THE USE OF CONSOLIDOMETER TESTS TO ESTIMATE SETTLEMENT IN RESIDUAL SOILS

L'UTILISATION DES TESTS DE CONSOLIDATION POUR ESTIMER LE TASSEMENT DES SOLS RESIDUELS

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SYNOPSIS: An examination is made of the way in which results of conventional consolidation tests are interpreted and used in estimating settlements and the relevance or limitations of these methods when applied to residual soils. The use of the standard deformation versus log pressure plot for residual soils is questioned; results are presented which show that it can easily lead to mistaken interpretations of soil behaviour. It is suggested that linear plots are likely to give a truer picture of the compressibility characteristics of residual soils. The rapid rate of consolidation revealed in many laboratory tests on residual soils is examined and it is pointed out that it is not possible to obtain reliable c_v values from consolidation tests with many residual soils. Finally, the difficulty involved in making settlement predictions because of the uncertain pore water pressure state in the ground when the water table is deep is discussed.

INTRODUCTION

The laboratory consolidation test is still widely used for estimating the settlement of foundations on all types of soil. The way in which the results of such tests are interpreted and used for making settlement estimates is based primarily on the behaviour of sedimentary soils, with emphasis on the stress history concept which divides behaviour into normally consolidated and overconsolidated components. This paper presents the results of consolidation tests on a variety of residual soils, and questions the use of the sedimentary soil "model" for residual soils. Methods of estimating both magnitude and rate of settlement are examined, and the specific factors influencing such estimates with residual soils are identified and discussed.

STRESS DEFORMATION BEHAVIOUR

Log and Linear Plots

It is normal practice to plot results of conventional consolidation tests using a logarithmic scale for pressure. This practice stems from the stress history framework which forms the basis for interpreting the behaviour of sedimentary (transported) soils, and the approximately linear relationships which this plot produces between log stress and deformation, both for the virgin curve and the rebound curves. Whether this type of plot is the most appropriate for residual soils is open to question, as its use can easily lead to mistaken interpretations of the way in which the soil behaves.

Fig.1 shows the results of conventional consolidation tests on samples of residual soil from the Piedmont region in the southeastern United States. Fig.1(a) is taken directly from Barksdale et al (1982); the data has been replotted by the writer using a linear scale in Fig.1(b). On the basis of Fig.1(a) it is possible to interpret the behaviour in terms of normally and overconsolidated behaviour, and to estimate a preconsolidation pressure using the Casagrande construction. This approach has been used by the authors in order to make settlement estimates. Fig.1(b) however, does not indicate any increase in compressibility above an identifiable preconsolidation pressure. The compressibility remains the same or steadily decreases with increasing stress.

Fig.2 shows similar curves for a tropical red clay derived from volcanic material in Java, Indonesia. Although the log pressure curves suggest behaviour similar to that of an over consolidated soil, the linear plot shows the compressibility to be approximately linear with no suggestion of a preconsolidation pressure.

Fig.3 shows results of consolidation tests on samples of a clay or clayey silt soil derived from the weathering of a sandstone and mudstone formation ("Waitemata series") in Auckland, New Zealand.

In this case the existence of the "preconsolidation" pressure is real as the change in compressibility which occurs at this pressure is evident in both the log and linear plots. The "preconsolidation" pressure is in the vicinity of 200 to 250 kPa. It is important to recognise that this "preconsolidation" pressure is not the result of stress history affects; it is the result of the physical and chemical weathering process which has produced the soil. The terms "apparent" preconsolidation pressure or "pseudo" preconsolidation pressure are rightly used to designate this pressure. It is perhaps better to think of this pressure as a sort of yield pressure at which interparticle bonds of some sort start to break down with a subsequent increase in compressibility. The use of log and linear plots and the way in which the former can lead to mistaken interpretations has been pointed out on previous occasions (e.g. Wesley 1983, Vaughan 1985) but the profession appears to have been reluctant to abandon its almost universal use of the standard log plot.

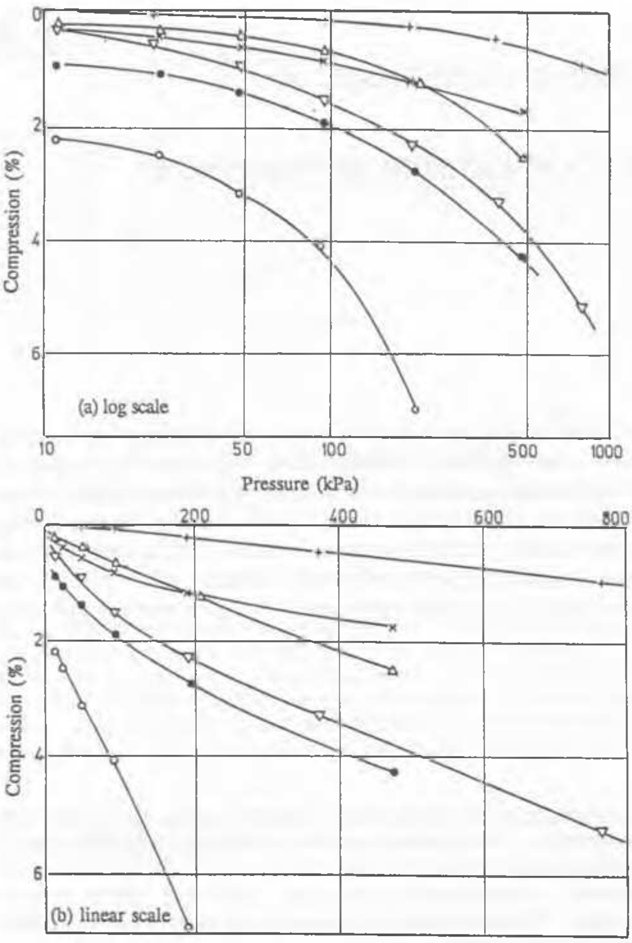

Fig. 1 Consolidation test results for Piedmont soil (after Barksdale et al (1982)

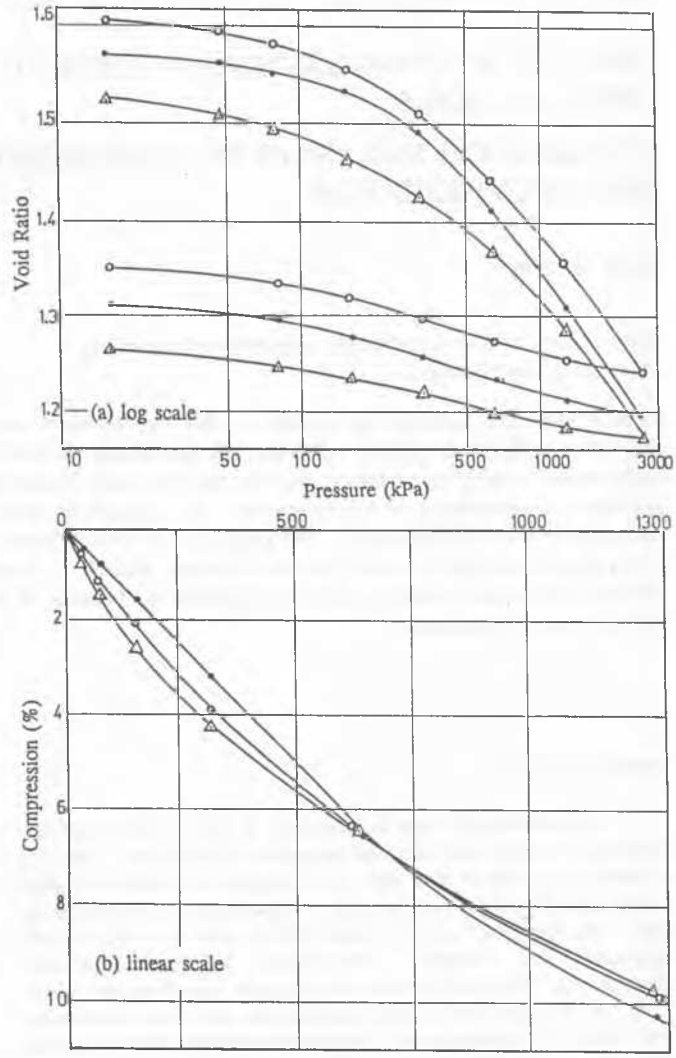

Fig. 2 Consolidation test results for tropical red clay

Compressibility Parameters

In order to make settlement estimates, it is normal to determine compressibility parameters from the consolidation test results. This can be done using either plot. From the log plot values of the compression index (C_c) and the swell index (C_s) can be determined. From the linear plot the coefficient of compressibility (m_v) can be determined. The choice of which of these parameters is more appropriate in a given situation should be made by carefully examining the test results after they have been plotted on both log and linear scales. For the type of behaviour observed in Figs 1 and 2 the use of m_v values would be the most appropriate. There is no basis for using C_c and C_s values as no "preconsolidation" pressure exists to separate stress levels to which these parameters are applicable. The m_v values must be selected for stress levels appropriate to the problem being analysed. For the behaviour illustrated in Fig.3, either set of parameters could be used, although care is needed in evaluating the parameter at the appropriate stress level.

Loading and Unloading Curves

If the parameters C_c and C_s are to be determined from the test result it is important to recognise that the unloading (or rebound) curves are not necessarily linear (on the log scale) and their inclination is not necessarily independent of stress level. Fig.4 shows a consolidation test result on a clay derived from the Weathering of Waitemata sandstone. An unloading and reloading loop was followed when the initial stress application had reached 216 kPa. The stress was then reduced again after the application of the peak stress of 1728 kPa. It is immediately apparent that the slope of the final unloading curve is much steeper than the loop obtained by unloading at 216 kPa. In other words the C_s values for these two unloading curves are quite different, the latter being over double the value of the former. This same effect is evident in Fig.3(a) where the slope of some of the final rebound curves is steeper than the initial loading curves.

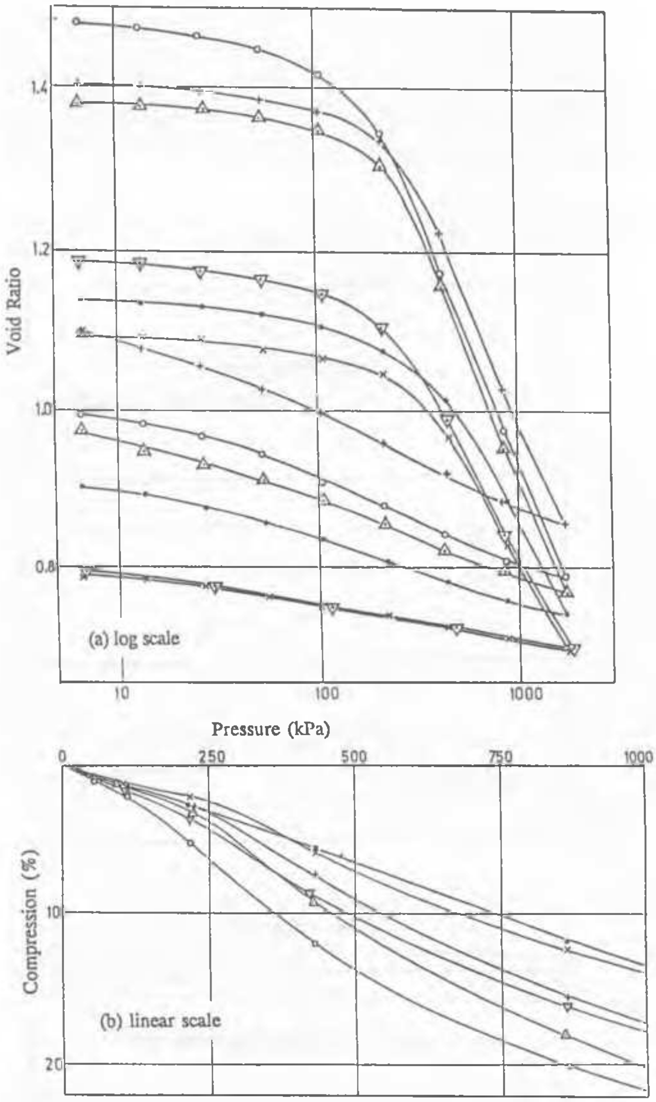

Fig. 3 Consolidation test results for Auckland clay

Corrections for Disturbance

Various attempts have been made to apply corrections to consolidation test results to take account of sample disturbance; the best known and most widely used appears to be that of Schmertman (1953). This method makes use of the observation that unloading and reloading curves all have the same gradient (corresponding to a constant C_s value), and that if the soil is overconsolidated then initial compression from the in situ stress level will follow this constant gradient. The swell index C_s is therefore used to estimate compression up to the preconsolidation pressure. The behaviour illustrated in Figs 1 to 4 shows that this concept has no general applicability to residual soils, firstly because there is no basis for assuming that the compressibility of residual soil can be divided into a normally consolidated component and an overconsolidated component and secondly because there is no reason to believe that the slope of the initial part of the compression curve should be parallel to later unloading and reloading curves.

The soil sample tested in Fig.4 was from a shallow depth where the in situ effective stress was in the region of 20 kPa. It is clear from the form of the curve that the use of a C_s value taken from the final unloading curve would result in a large overestimate of compression, at least for pressure increments up to about 100 kPa. It is not suggested here that the behaviour shown in Fig.4 is necessarily a typical of residual soil behaviour. it is simply one of many possible modes of behaviour, and it would be unwise to expect residual soils to conform to any tidy behaviour "models".

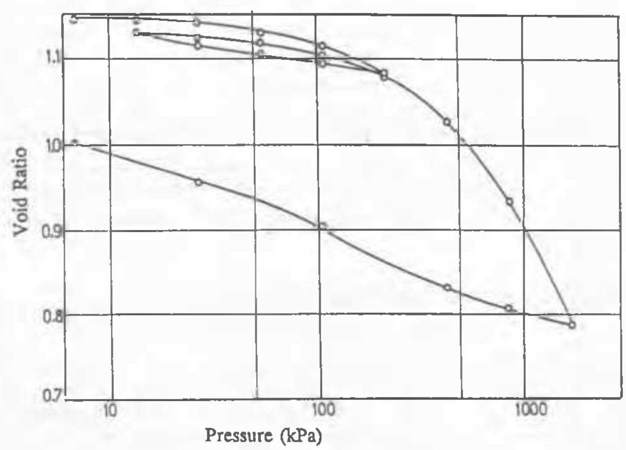

Fig. 4 Unloading and reloading cycle for Auckland clay

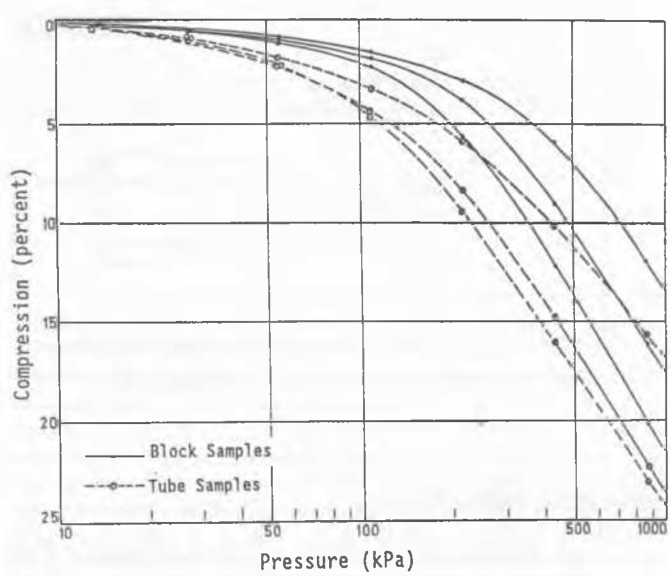

Fig. 5 Consolidation test curves from tube and block samples of Auckland clay

In the absence of any reliable methods for estimating the influence of disturbance on consolidometer test results with residual soils, or of correcting for such disturbance, it becomes imperative that undisturbed samples used for testing be as free from disturbance as possible. There is a strong case to be made for more frequent use of block samples, taken from relatively shallow surface pits. In many foundation situations, a large proportion of the settlement originates from compression of the upper soil layers close to the surface. Hence block samples from surface pits are the best hope of getting reliable consolidometer information. Fig.5 shows the results of consolidometer tests on tube samples from boreholes and on block samples of Auckland clay, taken from neighbouring sites. The compressibility of the block samples is only about half that of the tube samples, at least in the stress range of greatest interest, namely 0 to 200 kPa. This difference is believed to be due to the reduced disturbance of the block samples.

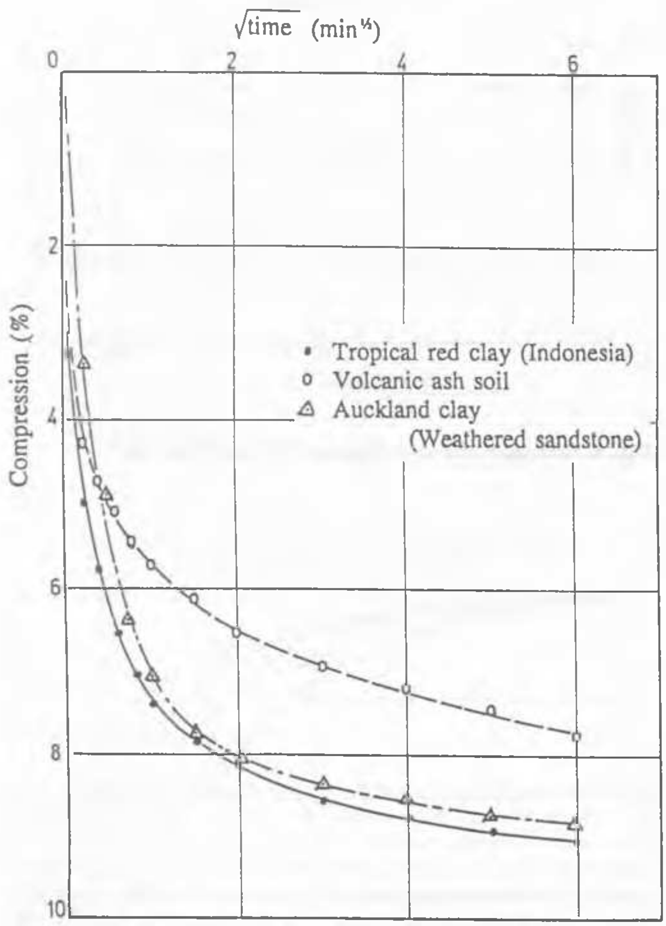

Fig. 6 Square root of time curves for residual soil

TIME RATE BEHAVIOUR

The coefficient of consolidation c_v is normally obtained from the consolidation test result by using the Taylor square root of time method or the Casagrande log time method. Fig.6 shows square root of time plots from standard consolidation tests on three residual soils. The sample thickness used in these tests was 19 mm (0.75in) and the curves are for loading increments from 108 kPa to 216 kPa. Two important factors are evident; firstly the rate of consolidation is very rapid and secondly the straight line portion of the curves is poorly established. In the authors experience, the behaviour demonstrated in Fig.6 is typical of many residual soils. It is thus difficult to extract reliable c_v values from normal consolidation tests.

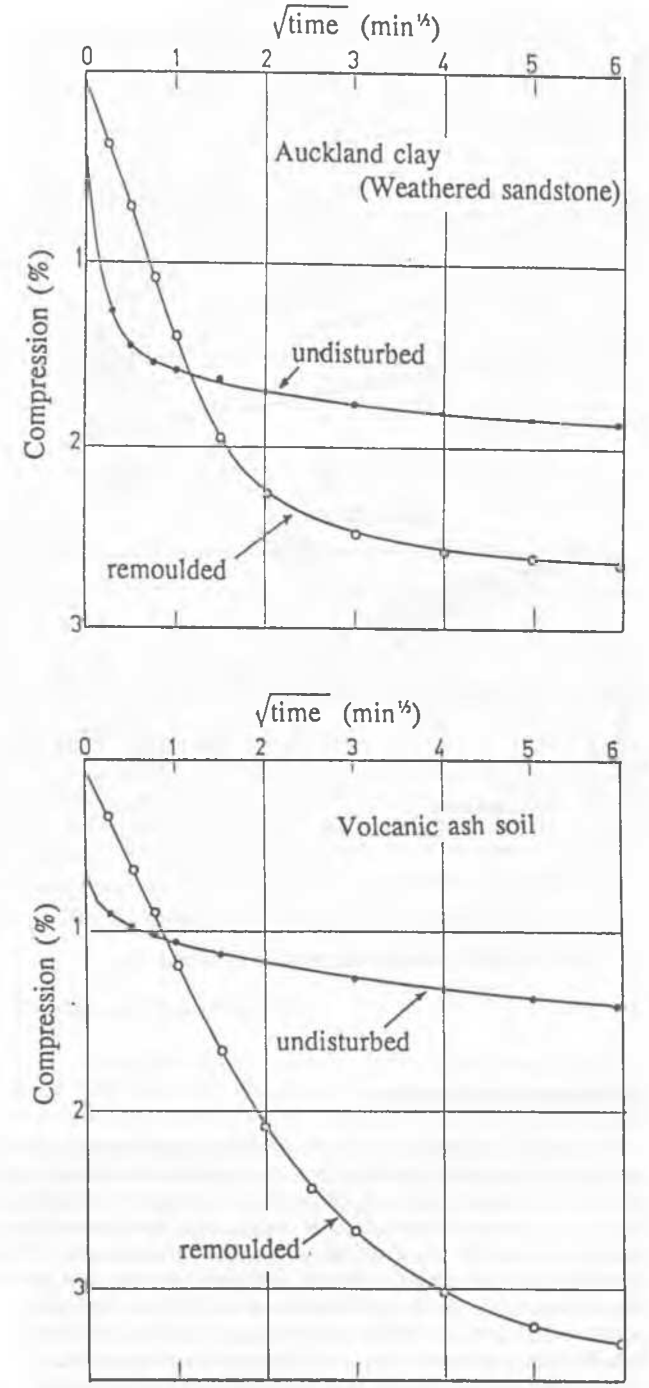

Fig. 7 Influence of remoulding on consolidation rate

The use of the log time plot does not provide a better means of obtaining reliable C_v values. Although it is not the purpose of this paper to compare undisturbed and remoulded behaviour, it is worth noting in passing that remoulding has a very significant effect on consolidation rates. The effect is illustrated in Fig.7. After remoulding the rate is much slower, and the root time plots show the normal initial straight line section.

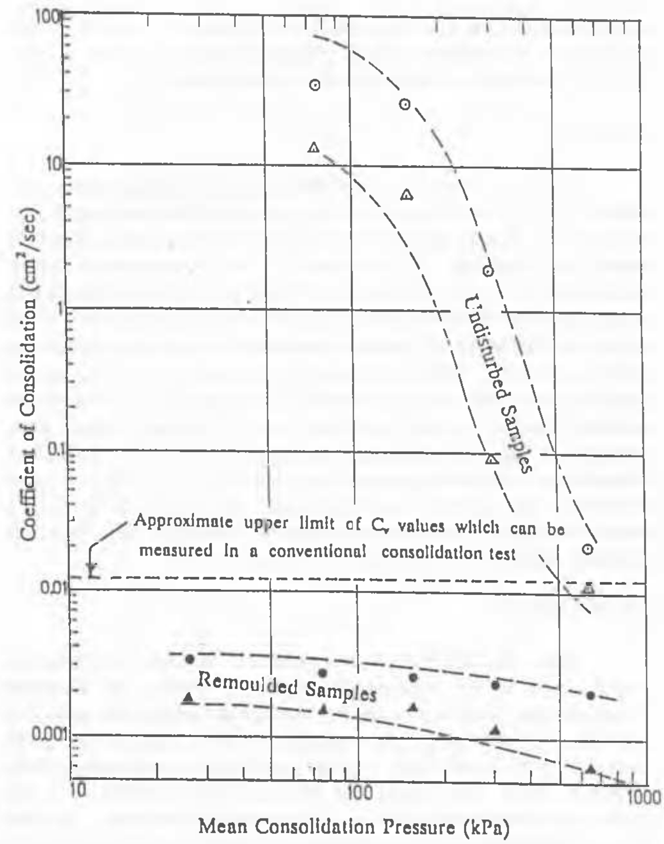

Fig. 8 Values of coefficient of consolidation for volcanic ash soil

There are two possible explanations for this very rapid rate of consolidation with the undisturbed samples. The first is that the samples are not fully saturated and it is compression of the air which occurs almost immediately; the second is simply that the soil has a very high permeability arising from certain structural effects. To investigate which of these two explanations is the more likely, a series of tests was carried out on two volcanic ash samples, in both their undisturbed and their remoulded state. Volcanic ash was used because of its very rapid consolidation rate in the undisturbed state. The tests conducted were pore pressure dissipation tests using cylindrical samples in a triaxial cell. By using triaxial samples of reasonable length and flow from one end only, the consolidation rate was reduced to the extent that c_v values could be determined. Back pressure was used to fully saturate the samples, and the pore pressure dissipation rate was used to calculate the c_v values. The samples were consolidated using a series of pressure increments similar to the procedure in a standard consolidation test. The results are illustrated graphically in Fig.8. This shows clearly the very high c_v values obtained from the undisturbed samples, and the dramatic reduction which occurs after remoulding. Also notable is the rapid reduction in the c_v value with increasing consolidation pressure.

These tests show conclusively that the rapid consolidation rate is due to the high permeability of the material and not because of the presence of air in the soil. The high permeability of the undisturbed soil appears to result from some specific structure in the soil which is destroyed on remoulding. The decrease in permeability and c_v value is typical of most soils, including sedimentary soils, but seems to be rather more pronounced with residual soils, especially the volcanic ash soil illustrated in Fig.8. The decrease in C_v value with consolidation pressure is also presumed to be due to destruction of structural effects as the stress level increases.

It is of interest to note in passing that there is an upper limit to the c_v value which can be measured in the conventional consolidation test, using a 19 mm (0.75 in) thick sample, when readings are taken manually. Analysis of the test shows that to obtain sufficient readings during the primary consolidation stage to enable reliable estimates of c_v to be made, the c_v value must be lower than approximately 0.012 cm^2/sec ($0.1 m^2/day$). With c_v values higher than this the consolidation rate is so rapid that the necessary readings cannot be taken. With instrumented electronic reading systems considerably higher c_v values could be measured. The limiting c_v value for manual readings is indicated on Fig.8.

Values of c_v from tests on undisturbed samples for three of the soils discussed n this paper have a range as follows:

Auckland ("Waitemata") clays and clayey silts	0.001 to 1 cm^2/sec
Indonesian red clays	0.008 to 0.08 cm^2/sec
Volcanic ash soils	0.01 to 50 cm^2/sec

SETTLEMENT ESTIMATES

Magnitude

To carry out estimates of settlement resulting from some specific foundation load it is necessary to have the following data:

(a) Soil Properties
(b) Initial stress state in the ground
(c) Final stress state in the ground after completion of the foundation and structure.

The question of soil properties has already been discussed. The initial stress in the ground is somewhat problematical, and depends on the combined effects of the depth of the ground water table and seasonal climatic effects. If the water table is close to the surface then the pore water pressure can be estimated accurately and there is no problem in determining the effective stresses. However, if the water table is deep then the pore water pressure above the water table is not known. It is likely to be negative (i.e. in tension) and its actual value may fluctuate considerably as the ground dries out in summer and takes up water in winter. The possible variation is indicated diagrammatically in Fig.9. It is thus not possible to determine the effective stress simply from a knowledge of the depth of the ground water table.

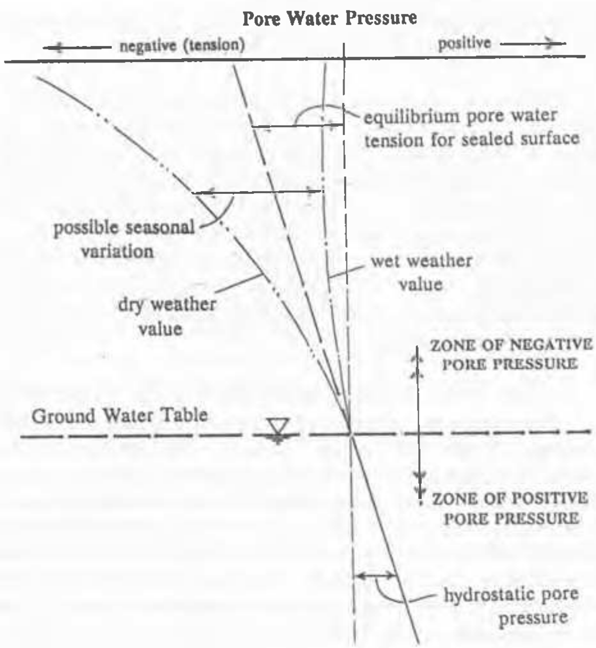

Fig. 9 Pore water pressure state above and below the water table

In addition to the uncertainty regarding the pore water tension above the water table there is the uncertainty regarding the depth of the water table itself. In very low permeability soils, such as sedimentary clays the water table may not undergo significant seasonal variation, but with many residual soils this is unlikely to be the case. Thus any calculation of initial effective stress based on a single observation of the ground water level may well be erroneous due to a change in water table between the time of observation and the time of construction of the foundation, or completion of the total structure.

It follows from the above considerations that the settlement which will occur in areas with deep water tables will depend on whether the foundation is constructed in summer or in winter. If the foundation is built at the end of a long dry summer, and the surrounding ground subsequently covered by on grade floor slabs, then the net result may be a large reduction in pore water tension to the equilibrium value for a sealed surface. The change in effective stress caused by the change in pore water tension may be greater than that resulting from the foundation loading itself, and hence may have a very significant influence on the foundation settlement. These effects are well recognised in the case of so-called "swelling soils" but do not seem to be so well appreciated in the case of residual soils in general.

Regarding the question of changes in the ground water pressures because of changing climatic factors, an interesting paper by Kenney and Lau (1983) suggests that with soils with a c_v value of 0.012 cm^2/sec (0.1 m^2/day) or less, there is unlikely to be large changes in groundwater pressures as a result of seasonal rainfall changes. The data on which this observation is based is quite limited so the relationship must be treated with caution; however, it does provide a tentative basis for evaluating the likelihood of groundwater pressure changes. This value of 0.012 cm^2/sec is in the centre of the range of values quoted above for three particular residual soil types, so that according to this criteria there is a clear possibility that there will be seasonal changes in groundwater pressure in these soils. This is in accordance with the field behaviour of these soils with respect to slope stability, as there is no doubt that slips are triggered in many of these soils by pore pressure rises associated with periods of heavy rainfall.

There appears to be considerable evidence to indicate that actual settlement of foundations on residual soils is usually less than that predicted on the basis of laboratory tests (e.g. Vaughan 1985). The difference is normally ascribed to the inability of laboratory tests (especially oedometer tests) to yield reliable estimates of field compressibility, and this is no doubt a major factor; however, it may also be that the failure to take proper account of the pore water pressure conditions in the soil is also a significant factor.

Time Rate

The high values of c_v reflected by the rapid consolidation rates obtained from consolidation tests suggest that field settlement with residual soils should occur rapidly, and in many if not most situations, should be complete by the end of the construction period. Unfortunately reliable records of settlement rate for foundations on residual soil are difficult to find. Records from an oil tank on volcanic ash soil in Omata, New Zealand, described by Wesley and Matuschka (1988), show that settlement occurred very rapidly and was almost complete when the tank load reached its peak value. Piezometers installed beneath the tank perimeter showed no rise in pore water pressure, so that full dissipation occurred as the load was applied. Limited data from other sources (e.g. Willmer et.al. 1982) suggests that with most residual soils, settlement will occur as the load is applied and post construction settlement will be negligible. There will however, certainly be exceptions to this trend.

CONCLUSION

There are some important aspects of residual soil behaviour which need to be appreciated when the results of laboratory consolidometer tests are used for making settlement estimates. In particular, there is no reason to assume that their compressibility will conform to the stress history pattern applicable to sedimentary soils, where normally consolidated and overconsolidated behaviour occurs below and above an identifiable preconsolidation pressure. To assist in the interpretation of test results it is useful to plot the pressure deformation curve using both linear and log scales for pressure.

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