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SESSION 9 — GENERAL REPORT

FIELD EXPLORATION AND SAMPLING (GEOTECHNICAL)

Reporter
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1 INTRODUCTION

The title chosen for this session of the Conference is "Field Exploration and Sampling (Geotechnical)". However, the eleven papers which have been assigned to this session deal with a much wider range of topics than the title suggests. Therefore, rather than being a state-of-the-art report on field exploration and sampling, this report presents a general review of the papers in the session.

The topics covered by the papers in this session can be summarised as follows:

- two papers on expansive soils affecting house foundations or street pavements (Pile, 1984; McInnes, 1984)
- three papers on instrumentation of roadway embankments on soft soil (Waterton and Ford, 1984), of a bridge foundation (Chandler and Harvey, 1984) and of a coal-waste dump (Dunbaven and Ross, 1984)
- two papers by Smith (1984 a and 1984 b) on the compressibility of the lateritic soils at Worsley
- formation of erosion tunnels in South Auckland, New Zealand (Goldsmith and Smith, 1984)
- evaluation of vibratory plate compactors (Jewell, 1984)
- control of saline groundwater using a permanent wellpoint system (Collingham and Newman, 1984)
- disposal of coal-mining waste (Laventhal and de Ambrosis, 1984).

It is intended to deal briefly with each of these topics and then to present a number of points for discussion drawn in the main from the papers presented.

2 EXPANSIVE SOIL

In recent years the problems associated with structures on expansive soils have been discussed in great detail in a number of special international and regional conferences and in a large number of papers in the geotechnical literature. In view of this extensive coverage, it is not proposed to do more here than reiterate some of the more important and more general points relating to expansive soil.

For volumetric changes to occur in expansive soil (or any clay soil), a change in effective stress is required. In arid and semi-arid climates, the in situ stress state of clay profiles is frequently dominated by high pore water suctions. Where the moisture loss from the ground by evaporation and transpiration exceeds the rainfall rate, a moisture deficit results which can extend to some depth, depending on other environmental factors. If there are distinct wet and dry seasons, the moisture deficit may be reduced or eliminated during the wet season, and then increase again during the dry season.

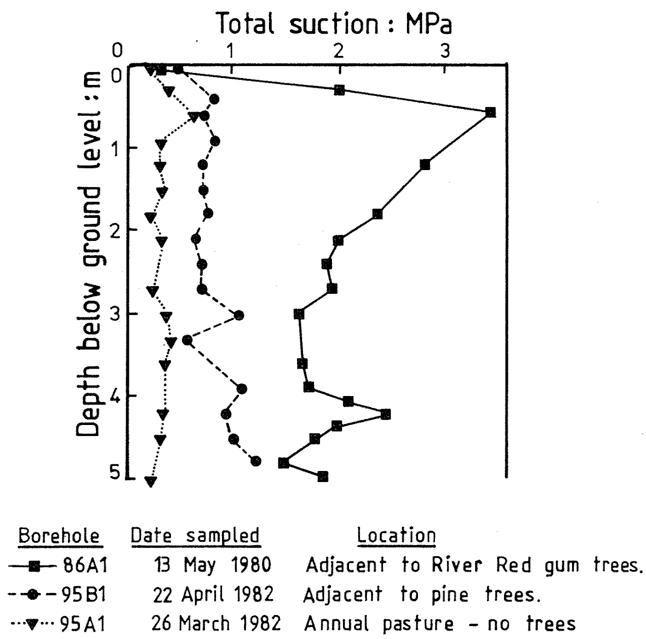

Figure 1. Measured values of total suction in red-brown earth profiles adjacent to River Red gum trees, next to pine trees and in an area void of trees

A primary factor in determining the magnitude of the suctions and the depth to which they extend is the nature of the vegetation cover (if any). Figure 1, taken from Richards et al (1983) shows a comparison between suctions in similar soil profiles measured at the same time of the year. Adjacent to a River Red gum tree, suctions of 2 MPa were measured to depths greater than 5 metres. In contrast, suctions adjacent to a pine tree and in open grassland were very much less. Figure 2, also from Richards et al (1983) shows that seasonal wetting and drying (decrease and increase of suctions) occurs both near the gum tree and in the open grassland. However, the cyclical change is seen to be confined to the upper 1.5 to 2 m. Under the gum tree, below this depth a permanent state of high suction exists which could be increased in a period of extended drought or reduced by a succession of high-rainfall years.

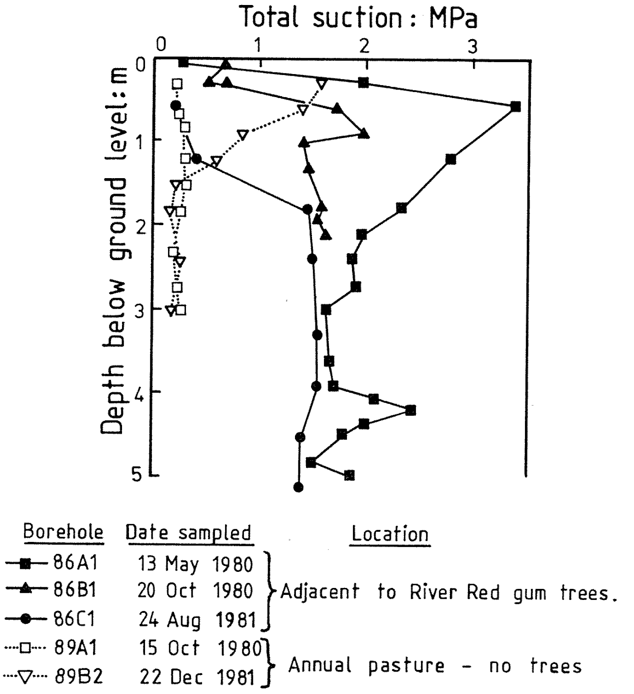

Figure 2. Measured values of total soil suction in red-brown earth profiles adjacent to River Red gum trees and in an area void of trees

Plants are generally assumed to be able to continue to extract moisture to a suction of about 1.5 MPa at which point they begin to wilt. However, plants in arid or semi-arid regions can create much higher suctions as shown for the gum tree in Figure 2. Indeed Aitchison and Richards (1965) report that Mulga and spinifex can remain healthy with suctions in the root zone as high as 31 MPa.

Since the suctions in the soil are due mainly to moisture loss by evaporation or transpiration from the surface, any changes in surface conditions can change the dynamic equilibrium conditions in the soil. De Bruijn (1973) illustrated this at an experimental site on black expansive clay at Onderstepoort. On an open grassed site, one area was stripped of all vegetation and left bare, another was covered with a blanket of clean sand and another was covered with an impermeable polyester cover. The movements of these areas over seven years compared with those of an undisturbed grassed area are compared in Figure 3. This illustrates the cyclical swelling-shrinking behaviour in the grassed area (and the effect of an unusually humid year in 1967), and the gradual heaving of the other areas due to prevention of moisture loss by transpiration. Note that in this case transpiration rather than evaporation appears as the main cause of moisture loss since the bare uncovered ground heaved in an identical manner to the polyester-covered ground.

The amount of potential heave depends among other things on the magnitude of the suctions and the depth to which they extend. For example, referring to the grassland in Figure 2, if the grass was removed or the surface was covered in the wet season, only a small amount of heave might result because suctions are small at this stage. However, if the gum tree was removed, a far greater amount of swell might occur, and the swelling could continue over a long period as the deep seated suctions were gradually dissipated. Samuels and Cheney (1975) reported that heave in high-plasticity London clay continued for about 20 years after large elm trees had been removed.

Obviously, introduction of vegetation, especially trees capable of generating high suctions, into a previously unvegetated area on expansive clay would have the opposite effect - ie. increasing the effective stress (suction) thereby causing shrinkage. It is also worth noting that since suction is an isotropic stress, shrinkage due to suction will tend to be three-dimensional, resulting not only in surface settlement but also in vertical cracking to accommodate the lateral shrinkage.

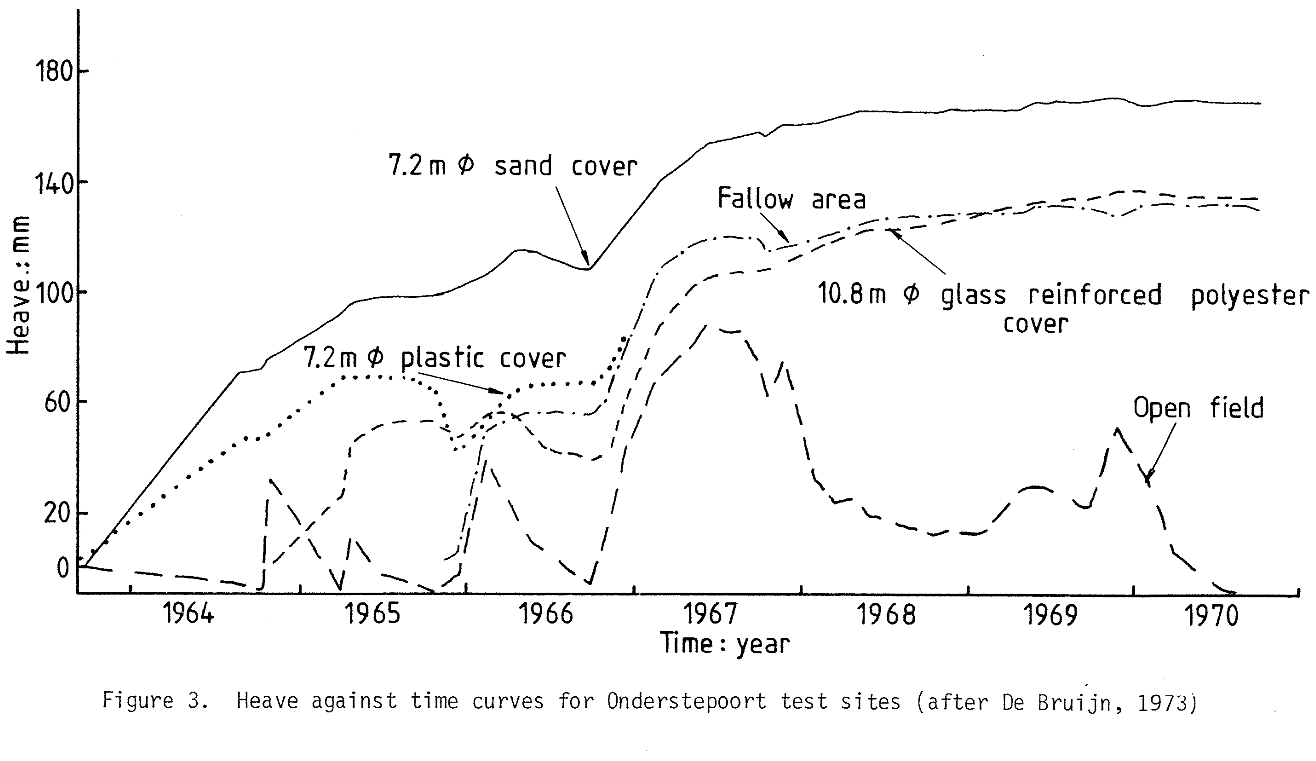

Figure 3. Heave against time curves for Onderstepoort test sites (after De Bruijn, 1973)

The stress state can also be changed dramatically by altering the availability of water to vegetation, for example by irrigation (especially excessive domestic irrigation of lawns and gardens), or by changing the site drainage.

Assuming that a change in stress state (ie. suction) has been brought about by removal or planting trees or by any other cause, a measure of the volume change which the soil will undergo is given by the instability index ($I_{p''}$) of the soil which is defined as (after Aitchison, 1973):

$$I_{p''} = (\Delta H/H)/\Delta \ell og h$$

where $\Delta H/H$ is the linear strain, and h is the suction in centimetres of water. The instability index is therefore analogous to the compression index C_c (or the recompression or swell indices C_r or C_s); it is a measure of the compressibility of the soil skeleton when effective stress changes are due to suction changes rather than to externally applied loads. Rather than measuring instability index directly, expansive soils are usually identified by Atterberg limit and linear shrinkage tests. High plasticity clays having high values of linear shrinkage are regarded as being the most expansive. (Continuing the analogy between compression index and instability index, it is worth noting that there is also a positive correlation between degree of plasticity and compression or swelling index; refer Lambe and Whitman, 1969).

It should be emphasized here that though highly expansive clays show the greatest volume change for a given change in suction, the effect of vegetation on total soil suction is much greater in the less expansive clays according to Richards et al, 1983. Thus, plasticity tests alone are not sufficient to identify sites with potential problems.

There are two courses of action which can be taken when locating low-rise structures on potentially troublesome sites: "firstly, to estimate the potential to swell and shrink and then try to avoid events which cause changes in moisture content; or secondly, to accept that swelling or shrinkage will occur and to take account of this in design. The foundations can be designed to resist resulting ground movements or the superstructures designed to accommodate movement without damage" (Building Research Establishment, 1980). However, a major problem in designing foundations for these conditions is estimating the shape of the deformation which will occur under the covered area. It is generally assumed that the deformation in the long term is in the form of a central dome, with seasonal effects extending some distance in from the edge as illustrated in Figure 4 (after Holland et al, 1975). In theory, the maximum central deflection could be predicted using an equation such as:

$$\delta = \sum_{n=0}^{m} I_{p''} \, \Delta \ell og\, h \, \Delta z_n$$

where z_n is the thickness of the n^{th} layer, $I_{p''}$ and $\Delta \ell og\, h$ are the instability index and change in suction of the soil in this layer, and m is the total number of layers. However, Richards et al (1983) suggest from their experience that "the practical use of this equation is unreliable at the present time and in fact has not always been able to predict even the direction of the movement". Pile (1984) presents results of long term observations of foundation movements of sixteen houses extending over a period of seventeen years. He shows in his Figure 1 profiles of total suction under three of the houses

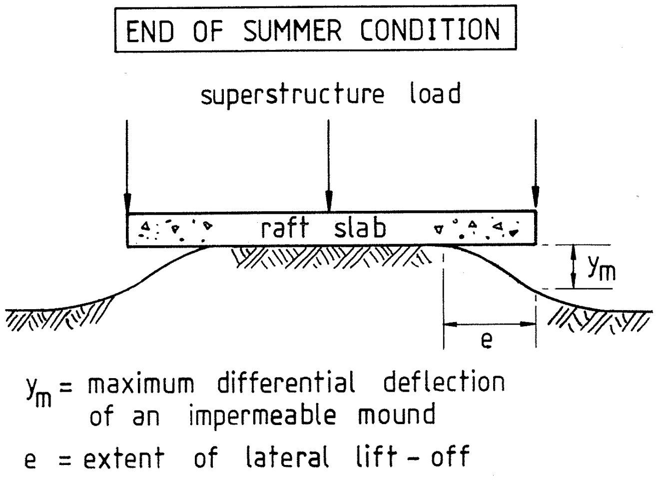

Figure 4. Idealized centre heave on expansive clay (after Holland et al, 1975)

and in parkland opposite the houses. This figure also shows soil profile, index properties and values of instability index. His Figure 2 shows the measured movements of the sixteen houses. The large variation in observed movements is probably due to the large number of variables influencing movement: the original suction profiles (perhaps still at construction time influenced by the trees which were cleared some unspecified time before construction); local variability in soil profile; site conditions at time of construction (some sites are noted as having been wet); and domestic activities such as tree planting and gardening and irrigation. Pile thus concludes that "the vertical movement of external footings and walls of houses on expansive clays may have many different patterns and is virtually impossible to predict". In contrast, McInnes (1984) found good agreement between the deflection of a street pavement due to the influence of a Coral gum tree and predictions of deflection based on measured suction values and instability indices. Perhaps this is due to a very much smaller number of variables.

In conclusion, it appears that to be able to accurately predict movement of expansive soil, it is necessary to:

- measure initial suctions throughout the soil profile
- determine the instability index of the soil - ie. the relation between suction change and volume change
- assess the likely changes in suction at all stages of construction and service life of the proposed structure due to the structure itself, removal or planting of vegetation, irrigation of gardens, changes in drainage etc.

In many instances, the interaction of many factors may make accurate prediction impossible, in which case it may be necessary to base the foundation design on the worst case of soil movement that can be foreseen (Pile, 1984).

3 INSTRUMENTATION

In geotechnical engineering, the quantities which are most commonly monitored by installing instrumentation are deformation, pore water pressure, and total earth pressure. Various aspects of measuring these quantities are dealt with in papers included in this session. Before dealing with the specific topics in these papers, some of the principles of instrumentation will be considered.

Three important benefits of installing instrumentation in full-scale structures have been stated by Burland (1977):

"Firstly, the accuracy of present analytical and predictive techniques can be evaluated and modified as necessary. Secondly, the in-situ properties of the ground can be deduced by back analysis and compared with laboratory and in-situ determinations. Thirdly, and perhaps most important, the measurements provide quantitative data which can be used as an aid to judgement in future design and construction of engineering works".

In order to achieve these aims, great attention must be given to design and selection of instrumentation, to methods of installation and to monitoring and interpretation of results. However, a large proportion of instrumentation is installed for reasons other than for research - eg., as a safety requirement in dams or to verify that structure deformations are within specified limits. Unless specifically installed for measurement of soil properties, the results obtained are frequently of little benefit in advancing our knowledge of soil behaviour.

Some of the basic requirements of instrumentation installed to measure soil response are:

(i) The presence of the instrument should not affect the value of the variable being measured

(ii) The response of the instrument must be sufficiently fast to follow the rate of variation in the variable being measured

(iii) Installation techniques must be such that the variable is unchanged, or at least return to its "undisturbed" value quickly after installation.

While these requirements can be met reasonably well for deformation and pore pressure measurements, at least in some situations, measurement of earth pressure is much more difficult. In the simplest situation - ie. where the earth pressure cell is installed in fill as it is being placed - accurate results can be obtained, though as illustrated by Dunbaven and Ross (1984) considerable care in cell design is necessary even then.

Measurement of contact stresses between the soil and a structure (footing, raft, retaining wall) can be achieved provided cell stiffness requirements are taken into consideration. However, the major problem is that such "point" measurements of earth pressure can be virtually meaningless unless a sufficiently large number of cells are installed to allow statistical methods to be used. As pointed out by Waterton (1982), "although contact pressures may be reasonably uniform as a whole, they are usually very non-uniform over areas the size of most cells".

A further degree of difficulty is encountered with measurement of in-situ earth pressures in the ground. It is virtually impossible to meet the requirement that insertion of the instrument should not alter the in-situ stress state, and once altered, it is usually impossible to know if the original conditions will, with time, be re-established around the instrument. A considerable research effort has gone into development of instruments for measuring in-situ stresses. These include the self-boring load cells developed in Cambridge (the original "Camkometer", Wroth and Hughes, 1973 a) and various push-in flat pressure cells (eg. Marchetti, 1979). However, inherent variability in in-situ stress and the lack of any method of establishing a standard against which to compare these instruments makes the results obtained open to question. It is frequently the case that if the results obtained agree with the anticipated values they will be accepted and used, otherwise some reason will be found for rejecting them.

Deformation measurements in geotechnical engineering consist predominantly of surface settlement monitoring of foundations. Such measurements may satisfy the requirements of the structural engineer or the building owner and perhaps reassure the geotechnical consultant that his settlement predictions were reasonable. However, determination of the underlying soil properties from surface measurements is far from straight forward, particularly if more than one stratum is involved. Unless the variation of settlement with depth is determined - for example by using a system such as the multi-point extensometer system developed by the BRE (Burland et al, 1972) - valid comparison cannot be made between back-figured elastic moduli and those measured by laboratory or in-situ methods.

The choice of system for measuring pore water pressure depends on a number of factors, a primary factor being the grain size of the soil. In coarse grained soils, simple standpipes are the cheapest system, but in fine grained soils, more sophisticated techniques are required to achieve a reasonable response time. In the examples cited by Waterton and Ford (1984), a further important consideration is that readings must continue to be obtained during and after placement of fill, and this dictated that remote-reading piezometers be used.

Measurement of negative pore pressures (suctions) in-situ is also an important aspect of the investigation of expansive clays, requiring techniques which are not familiar to most geotechnical engineers. A method of installing commerically-available thermocouple psychrometers to ensure long-term operation is described by Williams and Pidgeon (1983).

Before installing any instrumentation, some preliminary calculations must be undertaken to estimate the magnitude and range of the parameters of interest. This will frequently dictate the type and sophistication of the instrumentation required. In some cases, realistic prediction of response will require sophisticated numerical techniques and soil models, and these methods may also be able to show the areas where the most critical measurements are required. Then, provided the instrumentation is properly chosen and installed, the prediction techniques can be verified by comparison with the results.

Various aspects of the practicalities of choosing and installing instrumentation under roadway embankments in soft soils are outlined by Waterton and Ford (1984). They emphasize the need to carefully define objectives and to choose suitable equipment and installation techniques. In addition, the necessity for a certain degree of redundancy should be recognized - ie. that some cross-checking of results should be possible and that malfunction in some instruments does not result in a serious deficiency in the information obtained.

Chandler and Harvey (1984) describe instrumentation installed to monitor the foundations of the Bowen Bridge in Hobart, Tasmania. Pore pressures were monitored to guard against the possibility of uplift on cofferdam bases during construction. The necessity for this approach was demonstrated where for one pier the results of the monitoring dictated a major change in construction procedure. The settlements of the foundations and of a number of points under the foundations were also monitored to enable values of rock modulus to be determined. Having paid due attention to the type and accuracy of instrumentation required, satisfactory results were obtained and a meaningful comparison between backfigured moduli and moduli measured with a pressuremeter was possible.

4 LATERITIC SOILS AT WORSLEY

Two papers on the compressibilities and settlement characteristics of the lateritic soils at the site of the Worsley Alumina Refinery are presented by Smith (1984 a and b). A number of papers on other aspects of the soils at this site are included in other sessions of this Conference (Gordon, 1984 a and b, and Gordon and Smith 1984).

Smith (1984 a and b) attributes considerable significance to the results of self-boring pressuremeter (Camkometer) testing carried out at the site by the University of Western Australia. This testing is also referred to by Gordon and Smith (1984). It is therefore appropriate to re-examine some of the assumptions on which the interpretation of the pressuremeter test is based and to relate this to the tests at Worsley.

The derivation of a complete stress-strain curve from a pressuremeter test curve is based on a method proposed independantly by Palmer (1972), Layanyi (1972) and Baguelin et al (1972). The following assumptions must be fulfilled for the analysis to be valid:

- the soil must deform under conditions of zero volume change - ie. the test must be performed at a sufficiently fast rate (in relation to the soil permeability) that no drainage takes place during the test.
- the insertion of the instrument must cause no irreversible change in stress conditions so that at some stage of the test the original stress state is restored (all strains are then measured using this point as datum).

If both of these assumptions are fulfilled, the analysis shows that the shear stress (τ) in the soil at any stage of the test is given as

$$\tau \simeq \varepsilon \, d\psi/d\varepsilon \qquad (1)$$

Here, ψ is the cavity pressure and ε is the "cavity strain", defined as $(r-r_0)/r$, where r is the current cavity radius and r_0 is the initial cavity radius.

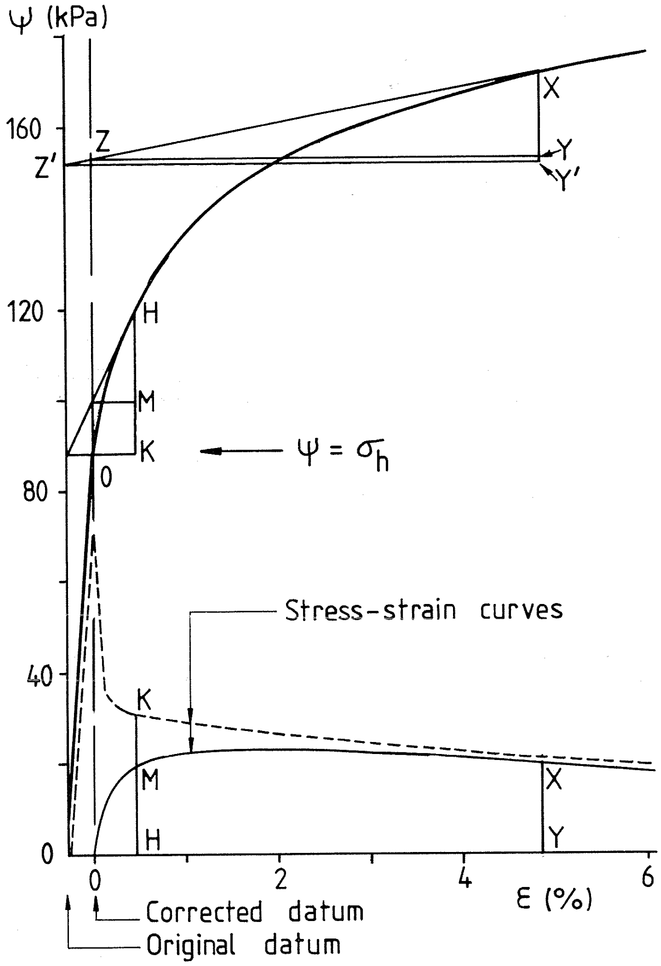

Figure 5. Deriving complete stress-strain curve from pressuremeter test (test from Hughes et al, 1975).

A graphical method of applying this relationship to a typical test curve is shown in Figure 5. In an ideal Camkometer test, the membrane of the instrument would not start to move until point 0, ie. when $\psi = \sigma_h$. A tangent drawn to the pressure-expansion curve at any point X has a gradient of $d\psi/d\varepsilon$, ie. in this case XY/YZ. Therefore, the distance XY is equal to $(YZ)d\psi/d\varepsilon$, or $\varepsilon \, d\psi/d\psi$, and is thus the current value of τ by equation 1. By drawing similar tangents to successive points on the curve, the shear stress versus strain curve can be constructed, as shown by the lower solid line in Figure 5. Note that this is a plot of shear stress versus (linear) "cavity strain" ε (which is numerically equal to circumferential strain ε_θ and is thus a principal strain). The (engineering) shear strain γ is approximately equal to 2ε . Therefore,

- the gradient of the initial part of the curve of τ versus ε is equal to 2G, and
- the plot of τ versus ε can be converted into a plot of τ versus γ by doubling the values of ε on the ε-axis.

In practice, as shown in Figure 5, the membrane of the Camkometer usually begins to expand at pressures well below σ_h. If the strain datum is now (erroneously) taken to be the initial radius of the instrument, then, using the graphical interpretation of Equation 1, the shear stress at point H would be calculated to be HK (instead of HM if the correct datum is used). In this way, using the incorrect datum, the stress-strain curve obtained would be as shown by the dashed curve in Figure 5.

Thus it can be seen that the shape of the derived stress-strain curve, at least at low strains, is extremely sensitive to the choice of strain datum. At larger strains it can be seen that the calculated shear stress XY would be increased only very slightly to XY' if the tangent XZ is extended back to the incorrect datum at Z'. Thus the derived stress-strain curves merge at larger strains and the "post-peak" value of shear strength is less sensitive to datum choice. As Wroth and Hughes (1973 b) noted in discussing Palmers method "Clearly, there are difficulties and limitations caused by the choice of origin for the derivation of the stress-strain curve, and caution must be exercised in using the method for obtaining stress-strain data".

It is again emphasized that if the soil is sufficiently permeable to allow significant drainage to occur during the test, the result will be an error in the derived values of "undrained shear strength"; the test does not give values of drained strength parameters unless the actual volume changes which occur can be measured.

Smith (1984 b) also states "An average Poisson's Ratio of 0.4 was computed from the results of the Camkometer testing". To the writer's knowledge, no method of obtaining Poisson's ratio from Camkometer tests has previously been published, so it would be of interest if details of Smith's method could be presented. It should also be noted that the pressuremeter test gives directly a value of the shear modulus G, and not of Young's modulus, E. The shear modulus G is a relationship between principal stress and strain differences, so that for an isotropic elastic material, the value of G is the same whether it is calculated in terms of total or effective stresses (Wroth 1975): ie.

$$2G = \frac{\sigma_1 - \sigma_3}{\varepsilon_1 - \varepsilon_3} = \frac{\sigma_1' - \sigma_3'}{\varepsilon_1 - \varepsilon_3}$$

To obtain a value for Young's modulus, an appropriate value of Poisson's ratio must be used: ie.

$$\frac{E}{2(1+\mu')} = G' = G_u = \frac{E_u}{2(1+\mu_u)}$$

giving $E' = 2.4G$ for $\mu' = 0.2$
and $E_u = 3G$ for $\mu_u = 0.5$

Thus the drained values of Young's Modulus quoted by Smith (1984 a and b) would have been obtained from the measured values of G using (presumably) the quoted value of Poisson's Ratio of 0.4, giving $E' = 2.8G$. It is not clear if the same value of Poisson's Ratio was used to obtain values of Young's Modulus from the values of constrained modulus measured in the oedometer. Lambe and Whitman (1969) show that constrained modulus D and Young's Modulus E are related by:

$$D = \frac{E'(1-\mu')}{(1+\mu')(1-2\mu')}$$

For $\mu' = 0.4$, this would give $D = 2.14\,E'$. For more usual values of drained Poisson's ratio (say $\mu' = 0.3$), this relationship becomes $D = 1.35\,E'$.

With regard to Smith's assertion that the back-calculated Young's Modulus (ie. 55 MPa) is more consistent with values for medium dense to dense sands than for clays or silts, reference should be made to the discussion session on stiff clay properties at the 7th European Conference on Soil Mechanics and Foundation Engineering (eg. Simpson et al, 1979), or to the work of Ward et al (1959) and others in London Clay, which shows that values of Young's Modulus as high as 100 MPa are not uncommon for stiff clays.

Referring to Figure 2 in Smith (1984 b), it is intersting to note that values of liquidity index derived from the quoted results (L.I. = $(w-w_p)/(w_\ell-w_p)$) for the soil under the trial embankment at Worsley are generally close to or greater than 0.5, and in one case (at about 13.5 m depth) is about 0.8. For remoulded soil the shear strength is generally assumed to vary from about 170 kPa at the plastic limit to about 1.7 kPa at the liquid limit*, with the shear strength for intermediate moisture contents given by (Wroth and Wood, 1978).

$$c_u = 170 \exp(-4.6\ \text{L.I.})\ \text{kPa}$$

For a liquidity index of 0.8, this equation implies the shear strength of the remoulded soil would be about 4 kPa. For L.I. = 0.5, the corresponding remoulded shear strength would be 17 kPa. This, perhaps more than the Camkometer test results, shows the extreme sensitivity of these soils, with the ratio of peak undisturbed shear strength to deduced remoulded shear strength being of the order of 20 to 100.

In fact, mapping these values of liquidity index onto Schofield's (1983) chart of "Equivalent Liquidity" suggest that the soils would be unstable after the fashion of the Champlain Sea Clay, which, when disturbed, tends to fail by a liquefaction mechanism. Thus the high liquidity index values for the Worsley site compared to the high strengths perhaps warrant some further discussion.

5 TUNNELLING SOILS

The paper by Goldsmith and Smith (1984) describes an investigation of an area in South Auckland, New Zealand in which the occurrence of erosion tunnels has been identified. A review of the factors which are considered essential for the formation of such tunnels is also included. One of these factors is the degree of dispersivity of the soil - a factor which is also of prime importance in the more general topic of piping and erosion in earth dams.

In natural terrain, there is probably little that can be done to prevent formation of erosion tunnels if all the essential conditions for their formation are present, apart perhaps from altering the drainage patterns in the area. Indeed, the main problem appears to be to identify areas in which erosion tunnels might be present. It is therefore interesting that Goldsmith and Smith conclude "the methods for determining the propensity for tunnelling to develop within a soil are not well defined" and emphasize the need for further research on this topic and for the development of suitable laboratory tests to better define the soils which present a risk.

* Though the concept of unique strengths at the plastic and liquid limits is generally accepted, the actual strength values are a subject of current discussion in the geotechnical literature (eg. Whyte 1983).

The commonest test used by geotechnical engineers to determine dispersivity is perhaps the Emerson Crumb Test (AS1289 C8.1-1980). Though this test is simple to perform, it does not give geotechnical engineers who are unfamiliar with the work of soil scientists in this area any appreciation of the complexity of the factors which determine dispersivity.

While the tendency of certain soils to deflocculate can be controlled in earth dams by modifying the cation concentration of the pore water or the sodium adsorption ratio of the soil, such solutions may be expensive to achieve initially and may require ongoing control of the chemistry of the imponded water. However the cheapest solution and the best from the engineering point of view may be to achieve the lowest possible seepage velocities through the dam by moisture content control during construction - ie. the material should be compacted on the wet side of Standard O.M.C.

Contractors have naturally a tendency to place core material at or dry of Standard O.M.C. because of the natural dry state of the soil, the difficulty of handling and working wetter soil or to be able to achieve the specified degree of compaction. In many cases, the designer may be at fault in specifying too high a degree of compaction rather than concentrating on ensuring a high enough moisture content. The success of the "puddled clay" technique in wetter climates shows that satisfactory cores can be constructed of material well wet of O.M.C. provided it is sufficiently worked to achieve a homogeneous mass. The flexibility and ductility of such material may also be a desirable feature if ground movement is anticipated, though in dry climates the greater shrinkage potential of wetter soil is a possible disadvantage if dams are only partially filled during prolonged dry spells.

6 VIBRATING PLATE COMPACTORS

The characteristics of the ground vibrations produced by four vibrating plate compactors have been studied by Jewell (1984). It is generally believed that it is the peak particle velocity which is the major factor governing damage to structures. Jewell quotes a proposed German Standard (DIN 4150) which puts a limit of 4 mm/s on the peak particle velocity which already visibly damaged buildings can tolerate. It is of interest to note that according to his Figure 2, humans would find vibrations at or just below this limit to be "annoying". Where construction activity is taking place close to private dwellings, the sensitivity of the householders to vibrations will naturally be greatly increased. Unless the builder has taken the precaution to survey adjacent structures for existing damage before commencing soil compaction, it may be difficult to prove after the event that the construction activity was not the cause of this damage.

Jewell concludes that to satisfy the requirements of the German Standard DIN 4150 for already cracked masonry structures "....the compactors would need to be operated at distances of up to 6 metres or more to minimize the probability of damage occurring". Such a conclusion has implications for the house construction industry and for house owners at least in Perth where compaction of sand pads for raft foundations is a routine procedure.

7 PERMANENT DEWATERING SCHEME

The paper by Collingham and Newman presents a case study covering the investigation, design, construction and commissioning of a permanent wellpoint scheme for the control of saline groundwater near Lake Victoria. Besides being an interesting account of a rather unique project, the studies performed illustrate the importance of careful design to eliminate long-term problems due to corrosion of metalic parts or clogging or chemical cementing of screens or filters. For any large-scale permanent or semi-permanent system, chemical analysis of the groundwater must be performed and any tendency for cementing or clogging determined by long-term pumping trials.

The authors advocate the use of finite element modelling to perform sensitivity analysis during the design phase. Unfortunately in most groundwater studies, while the geometry of the problem may be easy to define with reasonable accuracy, represntative global permeability values are more difficult to determine except by extensive in situ testing.

8 DISPOSAL OF COAL MINING WASTE

The paper by Laventhal and de Ambrosis (1984) is entitled "Geotechnical Aspects of Coal Mining Waste Disposal in the Sydney Basin". This paper concentrates on the problems associated with the disposal of the coarse fraction of coal mining waste. To complete the picture, it is appropriate to give some consideration to the geotechnical aspects of the disposal of fine tailings from the mining industry.

In many areas of coal and metaliferous mining, disposal or storage of fine tailings or process residue is a major economic and environmental problem. From the geotechnical viewpoint, the properties of the tailings which are often of interest are:

- the rate at which sedimentation occurs if the tailings are pumped to disposal as a dilute slurry
- the clarity of the supernatant liquor (deflocculants may be required)
- the propensity of the material to "beach" at discharge points
- the permeability of the settled tailings and the rate at which it consolidates
- the strength profile of the consolidated material
- the density profile of the consolidated material (which determines the volume of storage area required)
- the time which must be allowed after completion of tailings disposal before final rehabilitation can be achieved (by covering with fill and topsoil).

Some aspects of the performance of fine grained tailings have been dealt with in an earlier session of the Conference (eg. Glenister and Cooling, 1984, Parker et al, 1984 and Corless et al 1984). However, one important aspect which perhaps merits some further discussion is the benefit which can be derived from the high net rates of evaporation common to many mining areas of Australia.

Where fine tailings have been deposited under water in a low-permeability storage area, some surface "crusting" is generally required before any backfilling can be placed. The factors which determine the time for a crust to form, the depth of crusting, and the strength of the crust do not appear to be well documented. A recent project in which the writer was involved (the results of which are unpublished) suggests that a factor of primary importance is the depth to the water table (in the tailings). Where removal of free water has been accomplished by evaporation, precipitation of salts combined with the tendency of the fines fraction to sediment out last results in a thin layer of extremely low permeability being formed on the surface. If subsequent lowering of the water table is to be achieved solely by evaporation, this impermeable surface layer can greatly impede the process. If groundwater lowering can be achieved by some other means - base drainage, dewatering from wells or wellpoints for example - the pore water suctions which develop above the water table may be sufficient to produce vertical cracking at the surface. This greatly increases the surface area exposed to evaporation, and since these vertical faces may be more permeable than the very fine layer on the surface, the rate of water removal by evaporation is increased, thereby reducing the time to eventual rehabilitation.

Thus lowering the groundwater table in the disposal area has a number of benefits:

- the water or liquor removed may in itself be a resource (eg. in arid regions, or in the alumina industry)
- higher density and strength at depth due to the increase in effective overburden pressure (Glenister and Cooling, 1984)
- greatly increased rate of crust formation.

9 CONCLUDING REMARKS

The papers which have been included in this session of the Conference deal with a wide variety of topics, and therefore it has not been possible to adequately review each of these in this report. The approach taken has been to expand or highlight some of these topics with the view to stimulating discussion from the authors themselves and from the Conference participants.

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