of problems. It has been found that even when compliance effects are taken into account the strains inferred from external measurements are much larger than values measured locally on triaxial samples, Jardine et al. (1984). The reasons for this were given as the tilting of the sample and bedding errors at the end platens. It was also found that the disagreement between external measurements and local measurements was most severe for unconsolidated tests.

The determination of local strain requires the mounting of local gauges on the sides of the triaxial samples and the equipment needed for this was not readily available at Oxford. The shear modulus was therefore determined from the external measurement of displacement during an unload-reload loop. It was hoped that the investigation of this particular modulus would counteract some of the limitations of the measurement system, because it is in the initial phase of the test, that tilting and bedding errors are most pronounced. Some support for this is given by Wroth et al. (1984) who found that laboratory determined values of G/s_u from triaxial tests were three to eight times lower than those from the self-boring pressuremeter in contrast to the reloading loops where the correlation was much better.

The shear modulus, measured on an unload-reload loop, as well as being the appropriate modulus for correlation with the dilatometer readings, is also less sensitive to sample disturbance than the initial modulus. This is analogous to the unload-reload loop carried out in pressuremeter tests which is almost independent of disturbance and represents the elastic

response of the clay beneath the yield surface. The unload-reload modulus also has the advantage over the initial modulus that the compliance movements are linear with respect to changes in load over the entire range of the loop. The shear modulus was determined from a plot of the deviator stress against the axial strain of the sample; a typical example of which is shown in Figure 4.32. The slope of the loop, which is equal to $3G$, was taken as the slope of a line joining the bottom of the loop to the cross-over point.

The shear stiffness of a sample of soil can be expected to increase with closer packing of the soil particles, and to reduce with distortion. Figure 4.32 shows this second aspect clearly. The alignment of the soil particles, as the shear strain rises, reduces the slope of the shear stress-shear strain curve. When the direction of shearing is reversed the stiffness of the response shows a dramatic jump as the soil particles are required to move on a different path. As the soil is strained in this new direction the particles begin to align themselves again and the stiffness reduces. This phenomena is widely recognised, Whittle et al. (1992), Muir Wood (1990) and Simpson (1992), and it has been suggested that shear moduli are best presented with some indication of the strain level over which they were measured. For a particular density, the higher the stress level, the more resistance one would expect there to be to shear distortion.

The mechanics of the shear stiffness behaviour of soil therefore suggest a dependence on the mean stress level, the maximum past mean stress level (a measure of the density of soil in the case of clays) and the shear strain over which the measurement is taken. To avoid obscuring the effects of density and stress level on the measured modulus with random variations in the unload-reload loop strain, unloading and reloading were carried out between stress ratios q/p' of 0.6 and 0.2. This effectively links the drop in deviator stress and therefore the strain level, to the mean stress level at the unload-reload loop. It is therefore only necessary to consider the variation of the shear modulus with the mean stress level and the maximum previous stress level, as a systematic variation of the strain level has been

imposed on the tests.

Figure 4.33 shows a plot of the shear modulus against the mean stress level at which the unloading-reloading loop was executed. The results of Davidson (1980), who conducted ten triaxial tests on speswhite kaolin, have been included in the plot. His tests were made at a variety of stress states and stress histories. It can be seen that there is a slight correlation which appears to be independent of whether or not the samples are overconsolidated. On the basis of her own data and those of Davidson (1980), Al-Tabbaa (1987) proposed a linear relationship between the shear modulus and the mean stress which has also been shown in the plot. However the data from the tests reported here suggest a power relationship is more appropriate, e.g.:

$$G = 374\ (p')^{0.62} \qquad \text{(Eqn 4.9)}$$

Davidson (1980) found that there was a relationship between the maximum mean stress experienced by a sample and the shear modulus. All his samples were prepared isotropically, and therefore the maximum mean effective stress was coincident with p_c', which denotes the current size of the yield locus in the theory of modified Cam-clay, Figure 4.28. The value of p_c' is associated with the density of a sample. The difficulty of selecting an appropriate stress to characterise the size of the current yield locus for samples which have experienced anisotropic stress states has been discussed in the previous section. An alternative to comparison with p_c', is to use the undrained strength of a sample which closely reflects the size of the current yield locus and also has the advantage that it is dependent on the initial stress state as well. Figure 4.34 shows a plot of the shear modulus versus the undrained strength. The data have been differentiated according to their reconsolidated mean stress level. It can be seen that the samples that were reconsolidated to a mean effective stress of 200 kPa nearly all have rigidity indexes which fall between 250 and 100. The samples

reconsolidated to 450 kPa show less scatter (perhaps because of there are fewer points) and have indices between 167 and 100.

The shear modulus has been related to the degree of overconsolidation through normalisation with the undrained strength by Houlsby & Wroth (1985). Experimental and theoretical evidence suggests a relationship of the form:

$$\left(\frac{G}{s_u}\right) = \left(\frac{G}{s_u}\right)_{nc} R^{\,n} \qquad \text{(Eqn 4.10)}$$

However the results here do not exhibit a detectable dependence on overconsolidation. This does not contradict previous work because the scatter associated with the results may be large enough to conceal the effect of overconsolidation and in any case the tests represent only a small variation in the value of R. It is perhaps unfortunate that no triaxial tests were conducted on samples taken from bed OCDX because this had the highest value of R. Another aspect of these tests which might explain the absence of a dependence on the degree of overconsolidation is that the shear modulus was measured after the soil had been brought to a high proportion of its failure stress. Thus the preconsolidation effects on the soil structure would have been substantially erased by the time of the unload-reload loop.

4.6 SUMMARY

The contour plots of water content show that reasonably homogenous clay beds were produced. The variations of water content that were found, can be explained by radial drainage or an initial variation in water content. However, the increasing water content towards the centre of the sample suggests that slight radial drainage is the most likely cause. Confirmation of the uniformity of the clay beds was provided by the distributions of the vane

undrained strength.

Figure 4.35 shows a summary of the undrained strengths for each type of test against the appropriate water content. The results of the consolidated undrained triaxial tests and unconsolidated undrained tests show the greatest scatter. This is due to each point being the result of a single measurement of strength and water content. The strengths obtained from the vane tests conducted in the still stressed samples follow the same trend as the triaxial tests but with lower strengths. They also show less scatter because each point is the average of two strengths and two water contents, at each elevation within the clay bed. The vane tests conducted in the unstressed sample show the tightest grouping and the clearest trend of increasing strength with decreasing water content. Each point represents the average of nine vane tests and three water contents at a single elevation within the clay bed. In the case of the vane tests conducted in the unstressed beds, the improved definition of the relationship between water content and undrained strength, is achieved at a price of reduced appropriateness. The simplicity of the vane, which enabled a large number of tests to be carried out, also impaired the accuracy of the results. The chief reasons for this were the relatively large blade thickness of the vane tip, and that the clay beds were unstressed at the time of testing. Nevertheless, the vane strengths provide a convenient index to the undrained strength distribution within each clay bed, even if their absolute values might be in error. For this reason they have been used in the correlations with the dilatometer readings.

The oedometer tests revealed a dependence of the consolidation characteristics on the direction of loading.

There was too great a scatter in the recorded unload-reload behaviour of the clay samples to discern anything other than a dependence of G on the mean effective stress level and a slight dependence of G/s_u on the initial mean effective stress level.

CHAPTER 5

INTERPRETATION OF DILATOMETER TESTS IN CLAY

5.1 INTRODUCTION

In each clay bed dilatometer tests were conducted every 100 mm to a depth of 800 mm. In the upper 600 mm membrane expansions were carried out to obtain the *A* and *B* readings. In this study, in addition to the standard equipment described in Section 2.1, an air-regulator was used to allow the controlled depressurization of the blade from the *B* reading to give the *C* reading. Because of the relatively high permeability of speswhite kaolin care was taken to use a consistent rate of movement for the membrane during its inflation and deflation; the *A*, *B* and *C* readings were taken as near as possible to 20, 40 and 60 seconds respectively after reaching the test depth. The three membrane readings together with the force required to advance the blade P_D, comprise the complete set of measurements taken with the dilatometer in the top 600 mm of each clay bed.

The bulk of this chapter consists of a comparison of these dilatometer readings with each bed's geotechnical properties. The results of this study are used to establish a better understanding of the empirical correlations presented by Marchetti (1980).

Two dilatometer dissipation tests were carried out in the bottom portion of each clay bed at the 600 and 800 mm levels. The first consisted of a timed sequence of *A*, *B* and *C* readings and the second consisted of a timed sequence of *A* and *C* readings without an intermediate *B* reading. Section 5.6 shows how the results of these tests can be used to estimate a coefficient of consolidation for the clay.

5.2 DILATOMETER READINGS

In general the raw dilatometer readings are converted to the P_0, P_1 and P_2 readings using Equations 2.1, 2.2 and 2.3 respectively. The equations for P_0 and P_2 correct for the membrane movement required to activate the lift off signal (set at about 0.05 mm in the standard device) to give the pressure against the membrane at zero displacement. The P_1 reading is taken at a displacement of 1.1 mm. However, for the particular device used in the present study the displacement of the membrane at lift off was 0.095 mm and the full displacement was 1.00 mm. Assuming a linear pressure-expansion relationship these movements yield the following equations for P_0 and P_2:

$$P_0 = 1.105\ A' - 0.105\ B' \qquad \text{(Eqn 5.1)}$$

$$P_2 = 1.105\ C' - 0.105\ B' \qquad \text{(Eqn 5.2)}$$

where

A' = Corrected A reading

B' = Corrected B reading

C' = Corrected C reading

These two equations together with Equation 2.1 were used to process the three readings taken using the dilatometer membrane. It should be noted that there was a 0.1 mm difference between the displacement at which the B reading is taken in the standard device and the displacement at which it was taken in the device used in this study. However, the effect of small differences in the displacement at which the B reading is taken was shown by Marchetti & Crapps (1981) to be unimportant.

Profiles of the P_0, P_1 and P_2 readings from the test on clay bed OE are shown in Figure 5.1. In the analysis of the following sections the readings taken with the dilatometer membrane at 100, 700 and 800 mm depths have not been used. It can be seen in Figure 5.1 that the readings at the 100 mm depth are subject to interference from the chamber lid. Also the readings at the 700 and 800 mm levels are taken in clay which has been disturbed and then allowed to reconsolidate during the dissipation tests. Thus only five sets of reliable readings at depths of 200, 300, 400, 500 and 600 mm were obtained from each clay bed.

The rod which was used to push the dilatometer into the clay beds was equipped with two load cells, Figure 3.11. One load cell was used to measure the pushing force immediately behind the blade (the rod load cell) and the other was used to measure the total pushing force (the S load cell). Figure 5.2 shows the loads recorded during the penetration of clay bed OEF. At dilatometer tip penetrations up to approximately 400 mm the readings from the two load-cells are virtually the same; they gradually increase as more of the blade enters the clay bed. The spikes in the load-penetration curve coincide with breaks in the pushing for approximately 60 seconds so that membrane readings could be taken.

For penetrations greater than 400 mm the rod load cell was entirely within the clay bed and a length of rod above it was subject to friction from the clay. In consequence the load cell readings diverge as the S load cell begins to record the effects of soil friction on the rods in addition to the load on the dilatometer blade. When the centre of the dilatometer membrane was at the 600 mm depth level, which corresponds approximately to a dilatometer tip penetration of 700 mm, the penetration was stopped for 24 hours in order that a dissipation test could be carried out. When the penetration was resumed to take membrane expansion readings at the 700 and 800 mm levels, there was a jump in the penetration resistance. This was due to the disturbance and then reconsolidation to a higher strength of the clay surrounding the rods and dilatometer blade. The penetration load decreased rapidly as the blade entered the lower strength clay outside the previously disturbed zone and also as it

approached the base membrane.

Prior to the penetration of the dilatometer the vertical equilibrium of the clay bed is satisfied by the calibration chamber lid reaction and self weight of the clay acting against the upward push of the base membrane. The insertion of the dilatometer modifies this equilibrium. The membrane which encloses the lateral boundary of the clay bed does not allow the development of any significant shear stresses on the vertical boundaries and therefore the pushing force from the dilatometer causes the reaction against the lid of the chamber to decrease. The dilatometer is a total stress measuring device and could be expected to be sensitive to this effect. However the loads for dilatometer penetration into clay are relatively small and the change to the boundary conditions is not great. The highest penetration load was recorded in sample OAB and was equivalent to a 1.0 % reduction in the total vertical stress. However, for the calibration chamber tests in sand which are reported in Chapter 6, the penetration loads were much greater and the stresses applied at the sample boundaries were also smaller. In these tests the reduction in the boundary stress on the upper surface was very significant, and a discussion of this is given in Section 6.5.

A summary of the readings taken with the dilatometer, and the parameters which describe the initial state of each clay bed, is given in Table 5.1. The relevant data at depths of 200, 300, 400, 500 and 600 mm have been included. The stresses have been corrected for their level in the calibration chamber. The initial pore water pressure was taken to be hydrostatic except when water was still being expelled from the sample at a significant rate at the time of testing. In these cases a parabolic excess pore pressure distribution which was consistent with the rate of water outflow, was estimated using the method outlined in Section 3.4.5. The maximum excess pore water pressure calculated using this method was 5 kPa for the centre of sample OCD. Each water content is the average of three readings at radii of 100, 200 and 300 mm; and each undrained strength is the average of nine vane readings, three each at radii of 100, 200 and 300 mm.

5.3 INDEPENDENCE OF DILATOMETER READINGS

To obtain information about a soil deposit using an in-situ device it is necessary in principle to measure a number of responses at least equal to the number of unknown soil parameters which dominate the soil behaviour. This is because the chief soil parameters which describe soil behaviour (for example M, λ, and others in the critical state model) exert some influence in all but the simplest measurements. Jamiolkowski & Robertson (1988) took this view when they asserted that the penetration resistance was influenced to different degrees by almost all of a soil deposit's initial state factors, and it was therefore difficult to measure individual factors using a penetration test.

However, in practice the number of in-situ measurements required for the determination of a single soil parameter is often not as many as suggested above. Some measurements are effectively dominated by only one or two soil parameters though they are influenced by a number. All attempts to minimise the insertion disturbance of in-situ devices are in fact attempts to minimise the number of parameters required to describe the measurement; and it is the understanding of the complex nature of the disturbance effects which presents the greatest difficulty in the interpretation of in-situ tests. Where disturbance is significant, it is therefore measurements which are influenced by soil parameters that are least sensitive to the disturbance which are the most successful.

When more than one measurement is taken a second consideration is the independence of the readings. If two readings are similar in that they are dependent on the same factors, then there is a chance that one of them will prove to be redundant because it will contain no new information about the soil. In this study four measurements were taken with the dilatometer: P_0, P_1, P_2 and P_D. Of these it is the mechanisms behind the P_0 and P_1 readings which share the most similarities. Figure 5.3 shows a plot of the P_0 and P_1 readings; both have

been corrected for the hydrostatic water pressure. Soil behaviour is determined by its effective stress level and so in many of the plots which follow the initial pore water pressures have been subtracted from the dilatometer readings. It can be seen in Figure 5.3 that if one reading is known the other is given automatically. This applies to the entire range of stress histories, stress states and soil strengths which were examined. Nevertheless, Figure 5.3 does contain some information, as the position and slope of the linear relationship between the two readings are dependent on the type of material. A different relationship was found for instance for the tests in sand reported in Chapter 6. It is straightforward to show that Marchetti's material index I_D is directly related to the slope of this plot. Because of the close correlation between the P_0 and P_1 readings, it is only worthwhile to consider one of the readings when comparisons are being made with parameters which describe the condition of the soil deposit. For this reason the P_1 reading has been excluded from further consideration.

The P_2 reading is a measure of the response of the soil during unloading, but it has much in common with the other membrane readings. In Figure 5.4 the P_0 and P_2 readings are compared. The plot reveals a loose correlation between the two readings which suggests they are dependent on the same parameters but to different degrees. The trend of the results from the overconsolidated beds is different to that from the normally consolidated beds. So, in contrast to the P_1 reading it appears that the P_2 reading measures a soil response which is different from that recorded by the P_0 reading.

Figure 5.5 shows the P_0 and P_2 readings plotted against the P_D value, and at first sight they appear to be independent. However, both readings show similar patterns and in fact the quantity $P_0 - P_2$ is connected to P_D. Figure 5.6 shows that P_D is linearly related to

$P_0 - P_2$. It is possible, therefore, to describe P_D in terms of P_0 and P_2 without the need to consider other factors such as the degree of overconsolidation, undrained strength or stress state. This would seem to limit the usefulness of P_D as a source of additional information about the properties of the soil. However, the author has one reservation about entirely dismissing the P_D reading which is discussed in the next section.

In the following sections the analysis has concentrated on comparison with the P_0 and P_2 readings. It could be argued that P_D would be a better choice than P_0 because of the greater independence it shows from P_2 in Figure 5.5. The difference is not great however, and the P_0 reading has been preferred to P_D because it has been the subject of nearly all the previous work on the dilatometer.

5.4 COMPARISON WITH PARAMETERS DESCRIBING THE SOIL'S INITIAL STATE

The description of the initial state of a soil deposit includes the following:

- Macro and micro-fabric
- Vertical and horizontal stress
- Pore pressure
- Initial void ratio
- Overconsolidation ratio

The last four of these can be quantified and are known accurately for each clay bed. It is logical to compare the dilatometer readings with these separate quantities before considering

parameters such as the undrained strength and soil stiffness, which are soil responses which reflect the effects of many of the initial state quantities. Figure 5.7 shows the P_0 and P_2 readings plotted against the vertical effective stress. The wide scatter of the points demonstrates that neither reading is controlled primarily by the level of the vertical effective stress. Figure 5.8 shows the same dilatometer readings plotted against the horizontal effective stress. It is apparent that the P_2 reading gives a good indication of the total horizontal stress. A weak correlation between the P_0 reading and the horizontal stress suggests only a partial dependence between the two. Figure 5.9 also shows the same dilatometer readings but plotted against the mean effective stress. Not surprisingly the P_2 reading is also a good indication of the mean stress of which the horizontal stress is a major component. There is an obvious but less strong correlation between P_0 and the mean stress.

It is evident from the foregoing observations that the importance of the horizontal stress level in the P_2 reading outweighs the influence of all other parameters; the *OCR* and strength appear to have little or no influence. However, the P_0 reading cannot be explained by the horizontal stress level alone. Other parameters need to be considered to explain the reading, and what these should be and how their influence should be addressed are not immediately obvious.

Wroth (1988) sets out the basic requirements for correlations between the results of in-situ tests and soil parameters. They are as follows:

- A physical appreciation of why the correlation is expected to work,
- A theoretical background, however simplified or idealized,
- Correlations must be expressed in terms of dimensionless variables.

Marchetti (1980) was broadly following these recommendations when he proposed a single correlation between the horizontal stress and the P_0 reading, Equation 2.9 and Figure 2.6. In this correlation the increase in the horizontal stress caused by the insertion of the dilatometer was assumed to be related to the initial horizontal stress. The results shown in Figure 5.8 do not support this approach. Other parameters are required to determine the modification to the horizontal stress.

Following through Wroth's recommendations a simple idealisation is to assume that the disturbance caused by the insertion of the dilatometer is similar to that caused by the insertion of a cone-penetrometer or cone-pressuremeter. Support for this approach is provided by Luteneggar & Blanchard (1990) who conducted cone-pressuremeter and dilatometer tests at a number of clay sites and found good agreement between the cone-pressuremeter lift-off pressure and the dilatometer P_0 reading. It can be inferred from this that the installation disturbance is similar in both instruments.

Cavity expansion theory predicts that in Tresca materials the cone-pressuremeter limit pressure P_L, is given by the following expression, Gibson and Anderson (1961):

$$\frac{P_L - \sigma_H}{s_u} = 1 + \ln(G/s_u) \qquad \text{(Eqn 5.3)}$$

where

P_L = Cone-pressuremeter limit pressure

σ_H = Total horizontal stress

G = Soil shear modulus

The expression indicates that the limit pressure is controlled by the rigidity index G/s_u and

the undrained strength, s_u. Whilst theoretically the lift-off pressure would be expected to be the same as the limit pressure, in practice a much lower lift-off pressure is observed. It may, however, be expected to be controlled by the same parameters as the limit pressure, i.e. the undrained strength and the rigidity index. With the results of Lutenegger & Blanchard (1990) in mind Yu et al. (1992) carried out a numerical analysis of the dilatometer (Section 2.6) and obtained results which approximately fitted the following expression:

$$\frac{P_0 - \sigma_H}{s_u} = 1.57 \ln(G/s_u) - 1.75 \qquad \text{(Eqn 5.4)}$$

The results of Yu et al. can be improved if the limited size of the finite element mesh used by them is taken into account. When an inclusion is introduced into a finite element mesh with rigid boundaries there is an increase in the stress level at the boundaries. This leads to an increase in the mean stress level within the mesh. The magnitude of the increase is principally dependent on the bulk modulus and the volume introduced as a fraction of the volume of the mesh. A close approximation to the increase in the mean stress level can be found using the following:

$$\Delta p = \frac{\Delta V}{V} K_s \qquad \text{(Eqn 5.5)}$$

where

Δp = Change in mean total stress

ΔV = Change in the volume of the mesh

V = Volume of mesh

K_s = Undrained bulk modulus

This leads to

$$\Delta p = \frac{\Delta V}{V} G \frac{2\,(1+\nu)}{3\,(1-2\nu)} \qquad \text{(Eqn 5.6)}$$

where

G = Shear modulus

ν = Poisson's ratio.

In the finite element calculations of Yu et al., a Poisson's ratio of 0.49 was used. If this is combined with the volume of the mesh and the volume of the rigid body used to simulate the insertion of the dilatometer blade, then the following is obtained:

$$\Delta p = \frac{G}{342} \qquad \text{(Eqn 5.7)}$$

This equation was used to correct the results of Yu et al. for the increase in the ambient stress level due to the limited extent of the mesh. When this was done the modified data were found to fit the following expression:

$$N_D = 1.1 \ln(G/s_u) - 0.14 \qquad \text{(Eqn 5.8)}$$

where

$$N_D = \frac{P_0 - \sigma_H}{s_u} \qquad \text{(Eqn 5.9)}$$

A comparison with Equation 5.9 has been made in Figure 5.10 where the undrained strength has been plotted against $P_0 - \sigma_H$. Two separate linear correlations were found with the data from the clay beds with initial mean stresses of 200 kPa and 80 kPa grouped around one line,

and the data from the clay beds with mean stresses of 450 kPa grouped around the other. According to Equation 5.9 the slopes of these lines are equal to N_D and are related to the rigidity index of the soil. In Section 4.5.2.2 it was found that triaxial tests conducted on samples taken from clay beds with mean stresses of 200 kPa yielded a higher range of unloading rigidity indices than samples taken from clay beds with mean stresses of 450 kPa.

In Figure 5.11 the slopes of the two lines in Figure 5.10 are plotted against the range in the rigidity index found in the triaxial tests. Equation 5.8 is also shown in the plot. The trend of the experimental results is the same as that of the numerical predictions in that there is some indication that higher N_D may be associated with higher G/s_u. The discrepancy between the two can in the most part be attributed to difficulties in measuring the appropriate reference soil properties; but there are also simplifications in the analysis of Yu et al. which result in a failure to model some important aspects of the soil behaviour. A simple loading of the soil is used in their analysis which contrasts with the real loading which evidence, which is discussed below, suggests is cyclic in nature.

The dilatometer membrane is positioned in the centre of the blade a short distance behind the sharpened tip. Cavity expansion theories predict that there is some stress relief behind the tip of any penetrating probe. This is because the stresses required to open the cavity at the tip are larger than those required to maintain it. Further evidence that this type of loading does occur is provided by Finno (1993), who studied the dilatometer numerically using the strain path method (Section 2.6). He investigated the extent of the failure zone around the dilatometer in a simulated lightly overconsolidated medium plastic clay. Figure 5.12 shows shear stress ratio contours in the $y = 0$ plane. The shear stress ratio is defined as the octahedral shear stress at a point divided by the octahedral shear stress at failure. With this definition a value of 1 indicates soil failure. The results show that the soil adjacent to the dilatometer membrane has passed through a failure zone, been unloaded and then brought to

failure again before the P_0 reading is taken. Finno indicates that the reduction of shear stress at the blade transition is related to strain reversal. The control of the pressures against the blade by a reloading process, and therefore an unloading modulus, would explain the apparent independence of the results in Figure 5.10 from the *OCR*.

Another explanation for the independence from the *OCR* of the pressures against the blade, is provided by analogy with the pore pressures caused by driven piles. The mechanism of disturbance caused to the surrounding clay during the penetration of the dilatometer and the driving of a pile has many similarities. It has been found by Campanella et al. (1985) and Robertson et al. (1988) that the insertion of the dilatometer severely disturbs the surrounding soil, with the result that the membrane readings are dominated by pore water pressures. Wroth et al. (1979a) modelled the driving of a pile into clay as the creation of cavity within the soil under undrained conditions. They showed that the values of excess pore pressure in the soil close to the pile were dependent on the initial undrained strength and were insensitive to the *OCR*. Indeed they showed that the magnitude of all the stresses acting on the pile were essentially independent of *OCR*. This is a surprising result as the pore pressure is strongly dependent on the rigidity index G/s_u, which experimental evidence (Wroth et al. (1979b)) suggests is a function of *OCR*. However, pore pressure changes take place due to the change in mean stress, as well as through the shearing associated with cavity expansion. Soils with an *OCR* greater than about 2.0 show dilative behaviour and the pore pressure changes due to shearing are therefore negative. According to the calculations of Wroth et al. (1979a), these two effects cancel each other out with the result that the excess pore pressure distribution of a soil with a given strength is almost unaffected by the past history of the soil. Wroth et al. also found that the radial stress acting on a pile was approximately six times the in-situ undrained strength. This value is in reasonable agreement with the range in N_D shown in Figure 5.10; further evidence in support of the fact that the two processes are similar.

The independence of P_0 from the *OCR* can explained by using a reload modulus which is independent from *OCR* in the analysis of Yu et al. (1992), or through analogy with driven piles with the dilatometer readings dominated by pore pressures which are independent of *OCR*. The results of the triaxial tests reported in Section 4.5.2.2 would seem to favour the former explanation.

The results presented earlier in Figure 5.8 suggested a clear dependence of the P_2 reading on the horizontal stress. This was an unexpected result, and is difficult to explain given the unloading processes involved in taking the *C* reading. The available evidence from field tests by Robertson et al. (1988) and Benoit & Lutenegger (1992) does not support this finding. Also Powell & Uglow (1988) carried out a series of dilatometer tests in which the membrane expansion was varied between zero and 1.4 mm. They found that the P_2 reading was highly dependent on the displacement of the membrane, reducing as the membrane expansion increased. The reduction was small in soft clays but significant in stiff clays. All that can be concluded therefore is that the P_2 reading gave an indication of the horizontal stress in this particular study. This does have certain implications, because P_D was eliminated from further consideration in Section 5.3 on the basis of its linear dependence on $P_0 - P_2$. The plot of $P_0 - \sigma_H$ versus P_D would also have produced a linear correlation.

Nevertheless, through analogy with Figure 5.10 it was thought useful to plot the quantity $P_0 - P_2$ against the undrained strength. This has been done in Figure 5.13 in which a clear relationship between them is revealed for the tests in this study:

$$P_0 - P_2 = 5\, s_u \qquad \text{(Eqn 5.10)}$$

Benoit & Lutenegger (1992) relate the quantity $P_0 - P_2$ obtained from a dilatometer sounding in a soft sensitive clay to the in-situ horizontal stress. Reference values for the horizontal stress were found using a self-boring pressuremeter. However, the data they present are also reasonably consistent with Equation 5.10.

5.5 MARCHETTI'S CORRELATIONS

It has been shown in the previous section that P_2 is related to the total horizontal stress prior to the insertion of the dilatometer, and that the quantity P_0-P_2 gives a reasonable indication of the undrained strength. These findings are applicable to normally consolidated and moderately overconsolidated clay over a wide range of stress ratios and stress levels. However, the work of Marchetti (1980) and most of the research that has been undertaken up to this time has concentrated on the P_0 and P_1 readings. In order that a comparison can be made between the results of this study and previous research it is necessary to consider the separate soil parameters which control the P_0 reading. This is because, in general, Marchetti's correlations are only applicable to deposits which have experienced simple one-dimensional stress histories. The clay beds tested in this study were subjected to complex stress paths during their preparation with no control over the lateral strain, and because of this the results cannot be used directly to assess the validity of Marchetti's correlations. An example of a direct comparison is shown in Figure 5.14. This shows a K_D : K_0 plot and, as one would expect, there is little agreement between the results of this study and Marchetti's correlation.

However, when the individual parameters which control P_0 are considered the differences between the clay beds used in this study and the deposits used by Marchetti to establish his

correlations can be taken into account. The following sections examine Marchetti's correlations for the determination of K_0, OCR and s_u, and show how they are consistent with Equation 5.9 when certain conditions are satisfied. Marchetti's correlation with the soil modulus is briefly discussed in Section 5.5.4.

5.5.1 Stress Ratio

It has been shown that the data of Figure 5.10 can be fitted by the following:

$$s_u = \frac{(P_0 - \sigma_H)}{N_D} \qquad \text{(Eqn 5.11)}$$

A single line with N_D equal to 5.18 provides an average fit to all the data and values for N_D of 4.05 and 6.97 give upper and lower bounds.

In clays which have been one-dimensionally normally consolidated a good estimate for K_0 is provided by the Jaky equation, Muir Wood (1990):

$$K_{0nc} = 1 - \sin\phi \qquad \text{(Eqn 5.12)}$$

where

ϕ = Friction angle

K_{0nc} = Stress ratio during one-dimensional consolidation

Simple overconsolidation of a one-dimensionally normally consolidated deposit such as would result from erosion of overburden will increase K_0. The variation of the stress ratio, in

relation to the degree of overconsolidation, can be expressed by empirical equations with the following form, Schmidt (1967):

$$K_0 = K_{0nc}\ OCR^{\,a} \qquad \text{(Eqn 5.13)}$$

Schmidt recommended using a value for a equal to $1.2\ \sin\phi$, while Meyerhof (1976) and Mayne & Kulhawy (1982) suggested taking a equal to $\sin\phi$. In general, in natural soil deposits the exponent a has been found to vary from about 0.4 to 0.5, Schmertmann (1985). For the present argument Schmidt's value of a has been preferred as it takes some account of the nature of the soil, and also was found to be accurate in modelling the unloading behaviour of kaolin by Al-Tabbaa (1987).

The undrained strength also varies with the degree of overconsolidation. The form of the variation has been successfully modelled by Ladd et al. (1977) using the following empirical expression:

$$(s_u/\sigma'_V) = (s_u/\sigma'_V)_{nc}\ OCR^{\,m} \qquad \text{(Eqn 5.14)}$$

Wroth (1984) shows how the exponent m, in Equation 5.14 is equivalent to the plastic volumetric strain ratio Λ, of critical state soil mechanics. If the expressions given by Equations 5.13 and 5.14 are combined the following expression is obtained:

$$\left(\frac{K_0}{K_{0nc}}\right)^{m/a} = \frac{(s_u/\sigma'_V)}{(s_u/\sigma'_V)_{nc}} \qquad \text{(Eqn 5.15)}$$

This indicates that for clay deposits which have experienced one-dimensional stress histories there is an empirical relationship between the stress ratio and the normalised undrained strength (s_u/σ'_V). Equation 5.15 can be combined with Equation 5.11 to eliminate the undrained strength:

$$K_D - K_0 = (s_u/\sigma'_v)_{nc} \left(\frac{K_0}{K_{0nc}}\right)^{m/a} N_D \qquad \text{(Eqn 5.16)}$$

where $K_D = (P_0 - u_0)/\sigma'_V$

Leaving out the dependent and independent variables K_0 and K_D, the remaining components of Equation 5.16 can all be expressed in terms of three variables: the friction angle ϕ, N_D and the plastic volumetric strain ratio Λ. The details of the necessary substitutions are given in Table 5.2. Typical values and the range of the values found in natural clays are also included in the table. By taking the typical values for the various components of Equation 5.16 it possible to make a comparison with Marchetti's $K_D : K_0$ correlation. This has been done in Figure 5.15. Also shown in the plot are the bounds to the expression which are found when extreme combinations of the variation in each component are used.

The agreement with Marchetti's curve when average properties are used in Equation 5.16 is good. It should be remembered that Marchetti's curve, Equation 2.8, is an empirical best fit to his data. This is in contrast to Equation 5.16 which is derived from the more general Equation 5.11, when the link between the undrained strength and stress ratio that exists in deposits with one-dimensional stress histories is taken into account. The close agreement has much to do with the fact that Marchetti derived his reference K_0 values using the *OCR* and the empirical equations of Brooker & Ireland (1965) and Ladd et al. (1977). This point is

discussed further in Section 5.7.

The bounds to the relationship between K_D and K_0 shown in Figure 5.15 are a reasonable reflection of the scatter that has been found by other researchers who have measured the reference horizontal stress, Powell & Uglow (1986) and Larsson & Eskilson (1989). It appears therefore that Equation 5.16, together with moderate variations in its component parameters, provides an explanation for a broad correlation between K_0 and K_D. Equations 5.13 and 5.14 only apply when the soil deposits have experienced simple unloading, so the correlation is only likely to exist when these conditions have been satisfied. It can now be appreciated why Marchetti (1980) found it necessary to stipulate that this correlation was only applicable in deposits which had experienced one-dimensional stress histories and which were unaged and uncemented.

Marchetti (1980) observed that in normally consolidated deposits K_D was nearly constant with depth and had a value between 1.8 and 2.3. Marchetti attributed this to a direct relationship between K_D and K_0. The insertion of parameters appropriate for normally consolidated conditions into Equation 5.16 provides an explanation for this. With the most extreme combination of the parameters in Table 5.2 a range in K_D of 1.49 to 2.45 is obtained for normally consolidated deposits using Equation 5.16.

5.5.2 Overconsolidation Ratio

Marchetti (1980) used the shape of the K_D profile to infer the stress history of the deposit. He also proposed a correlation between K_D and the overconsolidation ratio, Equation 2.17. With an understanding of the parameters which control the P_0 reading it is possible to show why this relationship exists. Dividing Equation 5.11 by the vertical effective stress and

substituting in Equation 5.13 and 5.14 yields the following.

$$K_D = K_{0nc}\ OCR^{\ a} + (s_u/\sigma'_V)_{nc}\ OCR^{\ m}\ N_D \qquad \text{(Eqn 5.17)}$$

Again the relationship between the *OCR* and K_D is not unique but depends on parameters which can be linked to the friction angle, the plastic volumetric strain ratio and N_D. By taking the average values for each of these properties given in Table 5.2 it is possible to produce a single correlation between the *OCR* and K_D. This is shown in Figure 5.16 together with Marchetti's original correlation. A variation in the friction angle and plastic volumetric strain ratio which are reasonable for natural clays (Table 5.2) together with the variation in N_D which was found in this study, have been used to produce bounds to the relationship between *OCR* and K_D. These are also shown in Figure 5.16.

Equation 5.13, which is used in the derivation of Equation 5.17, is theoretically only valid up to *OCR*'s of about 15, because at this degree of overconsolidation passive failure occurs. Equation 5.17 has therefore been stopped in Figure 5.16 at an *OCR* of 15. Again, the correlation between the *OCR* and K_D, because it involves the application of Equations 5.13 and 5.14, only applies to simply unloaded uncemented clays. Marchetti also placed this restriction on the application of his own correlation.

5.5.3 Undrained Strength

Marchetti (1980) proposed a single correlation between the undrained strength and K_D (Section 2.5.5.1, Equation 2.20). Equation 5.11 can also be used as the basis for an expression between s_u and K_D. Dividing through Equation 5.11 by the vertical effective

stress gives:

$$K_D = (s_u/\sigma'_V)\ N_D + K_0 \qquad \text{(Eqn 5.18)}$$

The substitution of Equation 5.15 into Equation 5.18 leads to the following:

$$K_D = (s_u/\sigma'_V)\ N_D + K_{0nc} \left(\frac{(s_u/\sigma'_V)}{(s_u/\sigma_V)'_{nc}} \right)^{a/m} \qquad \text{(Eqn 5.19)}$$

A comparison between this expression after the substitution of the average soil properties from Table 5.2, and Marchetti's correlation is shown in Figure 5.17. The bounds to the expression that were found when the ranges of the parameters in Table 5.2 were used, are also shown. It can be seen that Equation 5.19 is in close agreement with Marchetti's correlation. This is not surprising as the equation of Ladd et al. (1977), Equation 5.14, was used by Marchetti to derive his $s_u : K_D$ correlation from his $OCR : K_D$ correlation. This same equation has been used in the derivation of Equation 5.19 from 5.11.

Marchetti indicates that his correlation appears to apply to clays overconsolidated by processes other than the removal of overburden (e.g. aging and cementation). This seems curious given that Equation 5.14 for the *OCR*, upon which his correlation and Equation 5.19 for s_u are based, is only applicable to simply unloaded uncemented soils. In fact this result is reasonable because Marchetti's equations work in the opposite direction from that which he supposed, i.e. his correlation with the undrained strength is in fact the more fundamental, and the restrictions which are attached to the application of the equation of Ladd et al. (1977) are therefore unnecessary.

Equation 5.14 and Equation 5.13, were used to eliminate the stress ratio from Equation 5.18 to form an expression directly linking s_u and K_D, Equation 5.19. However, the value of the stress ratio is relatively unimportant to the magnitude of K_D, and so this step can be omitted without significantly impairing the agreement with Marchetti's expression. Figure 5.18 compares Equation 5.18 with Marchetti's expression with a K_0 value of 0.6, 1.0 and 2.0. Equation 5.18 does not include empirical expressions and therefore, like Marchetti's expression, is applicable to soil deposits irrespective of their stress history.

5.5.4 Soil modulus

The quantity $P_1 - P_0$ has been used by Marchetti in his "dilatometer modulus", Equation 2.5, to correlate with the one-dimensional drained modulus of soil deposits. This study has shown that for a given material P_1 is in fact dependent on P_0. This indicates that, as with P_0, the quantity $P_1 - P_0$ is dependent on the horizontal stress, undrained strength and N_D. Figure 5.10 shows how N_D is independent of the *OCR*, a parameter which is highly correlated with a soil's stiffness. It would seem therefore that the "dilatometer modulus" has little potential to measure a soil modulus directly. Evidence suggests that the soil is completely remoulded during the insertion process, destroying its strain-sensitive parameters - such as a high stiffness associated with past overconsolidation. The moderate success the dilatometer has shown in determining the stiffness of natural cohesive deposits may therefore have much to do with its sensitivity to the undrained strength and horizontal stress, because at a given depth an overconsolidated deposit will have a higher strength and a higher horizontal stress than a normally consolidated deposit. Its potential for the evaluation of the soil modulus is therefore through secondary correlations and not through direct measurement. This is an important point because secondary correlations do not always give a good indication of the soil modulus; and therefore the use of an empirical correlation, in which a hidden assumption that

they do, plays an integral part, should be avoided.

5.6 CONSOLIDATION

It has been recognised for some time that the pore pressures generated during the insertion of the dilatometer in normally consolidated and lightly overconsolidated clays strongly influence the membrane readings, Campanella & Robertson (1983). It has been observed that a series of the membrane readings will decay with time, producing a curve similar in form to the pore pressure dissipation curves that can be obtained with the piezocone. Based on this observation a number of researchers have proposed methods for the estimation of the consolidation properties of soil deposits using the dilatometer: Lutenegger & Kabir (1988), Robertson et al. (1988), Schmertmann (1988) and Marchetti & Totani (1989). The dilatometer measures the reduction of a total stress with time, and not the dissipation of pore pressure; and Baligh et al. (1985), have shown that the dissipation rates of excess pore pressure, and horizontal total stress acting on the side of a cylindrical probe, are not the same. Nevertheless, it has been found by Lutenegger & Kabir (1988) that the shape of the pore pressure dissipation curves from the piezoblade closely matches that of curves of repeated P_2 readings. Also, it is relatively simple to take measurements with the dilatometer in comparison with the piezocone, where problems exist related to the saturation and clogging of porous elements, and the stability of electronic systems over long periods; and for these reasons the theoretical objections have been put to one side.

Three recognised readings can be taken with the dilatometer and these yield the P_0, P_1 and P_2 pressures. Each of these pressures has been shown to decay with time by Powell & Uglow (1986). At present there are two distinct methods for using the dilatometer readings to estimate a soil's consolidation characteristics. Marchetti & Totani (1989) suggested a

technique in which a timed sequence of A readings are taken with no intermediate B reading. An s-shaped curve was obtained by these authors when the A readings were plotted against log t. They recommended using the time to the point of contraflexure in the curve as an index of the rate of consolidation, and proposed the following tentative empirical correlation:

$$c_h \cdot t_{flex} = 5 \text{ to } 10 \text{ cm}^2 \qquad \text{(Eqn 5.20)}$$

where

t_{flex} = Time to point of contraflexure

c_h = Horizontal coefficient of consolidation

In contrast to the above method, procedures which involve expansion-deflation cycles of the membrane were proposed by Robertson et al. (1988), Lutenegger et al. (1988) and Schmertmann (1988b). In these methods the shape of the P_2 : t curve is analysed through comparison with the theoretical shapes of pore pressure dissipation curves for the piezocone. An advantage of using the P_2 reading for this is that, in general, the values tend to the equilibrium pore pressure. This allows the test to stopped when the time taken for a 50 % reduction in P_2 has been recorded.

The principal theories of dissipation around a cone that have been used to evaluate the reduction of dilatometer readings over time, are those of Torstensson (1977) and Baligh & Levadoux (1986). The adaptation of the theories to the investigation of the dilatometer has been done by assuming that the dissipation around the dilatometer is equivalent to the dissipation around a cone with the same cross-sectional area. This allows an equivalent radius for the dilatometer to be used in the time factor, Equation 5.21, without any need to modify the analysis.

$$T = \frac{c_h \, t}{a^2} \qquad \text{(Eqn 5.21)}$$

where

a = Probe radius

A value for a equal to 20.5 mm is found when the dilatometer is approximated to a cylinder with the same cross-sectional area. There are important problems, mainly regarding the different geometries of the two instruments, which are neglected when these analyses for the cone penetrometer are applied to the dilatometer. Their resolution requires nothing less than a solution for the generation and subsequent dissipation of pore pressures around the dilatometer blade. Nevertheless, it is possible to make a small improvement on the two previously mentioned analyses by taking the soil's rigidity index into account. This has been done by Teh & Houlsby (1989) who used the strain path method to study the penetration of a cone penetrometer into clay. The dissipation of excess pore pressure was then analysed using uncoupled Terzaghi-Rendulic consolidation theory. The rigidity index determines the extent and gradient of the initial pore pressure distribution and therefore the process of consolidation as characterised by the shape of the time-dissipation curve. Teh & Houlsby found that the most satisfactory way to incorporate these effects was by using the following time factor:

$$T^* = \frac{c_h \, t}{a^2 \sqrt{I_r}} \qquad \text{(Eqn 5.22)}$$

For 50 % consolidation at five radii behind the cone shoulder their analysis gave T = 1.11. This location can only be a crude idealisation of the position of the centre of the membrane relative to the shoulder of the dilatometer blade, as what would constitute a suitable

approximation is unclear. In the analysis of Teh & Houlsby the dissipation around the shaft of a cone was predicted to be considerably slower than around the cone tip. This was because lower pore pressures and flatter gradients were predicted around the shaft; this is related to a slight unloading of the soil which occurs behind the cone shoulder. Practical evidence in support of comparing the slower dissipation around the shaft of a cone to that around the dilatometer is provided by Robertson et al. (1988) who found that the dissipation of pore pressures around the tip of a cone was approximately twice as fast as around the dilatometer membrane. They also found that the piezoblade measured very similar pore pressures to those recorded on the shaft of the piezocone. The theoretical solution for the dissipation on the shaft of the cone would therefore seem the most appropriate one to consider.

Both P_0; P_2 and P_0; P_1; P_2 dilatometer dissipation tests were carried out in each clay bed. Figure 5.19 shows the P_0; P_2 : ln t plot obtained for clay bed OA. Minor corrections to the dilatometer readings have been made, equal to the hydrostatic water pressure. A series of P_0 readings obtained in this way forms the basis of the method proposed by Marchetti & Totani (1989) for the estimation of the coefficient of consolidation. The advantage of this method, according to Marchetti & Totani, is that there is no requirement to know the P_0 reading corresponding to zero time after the break in penetration, as it is the shape of the curve and not the change in reading which is used. However, Marchetti & Totani reported that on occasions it was difficult to discern the point of contraflexure from the results. This was also found to be the case with the results obtained in this study, and the use of the P_0; P_2 : ln t plots for determining the soil's consolidation characteristics was not attempted. In general the P_0 readings showed a steady decrease with increasing time, with a slight recovery when the rate at which readings were taken was decreased. This suggests that even though only very small movements of the membrane were needed to take the readings, they were

sufficient to affect the behaviour of the soil. The P_2 readings followed the same trend as the P_0 readings, but were found to be sensitive to slight over-expansions of the membrane and generally showed greater scatter.

Figure 5.20 shows the P_0; P_1; P_2 : ln t plot obtained for clay bed OA. All three readings show a reduction with time though P_0 and P_1 show a partial recovery as the rate of taking the readings decreases. The increase in effective stress around the dilatometer is reflected by the increasing difference between P_0 and P_1 as consolidation proceeds. When it is corrected for the hydrostatic water pressure, the P_2 reading decreases with time and eventually reaches zero. If comparison is made with Figure 5.19, it is apparent that an intermediate expansion is required in order that the decrease in pore pressure can be traced with the P_2 reading. In the analyses of Robertson et al. (1988), Lutenegger & Kabir (1988) and Schmertmann (1988b), the time for a 50 % reduction in the pore pressure t_{50}, is taken to be equal to the time required for $P_2 - u_0$ to reduce to half its initial value. An error is introduced by this because the first P_2 reading is not taken until one minute after the penetration has stopped and so the first minute of the P_2 : ln t curve is missing. However, given the general empiricism of the method this is not a major problem.

By normalising each P_2 curve with respect to t_{50} and also by plotting the degree of reduction in $P_2 - u_0$ it was possible to compare all of the curves. This has been done in Figure 5.21. It can be seen that in general the shapes of the P_2 curves are similar. The curves do not precisely pass through the point $P_2 - u_0$ = 50 % and t/t_{50} = 1 because the t_{50} values were obtained graphically and are therefore not exact.

Each of the values for the coefficient of consolidation calculated from the dilatometer dissipation tests using Equation 5.22, have been compared in Figure 5.22 with two values obtained from oedometer tests on samples taken from each clay bed, Section 4.3. A rigidity index of 175, which is the average of the range in rigidity indices found from the triaxial tests, together with an a of 20.5 mm and a time factor of 1.1 were used in Equation 5.22. The oedometer c_v values corresponding to the increments in applied vertical stress covering the effective vertical stress level in each clay bed have been used in the comparison. It can be seen that the same trend has been identified by the dilatometer and oedometer tests, for the variation in the coefficient of consolidation between the clay beds. However, there is almost an order of magnitude difference between the two values.

Oedometer tests are generally recognised to underestimate the coefficient of consolidation because of the disturbance caused during sampling. Also the samples were tested for their vertical coefficient of consolidation c_v. It has been found by Teh & Houlsby (1989), that it is the horizontal coefficient of consolidation c_h, which theoretically controls the dissipation process around a probe, and a difference between the two coefficients can be expected to exist in anisotropically stressed samples because of the preferential orientation of clay particles. However, in kaolin the difference is not great and for this reason only the vertical compression behaviour was measured. The discrepancy between the two sets of measurements of the coefficient of consolidation may be the result of disturbance of the oedometer samples, but there are also weaknesses in the analysis.

The equivalent radius a, is raised to the power 2 in Equation 5.22 and therefore has a strong influence over the calculation of c_h. The strain patterns around the dilatometer predicted by Finno (1993) Figure 2.12, approximate to plane strain conditions at the membrane. This suggests firstly that, even with a suitable equivalent radius, an axisymmetric analysis is unlikely to be successful in modelling the consolidation behaviour around a dilatometer, and

secondly that it would be more appropriate to use the half-width of the dilatometer blade as a dimension with which to characterize the device and the surrounding consolidation behaviour. It is difficult to judge the adequacy of the analysis because of the uncertainty attached to the reference values of the coefficient of consolidation.

5.7 SUMMARY AND DISCUSSION

Of the four dilatometer readings taken in this study only two, P_0 and P_2, are sufficiently independent to be useful in the evaluation of parameters describing the state of a soil deposit. The P_1 reading is related to P_0 for a given material and P_D is related to the difference $P_0 - P_2$. However, although P_1 provides no information about the state of a particular clay deposit, Marchetti's material index yields a range of values for natural clays. This suggests that the slope of the line in Figure 5.3 may contain some information about a soil's intrinsic properties. Tests on a range of clays with identical states would be required to attempt to identify what this information is.

This study has revealed that P_0 is controlled by the undrained strength and the horizontal stress and is slightly influenced by the mean stress level. The P_2 reading has been found to give an indication of the total horizontal stress, though this result is not supported by the results of field tests. The quantity $P_0 - P_2$ was found to give a good indication of the undrained strength, as does P_D by implication. Perhaps surprisingly neither individual readings nor combinations of them appear to be sensitive to the *OCR* or soil modulus. A possible explanation for this is the dependence of the pressure against the dilatometer blade on the reloading modulus of the destructured soil.

Section 5.5 has shown how Marchetti's correlation between K_0 and K_D for clay deposits with one-dimensional stress histories is highly dependent on the existence of a secondary relationship between the undrained strength and the horizontal stress. It is only when this secondary relationship exists that Marchetti's chart can be expected to provide reasonable results. Even within the limited number of deposits which conform to these specifications, and follow the empirical relationship between undrained strength and horizontal stress, there will be scatter around the K_D : K_0 correlation because of variations in the friction angle, the plastic volumetric strain ratio and N_D.

A high proportion of deposits exhibit complex profiles of horizontal stress and undrained strength which deviate considerably from that which would be predicted from the application of Equations 5.13 and 5.14. This is because the character of individual beds within a single deposit affects how each reacts to aging, stress changes and weathering. The final profile of horizontal stress is the result of an accumulation of a multitude of subtle effects. For these deposits Marchetti's correlation, and all other correlations which rely solely on a relationship between K_D and K_0, cannot be expected to provide a reliable estimate of the horizontal stress.

In a single deposit, which will have usually experienced a uniform geological history, one could expect there to be a local relationship between the undrained strength and the horizontal stress. The dilatometer would be sensitive to this relationship and this is perhaps the reason why it has been found to produce good site-specific correlations. The scatter found when results are compared from different geological regions, is an indication of the range over which the horizontal stress varies for a particular value of the undrained strength. For deposits with one-dimensional stress histories the variation is linked to the soils intrinsic properties, which are partly reflected by the plasticity index. Indeed, a small improvement in the correlation between the *OCR* and K_D has been achieved by Lacasse & Lunne (1988)

by taking the plasticity index into consideration.

Wroth (1979) has shown how empirical and semi-empirical correlations between soil properties can be useful in certain well defined situations; but in general soil deposits with simple geological histories are the exception, and it is therefore important to measure the properties in question directly. It was mentioned in Section 5.5 that Marchetti used the empirical equations of Brooker & Ireland (1965) and Ladd et al. (1977) to estimate his reference values for the horizontal stress. It can be appreciated that in any correlation it is undesirable to estimate rather than measure a reference parameter. In this case the problem was compounded by the uncertainty surrounding which parameters the dilatometer readings were actually sensitive to. By using empirical expressions Marchetti artificially eliminated much of the scatter that would have been present had he used direct measurements of the horizontal stress. The apparent uniqueness of Marchetti's $K_D : K_0$ correlation has encouraged the retention of its general form in most of the research that has been carried out subsequently. As mentioned in Section 2.5.3.1 all of the revisions to Marchetti's correlation have presumed a unique correlation between K_D and K_0.

The results of this study have shown that the P_0 reading is not well suited to evaluation of K_0 as it is controlled by the undrained strength as well as by the horizontal stress. Use of the P_2 reading would seem to hold more promise.

It might be imagined that the dilatometer is well suited to the determination of stress history because its readings are governed by the undrained strength and the horizontal stress, both of which are heavily dependent on the *OCR*. However it is only possible to estimate the *OCR* of a deposit from these two parameters when it has experienced a simple one-dimensional stress history. Equations 5.13 and 5.14 need to be invoked in order to do this. There are other more complex forms of geological unloading: geological unloading with reloading and

non one-dimensional unloading, which will give rise to different relationships between the stress ratio, undrained strength and *OCR*. In these cases, and where other geological processes have taken place, the profiles of undrained strength and horizontal stress may contain little useful information about the stress history.

A separate consideration is the advisability of using the dilatometer readings in a correlation with the *OCR*, even when the limits of its applicability are clearly stipulated. It would seem prudent to leave the step required to infer the *OCR* from profiles of undrained strength and horizontal stress as an active one taken by the engineer. The assumptions and limitations would then be obvious.

The ability of the dilatometer with respect to the measurement of the undrained strength shows more promise. The P_0 reading is dominated by the undrained strength, which explains why Marchetti's $K_D : s_u$ correlation, Equation 2.20 has been reasonably successful for a wide range of soils. However, it is possible to improve on Marchetti's expression by introducing the P_2 reading which appears from this study to be a reasonable measure of the horizontal stress. Figure 5.13 indicates that the quantity $P_0 - P_2$ has great potential for the evaluation of the undrained strength. However, the absence of comparable field results prevents confirmation of this.

The P_0; P_1; $P_2 : t$ tests yielded coefficients of consolidation an order of magnitude higher than the reference values when they were analysed using the solution for the dissipation of pore pressure around the shaft of a cone obtained by Teh & Houlsby (1989). However, the dilatometer appeared to be sensitive to the variation of the consolidation characteristics between the clay beds, which supports the findings of Lutenegger & Kabir (1988), who observed that the dilatometer P_2 reading was able to detect the dissipation of pore pressure.

It was not possible to decide whether it was the reference c_v values or the analysis which was used to evaluate the data, which was responsible for the poor agreement. There were doubts about the choice of the effective radius used to characterize the geometry of the dilatometer blade, and also about the relevance of an axisymmetric solution to the analysis of the dissipation around a blade; but there were also uncertainties about the reference c_v values. The P_0;P_2 test showed little potential in this study as a method for estimating a soil's consolidation characteristics.

Time constraints limited the number of the clay beds that could be prepared in this study and only one, bed OCDX, was tested in a moderately as opposed to lightly overconsolidated condition. The results of this test were consistent with the results of the tests in the normally consolidated and lightly overconsolidated clay beds. However, it was not possible to test a clay bed in a heavily overconsolidated condition. Powell & Uglow (1986) found that the results of dilatometer tests carried out in heavily overconsolidated soils did not exhibit the same trend as tests carried out in soils with less extreme stress histories. The results from fissured and cemented clays also show a deviation. Confidence in the results of this study is therefore confined to intact uncemented clays with *OCR*'s less than about 5.

CHAPTER 6

DILATOMETER TESTS IN SAND
RESULTS AND ANALYSIS

6.1 PREAMBLE

During the period November 1985 to June 1986, 31 sets of dilatometer tests were conducted in a calibration chamber for sand at Oxford University. The work was carried out by Mr. R. Hitchman under the supervision of Professor G. T. Houlsby. An analysis of the results of the tests is presented in this chapter.

6.2 INTRODUCTION

For a number of years the standard penetration test (SPT) and cone penetration test (CPT) have been used extensively for the determination of soil parameters in cohesionless deposits. The extreme difficulty of sampling cohesionless soils for laboratory tests has been the chief reason for the popularity of these in-situ techniques. Before use can be made of the results of in-situ tests it is generally necessary to correlate their readings with known soil parameters. The presence of complicating factors in the field and sampling difficulties have led to the construction and use of numerous calibration chambers for this purpose (Section 3.1). The Oxford University calibration chamber for sand operates using the same principles as most of the other chambers. A description of the chamber is given in Section 6.3.

The ability, in calibration chamber testing, to measure and control the sand density and type as well as the stress state and stress history gives it a great advantage over field calibration.

However, the finite size of the chamber and type of boundary conditions have been shown by Parkin & Lunne (1982) to influence the results significantly. They investigated the effect of boundary conditions and sample size on the laboratory calibration of a cone penetrometer in sand. This was carried out through the use of two sizes of calibration chamber and two sizes of penetrometer probe. Boundary conditions of zero strain and zero stress change were examined. These two boundary conditions can be considered as approximate upper and lower limits to the behaviour that would be measured in a large soil body; stress-controlled boundaries give too flexible a response and displacement controlled boundaries are too rigid. The results of their research were presented in a plot (Figure 6.1) relating the cone resistance to the ratio of the diameters of the chamber and the probe, for samples with different densities and different stress histories. They concluded that boundary condition type only has a large effect when probe induced distortions extend as far as the boundaries and also that, for in-situ tests at least, side restraint is more important than base restraint. The results suggested that the diameter ratio at which the results ceased to be affected by the presence of the sample boundaries was less than 20 for loose sand ($R_d \approx 30\%$) and greater than 50 for dense sand ($R_d \approx 90\%$). These results are for cone penetrometers, but if the dilatometer blade is approximated to a cylindrical probe with the same cross-sectional area, they can be used as a rough guide to dilatometer behaviour. Using this simplification the diameter ratio of the dilatometer and Oxford calibration chamber combination is 22 suggesting that only in loose sand would the chamber be of adequate size to avoid boundary problems.

Ekström (1986) carried out a series of dilatometer tests in a large rigid-walled pit filled with a compacted coarse sand. Glötzl cells were positioned at the pit walls and at points within the sand. By conducting the tests at various distances from the Glötzl cells Ekström was able to produce a contour plot of the increase in horizontal stress induced by the dilatometer. Stress changes were observed 700 mm below the level of the dilatometer blade and 750 mm from it horizontally. Even with consideration of the error caused by the use of stiff measuring devices, these findings provide additional evidence that the disturbance caused by

the dilatometer would have reached the sample boundaries in the Oxford calibration chamber tests.

In a laboratory investigation, Davidson & Boghrat (1983b) found the horizontal displacements around a penetrating dilatometer could only be detected 100 mm away from its edge in loose sand and 160 mm away in dense sand. A difference in the ability to detect movements and stress changes may explain the apparent discrepancy between these findings and those of Ekström (1986) as even very small displacements can cause significant stress changes in sand.

Bellotti et al. (1979) conducted eight dilatometer tests in a calibration chamber. In six of the tests the boundary conditions of constant horizontal and vertical stress were used. For the remaining two tests the horizontal boundary condition was changed to one of zero average strain. The use of a different boundary condition in an otherwise identical test resulted in a K_D value 1.43 times higher. This was observed in a chamber 1200 mm in diameter which suggests that, even at this scale, boundary conditions affect the result of the dilatometer test. This may provide an additional explanation to that of the effect of soil aging cited by Marchetti (1985), for the discrepancy between laboratory and field tests.

Baldi et al. (1986) found that tests conducted under conditions of constant horizontal and vertical stress gave more consistent results than those conducted with the average horizontal strain set to zero. He attributed this to an inconsistent increase in the horizontal stress at the boundary required to satisfy the zero average-strain boundary condition during the dilatometer penetration.

Schnaid (1990) investigated chamber size effects using three cone-pressuremeters with different diameters. He also carried out a parametric study using a one-dimensional cavity expansion analysis. His experimental results are particularly relevant because the cone resistance q_c, and limit pressure P_L, of the cone-pressuremeter are comparable to the

dilatometer P_D and P_1 values respectively, and also his tests were carried out using stress-controlled boundaries. The results clearly showed that both q_c and P_L were affected by the proximity of the sample boundary though the effect was only significant in medium ($R_d \approx$ 65%) and dense sands ($R_d \approx 90\%$). He found, as did Parkin & Lunne (1982), that in loose sand ($R_d \approx 30\%$) the chamber size was unimportant. His numerical analysis produced good agreement with his experimental findings. The effect of soil stiffness and friction angle on the variation of the limit pressure were investigated for different cone-pressuremeter diameter to sample diameter ratios. He found that an increase in sample stiffness only has an important effect in loose sand. The friction angle, which is highly correlated with the soil density, was found to exert a major influence on the sensitivity of the limit pressure to variations in the probe/chamber diameter ratio. High friction angles increased the sensitivity.

These observations taken together lead one to the conclusion that the size of specimens presently used in calibration chambers is inadequate to avoid significant movement at the boundaries and that because of this, the most consistent results are obtained if the stresses are kept constant at the boundaries. Little can be done at present to account for the sample size effect on the dilatometer readings, other than to recognise its existence. On a positive note Schnaid (1990) found that though cone-pressuremeter readings were affected, the ratio between them was relatively independent of sample size. In the dilatometer tests reported here boundary conditions of constant vertical and horizontal stress were used throughout.

6.3 TEST APPARATUS AND METHODOLOGY

The purpose of calibration chambers has changed subtly from its original which was the production of artificial sand deposits with simple stress histories for empirical correlation, to the parametric study of in-situ devices. In this series of dilatometer tests, in contrast to other

work using calibration chambers, Parkin & Lunne (1982), Baldi et al. (1986) and Last et al. (1987), no effort was made to prepare the samples under conditions of zero lateral strain; only stress control was used. The broad link between the overconsolidation ratio and the stress ratio K, of the sand bed was thus broken, permitting the exploration of a wider range of stress states. There is a drawback to this approach, namely that some of the laboratory test stress states are unlikely to exist in natural sand deposits; the advantage is that it offers the possibility of the independent examination of the influence of the vertical and horizontal stresses, the stress ratio, the overconsolidation ratio and the mean stress level.

A series of 31 dilatometer tests were carried out in 14/25 Leighton Buzzard sand at three different densities and a variety of stress states. The philosophy behind this series of tests differed slightly from the philosophy behind the tests in clay which was described in Section 1.2. The same systematic examination of stress level, stress ratio and degree of overconsolidation, was carried out but because in sands there is no unique correlation between the stress conditions and the soil density, it was also necessary to vary the initial densities. For loose, medium and dense samples, tests were conducted nominally at three mean stress levels, three stress ratios and at the lowest mean stress level in an overconsolidated state. Figure 6.2 shows the nominal stress states in the p' : q plane. The series of tests was not comprehensive for each density; the 31 tests comprised 13 dense tests ($R_d \approx 83\%$); 14 medium tests ($R_d \approx 62\%$); and 4 loose tests ($R_d \approx 28\%$).

Figure 6.3 shows a particle size distribution curve for the Leighton Buzzard sand. It shows that, excepting a small fraction of fines, the distribution of particle sizes is reasonably uniform between the No. 25 sieve (0.6 mm aperture size) and the No. 14 sieve (1.18 mm aperture size). The mechanical properties of Leighton Buzzard sand have been investigated by Stroud (1971), Palmeira (1987), Pedley (1990) and Schnaid (1990). Two separate batches of Leighton Buzzard sand have been used at Oxford and these were found by Pedley (1990) to

exhibit slightly different peak strengths. This has been attributed to differences in the angularity of the sand grains in each batch. The particular batch used in this study was yellow Leighton Buzzard sand, so called because of its iron staining. The properties of this sand were studied by Palmeira (1987) and its mechanical behaviour was described as being similar to the Leighton Buzzard sand tested by Stroud (1971). Using the standard procedures described in BS1377 (1975), Palmeira established the following properties for yellow Leighton Buzzard sand: e_{min} = 0.49, e_{max} = 0.79 and G_S = 2.65.

The Oxford calibration chamber for sand has been described by Houlsby & Hitchman (1988) but for completeness a description of the equipment and its operation is given here. Figure 6.4 shows the main features of the chamber. As with other similar calibration chambers, Chapman (1974) Bellotti et al. (1982) and the calibration chamber for clay described in Chapter 3, it uses water pressurized rubber membranes, around the sample of sand, to apply stress-controlled vertical and horizontal boundaries. The sand sample is cylindrical and measures 990 mm high and 930 mm in diameter. The water pressures were monitored using conventional pressure gauges and electronic transducers. Sample boundary movements were measured locally using LVDTs. These were mounted in the chamber walls with their tips resting on the rubber membranes. Three were situated in a vertical plane at the sample circumference at depths of 250, 500 and 750 mm and a fourth was positioned at the centre of the base of the sample. No attempt was made to measure the overall boundary strain by monitoring the water volume change at the air-water interface.

The chamber was filled with sand using pluvial deposition. Dry sand was rained from a hopper through a perforated plate and diffuser sieves, directly into the calibration chamber. Perforated plates with different sized apertures were used to control the intensity of sand flow, which in turn controls the sand density. Three rates of deposition were used to prepare the three different density samples; the highest flow rate gave the loosest sand specimen. The advantages of this preparation technique are that samples of good repeatability and

homogeneity can be formed over a large range of relative densities. The samples are also similar in structure to natural sand deposits, Bellotti et al. (1982). The sample was prepared inside a former to preserve its cylindrical shape, and was also enclosed in an air-tight membrane. This was needed because it was necessary to apply a 30 kPa vacuum to maintain the shape of the sample while the former was removed and until the lateral membranes were filled with water and could provide support. The sand was levelled to 5 mm below the top of the chamber before the top lid was bolted down. This gap was left to guarantee that, even if the height of the sample did not decrease significantly during the application of the boundary stresses, there would be a water filled gap between the base of the sample and the base of the chamber sufficient to ensure that the bottom boundary was flexible and stress controlled. A grid was placed between the base membrane and the base of the chamber to allow water to spread laterally and thereby ensure that the sample was lifted uniformly.

The sample was brought to the required stress state by applying incremental stress changes with short time intervals between them to allow the sample to adjust. For normally consolidated samples the loading was monotonic up to the desired stress level. For overconsolidated samples the stresses were increased up to their maximum values and then decreased to their test level. Samples were prepared with constant K loading conditions and tested at constant boundary stresses. As with the tests in clay, the dilatometer was jacked through a hole in the top lid of the chamber into the sand sample at 20 mm/sec and a standard test (Section 2.3) was conducted every 100 mm. Outputs from the transducers measuring boundary pressures and movements, and dilatometer penetration force and displacement were monitored by a data logger and recorded on magnetic disk.

At the end of each loose and medium test the sand density was obtained by weighing the sand in the chamber and dividing by the volume of the sample prior to testing. The relative densities of the first four dense samples were calculated in the same way and were found to be very consistent (± 0.5%). The densities of the remaining dense samples were not

determined and so in the analysis a single relative density of 83% has been ascribed to each dense sample. Schnaid (1990) prepared his samples using the same apparatus and technique as described above but found a maximum variation of relative density for the dense tests of ± 3%.

Catching experiments conducted during the pouring of a dense sample indicated a variation in relative density of 1% between the top and base of the sample. Schnaid (1990) also conducted catching experiments using thin-walled containers placed inside the chamber and found variations of relative density within 2% for loose and dense samples and within 4% for medium samples. He detected no tendency for the density to increase or decrease with depth.

6.4 ANALYSIS OF RESULTS

A complete set of results for the 31 dilatometer soundings is given in Table 6.1. Three values for P_0, P_1, P_D, the stress state, the maximum past stresses, and the relative density are given for each sample. These correspond to dilatometer tip penetrations of 500, 600 and 700 mm.

6.4.1 P_0 and P_1 Readings

The values of P_0 and P_1 were calculated from the A and B readings using Equations 5.1 and 2.1 respectively. Figure 6.5 shows a plot of the variation of the P_0 and P_1 readings with depth in normally consolidated dense, medium and loose samples subjected to the same stress regime. The readings generally reach a plateau after the dilatometer tip has penetrated 400 to 500 mm. The depth relevant to the pressure readings is that of the centre of the membrane which is 90 mm above the dilatometer tip level. The membrane pressure readings therefore stabilise at depths of approximately 300 to 400 mm which is consistent with the action of

interference from the rigid chamber lid boundary. In the comparisons made in this and the following sections readings at three depths have been used from each sample. These are at dilatometer tip penetrations of 500, 600 and 700 mm which correspond to centre of membrane depths of 410, 510 and 610 mm. In the following analyses the horizontal and vertical stresses at the level of the centre of the membrane have been used in combination with the dilatometer thrust taken at tip penetrations 90 mm below this level; i.e. when the blade has reached its test depth.

Returning to Figure 6.5 it can be seen that an increase in sample density causes the P_0 and P_1 readings to rise. The amount by which the two readings go up is in direct proportion. This can be seen clearly in Figure 6.6 which shows a plot of P_0 against P_1 for all the tests. Closely spaced parallel lines can be drawn through the data for the dense, medium and loose samples, so that, other than a vague indication of density, the P_1 reading provides no additional information about the state of the soil as it is directly related to the P_0 reading.

The P_0 and P_1 readings have been used by Marchetti (1980) in the form of the material index, I_D, Equation 2.6 and the horizontal stress index, K_D, Equation 2.7. He has related these two indices to a description of the material which is summarised in the chart shown in Figure 2.5; I_D is linked to the grading of the material and K_D to its density. Figure 6.7 shows a plot of I_D against the relative density for each sample. The column of data at a relative density of 83 % represent the dense samples to which a single value of the density was ascribed. The plot shows that for Leighton Buzzard sand, which is a coarse sand, the material index generally varies between two and four and that it is independent of the relative density. According to Marchetti's chart this range of material index indicates a silty-sand to sand material.

Marchetti (1980) uses the difference between the P_0 and P_1 readings to find the dilatometer modulus, E_D, Equation 2.5, which has been correlated to the soil's one-dimensional constrained modulus. It has been found that the dilatometer modulus increases with relative density and horizontal stress, Konrad (1989). Figure 6.8 shows a plot of P_1-P_0 against mean stress for the dense and medium tests. Different symbols have been used for the normally consolidated and overconsolidated tests. The loose tests have not been included because none were tested in an overconsolidated state. The results of overconsolidated tests plot within the scatter of the normally consolidated tests. No independent measurements of soil modulus were made in this study but it is well known from the results of triaxial tests that large increases in modulus can be expected if sands are prestressed. The dilatometer modulus appears to have no ability to detect this. Figure 6.9 compares the P_0 and P_1 profiles of two medium density sand samples existing under the same stress conditions. One is in a normally consolidated state and the other has been subjected to prestressing which has left it with an *OCR* of 2.5. There is very little difference between the two sets of profiles indicating that one of the chief effects of overconsolidation, namely an increase in soil stiffness, is not detectable with the dilatometer. Jamiolkowski et al. (1988) also showed that the dilatometer results are not influenced significantly by the soil stiffness.

6.4.2 Penetration Thrust

The Penetration thrust was measured by means of a load cell located in the rods just above the dilatometer blade, Figure 3.11. The results of penetration thrust measurements for dilatometer soundings in similarly stressed dense, medium and loose samples are presented in Figure 6.10. The plot shows that the penetration resistance increases with density and with depth for each sounding. The trend of increasing resistance with depth is more marked in the denser samples. For tip penetrations of 400 mm or less, part of the blade or rods below the level of the load cell is free from frictional resistance and one would therefore expect the

resistance to increase until this penetration has been exceeded. In the medium and loose samples the penetration load is constant for tip penetrations greater than 400 mm suggesting that they are uniform in density. The continued, though reduced rate of, thrust increase with penetration beyond tip depths of 400 mm for the dense test could be the result of sample non-uniformity. Typically the dense samples produced similar shaped profiles to the medium and loose samples. There is a slight drop in the penetration load at the end of each profile which was also found in profiles of cone end bearing stress obtained using the same calibration chamber and reported by Hitchman & Houlsby (1988). They attributed this to the effect of the proximity of the base membrane.

Figure 6.11 shows a plot of P_0 against P_D. The data are from tests at three different nominal densities, three stress levels, three stress ratios and sand in overconsolidated and normally consolidated states. The simple relationship between the two measurements indicates that they are directly dependent and therefore no additional information regarding the above mentioned variables can be determined by taking both as opposed to one of the readings. Apart from providing an indication of soil type it appears therefore that two of the three readings, P_0, P_1 and P_D are redundant. Equation 6.1 gives a best fit line to the data of Figure 6.11.

$$P_0 = 18.75\ P_D \qquad \text{(Eqn 6.1)}$$

Results from calibration tests in Italy and field tests reported by Campanella & Robertson (1990) also suggest that there is a dependence of the P_0 reading on the thrust required to advance the blade.

6.4.3 Horizontal Stress

In the previous two sections it was shown that the three dilatometer readings recorded in this series of tests, are directly related to one another; and that in some cases, the relationships between them are sensitive to density. Unfortunately the sensitivity is not sufficient to make it worthwhile to attempt to infer the soil density from the dilatometer readings alone. To make use of the dilatometer readings it is necessary to introduce another independent parameter and in this section the horizontal stress level has been selected for this purpose. The reason for choosing the horizontal stress in preference to the vertical stress, which is far easier to estimate in the field, is illustrated by Figures 6.12 and 6.13. Figure 6.12 shows a plot of the P_0 value against the vertical stress (any of the three dilatometer readings P_0, P_1 or P_D could have been chosen as they are directly related); no correlation appears to exist. However, in Figure 6.13, which shows a plot of the P_0 value against the horizontal stress, a clear pattern of dependence on the density can be seen. The remainder of this section is concerned with an attempt to estimate the horizontal stress from the dilatometer readings.

The importance of the horizontal stress in influencing the results of in-situ tests in cohesionless soils has been well established experimentally; Bellotti et al. (1982), Baldi et al. (1985), Jamiolkowski et al. (1985), Houlsby & Hitchman (1988), Schnaid (1990) and Nutt (1993) have all found that the magnitude of the horizontal stress governs the CPT end bearing stress q_c. However, because the readings are also dependent on the soil density, no satisfactory method has yet been found to back-calculate the value of the horizontal stress from the results of a CPT. However, the cone-pressuremeter, which in addition to measuring the end bearing stress q_c, also measures a limit pressure P_L, can be used to estimate the horizontal stress with reasonable accuracy, Schnaid (1990). The readings taken with the cone-pressuremeter are both dependent on the horizontal stress and soil density and can be combined to eliminate the soil density to give an estimate of the horizontal stress.

The perceived advantage the dilatometer has over the CPT, in common with the cone-pressuremeter, is that it can measure a post-installation horizontal pressure in addition to a penetration thrust. In consequence the problem of determining the horizontal stress in cohesionless deposits with the dilatometer is reduced to one of estimating the increase in stress caused by insertion. The methods described in Section 2.5.3.2 all use the end bearing capacity of the dilatometer q_D or the cone penetrometer, q_c, to predict the insertion generated increase in horizontal stress which must be subtracted from the P_0 reading to give the original in-situ horizontal stress. However none of the methods gives a prediction with an acceptable degree of accuracy; even when they are applied to the results of calibration chamber tests. Baldi et al. (1986) used an empirical fit to their calibration chamber results to back calculate the applied stress ratio. Though the range of applied stress ratios was small (0.4 to 1.2) they were only able to predict a value of stress ratio to within 25 %.

The suggested procedure for the determination of the horizontal stress from the dilatometer test put forward by Jamiolkowski et al. (1988) and detailed is Section 2.5.3.2 has been used with minor modifications to examine the results of this study. The method illustrates well the problems inherent in trying to establish the horizontal stress in cohesionless deposits through use of the dilatometer readings.

The increase in horizontal stress produced by insertion of the dilatometer, which typically varies between one and ten times the original horizontal stress, has been found to depend on the stress level and the soil density. These two variables have been encapsulated in a single parameter, namely the state parameter, by Been & Jefferies (1985). The state parameter is defined as the difference between the void ratio of the sand and its critical state void ratio at the same mean effective stress (Equation 6.2) where the critical state is defined as the void ratio at which the material deforms under conditions of constant volume and effective stress. The state parameter has precisely the same definition as that of the parameter D introduced

by Wroth & Bassett (1965).

$$\psi = e - e_{cs} \qquad \text{(Eqn 6.2)}$$

Where positive the state parameter indicates contractive behaviour and where negative it indicates dilative behaviour. Stroud (1971) gives the critical state line for Leighton Buzzard sand in the e - $\ln(s')$ plane as:

$$e = 0.88 - 0.0248 \ln(s') \qquad \text{(Eqn 6.3)}$$

In a series of simple shear tests on Leighton Buzzard sand he found that upon shearing the ratio σ_2/s' increases from the initial K_0 value to a constant approximately equal to 0.74. He found this value to be independent of the initial void ratio and of the stress level. If the constant ratio between σ_2 and s' at the critical state is used, the equation for the critical state line in the $e - \ln(p')$ plane becomes:

$$e = 0.878 - 0.0248 \ln(p') \qquad \text{(Eqn 6.4)}$$

In the calibration chamber tests the stress level and sand density are known accurately and it is a simple matter using Equation 6.4 to calculate the state parameter for each sample. Figure 6.14 shows a plot of the ratio K_D/K against the state parameter. Equation 2.13 which was fitted to the results of tests carried out in Hokksund and Ticino sand by Jamiolkowski et al. (1988) is also shown on the plot and fits the Leighton Buzzard sand data well enough to exclude the need for a separate correlation. The K_0 of Equation 2.13 has been replaced

by K in Equation 6.5 because no attempt was made to prepare the sand samples under conditions of zero lateral strain.

$$\frac{K_D}{K} = 1.35\ e^{-8.08\psi} \qquad \text{(Eqn 6.5)}$$

However Equation 6.5 cannot be used to predict the horizontal stress in the field because knowledge of the void ratio and the critical state line parameters is required. To overcome this problem Jamiolkowski et al. (1988) used a second relationship between the cone end resistance q_c and the state parameter (Equation 2.14). In principle q_D could be used to establish a similar relationship. However the dilatometer penetration thrust was measured just above the blade and assumptions which involve the use of the P_0 pressure and the sand friction angle are required to estimate q_D, Schmertmann (1982). A simple approximation to q_D was made by dividing the dilatometer penetration thrust P_D by the cross-sectional area of the blade a_D. A plot of this approximate dilatometer end bearing stress normalised by the mean stress level against the state parameter is shown in Figure 6.15. The data approximately fit Equation 6.6.

$$\frac{P_D}{a_D\, p'} = 44.58\ e^{-9.91\psi} \qquad \text{(Eqn 6.6)}$$

The state parameter can be eliminated from Equations 6.5 and 6.6 to give Equation 6.7.

$$\frac{K_D}{K} = 0.061 \left(\frac{3\ P_D}{a_D\ \sigma'_V(1+2K)}\right)^{0.816} \qquad \text{(Eqn 6.7)}$$

Figure 6.16 shows a comparison between stress ratios determined through use of Equation 6.7 and the stress ratio of the sand sample. The predictions are very poor. The reason for the large scatter is the dependence of P_0 on P_D shown in Figure 6.11. Essentially this makes Equations 6.5 and 6.6 highly ill-conditioned for the elimination of the state parameter involved in forming Equation 6.7.

6.4.4 Friction Angle

As mentioned in Section 6.2, there are good reasons to expect similarities between the dilatometer P_1 and P_D, and the cone-pressuremeter P_L and q_c; both sets of readings are dependent on the horizontal stress and the soil density. Schnaid (1990) was able to correlate the ratio $(q_c-\sigma_H)/(P_L-\sigma_H)$ to the friction angle. Unfortunately the P_0, P_1 and P_D readings are related to the same parameters in too similar a way, as demonstrated by Figures 6.6 and 6.11, for a comparable method to be successful with the dilatometer. However the friction angle is highly dependent on the density and one would therefore expect the P_0 value to correlate well with the friction angle and horizontal stress as it was shown to do with the density and horizontal stress in Figure 6.13. This section explores the value of this approach.

No measurements of the friction angle were made in connection with this test series, but by using the empirical expression (Equation 6.8) suggested by Bolton (1986) a reasonable value can be ascribed to each sand sample on the basis of the density, the stress state and the critical state friction angle of the material.

$$\phi = \phi_{cs} + 3\,[R_d\,(Q-\ln(p'))-1] \qquad \text{(Eqn 6.8)}$$

where

ϕ = Triaxial peak friction angle

ϕ_{cs} = Angle of friction at the critical state

Q = A constant depending on the compressibility and mineralogy of the sand

p' = Mean stress in kPa

Bolton also suggested an empirical expression for the plane strain friction angle which has been recommended by Schmertmann (1982) as the most appropriate angle to try to compute from the dilatometer readings. The data upon which Bolton's plane strain friction angle is based is limited, however, and Pedley (1990) has expressed doubts about its reliability. The expression for the triaxial friction angle was therefore used to estimate an appropriate friction angle for each level within the calibration chamber.

Bolton advised using a value for Q of 10 in silica sands. Schnaid (1990) conducted a series of triaxial tests on Leighton Buzzard sand and derived a critical state friction angle of 34° from the results. Schnaid's tests were conducted on a different batch of Leighton Buzzard sand and so a difference in peak strength would be expected. The critical state line however remains the same, because the variation in angularity between batches only affects the rate of dilation. Schnaid's critical state friction angle of 34°, which lies within the range $33° < \phi_{cs} < 35°$, typically quoted for Leighton Buzzard sand, was used with Equation 6.8 to calculate the friction angle for the 500, 600 and 700 mm levels in each sample.

Bolton recommended that Equation 6.8 should only be used in situations where the mean stress is greater than 150 kPa. This was because his data below this stress level were limited. However Pedley (1990) found that Bolton's expression was valid for the lower stress levels that were used in some of the tests in this study.

A relationship, analogous to the one shown in Figure 6.13, was found to exist between the P_0 reading, the horizontal stress and the friction angle:

$$\frac{P_0}{\sigma'_H} = 0.741\ \phi\ -\ 24.1^\circ \qquad \text{(Eqn 6.9)}$$

A comparison of the friction angles derived from Equation 6.9 with those obtained using Bolton's formula, Equation 6.8, is shown in Figure 6.17. The figure shows that if the horizontal stress is known the friction angle can be determined to ±3°. However the requirement to know the horizontal stress limits the usefulness of Equation 6.9.

6.4.5 Sample Strains

During the dilatometer penetration measurements were made using LVDTs of the movement at the lateral sample boundary at depths of 250, 500 and 750 mm. A single measurement, also using an LVDT, was made of the vertical movement at the centre of the base of the sample. Records of the boundary movements were made for 15 of the 31 soundings, but unfortunately these comprise only loose and medium density samples. Figure 6.18 shows the variation of the recorded boundary movements with depth of dilatometer penetration for Test 29 which was conducted in a medium density sample. Figure 6.19 shows the boundary movements for test 26 which was a loose sample. There was some variation in detail between the results of individual loose and medium tests but the following comments regarding Figure 6.18 and Figure 6.19 apply generally to all the tests in which boundary movements were recorded.

In the medium density tests the movements were predominantly dilatant and in the loose tests they were predominantly contractive. In the medium density tests the overall movements increased with the depth they were recorded at i.e. the maximum movements were recorded at the 750 mm depth. In the loose tests there was no distinct correlation between the position and the magnitude of the movements recorded. After the dilatometer blade reaches and

passes the 500 mm depth, the movements recorded by the horizontal LVDT at this level reverse direction in the medium density samples but not in the loose samples. The base movements are higher in the medium density samples than the loose samples.

The pattern of behaviour indicated in Figures 6.18 and 6.19 can be explained once it has been recognised that each series of readings from a particular LVDT represents only a portion of the total horizontal movement profile. This is due to the finite size of the sand specimen. A larger part of the total horizontal movement profile can be reconstructed, albeit in an approximate way, if the results of Figures 6.18 and 6.19 are plotted using the depth of the reading above the dilatometer tip, with the displacement profiles from the high and middle LVDTs aligned with the profile from the low LVDT. This effectively unifies the measurements at the three depths. The resulting profiles for the loose and medium density samples are shown in Figure 6.20. The readings, presented in this way, clearly indicate that the dilatometer is densifying the loose sample and loosening the medium density sample. The reversal of the movements in the medium sample is probably due to the closure of the cavity behind the dilatometer blade. The effect is obscured in the loose sample by its overall contraction.

For the purpose of comparison the horizontal movement at the level of the dilatometer tip taken from the reconstructed horizontal movement profile, was used as an index of the dilation of each sample. Figure 6.21 shows that there is a rough relationship between this horizontal movement and the state parameter (Section 6.4.3) for each sample. The state parameter incorporates the void ratio and mean stress level and gives an indication of a sand samples tendency to dilate. The scatter can be attributed to the dilatometer induced disturbance of the stress system and the very small movements recorded.

The orientation of the dilatometer blade to the vertical profile in which the measurements were made was varied in some of the tests but no effect of this could be detected in the

results.

Konrad (1988) conducted a series of laboratory dilatometer tests in a chamber in which the sand was poured around the blade in a known state. Konrad (1989) compared the results of these tests with standard disturbed tests in which the blade was pushed into the chamber and was able to infer the sand state after insertion. The results showed that loose sand ($R_d \approx$ 34%) contracts around the dilatometer blade while dense sand ($R_d \approx 88\%$) dilates. His findings agree with the change in state inferred from the observation of external boundary movements found in this study.

6.5 DISCUSSION

During dilatometer penetration, particularly in high density samples or those under a high lateral stress, the force needed to advance the blade leads to a significant unloading of the sample at the level of the dilatometer. This is chiefly because the sample boundaries cannot support shear stresses. Even though the dilatometer penetration is halted during the expansion of the membrane the rods are prevented from moving by the driving assembly and still carry a high proportion of the penetration load. The relatively small drop-off in load during the break in penetration can be seen clearly in Figure 6.10. The load taken by the blade and rods reduces the stress against the top of the sample. A qualitative indication of the equilibrium stresses against the top, base and side of the sand sample is shown in Figure 6.22. The distribution of the reduction in the vertical stress within the sample is complex but it is probable that the vertical stress experienced by the sand adjacent to the dilatometer membrane is reduced by this effect. Due to the method by which vertical stresses are applied to the sand sample (by reaction against the lid of the chamber) this is the opposite to what would happen in the field i.e. the vertical stress around the blade would increase. In the field the in-situ stresses in the soil surrounding the dilatometer blade are modified by soil disturbance and the

high stiffness of the blade. In calibration chambers there is the additional problem of different stress changes external to this disturbed zone due the phenomenon described above. If more of the dilatometer penetration load were taken by the blade below the level of the membrane, as seems likely, calibration chambers in which the vertical stress is applied from the top of the sample would be more realistic in this respect. This problem is greater in the dilatometer than cone-penetrometer because of its higher cross-sectional area which produces correspondingly higher penetration loads.

The LVDT measurements described in Section 6.4.5 indicate that distortions induced by the dilatometer reach the boundaries of medium density and loose samples. On the basis of the findings of Parkin & Lunne (1982) it could be expected that if there were an effect on the dilatometer readings from the interference of the penetration distortions and the sample boundaries, it would be more pronounced in the denser samples. However, without conducting dilatometer tests in different sized samples it is impossible to estimate the separate influence of different sample size and density on the readings.

Figure 6.20 shows how the boundary movements are quite complex and how applying a zero average displacement boundary is unlikely to control the lateral straining of different density samples in the same way. This provides an explanation of why the results of in-situ devices in calibration chambers where strain controlled boundaries have been used are generally less consistent than tests where stress control was used. Foray (1986) attempted to tackle this problem by constructing a calibration chamber with three separate lateral membranes which could be monitored and controlled independently.

It has been shown that a knowledge of the horizontal stress is important in the evaluation of the dilatometer results, but that it is difficult to exploit this relationship to derive the horizontal stress from the results with useful accuracy. This is because of the interdependence of the three dilatometer readings P_0, P_1 and P_D. The three readings are essentially

measuring the same response from the sand: that of the pressure required to open a cavity. It is significant that the relationships based on the use of q_c produce less scatter in the evaluation of K, Baldi et al. (1986). This is probably because the cone penetration mechanism differs from the dilatometer penetration in a way which makes q_c more independent from P_0 than P_D.

The following are cited by Jamiolkowski et al. (1988) as being of fundamental importance in determining the geotechnical behaviour of a sand deposit: the initial total vertical and horizontal stresses, the initial pore pressure, the macro and micro fabric, the initial void ratio, and the preconsolidation pressure. This study has shown that the dilatometer expansion pressures are sensitive to the stress state and the density but insensitive to the degree of overconsolidation. It appears that the penetration of the dilatometer causes sufficient disturbance to obliterate the stiffening effects of prestressing. It can be presumed that in the field the disturbance will also destroy the soil fabric limiting the ability of the dilatometer to detect some of the important aspects of the soil behaviour.

The results have shown that to evaluate the horizontal stress from the dilatometer readings an additional independent relation involving the horizontal stress and the state parameter is required. It may be that in some natural deposits of sand which have simple stress histories such a relationship exists. There would then be a unique correlation between P_0 and K_0 which would be potentially detectable with the dilatometer. The interdependence of the penetration thrust and the dilatometer expansion readings in the laboratory also does not preclude the possibility that in the field the P_D reading may provide some valuable information.

It is important to emphasize the limitations of calibration chamber tests. The finite size of the samples; the simplicity of the boundary conditions that are practical; and the inability to

model the important environmental factors such as aging, cementation and cyclic strain history found in natural deposits, all combine to decrease the relevance to field testing of correlations derived from their use. In their favour they provide the opportunity to identify the parameters of greatest importance to the measurements being made and thereby the best form of any correlation.

In the field prestressing of a sand deposit increases its stiffness and stress ratio. The dilatometer has been shown to be incapable of measuring the increase in stiffness but is able to detect the influence of the horizontal stress level. If it is able to detect the increase in stiffness associated with prestressing in the field as has been suggested by Schmertmann (1986), this must be achieved through the detection of the increase in horizontal stress associated with overconsolidation and not through direct measurement. This is an important finding because it allows the engineer to anticipate in which particular geological situations use of the device will be most advantageous.

CHAPTER 7

CONCLUDING REMARKS

Geological time and fabric features influence the mechanical behaviour of natural soil deposits. Samples prepared in calibration chambers differ from natural deposits in this respect, in their finite size and in the simple boundary conditions that are possible. Nevertheless, calibration chambers enable the soil parameters which influence the measurements taken with in-situ devices to be identified through the high degree of control they allow over the stress state, stress history and sample density.

The one-dimensional consolidation apparatus and calibration chamber for clay were found to be capable of producing reasonably uniform beds of clay. The model of critical state soil mechanics provided a useful framework for the understanding and control of the clay bed preparation procedures. The dilatometer tests yielded consistent readings in the central portion of each clay bed. Uniform beds of sand, which also gave consistent dilatometer readings in their central portions, could be produced in the calibration chamber for sand; though movements at the boundaries and unloading of the sample due the high dilatometer thrust were significant problems.

In the tests in clay only two out of the four measurements taken with the dilatometer were independent, and the analysis of the results in Chapter 5 has concentrated on the P_0 and P_2 readings. The conventional interpretation of the dilatometer test in clay relies heavily on the value of P_0. The results of this study indicate that P_0 is chiefly controlled by the undrained strength and the level of the horizontal stress. It is only when additional information about the soil is available that it is possible to use the P_0 reading to find either s_u or σ_H explicitly. In Marchetti's correlation for the estimation of K_0, this information takes the form (tacitly)

of a natural link between the undrained strength and the horizontal stress which exists for soil deposits with simple one-dimensional stress histories. Even so there is uncertainty associated with this relationship due to its dependence on other soil parameters. A general correlation between P_0 and K_0 does not exist. Marchetti's $K_D : s_u$ correlation is more sound than the $K_D : K_0$ correlation because the undrained strength dominates the P_0 reading. However, the importance of the horizontal stress to the P_0 reading is sufficient to cause scatter in the correlation. The P_2 reading gave a good indication of the horizontal stress in this study though this result is not supported by the results of field tests. The quantity $P_0 - P_2$ was found to be strongly related to the undrained strength.

In the tests in sand, the three measurements taken were interdependent in that they appeared to be measuring the same soil response, that of the pressure required to open a cavity in the sand. The analysis in Chapter 6 concentrated on the P_0 reading as this is the most important reading in the conventional interpretation of the dilatometer, but the use of P_1 or P_D would have yielded similar results. The P_0 reading in sand appeared to be controlled by the state parameter and the horizontal stress, but it was not possible to take advantage of this and evaluate the horizontal stress from the readings. In sands the interdependence of the readings presently taken using the dilatometer severely restricts the usefulness of the device. In most cases the in-situ measurement of individual parameters using the dilatometer requires a knowledge of other soil parameters in order that the disturbance caused during the penetration process can be accounted for.

In both sands and clays none of the readings nor combinations of them were found to be sensitive to the stress history of the deposit; and by implication they were therefore insensitive to the soil stiffness also. The correlation of the dilatometer readings to parameters to which it is not sensitive, such as the soil stiffness and degree of overconsolidation, is an indication

of the existence of inter-relationships between these parameters and those which do effect the readings. In general inter-relationships between soil parameters which are well defined enough to be useful do not exist, and therefore correlations which depend heavily on their existence cannot be relied upon. Correlations between the dilatometer readings and the *OCR* and soil stiffness fall into this category.

The dilatometer appeared to be able to detect variations in the coefficient of consolidation of clay beds. A method of analysis is needed which takes proper account of the geometry of the device, as approximating it to a cone with the same cross-sectional area and using an axisymmetric analysis, gave predictions of c_h which were nearly an order of magnitude too high.

It should be noted that previous studies of the dilatometer in calibration chambers for sand have concentrated on the preparation of samples under conditions of zero lateral strain. In practice this results in a limited range of stress ratios, with high K_0 values associated with a cycle of overconsolidation. A different philosophy was adopted in the calibration chamber tests for sand and clay described in this study. The stress states and stress histories were chosen to allow the independent examination of their effects. Had the samples been prepared under K_0 conditions it would have been difficult to identify the importance of the undrained strength and horizontal stress in the tests in clay, and the importance of the state parameter and horizontal stress in the tests in sand, because of the link between these properties and the *OCR* which one-dimensional stress histories induce.

The dilatometer is a robust and simple device and it has been found to provide useful information in a wide range of soil deposits. It appears therefore that in many soil deposits inter-relationships between the soil parameters, upon which the present dilatometer correlations are dependent, exist to a useful degree.

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