REVIEW PAPER: SESSION III

Heavily over-consolidated clays

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Introduction

Many major towns and cities are underlain by substantial depths of heavily overconsolidated clay. The estimation of settlements of structures on heavily overconsolidated clays has therefore been a subject of particular concern to engineers who contemplate differential settlement rather than bearing capacity as the limiting factor in determining what they can achieve. Thus it might seem reasonable to anticipate that case histories would be plentiful and the response to the Committee's call for such information would be good. This expectation has, however, been only partially realised.

Geological history

As with other soils and rocks, the behaviour of heavily overconsolidated clays is dictated by geological and geomorphological history. The terminology adopted by Morgenstern (1967) is apt in differentiating between the geological history of heavily overconsolidated clays, which fall broadly into two groups.

Firstly, there are marine or lacustrine deposits overconsolidated by the pressure of later sediments which have subsequently been eroded, which Morgenstern termed *simple overconsolidation*, i.e. subjected to a single loading and unloading cycle. These clays are usually relatively uniform in texture and grading and are amenable to some degree to sampling, laboratory or *in situ* testing, and to settlement analysis with a measure of confidence. The London Clay, the Gault Clay, and it seems, the Frankfurt Clay settlement records which are contained in papers to this Conference, are representative of simple overconsolidation.

The second group, which have been subjected to what Morgenstern termed *complex overconsolidation*, are typified by moraine and boulder clays. Their stress history contains major reloading and unloading cycles due to advancing and retreating ice sheets which may be interspersed with periglacial deposits. Because of their heterogeneous nature and the depositional and erosional loading and unloading they have undergone, it is unlikely that empiricism based on local practice will be fully replaced by more definitive methods of settlement analysis.

Methods of settlement analysis

Analytical problems in soil mechanics fall essentially into two groups: those of stability and those of settlement. Terzaghi's published work on consolidation in the period from 1921 to 1925 summarised by Skempton (Terzaghi, 1960) is the starting point of modern soil mechanics.

Terzaghi developed a theory of one-dimensional consolidation with which, using results obtained in the oedometer test, an estimate of consolidation settlement could be made. In cases where the compressible layer is thin relative to the extent of the loaded area or where the compressible layer was contained by layers of sand or rock above and beneath—conditions where lateral strain is negligible—this approach has been shown to be fairly reliable.

The method assumes $\Delta u = \Delta\sigma_v$ and that the relationship between axial compressibility m_v, and vertical effective stress given by the oedometer is applicable. The settlement is given by:

$$\delta_c = \int_o^z m_v . \Delta\sigma_z . \mathrm{d}z$$

However, the more general case, where lateral strain can occur is clearly outside this treatment. In the case of overconsolidated clays the method was suspected of providing a substantial overestimate of settlement.

Skempton and Bjerrum (1957) identified the need for a more generalised approach and proposed a semi-empirical method in which recognition of the nature of the clay and its thickness relative to the size and shape of the loaded area could be allowed for and of which the Terzaghi theory remained a valid particular case. This they achieved by subdivision of the overall settlement into two components, viz. 'immediate' settlement, which was considered to occur in saturated clays due to deformation under no-drainage conditions; 'consolidation' settlement which was controlled by dissipation of pore pressure derived from changes in σ_1, σ_2 and σ_3 rather than by change in vertical stress. This marked an important improvement in the method of estimating settlements; it contained a strong element of logic yet the laboratory testing requirements remained undemanding and the analysis, in computational terms, elementary. It has thus become established as a standard office procedure for the estimation of settlements in overconsolidated clays. However, a structure cannot be built instantaneously, the imposition of the building load can take a significant span even on a primary consolidation time-scale. During construction, immediate primary consolidation and secondary consolidation are occurring simultaneously. Nevertheless, it is convenient to persevere with the notion of instantaneous construction as a means of quantifying the effects of various stages in the construction process.

Immediate settlement

For the purposes of estimating immediate settlement, it is generally assumed that we may resort to elastic theory; to the classical expression for the settlement at the corner of a flexible loaded rectangular area on the surface of an elastic half-space:

$$\rho = \frac{q.B.(1-\nu^2).I_\rho}{E} \qquad (1)$$

where
q = surface stress
B = breadth of loaded area
ν = Poissons Ratio
E = bulk modulus
I_ρ = influence factor

The assumptions implied by this relationship are homogeneity (the soil properties are constant from point to point), linear elasticity (that strain is proportional to stress) and isotropy (its properties are identical in all directions through any point). The half-space is also assumed infinite in extent. Values of the influence factor $I\rho$ are dependent upon the shape of the loaded area and, in the Steinbrenner adaptation (1934), also the thickness of the elastic layer. For saturated clays no volume change occurs so long as there is no dissipation of pore pressure. Therefore in the calculation of immediate settlements $\nu = 0.5$ is assumed. The search for appropriate values of E_u (the undrained Young's Modulus) has concentrated work upon a variety of laboratory and, latterly, field testing techniques.

Consolidation settlement

In order to compute consolidation settlement, Skempton and Bjerrum proposed that the induced pore pressure due to loading the foundation should be estimated through the use of the pore pressure coefficients A and B where

$$\Delta u = B.[\Delta\sigma_3 + A.(\Delta\sigma_1 - \Delta\sigma_3)]$$

$B = 1$ for saturated clays. The changes in stress $\Delta\sigma_1$, and $\Delta\sigma_3$ at points below the centre of the foundation are derived by elastic theory and the parameter A from observed laboratory tests. In order to retain the practicability of the oedometer test, Skempton and Bjerrum related one-dimensional settlement (ρ_{oed}) to consolidation settlement (ρ_c) by means of the device μ where

$$\rho_c = \mu.\rho_{oed}$$

i.e.

$$\rho_c = \mu \int_o^z m_v.\Delta u.dz$$

The constant μ is a function of a shape of load distribution (breadth, b), depth of compressible layer (z) and pore pressure coefficient A.

The variation of μ with pore pressure coefficient A for various values of z/b and for circular and strip loading were produced in graphical form. Scott (1963) has recalculated and corrected errors relating to strip loading in the original text and the subsequent correction.

Despite the widespread acceptance of the Skempton and Bjerrum method in European foundation practice and the confirmation of its reliability at least in fairly familiar circumstances, the sensitivity of μ to the value of A and the philosophical inconsistency in combining one component based upon elastic theory with another based upon a semi-empirical application of laboratory data prompts the need for a more rational approach.

Stress-path methods

That proposed by Lambe (1964) transports the settlement analysis entirely to the laboratory. He has used the stress-path device to illustrate diagrammatically the shortcomings of the earlier methods of settlement analysis. He demonstrated, by careful triaxial testing, the influence on the behaviour of the soil of the stress history that the soil element has undergone in reaching its final state. The attempted simulation on laboratory specimens of the stress changes on soil elements in the field is, at least in principle, a step in the right direction. A similar line of approach has been advanced independently by Davis and Poulos (1963). However, as a practical means of estimating settlement, the endeavour to find a more rational method merely substitutes the difficulties of identifying 'typical' soil elements and a complex laboratory test programme and analysis for those of estimating E_u and A.

The pursuit of a stress-path technique for estimating settlements particularly in relation to London Clay has been given much impetus by the work of Som (1968), Simons and Som (1969 and 1970). In a very detailed study of the elastic parameters of London Clay, they have examined the extent to which the changes in effective stress in the ground during consolidation depart from the process of one-dimensional consolidation. They propose a correction factor

$$\lambda = \varepsilon_1/\varepsilon_v$$

where ε_1 = vertical strain and ε_v = volumetric strain, to allow for the effect of lateral strains, their revised equation being

$$\rho_c = \int_o^z \lambda.m_v.\Delta u.dz$$

The use of this equation still assumes that $m_v\Delta u$ represents the volumetric strain even though it is applied to strain increments which are not one-dimensional. The results of consolidation tests (Som (1968)) with differing values of $\Delta\sigma_3'/\Delta\sigma_1'$ broadly confirm this assumption. Thus m_v is given the wider definition of $\varepsilon_1/\Delta\sigma_1'$ independent of $\Delta\sigma_3'/\Delta\sigma_1'$, and only a function of stress level. The value of λ has been found experimentally for a variety of stress increment ratios $\Delta\sigma_3'/\Delta\sigma_1'$.

The methods so far considered are illustrated in stress-path terms as in *Fig. 1*. For simplicity let us consider an element of soil on the vertical axis through the centre of a circular loaded area. The *in situ* pressure on the element before applying the surface load is in the vertical direction p_0. If the *in situ* pore pressure is u_0 then the effective stress in the vertical direction is p_0' ($= p_0 - u_0$) and in the horizontal direction k_0p_0'. In total stress terms, the state of stress of the element is thus represented by the point A, and in effective stress terms by the point A'.

A loading, of intensity q, is then applied, without drainage, causing an increase in vertical stress of $\Delta\sigma_v$ and in horizontal stress of $\Delta\sigma_h$ on the element. From Skempton (1954), assuming a fully saturated clay, the change in pore pressure at the element is given by

$$\Delta u = \Delta\sigma_h + A\,(\Delta\sigma_v - \Delta\sigma_h)$$

Immediately after application of the load the effective stresses are therefore

$$(\sigma_v')_{ab} = p_o' + \Delta\sigma_v - \Delta u$$
$$(\sigma_h')_{ab} = k_o p_o' + \Delta\sigma_h - \Delta u$$

and the state of stress in effective stress terms is represented by the point B′. Note that since, for most clays, the value of A is positive and less than 1.0 within the range of stresses encountered in foundation problems the change in pore pressure Δu is greater than the change in horizontal stress. Thus, during the undrained loading the element experiences an increase in vertical effective stress and a decrease in horizontal effective stress. The vertical strain undergone by the element in undrained loading is therefore that associated with the stress path A′–B′.

The element is now allowed to consolidate: in the Skempton and Bjerrum case we assume one-dimensional consolidation. Thus the dissipation of the excess pore pressure Δu results in the stress path B′C′, the final state after primary consolidation, in effective stress terms being represented by the point C′, and in total stress terms by the point C.

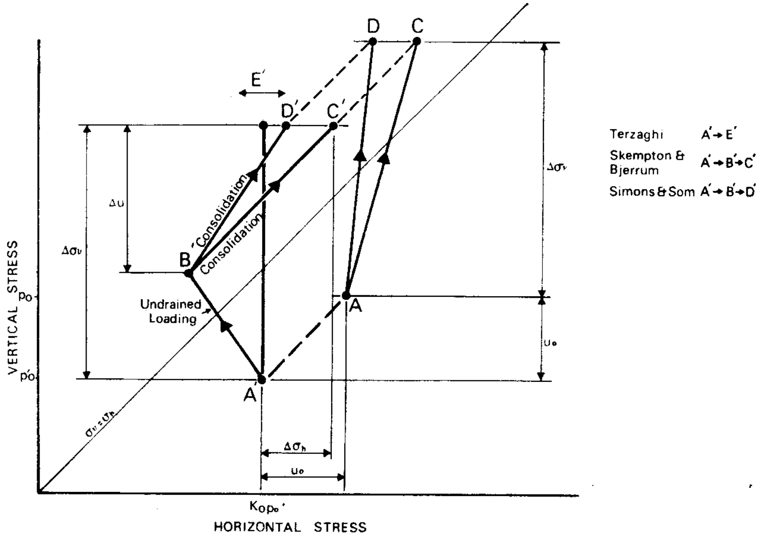

Fig. 1. Stress path settlement analysis

Simons and Som (1970) have taken account of the lateral strain and of the resulting change in stress distribution during consolidation. Thus the true final state of stress of the element is represented by the point D′ and the stress path C′–D′ is the correct path for the consolidation process.

Referring to *Fig. 1*, since Terzaghi assumes $\Delta u = \Delta\sigma_v$, the stress path followed is A′–E′ where E′ is not located in terms of horizontal stress. Skempton and Bjerrum ignore lateral strain in the consolidation phase, and follow the stress path A′B′C′. Simons and Som identify the stress path A′B′D′.

The practical application of the Simons and Som method relies upon a simplification of the consolidation properties so that simple oedometer results can be used. This simplification which has been justified by experimental evidence is, strictly in terms of elastic theory, inconsistent with the proposed relationship between λ and $\Delta\sigma_3'/\Delta\sigma_1'$.

Settlement prediction by elastic methods

The use of elastic theory to estimate immediate settlements has already been describe as part of the Skempton and Bjerrum method of analysis. A modulus E_u, and influence coefficient $I\rho$, a function of Poissons ratio ν, have to be selected for any given loading condition. In estimation of immediate settlements $\nu = 0.5$ is used for saturated clays. To estimate total settlement it is necessary to use a modulus E' appropriate to drained loading and the ν of the mineral skeleton.

The use of Equation 1 implies a homogeneous elastic and isotropic material, which soil is not. In the case of overconsolidated clays, however, total settlements are generally relatively small and there is usually a substantial factor of safety in terms of bearing capacity so that the stress–strain behaviour is sensibly linear. Compressible layers which are anisotropic and/or with non-uniform elastic modulus with depth cannot correctly be treated by the application of Equation 1. However, since the value of Poissons ratio for the mineral skeleton will be low, the error in using a vertical elastic modulus for vertical loading will not be great. Barden (1963) has investigated the effect of anisotropy, using a solution by Mitchell (1900) to show that in the case of surface loads settlements are affected by only 25% within the range $E_h/E_v = 1.0$ to 2.5. It would therefore seem justifiable to assume Equation 1 valid for the present purposes.

Wroth (1971) discusses the elastic behaviour of overconsolidated clays both in terms of undrained and drained parameters. From the results of the sampling at Ashford Common shaft (Bishop, Webb and Lewin (1965)), he deduced that the elastic moduli were a function of pressure and overconsolidation ratio and, more importantly, that they increase linearly with depth. Using the relationships between the elastic constants in the laboratory and the field deduced by Henkel (1971) and in particular for triaxial loading in undrained and drained conditions, where

ν_{hv} = Poissons ratio for the effect of vertical strains on horizontal strains
E_v' = Effective stress Young's modulus in the vertical direction
E_h' = Effective stress Young's modulus in the horizontal direction
$R = E_h'/E_v'$
E_{uI} = Undrained Young's modulus for triaxial loading on vertical specimens

$$\frac{\Delta\sigma_v'}{\Delta\sigma_h'} = -2\,\frac{\left(1 - \nu_{hv}\dfrac{1+3R}{2}\right)}{R(1-2\nu_{hv})} = M_I$$

and

$$E_{u_I} = \frac{E_h'}{R}\left(\frac{M_I - 1}{M_I - 2\nu_{hv}}\right) \qquad (2)$$

Given a simplified C_u–depth profile for any site and assuming a relationship between C_u and E_{uI} and a value of R, it is therefore possible to establish a corresponding profile of E_v' with depth. Commonly, in overconsolidated clay the C_u–depth profile, and thus the E_v' profile, shows a linear increase with depth.

Stress and deformation in a non-homogeneous elastic half-space has been investigated by Gibson (1967) who published solutions for the particular case where the shear modulus G varied linearly with depth and of which the Winkler model was a special case. Gibson, Brown and Andrews (1971) extended this work to examine the case when the elastic medium is of restricted depth and adhering to a rigid base. Brown and Gibson (1972) have since extended Gibson's original work to the analysis of settlement of circular and strip loading on the surface of a half-space with Young's modulus

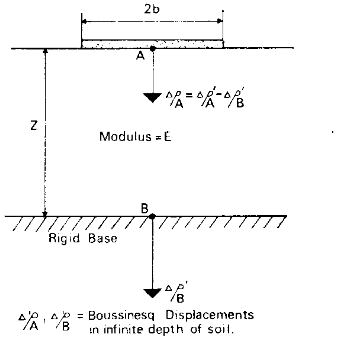

Fig. 2. Steinbrenners adaption of Boussinesq

varying linearly with depth and Poissons ratio of fixed value within the range 0 to 0.5.

Brown and Gibson (1973) have now published the solution to the problem of corner settlement due to a rectangular loading on an isotropic linearly non-homogeneous elastic half-space. However, no definitive analysis has been published which provides influence factors where this compressible layer is of limited depth. An approximate analysis can be undertaken by a simple extrapolation of Steinbrenner (1934). In Steinbrenner the displacement $\Delta\rho_A$ of the point A on the surface of limited depth z is computed from Bousinesq's expressions for the displacement of points A and B ($\Delta\rho_A'$ and $\Delta\rho_B'$) for an infinite depth of soil (*Fig. 2*). Since the actual deflection of point B is zero, $\Delta\rho_A$ is given by

$$\Delta\rho_A = \Delta\rho_A' - \Delta\rho_B'$$

Consider a multi-layered system as in *Fig. 3* and let us calculate the displacements $\Delta\rho_1$, $\Delta\rho_2$, $\Delta\rho_3$, etc., for an infinite depth of soil of modulus E

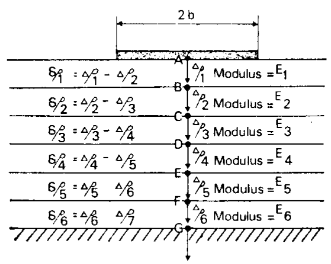

Fig. 3. Adaptation of Steinbrenner to give approximate analysis for variation of E with depth

(i.e. as if the whole half-space had the modulus of the upper layer). The settlement in each layer is then given by

$$\delta\rho_1 = \Delta\rho_1 - \Delta\rho_2$$
$$\delta\rho_2 = \Delta\rho_2 - \Delta\rho_3$$
$$\delta\rho_3 = \Delta\rho_3 - \Delta\rho_4 \text{ etc.}$$

But all these settlements are based upon the use of a uniform value of $E = E_I$: to accommodate variations in E with depth we should therefore multiply the nth layer of E_I/E_n. Hence the total settlement $\Delta\rho$ is given by

$$\Delta\rho = \frac{E_1}{E_1}.\delta\rho_1 + \frac{E_1}{E_2}.\delta\rho_2 + \frac{E_1}{E_3}.\delta\rho_3 \ldots \text{etc.} \qquad (3)$$

We actually require influence factors where

$$I_\rho = \frac{\rho/B}{q/E_1}$$

We can therefore rewrite Equation 1

$$I_\rho = \frac{E_1}{E_1}.\delta I_{\rho_1} + \frac{E_1}{E_2}.\delta I_{\rho_2} + \frac{E_1}{E_3}.\delta I_{\rho_3} \ldots \text{etc.} \qquad (4)$$

For the case where E increases linearly with depth the soil can conveniently be divided into finite layers of equal thickness and ascribed an average E for each, E_1, E_2, E_3, etc. The rate of increase of E with depth is expressed in terms of E_0 (at the surface) thus

$$E = E_0\left(1 + K.\frac{z}{b}\right)$$

and Equation 4 then becomes

$$I_\rho = \frac{E_0}{E_1}.\delta I_{\rho_1} + \frac{E_0}{E_2}.\delta I_{\rho_2} + \frac{E_0}{E_3}.\delta I_{\rho_3} \ldots \text{etc.}$$

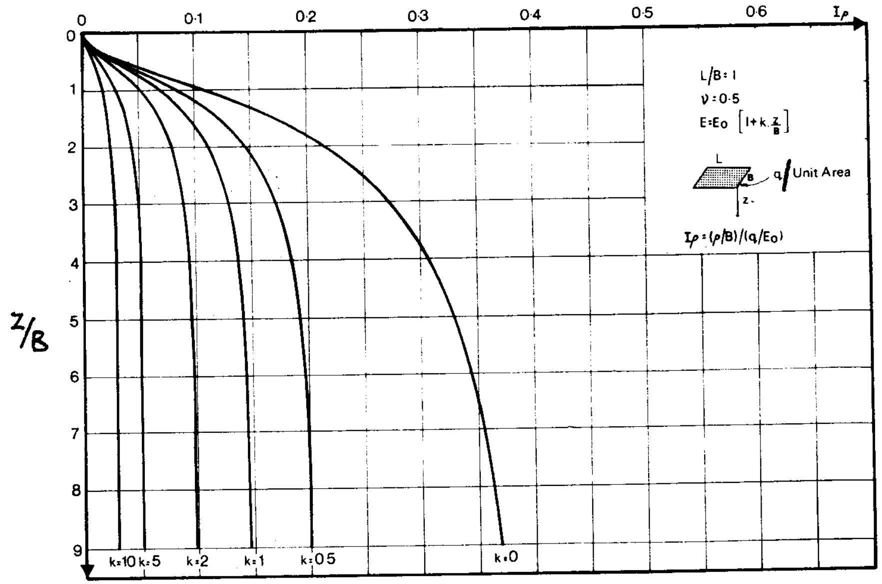

Fig. 4

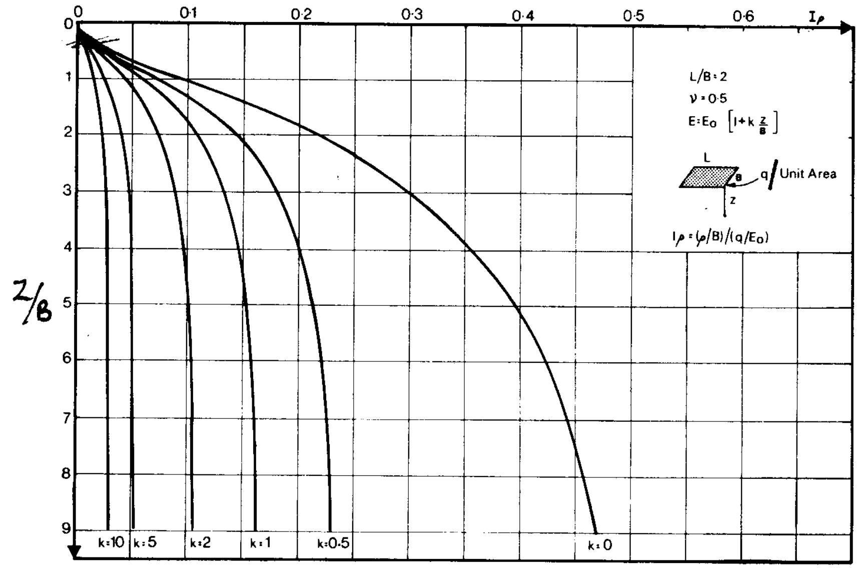

Fig. 5

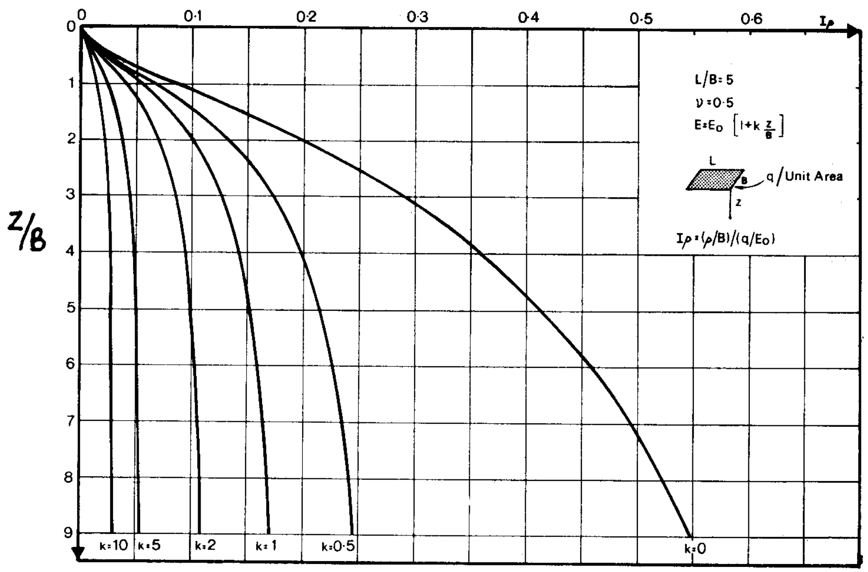

Fig. 6

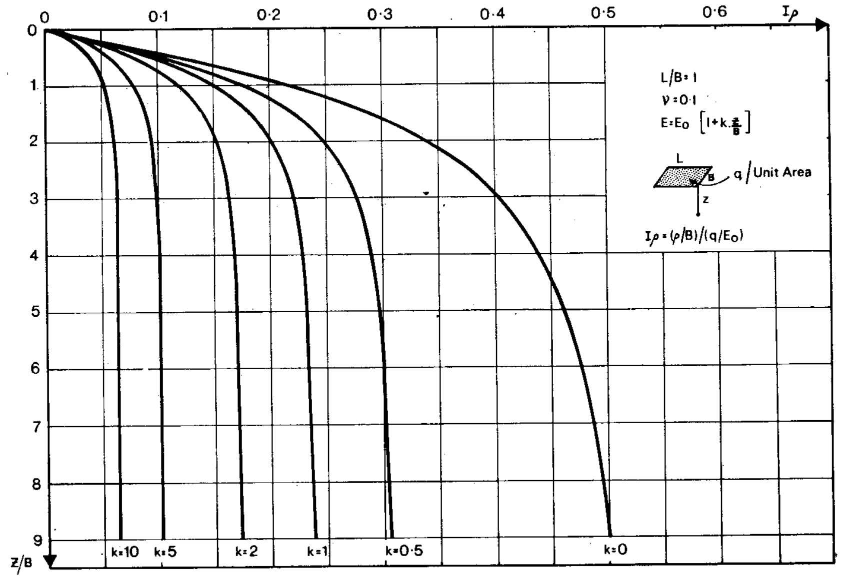

Fig. 7

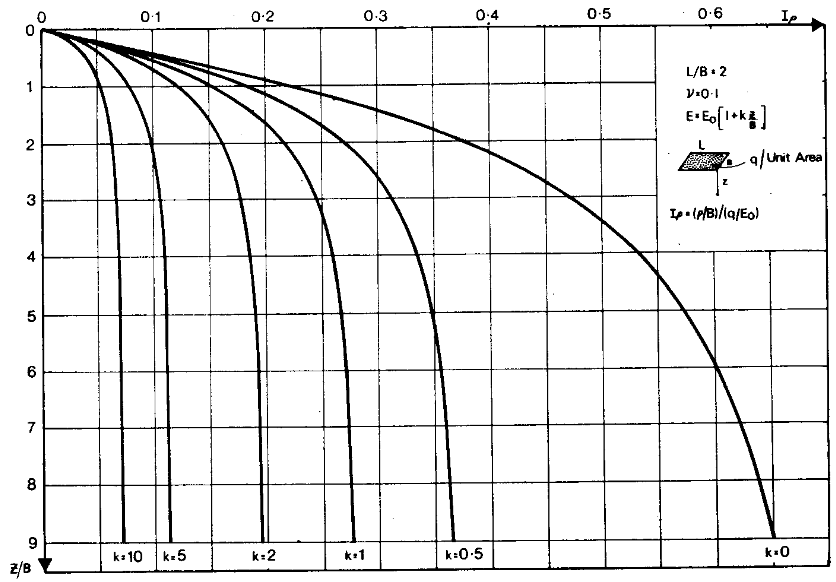

Fig. 8

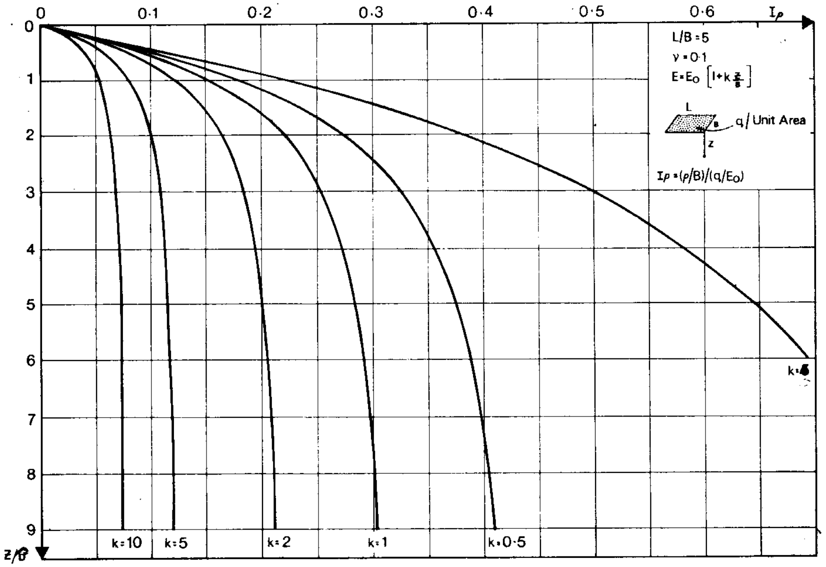

Fig. 9

This expression has been programmed, using a Hewlett Packard computer to provide values of $I\rho$ for various thicknesses of layer (expressed in terms of breadth b) for a range of values of $K\,[\delta E/\delta(z/b)]$ for rectangular shapes with $L/B = 1.0$, 2.0 and 5.0 and with $\nu = 0.5$ and 0.1, for the undrained and drained case respectively (*Figs. 4 to 9*).

Since these curves are the result of an approximation it was decided to attempt to assess their accuracy by means of a check against an axi-symmetric case solved by the finite element method. A circular flexible and rough footing radius (B) was modelled, founded on the surface of an elastic half-space of varying depth (Z) with a rigid rough base. With constant E_u

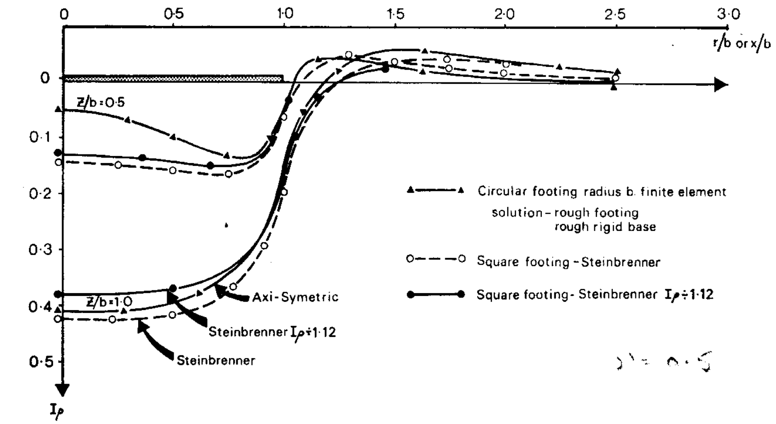

Fig. 10. Comparison between Steinbrenner and axisymmetric finite element analysis, $K = 0$

(i.e. $k = 0$), Poissons ratio $= 0.5$ and $Z/B = 0.5$ and 1.0, values of $I\rho$ defining the deflected shape of the surface were obtained. Similar values of $I\rho$ were obtained for a square footing from the $k = 0$ curve in *Fig. 4*. Using the ratio of influence factors for square and circular footings $= 1.12$, i.e.

$$I_{\rho\ \text{circ.}} = \frac{I_{\rho\ \text{square}}}{1.12}$$

a 'corrected' curve for a circle was obtained for comparison with that obtained by the finite element method. The results of this comparison are shown in *Fig. 10*. It can be seen that the error is no more than 8% for $Z/B = 1.0$ but this rises significantly for $Z/B = 0.5$. Additionally, the definitive values computed by Brown and Gibson (1973) provide a special case where the Z/B in the curves in *Figs. 4 to 9* tend to infinity. The curves show agreement within 10% with Brown and Gibson's calculated values.

Estimation of E_u

The direct determination of E_u from laboratory testing is subject to very large and virtually inescapable errors (Ward, Samuels and Butler (1959), Marsland (1971)) and attention is now focused primarily upon field testing and observation (Burland, Butler and Dunican (1966), Marsland (1971), Ward (1971), Marsland (1973)).

Cooling and Skempton (1942) first suggested from laboratory test data and later Skempton and Henkel (1957) substantiated a relationship $E_u = 140\ C_u$ from a limited number of case histories of settlement in London Clay. More recently Wroth (1971), reconsidering the published data of Bishop, Webb and Lewin (1965) and Webb (1966) derived a relationship $E_u = 150\ C_u$.

The rapid development of numerical analysis has significantly improved our ability to interpret the results of field observations (Burland (1966), Cole and Burland (1972), Hooper (1973)).

Marsland has reported on the results of an extensive series of tests on the London Clay by a constant-rate-of-penetration testing on large diameter plates at the base of lined and unlined shafts. A summary of some of his results from a site at Hendon is shown in *Table 1*.

Table 1

C_u (kN/m^2)	E_u (MN/m^2)	E_u/C_u
80	55	690
90	75	830
110	65	600

Earlier plate bearing tests reported by Burland, Butler and Dunican (1966) at Moorfields are shown in *Table 2*. These results indicate far higher ratios of E_u/C_u than have been obtained in the laboratory. However it has been shown that the modulus is very sensitive to the length of time between excavation and testing. For plate loading tests at Ashford Common, Marsland (1973) showed that 10 hr after excavation the modulus had halved.

The impossibility of performing a field test in truly undrained and undisturbed conditions and the difficulties of obtaining representative undisturbed samples for laboratory testing leaves the results of both methods open to question. Certainly from the behaviour of London Clay in the mass a value of E_u/C_u between these two extremes seems more likely.

More recently Hooper (1973) has reported the results of *in situ* measurements of pile load, raft pressure and settlement observations on the Hyde Park Cavalry Barracks. By relating his observations to a finite element analysis of the problem, Hooper established by trial and error a relationship $E_u = 10 + 5.2Z$ where Z = depth below ground surface. Comparing this expression in E_u to the measured values of C_u from the site we obtain the relationship between E_u and C_u tabulated in *Table 3*.

Table 2

C_u (kN/m^2)	E_u (MN/m^2)	E_u/C_u
140	100	800
300	200	650

Table 3

C_u (kN/m^2)	E_u *(computed)* (MN/m^2)	E_u/C_u
170	530	310
256	1050	410
328	1560	480

A back-analysis of the movements recorded in the basement wall of Britannic House (Cole and Burland (1972)) using a finite element mesh and assuming plane-strain conditions showed a value of $E_u/K_{ot} = 295$. Assuming $K_{ot} = 2.0$ and for an average shear strength ($U4$ samples) of 190 KN/m^2 gives

$$\frac{E_{uIII}}{C_{u(U4)}} = \frac{2.0 \times 295 \times 1000}{190} = 310$$

Pseudo-plane-strain and triaxial tests on specimens cut from block samples from shafts at Barbican and $U4$ samples from Swiss Cottage (Ove Arup & Partners (1971)) showed the relationship

$$\frac{E_{uIII}}{E_{uI}} = 1.4$$

Hence, from Cole and Burland (1972),

$$\frac{E_{uI}}{C_{uU4}} = \frac{310}{1.4} = 222$$

Some corroboration for this relationship is drawn from the ratio of E_{uI} to undrained shear strength of block samples of 180 obtained in particularly carefully executed sampling and testing at the Building Research Station. In this case, the value of undrained strength of block samples to that of $U4$

samples was found to be about 1.3. Thus the ratio of E_{uI} to undrained strength of $U4$ samples would be about $1.3 \times 180 = 234$.

Estimation of E_v'

For the purpose of estimating total settlements we are concerned with the relationship between C_{uU4} and E_v'.
From the foregoing, let us begin by assuming:

$$E_{uI} = 220\ C_{uU4}$$

Atkinson (1973) from extensive laboratory tests showed $R = 2.0$, $\nu_{hv} = 0.19$. Hence, from Equation 2,

$$E_{uI} = 1.67\ E_v'$$

Thus,

$$E_v' = \frac{220}{1.67} C_{uU4} = 130\ C_{uU4}$$

This compares with Wroth (1971) who quotes $\nu_{hv} = 0.12$, from which Henkel (1971) deduces $R = 1.6$ and hence $E_{uI} = 1.58\ E_v'$ which would give

$$E_v' = \frac{220}{1.58} C_{uU4} = 139\ C_{uU4}$$

Constructional history

In considering case histories of settlement, due account must be taken of the constructional history of the foundation since this may significantly affect its settlement behaviour. For example, let us suppose that a tall building has a two-storey basement and is supported on a piled raft (*Fig. 11*).

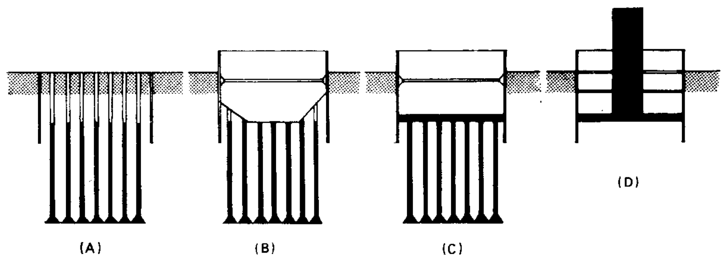

Fig. 11. Construction sequence

Commonly the piles are placed from ground level, with empty bore above raft level as in (a): the excavation process for the piles tends to release lateral pressure which may cause settlement of the site (Burland (1973) and Appendix B) during this stage, and will certainly inhibit heave when the excavation takes place, *Fig. 11* (b). A continuous raft anchored by piles, *Fig. 11* (c), will also inhibit heave effects; the swelling pressures developed under the raft give rise to tension in the piles (Hooper 1973) and the soil is effectively 'pre-stressed' until a net increase in load is applied to the foundations.

Without the piles heave effects would be greater, beginning during stage (b), through stage (c) and into stage (d) until attenuated as a net increase in load is approached. If the basement is much larger in plan than the building above, both initial and long term unloading effects may greatly complicate the settlement behaviour. Thus the response of an element of soil immediately beneath foundation level will be greatly influenced by the sequence of construction which is chosen (*Fig. 12*). The monitored settlement behaviour will

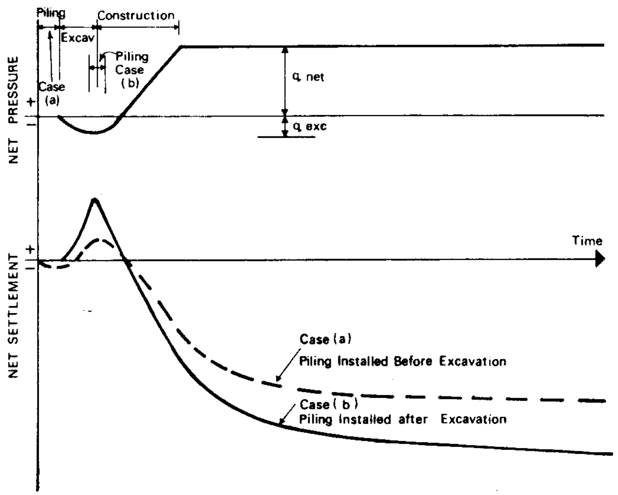

Fig. 12. Foundation settlement: effect of construction sequence

also obviously depend upon the point in time at which the observation points were installed. It follows that it is necessary to examine case histories very closely before applying any form of re-analysis.

The case histories presented to the Conference

The papers which have been submitted in this Session provide us with case histories in a wide variety of overconsolidated clays. The range includes clays with various modes of formation including continuous marine deposition (London Clay, Gault Clay, Woolwich and Reading), glacial action (boulder clay), periglacial deposition (flintz), diluvial clays (Nagoya, Japan).

London, Woolwich and Reading and Gault Clays

The papers by Green and Cocksedge, Hyde and Leach, Morton and Au, and Mould, provide a large and valuable contribution to the available data