THE MASS COMPRESSIBILITY OF FRACTURED CHALK

A thesis submitted to the University of Surrey for the Degree of Doctor of Philosophy in the Department of Civil Engineering

by

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NOTATION

$A_j$ Contact area ratio.
$a$ Cross sectional area of cone
$B_f$ Width of foundation
$B_p$ Width of foundation or diameter of plate
$b$ Dia. of asperities
$C_c$ Compression index
$D_m$ Constrained modulus ($1/m_r$)
$D_p$ Plate diameter
d Geophone spacing
$\delta_n$ Normal deformation.
$\delta_{nj}$ Normal closure of a single discontinuity.
$\delta_{nr}$ Normal deformation of intact rock.
$\delta_{nt}$ Total normal deformation of intact rock and discontinuity.
$\delta_s$ Shear deformation.
$E$ Young's modulus.
$E^+$ Unload-reload modulus from pressuremeter tests
$E_c$ Post-collapse modulus.
$E_{dynamic}$ Dynamic modulus
$E_r$ Re-load modulus.
$E_i$ Intact modulus.
$E_{ih}$ Initial horizontal modulus
$E_{iv}$ Initial vertical modulus
$E_j$ Joint modulus.
$E_{LM}$ Lower bound modulus.
$E_m$ Mass modulus.
$E_o$ Young's modulus at ground surface or foundation level
$E_s$ Secant modulus.
$E_{static}$ Static modulus
$E_i$ Initial tangent modulus
$E_{t50}$ Tangent modulus at 50% of the uniaxial compressive strength.
$E_y$ Post-yield modulus.
$E_{UM}$ Upper bound modulus.
$E_{0.001}$ Secant modulus at 0.001% axial strain
$E_{0.01}$ Secant modulus at 0.01% axial strain
$\varepsilon$ Normal strain
$\varepsilon_c$ Cavity strain
$\varepsilon_v$ Vertical strain
e Void ratio.
f Frequency
$f_f$ Fracture frequency
$f_i$ Empirical factor (Stroud, 1988)
g Acceleration due to gravity
G Shear modulus
$G_h$ Horizontal shear modulus
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\( \sigma_t \) Brazilian tensile strength.
\( \sigma_y \) Yield stress
\( \sigma_1 \) Major principal stress.
\( \sigma_2 \) Intermediate principal stress.
\( \sigma_3 \) Minor principal stress.
\( \sigma_1' \) Major principal effective stress.
\( \sigma_2' \) Intermediate principal effective stress.
\( \sigma_3' \) Minor principal effective stress.
\( \theta \) Phase difference
\( T \) Corrected phase difference
\( \tau \) Shear stress.
\( q \) Bearing pressure
\( q_{le} \) Yield bearing pressure (based on the onset of yield)
\( q_{net} \) Nett average bearing pressure
\( q_y \) Yield bearing pressure (based on the establishment of \( E_y \))
\( q_{ult} \) Ultimate bearing pressure
\( q_{ultu} \) Ultimate bearing pressure (undrained)
\( V_I \) Relative volume of intact material
\( V_j \) Relative volume of joint material
\( V_0 \) Initial volume
\( V_p \) Compressional wave velocity
\( V_r \) Rayleigh wave velocity
\( V_s \) Shear wave velocity
\( w \) Moisture content
\( w_l \) Liquid limit
\( w_{sat} \) Saturation moisture content
\( z \) Depth
5.0 DISCUSSION

Introduction

The review of literature (Chapter 2) has shown that the chalk displays a relatively uniform chemical composition together with a wide range of porosity. This variation in porosity is associated with depositional environment, diagenesis and tectonism.

Correlations exist between porosity and the intact mechanical properties of chalk such as strength and stiffness. Since the chemical composition of chalk is reasonably uniform the intact porosity of chalk may be expressed in terms of dry density, as this parameter is easier to measure.

For the major part of the area in which the chalk outcrops the dip is less than 10°. In such areas the discontinuity pattern is dominated by sub-horizontal and sub-vertical sets. Weathering processes such as stress relief and frost action have served to introduce new fractures to the rock mass. These secondary fractures are generally most abundant near the ground surface and are often more or less parallel to the ground surface and the major primary discontinuities. A typical weathering profile in the chalk is characterized by an increase in discontinuity spacing and reduction in aperture with depth.

Laboratory model studies and observations from in-situ loading tests reveal that the introduction of discontinuities in a rock mass increases the mass compressibility and the direction of the applied loading relative to that of the discontinuities has a significant affect on the load-deformation behaviour. Other factors found to be important in controlling rock mass compressibility include:-

(i) Discontinuity spacing
(ii) Discontinuity aperture
(iv) Discontinuity infill
(v) Intact stiffness
The contact area across discontinuities is thought to contribute to the compressibility of individual fractures. However no systematic study of the influence of contact area on the compressibility of individual discontinuities or of rock masses has been reported in the literature.

The general state-of-the-art of rock mass compressibility indicates that in chalk where sub-horizontal and sub-vertical discontinuities are dominant, the load-deformation curve should be concave (ie increasing stiffness with increasing load) and that in general mass stiffness should increase with depth as a result of the reduction in the effects of subaerial weathering processes. In reality, the results of limited load tests on the chalk are significantly different, as discussed below.

The mass compressibility behaviour of chalk has been observed in only six well documented case records for full scale foundations and large scale in-situ loading tests. These observations indicate the following aspects of behaviour:-

(i) The rock mass may display a significant component of time-dependant deformation. This is based on the results of only one long term test.

(ii) The load-deformation curve for the rock mass is typically convex. The initial part of the curve is more or less linear and most of the deformation is recoverable. The change in gradient of load-deformation curve, which gives rise to the characteristic convex shape, is thought to be associated with yielding. The mechanisms involved are not fully understood and this yielding behaviour has only been observed for one full-scale foundation.

Although the yielding behaviour has only been observed in the case of one full-scale foundation, it has been observed extensively in plate loading tests on chalk. Such observations from plate loading tests at Mundford, Norfolk
resulted in Burland and Lord (1970) proposing a simple bi-linear model for the load-settlement behaviour of chalk in the mass. This model permits the load-settlement behaviour to be described using 4 parameters:

\[ \begin{align*}
E_i & \quad \text{Initial modulus} \\
E_y & \quad \text{Post yield modulus} \\
q_e & \quad \text{Yield bearing pressure (based on the bearing pressure at the point at which the pressure-settlement curve starts to become steeper i.e. the onset of yield)} \\
q_y & \quad \text{Yield bearing pressure (based on the bearing pressure at zero settlement defined by extrapolation of the straight line representing the post yield portion of the pressure-settlement curve)}
\end{align*} \]

The plate loading tests in which this yielding behaviour has been observed have employed plate diameters of between 152mm and 910mm, founded in weathered chalk. It is known that the plate diameter influences the observed load-settlement behaviour (Lake & Simons, 1975, Hodges, 1976). Large scale loading tests and settlement measurements of full scale foundations provide the only conclusive data on the mass compressibility behaviour of chalk, and these are very limited in number. At the start of this research it was not clear whether this yielding behaviour occurs in all chalks, particularly since rock mechanics research on mass compressibility suggests that most chalk should display a concave load-settlement curve. However the Burland & Lord bi-linear model has found general acceptance in describing the mass compressibility behaviour of chalk.

Shallow foundations on chalk are generally designed on the basis of minimising settlements, since the rock mass is generally sufficiently strong that bearing capacity failure is of little concern. This approach to foundation design requires the stiffness of the rock mass to be estimated or measured. The fractured nature of the rock mass precludes the measurement of stiffness using intact specimens of chalk in the laboratory. In-situ tests provide the
only meaningful measurement of rock mass stiffness parameters for use in design. The most commonly used in-situ test in current practice for this purpose is the Standard Penetration Test. This test does not provide a direct measure of stiffness. However there are several empirical relationships between SPT 'N' value and initial modulus (Ei). Most popular amongst such relationships are those proposed by Wakeling (1970) and Kee and Clapham (1971), based on a very limited database which includes very few observations of full scale foundation behaviour. There are no published empirical relationships which permit the yield bearing pressure (qye, qy) or post yield modulus (Ey) to be predicted with any degree of confidence. Although the SPT has been found to be unreliable as a tool for predicting foundation settlements on weathered chalk, this appears not to have resulted in any reduction in its use for this purpose.

In many cases the stiffness of chalk in the mass is estimated from a visual examination of borehole samples or exposed faces (eg trial pit faces). The rock mass is generally classified according to the engineering grade classification developed for the chalk at Mundford, Norfolk (Ward et al., 1968). This rather complicated classification scheme is based on a visual examination of the rock mass in-situ at a site situated in a somewhat special stratigraphic and tectonic setting. It is therefore site specific. However it is used by engineers to describe chalk from any location within the chalk outcrop. At Mundford empirical relationships were established between visual grading of the rock mass and stiffness. But the database is not sufficiently comprehensive to permit the predict the stiffness parameters (Ei, Ey, qe and qy) with any degree of confidence outside of the Mundford area. In many cases the rock mass is graded on the basis of the visual examination of borehole samples obtained by cable percussion techniques and SPT 'N' values. The mechanical disturbance to the borehole samples together with unreliable nature of the SPT in chalk suggest that stiffness parameters derived in this manner should be treated with great caution.
Other methods currently available for the measurement of stiffness parameters for the chalk include the pressuremeter test, plate loading tests and seismic tests. The pressuremeter has not been used extensively in the chalk even though it has the potential to measure both pre-yield and post-yield stiffness parameters. The major problem with the use of the pressuremeter in chalk is providing a suitable test pocket. Menard Pressuremeters require a preformed test pocket. Drilling in chalk can cause considerable overbreak which would preclude the use of such pressuremeters. Self boring pressuremeters overcome this problem but suffer from the fact that they are unable to penetrate large flints. In addition to the problems of providing a suitable test pocket early pressuremeters were not sufficiently sensitive to provide accurate stiffness measurements in rock. The recent development of a rock pressuremeter overcomes this problem but is still considered by many engineers to be a research tool and hence it is currently not used extensively. The problems of the cost, inappropriate direction of loading, anisotropy and penetration of large flints may account for the lack of enthusiasm amongst engineers for using the pressuremeter in chalk.

Plate loading tests are very expensive tests to perform and hence their use is restricted to large projects. However this type of test not only provides the best similitude with the full-scale foundation but also permits the measurement of all four stiffness parameters associated with the Burland and Lord model ($E_p$, $E_y$, $q_e$, and $q_y$).

Seismic tests permit the initial modulus ($E_i$) to be determined at very low strains. However such measurements are rarely used in the prediction of foundation settlements on chalk. Traditionally geophysical methods have been employed to target features of geotechnical interest rather than to provide parameters for design. The limited success rate of geophysics in the targeting role in the past has bred scepticism amongst engineers. Hence it is not surprising that seismic techniques are not very popular for determining stiffness parameters for chalk. The reliability of seismic tests in predicting foundation settlements is uncertain since there are no published case records.
that compare predicted and observed settlements for large scale loading tests or full scale foundations.

In general the state-of-art concerning the mass compressibility of chalk was as follows, at the start of this research:

(i) The chalk was known to display a convex load-settlement curve. The change in gradient of the load-settlement curve which gives rise to the characteristic convex shape was thought to be associated with yielding.

(ii) The mechanisms associated with the yielding of the rock mass were not understood.

(iii) The chalk was known to display significant time-dependant settlements. The mechanisms controlling this are not understood.

(iv) The mass compressibility behaviour of chalk was described using a idealised bi-linear representation of the characteristic load-settlement curve. This permits the load-settlement relationship to be modelled using four stiffness parameters ($E_r$, $E_y$, $q_e$, and $q_y$).

(v) There were very few published records of the load-settlement behaviour from large scale loading tests or full scale foundations.
(vi) The stiffness parameter $E_i$ was (and remains) commonly determined from Standard Penetration Test results and visual examination of borehole samples and exposed faces. These methods make use of empirical relationships, most of which are based on the results of visual descriptions and in-situ tests carried out at Mundford, Norfolk.

(vii) The parameters $E_y$, $q_e$, and $q_y$ cannot be determined with confidence from the empirical relationships with SPT 'N' value or classification based on visual examination.

(viii) Other methods available for measuring the stiffness parameters were not used on a routine basis. This is either due to expense or the fact that the reliability of the results had not been fully evaluated.
5.1 Mechanical Properties of Intact Chalk

The review of literature (Chapter 2) has shown that correlations exist between mechanical properties of intact chalk, such as strength and stiffness, and porosity. In many cases the porosity has not been measured directly since such measurements are not carried out routinely. Moreover the accuracy of direct porosity measurements is related to the degree of interconnection between voids. In most of the cases reported in Chapter 2 the porosity was calculated from the dry density which is a more commonly measured parameter. The method by which the porosity is calculated is given in Chapter 4. This is based on the assumption that the specific gravity of the solid particles is constant ($G_s = 2.70$ which is the average value for calcite) which is not strictly true. However it provides a reasonable estimate of porosity of chalk where the calcium carbonate content is greater than 95%.

**Dry Density**

The range of dry densities measured at each of the three test sites are given in Table 4.3/1. These dry densities are compared with the range given by Clayton (1978) for each fossil zone in Fig. 5.1/1. It will be seen from Fig. 5.1/1 that all the test sites are within the Upper Chalk. The dry densities for site A are above average for the *Holaster planus* zone of the Turonian chalk. However, it is well known that the chalks of Lincolnshire and Yorkshire display a uniformly low porosity regardless of fossil zone, although the mechanisms which have brought this about are not understood. The densities measured at site A are not therefore unusually high since the site is located in Lincolnshire.

Site B is located within the *Micraster coranguinum* zone of the Senonian chalk. Fig. 5.1/1 indicates that the dry densities measured at this site were close to the average for this zone.

Site C is located within the *Actinocamax quadratus* zone of the Senonian chalk. The dry densities measured at this site were all around the minimum
values measured by Clayton (1978) (see Fig. 5.1/1). The chalks of Suffolk are characterized by high porosities and the chalk found at site C is in no way atypical. However when compared with chalk of similar age elsewhere it is unusual. Indeed these Suffolk chalks possibly display the highest porosities of any chalk found in the U.K..

The chalk at all three test sites displayed a relatively uniform dry density (see Fig. 4.3/1) when compared with the possible range of density shown in Fig. 5.1/1.

**Uniaxial Compressive Strength**

Whilst it may seem desirable to correlate porosity with uniaxial compressive strength and stiffness it is necessary to consider whether these parameters (measured in uniaxial compression) represent fundamental material properties, or whether they are test-method dependent. Uniaxial compression of cylindrical samples or drill core is probably the most widely performed test on rock. It is used to determine the uniaxial compressive strength (σc) and stiffness parameters Young’s modulus (E) and Poisson’s ratio (v). Despite its apparent simplicity, great care must be exercised in interpreting the results obtained from the test (Hawkes & Mellor, 1970). The results will depend upon the nature of the rock and on the condition of the test specimens, test instrumentation, platen friction and loading conditions. The conclusion that may be drawn from this is that the strength and stiffness measured in this manner are not fundamental material properties, and hence should be regarded as index properties. Clearly in order to make the comparison of strength and stiffness measurements for different specimens meaningful they must all be tested under the same conditions. Although recommendations have been made regarding standardizing the methods of specimen preparation and testing, it is almost impossible to cater for the almost infinite variety conditions under which rock is encountered.
For most rocks the uniaxial compressive strength is either measured directly from compression tests on cylindrical test specimens or indirectly from point load tests (Broch & Franklin, 1972). In the case of the weaker varieties of chalk the point load test may give unreliable results since these materials tend not to fail in a brittle manner. Hence the use of the point load test is restricted to the stronger varieties of chalk.

The suggested techniques for determining the uniaxial compressive strength of rock material are given by the International Society for Rock Mechanics Commission on Standardization of Laboratory and Field Tests (ISRM Commission, 1979) and the American Society for Testing and Materials (ASTM, D2938, 1979). Since there is no British Standard for uniaxial compressive strength tests, UK research and commercial rock testing laboratories adopt one of these suggested methods. The most common method used in the UK is that suggested by ISRM.

The height to diameter ratio of 2.5:1 recommended by ISRM (1979) is intended to minimise the effects of platen restraint which produces non-uniform stresses and strains within the test specimen. However in many cases specimens of rock which have been prepared in accordance with the ISRM recommendations display vertical fracturing, indicating failure in tension, probably due to end effects (see for example Kirkpatrick et al., 1974). The use of lubricated ends to minimise this effect is not recommended by ISRM, since it is considered to produce unreliable results.

It has been observed experimentally (Hawkes & Mellor, 1970) that for similar specimen geometry the uniaxial compressive strength of rock material varies with specimen volume. Generally, it is observed that $\sigma_c$ decreases with increasing specimen volume (except at very small specimen sizes). This, coupled with the requirement that the specimen diameter should be at least 10 times the size of the largest grain, provides the reason for the minimum specimen diameter of 54mm recommended by ISRM.
In many cases where rotary coring is employed to sample the chalk S or P size core barrels are used in order to improve recovery. Hence test specimens may be greater than the minimum 54mm recommended by ISRM (1979). However, in the UK chalk is frequently sampled using light cable percussion methods which may result in significant mechanical disturbance of the rock. This often results in test specimens with diameters less than 54mm being tested in order to achieve the appropriate H:D ratio. This is certainly the case for samples taken within the weathering profile regardless of the method of sampling, since the joint spacing is generally too close to achieve an H:D ratio of 2.5 (or even 2.0) with a 54mm diameter specimen. The typical grain size of chalk is $< 1.0 \mu m$. This means that test specimen diameters of less than 54mm may be used without affecting the measured strength provided the rock material is reasonably homogeneous.

The ISRM recommends that a loading rate of 0.5 to 1.0 MPa/s be used in uniaxial compression tests. This corresponds to a time to failure for most competent rocks of between 5 and 10 minutes and less than 1 minute for weak rocks. This is generally too fast for reliable data acquisition. These times correspond to average strain rates of 0.6% to 6% per minute. For rocks other than those which exhibit significant time-dependant deformation, departures from the prescribed strain rate by one or two orders of magnitude may produce little discernable effect. For example a change in strain rate from $6 \times 10^{-5} \%$ per minute to 60% per minute may only increase $\sigma_c$ by a factor of 2 (Brady & Brown, 1985).

In many cases the typical stiff compression testing apparatus for rock is too insensitive to give reliable results for the weaker varieties of chalk. For this reason, and the fact that most engineers in the UK tend to treat chalk as a soil rather than a rock, chalk is often tested under constant rate of deformation conditions. Indeed in many site investigation reports the strength of chalk is quoted as an undrained strength ($\sigma_c/2$) rather than uniaxial compressive strength. Despite this departure from the ISRM recommendations it would appear from discussion above that testing the rock under
constant rate of deformation conditions should not affect the measured uniaxial compressive strength significantly.

The sensitivity of uniaxial compressive strength and stiffness to moisture content changes has been observed in many different rock types (Colback & Wiid, 1965, Burshtein, 1969, Van Eeckhout & Peng, 1975, Priest & Selvakumar, 1982, Dyke & Dobereiner, 1991, Hawkins & McConnell, 1992). Priest & Selvakumar (1982) demonstrated that the sensitivity of different rock types to moisture is highly variable. Hawkins & McConnell showed that the sensitivity of strength and deformability to changes in moisture content for sandstones is highly variable. A number of mechanisms of strength reduction have been suggested including capillary tension decrease, pore pressure increase, reduction in friction and chemical and physical deterioration and these are described in detail by Van Eeckhout (1976). Clearly as a result of this sensitivity to changes in moisture content, rocks should be tested at their natural moisture content or fully saturated.

The uniaxial compressive strengths for chalk reported in the literature (Masson, 1973, Bell, 1977, Bonvallet, 1979, Woodland et al, 1988, Blight, 1990, Clayton & Saffari-Shooshtari, 1990, Kronieger, 1990, Mortimore & Fielding, 1990, Nienhuis & Price, 1990, Varley, 1990) were measured in a variety of ways. Bell (1977) reported uniaxial compressive strengths for a number of different chalks measured from 37.5mm diameter specimens with an H:D ratio of 2:1. Bell gives no indication as to whether the tests were conducted under stress control or strain control. Bonvallet (1979) performed uniaxial compressive strength tests under a constant rate of deformation of 0.05mm/min. However most authors do not give any indication of the specimen size or the way in which the test was carried out. There is in addition, uncertainty that the recommendations given in ISRM (1979) were followed unless stated specifically by the author.

The uniaxial compressive strength ($\sigma_c$) of the chalk at each test site is shown plotted against dry density and porosity in Fig.5.1/2. The data shown in Fig.
5.1/2 include that which has been reported in the literature. It will be seen from Fig. 5.1/2 that the uniaxial compressive strengths measured at the test sites are in good agreement with published values and follow a clearly defined trend of increasing strength with increasing dry density (or reducing porosity). The trend for dry chalk is more or less linear, whereas that for saturated chalk is curved. For saturated chalk with dry densities less than 1.5 Mg/m$^3$ there appears to be little change in strength with increasing dry density, which is the major cause of the curved trend line. This may result from the influence of effective stress since such chalks will be only weakly cemented. Indeed Safari-Shooshtari (1989) found high porosity chalk from Needham Market, Suffolk (Test site C) when subjected to triaxial tests to behave like soil in undrained compression. These trends however, are by no means conclusive since the database on which they are based is limited.

The sensitivity of strength to moisture content at the time of test is clearly shown in Fig. 5.1/3. For the chalks considered in Fig. 5.1/2 the ratio of $\sigma_c$ (dry) to $\sigma_c$ (saturated) is generally about 2:1. However for chalks of low strength (ie $\sigma_c$ (saturated) < 5 MPa) the dry strength can be as much as 4 times the saturated strength. This is mainly due to the saturated strengths remaining relatively constant for dry densities less than 1.5 Mg/m$^3$.

The data shown in Fig. 5.1/2 demonstrate that despite the differences in specimen size and loading conditions the influence of porosity and degree of saturation dominates the uniaxial compressive strength of chalk. The influence of specimen size and loading conditions may explain to some extent the scatter of strength data, although this is more likely to be associated with nonhomogeneity.

Clayton (1983) has reported a range of dry density for English chalk of 1.29 - 2.46 Mg/m$^3$. Based on this range and the trends established in Chapter 2 (Fig. 2.1/3) between strength and dry density, the range of uniaxial compressive strength is 1.5 - 40 MPa for dry chalk and 0.70 - 20 MPa for saturated chalk. The range of dry density for the three test sites is 1.34 - 1.93
Mg/m$^3$ which represents only 50% of the range reported by Clayton (1983). It will be seen from Chapter 4 and Fig. 5.1/2 that this range of dry densities gives rise to uniaxial compressive strengths between 3 MPa and 22 MPa for dry chalk and 0.72 MPa and 15 MPa for saturated chalk. These ranges represent 50% and 74% of the expected ranges for dry and saturated chalk respectively. The relatively high proportion of the expected range for saturated chalk represented by the three test sites reflects the insensitivity of uniaxial compressive strength to changes in dry density at low dry densities discussed earlier.

The ranges of dry densities measured at each site are shown in Fig. 5.1/1 which illustrates that although the chalks at each site are very different, they are not in any way unique or unusual. The high density chalk tested is typical of the chalk found in Lincolnshire and Yorkshire. The intermediate density chalk tested is representative of that commonly seen by engineers working in South East England. The low density chalk is typical of the chalk found over much of Suffolk. The chalks of very high dry density (> 2.0 Mg/m$^3$) are not represented in this research since they tend to be associated with hardgrounds. These hardgrounds may have considerable lateral extent but since they are generally relatively thin they only represent a very small proportion of the total area over which the chalk outcrops.

**Brazilian Tensile Strength**

The recommendations given by ISRM for the measurement of the Brazilian tensile strength (ISRM, 1978) are more easily adhered to than those given for the measurement of uniaxial compressive strength. Hence generally there is little or no deviation from the recommendations when measuring Brazilian tensile strength.

The Brazilian tensile strength ($\sigma_t$) of chalk at each test site are shown plotted against dry density and porosity in Fig.5.1/4. The data shown in Fig. 5.1/4 include that which has been reported in the literature. It will be seen from
Fig. 5.1/4 that the trends between tensile strength and dry density are clearly defined and similar in nature to those observed for uniaxial compressive strength in Fig. 5.1/2.

The range of Brazilian tensile strengths for the three test sites is 0.52 - 4.59 MPa for dry chalk and 0.16 - 1.52 MPa for saturated chalk. This represents 83% and 70% of the ranges (0.4 - 5.3 MPa for dry chalk, 0.2 - 2.1 MPa for saturated chalk) based on the overall range of dry density for English chalks given by Clayton (1978) and the trends shown in Fig. 5.1/4. It is surprising that although only 50% of the overall range of chalk dry density is represented by the three test sites such a high proportion of the expected range of Brazilian tensile strength was measured. However, the amount of published data was somewhat limited for the Brazilian tensile strength resulting in most of the data shown in Fig. 5.1/4 being clustered below a dry density of 1.6 Mg/m³. Hence the trend lines shown may be less reliable than those for uniaxial compressive strength. It will be seen from Fig. 5.1/4 that the Brazilian tensile strengths for dry chalk with the highest dry densities at test site A are all greater than the published values. The limited amount of published Brazilian tensile strength data for chalk with a dry density greater than 1.6 Mg/m³ make meaningful comparisons difficult.

The relationship between uniaxial compressive strength and Brazilian tensile strength for chalk which share the same dry density is shown in Fig. 5.1/5. It will be seen from Fig. 5.1/5 that the ratio of \( \sigma_c: \sigma_t \) is about 8:1 regardless of whether the chalk is dry or saturated at the time of testing.

Given that the uniaxial compressive strength and Brazilian tensile strength of intact chalk show a clearly defined relationship with dry density. It seems logical that there should a relationship between stiffness and dry density. However there are problems associated with the measurement of stiffness in weak porous rocks such as chalk which need to be considered before attempting to establish such relationships.
Stiffness in Uniaxial Compression

Stiffness measurements are often made in uniaxial compression. Indeed ISRM (1979) recommends the measurement of axial and radial strain during the measurement of uniaxial compressive strength. In hard rocks the tolerances for sample parallism and flatness recommended by ISRM (1979) provide a means of improving stress-strain measurements. When combined with the use of electrical resistance strain gauges (ERS gauges) careful sample preparation can lead to reliable stiffness measurements at strains as low as 0.001%. However the use of ERS gauges may not be feasible with some weak rocks.

Both sample grinding and the attachment of bonded ERS gauges can be impractical with some weak sandstones or closely fissured materials (Dobereiner, 1984). The attachment of bonded ERS gauges can be particularly problematic in the more porous varieties of chalk.

The principal problem is associated with bonding ERS gauges to a water saturated porous surface. The presence of water can result in an inadequate bond between the gauge and the rock unless the appropriate bonding agent is employed. Drying the rock before attaching the gauges is not advised since in weakly cemented rocks resaturation result in irrecoverable damage to the specimen (Maccarini, 1987 and Bressani, 1990). Local drying of the surface to which the gauge is to be bonded has been used successfully (Koshima et al, 1981). However this will increase the likelihood of the bonding agent penetrating the rock to such an extent that stiffness characteristics of the rock are modified at the points of strain measurement resulting in unreliable stiffness measurements. As a result of these difficulties it is not uncommon for deformation measurements to be based on platen to platen movement using a dial gauge or other types of linear displacement transducer. Weak rocks such as chalk are often tested in soil mechanics laboratories where internal load cells are used in triaxial testing and measurements of axial
displacement are made externally. In such cases test system compliances and bedding effects can completely mask the straining of the specimens.

Reliable strain measurements in weak rocks may be made locally to the specimen without the difficulties described above, using electrolevel transducers (Jardine et al., 1985) or Hall Effect local strain gauges (Clayton and Khatrush, 1986). These gauges are bonded to the specimen at discrete points above and below the gauge length and will resolve strains less than 0.001%. In the stiffness measurements made on intact chalk from the test sites described in Chapter 4 particular attention was given to need to measure strains locally without altering the stiffness within the gauge length and the measurement of small strains. The majority of chalks lie on the boundary between hard soil and weak rock. For most hard rocks the stress-strain behaviour is generally linear elastic almost to failure. However for soils the stress-strain behaviour is strongly non-linear even at very low strains (see Fig. 5.1/6). The results for dry chalk shown in Fig. 4.3/4 indicate, as might be expected, that the chalk fails at low strains (< 0.15%) and as the strength decreases (with increasing porosity) the stress strain behaviour becomes less linear.

The non-linear stress-strain behaviour of chalk in undrained triaxial compression was investigated in a limited way by Jardine et al. (1985). Their experiments showed the pre-peak behaviour of the chalk to be almost linear with a gradual drop in secant modulus from 6 GPa to 3 GPa over 0.07% strain.

In soil mechanics it has become traditional to emphasise the non-linear behaviour by plotting secant modulus ($E_{sec}$) against log axial strain, as was seen in Fig. 5.1/6. Fig. 4.3/5 shows the results of the tests shown in Fig. 4.3/4 plotted in this manner. It will be seen that in general the material behaves in a linear manner over most of the range but the high porosity chalk shows some deviations. This behaviour is completely different to that of soil. The degree of non-linearity may be examined from the ratio (L) of secant
modulus measured at 0.01% axial strain \( (E_{0.01}) \) to that measured at 0.001% axial strain \( (E_{0.001}) \). Fig. 5.1/7 shows the relationship between \( E_{0.01} \) and \( E_{0.001} \) for the chalk from the three test sites. It will be seen from Fig. 5.1/7 that although the stiffness ratio \( L \) varies between 0.5 and 2.0, much of the data lies close to the line of equality indicating the high degree of linearity displayed by the chalk. Typical values of \( L \) for hard soils are 0.3 to 0.4 which emphasises the differences in stress-strain behaviour between these materials and chalk.

Fig. 5.1/8 shows the relationship between the stiffness ratio \( L \) and dry density for the chalk from the three test sites. It will be seen from Fig. 5.1/8 that there is a general trend of increasing \( L \) with increasing dry density. \( L \) is generally less than 1 for chalks with low dry densities \( (<0.6 \text{ Mg/m}^3) \) indicating an initial high stiffness which reduces with increasing strain. This behaviour is similar to that of soil but much less pronounced. Some chalks of high density exhibit relatively low stiffness at small strains \( (ie < 0.001\%) \) which give rise to values of \( L \) greater than 1. Such behaviour is common in hard rocks and is thought to be associated with the closure of microfractures. For the chalks tested however, most values of \( L \) were less than 1.

In soil mechanics practice, for materials considered to be relatively unaffected by cementing, it is common to normalize \( E \) with respect to the mean effective stress \( (p'_m) \) immediately before shearing, but for materials which are considered to be cemented to a significant degree it is thought better to normalize with respect to the undrained strength \( (s_u) \). It can be seen in Fig. 5.1/9 that normalizing \( E \) with respect to \( \sigma_c \) for the chalk produces stiffness ratios between 1000 and 1500. For comparison the highly non-linear behaviour of London Clay is also plotted on this graph assuming that \( \sigma_c = 2s_u \) and it will be seen that normalized stiffness for this material is similar to that for the chalk for strains less than 0.01%.

Fig. 5.1/10 shows \( E \) for intact chalk determined in a variety of different ways. In the case of the local strain values the values plotted correspond to the
initial tangent modulus \( (E_t) \) as shown in Fig. 4.3/4. The \( E_{150} \) values correspond to a secant modulus at \( 0.5 \sigma_c \). It can be seen that although the data show some scatter there is nevertheless a strong trend of increasing stiffness with decreasing porosity. It will also be seen that the \( E_t \) values determined using the Hall Effect local strain gauges are in excess of those determined by other methods such as ERS gauges. As explained earlier the type of strain gauge and the method of test can have a significant effect on the measurement of stiffness. It is likely that stiffness determined without local strain measurement will underestimate the correct value. Fig. 5.1/11 shows that if the values of modulus determined by other methods are doubled a good correlation is obtained between \( E \) and porosity.

Fig. 5.1/12 shows the sensitivity of stiffness to moisture content at the time of test. It will be seen that stiffness is much less sensitive to moisture content than strength. The stiffness of dry chalk is about 1.2 times that of saturated chalk. Figs. 5.1/7 and 5.1/8 show that the stress-strain characteristics of the chalk are not influenced by the moisture content at the time of testing.

*Load-Deformation Behaviour in Uniaxial Strain*

It will be seen from the literature review in Chapter 2 that when chalk is compressed under conditions of uniaxial strain \( (K_u) \) it displays the following characteristics:

(i) At low stresses the cemented structure of the chalk deforms in a stiff manner and strains are generally recoverable upon unloading.

(ii) At a certain stress level cementation breaks down and the material undergoes yield. The yield stress \( \sigma_y \) has been shown to increase with reduction in porosity (Addis, 1987, Leddra, 1990).
If the stresses are increased beyond $\sigma_y$, the compressibility increases initially but reduces with increasing stress in the same manner as an uncemented soil. The amount by which the compressibility increases after yield has been shown to be related to the initial (pre-yield) porosity (Addis, 1987, Leddra, 1990).

Typical results of the uniaxial strain compression tests carried out on the chalk from the three test sites are shown in Fig. 4.3/11 as void ratio plotted against log vertical stress. It will be seen from Fig. 4.3/11 that the load-deformation behaviour of the chalk from test sites B and C displays the general features described above. The chalk from site A did not display any sudden increase in compressibility indicative of post yield behaviour even at vertical stresses of 36 MPa. However, Leddra demonstrates that for a chalk of similar porosity to that at site C the yield point is almost imperceptible from the plot of void ratio versus mean effective stress and can only be identified from the stress path plot (see Fig. 2.1/9). Since lateral stresses were not measured in the author's tests it was not possible to plot stress paths. It will be seen from Leddra's experiments (Fig. 2.1/9) that the yield stress for the chalk of similar porosity to that from site A (28%) yield occurs at a mean effective stress of 20 MPa. Since the typical stress ratio ($\sigma_{h}^{/}/\sigma_{v}^{/}$) is 0.3 up to yield the vertical effective stress at yield was 38 MPa. This is very close to the maximum vertical stress 36 MPa imposed on the chalk in the author's tests.

It will be seen from Fig. 4.3/11 that before yield there is very little change in void ratio with stress indicating a high stiffness. The point at which the chalk yields is seen clearly by the sudden drop in voids ratio for sites B and C. The high porosity chalk from site C shows the most significant drop in void ratio after the yield stress is exceeded. The chalk from site B shows a less significant drop in voids ratio after yielding which reflects the lower porosity of this material. The abruptness of yield is known to increase with the
porosity at which it occurs, and with increase in bond strength and yield stress (Leroueil and Vaughan, 1990). Both effects need to be referenced to the loosest possible void ratio-stress line for the uncemented (destructured) material. This relationship is shown schematically in Fig. 5.1/13. The cemented (ie structured) material can exist at void ratios greater than are possible for the destructured material at the same stress. It is thus convenient to define two void ratio-stress spaces (Fig. 5.1/13), the space bounded by the line that defines the loosest possible packing for the destructured material, and the space outside this line in which the material can only exist due to cement bonds between particles. Since the yield stress is reported to be related to initial porosity (Leddra, 1990) the abruptness of yield will be associated with relationship between the yield stress curve (ie the relationship between yield stress and void ratio) and the 'virgin' compression curve for destructured chalk. This is shown schematically in Fig. 5.1/14.

The above discussion suggests that a unique void ratio-stress relationship exists for white chalks after yield regardless of the initial porosity. The results of Leddra’s experiments indicate such a common relationship (see Fig. 2.1/9). It will be seen from Fig. 5.1/15 that the post-yield void ratio-log stress lines for the chalk from sites B and C tend towards a common line. The average gradient of this line is about 0.3. Coop (1990) showed that at high vertical stresses (ie > 1MPa) uncemented carbonate sand displayed a unique void ratio-log stress line with a gradient of 0.34 (see Fig. 5.1/16).

Based on the concept of a 'virgin' compression line for chalk in void-ratio-log stress space it is clear from Fig. 5.1/15 that the yield stress for the chalk at site A must be greater than 100 MPa. This is significantly greater than that deduced from Leddra’s experiments.
The results of the uniaxial strain compression tests are characterised using the following parameters:

(i) Constrained modulus $D_m \,(1/m,)$
This is used to describe the pre-yield behaviour since the load-deformation curve is more or less linear in this region.

(ii) Yield stress $\sigma_y$
This is taken as the point at which the voids ratio begins to drop significantly.

(iii) Compression index $C_c$
This is used to describe the post-yield behaviour. After destructuring the material behaves like an uncemented soil and hence the compressibility is stress dependent.

The values of these parameters derived for each test site are summarised in Table 5.1/1.

Table 5.1/1 Summary of uniaxial strain test results

<table>
<thead>
<tr>
<th>Site</th>
<th>Dry density $,(Mg/m^3),$</th>
<th>Constrained modulus $,(MPa),$</th>
<th>Yield stress $,(MPa),$</th>
<th>Compression index $,$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.89 - 1.93 (1.91)</td>
<td>3.23 - 4.00 (3.52)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>1.52 - 1.57 (1.54)</td>
<td>0.68 - 1.32 (1.02)</td>
<td>9.0 - 12.5 (11.2)</td>
<td>0.22 - 0.32 (0.28)</td>
</tr>
<tr>
<td>C</td>
<td>1.30 - 1.38 (1.34)</td>
<td>0.15 - 0.35 (0.26)</td>
<td>2.0 - 2.3 (2.2)</td>
<td>0.26 - 0.27 (0.26)</td>
</tr>
</tbody>
</table>

() = Average value
The constrained modulus values for the chalk is plotted against dry density in Fig. 5.1/17. It will be seen from Fig. 5.1/17 that the constrained modulus measured at the test sites follow a clearly defined trend of increasing modulus with increasing dry density (or reducing porosity). However the values are much lower than the values of Young’s modulus measured in uniaxial compression. If the constrained modulus values are multiplied by 7 they show good agreement with the other stiffness measurement (see Fig. 5.1/18). It is likely that the low values of constrained modulus result from bedding errors between the platens and the test specimen. Deformation measurements in these tests were made by measuring the displacement of the upper loading platen. Such external deformation measurements can lead to significant bedding errors, particularly in stiff materials such as rock (Tatsuoka and Shibuya, 1992).

The yield stress for the chalk from the test sites is plotted against dry density in Fig. 5.1/19. Included in Fig. 5.1/19 are yield stresses derived from results of $K_o$ tests on chalks of different porosities reported by Leddra (1990) assuming a $K_o$ of 0.3 up to yield. It will be seen from Fig. 5.1/19 that the yield stresses follow a clearly defined trend of increasing yield stress with increasing dry density. The yield stresses derived from Leddra’s tests show reasonably good agreement with the author’s data with exception of the high porosity chalk which has a much higher yield stress than that determined by the author for chalk of similar porosity. Yield stress is known to be strain rate dependent in weak rocks and the yield stress increases with increases in strain rate (Leroueil and Vaughan, 1990). Hence any lack of agreement may be attributed to the fact that the strain rates used in the author’s tests were different to those employed by Leddra (1990).

It is clear from Fig. 5.1/19 that the yield stress is very sensitive to small changes in dry density. This is largely due to the relationship the yield stress has with the ‘virgin compression curve’ for the chalk. This relationship is shown in Fig. 5.1/20. Since the chalk is structured the ‘virgin compression curve’ forms a lower bound for the yield stress. The positions of the yield
stresses in relation to this lower bound line at different void ratios suggest that the line representing the upper bound is non-linear and asymptotic to the 'virgin compression curve' at void ratios less than about 0.4. Clearly the yield stress for Leddra's low porosity chalk is too low since it is below the lower bound for yield stresses. This yield stress may be in error due to the assumption made about the typical pre-yield value of $K_o$ used in converting the mean stress to a vertical stress. The lower bound line shown in Fig. 5.1/19 suggests that the yield stress for the low porosity chalk from site A should be greater than 100 MPa.

Fig. 5.1/21 shows the relationship between compression index for destructured chalk and dry density. It will be seen from Fig. 5.1/21 that for the chalk from sites B and C which displayed yield and destructuring the compression index is close to 0.3. This agrees well with the values of $\lambda$ for carbonate sands (Coop, 1990).

**Summary**

- Since most of the chalk (with the exception of the Lower chalk) is monomineralic the porosity may be related to the dry density which is easier to measure.

- The dry density of the chalk at each test site was relatively uniform in relation to the range of dry density for the fossil zones represented.

- The overall range of dry density for the three test sites was from 1.30 to 1.93. This represents the typical range likely to be encountered in engineering works.

- The intact mechanical properties of the chalk are affected by the way in which the apparatus and the test specimen are configured.
• It has been shown that in white chalk dry density has a profound influence on strength, stiffness and yield.

• The degree to which the chalk is saturated with water has been shown to influence both strength and stiffness. It has been shown that:

\[
\frac{\sigma_{c\text{ dry}}}{\sigma_{c \text{ sat}}} = 2.0
\]

\[
\frac{E_{\text{dry}}}{E_{\text{sat}}} = 1.2
\]

• It has been shown that the ratio of \(\sigma_c : \sigma_t\) is 8:1

• It has been shown that local strain measurements using Hall Effect gauges or electrolevel gauges provide a more reliable means of measuring stiffness in chalk.

• Most chalk behaves in a linear elastic manner. However the linearity generally decreases with increasing porosity.

• Chalks of intermediate and high porosity (ie >35%) display yield associated with the breakdown of the pore structure when compressed under conditions of uniaxial strain.

• The stress at which yield occurs increases with increasing dry density.

• The post-yield load deformation behaviour is similar to that of uncemented soil, displaying a unique void ratio-log stress line typical of clay or sands at high stresses. The gradient of this line (compression index) is similar to that reported for uncemented carbonate sand. This line represents a lower bound for yield stresses.
Range of dry density and porosity found at sites A, B and C in relation to the overall range according to biozone.
Fig. 5.1/2 Variation of uniaxial compressive strength of chalk with dry density and porosity (data from chapter 2 combined with the results given in Chapter 4).
Fig. 5.1/3  Relationship between the uniaxial compressive strength of dry and saturated specimens of chalk.
Fig. 5.1/4 Variation of Brazilian tensile strength of chalk with dry density and porosity (data from chapter 2 combined with the results given in Chapter 4).
Fig. 5.1/5  Relationship between uniaxial compressive strength and Brazilian tensile strength of chalk.

\[ \sigma_c = 8 \sigma_t \]
Fig. 5.1/6  Stiffness of undisturbed London Clay during undrained shearing in triaxial compression.
Fig. 5.1/7 Relationship between secant modulus at 0.01% axial strain and that at 0.001% axial strain for specimens of chalk tested in uniaxial compression.
Fig. 5.1/8  Relationship between L ($E_{0.01}/E_{0.001}$) and dry density for specimens of chalk tested in uniaxial compression.
Fig. 5.1/9 Normalised stiffness of chalk ($E/\sigma_c$) and London clay ($E/2S_u$) during shear.
Fig. 5.1/10  Variation in stiffness of chalk with dry density and porosity (data from chapter 2 combined with the results given in Chapter 4).
Fig. 5.1/11 Variation in 'corrected' stiffness of chalk with dry density and porosity.
Fig. 5.1/12  Relationship between the stiffness of dry and saturated specimens of chalk.
Fig. 5.1/13 The comparison of structured and destructured compression in the oedometer test (after Leroueil and Vaughan, 1990)
Fig. 5.1/14  Yield locus for chalk in void ratio-effective stress space.
Fig. 5.1/15 Relationship between void ratio and vertical stress for specimens of chalk with different porosities.
Fig. 5.1/16  (a) Isotropic and (b) one-dimensional compression data for a carbonate sand (after Coop, 1990)
Fig. 5.1/17 Variation in constrained modulus ($1/m_\nu$) of chalk with dry density and porosity.
Fig. 5.1/18  Comparison between the constrained modulus of chalk and Young's modulus measured in uniaxial compression.
Fig. 5.1/19  Relationship between yield stress and dry density for specimens of chalk.
Fig. 5.1/20  Relationship between initial void ratio and yield stress for chalk.
Fig. 5.1/21
Variation of compression index $C_c$ of chalk with dry density and porosity.
5.2 Rock Mass Description

A visual description of the rock mass is fundamental to any assessment of engineering performance. Such a description would include the rock material as well as the discontinuities which dominate most rock masses. Emphasis may be placed on certain features of the rock mass known to be important in influencing its behaviour with respect to a particular engineering application such as underground excavations or foundation engineering. The importance of visual inspection of the chalk is now widely recognised. The factors which were identified in Chapter 2 as being important to the prediction of foundation settlements in chalk are shown in Fig. 5.2/1. It will be seen from Fig. 5.2/1 that the principal factor controlling mass compressibility is whether the chalk is structured or not. The term unstructured chalk is used to describe highly weathered chalk comprising lumps of intact chalk in a matrix of putty chalk or clay. This material is essentially an engineering soil and hence its performance as a foundation material may be predicted using soil mechanics principles. In the case of structured chalk, in which the rock is divided up into discrete blocks by discontinuities, the prediction of foundation settlements can be more complicated.

The compressibility of structured chalk will be controlled primarily by the structure of the discontinuities. Such features as the orientation of discontinuities with respect to the direction of applied load, and the spacing of discontinuities in critical orientations, are known to have a significant influence on load-settlement behaviour and this has been discussed in Chapter 2.

The orientation of discontinuities may be measured using a geological compass. If the discontinuities display a complex geometry it may be necessary to carry out a statistical analysis of orientations using lower hemispherical projections as described by Hoek and Bray (1981) or Priest (1993) in order to identify discontinuity sets. However it has been shown in the literature review that in most cases the chalk displays a relatively simple
discontinuity system. This typically comprises sub-horizontal discontinuities primarily associated with bedding and two sets sub-vertical joints. Superimposed upon this system is another which associated with mechanical weathering processes. These display similar orientations to the primary sets and reduce in abundance with depth below the natural ground level. For shallow foundations settlements will be dominated by the closure of the sub-horizontal discontinuities. Hence attention tends to focus on these in describing the rock mass. The sub-vertical joints should not be ignored however, since these play an important role in controlling the typical block size and shape and the looseness of the fracture block system. It has been shown in the literature review that these joints can have a significant influence upon the stress distribution beneath a foundation.

The literature review demonstrated the importance of sub-horizontal discontinuity spacing in controlling the mass compressibility of chalk. In general a chalk mass with closely spaced sub-horizontal discontinuities will be much more compressible than the same chalk with widely spaced sub-horizontal discontinuities. However it is not the spacing of these fractures alone which is significant but the number of fractures which occur within the zone of influence of the foundation. Hence if the spacing is greater than the dimensions of the foundation it is likely that the fractures will have little effect on the compressibility of the rock mass and the intact stiffness of the rock may be used to predict settlements. An example of a such a rock mass is given by Nienhuis and Price (1990). If, as is frequently the case in weathered near-surface chalk, the spacing is much less than the dimensions of the foundation the rock mass stiffness will be a small fraction of the intact stiffness.

Fig. 5.2/2 shows the relationship between sub-horizontal discontinuity spacing and depth for the three test sites. At sites B and C the average spacing of primary discontinuities associated with bedding was between 260mm and 300mm. Bedding discontinuities could not be distinguished from secondary fracturing associated with weathering at site A. It will be seen from Fig. 5.2/2
that generally within 4m of the ground surface the spacing is much less than
that displayed by the primary bedding features. This is due to the formation
of new fractures by frost weathering during the Pleistocene and general stress
relief. The range of sub-horizontal fracture spacings which occurred within a
depth of one plate diameter (1800mm) was between 10 and 160mm. Typically
there would be between 30 and 46 sub-horizontal discontinuities within one
plate diameter of the surface at any of the three sites. Of these up to 8 would
be primary bedding discontinuities.

All the profiles shown in Fig. 5.2/2 show an increase in spacing with depth,
although the trends are often somewhat complicated by the way in which the
spacing has been described.

Discontinuities are rarely regularly spaced and hence spacing is often
described by maximum, minimum and average values for a zone displaying
relatively uniform rock mass characteristics. The ranges used in the
description of spacing are not necessarily those recommended by
on Rock Mass Description. The ranges used are often biased by those
employed in the Mundford Grading system (see discussion in section 5.4). It
is relatively rare to find actual measurement of discontinuity spacings.
Frequently the spacings are simply assessed by eye rather than by direct
measurement.

The rock mass is normally divided into zones during the logging processes
rather than on the basis of all the measurements and observations. There is
no guarantee that the range of spacings given for a zone will be wholly
representative. However it is common practice to zone the rock mass in the
field largely on the basis of discontinuity spacing when attempting to assess
compressibility. A more objective approach to rock mass description would be
to use line surveys. This would ensure that every discontinuity which is
reasonably persistent would be recorded in some way. Such an approach,
although time consuming in the field, would at least provide sufficient data to
enable the preliminary field zonation to be checked and modified if necessary. It would also permit fracture frequency to be plotted against depth which would aid the identification of different weathering styles.

A feature of rock mass structure found by this research to be important is that of the looseness of the fracture block system. The fracture block system refers to the fracture bounded blocks that make up the rock mass. At Test Site A (North Ormsby) the fracture block system was found to be so loose that once the surface of the rock had been broken it could be excavated easily by hand, simply by lifting out the blocks from the rock mass. This chalk had the highest intact stiffness of all three test sites and was expected to display a low mass compressibility. Some of the lowest values of initial modulus ($E_i$) for all the test sites were measured at this site. By contrast some of the highest values were measured at Test Site C which, although characterized by chalk of low intact stiffness, displayed a relatively tight fracture-block system which precluded excavation without the use of tools. This highlights the importance of the relative looseness of the fracture block system and demonstrates the small influence the intact stiffness has on mass compressibility when the discontinuity spacing is small relative to the size of the foundation.

Table 5.2/1 outlines how the looseness of the fracture block system may be assessed in the field.
### Table 5.2/1 Assessment of looseness of the fracture-block system

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tight</td>
<td>Blocks bounded by natural fractures cannot be removed from an exposed vertical face or from the base of an excavation by hand without breaking the block.</td>
</tr>
<tr>
<td>Intermediate</td>
<td>Blocks bounded by natural fractures can be removed from an exposed vertical face or from the base of an excavation by working them loose by hand without breakage.</td>
</tr>
<tr>
<td>Loose</td>
<td>Blocks bounded by natural fractures can be removed from an exposed vertical face or from the base of an excavation easily by hand without breakage.</td>
</tr>
</tbody>
</table>

The structure of the rock mass is shown in Fig. 5.1/1 to be the most important factor controlling the mass compressibility of structured chalk. Next in this hierarchy of importance comes the stiffness characteristics of the individual discontinuities. The factor which is likely to have the greatest influence on the stiffness behaviour of discontinuities is the area of contact across adjacent discontinuity walls. This is of particular importance in the chalk since it is prone to solution weathering which results in a reduction in contact area.

If the contact area is small then high stresses may be generated at the asperities of contact at relatively low foundation bearing pressures. Such high contact stresses are likely to result in local yielding or collapse of the asperities. It is this mechanism which is likely to provide a significant contribution to rock mass compressibility, particularly in weak rocks such as chalk. The behaviour of the asperities of contact will depend to a large extent on the intact strength and stiffness of the rock forming the discontinuity walls, hence the inclusion of these in Fig. 5.1/1.

Although contact area is perhaps the most important factor controlling discontinuity stiffness it is impossible to measure directly in a visual assessment of the rock mass. However contact area may be inferred from the measurement of aperture which is discussed below. To date there has been
no systematic study of contact area and the load-deformation characteristics of discontinuities in chalk. This means that even if contact area could be measured it would be of limited use in predicting rock mass compressibility.

The presence of infill within a discontinuity will have the most significant effect the stiffness when it completely fills the void space between the discontinuity walls. The nature of the infill material will control the discontinuity stiffness in such cases. Completely infilled discontinuities are most likely to be found in weathered chalk close to the ground surface where soil has infiltrated discontinuities opened by stress relief. Only partially infilled discontinuities were observed at the test sites. This infill was generally coarse sand and fine gravel size rounded fragments of chalk indicative of transport by water within the discontinuities. This infill comprised only a small percentage (<10%) of the total void space. Hence it was considered to have no influence on the discontinuity stiffness.

Aperture is simply defined as the perpendicular distance between adjacent walls of a discontinuity. The measurement of aperture can provide the following information relating to mass compressibility:

(i) The maximum closure that can occur across a discontinuity. It is unlikely that an aperture would close up completely even under very high bearing pressures so maximum aperture measurements would not necessarily provide a meaningful measure of discontinuity closure.

(ii) The degree of weathering. Stress relief and frost weathering near the ground surface will tend to open up discontinuities and loosen the fracture block system mentioned earlier.
An indirect indication of the degree of contact across discontinuities
It is reasonable to assume that large apertures are associated with relatively small contact areas.

In most of the chalk in the UK foundation settlement will be controlled to a large extent by the closure of sub-horizontal discontinuities. In most cases there will be a certain amount of contact across these. Hence the discontinuity apertures observed in an exposed face will not be continuous across the extent of the discontinuities. The sub-horizontal discontinuities observed in the faces of trial pits at Test Sites B and C exhibited wavy discontinuity walls resulting from solution weathering. These discontinuities displayed apertures which ranged from 0 to more than 10mm over distances of between 200 to 300mm. This variation makes the measurement of aperture complicated. Recording the maximum aperture in such a case would not provide any meaningful information since it implies a low contact area. In reality the contact area is likely to be much higher since the contact zone is often of a similar extent to the void space in one dimension. Hence where these features are observed it is necessary to record the average maximum aperture and the average extent of the void space along the exposed part of the discontinuity. Other sub-horizontal discontinuities observed at the test sites tended to have apertures less than 3mm apart from those close to (within 1m) of the ground surface.

In most cases apertures are only measured on sub-horizontal discontinuities in chalk and the sub-vertical discontinuities are often overlooked. However, the apertures associated with sub-vertical discontinuities can be equally important. These are more likely to display little or no contact over distances greater than 1m if open more than about 4 to 5mm. If the rock mass is dominated by sub-vertical discontinuities such apertures will cause the rock mass beneath a foundation to act as a series of discrete column. The load-settlement behaviour of the foundation will be controlled to some extent by
yielding and failure of these columns in uniaxial compression until contact is re-established across these discontinuities.

The intact mechanical properties of the chalk appear to be of little importance near the ground surface where the discontinuities are closely spaced relative to the dimensions of the foundation. In cases where the discontinuities are widely spaced (ie sub-horizontal fracture spacing > 2 * foundation width) the intact stiffness would control the rock mass compressibility. The intact strength and stiffness play an important role in controlling the deformation of asperities of contact which as discussed earlier influence the stiffness of discontinuities.

The intact mechanical properties of the chalk have been found to play an important role in controlling the rock mass structure in weathered chalk by influencing the style of weathering. A distinct difference in the weathering profiles was observed between Test Sites A and B and Test Site C. The soft chalk at Test Site C is characterised by high porosity (n > 45%) and poorly cemented particles. When the intact rock freezes the associated volume change causes a breakdown of the weak bonds between the particles. Small tight randomly orientated hair-line cracks were observed in this soft chalk. These cracks remain tight and only become noticeable in hand specimen since the rock tends to break readily into medium gravel sized fragments along them. It is believed that these cracks are associated with frost action since they are generally not found at depth. Since the rock material does not behave in a brittle manner freeze thaw action in existing discontinuities tends not to promote the growth of new fractures except when close to an exposed face. The lack of brittleness also results in fewer discontinuities formed as a result of stress relief.

The harder chalks at Test Sites A and B are characterised by the development of flaggy chalk close to the ground surface (usually within 2m) which was absent in the soft chalk of Test Site C. The way in which soft chalk weathers results in smaller apertures, higher degree of contact and
higher degree of interlocking of joint bounded blocks than is commonly found in harder chalks. The consequence of this is a high initial modulus $E_i$. However once the rock mass yields both the hard and soft chalks exhibit similar stiffness characteristics.

Based on observations made at the three test sites it would appear that the intact properties of the chalk exert some influence over the style of mechanical weathering of the rock mass. Although this hypothesis does not seem unreasonable it is based on limited evidence. To date there has been no systematic study of weathering styles in chalk.

The intact strength and stiffness of chalk may be assessed relatively easily in the field using the simple scale of hardness described in Chapter 3 (Table 3.1/2). Based on the author's experience the hardness scale can be related to dry density as shown below.

<table>
<thead>
<tr>
<th>Average dry density (Mg/m$^3$)</th>
<th>Hardness scale (Table 3.1/3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.35</td>
<td>3</td>
</tr>
<tr>
<td>1.54</td>
<td>2</td>
</tr>
<tr>
<td>1.85</td>
<td>1</td>
</tr>
</tbody>
</table>

Having converted the hardness to a dry density the strength and stiffness may be estimated from the trends established in Figs. 5.1/2 & 5.1/11.

It is clear from the above discussion that many of the factors listed in Fig. 5.2/1 cannot be adequately assessed or measured in the field. This is particularly true of principal factors influencing discontinuity stiffness. Fig. 5.2/3 outlines those factors which are considered to be important either directly or indirectly to rock mass compressibility which can be readily
assessed in the field. The most important features of a rock mass to describe for the purposes of assessing mass compressibility are:

(i) Discontinuity orientation
In most chalk outside areas affected by intense tectonic activity (see Chapter 2) the orientations will be sub-horizontal and sub-vertical.

(ii) Spacing
For foundations the spacing of the sub-horizontal discontinuities will be of paramount importance. If possible the fracture frequency of these discontinuities should be established to a depth of at least 2 times the breadth of the proposed foundation.

(iii) Looseness of the Fracture-Block System
It will be shown in later discussion that this factor has a strong influence on mass compressibility. It is the interaction of the intact mechanical properties and the weathering processes which gives rise to different weathering styles which are identified primarily on the basis of this factor and fracture spacing. It is likely that the different weathering styles result in different degrees of contact across fractures. Looseness of the fracture block system may be assessed qualitatively using the definitions given in Table 5.2/1.

(iv) Hardness and dry density
The hardness of the intact rock may be assessed by handling the rock using Table 3.1/2. The dry density may be estimated using the relationship given in the above discussion. It has been demonstrated in section 5.1 that the strength and stiffness of the rock material is related to dry density.

Very little can be deduced about discontinuity stiffness from infill and aperture. Infill is only really important when it totally fills the void space. In many cases it only partially fills the void space and hence the asperities of
contact will still dominate the deformation process. It may be misleading to simply state that infill has been observed unless the extent to which the void space is filled is made clear. Aperture, as mentioned earlier can give an indirect indication of contact area provided the maximum aperture and the lateral extent of the void space is described.
Fig. 5.2/1 Principal factors influencing chalk mass compressibility.
Test site A
Sub-horizontal discontinuity spacing (mm)

Test site B
Sub-horizontal discontinuity spacing (mm)

Test site C
Sub-horizontal discontinuity spacing (mm)

Fig. 5.2/2 Variation of horizontal discontinuity spacing with depth observed at test sites A, B and C.
Fig. 5.2/3 Principal factors influencing the mass compressibility of chalk which can be assessed by visual inspection of the rock mass.
5.3 Load-Settlement Behaviour of Chalk

The available literature suggests that, for a rock mass with joints parallel and normal to the direction of loading

- the details of contact geometry, and the yield strength of joint walls normal to the direction of loading, will dominate the stiffness of the rock mass

- stiffness will increase with increasing load.

Stiffness can be expected to decrease with increasing frequency of the joint set normal to the direction of loading, but this may well be a secondary effect if there is significant spatial variation of contact area and aperture of the joints.

In-situ loading tests on limestone and sandstone (Zienkiewicz and Stagg, 1965 and Hobbs, 1973) with discontinuities parallel and perpendicular to the direction of loading gave nearly linear or strongly concave load-settlement curves (Fig. 2.2/8). These results are indicative of normal closure of the discontinuities perpendicular to the loading direction in accordance with the findings of Bandis et al. (1983).

Tests on cylindrical specimens of intact chalk, using local strain measurement have shown that in uniaxial compression the stiffness of chalk is significantly reduced by the introduction of smooth discontinuities perpendicular to the major principal stress direction (Matthews and Clayton, 1992). The stress strain curves are typically concave and confirm the findings of Bandis et al (1983). The initial stiffness is controlled mainly by the number of horizontal discontinuities present. A simple model for rock mass compressibility can be derived from this, taking account of the stiffness of the rock material, the normal stiffness of the discontinuities and the aperture. This forms the basis
of the rock mass factor for chalk proposed by Hobbs (1975) shown in Fig. 2.2/15.

The introduction of slightly roughened horizontal discontinuities with contact area ratios ([average contact area/specimen cross sectional area] *100%) of more than 90% appears to have very little effect on the stiffness behaviour. However if the contact area ratio is reduced to 30% a yield point is observed (Matthews and Clayton, 1992) and the overall behaviour is similar to that of intact chalk compressed under uniaxial strain conditions (Fig. 4.3/10).

The yield behaviour observed in chalk specimens with horizontal discontinuities of limited contact area ratio is associated with local yielding, and eventual pore structure collapse, of the chalk in the area of contact. The available literature shows that yielding is observed often in plate loading tests but it has only been seen in one case concerning a full-scale foundation (see Chapter 2). The large-diameter plate loading tests carried out at the three test sites all displayed convex shaped load-settlement curves (see Figs. 4.1/24-26) confirming that yield can occur over a wide range of intact porosity. The load-settlement data from the in-situ loading tests when compared with that from the laboratory tests on idealised discontinuities suggests that there is only a very limited amount of contact (<50%) across the sub-horizontal discontinuities within the rock mass. The rock mass observed in trial pits at test sites B and C showed clear evidence of solution weathering of primary bedding discontinuities (see Chapter 3) and such a mechanism would account for a significant reduction in contact area. It is likely that solution weathering has also affected the secondary discontinuities (ie stress relief and frost weathering fractures) although to a much lesser extent than the primary bedding and tectonic discontinuities. It should be pointed out however, that the laboratory tests on artificial discontinuities in chalk discussed above were limited in nature and hence they only provide a clue to the mechanisms causing yield in a fractured rock mass.
The load-settlement behaviour observed in the plate loading tests carried out at the three test sites confirms the bi-linear model proposed by Burland & Lord (1970). Hence the load-settlement behaviour may be described using the parameters \( E_i \), \( E_y \), \( q_e \) and \( q_y \). A summary of the parameters measured at the test sites is shown in Table 5.3/1.

**Table 5.3/1 Summary of rock mass compressibility parameters derived from plate loading tests at the three test sites.**

<table>
<thead>
<tr>
<th>Site</th>
<th>Average dry density (Mg/m³)</th>
<th>( E_i ) (MPa)</th>
<th>( E_y ) (MPa)</th>
<th>( q_e ) (kPa)</th>
<th>( q_y ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.85</td>
<td>347 - 579</td>
<td>54 - 75</td>
<td>200 - 250</td>
<td>295 - 325</td>
</tr>
<tr>
<td></td>
<td>(424)</td>
<td>(62)</td>
<td></td>
<td>(217)</td>
<td>(318)</td>
</tr>
<tr>
<td>B</td>
<td>1.55</td>
<td>463 - 627</td>
<td>63 - 70</td>
<td>200 - 300</td>
<td>363 - 447</td>
</tr>
<tr>
<td></td>
<td>(564)</td>
<td>(66)</td>
<td></td>
<td>(233)</td>
<td>(391)</td>
</tr>
<tr>
<td>C</td>
<td>1.35</td>
<td>361 - 1009</td>
<td>45 - 73</td>
<td>200 - 400</td>
<td>375 - 675</td>
</tr>
<tr>
<td></td>
<td>(687)</td>
<td>(58)</td>
<td></td>
<td>(300)</td>
<td>(550)</td>
</tr>
</tbody>
</table>

() = Average

**Initial Modulus \( E_i \)**

The initial modulus \( E_i \) is normally taken as a tangent modulus and is based on the assumption that the load-settlement curve is linear up to the yield point \( q_e \). However, difficulty was experienced using this method for determining \( E_i \), since the load-settlement curves where not always linear up to \( q_e \). In most cases this appears to be associated with errors in the precise levelling since small errors become more pronounced during the early stages of a plate loading test when the settlements are small (generally < 1mm). In some cases however, this appears to be due to the compressibility characteristics of the rock mass (eg at test site C).
It was found that $E_i$ could be determined with more confidence by plotting the secant modulus ($E_s$) against bearing pressure (see Figs. 4.1/29 to 31). Two types of curve are seen and idealisations of these are shown in Fig. 5.3/1. The Type I curve shown in Fig. 5.3/1a was generally observed in the more brittle chalks of intermediate and high dry density (ie test sites A and B). The linear relationship between points A and B is indicative of a linear load-settlement curve. The change in gradient at point B corresponds with the change in gradient of the load-settlement curve and may therefore may be taken to represent the yield bearing pressure $q_e$. It will be seen from Figs. 4.1/29-31 that the portion of the curves below $q_e$ is rarely linear. The curves are generally irregular, highlighting the errors made during levelling. However $E_i$ may be calculated by averaging the values of secant modulus up to the yield bearing pressure $q_e$ is clearly identified on these plots.

The Type II curve shown in Fig. 5.3/1b displays two concave potions. The cusp a point B corresponds with the more sudden change in gradient of the load-settlement curve and hence has been taken to represent the yield bearing pressure $q_e$. However this may be a misnomer in this case since the shape of the curve AB suggests that yielding is occurring from the start of the plate loading test.

The Type II curve was observed mainly at test site C which is characterised by chalk of low dry density (1.35 Mg/m$^3$) and a relatively tight fracture block system. It was also observed in a single test (test 3, Fig. 4.1/29) at site A which is characterised by a high dry density and a loose fracture block system. The Type II curve is considered to be an unusual result for site A. The results of the surface-wave seismic tests at this site indicate a change in rock mass characteristics for the location of test 3. It will be seen from Fig. 4.2/16 that the shear modulus close to the ground surface for test location 3 is much greater than that for test location 2. This suggests that the fracture block system may be tighter at this location. It will be seen from Fig. 4.1/29 that for test 3 the secant modulus at 50 kPa bearing pressures was less than 1 GPa which is much less than the intact modulus for this chalk (see Table 516.
4.3/3) indicating the dominance of the discontinuities in the deformation process.

It will be seen from Fig. 4.1/31 that for test 1, the secant modulus measured at a bearing pressure of 50 kPa was 2.7 GPa. This value is close to the intact modulus for this chalk (see Table 4.3/4). The loading increment was increased in test 2 such that the secant modulus for a bearing pressure of 50 kPa cannot be calculated. However the shape of the curve for test 2 shown in Fig. 4.1/31 indicates that the secant modulus at 50 kPa bearing pressure may be similar to that measured in test 1. These high initial secant moduli reflect the tightness of the secondary fractures and the widely spaced bedding plane discontinuities which display the greatest apertures. Test 3 at site C displayed a Type I curve. The results of this test are considered to be unusual and are discussed later.

The above evidence suggests that the type II curve may be related directly to the contact area across discontinuities.

It is clear from Fig. 5.3/1b that in the case of a type II curve $E_i$ is stress dependent making its determination difficult. The value of $E_i$ given in Table 4.1/1 for the cases where a type II curve was observed were calculated from averaging $E_s$ up to bearing pressure corresponding to $q_e$.

In order to compare the load-settlement data derived from these tests with that found in the literature for full scale foundations and large scale in-situ loading tests it is necessary to normalise the settlements with respect to the plate diameter in the manner described in Chapter 2. The load-settlement behaviour observed at the three test sites together with the data from the case records considered in Chapter 2 for weathered chalks with similar rock mass characteristics (ie discontinuity spacing between 10 and 200mm and apertures between 0 and 20mm) is shown in Fig. 5.3/2 up to a settlement ratio of 0.1%.
The pre-yield load-settlement behaviour may be seen in Fig. 5.3/2. The typical range of initial modulus \( (E_i) \) values shown by the envelope in Fig. 5.3/2 is from 300 MPa to 1000 MPa. It will be seen from Fig. 5.3/2 that most of the plate loading test data from the test sites fall within this range.

The test sites and the case histories considered all share similar rock mass characteristics in terms of discontinuity spacing and aperture. Furthermore although the range of spacings appears large (10 - 200mm), it is small relative to the size of the loaded areas considered. It would seem reasonable to assume that since the intact stiffness increases with dry density, the initial modulus \( E_i \) should display a similar relationship with dry density. However it will be seen from Table 5.3/1 that \( E_i \) decreases with increasing dry density. This trend is also seen in Fig. 5.3/3 which shows the same data as in Fig. 5.3/2 but classified according to dry density. This trend is associated with variations in the looseness of the fracture block system (see section 5.2).

The low porosity (high dry density) chalk at test site A exhibited a very loose fracture block system near the ground surface where the plate tests were carried out, such that the rock could easily be excavated by hand (ie without the use of hand tools). The initial closure of open sub-horizontal discontinuities (both primary and secondary) result in low values of initial modulus despite the high stiffness of the intact rock. Clearly in such cases the intact stiffness is relatively insignificant in controlling the rock mass compressibility.

The high porosity (low dry density) chalk at test site C does not display the same degree of brittleness as the low porosity varieties and hence secondary fracturing is associated mainly with frost weathering. Those fractures observed in trial pits at test site C tended to be irregular, in persistant and generally tight. Hence at this site the primary bedding discontinuities provided the major contribution to the compressibility of the rock mass. As a result of the bedding discontinuities being widely spaced (300mm average) the values of initial modulus are very high values of \( E_i \). In this case the intact
stiffness plays a much more significant role in controlling the mass compressibility.

The values of $E_i$ measured at test site C at first site appear to cover a surprisingly wide range (361 to 1009 MPa). However a low value of $E_i$ (361 MPa) was only measured for one test (Test Location 3, see Chapter 4). It should be noted that this was the only test location where general crushing of the rock mass was observed beneath the plate (see Plate 4.1/6). Although no unusual features were observed when preparing the surface for the plate loading test the fact that the rock mass was crushed during the test suggests that the rock mass at this location was in some way different to that at the other locations. The most likely explanation is that a relatively large solution void (>20mm max. aperture and with an extent approaching that of the plate diameter) may have been present within a bedding discontinuity close to the underside of the plate. The progressive collapse of this void would result in a low initial modulus and crushing of the overlying chalk. This hypothesis is supported to some extent by the relatively low yield bearing pressure ($q_e$ and $q_y$) measured at this location. However it should be pointed out that these yield stresses were not unusually low but simply the lowest measured at this site.

The reduction in stiffness of the rock mass as a result of fracturing may be seen from the ratio of $E_i$ to $E_r$. This ratio known as the rock mass factor (Hobbs 1975). The rock mass factors for the sites investigated are given in Table 5.3/2. It will be seen from Table 5.3/2 that the intact stiffness is reduced by about 40 times at test site A because of the loose fracture block system, whereas it is only reduced by a factor of 5 at test site C because the fracture block system is relatively tight. This clearly highlights the relative importance of the intact stiffness in predicting the mass compressibility of chalk where the rock mass characteristics are known. The rock mass factors given in table 5.3/2 are plotted against fracture spacing in Fig. 5.3/4. It will be seen that the data from the test site show good agreement with Hobbs (1975).
Table 5.3/2  Rock mass factors for the three test sites

<table>
<thead>
<tr>
<th>Site</th>
<th>Dry density (Mg/m³)</th>
<th>Rock mass factor E/E₁</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.85</td>
<td>0.027</td>
</tr>
<tr>
<td>B</td>
<td>1.55</td>
<td>0.093</td>
</tr>
<tr>
<td>C</td>
<td>1.35</td>
<td>0.202</td>
</tr>
</tbody>
</table>

**Post-yield Modulus Eₚ**

Fig. 5.3/5 shows the load-settlement behaviour up to a settlement ratio of 0.4% and hence includes the post-yield behaviour. It will be noted that the data for both foundations and in-situ loading tests shown in Fig. 5.3/5 fall into a relatively narrow band, with similar post-yield modulus Eₚ values (typical range 44 to 68 MPa) which are generally an order of magnitude less than the initial modulus Eᵢ. However it will be seen from Fig. 5.3/5 that the majority of the post-yield settlement data comes from the large diameter plate loading tests carried out by the author. This highlights the lack of published post-yield data from full scale foundations and large scale in-situ loading tests.

It will be seen from Fig. 5.3/5 that the only published case record in which settlement ratios greater than 0.2% have been recorded is for the Sugar silos at Bury St Edmunds. In this case history the contact stresses are assumed to be uniform. Although the loads imposed by the silos and contents are well known, the contact stresses are not. Nicoletto (1979) showed using a soil-structure interaction analysis that the contact stress distribution under the slab was not uniform. In addition the analysis showed that the distribution varies as the total load is increased. When the silos are empty the dead weight of the structure is distributed over the slab giving a uniform contact stress distribution. This uniform distribution is maintained as a silo is loaded with sugar. However there is a point during the loading when arching within the mass of sugar causes much of the live load to be transmitted through the
wall of the silo. Hence high contact stresses occur beneath the silo wall. Evidence for this mechanism is may be found in settlement measurements made of the foundation slabs. The settlement profiles of the base of the silos during the loading stages (Fig. 2.2/21) show the centre of the slab settling more than the edge at the start of the loading sequence. As the load increased the maximum settlement moved from the centre to the edge of each slab.

The implication of the above mechanism is that yielding in the chalk was probably caused by the contact stresses developed at the edge of the slabs which was much greater than the contact stress based on the known load and the area of the slab. Hence the reliability of this case record is brought into question particularly with respect to the post-yield load-settlement behaviour. Based on the mechanism described above it is likely that the actual post-yield stiffnesses are lower than those determined using a uniform contact stress distribution.

Another factor which brings into question the reliability of this case record is the lack of control over the rate of loading and unloading of the rock mass. The initial load was applied rapidly giving rise to high pre-yield stiffnesses since little creep is permitted. The creep is seen to dominate in the post-yield section due to reduction in loading rate. If this case record is removed from the database the only reliable data left for post-yield load-settlement behaviour at large or full scale is that of the author.

The post-yield load-settlement behaviour is likely to be associated largely with the compression of destructured chalk at the asperities of contact within the sub-horizontal discontinuities. The load-deformation behaviour of destructured intact chalk discussed in section 5.2 was found to be independent of initial dry density. This suggests that the in-situ load-settlement behaviour may also display such a lack of dependence on dry density. There is certainly no relationship between dry density and $E_v$ indicated from the plate loading test data. This may explain the relatively
narrow range of $E_y$. However in the case of high porosity chalk at test site C there is evidence of more general pore structure collapse giving rise to exceptionally low values of post-yield modulus. This collapse phenomenon is discussed further later.

**Yield Bearing Pressure $q_e$**

The most noticeable feature of the load-settlement behaviour of chalk is change in gradient of the load-settlement curve. This feature is generally associated with yielding within the rock mass since large irrecoverable deformation is known to occur if the yield stress is exceeded. However little is known about the mechanisms involved and at the present time there are no reliable means of predicting when the rock mass will yield. The most likely mechanism is that of local yielding of the asperities of contact within the sub-horizontal discontinuities. The laboratory experiments described earlier suggest that a critical contact area must exist below which the load-compression behaviour of a single discontinuity displays yield whereas if the contact area is above the critical value there is no yield and a concave load-compression curve is observed. It seems logical to assume that the critical contact area is related to the dry density of the chalk. However there has been no systematic study to investigate either of these hypotheses and hence they should be regarded as subjective.

It will be seen from the literature review (Fig. 2.3/19) that the yield bearing pressure $q_e$ derived from plate loading tests on near-surface weathered chalk has a very limited range (200 to 600 kPa). The values of $q_e$ derived from the plate tests carried out by the author fall within this range (see Table 5.3/1). The highest values of $q_e$ were measured at test site C, whereas the lowest were measured at test site A. This implies a relationship between contact area and $q_e$. The ratio of the $q_e$ to the yield stress of intact chalk ($\sigma_y$) may give some indication of the degree of contact across the sub-horizontal discontinuities. The average ratios are given in Table 5.3/3. These ratios suggest that at sites A and B the contact area ratio (ie area of discontinuity /
area of contact) is 2% or less and at site C it is 14%. Experimental work by Duncan & Hancock (1966) and Pyrak-Nolte et al. (1990) has shown that for a wide range of rock types including chalk the contact area ratio at low stress levels (<3MPa) is generally less than 10%. This is agrees well with the ratios determined indirectly for the test sites.

Table 5.3/3 Ratio of $q_e$ to intact yield stress ($\sigma_y$)

<table>
<thead>
<tr>
<th>Site</th>
<th>Dry density (Mg/m$^3$)</th>
<th>$q_e/\sigma_y$ (%)</th>
<th>Fracture-block system</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.85</td>
<td>&lt; 2</td>
<td>Loose</td>
</tr>
<tr>
<td>B</td>
<td>1.55</td>
<td>2</td>
<td>Intermediate to tight</td>
</tr>
<tr>
<td>C</td>
<td>1.35</td>
<td>14</td>
<td>Tight</td>
</tr>
</tbody>
</table>

Yield Bearing Pressure $q_y$

The yield bearing pressure $q_y$ is found from the load-settlement curve by extrapolating the post-yield line back to zero settlement. Although it has no real physical meaning it is used to define the position of the linear portion of the post-yield load-settlement curve. It is also useful for comparison with $q_e$ since the difference between them gives an indication of the curvature of the load-settlement curve. Ideally if the two pre-yield and post-yield curves were entirely linear $q_y$ would occur at a lower stress than $q_e$ causing $(q_y - q_e)$ to be negative. However the curvature between the two linear sections of the curve general gives rise to positive values of $(q_y - q_e)$. For test site A the curvature between the pre and post-yield sections is relatively small and $q_y - q_e$ is about 100 kPa. The greatest curvature was observed at test site C. Here the difference $q_y - q_e$ was as much as 375 kPa. At Mundford the difference between $q_y$ and $q_e$ for the plate loading tests was between 300 and 500 kPa (Burland and Lord, 1970) indicating a high degree of curvature prior to the establishment of $E_i$. For the silos at Bury St Edmunds the difference was between 5 and 80 kPa indicating a more sudden change in gradient. However
in the light of the earlier discussion relating to this case record it is doubtful whether these results are reliable.

The measurement of $q_y$ is subjective since it depends upon the length of the post-yield load-settlement curve. For example at test site C (Needham Market Fig. 4.1/26 and 4.1/27) if test 2 were terminated at 600kPa $q_y$ may be calculated as being 330 kPa respectively. However this tests were taken to a bearing pressure in excess of 1MPa. Under these conditions $q_y$ would be measured as 800 kPa.

**Collapse Settlement at Needham Market (Site C)**

The chalk at test site C showed unusual load-settlement behaviour. Plate loading tests 2 and 3 at this site displayed excessive post-yield settlements. The maximum bearing pressures applied in tests 2 and 3 were 1.2MPa and 1.4MPa respectively. The average settlements measured in each test were greater than 80mm. This is more than double the settlements measured at similar bearing pressures at the other test sites. It will be seen from Fig. 4.1/27 that the initial stages of the load-settlement curves for these tests follow the normal yielding behaviour. However before $E_y$ can be fully established a further change in gradient occurs. This is shown as point A in Fig. 4.1/27. At bearing pressures greater than that at point A the stiffness drops to about 10 MPa (shown as $E_c$ in table 4.1/1), which is more typical of a soil than a weak rock. During the tests plate settlements associated with each loading increment beyond point A often occurred so rapidly during the application of the increment that the target load could not be reached without a rest period of up to 20 minutes. The evidence points towards some form of collapse. Such a mechanism is supported for test 3 by the crushed chalk observed beneath the plate in the trial pit. However no evidence of general crushing was seen in the trial pit at test location 2.

The load-settlement curve for test 3 in Fig. 4.1/27 is concave between bearing pressures of 600 and 900 kPa which is indicative of compression of
destructured chalk. This is consistent with the evidence of crushing seen beneath the plate. However the gradient of the load-settlement curve steepens at a bearing pressure of 900 kPa. This may be associated with the fact that the loading increment was doubled at this point. The shape of the load-settlement curve for test 2 is a more or less linear for bearing pressures greater than 900 kPa. This may be a result of local collapse at the asperities of contact within the sub-horizontal discontinuities spreading as the bearing pressure increases towards the yield stress for the intact chalk which is only about 2 MPa.

The load-settlement curve for test 1 shown in Fig. 4.1/26 is indicative of a greater degree of contact across the discontinuities. It displays a highest values of $E_i$ and $q_e$ for site C and does not exhibit collapse. However if the load-settlement curve is examined carefully it will be seen that the curve continues to steepen up to the maximum bearing pressure of 1000 kPa. This is clearly seen in the secant modulus-bearing pressure plot shown in Fig. 4.1/31. It is likely that the chalk at test location 1 was approaching collapse when the loading stage was terminated.

The large settlements observed at site C did not result in the complete closure of the major bedding plane discontinuities. Maximum apertures of between 5 and 10mm were observed in the trial pits within 2m below the underside of the plate.

The collapse phenomenon observed at site C has not been recognised in the literature. The Silos at Bury St Edmunds were founded on a similar material to that at site C. They suffered settlements of over 30mm at bearing pressures less than 400 kPa. The very low values of post-yield stiffness were not observed at Bury St Edmunds. It is not clear whether collapse of asperities of contact played was significant at this site although the likelihood is great based on the experience at test site C.
Creep

Little attention has been given to the time-dependent behaviour of the chalk. Based on time-settlement behaviour of in-situ loading tests at Mundford, Burland and Lord (1970) found that near surface weathered chalk displayed marked time-dependent behaviour. This was attributed to creep and it was noted that the creep rate increased with load intensity. This phenomenon was also observed in near surface weathered chalk at Luton (Marsland and Butcher, 1983, Powell et al, 1990).

Kee (1974) observed significant time-dependent settlements for the sugar silos at Bury St Edmunds and suggested that primary consolidation can be up to 100% of the immediate load-dependent settlement. However it is unlikely that Terzaghi's model of time-dependent settlement based on hydrodynamic time-lag would be applicable in the case of cemented materials such as chalk. Moreover the interpretation of the time-dependent settlement behaviour at Bury St Edmunds is complicated by the lack of control over the sequence and rate of loading and unloading of the four silos.

Burland and Lord (1970) suggest that the rate of creep may be expressed as a proportion, R, of the immediate settlement for an applied pressure, as:

\[
(\text{Creep ratio}) \quad R = \frac{\text{Settlement}}{\text{cycle of log time}} \times \frac{1}{\text{Observed total immediate settlement}} \times 100\%
\]

The above equation is based on the fact that the plate loading tests carried out at Mundford in weathered near surface chalk generally showed a linear relationship between settlement and log time for times after loading in excess of 20 minutes (Burland and Lord, 1970). As the loading intensity is increased
the amount of settlement which occurs also increases. Hence the settlement which occurs over a log cycle of time (ie the gradient of the log time-settlement curve) is normalised using the initial settlement. The implication is that R represents some fundamental property of the rock mass. In support of this values of R for plate loading tests reported in the literature are generally fall within a narrow range over a wide range of loading intensities (10 to 15% see Fig. 2.2/50). However it will be seen from Fig. 2.2/50 that most of the data are for bearing pressures greater than 500 kPa. For most weathered near surface chalks this is greater than the yield bearing pressure q_e. The values of R for bearing pressures less than 500 kPa the values of R tend to be greater than 30%.

Values of R calculated for the plate loading tests carried out at the three test sites have been plotted against bearing pressure in Fig 4.1/44. Difficulty was experienced in calculating the creep ratio R due to the following factors:

(i) The log-time settlement curves are not always linear. This is particularly so between the yield bearing pressure q_e and the establishment of E_y.

(ii) The term 'immediate settlement' has not been defined adequately.

In the case of the log time-settlement curve being non-linear beyond the first 100 minutes after the application of a loading increment the average gradient was calculated. In general the immediate settlement was taken either at 20 minutes after the application of the applied load or at the break in slope of the log time settlement curve within the first 100 minutes after the increment was applied.

Fig. 5.3/6 shows values of the creep ratio R reported in the literature superimposed on those derived from the author's tests. It will be seen from Fig. 5.3/6 at bearing pressures below 500 kPa there is a wide scatter of R values ranging from 3% to 55% The majority of R values are greater than
20% which agrees with the limited data from Mundford and Luton. The scatter of data possibly reflects the subjective nature of the immediate settlement used in the calculation of R. Between bearing pressures of 500 kPa and 1000 kPa the R values are between 5 and 20% and show reasonably is good agreement with data from Mundford and Luton. Above 1000 kPa the values of R from the author’s tests are generally between 2 and 4% which are lower than those obtained at Mundford.

There appears to be a trend of reducing R value with increasing bearing pressure. It is not possible to say whether this is associated with any fundamental behaviour since the calculation of R is in many subjective. The implication of using immediate settlement to normalise the gradient of the log time-settlement curve is that it is not associated with creep. However it is likely that creep occurs from the instant the load is applied.

Powell (1990) presented observed time-settlement behaviour by plotting creep rate (in mm/day) against log time (see Fig. 2.2/51) for different loading intensities. For a given loading intensity the creep rates are seen to reduce with log time in an exponential manner. At stress levels below yield the curves all showed very similar shape. Once the yield stress is exceeded the creep rates increase significantly.

Figs. 4.1/45 to 47 show typical relationships between creep rate and log time for the plate loading tests carried out by the author. It will be seen that they display features similar to those observed by Powell (1990). All the curves display a similar shape with creep rates for a given stress level generally reducing to less than 1mm per day after 1000 minutes. This would appear to justify the decision to maintain each loading increment for 24 hours. Burland and Lord (1970) suggest that the time interval between loading increments should be based on achieving a creep rate of 0.29 mm per day. However it will be seen from Figs 4.1/45 to 47 that this rate is only achieved after 24 hours in the case of the lowest bearing pressures (ie < 400kPa).
The creep rate curves shown in Figs. 4.1/45 to 47 fall into two distinct groups shown as envelopes in Fig. 5.3/7 based on whether the stress level is above or below yield. The pre and post-yield time-settlement behaviour appears to be independent of differences in rock material and rock mass characteristics since no trend can be identified within each group that permits sub-division on the basis of test site. However the collapse behaviour of the Needham Market chalk (site C) does stand out as exhibiting some of the highest post-yield creep rates.

Fig. 5.3/8 shows that the creep rate at 24 hours increases significantly with bearing pressure. The yielding phenomenon appears to be the cause of this increase in creep rate with bearing pressure. These high creep rates at 24 hours imply that the post-yield modulus $E_y$ will be overestimated. The fact that at stress levels greater than $q_e$ each loading increment was applied at a different creep rate may explain the linearity of the post-yield load-settlement curve. With the exception of the Needham Market chalk the data shown in Fig. 5.3/8 appear to be independent of the differences in rock material and rock mass characteristics observed at the three test sites.

The creep rate-log time plots do not permit the time-settlement behaviour to be studied in the early stages of the test when creep rates exceed 10 mm per day or in the later stages once the creep rate has fallen below 1 mm per day. In order to examine more fully the change in creep rate with time it is necessary to plot log creep rate against log time. The data from the author's tests have been plotted in this manner in Figs. 4.1/45b-47b. It will be seen from these figures that there is generally a linear relationship between log creep rate and log time. The irregular nature of some of the creep rate curves (eg Needham Mkt. Test 3 Fig. 4.1/47b) stems from the fact that during the tests the disc springs and the loading frame did not have sufficient compliance to maintain the load intensity constant and hence it required periodic adjustment. The sudden increase in load causes the creep rate to increase. However at some time after the load is applied the creep rate comes back to the original trend.
The log creep rate curves shown in Fig. 4.1/45b-47b can be divided into two groups on the basis of whether the applied stress is above or below the yield bearing pressure $q_e$. The envelopes shown in Fig. 5.3/9. The post-yield behaviour is shown by two envelopes in Fig. 5.3/9. This is because the Needham Market chalk displayed higher than normal creep rates whilst undergoing collapse. The gradients of the log creep rate lines within each group are very similar (-0.57 for pre-yield and -0.97 for post yield). The position of a log creep rate line within a given group is dependent upon the applied bearing pressure as indicated in Fig. 5.3/9.

Creep rate data from the long term test carried out at North Ormsby have been plotted in the manner discussed above in Fig 4.1/49. The data plotted in Fig. 5.3/10 have been derived from dial gauges and precise levelling (see chapter 4 for details about instrumentation). It will be seen from Fig. 4.1/49b that the trend observed in the first 24 hours after loading continues for 40 days. The test had to be terminated after 40 days so it was not possible to establish whether the increase in creep rate seen at the end of the test in Fig. 4.1/49 continued or not. The data from the long term test indicate that the log creep rate lines for the post-yield condition shown in Fig. 5.3/9 can be extrapolated beyond 1440 minutes. By extrapolating these lines it is clear that loading increments would have to be maintained for several days based on the creep rate suggested by Burland and Lord (1970).

Records of settlement for tests in which the load has been maintained for more than 5 days are rare. The only case history presented in the literature is that of the tank test at Mundford in which the load was maintained for about 1 year. The loading intensity in this case was less than $q_e$. The long term test carried out at North Ormsby under a bearing pressure of 900 kPa lasted 40 days. This represents the only record of long term post-yield creep in chalk.

The long term test demonstrated that at stresses above $q_e$ the plate was continuing to settle a month after the load had been applied (see Fig. 4.1/48). It will be seen from Fig. 4.1/48a that less than 50% of the
settlement observed during the 40 day period occurred in the first 24 hours. It should be noted however that since most structures taken more than 6 months to complete most of this creep settlement would be 'built out' during construction.

Summary

The above discussion has highlighted the following points concerning the load-settlement behaviour of the chalk.

- The yielding behaviour which gives rise to the convex load-settlement curves commonly seen in plate loading tests on chalk is generally not expected from a rock mass in which deformation is dominated by the normal closure of discontinuities.

- The yielding behaviour of chalk appears to be related to a combination of limited contact area across discontinuities and the yielding behaviour of the intact rock. Solution weathering is largely responsible for reducing the contact area across major primary discontinuities. It is the local yielding of the asperities of contact which is thought to bring about yield in the rock mass.

- The plate loading tests carried out at the three test sites all displayed yield and the load-settlement behaviour could be described using the simplified model proposed by Burland and Lord (1970) which makes use of the parameters $E_i$, $E_y$, $q_e$ and $q_y$.

- The range of initial modulus for weathered near-surface chalks is between 300 MPa and 1000 MPa based on this published settlement data for large scale loading tests and full scale foundations together with the results of this research.
The initial modulus $E_i$ was often difficult to measure accurately from the load-settlement curve due to limited number of settlement measurements prior to yielding and the small settlements being close to the limit of resolution of the instrumentation.

It was found that $E_i$ could be determined with more confidence from a plot of secant modulus ($E_s$) against bearing pressure. Two types of curve were identified from the secant modulus plots. Type I displayed a relatively constant modulus up to the yield bearing pressure $q_e$ indicating a relatively linear pre-yield load-settlement curve. Type II displayed two concave curves with a cusp coinciding with $q_e$ as interpreted from the load-settlement curve. It would appear that the rock mass is undergoing some form of yield from a very early stage in the loading test and that another mechanism causes the cusp in the secant modulus curve. In general the type II curves are associated with high initial stiffness. In the case of site C the initial stiffness approached that of the intact rock. It is thought that variations in contact area and looseness of the fracture block system are responsible the two types of secant modulus curves.

The initial modulus $E_i$ appears to be inversely proportional to the intact dry density of the chalk. This is largely the result of the variation in looseness of the fracture block system which is brought about by the intact mechanical properties influencing the style of mechanical weathering. Chalks of high dry density ($\rho_d > 1.60$ Mg/m$^3$) tend to be brittle and will support a loose fracture block system which gives rise to low values of $E_i$ (300 to 400 MPa). Chalks of low dry density ($\rho_d < 1.50$ Mg/m$^3$) generally will not support a loose fracture block system and are characterised by high values of $E_i$ (600 to 1000MPa).
Rock mass factors for the three test sites ranged from 0.027 to 0.202 (see Table 5.3/2) indicating the relative dominance of the discontinuities in controlling rock mass compressibility. The rock mass factors reduce as the fracture block system becomes more loose.

The post-yield stiffness $E_y$ is generally an order of magnitude lower than $E_t$ and has a much more limited range of values than $E_t$. The range of $E_y$ for large scale loading tests is generally between 40 and 70 MPa. This is based largely on the results of the author's tests since there is only one case record in the literature that covers post-yield behaviour at large or full scale. This case record is for the Bury St Edmunds Silos. The reliability of the post-yield settlement data is questioned on the grounds that the data have been interpreted on the basis of a uniform contact stress distribution, whereas both field evidence and the results of a full soil-structure interaction analysis suggest that it is non-uniform.

The yield bearing pressure $q_e$ has a limited range of between 200 and 600 kPa. A comparison of $q_e$ and the yield stress $\sigma_y$ of chalk under conditions of uniaxial strain for the three test sites suggest that $q_e$ may be controlled by the contact area across sub-horizontal discontinuities. However the mechanisms causing yield are not sufficiently well understood to permit the yield stress to be predicted for chalk.

The yield bearing pressure $q_y$ has no real physical meaning but is simply used to fix the position of the post-yield load-settlement curve.

The difference between $q_y$ and $q_e$ can give an indication of the degree of curvature of the load-settlement curve between the pre and post-yield portions.

The yield bearing pressure $q_y$ is considered subjective since its magnitude depends on the length of the post-yield load-settlement curve.
• Large post-yield settlements at site C (Needham Market) caused the stiffness to reduce to about 10 MPa which is similar to that of a soil. These settlements are thought to be associated with pore structure collapse within the asperities of contact associated within the major bedding discontinuities.

• The parameter R suggested by Burland and Lord is considered to be subjective since the log time-settlement curves are not always linear (particularly before and at yield) and the term immediate settlement is not defined. Furthermore the use of immediate settlement to normalise the gradient of the log time-settlement curve fails to recognise that creep is initiated the instant a loading increment is applied. In reality there is no "immediate settlement".

• Plots of creep rate against log time for the plate loading tests indicate that the creep rate has reduced significantly 24 hours after a loading increment has been applied. This would appear to justify the choice of such a period between loading increments. However the creep rates at 24 hours were generally greater than the target of 0.29 mm per day suggested by Burland and Lord (1970).

• Creep rate-log time curves display similar shape for different bearing pressures. Significantly higher creep rates are observed after the onset of yield. This gives rise to two distinct groups of creep rate curves based on whether the bearing stress in below or above $q_e$. Upper and lower bounds for each group are shown in Fig. 5.3/7.

• There is a trend of increasing creep rate at 24 hours with increasing bearing pressure. It is considered that since after yield the loading increments were applied at increasing creep rates that this may be largely responsible for the linear nature of the post-yield load-settlement curve.
Plots of log creep rate against log time indicate a linear relationship. The gradients of the lines for different bearing pressures are related to the yield stress. At bearing pressures below yield the lines have a common gradient of -0.57 whilst those at bearing pressures above yield have a common gradient of -0.97. Although the lines share a common gradient the position of the line is related to bearing pressure as shown in Fig. 5.3/9.

The long term test carried out at North Ormsby under a bearing pressure of 900 kPa lasted 40 days. This represents the only record of long term post-yield creep in chalk.

The long term test at North Ormsby demonstrated that at stresses above $q_e$ the that less than 50% of the settlement observed during the 40 day period occurred in the first 24 hours. Although the plate was still settling 40 days after the application of the loading increment the rate of settlement was only 0.0246 mm/day. The creep rate curve shown in Fig. 4.1/49 indicates the rate was still reducing steadily towards the end of the test.
Characteristic relationships between secant modulus and bearing pressure observed at sites A, B, and C.
Fig. 5.3/2  Pressure-settlement ratio results showing pre-yield behaviour for structures and 1.8m dia. plate loading tests on chalk.
Fig. 5.3/3 Pressure-settlement ratio results showing pre-yield behaviour of structures and 1.8m dia. plate loading tests on chalk classified according to dry density.
Fig. 5.3/4  Relationship between rock mass factor j and fracture spacing for the chalk (from Hobbs, 1975).
Pressure-settlement ratio results showing yielding behaviour for structures and 1.8m dia. plate loading tests on chalk.
Fig. 5.3/6  Relationship between bearing pressure and creep ratio R for the chalk based on published data and 1.8m dia. plate loading tests at site A, B and C.

Fig. 5.3/7  Characteristic relationships between creep rate and log time observed at sites A, B and C.
Fig. 5.3/8  Relationship between creep rate at 24 hours and bearing pressure.

Fig. 5.3/9  Characteristic relationship between log creep rate and log time observed at sites A, B and C.
5.4 Methods of Determining Mass Stiffness Parameters

It has been shown in the literature review that the load-settlement behaviour of the chalk can be described using the simplified model proposed by Burland and Lord (1970). It has been demonstrated in the previous section that the load-settlement behaviour observed in the author’s plate loading tests may be described using the Burland and Lord model. The model uses four parameters $E_v$, $E_y$, $q_e$ and $q_y$ to describe the load-settlement behaviour of chalk. Table 2.3/1 lists the methods that may be used to determine these parameters. These methods include:-

- Pressuremeter
- Plate loading test
- Geophysics
- Standard Penetration Test
- Visual Assessment

It will be seen from Table 2.3/1 that only the pressuremeter and plate loading test can be used to provide measurements of all four parameters. The pressuremeter loads the rock mass in the wrong direction to provide complete similitude with a foundation loading if the rock mass displays anisotropy. The plate loading tests however provides good similitude. The rigidity and geometry of the plate results in different elements of the rock mass beneath the plate following different stress paths. This gives problems in extrapolating from the test scale to that of a full-scale foundation. In general a plate diameter greater than five times the average sub-horizontal discontinuity spacing is recommended in order to minimise this sensitivity. In this research a plate diameter of 1800mm was employed. The plate diameter is more than 10 times the average sub-horizontal discontinuity spacing.
observed at the three test sites. This allows a better comparison to be made with the behaviour of full-scale foundations. It has been demonstrated in the previous section that the load-settlement behaviour observed in the author’s tests shows good agreement with that of large scale-loading tests and full-scale structures reported in the literature on chalk with similar rock mass characteristics.

The other methods listed in Table 2.3/1 are generally only used to determine the initial modulus $E_i$. In the case of geophysics this is a result of the very small strain levels imposed upon the rock mass by the seismic energy sources used. In the case of the SPT and visual assessment the stiffness parameters are determined from empirical relationships based largely on plate loading test results. The database for post-yield modulus is limited in comparison to that for pre-yield behaviour. Hence typically these methods are only used to determine $E_i$.

Geophysics, in the form of surface-wave seismic tests, together with the SPT and visual assessment were carried out at every test site at or adjacent to the plate loading test locations (see Chapter 4). The results of these tests and observations were used to predict the settlement of the plate at each test location. These predictions are discussed below.

It is clear from the above discussion that only predictions of pre-yield settlements can be compared. Hence plate settlements at bearing pressure of 200kPa have been used to compare observed and predicted settlements.

**Settlement Predictions using Surface-Wave Geophysics**

Surface-wave seismic tests were carried out over all but one (North Ormsby test 1) of the nine plate locations. The results of these tests are shown in the form of shear modulus depth profiles in Figs. 4.2/16, 17 and 18. It will be seen from these shear modulus depth profiles that for sites A and B the shear modulus increases steadily with depth, whereas at site C it is almost uniform.
It was demonstrated in Chapter 4 that the dry density was reasonably uniform for each of the test sites. This would suggest that the intact stiffness is also relatively uniform for each site and hence the measured changes in shear modulus with depth must reflect the spacing and tightness of the discontinuities. Very low values of shear modulus were measured close to the ground surface at site A (7 MPa) as a result of the loose fracture block system which is characteristic of the weathering style of this chalk. The shear modulus was seen to increase by about 400 MPa over a distance of 5m indicating a significant reduction in the looseness of the fracture block system with depth and increased discontinuity spacing. These features are confirmed by the description of the rock mass given in Chapter 3.

Site B displays shear stiffnesses of between 100 and 300 MPa for the structured chalk near the ground surface. This reflects the tighter fracture block system observed at this site. This is also reflected in the increase in stiffness with depth which is between about 400 and 600 MPa over 5m. This increase is greater than that observed at site A and is attributed largely to the increase in discontinuity spacing with depth.

The stiffness was more or less uniform at site C with values between about 350 and 480 MPa. This reflects the tight nature of the fracture block system at this site. The sub-horizontal discontinuities here are characterised by secondary fractures which are generally tight and moderately narrow to moderately widely spaced, and primary bedding discontinuities which are widely spaced and show evidence of solution weathering with apertures of up to 15mm. It is clear that the secondary fracturing had little influence on the stiffness measurements. The average Young's modulus for the rock mass at site C determined from the surface-wave seismic tests is 1.1 GPa (assuming $v = 0.25$). The rock mass factor based on this value of $E_m$ is 0.324 which is similar to that based on $E_m$ for the plate loading tests.

In cases where the stiffness increases with depth the commonly-used assumption that the stiffness is uniform will lead to errors in predicting
foundation settlements. A number of solutions are available that cater for foundation on a non-homogeneous elastic half-space (Carrier and Christian, 1973, Brown and Gibson, 1979, Rowe, 1981). All these solutions assume a linear increase in stiffness with depth. The method used in this case to predict the settlement of the plate at the different sites is that of Brown and Gibson (1979). This method was chosen because it makes use of the shear modulus G which is the parameter that is measured from the surface-wave seismic tests. The other models use Young's modulus E which can only be determined in this case by assuming a value of Poisson's ratio.

In order to use the Brown and Gibson model it was necessary to determine the shear modulus at the level of the plate ($G_0$) and the rate of increase of shear modulus with depth (m). This was done by determining the best fit straight line through the shear-modulus-depth data using the linear regression analysis in LOTUS 1-2-3. The values of $G_0$ and m determined in this manner are shown in Table 5.4/1.

The predictions of plate settlement at a bearing pressure of 200kPa are shown in Table 5.4/2 and Fig. 5.4/1. It will be seen from Fig. 5.4/1 that the geophysics generally under-predicts the plate settlement. The seismic energy subjects the rock mass to very small strain levels very rapidly hence creep is negligible. However in the plate tests where the load was maintained constant for 24 hours the creep component of deformation will be much more significant. Hence it is not surprising that the geophysics under-predicts the plate settlements.

In general the modulus-depth profiles derived from the surface-wave measurements under-predict the plate settlement by a factor between 1 and 2 (see Table 5.4/2). In some cases the yield stress $q_e$ was found to be 200 kPa. It will be seen from Fig. 5.4/1 that the ratios of observed to predicted settlement at 100 kPa bearing pressure display a greater degree of over-prediction and with exception of one result from site A (North Orsmby) at 300 kPa bearing pressure all the ratios indicate a under-prediction of
settlement. This clearly demonstrates the need to know \( q_e \) before assuming elastic behaviour.

**Table 5.4/1** Values of \( G_0 \) and \( m \) derived from surface-wave seismic tests

<table>
<thead>
<tr>
<th>Site</th>
<th>Test No.</th>
<th>( G_0 ) (MPa)</th>
<th>( m ) (MPa/m)</th>
<th>( G_{av} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1</td>
<td>7</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>2</td>
<td>84</td>
<td>78</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>3</td>
<td>175</td>
<td>116</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>1</td>
<td>207</td>
<td>89</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>2</td>
<td>323</td>
<td>91</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>1</td>
<td>380</td>
<td>8</td>
<td>460</td>
</tr>
<tr>
<td>C</td>
<td>2</td>
<td>486</td>
<td>0.3</td>
<td>487</td>
</tr>
<tr>
<td>C</td>
<td>3</td>
<td>331</td>
<td>1.6</td>
<td>336</td>
</tr>
</tbody>
</table>

In the case of plate loading test 2 at site A (North Ormsby) the settlement of the plate is over-predicted by a factor of 3 times (see Table 5.4/2). The ratios of observed to predicted settlement for test 2 are indicated by the label (A) in Fig. 5.4/1. It will be seen from Fig. 5.4/1 that the settlement of the plate is grossly over-predicted even at a bearing pressure of 300kPa which is greater than the yield stress \( q_e \) recorded for this test. It will be seen from Table 5.4/1 that the shear modulus \( G_0 \) directly beneath the plate is only 7MPa. As far as the settlement model is concerned such a small value of shear modulus is more or less equivalent to \( G_0 = 0 \). This special case has been studied by Gibson (1967) (Gibson, 1968 and Gibson, 1969) and Brown and Gibson (1979) assuming a Poisson's ratio of 0.5. The Poisson's ratio of intact chalk has been shown to be about 0.25 (see chapter 2). In cases where the stiffness is equal to zero at the plate level predictions of settlement have been shown to be sensitive to Poisson's ratio (Carrier and Christian, 1973). The sensitivity
of predicted settlements for a circular plate to Poisson's ratio has been studied using a finite element analysis (CRISP90) for different modulus-depth profiles. The results of the finite element analysis are summarised in Fig. 5.4/2. It will be seen from Fig. 5.4/2 that sensitivity of predicted settlement to Poisson's ratio increases as the stiffness at the plate level is reduced. The over-prediction of the settlement for plate loading test 2 at North Ormsby clearly highlights this sensitivity to Poisson's ratio since the predictions where based on a Poisson's ratio of 0.5.

Table 5.4/2  Ratio of observed settlement and predicted settlement based on stiffness profiles determined from surface-wave seismic tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Site A North Ormsby</th>
<th>Site B Leatherhead</th>
<th>Site C Needham Market</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.3446</td>
<td>1.5316</td>
<td>0.9945</td>
</tr>
<tr>
<td>2</td>
<td>0.9344</td>
<td>1.3277</td>
<td>2.1630</td>
</tr>
<tr>
<td>3</td>
<td>1.4036</td>
<td>2.6353</td>
<td></td>
</tr>
</tbody>
</table>

*Settlement Predictions using the Standard Penetration Test*

Standard Penetration Tests were carried out at all the tests sites in boreholes constructed using a variety of drilling methods. At each site Standard Penetration Tests were carried out in holes advanced using rotary drilling to allow the comparison between sites. The SPT results in the form of N values are given in Chapter 4 (Figs. 4.2/3, 7 and 9).

It will be seen from the data given in Chapter 4 that the lowest SPT N values were recorded at site C (Needham Mkt.). The SPT N values showed relatively little scatter and were generally between 5 and 10 indicating uniform conditions. The uniformity of the chalk at this site was clearly shown in the profiles derived from dynamic probing tests (Fig. 4.2/11b) and the
profiles of shear modulus with depth derived from the geophysics (Fig. 4.2/18). The low values of N derived from the Standard Penetration Tests reflect the low strength of this chalk which displays the lowest dry density of all the sites. The uniformity of the rock mass may be attributed to the tight fracture block system that is characteristic feature of the weathering style at this site.

It will be seen from Fig. 4.2/9 that the SPT N values measured at site C do not show any sensitivity to drilling method. The drilling methods employed at this site included light cable percussion and rotary. Since the chalk at this site is not very brittle the light cable percussion method of drilling does not tend to shatter and loosen the rock ahead of the bottom of the borehole. The SPT is not able to resolve the differences in the degree of mechanical disturbance that results from each drilling method in this chalk.

The highest SPT N values were recorded at site B (Leatherhead) although the chalk which exhibits the greatest intact strength is found at site A (North Ormsby). The reason for this may be associated with the looseness of the fracture block system. At site A the fracture block system was loose whereas at site B it was significantly tighter. This suggests that the SPT N value is sensitive to weathering style in the more brittle varieties of chalk with dry density greater than 1.50 Mg/m³.

The SPT N values measured in rotary drilled holes at sites A and B showed significant scatter making it difficult to establish whether there was a any general improvement in the quality of the rock mass with depth. An increase in the SPT N value might be expected to be associated with an increase in the sub-horizontal discontinuity spacing. Clearly if the discontinuity spacing is less than 300mm it is likely that the SPT N value may exhibit some sensitivity to the discontinuity spacing, although correlations with spacing are not possible due to the influence of aperture and the looseness of the fracture block system. In cases where the spacing less than 300mm and the discontinuities are generally tight or where the spacing is greater than 300mm
the SPT N value will be sensitive to changes in the intact properties of the chalk, particularly strength.

Fig. 4.2/7 shows the results of Standard Penetration Tests carried out in holes advanced using light cable percussion at site B to depths of up to 20m. It will be seen from Fig. 4.2/7 that the SPT N value generally increases with depth although there is significant scatter of results. This increase is not so clear within the top 5m, which is the maximum depth of investigation beneath the plate locations. However an increase in penetration resistance with depth was clearly observed in the dynamic penetrometer test results shown in Fig. 4.2/11a.

For the tests carried out in rotary drilled holes the SPT N values varied between 19 and 40 at site A (North Orsmby) and between 20 and 65 at site B (Leatherhead).

Fig. 4.2/7 clearly shows the sensitivity of SPT N value to drilling method at site B. The SPT N values measured in holes made using rotary drilling are generally about double those measured in holes advanced using light cable percussion. The difference is attributed to the mechanical disturbance which is known to occur ahead of the base of the hole when using light cable percussion boring in chalk. In brittle chalks such as that found at sites A and B the chalk at the base of the borehole is generally shattered and loosened by the action of light cable percussion boring. The bands of sheet flints which dominated the chalk at site A made light percussion boring impractical and hence such a comparison of SPT N values could not be made for this site.

A number of empirical correlations between SPT N value and elastic modulus have been proposed (Wakeling, 1966,1970 Lake and Simons, 1970, Kee and Clapham, 1971, Stroud, 1988 and Powell et al, 1990). The commonly used correlations are shown in Fig. 2.3/16.
The correlations proposed by Wakeling (1970) are based largely on the
results of Standard Penetration Tests carried out in boreholes advanced by
light cable percussion adjacent to the tank loading test and plate loading tests
at Mundford, Norfolk. Wakeling (1970) proposed two linear correlations
between SPT N value and log (elastic modulus). The first, known as line A,
was determined for large settlements (ie settlement ratios >0.5%) and is
based upon the post-yield modulus E_y derived from plate loading tests. The
second, known as line B, was determined for very small settlements (ie
settlement ratios < 0.01%) and is based upon the initial modulus E_i derived
from plate loading tests and the tank loading test at Mundford. Wakeling
(1970) noted that since in practice foundations are often designed for total
settlements not exceeding about 25mm, which is about 0.5% to 1% of the
foundation width, line A would be appropriate for predicting foundation
settlements on chalk.

Kee and Clapham (1971) essentially redefined Wakeling's line A on the basis
of more plate loading test results. The plate loading tests used plate
diameters less than 440 mm. Kee and Clapham criticise the correlation
proposed by Lake and Simons (1970) on the grounds that they used small
diameter (140mm) plate tests which resulted in relatively low E values. The
plate loading tests on which Kee and Clapham base their correlation are also
small diameter being less than 865mm. Some doubt must therefore be cast
upon the reliability of this correlation. The line proposed by Kee and
Clapham has a smaller gradient than Wakling's line A and crosses it at an
SPT N value between 20 and 30 (see Fig. 2.3/16).

Stroud (1988), recognising that both stiffness E' and N vary with mean
effective stress for soil, proposed that the ratio E'/N_60 should be considered
together with its variation with strain level or degree of loading by the ratio
q/q_{ult}. The variation of E'/N_60 with q_{net}/q_{(ult)u}, for the Chalk is summarised in
Fig. 2.3/17 based on available data from in-situ loading tests of shallow
foundations, piles and large plates. q_{net} represents the average net effective
bearing pressure. Stroud (1988) suggests that for q/q_{ult} up to 0.15 a value of
E'/N_{60} of 15MPa should be employed and a for greater degrees of loading this value should be reduced to 5.5MPa. This approach represents a significant departure with respect to the other correlations which use a semi-logarithmic relationship between E and N.

It will be seen from Fig. 2.3/17 that these empirical correlations between SPT N value and stiffness are clustered around Wakeling's Line A which represents post-yield stiffness. However it should be pointed out that most foundations will have contact stresses less than 200 kPa. In the previous section of this chapter (5.3) it was demonstrated that generally most chalks will not experience yield at bearing pressures less than 200 kPa. It would appear therefore that the adoption of line A for settlement predictions is likely to lead to overdesigned foundations.

Wakeling's line B is based on the values of initial modulus measured at Mundford, Norfolk. It was noted by Burland and Lord (1970) that the $E_i$ values determined from the plate loading tests were probably too high owing the adhesion between the plate and the walls of the auger hole due to the plaster of Paris squeezed up during bedding of the plate. It is likely that line B will underestimate settlements and hence the more conservative line A is preferred for settlement predictions.

In order to predict the settlement of the plate at each site it was considered that on the basis of an elastic stress distribution, most of the settlement would be associated with deformations occurring within a depth of approximately one plate diameter beneath the plate. Thus the range of depths over which SPT N values were considered was between plate level and 2m below plate level. Average SPT N values over this range of depths are given in Table 5.4/3.
Table 5.4/3  SPT N values

<table>
<thead>
<tr>
<th>Site</th>
<th>Minimum SPT N value</th>
<th>Maximum SPT N value</th>
<th>Average SPT N value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>13</td>
<td>42</td>
<td>27</td>
</tr>
<tr>
<td>B</td>
<td>24</td>
<td>67</td>
<td>47</td>
</tr>
<tr>
<td>B*</td>
<td>6</td>
<td>34</td>
<td>20</td>
</tr>
<tr>
<td>C</td>
<td>5</td>
<td>12</td>
<td>7</td>
</tr>
</tbody>
</table>

* SPT N values derived from tests in light cable percussion bored holes

Settlement predictions have been made for a bearing pressure of 200kPa on the basis of a uniform elastic modulus derived from the average SPT N value for each site using the following correlations:

(i) Wakeling (1970) Line A
(iii) Kee and Clapham (1971)
(iv) Stroud (1988) using $E = 15N$ and $E = 5.5N$

For Stroud’s correlation the $N_{60}$ value is required. However the values of N used in the following settlement predictions have not been corrected for free fall hammer energy. The trip hammer used by the author to measure the N values at the test sites (in rotary drilled and augured holes) delivered 72% of the free fall hammer energy (Clayton, 1990b). The N values should therefore be multiplied by 72/60 to give values of $N_{60}$. The error in predicted settlement as a result of neglecting this energy correction is about 10% which would not seriously affect the pattern of results.

The settlement predictions are expressed in terms of ratios of observed to predicted settlement in Table 5.4/4. These results are illustrated graphically in Fig. 5.4/3. It will be seen from Fig. 5.4/3 that Wakeling (1970) line A and
Kee and Clapham (1971) over-predict the plate settlements by factors between 2 and 117 times. In the case of site B (Leatherhead) Wakeling's line A under-predicts the plate settlement by a factor of about 2. Clearly this large range of over-prediction is of concern, since it could lead to overdesign which is clearly not cost effective.

Table 5.4/4  **Ratios of observed to predicted settlements for a bearing pressure of 200kPa based on the Standard Penetration Test.**

<table>
<thead>
<tr>
<th>Method</th>
<th>Site A North Ormsby</th>
<th>Site B Leatherhead</th>
<th>Site B Leatherhead (SI Data*)</th>
<th>Site C Needham Mkt.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.1781</td>
<td>2.2073</td>
<td>0.0769</td>
<td>0.0086</td>
</tr>
<tr>
<td></td>
<td>0.1945</td>
<td>1.7195</td>
<td>0.0599</td>
<td>0.0114</td>
</tr>
<tr>
<td></td>
<td>0.1480</td>
<td>1.7546</td>
<td>0.0611</td>
<td>0.0204</td>
</tr>
<tr>
<td>Wakeling Line A</td>
<td>4.5559</td>
<td>28.4372</td>
<td>2.2168</td>
<td>0.3650</td>
</tr>
<tr>
<td></td>
<td>4.9760</td>
<td>22.1528</td>
<td>1.7269</td>
<td>0.4866</td>
</tr>
<tr>
<td></td>
<td>3.7865</td>
<td>22.6056</td>
<td>1.7622</td>
<td>0.8699</td>
</tr>
<tr>
<td>Wakeling Line B</td>
<td>0.1068</td>
<td>0.5103</td>
<td>0.0544</td>
<td>0.0104</td>
</tr>
<tr>
<td></td>
<td>0.1167</td>
<td>0.3975</td>
<td>0.0424</td>
<td>0.0139</td>
</tr>
<tr>
<td></td>
<td>0.0888</td>
<td>0.4056</td>
<td>0.0433</td>
<td>0.0248</td>
</tr>
<tr>
<td>Kee and Clapham</td>
<td>0.3591</td>
<td>0.4992</td>
<td>0.2124</td>
<td>0.0418</td>
</tr>
<tr>
<td></td>
<td>0.3923</td>
<td>0.3889</td>
<td>0.1655</td>
<td>0.0557</td>
</tr>
<tr>
<td></td>
<td>0.2985</td>
<td>0.3968</td>
<td>0.1689</td>
<td>0.0995</td>
</tr>
<tr>
<td>Stroud (E = 5.5 N)</td>
<td>0.9795</td>
<td>1.3614</td>
<td>0.5793</td>
<td>0.1139</td>
</tr>
<tr>
<td></td>
<td>1.0698</td>
<td>1.0605</td>
<td>0.4513</td>
<td>0.1518</td>
</tr>
<tr>
<td></td>
<td>0.8141</td>
<td>1.0822</td>
<td>0.4605</td>
<td>0.2714</td>
</tr>
</tbody>
</table>

The greatest over-prediction plate settlement using Wakeling's line A and the Kee and Clapham correlation is seen in the case of site C (Needham Market). Here the plate settlement were generally over-predicted by factors ranging between 40 and 117. This is due to the inability of the SPT to identify the looseness of the fracture block system in chalk of low dry density. At Needham Market the SPT N values were generally very low (<10) due to the low intact strength of the chalk but the stiffness of the rock mass was high because the fracture block system was tight. This highlights the fact that
empirical correlations have been made between measurements that bear little or no fundamental relationship with each other.

If the results from site C are ignored then the factors by which the plate settlement is over-predicted range between 2 and 11 when using Wakeling's line A and the Kee and Clapham correlation. Although the prediction is much improved it is still not satisfactory. Indeed it is made worse by the fact that in the more brittle chalks the SPT N value is sensitive to drilling method. It will be seen from Table 5.4/4 and Fig. 5.4/3 that reduction in SPT N values through the use of light cable percussion boring at Leatherhead causes the factors by which the plate settlements are over-predicted to be doubled.

The use of the Wakeling (1970) line B correlation generally results in the plate settlements being under-predicted by factors ranging between 4 and 28 times. This is clearly unsatisfactory and is likely to lead to inadequate foundations. The situation is improved to some extent when the low values of N are measured in the light cable percussion bored holes at Leatherhead are used to make settlement predictions. In this case the plate settlement was under-predicted by a factor of about 2 times.

At site C (Needham Market) the plate settlements were overestimated by Wakeling's line B by a factor between 1 and 3 times. This apparently reasonable result is again due to the low strength of the Needham Market chalk dominating the SPT N value.

Stroud (1988) recommends $E' = 5.5N_{60}$ for conservative estimates of settlements at moderate degrees of loading. Using this correlation the plate settlements were over-predicted by factors between 2 and 24 times. This range is clearly unacceptable. However it will be seen from the range of values for $q_{net}/q_{(ult)u}$ given in Table 5.4/5 that the use of the above correlation is inappropriate to the degree of loading being considered. The degree of loading is either close to or much less than 0.15 suggesting that the relationship $E' = 15N_{60}$ might be more appropriate. This yields the best SPT-
based predictions of settlement. For sites A and B the plate settlements are either over-predicted by a factor of 1.22 times or under-predicted by a factor of 1.4 times. However Stroud's correlation fails to provide a satisfactory set of predictions for site C. Here the plate settlements are over-predicted by a factor of between 4 and 9 times.

Table 5.4/5  Degree of loading \( \left( \frac{q_{\text{net}}}{q_{(\text{ult})u}} \right) \) for a bearing pressure of 200kPa

<table>
<thead>
<tr>
<th>Site</th>
<th>( q_{\text{net}} ) (kPa)</th>
<th>( s_u ) ( (s_u = 25N) ) (kPa)</th>
<th>( N_c )</th>
<th>( q_{(\text{ult})u} ) ( (q_{(\text{ult})u} = c_u N_c) ) (kPa)</th>
<th>( q_{\text{net}}/q_{(\text{ult})u} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>200</td>
<td>600</td>
<td>6.27</td>
<td>3762</td>
<td>0.053</td>
</tr>
<tr>
<td>B</td>
<td>200</td>
<td>1175</td>
<td>6.27</td>
<td>7367</td>
<td>0.027</td>
</tr>
<tr>
<td>C</td>
<td>200</td>
<td>175</td>
<td>6.27</td>
<td>1097</td>
<td>0.182</td>
</tr>
</tbody>
</table>

It was demonstrated in section 5.3 that values of \( E_i \) for weathered near-surface chalk are generally between 300 and 1000MPa. Clearly such a range does not warrant a log scale of stiffness when attempting to make correlations with SPT N value. The use of such a logarithmic relationship results in the stiffness being far too sensitive to small variations in SPT N value particularly when N is relatively large.

Fig. 5.4/4 which shows the initial moduli derived from the plate loading tests plotted against SPT N value. Also shown in Fig. 5.4/4 are stiffness data from loading tests reported by Powell et al (1990). In general these data show a trend which supports the linear relationship between SPT N value and \( E \) proposed by Stroud (1988).

Although the correlation proposed by Stroud (1988) generally produced tolerable predictions of plate settlement it must be pointed out that in general the use of correlations between SPT N value and stiffness can be misleading. The failure to predict the settlement of the plate at the Needham
Market site is a case in point. In many cases the SPT is unable to differentiate between unstructured and structured chalk when the structured chalk is weak and displays a tight fracture block system or where the structured chalk has moderate strength and a particularly loose fracture block system. It must be remembered that the relationships that are commonly used to derive stiffness from SPT N value are purely empirical and do not have any fundamental basis.

*Settlement Predictions using Visual Assessment of the Rock Mass*

The rock mass at each test site was described in detail. These descriptions were used to classify the rock mass according to the Grading system devised for the Mundford chalk by Ward et al (1968) (see Table 2.3/5).

In assessing the mass compressibility of the chalk at Mundford, Norfolk Ward et al (1968) recognised the importance of:

- Discontinuity spacing,
- Discontinuity aperture,
- Infill
and Hardness.

A five-part (later extended to six by Wakeling (1970)) classification scheme was devised based on visual inspection of the rock mass in large diameter boreholes and calibrated in terms of compressibility through a series of 865mm diameter plate loading tests. The details of the classification are given in Chapter 2. Although Ward et al. (1968) emphasised the site specific nature of the classification it has been applied almost universally since 1968 as a means of predicting the mass compressibility of chalk.

The grading system fails to recognise the importance of orientation of discontinuities in relation to that of the applied load and the looseness of the
fracture block system in relation to weathering style. This simply highlights the site specific nature of this engineering grade classification.

Difficulties arise in grading the chalk using this classification since in many cases the features described do not all fall into the same category. For example in many near surface weathered chalks the sub-horizontal discontinuities display spacings of between 10 and 60 mm (Grade IV) and apertures less than 3mm (Grade III). This situation commonly results in a Grade IV/III characterization. Here equal importance is being given to the spacing and the aperture. In reality the spacing may dominate the compressibility of the rock mass with the aperture playing a relatively minor role since it is small. Thus the chalk would be Grade IV.

The problems of classifying the chalk using the Mundford grading scheme are highlighted by the descriptions of the rock mass given in Chapter 3. The rock mass at each test site was described in detail before attempting to assign an overall grade. In order to examine the difficulties of grading the chalk, grades were initially assigned on the basis of single commonly-used identifiers (ie discontinuity spacing, aperture and infill) and these were combined to give an overall grade. Other identifiers such as hardness and weathering could not be used because no definitions are given by Ward et al. (1968) relating the identifiers to specific grades. Some bias was involved in this exercise since the rock mass was generally divided into zones during logging which were based loosely on the loggers perception of grade. The results of this exercise are shown in Chapter 3 (Figs. 3.1/9,10, 3.2/11 to 14, 3.3/12 to 14).

It will be seen from Chapter 3 that in most cases the spacing of sub-horizontal discontinuities falls into a single Mundford grade. In assigning grades based on spacing no account has been taken of the relative importance of discontinuities to mass compressibility. At site C (Needham Market) for example the sub-horizontal discontinuities were characterised by relatively tight irregular and inpersitent secondary fractures and wavy persistent primary bedding discontinuities with highly variable apertures. The
primary discontinuities are likely to contribute most to the mass compressibility. On the basis of these primary discontinuities alone the chalk at this site is mostly Grade II below a depth of 1m. However if all the sub-horizontal fractures are considered the chalk is typically Grade III.

The range of apertures seen in the sub-horizontal discontinuities at the test sites was such that a combination of two grades are frequently required (Grade III/IV or II/III, see Figs. 3.1/9,10, 3.2/11 to 14, 3.3/12 to 14) to describe this feature. This is largely due to the solution weathering of the primary bedding discontinuities. The problems of measuring aperture and the relevance of this to mass compressibility have been discussed in section 5.2.

Discontinuities close to the ground surface (ie < 1.5m) were commonly infilled with weathered debris and chalk fragments making grading straightforward. At site C (Needham Market) the secondary sub-horizontal fractures generally did not display any infill, whereas the primary bedding discontinuities were frequently partially infilled with putty chalk and rounded chalk fragments. Grading on the basis of infill at site C was complicated by the lack of definition concerning this feature in the Mundford grading scheme. It is not clear from the descriptions of the grades given by Ward et al. (1968) whether partial infill should be considered in the same way as complete infilling. In general the chalk has been graded as III, III/IV or IV on the basis of infill (see Figs. 3.1/9,10, 3.2/11 to 14, 3.3/12 to 14).

The different weathering styles seen at the three test sites generally gave rise to different combinations of grades based on individual identifiers making the assignment of an overall grade difficult. For example, based on discontinuity spacing the chalk may be Grade IV, but based on aperture and infill it is Grade III and Grade IV respectively. In such a case the overall grade for the rock mass would be IV/III. In general the when assigning an overall grade spacing was considered to be the most important factor. Aperture was considered to be of intermediate importance and infill the least important factor. A summary of the final gradings for each site are given in Fig. 5.4/5.
It will be seen from Fig. 5.4/5 that the mismatch of grades based on different identifiers leads to Grade IV/III and Grade III/IV being assigned for rock mass units at the test sites.

The correlation between Grade and $E_i$ suggested by Ward et al. (1968) for Mundford is given in Table 2.3/6. The chalk directly beneath the plate locations (i.e., to a depth of 2m below the plate) was generally between grades V and III. It was considered reasonable to take the minimum and maximum values of $E_i$ given in Table 2.3/6 for grade IV chalk as a lower and upper bound for the prediction of plate settlement at the test sites. At site A (North Ormsby) the upper bound for $E_i$ was taken as 2000 MPa to account for the fact that the chalk at this site was significantly harder than that at sites B and C. The settlement predictions were carried out for a bearing pressure of 200 kPa assuming a uniform modulus with depth. The results of these predictions are shown in terms of the ratio of observed to predicted settlements in Table 5.4/6 and Fig. 5.4/6.

It will be seen from Table 5.4/6 and Fig. 5.4/6 that the lower and upper bound moduli both result in the plate settlements being under-predicted. However in general the ratios of observed to predicted settlement were between 1 and 2 for both the lower and upper bound modulus values. Such predictions are generally acceptable and are comparable with the predictions based on the stiffnesses derived from the surface-wave seismic tests. This suggests that 500 and 1000 MPa may represent reasonable bounds for predicting foundation settlements on a wide range of structured near-surface weathered chalks.
Summary of Settlement Predictions

A summary of the settlement predictions based on Surface-wave geophysics, the Standard Penetration Test and visual assessment are shown in Fig. 5.4/7. In general predictions of settlement that are between one and two times the observed settlement are considered to be tolerable. These tolerance limits are shown on Fig. 5.4/7. It will be seen from Fig. 5.4/7 that the best predictions of plate settlement were based upon the surface-wave geophysics, Stroud's correlation between SPT N value and stiffness for \( \frac{q_{net}}{q_{ult}} < 0.15 \) (ie \( E = 15N \)) and visual assessment. Of these the surface-wave geophysics provided the most consistent results. Although the surface-wave geophysics does not give a direct measurement of stiffness, the determination of stiffness from the shear wave velocity is based upon fundamental principles, unlike the Standard Penetration Test and visual assessment which rely on empirical relationships to derive values of stiffness. The only case where the predictions were not acceptable was for the case in which the value of shear stiffness approached zero at the level of the plate. The lack of agreement between observed and predicted settlement in this case results from the model used and not from the measurements of stiffness.

The correlation between SPT N value and \( E_i \) based on Stroud (1988) gave good agreement between observed and predicted settlements except in the case of site C. The low SPT N values recorded at this site reflected the low strength of the rock material but the correlation failed to recognise the influence of dry density upon weathering style which gave rise to a tight fracture block system at this site.

The upper and lower bound stiffnesses based upon visual assessment gave good agreement between observed and predicted settlement. However it is clear from the discussion that the Mundford grading system, although recognising some important factors influencing mass compressibility, is too complicated to be applied objectively. Some identifiers of grade such as weathering and hardness are inadequately defined such that they cannot be
Table 5.4/6  Results of settlement predictions for a bearing pressure of 200kPa based on visual assessment using the Mundford grading scheme devised by Ward et al. (1968)

<table>
<thead>
<tr>
<th>Test No</th>
<th>Site A North Ormsby (E = 2000MPa)</th>
<th>Site B Leatherhead (E = 1000MPa)</th>
<th>Site C Needham Market (E = 1000MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.4430</td>
<td>1.9309</td>
<td>1.0845</td>
</tr>
<tr>
<td>2</td>
<td>5.9449</td>
<td>1.5041</td>
<td>1.4459</td>
</tr>
<tr>
<td>3</td>
<td>4.5238</td>
<td>1.5349</td>
<td>2.5847</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test No</th>
<th>Site A North Ormsby (E = 500MPa)</th>
<th>Site B Leatherhead (E = 500MPa)</th>
<th>Site C Needham Market (E = 500MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.7205</td>
<td>1.9309</td>
<td>1.0845</td>
</tr>
<tr>
<td>2</td>
<td>2.9713</td>
<td>1.5041</td>
<td>1.4459</td>
</tr>
<tr>
<td>3</td>
<td>2.2610</td>
<td>1.5349</td>
<td>2.5847</td>
</tr>
</tbody>
</table>

The worst predictions based on visual assessment were those for site A (North Ormsby) in which the upper bound stiffness was employed. The upper bound stiffness of 200 MPa was selected because the chalk at site A was harder than the chalks at the other test sites. The fact that the fracture block system at this site was loose was neglected since it is not considered in the Mundford grading scheme.
readily determined in the field or from a detailed description of the rock mass. This may stem from the fact that this classification was never intended to be used at sites other than Mundford. The site specific nature of the classification is highlighted by its failure to recognise the importance of looseness of the fracture block system and intact dry density upon the mass compressibility of chalk.

It should be pointed out that in practice the classification of the chalk normally takes place on the basis of SPT N value, the presence or absence of structure and discontinuity spacing. The classification of the chalk based on SPT N values is compared with that derived from visual description in Figs. 4.2/3, 7 and 9. It is clear that the SPT alone is unable to predict the observed grades accurately. The sensitivity of the SPT N value to drilling method in the more brittle chalks can lead to significant errors in the prediction of grade particularly when rotary drilling methods are employed. The practice of classifying the chalk using the Mundford grading system based on SPT N values alone is not recommended.

It is clear from the forgoing discussion that the Mundford grading scheme is unsuitable because :-

(i) It is too complex.
   It contains too many grades to be applied accurately and objectively.

(ii) It is site specific
    The classification was developed to enable stiffness parameters determined from plate loading tests and the tank loading test to be extrapolated over a large site at Mundford, Norfolk and was never intended to be used elsewhere.
(iii) It does not recognise some important facets of the chalk fracture-block system and the importance of intact dry density (or porosity) due to the site specific nature of the classification.

A simpler system that better reflects the insight into the mass compressibility of chalk based on this research and published data is given in Table 5.4/7. This classification is based upon the factors identified as being important in influencing mass compressibility in section 5.2. It is assumed that the discontinuities described in the classification are sub-perpendicular to the direction of applied loading (eg sub-horizontal in the case of a vertical foundation loading). Emphasis has been placed on structure, discontinuity spacing, aperture (to a limited extent), looseness of fracture block system and intact dry density. It is not intended that this classification should in any way be correlated with the Mundford grades, although a broad correlation may be possible.

Fig. 5.4/8 shows the classification of the chalk beneath the plate locations at each test site based on the scheme given in Table 5.4/7. It will be seen from Fig. 5.4/8 that the classification is more easily applied since there is no need to extend any of the classes in order to embrace the range of features observed.

Cost Effectiveness of the Methods of Settlement Prediction

It has been shown above that all the methods commonly used to predict settlements of foundations on weathered chalk provided estimates of plate settlement ranging from tolerable to totally unacceptable. The choice of method will depend upon the general reliability of the prediction it provides and the unit cost of the stiffness measurements.

In terms of reliability no single method stands out as being 100% reliable. However since the stiffnesses derived from the geophysics are based upon fundamental principles it seems reasonable to assume that these will be more
Table 5.4/7  Assessment of mass compressibility of chalk based on visual assessment.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Description</th>
<th>Compressibility characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Very low compressibility.</td>
</tr>
</tbody>
</table>
| A     | Structured Chalk: discontinuities more than 200mm apart, and closed. | $E_i = 1000 - 10000$ MPa  
$q_y > 1000$ kPa  
Intact dry density significantly affects compressibility. |
$E_i = 500 - 1500$ MPa  
$q_y = 150 - 420$ kPa, normally greater than 200 kPa.  
$E_y = 50$ MPa  
Intact dry density is likely to have a limited effect on compressibility. |
$E_i = 300 - 500$ MPa  
$q_y = 150 - 420$ kPa, normally greater than 200 kPa.  
$E_y = 50$ MPa  
Intact dry density has little effect on compressibility. |
| C     | Structureless Chalk: a melange of fines and intact chalk lumps, with no regular orientation of bedding or jointing. | Relatively high compressibility.  
$E_i = 100 - 300$ MPa  
$q_y$ unknown, but probably less than 200 kPa.  
Intact dry density has no effect on compressibility. Compressibility behaviour is likely to be effected by the relative proportions of fines and intact lumps of chalk. |
reliable than those derived from methods that make use of empirical relationships. This is particularly so when the database for the empirical relationships is very limited as it is in the case of the Standard Penetration Test and visual assessment. It should be remembered that the geophysical method measures stiffness at very small strains and hence relies on the stress-strain curve for the rock mass being linear. This may not be the case in structureless chalk or structured chalk of high porosity. In such cases the stiffness values derived from the geophysical measurements may require correcting for strain level to avoid the predicted settlements being significantly under-estimated. This is likely to involve some degree of empiricism which may reduce the reliability of the method.

The unit cost of stiffness measurements depends largely on the equipment and personnel required. The equipment and personnel required for each of the methods considered above are outlined in Table 5.4/8. It will be seen from Table 5.4/8 that surface-wave geophysics requires the least number of personnel since the method does not require a borehole or trial pit. However the equipment employed is generally expensive and is generally hired on a daily or weekly basis. The cost of hiring the equipment is offset by the speed at which measurements can be made. At most sites provided background noise is not coherent and the level of noise is relatively low a modulus-depth profile to at least 10m can be provided for at least 3 separate locations in one day's work. This would generally represent a total of about 90 stiffness measurements. The data for each test location is processed as the test proceeds and stored on a lap-top computer. This means that the modulus-depth profiles are prepared mainly on site and the data are in a form that may be used directly in a settlement prediction. The unit cost per measurement is about £5 excluding mobilisation costs.

The Standard Penetration Test requires a borehole and hence the cost of a drilling rig and drilling personnel should be considered. However it is likely that the borehole would be required for profiling and sampling as part of a normal site investigation programme and hence need not be considered as
special items required for the determination of stiffness. The SPT data from each borehole will require interpretation by an engineer before it is ready to be used in a settlement prediction which clearly adds to the unit cost of the measurement. In general about 10 to 15 Standard Penetration Tests can be carried out in a day since these are normally carried out in conjunction with sampling. The unit cost of an Standard Penetration Test is about £15.

Table 5.4/8  Logistics of in-situ measurements of stiffness

<table>
<thead>
<tr>
<th>Method</th>
<th>Equipment</th>
<th>Personnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface-wave geophysics</td>
<td>Spectrum analyser, vibrator, power oscillator, geophones, lap-top computer and generator.</td>
<td>Engineer</td>
</tr>
<tr>
<td>Standard Penetration Test</td>
<td>Drilling rig and SPT tools.</td>
<td>Driller and second man</td>
</tr>
<tr>
<td>Visual assessment</td>
<td>Digging machine (eg HiMac with 360° slew).</td>
<td>Engineer and digging machine operator</td>
</tr>
</tbody>
</table>

Visual assessment of weathered chalk is best carried in a trial pit. In general trial pits in the order of 5m deep will be required for this exercise. For health and safety reasons such deep pits must be properly supported before allowing anyone to log them. An engineer is required to carefully describe the rock mass exposed in the face of the trial pit for subsequent classification. A digging machine and operator are required to dig the pits. The maximum number of pits that can be dug and logged in a day is about 3. The detailed logs from each pit have to be interpreted and the rock mass classified back in the office in order to provide stiffness data for a settlement prediction. The unit cost of this exercise is about £200 per measurement assuming that each pit provides one stiffness measurement.

The unit cost of stiffness measurements are summarised in Table 5.4/9. It will be seen from Table 5.4/9 and the foregoing discussion on reliability that
the surface-wave geophysics appears to be the most cost effective method for settlement prediction. This method of investigation can also provide useful information about the variability of ground conditions which may result in savings in direct methods of ground investigation.

Visual assessment appears to be the least cost effective method. However it must be pointed out that there can be no substitute for the direct visual assessment of a rock mass. Although a trial pit may only provide a single stiffness measurement the wealth of useful information about ground conditions gained from careful and systematic description of the rock mass more than compensates for this.

**Table 5.4/9 Unit cost of in-situ stiffness measurements**

<table>
<thead>
<tr>
<th>Method</th>
<th>No. of measurements per day</th>
<th>Unit cost of measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface-wave geophysics</td>
<td>90</td>
<td>£5</td>
</tr>
<tr>
<td>Standard Penetration Test</td>
<td>10-15</td>
<td>£15</td>
</tr>
<tr>
<td>Visual assessment</td>
<td>3</td>
<td>£200</td>
</tr>
</tbody>
</table>

**Summary**

- Settlement predictions based on modulus-depth profiles derived from surface-wave geophysics were generally between one and two times the observed settlements. Plate settlements were generally under-predicted. However over-prediction of plate settlement by a factor of about 3 times occurred when the stiffness at the ground surface approached zero. This result was attributed to the model (ie Brown and Gibson, 1979) used to account for the increase in modulus with depth and not the values of stiffness measured.
The stiffnesses derived from Surface-wave geophysics are based on fundamental elastic theory and hence should be reasonably accurate for the strain levels involved. Provided the pre-yield stress-strain behaviour of the rock mass is not significantly non-linear there is no reason why settlement predictions made using this method should not show good agreement with observed settlements. The results of the unconfined compression tests on intact chalk and the plate loading tests indicate that the pre-yield stress-strain behaviour of chalk is not markedly non-linear, although the degree of non-linearity appears to increase with intact porosity.

Settlement predictions based on the Standard Penetration test and the empirical correlations with stiffness proposed by Wakeling (1970) and Kee and Clapham (1971) generally overestimated the observed plate settlements by as much as 100 times. The use of such correlations for predicting foundation settlement are likely to lead to costly overdesign.

Of the SPT-based methods of settlement prediction for chalk Stroud (1988) gave the best agreement with observed pre-yield plate settlement when the relationship $E = 15N$ was used. Ratios of observed to predicted settlements were generally 0.8 and 1.4 with the exception of site C (Needham Market) where the ratios were between 0.1 and 0.3. The lack of agreement at site C was due to the failure of the correlation to recognise that a tight fracture block system in a chalk of low intact strength gives rise to high pre-yield stiffnesses.

The Standard Penetration Test does not measure stiffness and hence the correlations proposed by various authors between SPT N value and $E$ have no fundamental basis whatsoever. The relationships are purely empirical and hence rely to a large extent on the scope of the database which forms the basis for the relationship. In general these databases are too limited to enable reliable settlement predictions to be made using the Standard Penetration Test.
Settlement predictions based on a classifying the rock mass by visual inspection according to the grading scheme developed for the Mundford chalk (Ward et al., 1968) generally showed good agreement with observed plate settlements.

It has been demonstrated that the Mundford grading scheme is too complex and does not reflect the current level of understanding of the mass compressibility behaviour of chalk. A simpler scheme based on the current state-of-the-art is proposed in Table 5.4/7.

It has been shown that the high cost of plate loading tests precludes the routine use of this method to determine stiffness parameters in chalk. In practice, for most shallow foundations on chalk settlements are predicted on the basis of stiffness parameters derived from the Standard Penetration Test. It has been shown that the SPT results can be misleading and may lead to overdesigned foundations. In terms of cost effectiveness surface-wave geophysics provides best means of predicting settlements on chalk. However it should not replace conventional means of investigation particularly the visual assessment of the rock mass.
Fig. 5.4/1: Ratio of observed to predicted settlement for bearing pressures of 100, 200 and 300 kPa based on continuous surface-wave seismic tests.
Fig. 5.4/2 Relationship between predicted settlement (normalised by the settlement assuming a Poisson's of 0.5) and Poisson's ratio derived from finite element analysis using CRISP90.
Fig. 5.4/3  Ratio of observed to predicted settlement for a bearing pressure of 200kPa based on empirical relationships between E and SPT 'N' value.
Fig. 5.4/4  Empirical and observed relationships between $E$ and SPT 'N' value.
Fig. 5.4/5  Visual classification of the chalk at each test site using the Grading scheme for Mundford (Ward et al., 1968).
Fig. 5.4/6  
Ratio of observed to predicted settlement for a bearing pressure of 200kPa based on visual assessment of the rock mass.
Summary of settlement predictions derived from geophysics, SPT and visual assessment.
Fig. 5.4/8  Classification of the chalk at sites A, B and C using the scheme proposed in Table 5.4/7.
5.5 Design of Plate Loading Tests

Of all the in-situ loading tests considered in Chapter 2, plate loading tests offer the best similitude with the full-scale foundation and enable the parameters $E_r$, $E_y$, $q_e$ and $q_y$ to be measured. However they suffer from several disadvantages. These include:-

(i) The rigidity and geometry of the plate results in different elements of ground beneath the plate following different stress paths. This results in problems extrapolating from the test scale to that of a full scale foundation.

(ii) Plate loading tests are generally very expensive.

The scale problem outlined in (i) may be overcome to a large extent by using a plate with a diameter at least 6 times the average sub-horizontal discontinuity spacing. This rule of thumb generally results in a minimum plate diameter of about 1000mm. Increasing the plate diameter, however, also increases the cost of performing the test in that higher loads are required. These large plate diameters generally preclude the use of kentledge as a means of applying load which means that tension piles must be used which adds more to the overall cost of the test. It is the cost of plate loading tests that generally limits their use to only the most sensitive structures.

The constant rate of penetration test described in chapter 2 would appear to be the cheapest form of plate loading test since the duration of the tests are generally only a few days. However the deformation rates used are so high that there is little or no component of creep and as a result the stiffnesses determined are generally too high will under-predict the settlement of the full-scale foundation. In general these tests are only really suitable for obtaining the ultimate bearing capacity of the rock mass.
Maintained incremental load tests permit creep effects to be studied and give more realistic values of stiffness and yield stress. However due to the greater duration required for this type of plate loading tests they are generally more expensive than the constant rate of penetration test outlined above. The tests carried out by the author each lasted between 12 and 18 days.

The typical cost of a plate test of the type carried out by the author lasting 12 days would be £14,000 for the first test inclusive of mobilisation and demobilisation and £10,500 for each additional test of the same duration at a different location at the same site. Nearly 40% of these costs are for personnel since a test requires two people to run the test and make daily measurements of plate settlement. The balance is taken up in the cost of the instrumentation, site preparation (construction of tension piles), preparation of the test location, hire of plant (cranes for mobilisation, erection, moving rig on site and demobilisation), hire of transport for mobilisation and demobilisation and hire of plate loading rig. The cost of carrying out plate loading tests at various depths in a large-diameter borehole would prove even more expensive since provision must be made for the cost of a piling rig to advance the hole together with the equipment necessary to extend the loading column and the extra instrumentation required for the measurement of plate settlement when the plate is located at some depth below ground surface.

It is clear from the above costs that plate loading tests can only be justified on projects valued at more than 2 million pounds. The high cost of large scale in-situ loading tests explains lack of published and unpublished data from such tests and the attraction of the Standard Penetration Test and visual assessment as a means of predicting foundation settlements. However it is this lack of data from large-scale loading tests that renders these empirical techniques unreliable.

The instrumentation used to measure plate settlement for the author's tests is described in Chapter 4. It was chosen on the basis of simplicity and reliability. These factors are of particular importance when performing tests
in remote areas with no access to a reliable electrical power supply over the total duration of a suite of three loading tests (which was typically about 36 days).

Precise levelling was employed for the measurement of absolute plate settlement. However although this method of measurement proved to be generally reliable and not prone to significant temperature effects it suffered from several disadvantages which are given below:

(i) It was difficult to make accurate measurements on windy days. This was due to severe vibrations set up in the pendulum mechanism of the level (see Fig. 4.1/12) and difficulties in holding the staff steady.

(ii) During the early stages of a test (ie pre-yield) the measured settlements were often close to the limit of resolution of the level. The table below lists the expected settlements for chalk with $E_i = 300$ and $1000$ MPa for a number of different bearing pressures.

<table>
<thead>
<tr>
<th>Bearing pressure (kPa)</th>
<th>Settlement (mm) of a 1.8m dia. plate for $E_i = 300$MPa</th>
<th>Settlement (mm) of a 1.8m dia. plate for $E_i = 1000$MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0.2200</td>
<td>0.0663</td>
</tr>
<tr>
<td>100</td>
<td>0.4400</td>
<td>0.1325</td>
</tr>
<tr>
<td>150</td>
<td>0.6600</td>
<td>0.1988</td>
</tr>
<tr>
<td>200</td>
<td>0.8800</td>
<td>0.2651</td>
</tr>
</tbody>
</table>

The analysis of the precise levelling data given in Chapter 4 suggests that movements greater than 0.1mm can be resolved with confidence. It is clear from the above table that this limit is approached for sites B and C (Leatherhead and Needham Market) since the initial modulus is relatively high at these sites. If a 1m diameter plate had been used the settlements would be nearly 50% of those shown above. In such a
case the pre-yield movements would be at or below the limit of resolution of the precise level.

Clearly if the initial modulus $E_i$ at the test sites had exceeded 1000MPa to any large degree this form of settlement measurement would have proved inadequate.

The settlement was measured at four points located symmetrically around the plate. These points were generally located at similar distances (700 to 800mm) from the centre of the plate. It was not possible to have measurement points closer than 500mm from the centre of the plate because of the stacked plate arrangement (see Fig. 4.1/2) and the proximity of the loading column. It was not possible therefore to measure the settlement at the centre of the plate.

This arrangement of measurement points on the plate permitted tilting to be monitored. In general plate tilt was found to be minimal up to yield (see Figs. 4.1/40 to 43). Beyond yield the plate experienced differential settlements of up to 10mm.

Dial gauges were used to supplement the precise levelling and provide time-deformation data for each loading stage. Six dial gauges were mounted in a symmetrical fashion over the exposed part of the plate. The dial gauges were generally positioned on the metal fins used to provide rigidity to the plate since these gave the greatest stability. The dial gauges were connected to a rectangular frame formed of scaffold bars (see Fig. 4.1/9). The frame was fixed to four datum posts located 2m away from the centre of the plate. This arrangement was chosen in order to minimise the length of the datum beams and hence avoid significant temperature effects. The penalty for this was that the dial gauges could not be used to measure absolute settlement. However based on the work of Jardine et al. (1985) the maximum ground deflexions take place within a radial distance of one plate diameter. Hence the datum posts were positioned at this radial distance from the centre of the plate. It
will be seen from Figs. 4.1/35 to 38 that the load-settlement curves for the
dial gauges and precise levelling are more or less parallel. Figs. 4.1/39 show
that for bearing pressures up to yield (q_e) the difference in average
settlement derived from precise levelling and the dial gauges is less than
0.3mm. At bearing pressures greater than the yield stress the difference
increases but remains generally less than 1mm. This demonstrates that choice
of datum post position was sound, and that the dial gauges provide
meaningful data. However the dial gauges cannot be used to determine
absolutely accurate stiffness parameters.

Readings of the dial gauges were taken at regular intervals during each
loading increment. However in order to study creep behaviour it would have
been better to have installed electrical linear displacement transducers such
that plate settlement could be logged continuously during periods of
maintained load. Such a system of measurement is more sensitive to
temperature effects than the dial gauges and furthermore it is likely that it
would be less reliable due to the need to provide a generator which might
break down or be prone to voltage fluctuations. The disadvantages of not
being able to continuously log the dial gauges were far outweighed by their
reliability in all weather conditions.

In this study no measurements were made of sub-plate deformations. Such
measurements have been used in previous studies of the load-deformation
behaviour of chalk in order to determine strain. These measurements of
strain could then be used with the appropriate stress for that depth
determined from elastic theory to calculate stiffness at various levels beneath
the plate. Such measurements have also been used to determine a yield locus
for the chalk (Burland and Bayliss, 1990).

The calculation of stiffness and yield based on sub-plate deformation
measurements assume that the distribution of stresses beneath the loaded
area can be predicted from elastic theory. Whereas this may be true for soil
there is evidence to suggest that it is not true for fractured rock (Gaziev and Erlikhman, 1977).

When considering whether or not to measure sub-plate deformations the extra expense of the instrumentation, the provision of a data logger and power supply and the logistics of installing the instrumentation were considered carefully against the usefulness of the data such measurements would provide. The fact that the stress distribution beneath the plate could not be predicted accurately from elastic theory meant the sub-plate deformation measurements were only of limited value. The measurements could be used to show which elements of the rock mass were contributing the most to the settlement of the plate and provide some useful data on creep behaviour. However the value of this information was outweighed by the problems of supplying and protecting a reliable power source at remote sites such as North Ormsby.

The diameter of the plate used by the author was 1800mm. The choice of plate diameter was based on the need to simulate as closely as possible the behaviour of a full-scale foundation. The diameter chosen represented a compromise between this requirement and the maximum plate diameter that could be used with BRE's 500 tonne capacity loading frame. When using a reaction frame that is fixed to the ground with tension piles it is important that there should be little or no interaction between the plate and the tension piles. BS 1377:Part 9:1990 suggests that it is normal practice for the centres of the tension piles should be at least three times the plate diameter from the centre of the plate. In the case of the tests carried out by the author the two rows of tension piles were positioned 4m from the centre of the plate which is only 2.2 plate diameters. However the recommendations for normal practice are based upon a stress distribution beneath the plate based upon elastic theory. The chalks tested all had open sub-vertical discontinuities suggesting that the rock mass is likely to behave as a series of columns with little transfer of horizontal stress at least at bearing pressures below the yield stress $q_y$. The stress distribution beneath the loaded area is likely not to
extend far beyond the diameter of the plate. Hence the likelihood of significant interaction between the piles and the loaded area is minimal.

When designing a plate loading test on chalk attention should be given to the following factors:-

(i) The choice of plate diameter.
The plate diameter should be at least five times the maximum subhorizontal discontinuity spacing. Typically in near surface weathered chalk a plate diameter of 865mm would represent the lower limit of suitability. Plates of smaller diameter are likely to give unreliable results when attempts are made to extrapolate to a full-scale foundation.

(ii) The preparation of the chalk surface
Mechanical excavation of pits auguring in boreholes severely disturbs the blocks of chalk, particularly if the fracture block system is loose. It is strongly recommended that the pits be excavated by hand. In pits and augured holes the area under the plate should be carefully prepared by removing by hand all loose chalk lumps, crumbs and putty chalk. The surface should be sealed with 25 to 50mm thick layer of blinding within a mould of the same diameter as the plate. The surface of the blinding should have a smooth finish. The plate is bedded on a layer of liquid plaster. The blinding and plaster bedding should be strong enough to withstand the proposed stress levels. These procedures are of critical importance in order to minimise bedding errors such that the initial (pre-yield) load-settlement behaviour can be accurately measured.
(iii) Loading increments

Loading increments should be chosen to give bearing pressure increments of not more than 50 kPa. This will enable the yield stress $q_y$ to be measured with reasonable accuracy. Once $E_y$ has been established the loading increment may be increased.

(iv) Duration of loading increments

Ideally each loading increment should be maintained until the creep rate is less than or equal to 0.3 mm/day. This may prove impractical particularly at bearing pressures greater than the yield stress $q_y$. In such cases a minimum duration of 24 hours is recommended.

(v) Maximum bearing pressure

The depends very much upon the objectives of the test. The ultimate bearing capacity of the chalk may range from 1.5 MPa to 8 MPa. Even with the lowest ultimate capacities recorded and a conventional factor of safety of 3, the minimum allowable bearing pressure would be 500 to 600 kPa. Such values are seldom applied by foundations. However it is necessary to establish the values of $q_e$ and $q_y$ since these will indicate the bearing pressures at which large settlements may be expected. When attempting to measure $q_y$ it is important that $E_y$ is fully established. This is particularly important in chalk of high porosity which may show collapse behaviour. In such cases a lower limit of 1000 kPa is recommended as a maximum bearing pressure.
(vi) Instrumentation

The following measurements are essential:

- Load applied to plate
- Settlement of plate
- Temperature

Sub-plate deformation measurements are valuable but are not considered to be essential until such time the stress distributions beneath the plate can be predicted with accuracy.

Since the test is likely to go on for several weeks it is of the upmost importance that the instrumentation should be reliable. It is recommended that the instrumentation kept as simple as possible. Load may be measured with a mechanical load cell (e.g., Macklow-Smith cantilever beam type load cell). Settlement may be measured using precise levelling if the plate is founded near the ground surface and the plate is greater than 1m in diameter. For plates with diameters less than 1m an optical levelling system with a resolution better than 0.05mm must be employed. For 865mm or 1000mm diameter plates it may be necessary to use dial gauges or other forms of high resolution linear displacement transducer with a suitably long datum beam which is protected as far as possible from temperature effects. At least three stable temporary bench marks should installed for the purpose of precise levelling. If the plate is positioned at depth then plate movements may be measured using a tensioned invar tape and a suitable datum beam. Details of such a system are described by Ward et al. (1968). If time-deformation measurements are required dial gauges may be used in the manner described in chapter 4. For both precise levelling and the use of dial gauges at least 4 measurement points on the plate is recommended. Temperature measurements should be made at the level of the plate and at the...
position of the jack and load cell. A simple digital thermometer may be employed.

Sub-plate deformation measurements may be made in the manner described by Marsland and Eason (1973).

(vii) Maintained load system

The most simple and effective means of maintaining the load on the plate constant is by the combination of a locking ring ships jack and a set of disc springs in the appropriate configuration. Details of this system are given in chapter 4. This system, although simple and reliable, is not able to maintain a constant load when the post-yield settlements are large. In such cases a semi-dynamic jacking system may be required. Such a system will provide a constant load but requires a hydraulic power pack to run it. This will result in a significant increase in the unit cost per test and may present problems with noise pollution when testing in urban areas.

(viii) Tension piles

With the recommended minimum plate diameter of 865mm it is likely that tension piles will be required as a reaction. Since little is known about the behaviour of tension piles in chalk it is recommended that prior to carrying out the plate test a set of piles of different lengths are constructed and subjected to pull out tests. This will enable the optimum length of pile to be chosen. The experience gained in this research suggests that a reaction suitable for a maximum plate load of 470 tonnes in a variety of chalks is provided by 18 No. 200mm diameter piles of 5m length each with a single 26.5mm diameter Dywidag threadbar. Details of the pile layout are given in chapter 4.
5.6 Design of Spread Foundations on Chalk

The ultimate bearing capacity of a shallow foundation on chalk may be estimated from:

\[ q_{ult} = \frac{\sigma_c}{2} N_c + p \]

Where:
- \( q_{ult} \) = ultimate bearing capacity
- \( \sigma_c \) = unconfined compressive strength
- \( N_c \) = bearing capacity factor
- \( p \) = overburden pressure at the level of the foundation

The bearing capacity factor \( N_c \) depends only on the shape and depth of the foundation and values are given by Skempton (1951). For a circular or square foundation at the ground surface the value of \( N_c \) is 6.15 and for a strip footing at the ground surface \( N_c \) is about 5.29. These values increase to as the depth of the foundation increases. Hence the ultimate bearing capacity is a minimum for a foundation located at the ground surface.

It has been shown in chapter 2 that the unconfined compressive strength of saturated chalk varies between 0.3 and 22MPa. Using the appropriate bearing capacity factors for a foundation located at the ground surface the ultimate bearing capacity of chalk is between 0.9 and 68MPa. The ultimate bearing pressure derived from plate loading tests on chalk (based on settlements between 10 and 15% of the plate diameter) varies between 1.3 and values in excess of 16MPa (Lake & Simons, 1970, Butler & Lord, 1970, Hodges, 1976 and Powell et al., 1990). Even with the lowest ultimate bearing pressure recorded and a conventional factor of safety of 3, the minimum allowable bearing capacity would be about 400kPa. Such high stresses are rarely used in practice.
It is clear from the above discussion that the bearing capacity of chalk is rarely a problem in foundation design on structured chalk. However, it may be a problem in structureless chalk. The major issue in the design of foundations on chalk is settlement since this controls the allowable bearing pressure.

The load-settlement behaviour of structured chalk may be adequately described using the simplified model proposed by Burland and Lord (1970). The principal load-settlement characteristics include at relatively low bearing pressures stiff more or less elastic behaviour followed by yield and a reduction in stiffness by about one order of magnitude. Clearly in order to limit settlements to an acceptably small level (eg < 10mm) it is necessary to distribute the foundation loading such that bearing pressure is less than the yield stress $q_y$.

The experience gained from the author’s experiments and published data suggest that the yield stress $q_y$ varies between 150kPa and 400kPa. The mechanisms involved in the yielding process are still not fully understood and the database from large diameter plate loading tests and full scale foundations is very limited. Hence it is not possible given the current state-of-the-art to make reliable predictions of $q_y$ based any test other than a plate loading test. Such tests are generally very expensive (see section 5.4) and can only be justified for very large projects. In general most structures are not likely to impose contact stresses greater than 200kPa. The stiffness of chalk at stresses less than 200kPa are generally such that the settlement of a circular foundation of 20m will not exceed 10mm.

Foundation settlements for bearing pressures up to 200kPa may be predicted with reasonable confidence from the following in-situ tests:
(i) Surface-wave geophysics
   This is likely to under-predict foundation settlements by a factor of up to 2 times. The accuracy of this method is reduced when the test is not carried out at the level of the foundation. Caution should be employed when using this method with published models for settlement prediction (e.g., Carrier and Christian, 1973, Brown and Gibson, 1979 and Rowe, 1981) in rock masses with a loose fracture-block system in which $G_o/mB_p$ is less than or equal to $10^4$. The use of such models in such cases are likely to give rise to settlements being overpredicted by a significant amount.

(ii) Visual assessment.
   This involves the careful description of the rock mass from an exposed face in a trial pit or shaft. In general the use of samples boreholes or drillholes alone is not adequate for the assessment of rock mass compressibility.

   This method generally gives reasonably accurate predictions of settlement and is very useful for carrying the results of plate loading tests across the site. The Mundford Grading scheme commonly used to classify the chalk in terms of mass compressibility has been found to be site specific and over complicated. A simpler and more general classification scheme based on our current state of knowledge is proposed in Table 5.4/7. The features of the rock mass which need to be described in detail are outlined in sections 5.3 and 5.4.

(iii) Standard Penetration Test using the correlation proposed by Stroud (1988).
   Using $E = 15N$ the SPT is likely to over-predict settlements by a factor up to 2 times. Caution should be employed in the interpretation of SPT results since in chalks with dry densities
greater than 1.50Mg/m$^3$ the SPT N value is sensitive to drilling method. It has been shown that light cable percussion drilling can give rise to SPT N values that are about half those obtained in rotary cored holes. It has been shown that settlement predictions based on the SPT are likely to be unreliable in chalks of high porosity which exhibit a tight fracture block system. In general it is not recommended that settlement predictions should be based on the SPT alone.

Of the above methods, surface-wave geophysics has been shown to be the most cost effective. However it is not as yet considered a routine or a conventional method of investigation and hence some engineers are likely to be sceptical about its use. The most commonly used method is the Standard Penetration Test. However if used alone the results can be misleading and may lead to grossly overdesigned foundations. This is particularly so if the more well-established correlations such as Wakeling (1970) or Kee and Clapham (1971) are employed. Although relatively expensive visual assessment of the rock mass is highly recommended since it permits the assessment of discontinuity geometry and looseness of the fracture block system which have been shown to be important factors in the mass compressibility of chalk.

If the bearing pressures cannot be limited to at least 200kPa the designer has the choice of determining the yield stress from a series of plate bearing tests or base the design on the post-yield stiffness. The high cost of plate loading tests must be considered against the cost of gross overdesign.

For most chalks the post-yield stiffness has been shown to vary between 44 and 68MPa. As a rule of thumb the post-yield stiffness $E_y$ may be estimated from $E_u/10$. However if the fracture block system is tight the value derived from this relationship may be too high. Low density chalks (ie dry density < 1.50Mg/m$^3$) are likely to exhibit collapse at stresses above $q_y$ and display
stiffnesses as low as 10MPa. By virtue of this phenomenon plate loading tests should be considered for any heavy structure founded on low density chalk.

Time-dependent settlement or creep is an important consideration although very little long term settlement data of foundations are available. On the basis of the available data for bearing pressures less than $q_e$ the long term settlements are of the of 2 to 5 times the short term settlement in weathered near surface chalk (Burland, 1975). For bearing pressures greater than $q_y$ the data from this research indicates that the long term (10 years) settlement is only 1.5 times the short term (24 hours) settlement. Intuitively the long term post-yield creep settlements should be greater than the pre-yield creep settlements. The fact that they are not in this case may arise from the differences in rock mass characteristics between the sites at which long term settlements were monitored. This highlights the limited amount of creep settlement data and the lack of understanding of the mechanisms causing time dependant deformations. In general if settlement measurements are based on plate loading tests in which loading stages have been maintained for 24 hour periods a conservative estimate of long term pre-yield settlement would be $P_{24}/3$.

It can be seen that much is known about the magnitude of $E_i$ and $E_y$ in structured chalk and hence pre and post-yield short term settlements may be predicted with reasonable confidence. With respect to yielding bounds may be placed upon the yield bearing pressure $q_y$. Unfortunately these bounds fall directly within the typical loading range of foundations. Clearly there is an urgent need to obtain more data for $q_e$ and $q_y$ from large scale loading tests and to gain a fundamental understanding of the mechanisms which control yielding within the rock mass.

Virtually nothing is known about creep in chalk. Only 2 long term settlement records for a loaded area of adequate size are known to exist. There are settlement records for plate loading tests with loads maintained for periods of
72 hours, but this is generally too short a period to facilitate an adequate study of creep.
Prior to this research the knowledge of the load-settlement behaviour of chalk was based on the published results of 19 small diameter (<865mm) plate loading tests and only 6 published case records dealing with load-settlement behaviour derived from large-scale loading tests and full-scale foundations. At first sight this database (see Chapter 2) looks reasonably comprehensive. However the small diameter plate loading tests generally yield unreliable results when attempts are made to scale the results up to that of a full scale foundation. The records of large-scale loading tests and foundation settlements are difficult to interpret in terms of behavioural patterns since the ground conditions are not always described in sufficient detail. In the case of full scale foundations there is little control over rate of loading and in some cases the contact stress distribution cannot be determined accurately using classical methods. The most significant contribution to our understanding of the load-settlement behaviour of chalk comes from one site at Mundford, Norfolk (Ward et al., 1968). This work demonstrated that fractured chalk undergoes yield at a certain stress resulting in a significant increase in compressibility and that the rock mass is prone to time-dependent settlements.

The object of this research is to improve the general understanding of the load-settlement behaviour of spread foundations on fractured chalk through a combination of field and laboratory techniques which provided data on both intact and mass properties of the chalk. The field techniques included the visual description of the rock mass, geophysics, penetration tests and large diameter plate tests. The laboratory techniques included the measurement of dry density, strength and stiffness.

At the time of starting this research it was known that factors such as fracture spacing and aperture played an important role in controlling the load-settlement behaviour of spread foundations. This concept was based largely upon the experience gained from in-situ loading tests carried out at
Mundford, Norfolk (Ward et al., 1968). Little attention was paid to the large variation in intact properties displayed by the chalk and the influence this might have on the engineering performance of the rock mass, particularly in the weathered profile. In order to carry out a systematic investigation of load-settlement behaviour it was necessary to locate a number of chalk sites at which the rock mass exhibits similar discontinuity characteristics but different intact properties. Three sites were selected. These were located at North Ormsby, Lincolnshire (Site A), Leatherhead, Surrey (Site B) and Needham Market, Suffolk (Site C).

The rock mass at the test sites was characterised initially from drill hole core or from surface exposures before any in-situ tests were carried out. The principal in-situ test used to investigate the load-settlement behaviour of the chalk was the plate loading test. This was chosen because it offers the best similitude with a spread foundation. In order to minimise problems of data interpretation associated with scale a plate diameter of 1.8m was used. This is the largest plate diameter ever used on chalk in the UK. Three maintained load tests were carried out at each test site.

The Standard Penetration Test is used routinely in practice for the prediction of foundation settlement on chalk. Hence it was necessary to carry out such tests at each test site. In addition to this surface-wave geophysics tests were carried out over each plate test location in order to produce a profile of shear modulus with depth. Although seismic methods are not generally used routinely they can provide valuable stiffness data. Upon completion of the plate loading tests the rock mass was described in detail. In most cases access to the rock mass directly beneath the plate locations was provided using deep trial pits. These observations allowed the rock mass to be classified using the Mundford Grading Scheme (Ward et al., 1968). In order to fully characterise the rock mass at each test site the intact physical and mechanical properties of the chalk were measured in the laboratory.
The effectiveness of the field tests was evaluated by using the results to predict the settlements measured by the plate loading tests.

The conclusions arising from this research may be considered in terms of:

(i) the literature review;
(ii) the laboratory studies;
(iii) the field studies;
(iv) implications for the design of spread foundations;
and (v) recommendations for further work.

6.1 Conclusions from Literature Review

The following conclusions may be drawn from the literature review:-

- Most white Cretaceous chalk displays a relatively uniform chemical composition (generally > 95% CaCO₃) together with a wide range of porosity (generally 9% ≤ n ≤ 52%).

- From the database of published mechanical properties of the chalk correlations have been established between porosity and strength and between porosity and stiffness. The wide range of porosity displayed by the chalk gives rise to compressive strength and stiffness values ranging between 0.3 to 22 MPa and 0.17 to 25 GPa respectively for saturated chalk. These mechanical properties are sensitive to moisture content. The magnitude of strength and stiffness for dry chalk is greater than that saturated chalk.

- The intact mechanical properties of the chalk are affected by the way in which the apparatus and the test specimen are configured. However porosity has such a profound influence on strength and stiffness that relationships are not masked by the differences in apparatus and test specimen configuration used by different authors.
Laboratory model studies show that the introduction of discontinuities in a rock mass increases the compressibility significantly.

In general a rock mass will deform by a combination of normal closure and shear displacement of discontinuities together with the deformation of the rock material. As a consequence the orientation of discontinuities with respect to the direction of the applied load will have a significant effect on the deformation behaviour of a rock mass.

The rock mass factors considered to be important in controlling compressibility include:-

- compressibility of the rock material
- discontinuity geometry
  - Number of sets
  - orientation (with respect to the direction of applied load)
  - spacing
  - aperture
  - surface topography
  - degree of contact
- discontinuity infill
- compressive strength of discontinuity walls
- shear strength of discontinuities

A system of sub-horizontal and sub-vertical discontinuities characterise the rock mass over the major part of the chalk outcrop (i.e. outside areas affected by intense tectonic activity). Hence the deformation of the rock mass subject to vertical foundation loads will be dominated by the normal closure of sub-horizontal discontinuities. The shear displacement of discontinuities is therefore of little concern in the mass compressibility of chalk and hence the shear strength of discontinuities need not be considered.
Typical weathering profiles in chalk are characterised by an increase in discontinuity spacing and a reduction in aperture with depth. The dominant weathering mechanisms are mechanical through stress relief and frost action and chemical through dissolution. This clearly highlights the importance of discontinuity spacing, aperture and degree of wall contact in controlling the mass compressibility of chalk.

The load-settlement behaviour of chalk has been observed mainly from the results of plate loading tests. In order to test a representative volume of the rock mass the plate diameter must be greater than five times the average sub-horizontal discontinuity spacing. In general the minimum conventional plate diameter that may be used on most chalk within the weathering profile is 865mm. Of the published records of plate loading tests 63% used plate diameters less than 865mm and 42% used plate diameters less than 600mm.

The interpretation of plate loading tests is complicated by the fact that the rigidity and geometry of the plate results in different elements of ground beneath the plate following different stress paths. This results in problems extrapolating from the test-scale to that of the full-scale foundation. The results of conventional plate loading tests may not yield reliable data for the studying the load-settlement behaviour of foundations.

The number of published load-settlement records for large-scale loading tests (ie plate dia > 1000mm) or full-scale foundations is extremely limited (6 records). With the exception of 3 records the information given concerning the rock mass is very limited. This combined with the limited number of records and differences in loading conditions does not permit a systematic and comprehensive study to be made of the mass compressibility behaviour of the chalk in general.
A single published record of a long term maintained load test on chalk (Ward et al. 1968) together with some shorter term maintained load test records (Kee, 1974 and Powell et al., 1990) indicate that significant settlements may occur at constant load as a result of creep. There is insufficient data to enable the mechanisms to be understood such that magnitude of creep settlements can be predicted for other structures.

The results of plate loading tests on chalk show a convex load-settlement curve. This behaviour has been observed for a wide range of plate diameters and for a full-scale foundation (Sugar silos at Bury St Edmunds). At a certain bearing pressure the compressibility of the rock mass increases significantly and settlements are not recovered upon unloading. This change in gradient of the load-settlement curve that gives rise to this characteristic shape is thought to be associated with yielding. However the mechanisms are not understood.

The yielding behaviour observed in the chalk is contrary to the behaviour predicted on from the normal closure of discontinuities based on laboratory model tests and in-situ loading tests for other rock types such as sandstone. The reasons for this difference in behaviour have never been considered.

The mass compressibility of chalk may be described using the idealised bi-linear representation of the characteristic load-settlement curve proposed by Burland and Lord (1970). This permits the load-settlement relationship to be modelled using four stiffness parameters:

\[ \begin{align*}
E_i & \quad \text{Initial modulus} \\
E_y & \quad \text{Post-yield modulus} \\
q_e & \quad \text{Yield bearing pressure (marking onset of yielding)} \\
q_y & \quad \text{Yield bearing pressure (on establishment of } E_y) \\
\end{align*} \]
Since plate loading tests are expensive a number of other methods may be used to determine the mass compressibility characteristics of chalk. These include:-

- Standard Penetration Test (SPT)
- Visual assessment
- Pressuremeter
- Geophysics

The methods are listed in order of popularity. In most cases only $E_i$ may be determined. Table 2.3/1 outlines the parameters that may be determined using each method.

- The SPT is by far the most popular method used to determine the stiffness of the rock mass. The method relies upon an empirical relationship between SPT N value and stiffness. There are a number of these relationships based on limited databases. There has been no published systematic study carried out which identifies the most reliable relationship. The results of the Standard Penetration Test are affected by drilling method and are often insensitive to changes in rock mass conditions such as discontinuity spacing and aperture. In general the use of the SPT alone is considered to be misleading.

- The use of visual assessment alone is little used. It is generally used in conjunction with the SPT but is rarely carried out directly on the exposed rock mass. The method make use of an engineering grade classification described by Ward et al. (1968). The classification recognises the importance of structure, discontinuity spacing and aperture and rock material hardness in controlling mass compressibility. However some of the identifiers for the classes are ill defined making the classification difficult to apply in some cases. The grade classification and the empirical correlations with stiffness were developed for the chalk at Mundford, Norfolk. Since this site
represents a unique stratigraphic, tectonic and geomorphological setting it is unsuitable for general use throughout the chalk outcrop. However engineers in the UK persist in applying it to all chalk sites.

- The pressuremeter is rarely used to determine the stiffness of the chalk. Although theoretically it provides the best method for determining stiffness parameters it suffers from a number of serious practical disadvantages. The principal disadvantages include loading the rock mass in a different direction to the foundation loading, mechanical disturbance of the test zone (Menard pressuremeter) and penetrating large flints (self boring pressuremeter).

- Geophysics has been rarely used to provide stiffness parameters in chalk.

The principal conclusions from the literature review are the fact that the mechanical properties of intact chalk can be related to porosity, the lack of published load-settlement data from large-scale loading tests and full-scale foundations, significant settlements can occur as a result of creep and the fact that the characteristic load-settlement curve for chalk shows yielding which is contrary to the behaviour predicted from the deformations dominated by the normal closure of fractures.

6.2 Conclusions from Laboratory Studies

The conclusions that can be drawn from the laboratory tests on intact chalk include:-

- Since most of the chalk (with the exception of the Lower Chalk) is monomineralic the porosity may be related to the dry density which is easier to measure.
The dry density of the chalk at each test site was relatively uniform in relation to the range of dry density for the fossil zones represented.

The overall range of dry density for the three test sites was from 1.30 to 1.93. This represents the typical range likely to be encountered in engineering works.

The intact strength of the chalk from the test sites shows good agreement with published data. The relationship between strength and porosity fits that established using published data.

The values of stiffness measured for the chalk from the test sites was generally greater than that published for chalk of a similar porosity. This lack of agreement is attributed to the method used for measurement of axial strain. It has been suggested that local strain measurements using Hall Effect gauges or electrolevel gauges provide a more reliable means of measuring stiffness in chalk.

The degree to which the chalk is saturated with water has been shown to influence both strength and stiffness. It has been shown that:

\[
\frac{\sigma_{c\, dry}}{\sigma_{c\, sat}} = 2.0
\]

\[
\frac{E_{dry}}{E_{sat}} = 1.2
\]

It has been shown that the ratio of \( \sigma_c : \sigma_i \) is 8:1

Most chalk behaves in a linear-elastic manner. However the linearity generally decreases with increasing porosity.
Chalks of intermediate and high porosity (ie >35%) display yield associated with the breakdown of the pore structure when compressed under conditions of uniaxial strain. The stress at which yield occurs increases with increasing dry density.

The post-yield load deformation behaviour is similar to that of uncemented soil, displaying a unique void ratio-log stress line typical of clay or sands at high stresses. The gradient of this line (compression index) is similar to that reported for uncemented carbonate sand. This line represents a lower bound for yield stresses.

6.3 Conclusions from Field Studies

A total of nine 1.8m diameter plate loading tests of the maintained load type were carried out up to bearing pressures between 1 and 1.8MPa were carried out at three test sites which exhibited similar discontinuity spacings and apertures but different fracture block system characteristics and different intact strength and stiffness. This is the first systematic study of load-settlement behaviour to be carried out on the chalk using a model foundation.

At test site A (North Ormsby) plate settlements were monitored over a period of 40 days whilst a bearing pressure of 900 kPa was maintained on the plate. The settlement at the end of this period was more than two times that after 24 hours, indicating that significant settlements are associated with creep. This test represents the only long term maintained load test under controlled loading conditions other than the tank loading test at Mundford, Norfolk (Ward et al., 1968) and the only recorded maintained load test carried out at a bearing pressure in excess of the yield stress $q_y$. 
The conclusions that may be drawn from the results of the plate loading tests include:-

- The plate loading tests carried out at the three test sites all displayed yield and the load-settlement behaviour could be described using the simplified model proposed by Burland and Lord (1970) which makes use of the parameters \( E_j, E_y, q_e \) and \( q_y \).

- The yielding behaviour of chalk appears to be related to a combination of limited contact area across discontinuities and the yielding behaviour of the intact rock. Solution weathering is largely responsible for reducing the contact area across major primary discontinuities. It is the local yielding of the asperities of contact which is thought to bring about yield in the rock mass.

- The range of initial modulus for weathered near-surface chalks is between 300 MPa and 1000 MPa based on published settlement data for large-scale loading tests and full-scale foundations together with the results of this research.

- The initial modulus \( E_j \) was often difficult to measure accurately from the load-settlement curve due to limited number of settlement measurements prior to yielding and the small settlements being close to the limit of resolution of the instrumentation.

- It was found that \( E_j \) could be determined with more confidence from a plot of secant modulus (\( E_q \)) against bearing pressure. Two types of curve were identified from the secant modulus plots. Type I displayed a relatively constant modulus up to the yield stress \( q_e \) indicating a relatively linear pre-yield load-settlement curve. Type II displayed two concave curves with a cusp coinciding with \( q_e \) as interpreted from the load-settlement curve. It would appear that the rock mass is undergoing some form of yield from a very early stage in the loading
test and that another mechanism causes the cusp in the secant modulus curve. In general the type II curves are associated with high initial stiffness. In the case of site C the initial stiffness approached that of the intact rock. It is thought that variations in contact area and looseness of the fracture block system are responsible for the two types of secant modulus curves.

The initial modulus $E_i$ appears to be inversely proportional to the intact dry density of the chalk. This is largely the result of the variation in looseness of the fracture block system which is brought about by the intact mechanical properties influencing the style of mechanical weathering. Chalks of high dry density ($\rho_d > 1.60 \text{ Mg/m}^3$) tend to be brittle and will support a loose fracture block system which gives rise to low values of $E_i$ (300 to 400 MPa). Chalks of low dry density ($\rho_d < 1.50 \text{ Mg/m}^3$) generally will not support a loose fracture block system and are characterised by high values of $E_i$ (600 to 1000MPa).

Rock mass factors for the three test sites ranged from 0.027 to 0.202 indicating the relative dominance of the discontinuities in controlling rock mass compressibility. The rock mass factors reduce as the fracture block system becomes more loose.

The post-yield stiffness $E_y$ is generally an order of magnitude lower than $E_i$ and has a much more limited range of values than $E_i$. The range of $E_y$ for large scale loading tests is generally between 40 and 70 MPa. This is based largely on the results of the author's tests since there is only one case record in the literature that covers post-yield behaviour at large or full scale. This case record is for the Bury St Edmunds Silos. The reliability of the post-yield settlement data is questioned on the grounds that the data have been interpreted on the basis of a uniform contact stress distribution, whereas both field
evidence and the results of a full soil-structure interaction analysis suggest that it is non-uniform.

- The yield bearing pressure $q_e$ has a limited range of between 200 and 600 kPa. A comparison of $q_e$ and the yield stress of chalk under conditions of uniaxial strain for the three test sites suggest that $q_e$ may be controlled by the contact area across sub-horizontal discontinuities. However the mechanisms causing yield are not sufficiently well understood to permit the yield stress to be predicted for chalk.

- The difference between $q_y$ and $q_e$ can give an indication of the degree of curvature of the load-settlement curve between the pre and post-yield portions.

- The yield bearing pressure $q_y$ is considered subjective since its magnitude depends on the length of the post-yield load-settlement curve.

- Large post-yield settlements at site C (Needham Market) caused the stiffness to reduce to about 10 MPa which is similar to that of a soil. These settlements are thought to be associated with pore structure collapse within the asperities of contact associated within the major bedding discontinuities.

- The parameter $R$ suggested by Burland and Lord is considered to be subjective since the log time-settlement curves are not always linear (particularly before and at yield) and the term immediate settlement is not defined. Furthermore the use of immediate settlement to normalise the gradient of the log time-settlement curve fails to recognise that creep is initiated the instant a loading increment is applied.
Plots of creep rate against log time for the plate loading tests indicate that the creep rate has reduced significantly 24 hours after a loading increment has been applied. This would appear to justify the choice of such a period between loading increments. However the creep rates at 24 hours were generally greater than the target of 0.29 mm per day suggested by Burland and Lord (1970).

Creep rate-log time curves display similar shape for different bearing pressures. Significantly higher creep rates are observed after the onset of yield. This gives rise to two distinct groups of creep rate curves based on whether the bearing stress is below or above \( q_e \). Upper and lower bounds for each group are shown in Fig. 5.3/7.

There is a trend of increasing creep rate at 24 hours with increasing bearing pressure. It is considered that since after yield the loading increments were applied at increasing creep rates that this may be largely responsible for the linear nature of the post-yield load-settlement curve.

Plots of log creep rate against log time indicate a linear relationship. The gradients of the lines for different bearing pressures are related to the yield stress. At bearing pressures below yield the lines have a common gradient of -0.57 whilst those at bearing pressures above yield have a common gradient of -0.97. Although the lines share a common gradient but the creep rate increases with bearing pressure within certain bounds.

The long term test at North Ormsby demonstrated that at stresses above \( q_e \) the that less than 50% of the settlement observed during the 40 day period occurred in the first 24 hours. Although the plate was still settling 40 days after the application of the loading increment the rate of settlement was only 0.0246 mm/day. The creep rate curve
shown in Fig. 4.1/49 indicates the rate was still reducing steadily towards the end of the test.

The rock mass at each test site was carefully described from 4 to 5m deep trial pits dug beneath the plate locations or from exposed rock faces.

Fig. 5.2/3 outlines those factors which are considered to be important either directly or indirectly to rock mass compressibility which can be readily assessed in the field. The most important features of a rock mass to describe for the purposes of assessing mass compressibility are:

(i) Discontinuity orientation
In most chalk outside areas affected by intense tectonic activity (see Chapter 2) the orientations will be sub-horizontal and sub-vertical.

(ii) Spacing
For foundations the spacing of the sub-horizontal discontinuities will be of paramount importance. If possible the fracture frequency of these discontinuities should be established to a depth of at least 2 times the breadth of the proposed foundation.

(iii) Looseness of the Fracture-Block System
The results of the plate loading tests have demonstrated that this factor has a strong influence on mass compressibility. It is the interaction of the intact mechanical properties and the weathering processes which gives rise to different weathering styles which are identified primarily on the basis of this factor and fracture spacing. It is likely that the different weathering styles result in different degrees of contact across fractures. Looseness of the fracture-block system may
assessed qualitatively using the definitions given in Table 5.2/1.

(iv) Hardness and dry density

The hardness of the intact rock may be assessed by handling the rock using Table 3.1/2. The dry density may be estimated using the relationship given in Chapter 5 (section 5.3). The strength and stiffness of the rock material may be found from the relationships established with dry density and porosity.

Very little can be deduced about discontinuity stiffness from infill and aperture. Infill is only really important when it totally fills the void space. In many cases it only partially fills the void space and hence the asperities of contact will still dominate the deformation process. It may be misleading simply to state that infill has been observed unless the extent to which the void space is filled is made clear. Aperture can give an indirect indication of contact area provided the maximum aperture and the lateral extent of the void space is described.

The stiffness of the rock mass at each test site was determined using the geophysics and the standard penetration tests in order to evaluate the effectiveness of these methods for predicting foundation settlement. The Mundford Grading scheme was applied to each site on the basis of the visual description of the rock mass. The correlations between Grade and stiffness developed for the Mundford site were used to predict the plate settlements. The conclusions which may be drawn from these tests and observations include:-

- Settlement predictions based on modulus-depth profiles derived from surface-wave geophysics were generally between one and two times the observed settlements. Plate settlements were generally under-predicted. However over-prediction of plate settlement by a factor of
about 3 times occurred when the ratio $G_{cr}/m$ is less than 1. This result was attributed to the model (ie Brown and Gibson, 1979) used to account for the increase in modulus with depth and not the values of stiffness measured.

The stiffness derived from Surface-wave geophysics is based on fundamental elastic theory and hence should be reasonably accurate for the strain levels involved. Provided the pre-yield stress-strain behaviour of the rock mass is not significantly non-linear there is no reason why settlement predictions made using this method should not show good agreement with observed settlements. The results of the unconfined compression tests on intact chalk and the plate loading tests indicate that the pre-yield stress-strain behaviour of chalk is not markedly non-linear, although the degree of non-linearity appears to increase with intact porosity.

Settlement predictions based on the Standard Penetration test and the empirical correlations with stiffness proposed by Wakeling (1970) and Kee and Clapham (1971) generally overestimated the observed plate settlements by as much as 100 times. The use of such correlations for predicting foundation settlement are likely to lead to costly overdesign.

Of the SPT based methods of settlement prediction for chalk Stroud (1988) gave the best agreement with observed pre-yield plate settlement when the relationship $E = 15N$ was used. Ratios of observed to predicted settlements were generally 0.8 and 1.4 with the exception of site C (Needham Market) where the ratios were between 0.1 and 0.3. The lack of agreement at site C was due to the failure of the correlation to recognise that a tight fracture block system in a chalk of low intact strength gives rise to high pre-yield stiffnesses.

The Standard Penetration Test does not measure stiffness and hence the correlations proposed by various authors between SPT N value and
E have no fundamental basis whatsoever. The relationships are purely empirical and hence rely to a large extent on the scope of the database which forms the basis for the relationship. In general these databases are too limited to enable reliable settlement predictions to be made using the Standard Penetration Test.

- Settlement predictions based on a classifying the rock mass by visual inspection according to the grading scheme developed for the Mundford chalk (Ward et al., 1968) generally showed good agreement with observed plate settlements.

- It has been demonstrated that the Mundford grading scheme is too complex and does not reflect the current level of understanding of the mass compressibility behaviour of chalk. A simpler scheme based on the current state-of-the-art is proposed in Table 5.4/7.

- In terms of cost effectiveness surface-wave geophysics provides best means of predicting settlements on chalk. However it should not replace conventional means of investigation particularly the visual assessment of the rock mass.

6.4 Implications for the Design of Spread Foundations

The design of spread foundations on chalk is based on minimising settlements since the strength of the rock mass is such that bearing capacity is not usually a problem. This research has demonstrated that the initial modulus of the rock mass may be predicted with reasonable confidence from surface-wave geophysics and visual assessment. However the range of yield stress $q_e$ is within the range of bearing pressures that may be imposed by spread foundations. This has a significant implication to the design of spread foundations since yielding within the rock mass results in a marked increase in compressibility. Unfortunately little is known about the mechanisms involved in yielding. This research presents limited evidence to suggest that
contact area may be an important factor affecting the yield stress but this is by no means conclusive. The current state-of-the-art is such that the yield stress can only be determined from large diameter plate loading tests which are generally prohibitively expensive to perform.

If the contact stresses are going to be large (ie q > 300kPa) the settlements may be predicted on the basis of the post-yield modulus $E_y$. This research has shown that in general $E_y$ is generally about a tenth of the initial modulus $E_i$. Although not accurate this may provide a reasonable estimate for most chalks with the exception of high porosity chalk (ie n > 45%, $\rho_d < 1.50 \text{ Mg/m}^3$) which may exhibit collapse. Such materials display a post-yield stiffness similar to that of a soil (ie about 10MPa).

An important issue for the design of spread foundations is the amount of creep settlement. time-settlement observations under constant load for plate tests carried out at Mundford, Norfolk and for those carried out as part of this research indicate creep settlement increases significantly after yield. There is insufficient long term settlement data to gain an sufficient understanding of the fundamental mechanisms involved to permit the prediction of creep settlements.

6.5 Recommendations for Further Work

Three areas where further work is required in order to improve our fundamental understanding of chalk mass compressibility behaviour have been identified. These include:

(i) Weathering style

Observations made of chalk in the mass indicate that the intact porosity may have a profound influence upon the style of weathering particularly in relation to the looseness of the fracture block system. The classification of weathering styles
may permit may aid the prediction of $E_i$ and ultimately may aid the prediction of yield stress and creep settlement.

(ii) Yield stress
The prediction of yield stress is an important issue in the design of spread foundations. It is considered that the yield stress is controlled by the properties of individual discontinuities which can be readily studied in the laboratory.

(iii) Creep settlement
There is an urgent need to gain a fundamental understanding of the mechanisms controlling creep settlement. This involves long term in-situ maintained loading tests as well as laboratory and numerical studies.
REFERENCES

ABBISS, C.P. (1979)  
A comparison of the stiffness of chalk at Mundford from a seismic survey and large-scale tank test. Geotechnique, 29, 461-468.

Shear wave measurements of the elasticity of the ground.  
Geotechnique, 31, 91-104.

ADDIS, M.A. (1987)  

AKAI, K., OHNISHI, Y. and TAKAHASHI, K. (1979)  

ANON (1977)  

A.S.T.M. D1194 1972  

A.S.T.M. 1586 (1974)  


A.S.T.M. Designation D3967-81  


AYDAN, O., and KAWAMOTO, T. (1990)  
Discontinuities and their effect on rock mass. Rock Joints, Barton & Stephansson (eds) Balkema, Rotterdam, 149-156.
The Pressuremeter and Foundation Engineering. Trans Tech Publ., Ohio.

BALLARD, R.F. and MCLEAN, F.G. (1975)  
Seismic field methods for in-situ moduli. Miscellaneous Papers S-75-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg.


BARTON, N. (1973)  

BARTON, N. (1986)  


Engineering classification of rock masses for the design of tunnel support. Rock Mech. 6, 189-239.


BATHURST, R.G.C. (1975)  

BERRIDGE, N.G. (1988)  

BELL, F.G. (1977)  
A note on the physical properties of the chalk. Engng Geol. 11, 217-225.


BROCH, E. and FRANKLIN, J.A. (1972)

BROOKS, N.J. (1983)


BROWN, P.T. and GIBSON, R.E. (1972)

BROWN, S.D., BIDDULPH, R.B. and WILCOX, P.D. (1964)


BS C.P. 2001 (1957)

BS 1377 (1975)

BS 1377 (1990)
BS 5930 (1981)
Code of Practice for Site Investigation (formerly C.P. 2001), British Standards Institution, London.

BUISMAN, A.S.K. (1936)
Results of long duration settlement tests. Proc. 1st Int. Conf. Soil Mech., 1, 100-106.

BURLAND, J.B. (1975)
Discussion on Session IV (Problems associated with Chalk). Proc. Conf. on Settlement of Structures, Cambridge, 746-748.

BURLAND, J.B. and BAYLISS, F.V.S. (1990)


BURLAND, J.B., HANCOCK, R.J. and MAY, J (1983)


BURLAND, J.B., SILLS, G.C. and GIBSON, R.E. (1973)
A field and theoretical study of the influence of nonhomogeneity on settlement. Foundation and Soil Technology, 3, 250-256.

BURSHTEIN, L.S. (1969)
BUTLER, F.G. and LORD, J.A (1970)

CARRIER, W.D. and CHRISTIAN, J.T. (1973)
Rigid circular plate resting on a non-homogeneous elastic half-space. Geotechnique, 23(1), 67-84.


CAWSEY, D.C. (1977)

CHAPPELL, B.A. (1979)

Classification of rock mass related to foundations. Proc. Int. Conf. on Structural Foundations on Rock, Sydney, Balkema, Rotterdam, 1, 29-36.

CHATWIN, C.P. (1961)


CLAYTON, C.R.I. (1978)
A note on the effects of density on the results of standard penetration tests in chalk. Geotechnique, 26, (1), 119-122.


CLAYTON, C.R.I. (1990a)

CLAYTON, C.R.I. (1990b)
SPT Energy transmission: theory measurement and significance. Ground Engineering, 23,
A new device for measuring local strains on triaxial specimens.  
Geotechnique, 36(4), 593-597.

Deformation, diagenesis and the mechanical behaviour of chalk. in  


COLBACK, P.S.B. and WIID, B.L. (1965)  
The influence of moisture content on the compressive strength of rock.  

COOK, N.G.W. (1992)  

COOP, M.R. (1990)  

COOPER, M.A.R. (1971)  

Gelifrction expérimentale des calcaires de la Campagne de Caen;  
comparison avec quelques dépôts périglaciaire de cette région. Bull. du Centre de Géomorphologie du CNRS, 6, 7-44.


CURREY, D.T. (1968)  
DENNEHY, J.P. (1975)
Correlating the SPT N value with chalk grade for some zones of the Upper Chalk. Geotechnique, 25, 610-614.

DEUMLICH, F. (1982)

DIN 4094 (1980)


DOBEREINER, L. and DE FREITAS, M.H. (1983)

DOBEREINER, L. and DE FREITAS, M.H. (1986)

DOBRIN, M.B. (1960)

DUNCAN, N. (1969)

DUNCAN, N. and HANCOCK, K.E. (1966)

DYKE, C.G. and DOBEREINER, L. (1991)

The use of a high capacity pressuremeter for design of foundations in medium strength rock. Proc. Int. Conf. on Structural Foundations in Rock, Sydney, 1, 9-16.

FARMER, I.W. (1968)
Engineering Properties of Rocks. Spon, London, 180pp
FOOKES, P.G. and DENNESS, B. (1969)

Limestone weathering; its engineering significance and a proposed classification scheme. Q. J. Eng. Geol., 21(1), 7-31.

FOOKES, P.G. and HORSWILL, P. (1970)


GAZIEV, E. and ERLIKHAM, S. (1971)
Stresses and strains in anisotropic foundations. Proc. Symp. on Rock Fracture, ISRM (Nancy), Paper II-I.

GENTIER, S., and RISS, J. (1990)

GIBSON, R.E. (1967)
Some results concerning displacements and stresses in a non-homogeneous elastic half-space. Geotechnique, 17(1), 58-67.

GILBERT, G.K. (1904)

GOODMAN, R.E. (1970)

GOODMAN, R.E. (1976)

GOODMAN, R.E., TAYLOR, R.L. and BREKKE, T.L. (1968)
The application of the seismic refraction techniques to the study of the 
fracturing of the Middle Chalk at Mundford, Norfolk. Geotechnique,  
23(2), 219-232.

GREENWOOD, J.A. and WILLIAMSON, J.B.P. (1966)  
Contact of nominally flat surfaces. Proc. R. Soc. Lond. A295, 

GROOM, G.E. and EDE, D.P. (1972)  
Laboratory simulations of limestone solution. Trans. of the Cave 
Research Group of Great Britain, 14, 89-95.

HAKAMI, E. and BARTON, N. (1990)  
Aperture measurements and flow experiments using transparent 
replicas of rock joints. Rock Joints, Barton & Stephansson (eds) 
Balkema, Rotterdam, 383-390.

HANCOCK, J.M. (1975)  

HANCOCK, P.L. (1969)  
80, 219-241.

HANCOCK, P.L. (1985)  
437-457.

HARDIN, B.O. and DRNEVICH, V.P. (1972)  
Shear modulus and damping in soils: design equations and curves.  

HARRIS, J.F., TAYLOR, G.L. and WALKER, J.L. (1960)  
Relation of deformational fractures in sedimentary rocks to regional 


HAWKINS, A.B. and MCCONNELL, B.J. (1992)  
Sensitivity of sandstone strength and deformability to changes in 
moisture content. Quarterly Journal of Engineering Geology, 25, 115- 
130.

HIGGINBOTTOM, I.E. (1966)  
The engineering geology of chalk. Proc. Symp. on Chalk in Earthworks 
HIGGINBOTTOM, I.E. (1971)
Superficial structures in reconnaissance and feasibility studies:
Discussion, Engineering Group Annual meeting, Dublin, July 1970. Q.

Engineering aspects of periglacial features in Britain. Q. J. Eng. Geol.,
3(2), 85-117.

Personal communication.

HILLIER, R.P. (1992)
The plate test on clay - A finite element study.
Phd Thesis, Department of Civil Engineering, University of Surrey.

HOBBS, N.B. (1970)
Discussion on Session A. Conf. on In situ Investigations in Soils and

HOBBS, N.B. (1973)
Effects of non-linearity on the prediction of settlements of foundations

HOBBS, N.B. (1975)
Factors affecting the prediction of settlement of structures on rock:
with particular reference to the Chalk and Trias. Review Paper,
Session IV, Proc. Conf. on Settlement of Structures, Cambridge,

HOBBS, N.B. and HEALY, P.R. (1979)

HODGES, W.G.H. (1976)
In situ tests in chalk: A report on plate bearing tests, dynamic and
static sounding in chalk, carried out at Whitchurch and Otterbourne,

HODGSON, R.A. (1961)
Classification of structures on joint surfaces. Am. Journ. Sci., 259,
493-502.

HOEK, E. (1968)

Rock Sope Engineering. 3rd Ed. Instn Min. & Metall., London,
358pp.

HUDSON, J.A. and MORGAN, J.M. (1975)
Compressive failure of chalk. TRRL Laboratory Report 681, Transport and Road Research Laboratory, Department of the Environment, Crowthorne, Berks.

HUNGR, O. and COATES, D.F. (1978)

HYETT, A.J. and HUDSON, J.A. (1990)

INGOLDBY, H.C. & PARSONS, A.W. (1973)
The classification of chalk for use as a fill material. Transport and Road Research Laboratory report LR806, TRRL, Crowthorne, Berks.

ISRM Commission, (1978)

ISRM Commission, (1979)

JARDINE, R.J., BROOKS, N.J. and SMITH, P.R. (1985)


KERENSKY, O.A. and LITTLE, G. (1964)  

KIERSCH, G.A. and ASCE, F. (1964)  


KJELLMAN, W., KALLSTENIUS, T. and LILJEDAHL, Y. (1955)  

KNILL, J.L. and JONES, K.S. (1965)  


KNUDSON, F.P. (1959)  

Arenito-Bauro: Determinacao de suas propiedadeo geotecnicas e applicacao ao projetao de um canal de grandes dimensoes. Anais do 3 Congresso da Abge, 3, 119-142.

KOWALSKI, W.C. (1966)  
The interdependence between the strength and voids ratio of limestone and marls in connection with their water saturating and anisotropy. Proc. 1st Congr. I.S.R.M., Lisbon, 143-144.

KROKOSKY, E.M and HISAK, A. (1968)  

KRONIEGER, R.R. (1990)  
Investigations into the engineering properties of chalk at Welford
Theale, Berkshire. Proc. Conf. on In situ Investigations in Soils and

LAKE, L.M. & SIMONS, N. E. (1975)
Some observations on the settlement of a four-storey building founded
in chalk at Basingstoke, Hampshire. Proc. Conf. on Settlement of
Structures, Cambridge, 283-291.

LAMA, R.D. (1978)
Influence of clay fillings on shear behaviour of joints. Proc. 3rd Int.

LAUTRIDOU, J.P. (1970)
Gélivité de la craie de Trancarville: le head de l’estuaire de la Seine.
Bull. du Centre de Géomorphologie du CNRS, 6, 45-62.

LEDDRA, M.J. (1990)
Deformation of chalk through compaction and flow. PhD Thesis,
University of London.

Compaction and shear deformation of a weakly-cemented, high
the Engineering Geology of Weak Rocks, Leeds, Balkema, Rotterdam.

LEROUEIL, S. AND VAUGHAN, P.R. (1990)
The general and congruent effects of structure in natural soils and

LEWIS, W.V. (1954)
Pressure release and glacial erosion. J. Glaciology, 2, 417-422.

Observation and analysis of ground deformation adjacent to a deep
road cutting in chalk. Proc. 10th European Conf. Soil Mech. and
Found. Engng, Florence, 2, 819-824.

The misuse of SPT N value correlations with Upper Chalk grades.
Geotechnique, 26, 217-220.

LORD, J.A. (1990)
Keynote Address: Design and Construction in Chalk. Int. Chalk
LORD, J.A. and DAVIES, J.A.G. (1979)  

MACCARINI, M. (1987)  

MAIR, R.J. and WOOD, D.M. (1987)  

MALLARD, D.J. (1977)  
Discussion on Session I, Chalk. Piles in Weak Rock, Institution of Civil Engineers, London, 177-180.

MANDELBROT, B.B. (1977)  
Fractals, Form, Chance and Dimension, Freeman, San Francisco.


MARSLAND, A. AND EASON, B.J. (1973)  


MASSON, M. (1973)  


MEIGH, A.C., SKIPP, B.O. and HOBBS, N.B. (1973)  
MILLER, G.F. and PURSEY, H. (1955)  

In-situ impulse test for dynamic shear modulus of soils. Proc. ASCE Special Conf. on in-situ measurement of soil properties, 1,

MIMRAN, Y. (1975)  
Fabric deformation induced in Cretaceous Chalks by tectonic stresses. Tectonophysics, 26, 309.

MONTAGUE, K.N. (1990)  


MORTIMORE, R.N. & FIELDING, P.M. (1990)  


MOYE, D.G. (1955)  


NICOLETTO, F. (1979)  
Silos on chalk: a reappraisal using the finite element technique. M.Sc. Dissertation, Department of Civil Engineering, University of Surrey.


OBERT, L., WINDES, S.L. and DUVALL, W.I. (1946)  
Standardised tests for determining the physical properties of mine rock. U.S. Bureau of Mines, Rept Invest., 3891.

PALMER, D.J. (1966)  

PALMER, D.J. (1976)  


PATTON, F.D. (1966)  

PEREIRA, J.P. (1990)  


POWELL, J.J.M. (1990)  

Engineering properties of Middle Chalk encountered in investigations
for roads near Luton, Bedfordshire. Int. Chalk Symposium, Brighton,

PRICE, N.J. (1966)
Fault and Joint Development in Brittle and Semi-Brittle Rocks.

Relationship between fracture spacing and bed thickness. J Struct.
Geol. 3, 179-183.

Discontinuity Analysis for Rock Engineering. Chapman and Hall,

The failure characteristics of selected British rocks. Transport and
Road Research Laboratory Report.

PYRAK-NOLTE, L.J., MYER, L.R., COOK, N.G.W. and WITHERSPoon,
P.A. (1987)
Hydraulic and mechanical properties of natural fractures in low
permeability rock. Proc. 6th Int. Congr. on Rock Mechanics (eds G.
Herget and S. Vongpaisal), Montreal, Canada. Balkema, Rotterdam.

PYRAK-NOLTE, L.J., NOLTE, D.D, MYER, L.R. and COOK, N.G.W.
(1990)
Fluid flow through single fractures. Rock Joints, Barton & Stephansson
(eds) Balkema, Rotterdam, 405-412.

RAVEN, K.G. and GALE, J.E. (1985)
Water flow in a natural rock fracture as a function of stress and
251-261.

Joint origin as a predictive tool for the estimation of geotechnical
Stephansson (eds) Balkema, Rotterdam, 91-96.

Beachy Head cave systems survey. Records, 9.

REUSS, A. (1929)
Berechnung der fleigrenzevon mischkristallen auf grand der
ROBERTS, J.C. (1961)  


RODIN, S (1966)  
Discussion on Session A. Proc. Symp. on Chalk in Earthworks and foundations. Institution of Civil Engineers, London. 74-75.


RIUZ, M.D. (1966)  

RUXTON, B.P. AND BERRY, L. (1957)  

Constant normal stiffness direct shear testing of chalk-concrete interfaces. PhD Thesis, Department of Civil Engineering, University of Surrey.

SCHILLER, K.K. (1958)  

SCHNEIDER, B. (1967)  

SCHNEIDER, H.J. (1976)  

SCHOLLE, P.A. (1977)

SCHOLLE, P.A. and KINSMAN, D.J.J. (1973)


Stress deflection and fracture development in a multidirectional extensional regime. Mathematical and Experimental Approach with Field Example. 2, 21-32.

SIRIEYS, P.M. (1966)

SKEMPTON, A.W. (1951)


STOKOE, K.H. and NAZARIAN, S. (1985)

STRAHAN, A. (1898)

STROUD, M.A. (1988)
The standard penetration tests - its application and interpretation. Proc. ICE Conf. on Penetration Testing in the UK, University of Birmingham, Thomas Telford.

SUN, Z., GERRARD, C. and STEPHANSSON, O. (1985)

SWAN, G. (1983)
SYLVESTER-BRADLEY, P.C. and FORD, F.H. (1968)
The geology of the East Midlands. Leicester Univ. Press.

Deformation characteristics of soils and rocks from field and laboratory tests. Report of the Institute of Industrial Science, The University of Tokyo, 37(1), No. 235.

TAYLOR, D.K. (1967)

 TOMLINSON, M.J. (1966)

TOMILINSON, M.J. (1980)

TOYNTON, R. (1983)


TRICART, J. (1956)

VAN EECKHOUT, E. M. (1976)


VARLEY, P.M. (1990)

VOIGT, W. (1928)


WAKELING, T.R.M. (1975)
Discussion on Session IV (Problems associated with Chalk). Proc. Conf. on Settlement of Structures, Cambridge, 748-750.


WARD, W.H., BURLAND, J.B. and GALLOIS, R.M. (1968)
Geotechnical assessment of a site at Mundford, Norfolk for a Proton Accelerator. Geotechnique, 18 (4), 399-431.

Elastic wave velocities in heterogeneous and porous media. Geophysics, 21, 41-70.

WILLIAMS, R.B.G. (1980)

WILLIAMS, R.B.G. (1987)

WOOD, C.J. and SMITH, E.G. (1978)

WOODLAND, A.W. (1970)


637
XU, S, and de FREITAS, M.H. (1990)

YOUNG, G.W. (1905)


ZIENKIEWICZ, O.C. and STAGG, K.G. (1965)
APPENDIX A

Drill-hole logs, trial pit logs, and face logs
Date 31/05/89  
Site: North Ormsby  
Location: Quarry face adjacent to and below test site (Face Log 1)  
Logged by CSR/CRIC  

<table>
<thead>
<tr>
<th>Depth</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 - 0.10</td>
<td>Top soil mixed with putty chalk.</td>
</tr>
<tr>
<td>0.10 - 0.44</td>
<td>Structureless white CHALK, just scrtatchable with thumb nail.</td>
</tr>
<tr>
<td></td>
<td>Sub-horizontal joints spacing 5 to 30mm, aperture 5mm, partially</td>
</tr>
<tr>
<td></td>
<td>infilled with weathered chalk fragments.</td>
</tr>
<tr>
<td></td>
<td>Sub-vertical joints spacing 5 to 30mm, aperture 10mm, partially</td>
</tr>
<tr>
<td></td>
<td>infilled with weathered chalk fragments.</td>
</tr>
<tr>
<td>0.44 - 0.80</td>
<td>Flaggy greyish white CHALK, just scrtatchable with thumb nail.</td>
</tr>
<tr>
<td></td>
<td>Sub-horizontal joints spacing 10 to 75mm (Av. 25mm), aperture 1mm,</td>
</tr>
<tr>
<td></td>
<td>some walls iron stained. Some joints infilled with clay and putty</td>
</tr>
<tr>
<td></td>
<td>chalk.</td>
</tr>
<tr>
<td></td>
<td>Sub-vertical joints spacing 15 to 150mm, aperture 1 to 4mm (Av. 1mm),</td>
</tr>
<tr>
<td></td>
<td>some walls iron stained. Some joints infilled with clay and putty</td>
</tr>
<tr>
<td>0.70</td>
<td>Large tabular black flint nodules (up to 300mm wide and 100mm thick).</td>
</tr>
<tr>
<td>0.80 - 1.50</td>
<td>Blocky greyish white stylolitic CHALK, just scrtatchable with thumb</td>
</tr>
<tr>
<td></td>
<td>nail. Sub-horizontal joints spacing 50 to 150mm, aperture 0 to 1mm,</td>
</tr>
<tr>
<td></td>
<td>some walls iron stained. Some joints infilled with clay and putty</td>
</tr>
<tr>
<td></td>
<td>chalk.</td>
</tr>
<tr>
<td></td>
<td>Sub-vertical joints spacing 60 to 200mm, aperture 0 to 1mm, some</td>
</tr>
<tr>
<td></td>
<td>walls iron stained. Some joints infilled with clay and putty chalk.</td>
</tr>
<tr>
<td>1.50 - 1.60</td>
<td>Discontinuous band of nodular black FLINT</td>
</tr>
<tr>
<td>1.60 - 2.32</td>
<td>Blocky greyish white stylolitic CHALK, just scrtatchable with thumb</td>
</tr>
<tr>
<td></td>
<td>nail. Sub-horizontal joints spacing 130 to 250mm, aperture 0 to 1mm.</td>
</tr>
<tr>
<td></td>
<td>Sub-vertical joints spacing 20 to 150mm (Av. 45mm), aperture 0 to</td>
</tr>
<tr>
<td></td>
<td>1mm.</td>
</tr>
<tr>
<td>1.88</td>
<td>Large <em>Inoceramus.</em></td>
</tr>
<tr>
<td>2.32 - 2.50</td>
<td>Black sheet FLINT up to 200mm thick.</td>
</tr>
<tr>
<td>Depth</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>--------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>2.50 - 4.00</td>
<td>Blocky greyish white stylolitic CHALK, just scratchable with thumb nail. Sub-horizontal joints spacing 50 to 75mm, aperture 0 to 1mm. Sub-vertical joints spacing 50 to 300mm, aperture 0 to 0.5mm.</td>
</tr>
<tr>
<td>4.00</td>
<td>Face obscured by talus.</td>
</tr>
</tbody>
</table>
Date 21/08/89  
Site: North Ormsby  
Location: Quarry face adjacent to test site (Face Log 2)  
Logged by MCM/CSR

<table>
<thead>
<tr>
<th>Depth</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 - 0.70</td>
<td>Structureless CHALK. Irregular coarse gravel size fragments (Av. 40mm) of chalk in a putty matrix (matrix supported). No preferred orientation of fragments.</td>
</tr>
</tbody>
</table>
| 0.70 - 1.17 | Flaggy greyish white stylolitic CHALK, just scratchable with thumb nail. Sub-horizontal joints spacing 20 to 50mm (Av. 35mm), aperture 2 - 6mm (Av. 3mm). Two sets of vertical joints:  
Set 1  90/200, spacing 100 to 240mm (Av. 150mm), aperture 0 to 10mm (Av. 5mm), infilled with iron stained putty chalk.  
Set 2  90/270, less persistent and less well defined as set 1. Spacing 60 to 100mm (Av. 70mm), aperture 0 to 5mm (Av. 2mm) infilled with iron stained putty chalk. |
| @ 0.80 | Band of cobble size flints. |
| @ 1.40 | Inpersistant band of putty chalk up to 20mm thick. |
| @ 1.50 | Band of scattered cobble to boulder size flints (650mm*250mm). |
| 1.17 - 1.840 | Flaggy greyish white stylolitic CHALK, just scratchable with thumb nail. Sub-horizontal joints spacing 20 to 50mm (Av. 35mm), aperture 1 - 2mm (Av. 1mm). Two sets of vertical joints as described above. |
| 1.84 - 3.52 | Flaggy to blocky greyish white stylolitic CHALK, just scratchable with thumb nail. Sub-horizontal joints spacing 90 to 150mm (Av. 100mm), aperture 2 - 5mm (Av. 3mm), walls very rough. Two sets of vertical joints:  
Set 1  90/220, spacing 100mm, aperture 2 to 10mm (Av. 3mm).  
Set 2  90/270, spacing 50 to 100mm (Av. 70mm), tight. |
| @ 2.05 | Band of scattered cobble to boulder size flints. |
| 3.52 - 3.74 | White and greyish black sheet flint 220mm thick with scattered chalk inclusions. |
3.74 - 4.390 Blocky greyish white stylolitic CHALK, just scratachable with thumb nail. Sub-horizontal joints spacing 60 to 220mm (Av. 80mm), aperture 0 - 5mm (Av. 3mm), walls very rough. Two sets of vertical joints:

Set 1 90/220, spacing 100mm, aperture 3 to 5mm.

Set 2 90/270, spacing 70 to 200mm (Av. 120mm), aperture 3 to 6mm.

Below 4.390 Face obscured by talus.
Date: 14/12/88  
Site: Esso HQ, Ermyn Way, Leatherhead  
Location: Trial Pit 1 (TPS1) Plate Loc. 1  
Logged by CRIC/RJH

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 - 0.25</td>
<td>Top soil, brown loam</td>
</tr>
<tr>
<td>0.25 - 0.57</td>
<td>Structureless CHALK with top soil. Intact lumps of chalk up to 25mm Dia. in a buff mixture of fine gravel and course sand size chalk fragments and clay rich top soil.</td>
</tr>
<tr>
<td>0.57 - 0.84</td>
<td>Structureless brownish white (buff) CHALK. Intact chalk up to 25mm Dia set in matrix of coarse sand to fine gravel size fragments of chalk (Av. 1 to 3mm), breaks in fingers but does not crush in fingers, voids open to 5mm (Av. 2 to 3mm), joints stained light brown.</td>
</tr>
<tr>
<td>0.84 - 1.20</td>
<td>Structured greyish white flaggy CHALK, easily broken with fingers but cannot crushed. Sub-horizontal discontinuity spacing 15 to 20mm, aperture 2 to 3mm, infilled with coarse sand size chalk fragments. Sub-vertical joints spacing 50 to 75mm, aperture 2 to 3mm, infilled with coarse sand size fragments of chalk, walls iron stained.</td>
</tr>
<tr>
<td>1.20 - 1.60</td>
<td>Structured greyish white flaggy/blocky CHALK, easily broken with fingers but cannot be crushed. Sub-horizontal discontinuities, spacing 20 to 50mm, aperture 0 to 1mm. Sub-vertical joints, spacing 50 to 100mm, aperture 0 to 1mm, walls iron stained and often covered with spots of manganese.</td>
</tr>
<tr>
<td>@ 1.43</td>
<td>Wavy sub-horizontal bedding plane, aperture 0 to 3mm, partially infilled with coarse sand to fine gravel size rounded fragments of chalk. Rounded nature of infill suggests transport by water.</td>
</tr>
<tr>
<td>1.60 - 2.00</td>
<td>Structured greyish white blocky CHALK, easily broken with fingers but cannot be crushed. Sub-horizontal discontinuities, spacing 100 to 150mm, aperture 0 to 0.5mm. Sub-vertical joints, spacing 50 to 150mm, occasionally 15 to 20mm aperture 0 to 1mm, walls lightly ironstained and often covered with spots of manganese.</td>
</tr>
<tr>
<td>@ 1.90</td>
<td>Discontinuous band of coarse gravel to cobble size FLINT.</td>
</tr>
</tbody>
</table>
2.00 - 2.86 Structured greyish white blocky CHALK, easily broken with fingers but cannot be crushed. Sub-horizontal bedding discontinuities, spacing 150 to 200mm, aperture 0 to 2mm. Intermediate sub-horizontal joints, spacing 50 to 75mm, aperture 0 to 0.5mm. Sub-vertical joints, spacing 50 to 200mm, aperture 0 to 1mm, walls lightly ironstained and often covered with spots of manganese.

@ 2.10 Wavy sub-horizontal bedding plane, aperture 0 to 5mm, partially infilled with coarse sand to fine gravel size fragments of chalk.

@ 2.34 Wavy sub-horizontal bedding plane, aperture 0 to 10mm, partially infilled with coarse sand to fine gravel size fragments of chalk.

@ 2.73 Wavy sub-horizontal bedding plane, aperture 0 to 10mm, partially infilled with coarse sand to fine gravel size angular to sub-rounded fragments of chalk.

@ 2.86 Wavy sub-horizontal bedding plane, aperture 0 to 10mm, partially infilled with coarse sand to fine gravel size angular to sub-rounded fragments of chalk.

2.86 - 4.50 Structured greyish white blocky/massive CHALK, easily broken with fingers but cannot be crushed. Sub-horizontal bedding discontinuities, spacing 150 to 200mm, aperture 0 to 0.5mm. Sub-vertical joints, spacing 50 to 200mm, aperture 0 to 1mm.

@ 2.90 Discontinuous band of gravel size FLINT

@ 3.24 Wavy sub-horizontal bedding plane, aperture 0 to 5mm, partially infilled with coarse sand to fine gravel size rounded fragments of chalk.

@ 3.50 Wavy sub-horizontal bedding plane, aperture 0 to 1mm.

@ 3.55 Wavy sub-horizontal bedding plane, aperture 0 to 1mm.

@ 3.90 Discontinuous band of cobble to boulder size FLINT

@ 4.35 Discontinuous band of cobble to boulder size FLINT

@ 4.50 BASE OF PIT
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 - 0.20</td>
<td>Top soil, brown loam</td>
</tr>
<tr>
<td>0.20 - 0.40</td>
<td>Structureless light brownish white CHALK with sub-angular to sub-rounded medium to coarse gravel size lumps of intact chalk in a light brown matrix of medium sand to fine gravel size fragments of intact chalk mixed with top soil.</td>
</tr>
<tr>
<td>0.40 - 0.73</td>
<td>Structureless light brownish white CHALK. Medium to coarse gravel size lumps of intact chalk with voids open up to 6mm Dia. in a matrix of coarse sand to fine gravel size fragments of chalk; intact lumps are coated with light brown clay; lump will break with fingers but cannot be crushed.</td>
</tr>
<tr>
<td>@ 0.70</td>
<td>Base of pit for plate test.</td>
</tr>
<tr>
<td>0.73 - 1.00</td>
<td>Structured light brownish white rubbly CHALK easily broken with fingers but cannot be crushed. Horizontal discontinuities, spacing 10 to 25mm, aperture 1 to 2mm, infilled with medium to coarse sand size fragments of intact chalk. Discontinuity walls iron stained.</td>
</tr>
<tr>
<td>1.00 - 1.80</td>
<td>Structured light brownish white flaggy CHALK easily broken with fingers but cannot be crushed. Horizontal discontinuities, spacing 10 to 50mm (Av. 25mm), aperture 1 to 5mm (Av. 2mm), walls iron stained, partially infilled with medium to coarse sand size fragments of intact chalk. Sub-vertical joints, spacing 50 to 100mm, aperture 0 to 2mm, wall ironstained or covered with spots of manganese.</td>
</tr>
<tr>
<td>@ 1.75</td>
<td>Persistent band of coarse gravel to cobble size nodular FLINT. Seen in all faces of pit.</td>
</tr>
</tbody>
</table>
| 1.80 - 2.23 | Structured white blocky CHALK easily broken with fingers but cannot be crushed. Horizontal discontinuities, spacing 100 to 150mm, aperture 0 to 3mm, partially infilled with coarse sand size fragments of intact chalk. Sub-vertical joints, spacing 50 to 100mm,
aperture 0 to 2mm, light iron staining and sparse manganese spotting on joint walls.

@ 2.00 Wavy sub-horizontal bedding plane, aperture 0 to 5mm, partially infilled with coarse sand to fine gravel size fragments of sub-rounded intact chalk.

2.23 - 2.80 Structured white blocky CHALK easily broken with fingers but cannot be crushed. Horizontal discontinuities, spacing 50 to 100mm, aperture 0 to 1mm. Sub-vertical joints, spacing 50 to 150mm, aperture 0 to 2mm, light iron staining and sparse manganese spotting on joint walls.

@ 2.72 Wavy sub-horizontal bedding plane, aperture 0 to 8mm, partially infilled with coarse sand to fine gravel size fragments of sub-rounded intact chalk.

2.80 - 3.90 Structured white blocky CHALK easily broken with fingers but cannot be crushed. Horizontal discontinuities, spacing 150 to 300mm, aperture 0 to 1mm. Sub-vertical joints, spacing 100 to 250mm, aperture 0 to 2mm.

@ 2.60 Wavy sub-horizontal bedding plane, aperture 0 to 8mm, partially infilled with fine to medium gravel size rounded fragments of intact chalk. Bottom surface in cavity areas (where aperture is large) is coated with a thin layer (1mm) of soft brown putty chalk.

@ 3.10 Wavy sub-horizontal bedding plane, aperture 0 to 8mm, partially infilled with fine to medium gravel size rounded fragments of intact chalk. Bottom surface in cavity areas (where aperture is large) is coated with a thin layer (1mm) of soft brown putty chalk.

@ 3.46 Wavy sub-horizontal bedding plane, aperture 0 to 8mm, partially infilled with fine to medium gravel size rounded fragments of intact chalk. Bottom surface in cavity areas (where aperture is large) is coated with a thin layer (1mm) of soft brown putty chalk.

@ 3.80 Persistent band of cobble size nodular FLINT.

3.90 - 5.20 Structured white massive CHALK easily broken with fingers but cannot be crushed. Horizontal and sub-vertical discontinuities, spacing 200 to 300mm, aperture generally tight.

@ 4.30 Wavy sub-horizontal bedding plane, aperture 0 to 5mm, partially infilled with coarse sand to fine gravel size fragments of intact chalk.

@ 5.20 BASE OF PIT
Date: 14/12/88  
Site: Esso HQ, Ermyn Way, Leatherhead  
Location: Trial Pit 3 (TPS3) Plate Loc. 3  
Logged by MCM/AVDB

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 - 0.23</td>
<td>Top soil, brown loam.</td>
</tr>
<tr>
<td>0.23 - 0.50</td>
<td>Structureless light brownish white CHALK with sub-angular to sub-rounded medium to coarse gravel size (10 to 40mm) lumps of intact chalk in a friable matrix of coarse sand to fine gravel size fragments of intact chalk mixed with top soil.</td>
</tr>
<tr>
<td>0.50 - 1.00</td>
<td>Structureless light brownish white rubbly CHALK, with intact lumps of chalk 25 to 50mm Dia. in a matrix of coarse sand to fine gravel size fragments of chalk and brown clay with voids open to 5 to 6mm. Intact lumps of chalk are easily broken with fingers but cannot be crushed. Intact lumps of chalk are generally coated with light brown clay.</td>
</tr>
<tr>
<td>@ 0.65</td>
<td>isolated pockets (40 to 60mm Dia) of friable fine to medium clayey sand.</td>
</tr>
<tr>
<td>@ 0.71</td>
<td>Base of pit for plate test.</td>
</tr>
<tr>
<td>1.00 - 1.20</td>
<td>Structured light brownish white flaggy CHALK broken with high finger pressure but cannot be crushed. Sub-horizontal discontinuities, spacing 10 to 25mm, aperture 1 to 3mm infilled with coarse sand size fragments of chalk. Sub-vertical joints, spacing 25 to 60mm, aperture 0 to 1mm.</td>
</tr>
<tr>
<td>1.20 - 2.00</td>
<td>Structured light brownish white flaggy/blocky CHALK easily broken with fingers but cannot be crushed. Sub-horizontal discontinuities, spacing 30 to 70mm, aperture 0 to 3mm. Sub-vertical joints, spacing 25 to 80mm (Av. 40mm), aperture 0 to 2mm [Av. 260/80], some manganese spotting on walls.</td>
</tr>
<tr>
<td>@ 1.53</td>
<td>Wavy sub-horizontal bedding plane, aperture 0 to 6mm partially infilled with fine gravel size fragments of chalk.</td>
</tr>
<tr>
<td>@ 1.90</td>
<td>Persistent band of coarse gravel to cobble size nodular FLINT. Seen in all faces of pit.</td>
</tr>
</tbody>
</table>
Structured white blocky/massive CHALK easily broken with fingers but cannot be crushed. Sub-horizontal discontinuities, spacing 100 to 300mm (more closely spaced between 2.66 and 2.86m), aperture 0 to 3mm. Sub-vertical joints, spacing 25 to 80mm (Av. 40mm), aperture 0 to 2mm (Av. 258/82). Sub-vertical joint [052/80] with slickensides forms face of pit between 2.00 and 3.58m. Similar slickensided joints are seen on the face opposite to that being logged.

@ 2.05
Wavy sub-horizontal bedding plane, aperture 0 to 6mm partially infilled with coarse sand to fine gravel size fragments of chalk. Bedding surface is lined with a thin (1mm thick) coating of light brown putty chalk.

@ 2.41
Wavy sub-horizontal bedding plane, aperture 0 to 6mm partially infilled with coarse sand to fine gravel size fragments of chalk. Bedding surface is lined with a thin (1mm thick) coating of light brown putty chalk.

@ 2.63
Wavy sub-horizontal bedding plane, aperture 0 to 6mm partially infilled with coarse sand to fine gravel size fragments of chalk. Bedding surface is lined with a thin (1mm thick) coating of light brown putty chalk.

@ 2.84
Wavy sub-horizontal bedding plane, aperture 0 to 6mm partially infilled with coarse sand to fine gravel size fragments of chalk. Bedding surface is lined with a thin (1mm thick) coating of light brown putty chalk.

@ 3.21
Wavy sub-horizontal bedding plane, aperture 0 to 6mm partially infilled with coarse sand to fine gravel size fragments of chalk. Bedding surface is lined with a thin (1mm thick) coating of light brown putty chalk.

@ 3.55
Wavy sub-horizontal bedding plane, aperture 0 to 6mm partially infilled with coarse sand to fine gravel size fragments of chalk. Bedding surface is lined with a thin (1mm thick) coating of light brown putty chalk.

3.90 - 4.04 Persistent band of coarse gravel to cobble size nodular FLINT with scattered pockets of limonite sand.

4.04 - 4.50 Structured white massive CHALK easily broken with fingers but cannot be crushed. Horizontal and sub-vertical discontinuities, spacing >200mm, aperture generally tight. Sub-vertical joints [050/80] show slickensides and manganese spotting.

@ 4.50 BASE OF PIT
**Date:** 27/7/88  
**Site:** Esso HQ Leatherhead  
**Location:** Main foundation excavation, South face.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 0.25</td>
<td>Top soil, brown loam</td>
</tr>
<tr>
<td>0.25 - 0.80</td>
<td>Rubbly structureless CHALK. Intact fragments sand size to 20mm. Voids visible up to 4mm dia.</td>
</tr>
<tr>
<td>@ 0.5</td>
<td>Flints. Small amount of Iron staining, particles sub-angular.</td>
</tr>
<tr>
<td>0.80 - 1.10</td>
<td>Structured flaggy CHALK, heavily iron stained, Horizontal joint spacing 15 to 20mm, vertical joint spacing 50 to 75mm, horizontal joints open between 1 and 3mm and partially infilled with coarse sand size chalk fragments.</td>
</tr>
</tbody>
</table>
| 1.10 - 1.60 | Structured flaggy CHALK, horizontal joint spacing 15 to 20mm, vertical joint spacing 30 to 40 mm, horizontal joint apertures tight to 1mm.  
Joint walls lightly iron stained. |
| @ 1.90 | Scattered flints. |
| 1.60 - 2.70 | Structured flaggy to blocky CHALK, Horizontal joint spacing 40 to 60mm, horizontal joint apertures tight to 0.2mm.  
Joint walls lightly iron stained. |
| @ 2.00 | Flints |
| 2.70 - 4.30 | Structured Blocky CHALK, horizontal joint (bedding planes) spacing 100 to 300mm, vertical joint spacing 100 to 150mm, horizontal joint aperture tight to 3mm. Aperture of bedding planes appears to be the result of solution weathering. |
| 4.30 | BASE OF EXCAVATION |
Date: 20/11/89
Site: Needham Chalks Ltd
Location: Trial Pit 1 (TPS1)
Logged by: MCM/CRIC

Max depth of pit: 4.50m Pit dry

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 - 0.72</td>
<td>Structured greyish white (columnar) CHALK, easily crushed with light finger pressure. Sub-horizontal joint spacing 30 to 60mm generally irregular, inperisstant and tight. Major bedding discontinuities 300mm apart, wavy, aperture 0 to 5mm, partially infilled with coarse sand to fine gravel size fragments of chalk. Persistent sub-vertical wavy joint (90/300). Inclined joints (75/064) spacing 5 to 40mm, aperture 0 to 5mm (generally 3mm). All joints walls stained with manganese and some with iron oxide. @ 0.15 Sub-horizontal wavy bedding plane, aperture 0 to 8mm, partially infilled with coarse sand size fragments of chalk. @ 0.30 Sub-horizontal wavy bedding plane, aperture 0 to 5mm, iron stained, partially infilled with coarse sand size fragments of chalk. @ 0.72 Sub-horizontal wavy bedding plane stained with iron oxide, aperture 0 to 15mm with evidence of water flow, partially infilled with rounded fine gravel size fragments of chalk.</td>
</tr>
<tr>
<td>0.72 - 1.30</td>
<td>Structured greyish white columnar/tabular CHALK, easily crushed with light to medium finger pressure. Sub-horizontal joint spacing 80 to 150mm (generally 100mm) inperisstant, irregular and generally tight. Vertical joints open to 2mm (Average 1mm). Inclined joints (68/128) spacing 30mm (parallel to face), tight, stained with manganese. Typical block size 100 * 40 * 40mm</td>
</tr>
<tr>
<td>@ 0.86</td>
<td>Wavy to very wavy bedding plane aperture 0 to 5mm partially infilled with coarse sand to fine gravel size chalk fragments.</td>
</tr>
<tr>
<td>@ 1.30</td>
<td>Wavy bedding plane, aperture 0 to 15mm with some evidence of water flow, partially infilled with fine gravel size fragments of chalk. Walls stained with iron oxide.</td>
</tr>
</tbody>
</table>

Typical block size 60 * 50 * 30mm

Typical block size 100 * 40 * 40mm
1.30 - 2.00 Structured greyish white rubbly to blocky CHALK, easily crushed with light to medium finger pressure. Sub-horizontal joint spacing 80 to 150mm inspersistent, irregular and tight. Vertical joints (90/274), spacing 70 to 150mm, wavy, aperture 0 to 1mm, walls stained with manganese.

Typical block size 140 * 60 * 65mm

@ 1.50 Wavy sub-horizontal bedding plane, aperture 0 to 2mm.

@ 1.68 Wavy sub-horizontal bedding plane, aperture 0 to 5mm. Some iron oxide staining.

@ 2.00 Wavy sub-horizontal bedding plane, aperture 0 to 10mm, partially infilled with coarse sand to fine gravel size rounded fragments of chalk.

2.00 - 2.50 Structured greyish white blocky CHALK, easily crushed with medium finger pressure. Vertical joint (90/274), stained with manganese, forms face of pit (extends from 2.00m to 2.50m). Vertical irregular fractures daylighting in face are less persistent and terminate in rock 250mm above the bedding plane at 2.50m. The spacing of these fractures is 30 to 40mm.

Typical block size 65 * 45 * 45mm

@ 2.50 Wavy sub-horizontal bedding plane, aperture 0 to 15mm, heavily iron stained in open sections, partially infilled with fine gravel size rounded fragments of chalk. The material below the open sections tends to be highly fractured.

2.50 - 3.40 Structured greyish white columnar CHALK, easily crushed with medium finger pressure becoming stronger below 2.80m (requires high finger pressure to crush). Sub-vertical wavy joint (70-90/100) forms face of pit, spacing 90 to 130mm, aperture 3 to 4mm, walls stained with manganese.

Many inspersistent, irregular, tight sub-horizontal and sub-vertical fractures daylight in face of pit.

Typical block size 150 * 65 * 55mm

@ 2.80 Highly irregular sub-horizontal bedding plane, aperture 0 to 25mm (generally 10 - 15mm), heavily iron stained and partially infilled with fine gravel size rounded fragments of chalk.

@ 3.10 Wavy sub-horizontal bedding plane, generally tight but locally open to 7mm with no infill.
3.40 - 4.00 Structured greyish white tabular CHALK, crushes to rubble with light finger pressure. Vertical joints (90/270), heavily iron stained, spacing 20 to 70mm, aperture 0 to 15mm (Av. 5mm).

Typical block size 190 * 84 * 50

@ 4.00 BASE OF PIT
General description of pit face as viewed from the top:

Regular bedded greyish white CHALK with no flints, dominated by two orthogonal vertical joint sets. Bedding planes are generally stained with iron oxide. Some vertical joints are also stained with iron oxide.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 - 0.97</td>
<td>Structured greyish white blocky/rubbly CHALK, easily crushed with medium finger pressure. Top 200mm heavily fractured. Vertical and inclined joints, 3 sets:</td>
</tr>
<tr>
<td>0.395</td>
<td>Wavy sub-horizontal bedding plane, aperture 0 to 5mm, iron stained, partially infilled with coarse sand size fragments of chalk.</td>
</tr>
<tr>
<td>0.76</td>
<td>Wavy sub-horizontal bedding plane, aperture 0 to 5mm, no infill.</td>
</tr>
<tr>
<td>0.97</td>
<td>Sub-horizontal planar bedding plane, aperture 0 to 12mm (Av. 1mm). Iron staining penetrates 20mm into walls, particularly the upper surface. Manganese staining in places almost completely covers surface with a black mat, particularly when the aperture approaches zero. Typical block size 70 * 55 * 45</td>
</tr>
<tr>
<td>0.97 - 1.48</td>
<td>Structured greyish white blocky/rubbly? CHALK, easily crushed with medium finger pressure. Sub-vertical joint (84/136) forms face of pit. Vertical joints (90/210) spacing 10 to 50mm (Av. 25mm), aperture 0 to 4mm (Av. 0.5mm), partially infilled with coarse sand size fragments of chalk.</td>
</tr>
</tbody>
</table>
Wavy sub-horizontal bedding plane, aperture 0 to 6mm, partially infilled with coarse sand size fragments of chalk. Walls iron stained. Little manganese staining.

Wavy sub-horizontal bedding plane, aperture 0 to 15mm (Av. 1mm) partially infilled with coarse sand to fine gravel size rounded fragments of chalk (evidence of water flow).

Wavy sub-horizontal bedding plane, aperture 0 to 30mm (Av. 1mm), partially infilled with fine sand to fine gravel size fragments of chalk. Open parts of bedding plane are characterised by manganese staining.

Typical block size 58 * 40 * 35mm

Structured greyish white CHALK, easily crushed with medium finger pressure. Vertical joint (90/140) with some iron and manganese staining forms face of pit. Minor vertical joint set (90/260), spacing 8 to 50mm (Av. 20mm), aperture 0 to 1mm with some manganese staining.

Wavy sub-horizontal bedding plane, aperture 0 to 20mm, partially infilled with fine gravel size fragments of chalk. Walls iron stained only in open sections.

Typical block size 90 * 60 * 40mm

Structured greyish white CHALK, crushed with hard finger pressure.

Two sets of vertical and inclined joints:

Set 1 Major persistent vertical joints (90/140), spacing 50 to 100mm (Av. 80mm), aperture 0 to 1mm (Av. 0.5mm) with no infill and some manganese and iron staining.

Set 2 Inclined joints (78/218), spacing 10 to 60mm (Av. 25mm), aperture 0 to 8mm (Av. 1mm), partially infilled with coarse sand size fragments of chalk. Walls stained with manganese.

Wavy sub-horizontal bedding plane, aperture 0 to 10mm, partially infilled with coarse sand to fine gravel size fragments of chalk. Walls stained with manganese and iron in open sections (Iron staining penetrates 10-12mm).

Wavy sub-horizontal bedding plane, aperture 0 to 40mm (Av. 4mm), partially infilled with coarse sand to fine gravel size
fragments of chalk. Large apertures tend to be associated with sub-vertical iron stained joints running from the upper wall.

**Typical block size 110 * 83 * 60mm**

**3.05 - 4.00** Structured greyish white CHALK, crushed with high finger pressure.

**Three joint sets:**

**Set 1** Inclined joints (80/142), spacing 100mm, aperture 0 to 2mm (Av. 0.5mm) with no infill. Walls stained with manganese and iron.

**Set 2** Inclined joints (68/096), spacing 20 to 70mm (Av. 60mm), aperture 0 to 3mm (Av. 2mm) with no infill. Walls stained with manganese and iron.

**Set 3** Inclined joints (82/060), spacing 4 to 40mm (Av. 25mm), aperture 0.5 to 2mm (Av. 1mm) partially infilled with coarse sand size fragments of chalk. Some Manganese spotting and iron staining.

[@ 3.33] Wavy sub-horizontal bedding plane, aperture 0.5 to 10mm (Av. 4mm) partially infilled with coarse sand to fine gravel size fragments of chalk. Walls iron stained, discoloration penetrates up to 10mm.

[@ 3.46] Fossiliferous stratum, thickness 30mm with abundant casts of bivalves and wavy. Shell structure no longer intact.

[@ 3.79] Wavy sub-horizontal bedding plane, aperture 0.5 to 10mm (Av. 4mm), partially infilled with coarse sand to fine gravel size fragments of chalk. Walls iron stained, discoloration penetrates up to 10mm.

**Typical block sizes 140 * 70 * 58**

[@ 4.00] **BASE OF PIT**
Date: 20/11/89  
Site: Needham Chalks Ltd  
Location: Trial Pit 3 (TPS3) Plate Loc. 3  
Logged by CRIC/CSR

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Details</th>
</tr>
</thead>
</table>
| 0.00 - 0.35 | Structured greyish white CHALK, crushes under medium finger pressure.  
Vertical joints, spacing 1 to 30mm (Av. <10mm), aperture 0 to 3mm with some manganese and iron staining.  
Horizontal joints, spacing 10 to 20mm, aperture 0 to 3mm.  
@ 0.30 | Sub-horizontal discontinuity, aperture 3 to 4mm infilled with light brown clay. Walls iron stained with discolouration penetrating up to 20mm.  
Typical block sizes 30 * 15 * 10mm. |
| 0.35 - 1.00 | Structured greyish white columnar/tabular CHALK, crushes under medium finger pressure.  
Inclined joints (82/270), spacing 5 to 30mm, aperture 0 to 5mm with no infill. Walls show manganese staining.  
@ 0.34 | Horizontal joint |
@ 0.65 | Horizontal joint, aperture 0 to 4mm, partially infilled with coarse sand size fragments of chalk. |
@ 0.96 | Horizontal joint  
Typical block size 110 * 74 * 43mm |
| 1.00 - 2.00 | Structured greyish white CHALK, crushes under medium finger pressure.  
Vertical joints spacing 2 to 60mm, aperture 0 to 4mm.  
@ 1.03 | Horizontal joint, heavily iron stained with a 20mm thick zone of shattered chalk (coarse sand to medium gravel) immediately above. |
@ 1.55 | Horizontal joint |
@ 1.78 | Horizontal joint |
@ 1.97 | Wavy sub-horizontal bedding plane, aperture 0 to 10mm, partially infilled with coarse sand size fragments of chalk. |
Block size highly variable 50-80*30-60*20-40mm

Typical size 60 * 45 * 30mm.

2.00 to 2.85 Structured greyish white CHALK, crushes under high finger pressure.

Vertical joints (90/275 & 90/185), spacing 10 to 30mm, aperture 0 to 3mm. Walls stained with manganese.

@ 2.23 Wavy sub-horizontal bedding plane, aperture 0 to 5mm with no infill.

@ 2.44 Wavy sub-horizontal bedding plane, aperture 0 to 4mm partially infilled with coarse sand to medium gravel size fragments of chalk.

@ 2.70 Wavy sub-horizontal bedding plane, aperture 0 to 25mm partially infilled with light brown silt together with sand to medium gravel size fragments of chalk.

@ 2.85 Wavy inclined joint (40/270 Av.), aperture 0 to 20mm partially infilled with light brown silt together with sand to medium gravel size fragments of chalk.

Typical block size 75 * 44 * 30mm

2.85 - 3.43 Structured greyish white CHALK, crushes under high finger pressure.

Vertical joints spacing 20 to 170mm (Av. 60mm), aperture 2mm

@ 3.25 Wavy sub-horizontal bedding plane, aperture 0 to 2mm with no infill.

@ 3.42 Wavy sub-horizontal bedding plane, aperture 0 to 20mm partially infilled with coarse sand to fine gravel size fragments of chalk.

Typical block size 140 * 80 * 67mm

3.43 - 4.00 Structured greyish white CHALK, crushes under high finger pressure.

Vertical joints (90/182), spacing 30 to 120mm, aperture 0 to 4mm, no infill.

Sub-vertical joints (82/090), spacing 60 to 100mm, aperture 0 to 2mm.
@ 3.57  Horizontal joint
@ 3.85  Horizontal joint
@ 4.00  Sub-horizontal joint, aperture 0 to 10mm.

Typical block size 145 * 100 * 84mm

4.00 - 5.50  Structured greyish white CHALK, crushes under high finger pressure.

Sub-vertical joints (80/078), spacing 100 to 180mm (Av. 100mm), aperture 0 to 10mm.

Inclined joints (66/214), spacing 100 to 180mm, aperture 0 to 10mm.

@ 5.49  Wavy sub-horizontal joint, aperture 0 to 3mm partially infilled with coarse sand size fragments of chalk.

@ 5.50  BASE OF PIT
### Description of Strata

**Depth (meters)**

<table>
<thead>
<tr>
<th>Depth</th>
<th>Symbolic Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.017</td>
<td><strong>Hard White Chalk Pebbles, Highly Fractured</strong></td>
</tr>
<tr>
<td>1.52</td>
<td><strong>Thick Black Flint Band</strong></td>
</tr>
<tr>
<td>3.3</td>
<td><strong>2.65 to 3.3 m - Hard White Chalk with Stylolites, Sub-horizontal Discontinuities 40-190 mm Spacing, Sub-vertical Discontinuities Present with Mg Flecking on Surfaces</strong></td>
</tr>
<tr>
<td>3.3</td>
<td><strong>3.3 - 4.3 m - Hard White Chalk with Stylolites and Flecked, sub-horizontal discontinuities 170-260 mm spacing, sub-vertical discontinuities almost absent. Where present Mg Flecking present on surfaces.</strong></td>
</tr>
</tbody>
</table>

**Key**
- D.B. - Diamond Bit
- T.T. - Tungsten Taper Bit
- N.C. - Nutty Crunch Bit

**Remarks**
- Poor recovery in first run due to Flint fragments eroding core in grade III Chalk.

**Logged By:**
- C.S. Russell

**Scale:**
- 1/25

**Client REF:**
- UOS 90
Test Site A: North Ormsby
Test 2

Bearing pressure (kPa)
- 200
- 400
- 600
- 800

$q_e = 250$ kPa

0.24 mm/day

Time (min)

Creep rate (mm/day)
Test Site C: Needham Mkt.
Test 2
Bearing pressure (kPa)
- 200
- 400
- 600
- 800
- 1000
- 1200

$q_e = 300 \text{kPa}$

0.24 mm/day

(a)

Test Site C: Needham Mkt.
Test 2
Bearing pressure (kPa)
- 200
- 400
- 600
- 800
- 1000
- 1200

$q_e = 300 \text{kPa}$

(b)
Test Site C: Needham Mkt.

Test 3

Bearing pressure (kPa)
- 200
- 400
- 600
- 800
- 1100
- 1400

$q_e = 200\text{kPa}$

0.24mm/day

(a)

Test Site C: Needham Mkt.

Test 3

Bearing pressure (kPa)
- 200
- 400
- 600
- 800
- 1100
- 1400

$q_e = 200\text{kPa}$

(b)