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This paper considers the application and interpretation of the Standard Penetration Test in the wide variety of soils and rock encountered in the UK. In particular, attention is focused on what information the SPT can provide on the strength and stiffness of these materials. Correlations are provided for overconsolidated sands and gravels, normally consolidated sands, overconsolidated clays, weak rocks and Chalk. The importance of ϕ'_{cv} in the interpretation of the strength of granular materials is demonstrated, as is the importance of strain dependency in the understanding of stiffness in all materials.

INTRODUCTION

The Standard Penetration Test is like the friend you've known for a long time: maybe a little taken for granted; there providing support when all else fails; given to frustrating habits. Neither is above criticism, but both perhaps get criticised more than they should.

Of course the SPT can be done badly. But we should not forget that ground is naturally variable. We should not necessarily expect tidily bunched data. The scatter of SPT results in Thames gravel is more likely to be a statement of reality in the ground than a foible of the test procedure.

In developing our correlations it is important to relate SPT data to fundamental soil properties if we can. It is more useful to relate N values to shear strength, for example, than to relate them directly to shaft friction of piles. It is more useful to relate N values to soil stiffness than directly to settlement. In both cases if we understand the relationship of N to the basic parameters we can extend this application to a whole variety of different geotechnical situations.

This report attempts to draw together and to develop some of the experience contained in the papers presented to this Conference, together with past work. Correlations of N value with two basic parameters perhaps most widely used in practice, strength and stiffness, will be examined for each of the following materials commonly found in the UK:

a) sands and gravels
b) clays
c) insensitive weak rocks
d) Chalk.

One of the principal advantages of the SPT is that it has something useful to tell us about all these materials and since three out of four of them can commonly be found in one borehole, the SPT has a big advantage over other less versatile forms of in situ testing. In some materials, such as many glacial tills, the SPT is the only in situ test which can readily and economically be relied on.

SANDS AND GRAVELS

Free draining granular materials are traditionally the materials most often tested by SPT but they also provide data which give rise to the greatest debate and criticism. The work of Burland and Burbidge (1985) is an important step forward in clarifying many of the issues. There remains, however, the difficulty of understanding the physical significance in foundation performance of their compressibility index I_c, upon which their correlations depend, defined as the inverse slope of the pressure/settlement curve divided by $B^{0.7}$, where B is the footing breadth, i.e., $I_c = \dfrac{\Delta\rho}{\Delta q} \times \dfrac{1}{B^{0.7}}$.

It may be helpful to look at a few basic relationships. Imagine a body of sand with constant relative density, such as that represented in Figure 1a. For this material SPT N values will increase with depth as indicated in Figure 1b, because N is a function of current mean effective stress level (Clayton, Hababa and Simons, 1985). Stiffness, represented by elastic Young's Modulus E' is also a function of mean effective stress level and in such a material gives a similar variation with depth as N (Figure 1c). Thus if we were to look for correlations:

a) with relative density, we would want to correct the N value for depth to give one characteristic value while

b) with stiffness E', we would look for relationships with N uncorrected for depth.

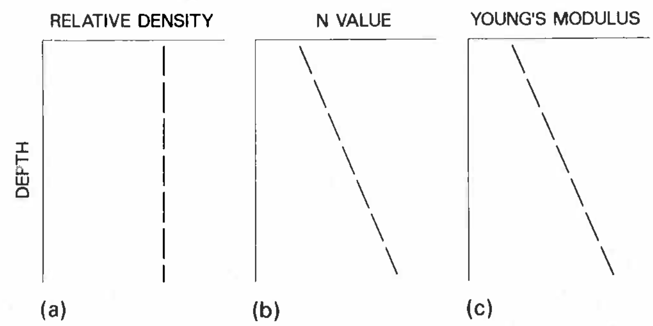

Fig 1 Sands and gravels: basic relationships

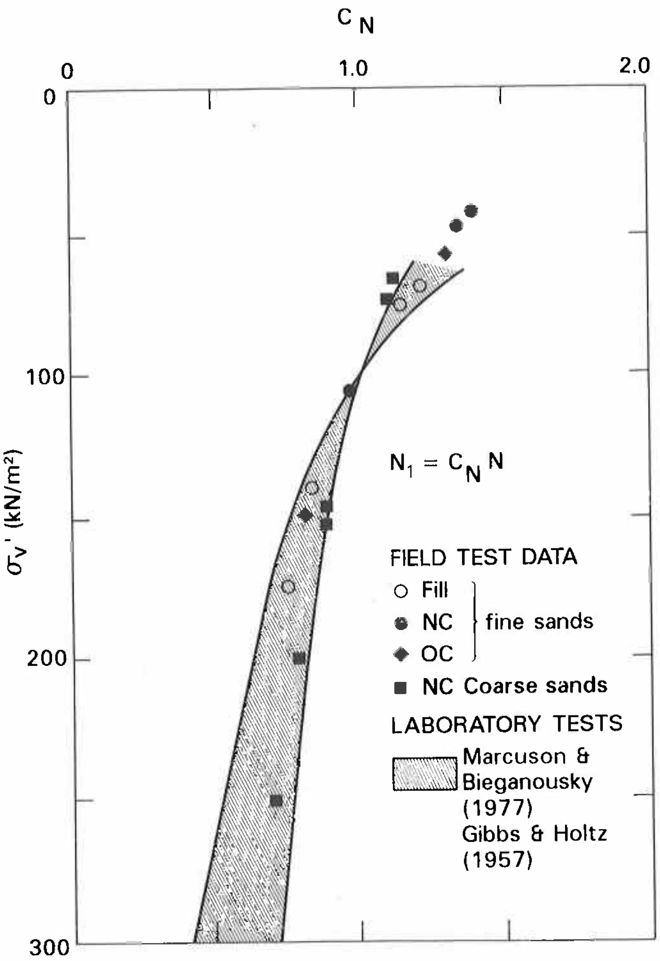

Fig 2 Correction for overburden pressure (after Skempton, 1986)

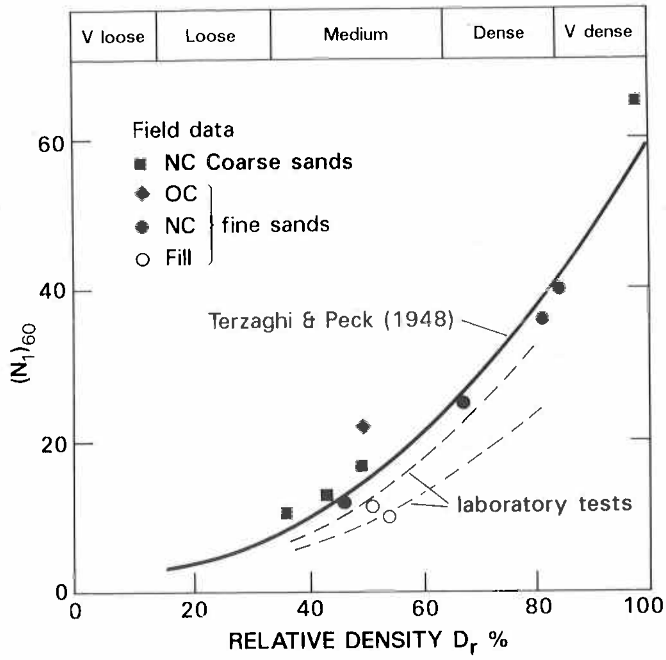

Fig 3 Effect of relative density, based on field data (after Skempton, 1986)

Strength

Let us first look at the parameters of strength which emerge from an appreciation of relative density.

We first need to correct for overburden pressure in the way which is well known, relating N to the corrected value, N_1, appropriate to a vertical effective stress of 100kN/m², using the expression $N_1 = C_N N$.

Figure 2 summarises the available field data collected by Skempton (1986) together with the variation of laboratory test data produced by Marcuson and Bieganousky (1977) and Gibbs and Holtz (1957). The plot indicates that for normally consolidated sands the variation of C_N with overburden pressure is relatively insensitive to grading. Limited field and laboratory data suggests that the effect of overconsolidation on C_N is also small.

Skempton (1986) pointed out that the original correlation between descriptions of relative density and N value proposed by Terzaghi and Peck (1948) should properly be corrected for the energy levels used in modern SPT practice. He arrived at the relationship shown in Figure 3 of relative density, D_r, against the standardised* SPT value, $(N_1)_{60}$. This relationship correlates well with the field data for normally consolidated sands of fine and coarse grading. Data for fill and for laboratory tests fall below this line.

This same relationship is shown in Figure 4 as the full line curve. Also shown is the correlation with Ø' proposed by Peck, Hanson and Thornburn (1953), modified as a result of using $(N_1)_{60}$.

If penetration resistance in a sand of given relative density is controlled by the mean effective stress as the work of Clayton et al (1985) suggests, then it is to be expected that the relationship in Figure 4 will be different for overconsolidated materials. Using working similar to that used by Skempton (1986) it can be shown to a first approximation (see Appendix A) that the relationship between relative density and $(N_1)_{60}$ varies with overconsolidation ratio as indicated in Figure 4. For a given value of $(N_1)_{60}$ it is evident that the effect of overconsolidation ratio on Ø' is more significant for dense materials than for loose.

An alternative method of estimating Ø' was proposed by Cornforth (1973) in which the critical state value of the angle of friction $Ø'_{cv}$ was first measured by static angle of repose tests. To this was added the dilatancy component $Ø'-Ø'_{cv}$ which was found to vary with relative density.

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* Modern UK practice using the automatic trip monkey gives N values equivalent to the standard N_{60} without correction. $(N_1)_{60}$ values are obtained by correcting for overburden using Figure 2.

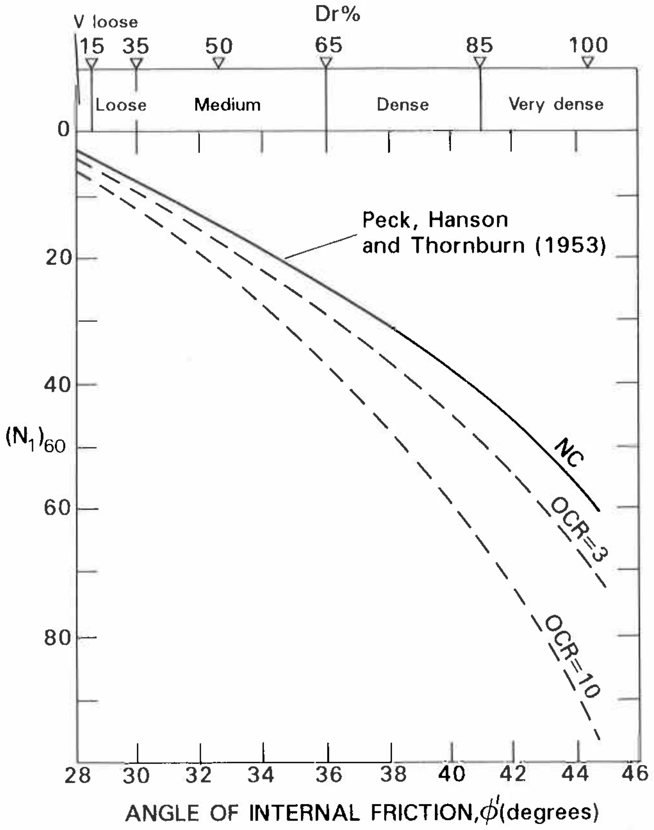

Fig 4 Effect of overconsolidation ratio on the relationship between $(N_1)_{60}$ and angle of friction Ø'.

Bolton (1986) collected together data for 17 different sands. Figure 5 shows the variation of $Ø'-Ø'_{cv}$ with relative density for a mean effective stress at failure in the range 150-600kN/m²**. Plane strain values were higher than triaxial values as would be expected. Bolton reported that values of $Ø'_{cv}$ varied from about 33° for the quartz sands to 37° for sands containing a significant proportion of feldspar.

Taking a value of $Ø'_{cv}$ = 33° relevant to quartz sands and assuming that the relationship between $(N_1)_{60}$ and relative density in Figure 3 is appropriate to quartz sands with $Ø'_{cv}$ = 33°, then a relationship between $(N_1)_{60}$ and Ø' can be obtained as plotted in Figures 6a and 6b for triaxial and plane strain configurations respectively. There is some evidence to suggest that for a given material $Ø'_{cv}$ in plane strain is a little higher than for triaxial loading. However, the conservative assumption is made here that they are the same. Also shown for comparison in Figure 6 is the relationship between $(N_1)_{60}$ and Ø' from Peck Hanson and Thornburn, replotted from Figure 4.

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Because of the curvature of the failure envelope, Ø' is here measured as a secant value. Ø' will be higher at lower stress levels and lower at higher stress levels (see Bolton, 1986).

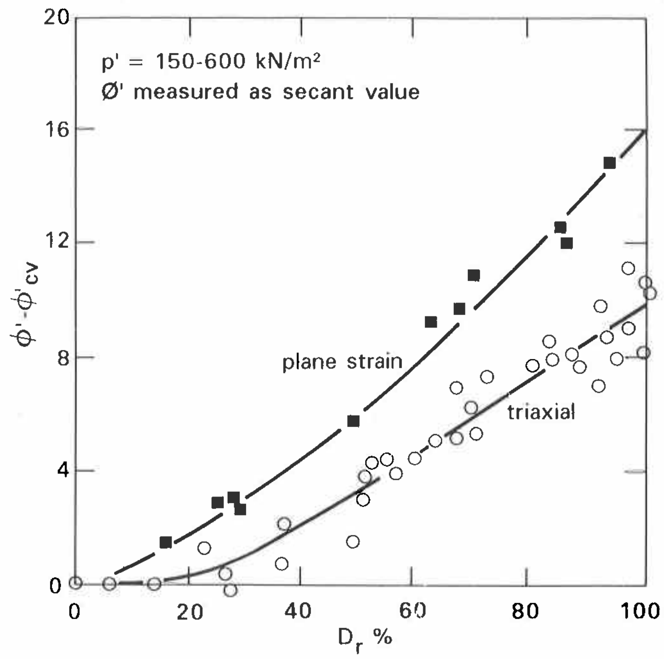


Fig 5 Variation of $Ø'-Ø'_{cv}$ with relative density (after Bolton 1986)

It is evident from Figure 6a that the relationships derived from triaxial testing are in broad agreement with the Peck, Hanson and Thornburn results, although these latter underestimate Ø' at low $(N_1)_{60}$ values. In plane strain, however, as would be appropriate for retaining wall design for example, the Peck, Hanson and Thornburn results significantly underestimate Ø'.

The correlations are clearly very sensitive to the value of $Ø'_{cv}$ appropriate to the material in question. Values of $Ø'_{cv}$ found in the literature are presented in Table C1 (Appendix C) for uniformly graded and well graded materials where description of particle shape are available. The variation of $Ø'_{cv}$ with particle shape is plotted in Figure 7 from this data for triaxial testing. Typical $Ø'_{cv}$ values may be summarised as follows:

<u>uniformly graded</u>	quartz	feldspar
rounded	30°	
sub rounded sub angular	32°	
angular very angular	34° 36°	 39°
<u>well graded</u>		
sub rounded angular	36° 38°	

It is clear from this range of values that at low relative densities, there will not be a unique relationship of $(N_1)_{60}$ with Ø'. An $(N_1)_{60}$ value of less than about 5, for example, will indicate a generally loose or very loose material, but will say nothing about its strength which could be anywhere in the range

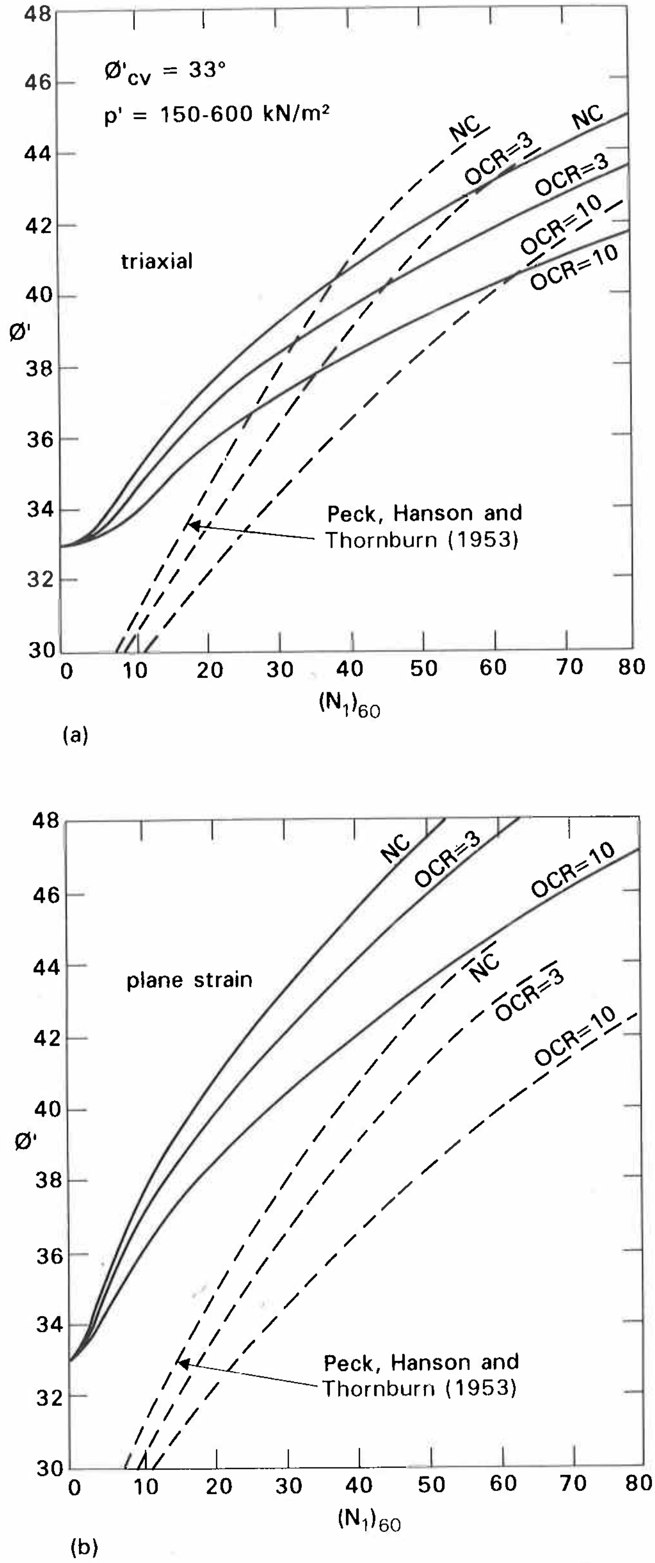


Fig 6 Relationship between $(N_1)_{60}$ and $\emptyset'$ for materials with $\emptyset'_{cv} = 33°$

30°-40°. This is probably because in very loose sands the SPT readily breaks down the metastable sand structure and the local confining pressures are greatly reduced leading to low N values. For loose materials consideration of $\emptyset'_{cv}$ is thus more important than N value. At higher relative densities, however, and for a given overconsolidation ratio it is probable that $(N_1)_{60}$ is

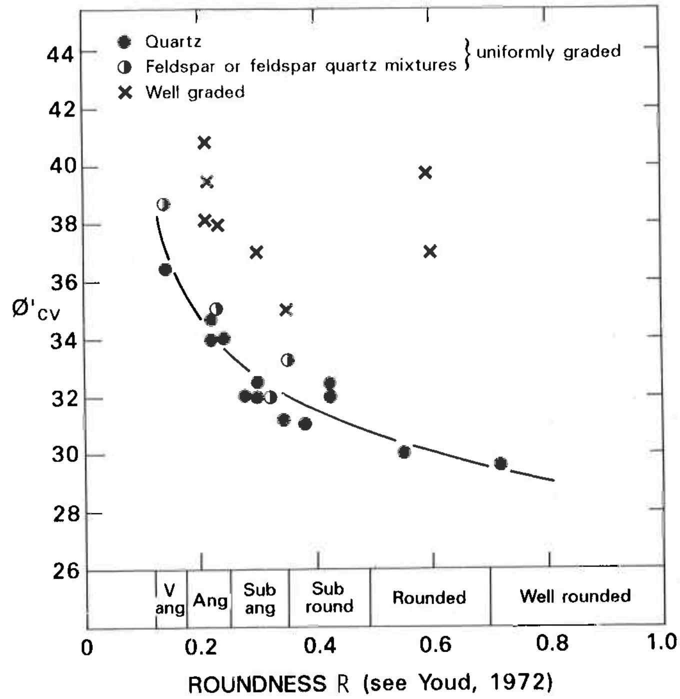


Fig 7 Relationship between particle shape and $\emptyset'_{cv}$ based on triaxial tests

proportional to the bearing capacity factor, N_q, and thus is uniquely related to $\emptyset'$.

This possibility is explored in Figure 8a where the $(N_1)_{60}$ v. $\emptyset'$ curve for normally consolidated materials under triaxial loading and having $\emptyset'_{cv} = 33°$ has been replotted from Figure 6a. Also shown is a curve AA of N_q against $\emptyset'$ from Berezantsev (1961) with the horizontal scale adjusted to provide the best fit with the SPT curve. The fit is close over much of its length supporting the view that $(N_1)_{60}$ is proportional to N_q at moderate to high relative densities. Tentative curves are drawn for materials with $\emptyset'_{cv}$ = 31°, 35° and 37°. The effect of overconsolidation is indicated in Figures 8b and 8c for over-consolidated ratio of 3 and 10 respectively. Similar curves could be drawn for plane strain loading.

The pattern of behaviour identified in Figure 8 has implications for the relationship between $(N_1)_{60}$ and relative density given in Figure 3. Taking each of the curves in Figure 8 and using the relationship between $\emptyset'-\emptyset'_{cv}$ and relative density shown in Figure 5, it is possible to construct curves of $(N_1)_{60}$ against relative density shown in Figure 9 for materials with different $\emptyset'_{cv}$.

Evidently the value of $\emptyset'_{cv}$ has a significant effect on the relationship between $(N_1)_{60}$ and relative density.

Corroborative data are hard to find but some indicators are given in Figure 9b where relative density has been measured in the field. Well graded gravels and sands were investigated by Yoshida et al (1988) giving average $(N_1)_{60}$ values in the range 50 to 60 for average relative density in the range 65 to

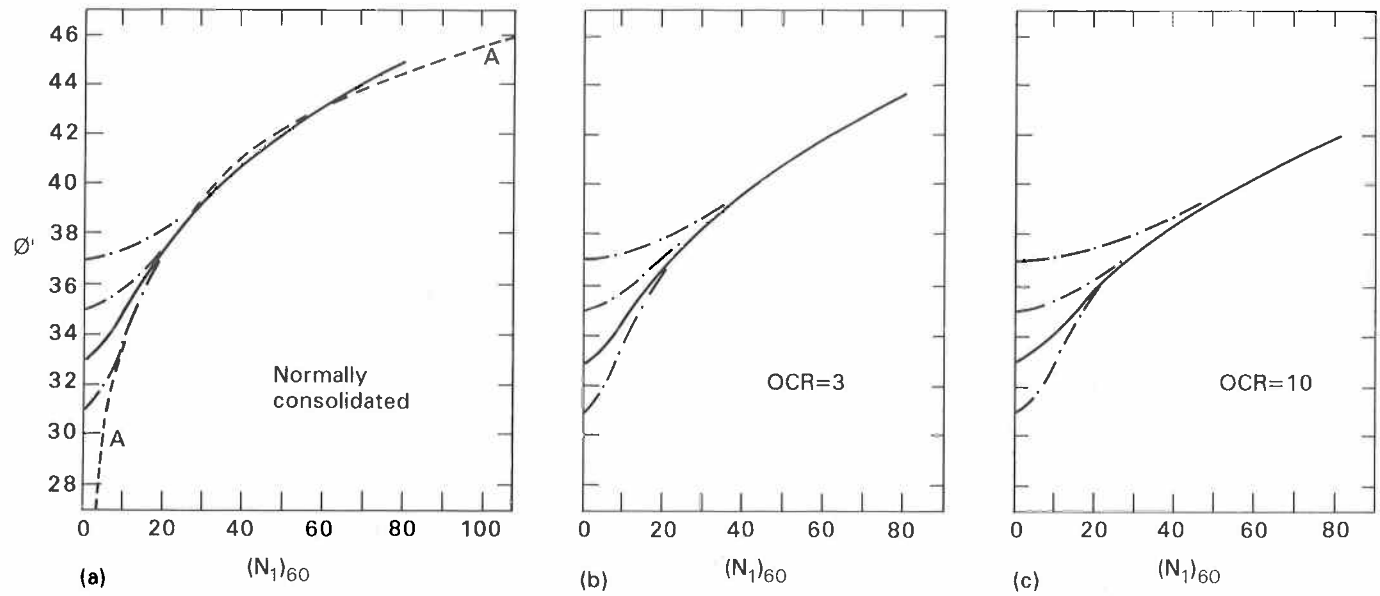


Fig 8   Variation of $Ø'$ and $(N_1)_{60}$ with $Ø'_{cv}$ and OCR

75%. No strength values were quoted, but a value of $Ø'_{cv}$ in the region of 36° would not be unreasonable for these well graded materials.

Data for normally consolidated sands are taken from Figure 3 and Skempton (1986). Judging by the maximum and minimum voids ratios for these sands (e_{max} = 1.0 to 1.2, e_{min} = 0.56 to 0.75) they are likely to be angular to sub angular in particle shape (see Table C1 and Youd, 1972). Thus the value of $Ø'_{cv}$ = 33° chosen for the line linking this data in Figure 3 and Figure 9a is likely to be appropriate.

Data for the heavily overconsolidated Norwich Crag sands in Figure 9c are also taken from Skempton (1986). Also shown is a point for the overconsolidated Bagshot sands provided in the paper to this conference by Barton et al (1988), for which $(N_1)_{60}$ = 85 and D_r = 88%. A $Ø'_{cv}$ of 34° is indicated which again is not unreasonable for this angular uniformly graded material.

Laboratory tests carried out by Yoshida et al (1988) on normally consolidated material showed $(N_1)_{60}$ values 35% higher for moderately well

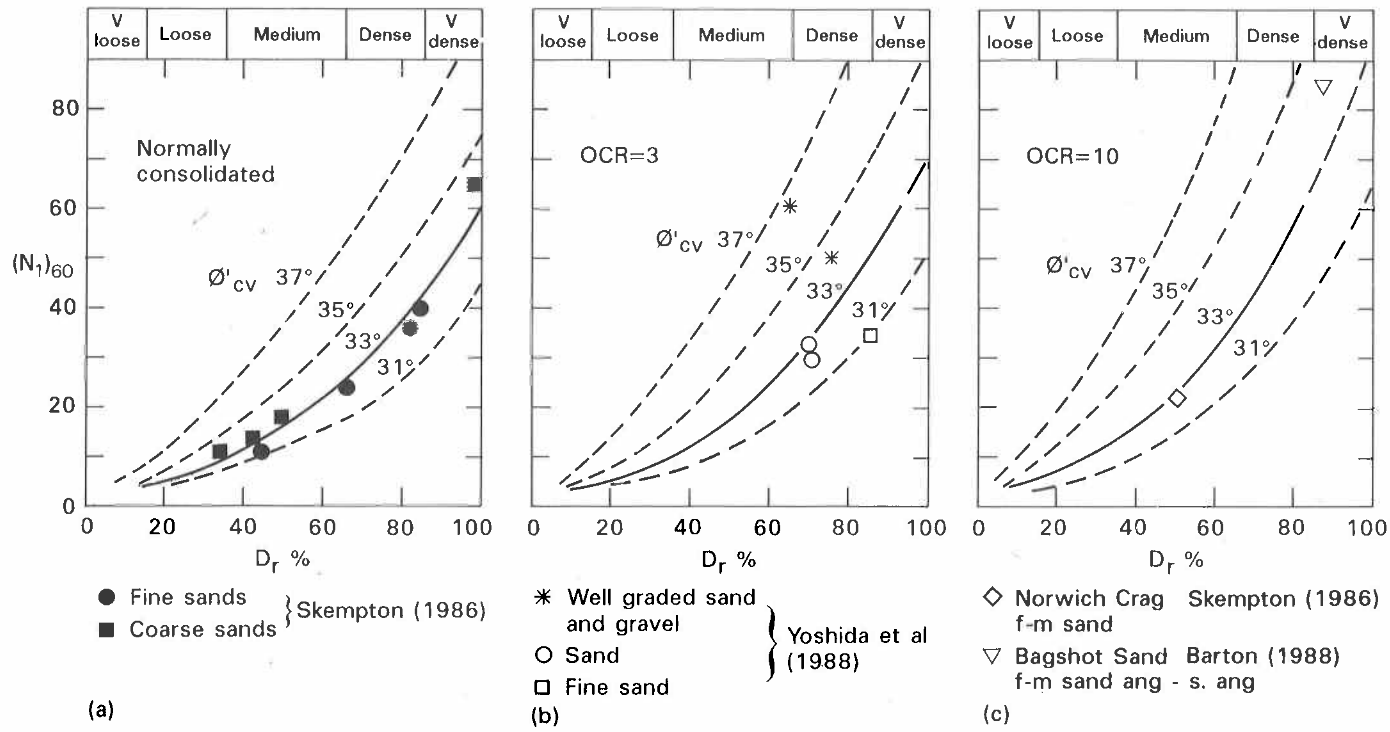


Fig 9   Variation of $(N_1)_{60}$ and relative density with $Ø'_{cv}$ and OCR

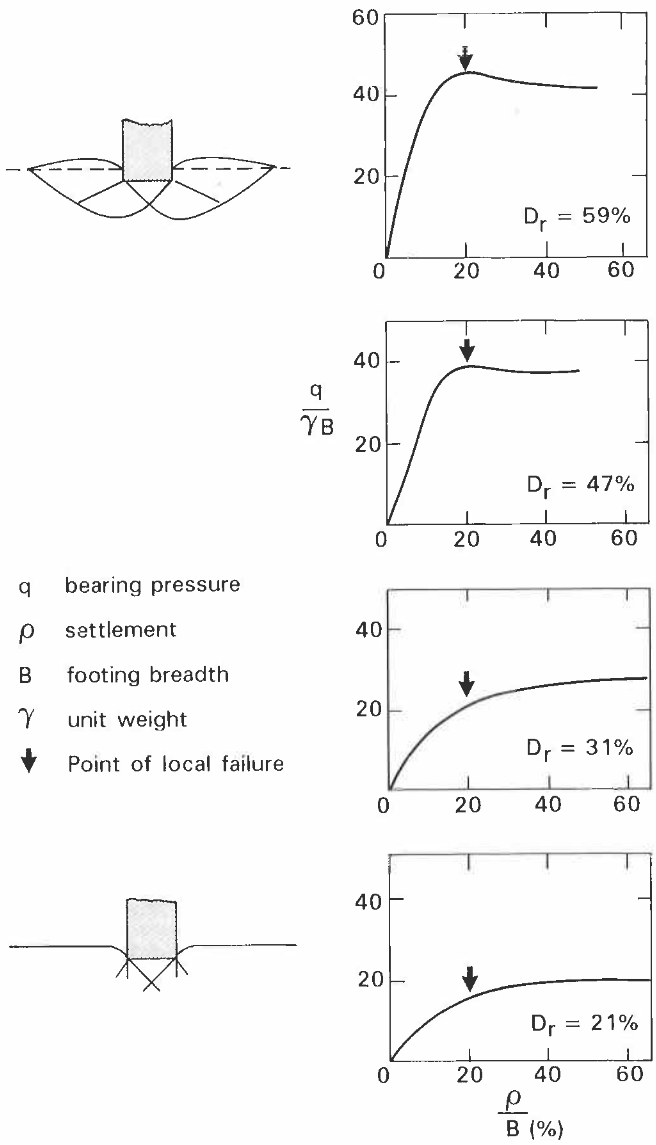


Fig 10 Behaviour of model footing on sand at various relative densities (after Vesic, 1973. B = 38mm)

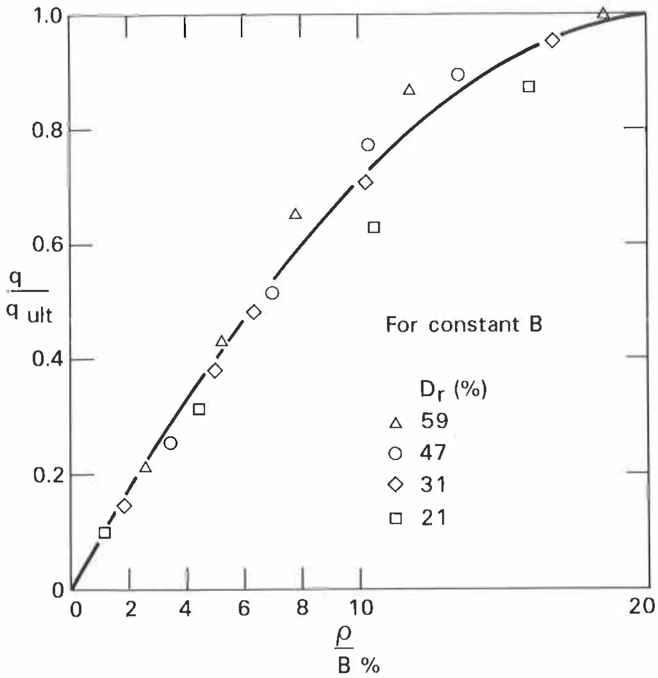


Fig 11 Normalised plot of settlement against bearing stress, for model footing tests by Vesic.

graded gravel than for uniform fine sand at the same relative density. Similarly, Holubec et al (1972) found for model penetration tests carried out in the laboratory on normally consolidated sands, that angular sands with ϕ'_{cv} = 34° showed N values twice those for rounded sands with ϕ'_{cv} = 30° at the same relative density and depth. A very similar pattern is evident in Figure 9a.

More field data is required to confirm the sensitivity of the relationships in Figures 8 and 9 to ϕ'_{cv} and overconsolidation ratio.

Stiffness

The prediction of settlement of footings on granular materials involves estimating stiffness. It is now widely accepted that stiffness in many materials is strain dependent, the stiffness at small strains being greater than at larger strains. A practical difficulty then arises of how to estimate strain level in a useful way in a loaded foundation.

A possible approach is suggested by tests carried out by Vesic (1973) on a model footing of constant breadth in sands of varying density. The results of Vesic's tests are summarised in Figure 10 where normalised bearing pressure - settlement plots are shown for four relative densities. The point at which local shear failure was observed to occur is indicated on each plot. Interestingly this point occurs at approximately the same settlement in each case. Figure 11 shows the data from each of the tests plotted on the same graph of q/q_{ult} against settlement where q_{ult} is the ultimate bearing stress at the point of local failure. It is evident that to a first approximation there is a unique relationship between "degree of loading" q/q_{ult} and settlement for varying density.

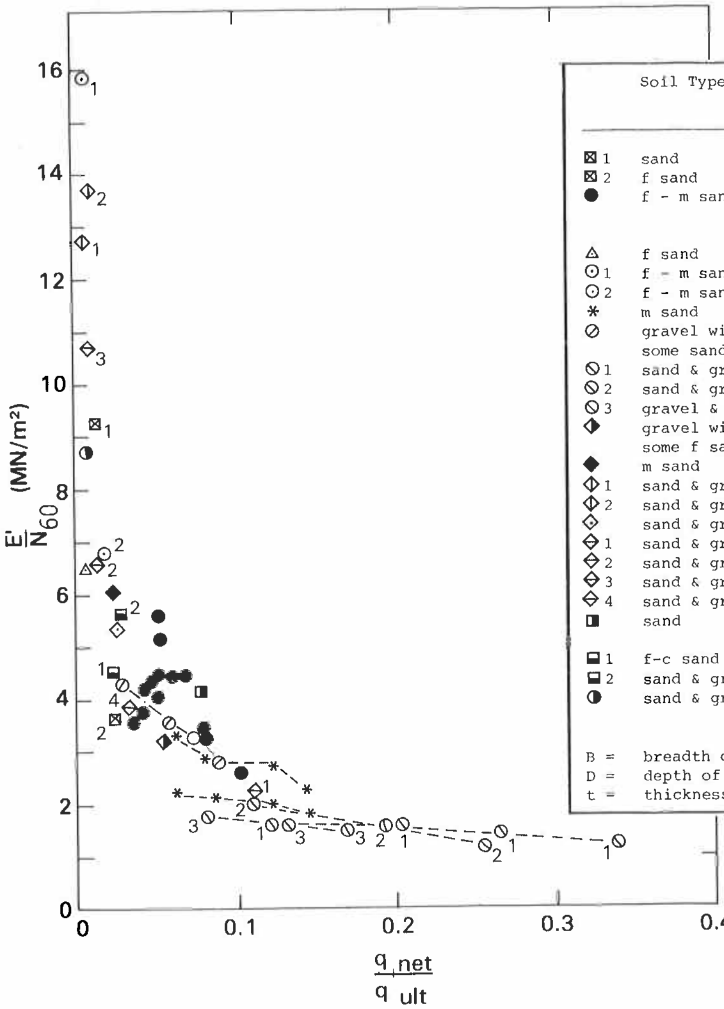
This suggests that q/q_{ult} is an indirect measure of shear strain.

In footing design q is known and q_{ult} can be readily estimated using bearing capacity factors incorporating an allowance for local failure, such as those in Figure B1 (see Appendix B).

We have seen that both stiffness E' and N vary with mean effective stress level in the ground. It may therefore be fruitful to consider the ratio E'/N_{60} and its variation with strain level or degree of loading q/q_{ult}.

Figure 12 shows data from a wide range of spread footings, raft foundations and large scale plate tests on overconsolidated sands.

The data is taken from those case histories referred to by Burland and Burbidge (1985)



	Soil Type	N_{60}	B (m)	D (m)	t (m)	q_{net} (kN/m²)	Reference
⊠ 1	sand	14	37	0	>12	175	Baker (1965)
⊠ 2	f sand	11	12	1	7	130	"
●	f - m sand	14-21	3-6.7	av3	8	av165	D'Appolonia et al (1968) each point represents av. of 20 footings
△	f sand	44	55	9	30	100	Dunn (1974)
⊙ 1	f - m sand	17	22	5	>50	82	Garga et al (1974)
⊙ 2	f - m sand	17	22	5	>50	154	"
*	m sand	26	3	1.5	9	<475	Greenwood et al (1970)
⊘	gravel with some sand	30	4	5	7	<465	Levy et al (1974)
⦸ 1	sand & gravel	40	1.2	0.5	4	<850	"
⦸ 2	sand & gravel	24	0.9	1.2	6	<630	"
⦸ 3	gravel & sand	16	0.9	3	6	<600	"
◈	gravel with some f sand	10	1.0-1.5	1.2	-	78	Meigh et al (1961) av. of 3 footings
◆	m sand	48	10	1.5	12	212	Ronan (1980)
◇ 1	sand & gravel	16	18	11	>37	47	Schultze (1962)
◇ 2	sand & gravel	11	16	7	>23	97	"
◇	sand & gravel	21	15	3.5	22	192	Schultze (1963)
◇ 1	sand & gravel	30	2.5	1	5	282	Schultze et al (1973)
◇ 2	sand & gravel	14	18	11	14	100	"
◇ 3	sand & gravel	12	16	7	12	132	"
◇ 4	sand & gravel	24	14	3.5	28	191	"
◧	sand	>40	2	av2.2	-	av510	Thorne (1970) av. of 7 footings
⬒ 1	f-c sand	10	30	2.7	20	310	Tschebotarioff (1951)
⬒ 2	sand & gravel	51	3.8	2.5	17	310	"
◑	sand & gravel	17	20	5	>10	161	Stuart et al (1974)

B = breadth of foundation
D = depth of embedment
t = thickness of stratum beneath foundation

Fig 12 Variation of E'/N_{60} with degree of loading for overconsolidated sands and gravels

where Standard Penetration Tests were carried out. The data relates to early American or UK practice for which the SPT rod energies were lower than they are now. Consideration of the data presented by Skempton (1986) suggests that for modern UK practice N_{60} will be lower by a factor of about 0.8. The N values given in the case histories have therefore been reduced by 80%. The value of q_{ult} has been calculated in each case using N_{60} values corrected for overburden to give $(N_1)_{60}$. Values of $\emptyset'$ were then chosen using Figure 8 together with bearing capacity factors appropriate to local shear failure as described in Appendix B. The bearing pressure used, q_{net}, is the average net effective bearing pressure acting on the foundation. The value of E' has been estimated from the data given in the case histories using linear elastic theory and is thus the average secant stiffness beneath the foundation under loading q_{net}. Further details of the assumptions made are given in Appendix B.

Working foundations generally were found to have values of q_{net}/q_{ult} less than about 0.1, while footing tests and large plate tests with breadths in the range 1 to 3m were taken to higher degrees of loading, giving q_{net}/q_{ult} in the range 0.1 to 0.4. Data for the larger raft foundations give low degrees of loading and corresponding high values of E'/N_{60}.

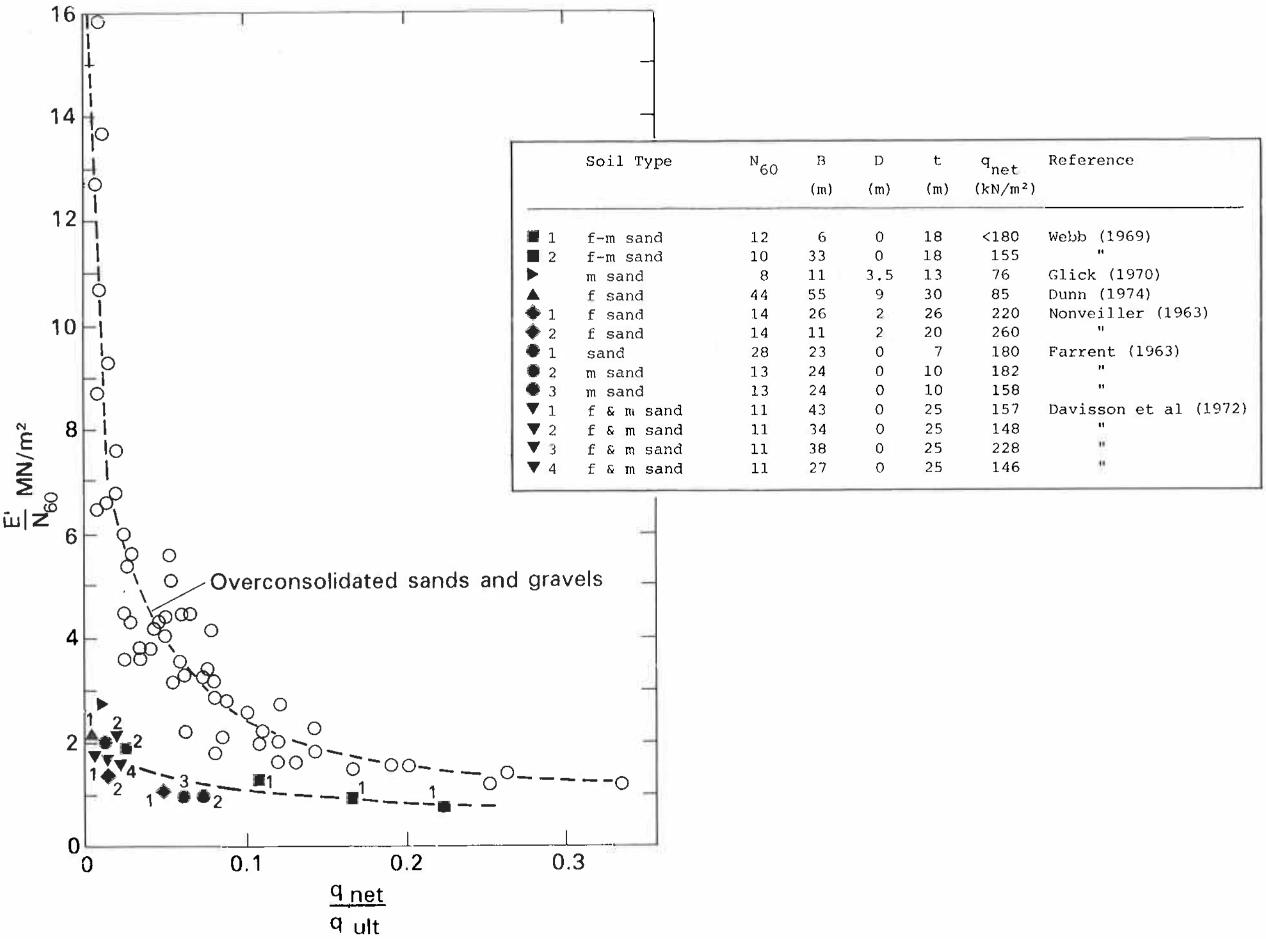
It is evident that the relationship between E' and N_{60} is strongly strain dependent. It is little wonder that the search for simple relationships between E' and N in the past has proved so frustrating in the absence of consideration of strain or degree of loading.

The data in Figure 12 have been replotted in Figure 13 and a mean trend line has been added.

Also plotted in Figure 13 are the data from case histories of structures on normally consolidated sands. Here the observed behaviour is somewhat different. While the data are rather more limited they suggest that stiffness is significantly less affected by shear strain in these materials, with the ratio of E'/N_{60} decreasing from about 2MN/m² to 1MN/m² as loading increases. For values of q_{net}/q_{ult} in excess of about 0.1 the stiffness of normally consolidated sands is roughly half that given by overconsolidated sands.

Corroboration from Laboratory Tests

Some corroboration of the trend in overconsolidated materials can be found in the field of soil dynamics. Let us first consider behaviour at very low strain levels. Figure 14 shows the variation of small strain shear stiffness G_o with N value as found by Imai and



	Soil Type	N_{60}	B (m)	D (m)	t (m)	q_{net} (kN/m²)	Reference
■ 1	f-m sand	12	6	0	18	<180	Webb (1969)
■ 2	f-m sand	10	33	0	18	155	"
►	m sand	8	11	3.5	13	76	Glick (1970)
▲	f sand	44	55	9	30	85	Dunn (1974)
◆ 1	f sand	14	26	2	26	220	Nonveiller (1963)
◆ 2	f sand	14	11	2	20	260	"
● 1	sand	28	23	0	7	180	Farrent (1963)
● 2	m sand	13	24	0	10	182	"
● 3	m sand	13	24	0	10	158	"
▼ 1	f & m sand	11	43	0	25	157	Davisson et al (1972)
▼ 2	f & m sand	11	34	0	25	148	"
▼ 3	f & m sand	11	38	0	25	228	"
▼ 4	f & m sand	11	27	0	25	146	"

Fig 13 Variation of E'/N_{60} with degree of loading for normally consolidated sands

Tonouchi (1982) based on measurement of the velocity of shear waves through sands and gravels in the field. A straight line relationship was proposed by the authors as best fit to the data, as indicated. However, the line representing G_o = 7N MN/m² gives arguably almost as good a fit and is more useful for present purposes.

Now $G_o = \frac{E'_o}{2(1+\nu)}$ and for Japanese SPT procedures it is reasonable to assume N_{60} = 1.1 N (Skempton 1986). Thus assuming ν = 0.25, the relationship G_o/N = 7 MN/m² becomes E'_o/N_{60} = 16MN/m². Such a value is consistent with the trend of data in Figure 13 for overconsolidated sands and gravels at very low values of q_{net}/q_{ult}.

The decrease of shear modulus with shear strain for sands has been studied by a number of authors. Curves from Seed and Idriss (1970) and Uchida et al (1980) are shown in Figure 15, based on dynamic and cyclic loading tests on a variety of sands. It is to be expected that the stiffnesses so measured will be roughly equivalent to the stiffness of overconsolidated sands since in both cases the loading takes place essentially below the yield locus. In order to relate these curves to the E'/N_{60} v. q_{net}/q_{ult} plot for overconsolidated materials, however, it is necessary to establish a relationship between shear strain γ and q_{net}/q_{ult}. This can be done in a approximate way as follows:

Eggestad (1963) measured the distribution of strain beneath a model footing on normally consolidated sand and showed a relationship of increasing strain with q/q_{ult} (Figure 16).

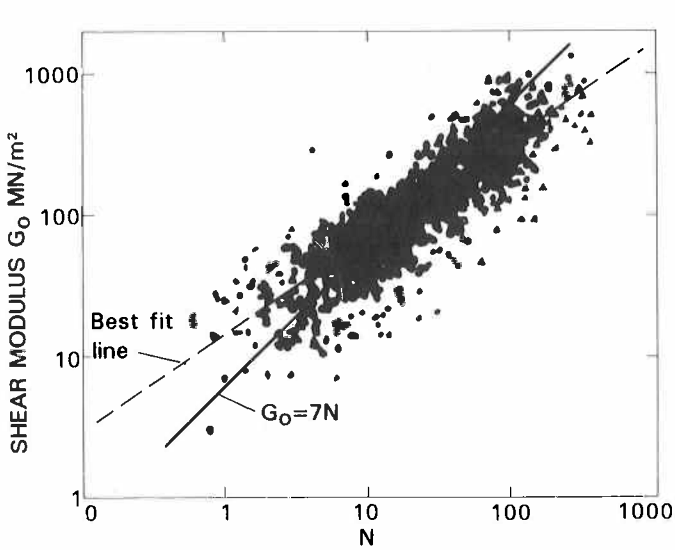


Fig 14 Relationship between small strain shear modulus G_o and N values (after Imai and Tonouchi, 1982)

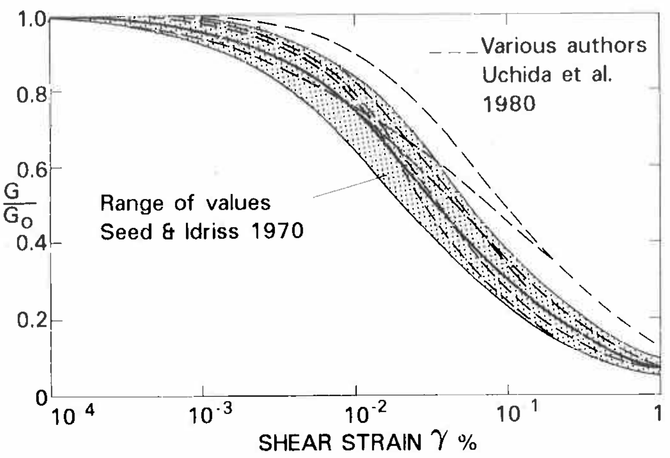


Fig 15 Variation of secant shear modulus G with shear strain

By averaging the strains to a depth of twice the footing width a relationship can be obtained between average shear strain γ and q/q_{ult}, as shown in Figure 17. This relationship is evidently broadly the same over a wide-range of relative densities. Now from the field data given in Figure 13 it is possible to estimate the ratio of stiffness between overconsolidated and normally consolidated materials at a given q_{net}/q_{ult}.

Thus the curve from Eggestad's data in Figure 17 for normally consolidated sands can be proportioned to give a corresponding curve for overconsolidated materials, as shown.

Using this curve and the value of E'_o/N_{60} = 16 MN/m² established earlier, and knowing that $G/G_o = E'/E'_o$, it is now possible

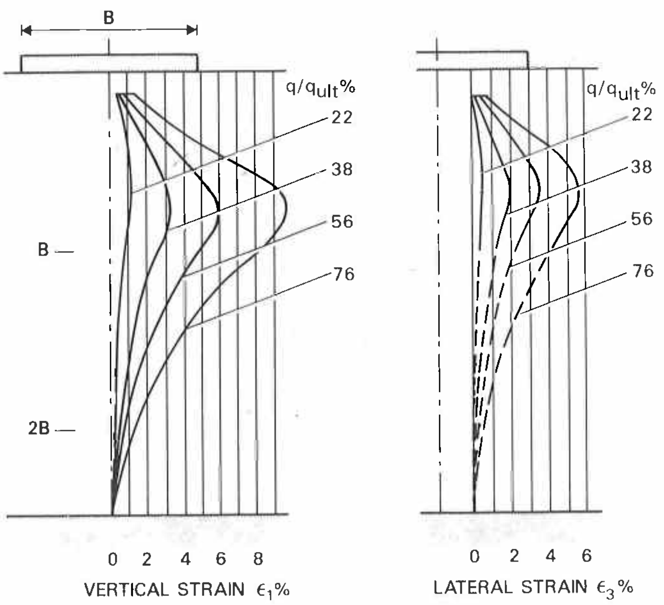


Fig 16 Measured strains beneath a footing on normally consolidated sands (after Eggestad, 1963. B = 200mm, D_r = 44%)

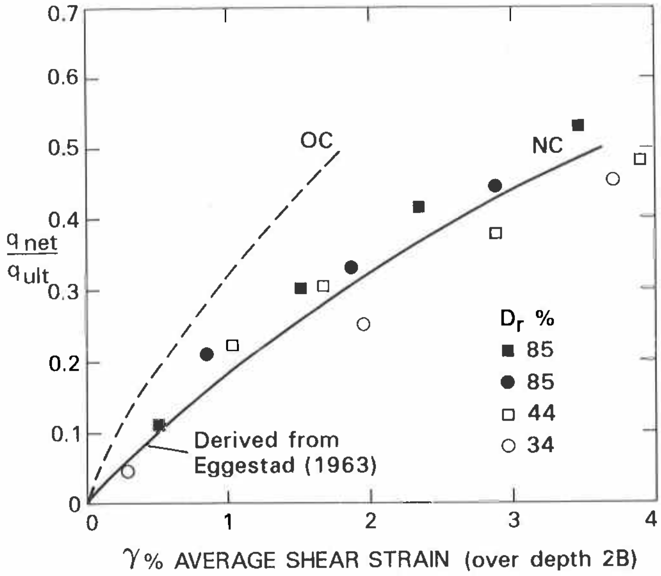


Fig 17 Relationship between degree of loading and average shear strain beneath a footing on sand

to transpose the envelope of curves from Figure 15 onto a plot of E'/N_{60} against q_{net}/q_{ult} as shown in Figure 18.

While it is recognised that this transposition has involved a number of rather sweeping generalisations, the trends are evidently similar and the broad agreement of the laboratory data with the back analysed case histories is encouraging.

CLAYS

Strength

Undrained shear strength of overconsolidated clays in the mass can be related to N values in the manner proposed by Stroud (1974), using the simple relationship $c_u = f_1 N$ where f_1 is a constant for a given material. The SPT data upon which the correlations were based were derived from modern UK practice and so the parameter f_1 is more properly defined by

$$c_u = f_1 N_{60}$$

The variation of f_1 with plasticity index is shown in Figure 19.

Stiffness

The drained stiffness of overconsolidated clays in vertical loading, E', can be back-figured from case histories as for sands. Figure 20 shows data from a number of raft and spread foundations for large structures on overconsolidated clays plotted against q_{net}/q_{ult} as before.

In each case q_{ult} has been estimated using drained strength parameters. In a number of

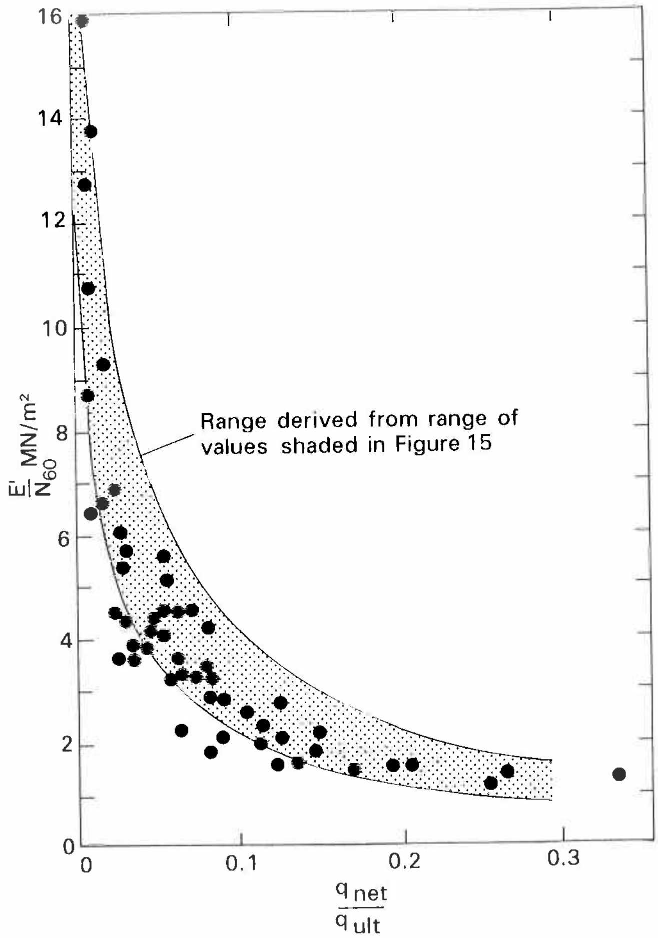


Fig 18 E'/N_{60} v. q_{net}/q_{ult} for overconsolidated sands based on dynamic and cyclic laboratory tests

cases, as indicated, it has been necessary to assume values of N_{60} based on strength measurements and appropriate values of f_1.

The trend of decreasing stiffness with increased loading is again evident. For the rafts on London Clay at a degree of loading of about 0.1 a value of E'/N_{60} = 900kN/m² appears to be representative. This is equivalent to E'/c_u = 200, since from Figure 19, f_1 = 4.5kN/m². On the other hand for the piled raft (No 5) q_{ult} is very much higher, the degree of loading is consequently less and the stiffness is correspondingly greater. Data for the materials of low plasticity appear to lie above those for materials of high plasticity.

A similar variation of stiffness with strain has been identified in the undrained loading of overconsolidated clays.

Simpson et al (1979) proposed a model for London Clay in which the ratio of undrained Young's Modulus to undrained shear strength, E_u/c_u, decreased with shear strain, see Figure 21. The model accounted for the range of stiffnesses typically measured around deep excavations, in plate bearing tests and on laboratory samples at the limit of accuracy of laboratory equipment at that time.

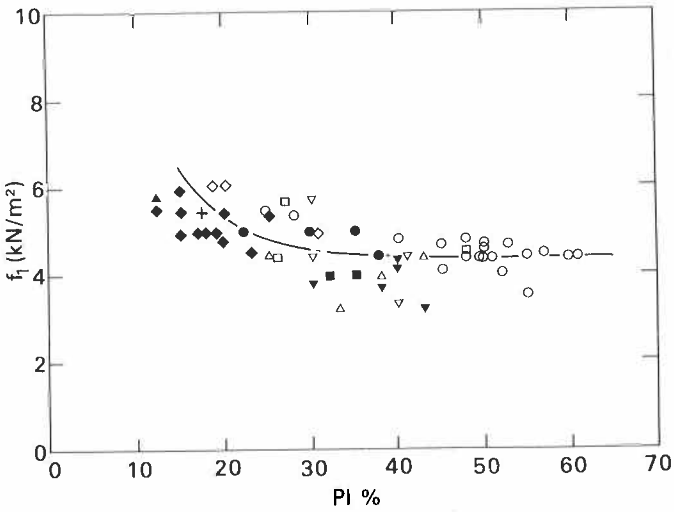


Fig 19 Variation of $f_1 = c_u/N_{60}$ with plasticity index for overconsolidated clays (after Stroud and Butler, 1975)

In recent years significant advances have been made in the accurate measurement of small strains in the laboratory (Burland and Symes, 1982). Figure 22 shows triaxial test data obtained by Jardine et al (1984) for undrained tests on London Clay and on an overconsolidated clay of low plasticity. The range of stiffness measured matches well that postulated in the Simpson model.

The general trend of results is also similar to that for the case histories in drained loading given in Figure 20.

In particular at very small strains the laboratory test data of Figure 22 gives values of E_u/c_u for London Clay and the clay of low plasticity in the region of 1400 and 1900 respectively.

Now, assuming that the shear modulus, G, is the same in drained and undrained loading,

$$E' = E_u \frac{(1+\nu)}{(1+\nu_u)}$$

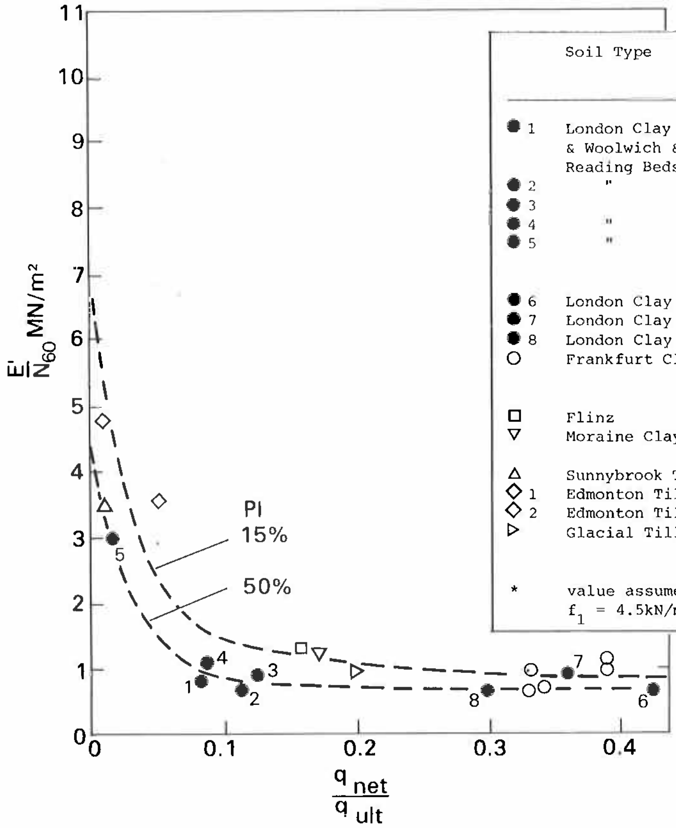
and $c_u = f_1 N_{60}$

$$\text{Thus } \frac{E'}{N_{60}} = \frac{E_u}{c_u} \frac{(1+\nu)}{(1+\nu_u)} . f_1 = \frac{E_u}{c_u} A.$$

Taking ν = 0.1 and ν_u = 0.5, and f_1 values from Figure 19 we arrive at:

	London Clay	Low plasticity clay
f_1 (kN/m²)	4.5	5.5
A (kN/m²)	3.3	4.0

Thus from the laboratory data we would expect at very small strains a value of E'/N_{60} = 3.3 x 1400kN/m² = 4.6MN/m² for London Clay and 4.0 x 1900kN/m² = 7.6MN/m² for low plasticity clays. These values are in keeping with the broad trend of case history data in Figure 20 at very low values of q_{net}/q_{ult}.



	Soil Type	PI (%)	B (m)	D (m)	t (m)	q_{net} (kN/m²)	Ø'	c' (kN/m²)	Reference
● 1	London Clay & Woolwich & Reading Beds	40-60	24	7	40	109	26°	20	Morton et al (1974)
● 2	"	40-60	18	6	34	120	"	"	"
● 3		40-60	23	5	19	165	"	"	"
● 4	"	40-60	30	11	7	226	"	"	"
● 5	"	40-60	27	9	75	124	"	"	Hooper (1973) Knightsbridge Barracks (piled raft)
● 6	London Clay	40-60	8.3	7	24	268	20°	15	MacDonald et al (1955)
● 7	London Clay	40-60	8.5	9	33	182	"	"	"
● 8	London Clay	40-60	1.5	2	>10	86	"	"	"
○	Frankfurt Clay	28	14-43	6-13	>85	90-260	20°	20	Breth et al (1974a) 5 large structures $*f_1 = 5kN/m^2$
□	Flinz	17	31	8	>73	265	30°	35	Breth et al (1974b)
▽	Moraine Clay	*16	20	4	20	250	*30°	30	Vefling (1974) $*f_1 = 5kN/m^2$
△	Sunnybrook Till	12	17	5	12	70	*33°		Crawford et al (1962)
◇ 1	Edmonton Till	14	34	7.5	21	203	40°		DeJong et al (1971)
◇ 2	Edmonton Till	14	4.5	5	23	805	40°		"
▷	Glacial Till	15	27	3	10	210	*30°	30	Holden (1988)

* value assumed
$f_1 = 4.5kN/m^2$ assumed in London Clay.

Fig 20 Variation of E'/N_{60} with degree of loading for overconsolidated clays

The laboratory data in Figure 22, also indicates that for a given strain, low plasticity clays exhibit greater stiffnesses than high plasticity clays.

Similar conclusions have been arrived at by others e.g. Koutsoftas et al (1980), Sun et al (1988) in resonant column and cyclic loading tests on clays in the triaxial apparatus. There is some confirmation for this trend in the back-analysed case history data in Figure 20. The trend lines for PI = 15 and 50% from Figure 20 are replotted in Figure 23 together with the trend lines obtained for granular materials. It is evident that the curve for overconsolidated sands and gravels, lies at higher stiffnesses still, which is consistent with this trend of increasing stiffness with decreasing plasticity.

The overall picture in Figure 23 suggests that the behaviour of overconsolidated silts will lie somewhere between that of low plasticity clays and overconsolidated sands, but data are needed to confirm this.

INSENSITIVE WEAK ROCKS

The SPT can be a very useful tool in weak rocks to obtain an approximate quantitative measure of rock properties. However, in rocks the influence of fissuring and jointing on the properties of the mass are even more important than for clays. Meigh and Wolski (1980) were right in emphasising that in these materials particularly, it is important to use our eyes to understand the structure.

Compressive strength

An attempt to correlate N_{60} values with the compressive strength of the mass of rock is presented in Figure 24. In these cases strength has been deduced from the back analysis of pile tests and pressuremeter tests,

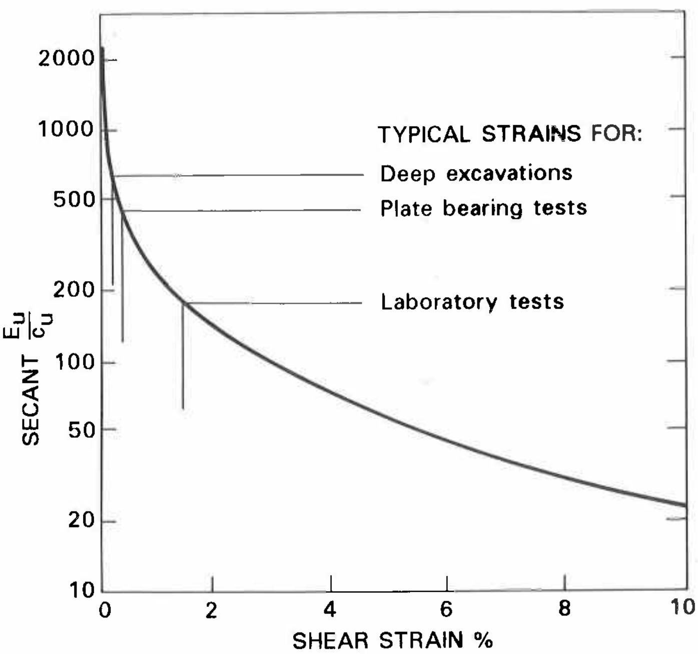


Fig 21 Variation of undrained Young's Modulus E_u with shear strain, derived from a mathematical model for London Clay (after Simpson and Sommer, 1980)

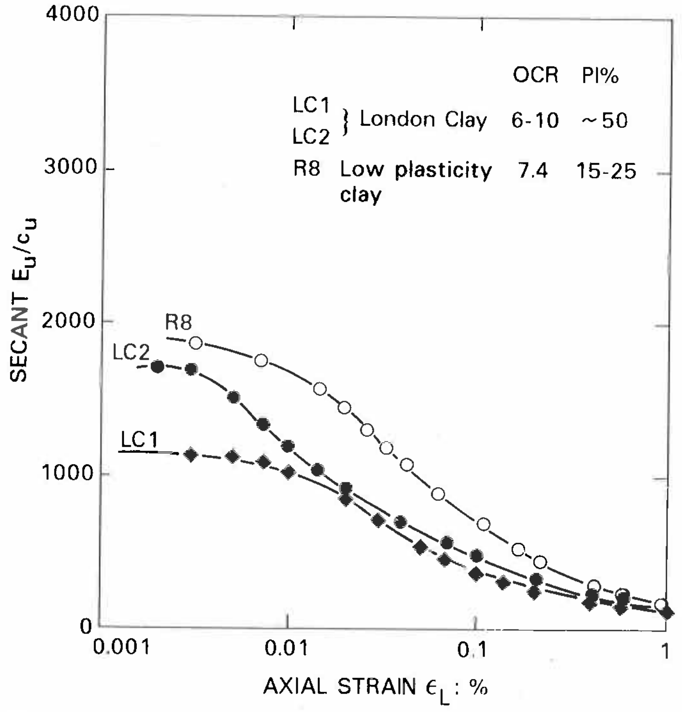


Fig 22 Stiffness of overconsolidated clay at small strains in the triaxial test (after Jardine et al, 1984)

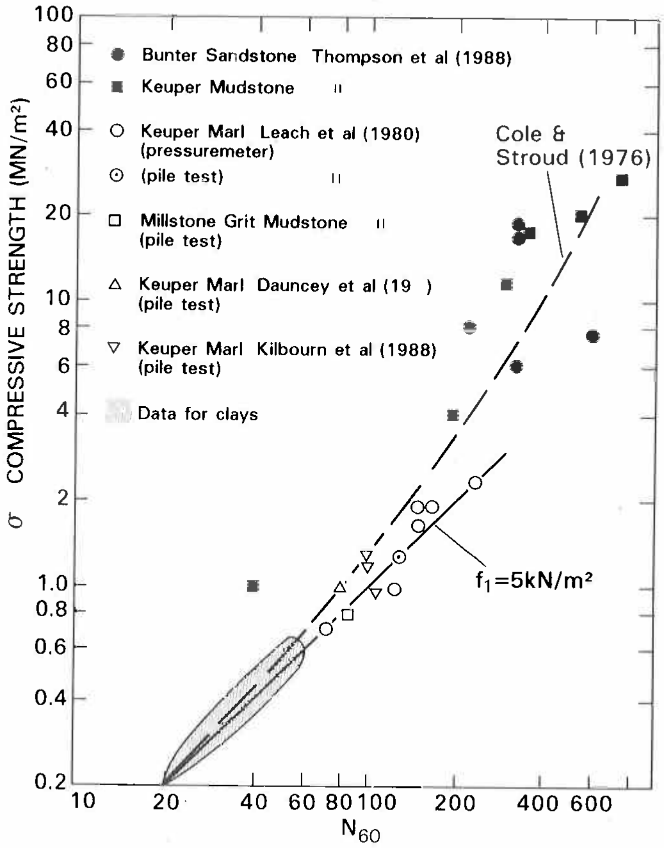


Fig 24 Variation of mass compressive strength with N_{60} for insensitive weak rocks

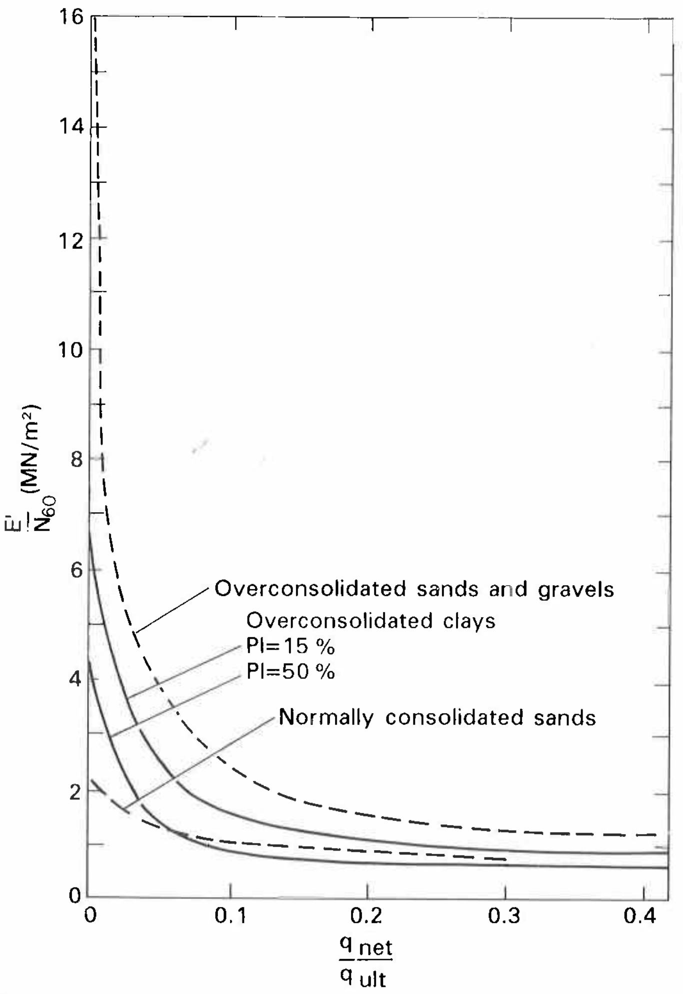


Fig 23 Variation of E'/N_{60} with degree of loading: summary plots

which, because of the volume of rock involved, go some way to measuring the strength of the mass rather than the intact rock.

The data suggest that for N_{60} <200 an extrapolation of the relationship found for clays is appropriate for weak rocks, using an average value of

$$f_1 = 5kN/m^2,$$

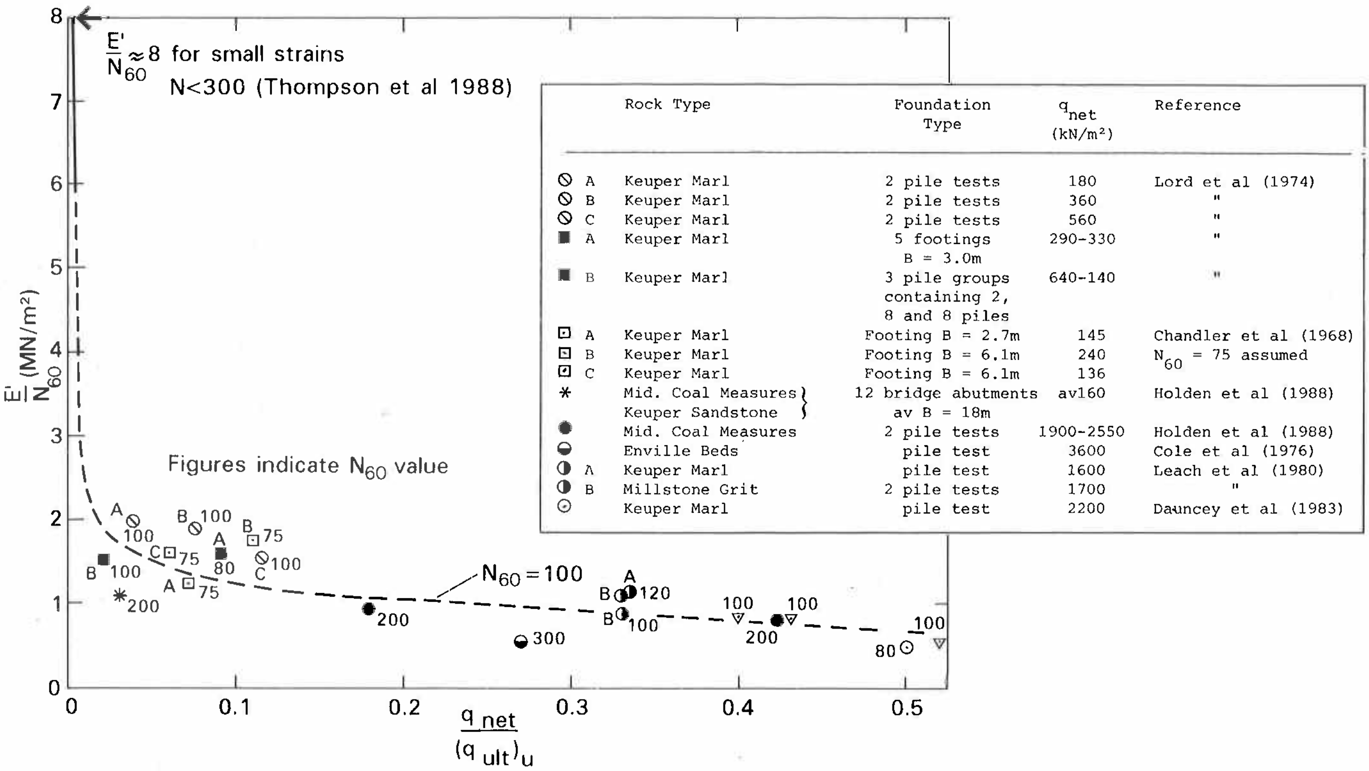
relating N_{60} to undrained shear strength, as for clays, and taking compressive strength $\sigma = 2c_u$.

Also shown is the relationship suggested by Cole and Stroud (1976) which forms an upper bound to the data for N_{60} <200.

Pile test data for the Keuper Marl from the paper to this conference by Kilbourn et al (1988) is shown, together with data by Thompson and Leach (1988) for Keuper Mudstone and Bunter Sandstone. It is understood that the strengths of the Mudstone were obtained from pressuremeter tests but the strengths of the Sandstones were based on point loading tests down graded by a factor of about 0.5 to take account of mass effects. The relationship represented by f_1 = 5kN/m² forms a lower bound to this data.

Stiffness

Figure 25 shows the results of back-analysis of a number of case histories of spread and piled foundations bearing on weak rocks. For these



Symbol	Rock Type	Foundation Type	q_{net} (kN/m²)	Reference
⊘ A	Keuper Marl	2 pile tests	180	Lord et al (1974)
⊘ B	Keuper Marl	2 pile tests	360	"
⊘ C	Keuper Marl	2 pile tests	560	"
■ A	Keuper Marl	5 footings B = 3.0m	290-330	"
■ B	Keuper Marl	3 pile groups containing 2, 8 and 8 piles	640-140	"
⊡ A	Keuper Marl	Footing B = 2.7m	145	Chandler et al (1968)
⊡ B	Keuper Marl	Footing B = 6.1m	240	N_{60} = 75 assumed
⊡ C	Keuper Marl	Footing B = 6.1m	136	
*	Mid. Coal Measures / Keuper Sandstone	12 bridge abutments av B = 18m	av160	Holden et al (1988)
●	Mid. Coal Measures	2 pile tests	1900-2550	Holden et al (1988)
⊖	Enville Beds	pile test	3600	Cole et al (1976)
◑ A	Keuper Marl	pile test	1600	Leach et al (1980)
◑ B	Millstone Grit	2 pile tests	1700	"
⊙	Keuper Marl	pile test	2200	Dauncey et al (1983)

Fig 25 Variation of E'/N_{60} with degree of loading for insensitive weak rocks

materials it has not been possible to calculate q_{ult} in effective stress terms because of the difficulties of estimating c' and Ø' and so ultimate bearing capacity in undrained loading $(q_{ult})_u$ has been estimated based on undrained shear strength. Where measurements of shear strength are not available, estimates have been made using N_{60} values and f_1 = 5kN/m².

While the scatter of points in Figure 25 is greater than for sands and clays the trend of decreasing stiffness with degree of loading is again evident. Thompson and Leach (1988) provide some data of shear wave measurements in mudstone and sandstones which indicate that, at very small strains the ratio of E'/N_{60} is in the region of 8MN/m².

The trend line for N_{60} = 100 is shown. There appears to be a tendency for points with higher values of N_{60} to fall below this line, see for example the data by Holden (1988) presented to this conference, where N_{60} values average 200 and that by Cole et al (1976) where N_{60} = 300.

CHALK

Strength

Chalk is a particular example of a weak rock with a sensitive structure that breaks down at failure to produce a material with a very much reduced shear strength. By its nature the SPT is likely to be influenced by this lower remoulded shear strength. Thus, for a given mass strength the N_{60} value will be lower for Chalk than for insensitive rocks, i.e. the f_1 value will be higher.

Nevertheless, it is still useful to correlate N_{60} values with the strength of chalk as determined in the mass.

Figure 26 summarises data from Hobbs and Healy (1979) for N_{60} against strength of Chalk determined by loading plates or piles to failure. The correlation gives a factor:

$$f_1 = 25\text{kN/m}^2,$$

relating N_{60} to undrained shear strength, as before.

The data presented to this conference by Woodland et al (1988) for very strong Upper Chalk from Hull, where N_{60} values are in the region of 200, are consistent with this correlation although in this case strength was determined from unconfined compressive tests in the laboratory.

Finally it is worth emphasising that SPT's in Chalk are likely to be influenced most by strength. There has been much confusion over relating N values to Chalk grades. It is true that N in a particular Chalk will vary with the degree of weathering, as the strength of the Chalk mass varies. However, N is also likely to vary from one Chalk to another even if the degree of weathering is the same (as measured by fracture spacing etc.) if the strength of the intact Chalk itself varies. Mortimore and Jones (1988) rightly conclude that there is a need to identify and record strength, fracture spacing and fracture tightness as three separate parameters in Chalk, as would be the case in any other rock.

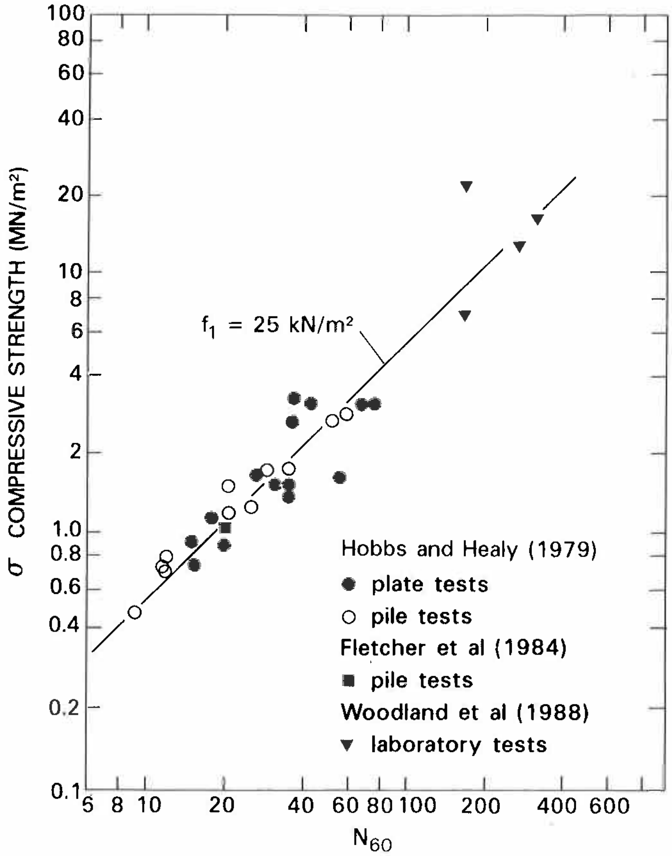


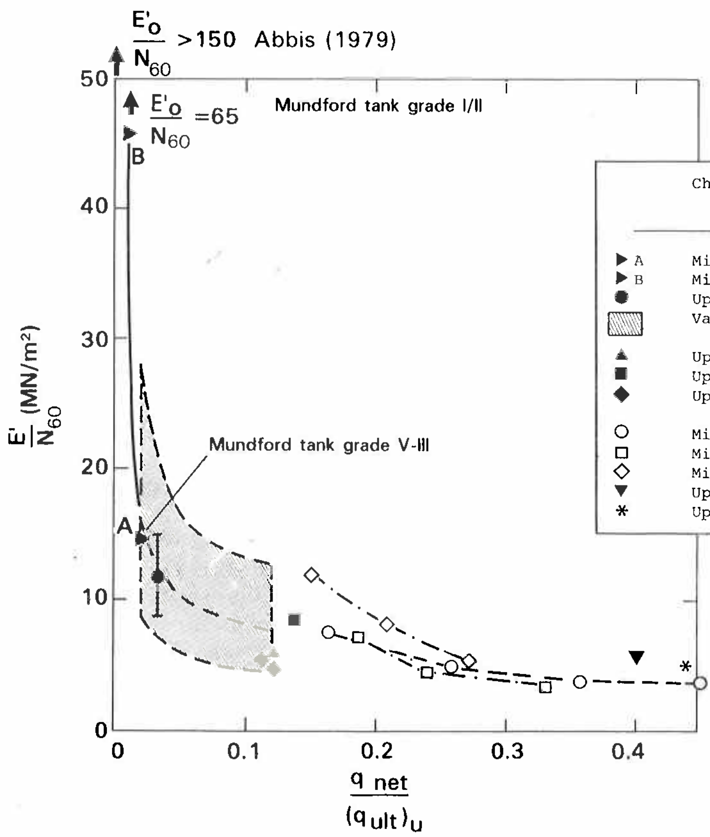
Fig 26 Variation of mass compressive strength with N_{60} for Chalk

Stiffness

Wakeling (1969) considered the relationship between E' and SPT values in Chalk and concluded that the correlation was strongly strain dependent.

The variation of E'/N_{60} with $q_{net}/q_{(ult)u}$ for Chalk is summarised in Figure 27 based on available data from in situ loading tests of shallow foundations, piles and large plates. Because of the difficulty of measuring accurately small deflections in this stiff material, data have not been included from small diameter plate tests or from plate tests where there is no evidence that the plates were bedded in mortar. Values of $q_{(ult)u}$ have been chosen in the same way as for weak rocks but using $f_1 = 25kN/m^2$.

Geophysical testing by Abbis (1979) shows that at very small strains E'/N_{60} can be in excess of 150 MN/m². Jardine et al (1984) made measurements of stiffness of chalk samples at small strains in the triaxial apparatus which leads to E'/N_{60} values in the region of 100MN/m², assuming f_1=25 kN/m². High values are also obtained from the Grade I/II Chalk at Mundford where at depth beneath the tank the degree of loading was also very low. In the upper levels where the strains are higher E'/N_{60} values of about 15MN/m² are obtained. This value is broadly consistent with data presented by Hobbs and Healy (1979) for the base performance of 30 pile loading tests and a number of large scale footings with degrees of loading up to about 0.15. The trend of decreasing E'/N_{60} with degree of loading is evident. An indication of the trend beyond



	Chalk Type	N_{60}	B (m)	q_{net} (kN/m²)	Reference
▶ A	Middle	18	18	149	Ward et al (1968)
▶ B	Middle	40	18	85	"
●	Upper	12	16	57	Burland et al (1974)
▨	Various	10-45	piles	150-500	Hobbs et al (1979) 30 pile tests
▲	Upper	24	2.6	620	Wakeling (1966)
■	Upper	24	9.4	500	"
◆	Upper	20	2-3.4	370	Lake et al (1974) 10 foundation bases
○	Middle	12	0.87	<1100	Burland et al (1969)
□	Middle	16	0.87	1050	plate tests (allowance
◇	Middle	19	0.87	1020	for creep to 4 months)
▼	Upper	230	-	8000	Woodland et al (1988)
*	Upper	20	1.05	2000	Fletcher et al (1984)

Fig 27 Variation of E'/N_{60} with degree of loading for Chalk

a degree of loading of 0.15 is given by the 0.87m diameter plate loading tests on the weaker Chalk at Mundford described by Burland and Lord (1969).

Finally, stiffness values for the high strength Chalk at Hull in dilatometer tests are reported by Woodland et al (1988) and give a value of E'/N_{60} of about 5.5MN/m² at a degree of loading of about 0.4. It should be pointed out however, that the dilatometer enables only a small area of chalk to be loaded, so this result may not be as representative of the mass properties as the other data.

The evidence suggests that a conservative stiffness for Chalk under a moderate degree of loading is given by E'/N_{60} = 5MN/m². This value should be compared with a value for insensitive weak rocks of about E'/N_{60} = 1MN/m² (Figure 25), i.e. a factor of 5 different. A similar factor is reflected in the values of f_1 with regard to strength, i.e. f_1 = 25 and 5KN/m² for Chalk and insensitive weak rocks respectively, which is perhaps not surprising.

CONCLUDING REMARKS

The application and interpretation of the SPT has been reviewed in four types of materials most commonly encountered in the UK. Two types of parameters have been considered: strength and stiffness.

Strength

In sands and gravels the value of the angle of shearing resistance $\emptyset'$ can be estimated from the SPT value corrected for overburden, $(N_1)_{60}$, and a knowledge of $\emptyset'_{cv}$. It is evident that at low relative densities there is no unique relationship between $(N_1)_{60}$ and $\emptyset'$. Here strength is closely influenced by $\emptyset'_{cv}$ which can lie within a wide range of possible values. The data indicates that $\emptyset'_{cv}$ is dependent principally on particle shape and grading. The interpretation of the SPT and for that matter of any other type of penetrometer is therefore greatly assisted by knowledge of $\emptyset'_{cv}$, particle shape and grading. These are easy parameters to measure in the laboratory on disturbed samples and should be part of basic site investigation practice.

The available data suggest that at higher values of relative density, $(N_1)_{60}$ is more directly related to $\emptyset'$, for a given degree of overconsolidation. The effect of overconsolidation is to increase the value of $(N_1)_{60}$ for a given value of $\emptyset'$.

The data indicate that the relationship between $(N_1)_{60}$ and relative density is not unique but depends on degree of overconsolidation and the value of $\emptyset'_{cv}$. More good quality data is needed to establish the relationships between $(N_1)_{60}$, $\emptyset'$, OCR and $\emptyset'_{cv}$.

The differences in behaviour between laboratory tests and field tests noted in Figure 3 may be, at least in part, due to differences in particle shape of the sands tested, as much as ageing as suggested by Skempton (1986). The shape of the laboratory tested sands was on average sub-rounded to rounded. Their $\emptyset'_{cv}$ value, from Figure 7, would be in the region of about 31°. Figure 9a then shows that it is to be expected that the laboratory data would plot below Skempton's field data in almost exactly the position found in Figure 3.

In overconsolidated clays, weak rocks and Chalk the undrained shear strength of the deposit in the mass can be estimated from the expression $c_u = f_1 N_{60}$.

Values of f_1 calibrated against plate loading tests, pile loading tests and pressuremeter tests may be summarised as follows:

Table 1	f_1 (kN/m²)
Overconsolidated clays	
PI = 50%	4.5
PI = 15%	5.5
Insensitive weak rocks	
N_{60} <200	5.0
Chalk	25.0

Stiffness

In all materials the relationship between drained Young's Modulus, E', and the SPT value uncorrected for overburden, N_{60}, has been shown to be strongly strain dependent. The value of E'/N_{60} decreases with increasing degree of loading q_{net}/q_{ult}. The effect is most striking in overconsolidated sands and gravels and least dramatic for normally consolidated sands.

The majority of full scale structures founded on overconsolidated sands and gravels, when back-analysed, showed stiffnesses greater than would be given by E'/N_{60} = 3MN/m². For structures where the factor of safety against local shear failure is about 3 (i.e., q_{net}/q_{ult} = 0.33), the stiffness drops to half this value.

In normally consolidated sands at a low degree of loading, E'/N_{60} = 2MN/m², but rapidly drops to E'/N_{60} = 1MN/m² as the level of loading increases.

Table 2 summarises typical values of E'/N_{60} for each of the materials considered, at q_{net}/q_{ult} of about 0.1.

General

Most of the considerations discussed in this report have relevance to other forms of penetration testing as well as to the SPT.

Many of the apparent inconsistencies of interpretation of the SPT in granular materials in the past are resolved by consideration of:

a) the influence of strain level on the relationship between N_{60} and E',

	E'/N_{60} (MN/m²)	E'/c_u*
Overconsolidated sands and gravels	2.5	-
Normally consolidated sands	1.0	-
Overconsolidated clays		
PI = 15%	1.4	250
PI = 50%	0.9	200
Insensitive weak rocks		
N_{60} <200	1.0	200
Chalk	5.0	200

* based on f_1 values from Table 1

Table 2: Stiffness at $q_{net}/q_{ult} = 0.1$

b) the importance of $\emptyset'_{cv}$ on the relationships between $(N_1)_{60}$, $\emptyset'$ and the relative density D_r.

In all materials the importance of using the eye to understand the fabric of material being tested cannot be over-emphasised. In granular materials it is important to observe carefully particle shape, grading and layering. In clays, weak rocks and Chalk the state of fissuring, jointing and bedding must be understood.

Neither the SPT nor any other type of penetrometer should be used as an alternative to visual inspection of the material, but as an adjunct to it. Boring holes to retrieve samples for inspection and testing will therefore always be a central part of site investigation. Carrying out SPT's is then a simple task which yields at little expense a great deal of additional information about the ground.

The versatility, simplicity and cheapness of the SPT in such a range of ground conditions will continue to make it a very attractive tool for site investigation in the UK and in many other countries in the world.

The limitations of the SPT of course should be recognised, but so too should the limitations of other in situ testing devices. All have a place in modern site investigation practice but none will free the geotechnical engineer from the need to use his judgement in the interpretation of the results. We have a friend in the SPT and we would be the worse off without it.

ACKNOWLEDGEMENTS

The writer is grateful to his colleagues in Arup Geotechnics for much helpful criticism during the preparation of this paper, in particular to Dr J A Lord and Dr J W Pappin; also to Mrs J Britton and Miss J Arnold whose sustained skill and patience turned words and diagrams into reality.

APPENDIX A

Note on the method of estimating $(N_1)_{60}$ values in overconsolidated materials

Skempton (1986) shows that N_{60} and relative density in a normally consolidated sand can be related by the expression

$$N_{60}/D_r^2 = a + b\,\sigma'_v$$

where a and b are constants, and σ'_v = vertical effective stress in tons/ft².

Assuming that for a given relative density penetration resistances is uniquely related to the mean effective stress $p' = \frac{1}{3}\sigma'_v(1 + 2K_o)$, this expression can be rewritten as

$$N_{60}/D_r^2 = a + C_{oc}\, b\,\sigma'_v$$

where $C_{oc} = (1 + 2K_o)/(1 + 2K_{oNC})$

and where K_o and K_{oNC} are the in situ stress ratios σ'_h/σ'_v in overconsolidated and normally consolidated sand respectively.

Now to a first approximation

K_{oNC} = 1-sin$\emptyset'$ and

$K_o = K_{oNC}$ (OCR) sin$\emptyset'$ (Mayne & Kulhawy, 1982).

Thus for given values of $\emptyset'$ and OCR, values of C_{oc} can be calculated.

Now $(N_1)_{60}/D_r^2 = a + C_{oc}b$.

Thus for a given D_r, the ratio of $(N_1)_{60}$ values in overconsolidated and normally consolidated sand is given by the expression

$$(N_1)_{60}/[(N_1)_{60}]_{NC} = (a + C_{oc}b)/(a + b).$$

This expression was used to derive the $(N_1)_{60}$ values for overconsolidated materials shown in the main text assuming representative field values of a = 36 and b = 27 for normally consolidated sands and C_{oc} values tabulated by Skempton.

APPENDIX B

Notes on the methods of analysis

Estimation of stiffness E'

The average apparent Young's Modulus was estimated for each of the foundations given in the case histories using linear elastic theory and the expression:

$$E = \frac{q\,B\,I}{\rho}\,\mu_1\,\mu_2$$

where ρ is the settlement at the centre of the foundation of width B loaded to pressure q. The influence coefficient I were estimated using Steinbrenner charts (Lambe & Whitman, 1969) which enabled account to be taken of the depth of compressible material beneath the foundation. Poisson's ratio for drained loading was taken as 0.25 in sands and 0.1 in clays.

A factor correcting for depth of embedment μ_1 based on Fox (1948) was applied where appropriate. A factor μ_2 = 0.8 was also applied where it was considered that the foundation was rigid.

In a limited number of cases it was appropriate to estimate the settlement of strata underlying the stratum in question and to deduct this from the total measured settlement before calculating E'.

Where possible values of q and ρ were chosen corresponding to loading net of previous overburden. In some cases for overconsolidated materials this was not possible and gross bearing pressure and the corresponding settlement have been used. For normally consolidated sands net pressures and corresponding settlements were used in every case.

Choice of N value

Burland and Burbidge (1985) defined the depth of influence below a foundation as being that depth at which the settlement is 25% of the surface settlement. They recognised that the depth of influence depends on the variation of stiffness with depth. Nevertheless the data which they were able to gather together, where the variation of settlement beneath foundation has been measured, indicates that depth of influence, lies at a depth breadth-ratio (Z_1/B) of between 0.75 and 1.25 for foundations of width 1m to about 30m.

In the analysis of the data in this paper it has been assumed that depth of influence extends to a depth of about B below a foundation and the N_{60} values within this zone have been averaged to provide characteristic N_{60} values used in evaluating E'/N_{60}.

In a number of cases the general ground level and/or the groundwater level was reduced to a lower level prior to the construction of the foundations. For granular materials where the SPT's were carried out from the original level it was necessary to correct the N_{60} values to give new N_{60} values appropriate to the new stress regime. This was done by using the correction curve given in Figure 2.

Otherwise no corrections to N_{60} values have been made for such factors as the presence of gravel or very fine sands and silty sands below the water table. There appears to be little evidence to support the validity of such corrections.

As mentioned in the main text N values obtained in early American and UK practise were adjusted to modern SPT rod energy levels by reducing N by a factor of 0.8 to give N_{60} values (see Skempton, 1986).

In order to estimate ϕ' and q_{ult} the characteristic N_{60} values obtained as described above were corrected to give $(N_1)_{60}$ values using Figure 2.

Estimation of q_{net}

The average net effective bearing pressure has been taken as the gross effective bearing pressure less the previously existing effective overburden pressure at foundation level.

Estimation of q_{ult} for granular materials

The ultimate bearing capacity in drained loading q_{ult} has been obtained as follows:

Firstly the $(N_1)_{60}$ values characteristic of the material in question to a depth of about B below foundation level were used to obtain values of ϕ' from Figure 8. Unless it was otherwise made clear from the case history data, it was assumed that ϕ'_{cv} = 32° applied for uniformly graded materials and ϕ'_{cv} = 36° applied to well graded materials. For normally consolidated deposits Figure 8a was used and for overconsolidated deposits an OCR of 3 was assumed and Figure 8b was used.

Having obtained appropriate values of ϕ' in this way, the ultimate bearing capacity with allowances for local shear was estimated in the manner described by Lambe and Whitman (1969) using bearing capacity factors shown in Figure B1(b).

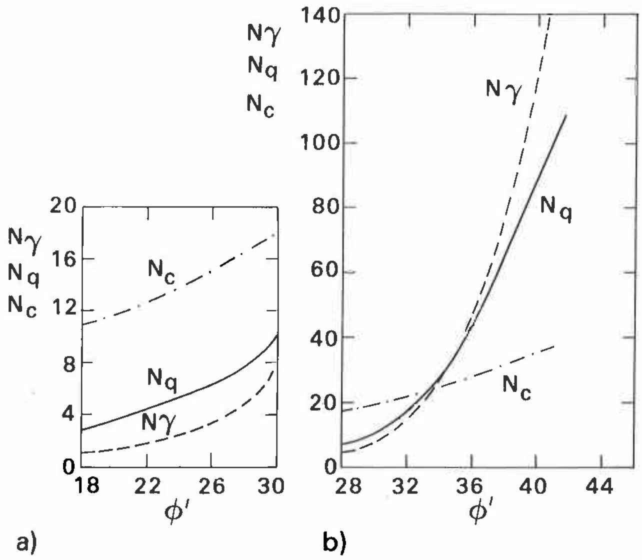


Fig B1 Bearing capacity factors for local shear failure based on Lambe and Whitman (1969) and Terzaghi (1943)

Estimation of q_{ult} for clays

The ultimate bearing capacity in drained loading for clays was based on measured effective strength parameters where possible or on estimated values as indicated in the table in Figure 20. Bearing capacity factors have again been estimated making an allowance for local shear in the manner suggested by Terzaghi (1943) taking the mobilised angle of friction at the point of local shear failure as $\phi'_m = \tan^{-1} ({}^2/_3 \tan \phi')$. Bearing capacity factors for local shear failure in a material of strength ϕ' are then taken as the same as those for a material of strength $\phi'm$ without allowance for local shear failure. Values so obtained are shown in Figure B1(a).

While the choice of factors for local shear in clays may be somewhat arbitrary it does provide continuity with the assumptions made for loose sands (see Lambe and Whitman, 1969 p209). Consolidation in clays is arguably a rather drawn out process of punching shear.

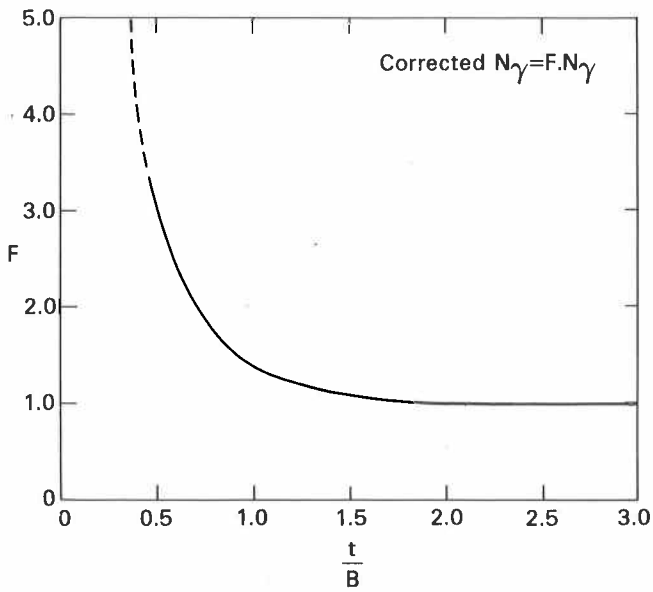


Fig B2 Effect of reduced thickness of bearing stratum, t, on bearing capacity factor N_γ (after Tournier and Milovic, 1977)

Estimation of $(q_{ult})_u$ for rocks

The ultimate bearing capacity in undrained loading for weak rocks and Chalk has been calculated using undrained shear strength and conventional bearing capacity factors. Shear strength has been calculated from measurements of mass compressive strength taking $\sigma = 2c_u$. Where mass compressive strength data is not available c_u has been estimated from N_{60} using the appropriate value of f_1.

Correction for limited depth

Tournier and Milovic (1977) show that for a foundation of breadth B underlain by a soil of thickness t the bearing capacity factor N_γ increases significantly where t reduces to values less than B. They arrived at the correction factors for N_γ shown in Figure B2. These correction factors have been used in analysing the case histories in those few cases where a stratum of significantly greater strength underlies the foundation at shallow depth.

APPENDIX C

TABLE C1 Values of ϕ'_{cv} for granular materials

	ϕ'_{cv}	Particle size	Mineralogy	Particle shape	D_{50} (mm)	D_{10} (mm)	Unif coeff.	e_{max}	e_{min}	Reference
UNIFORMLY GRADED										
Well rounded and rounded sand										
Ottawa	29.5	c	q	well rnd	0.75	0.65	1.2	0.08	0.49	Lee et al (1967)
Ottawa	30.0	m	q	rnd	0.53	0.35	1.7	0.79	0.49	Been et al (1987)
Sub rounded to sub angular sand										
Chattahoochee River	32.5	m	q	s ang	0.37	0.17	2.5	1.10	0.61	Vesic et al (1968)
Mol	32.5	f-m	q	s rnd	0.19	0.14	1.5	0.89	0.56	Ladanyi (1960)
Monterey No 0	32.0	m	q+sf	s rnd	0.37	0.25	1.6	0.82	0.54	Been et al (1987)
Ticino	31.0	c	q	s rnd	0.53	0.36	1.6	0.89	0.60	" " "
Sacramento River	33.3	f-m	q+f	s ang/s rnd	0.22	0.15	1.5	1.03	0.61	Lee et al (1967)
Reid Bedford	32.0	f-m	q+sf	s ang	0.24	0.16	1.6	0.87	0.55	Been et al (1987)
Hokksund	32.0	c	q+f	s ang	0.39	0.21	2.0	0.91	0.55	" " "
Welland River	35.0*	f	q	s rnd	0.14	0.10	1.4	0.94	0.62	Barden et al (1969)
Leighton Buzzard	35.0*	c	q	rnd-s rnd	0.82	0.65	1.3	0.74	0.49	Stroud (1971)
Toyoura	32.0	f	q	s ang	0.16	0.11	1.5	0.98	0.61	Tatsuoka (1987)
Toyoura	34.0*	f	q	s ang	0.16	0.11	1.5	0.98	0.61	" "
Toyoura	31.0	f	q	s ang	0.21	0.16	1.4	0.87	0.66	Been et al (1987)
Angular sand										
Milton Mines	35.0	f-m	q+f	ang	0.20	0.11	2.0	1.05	0.62	Been et al (1987)
Southport	35.0	f-m		ang	0.20	0.12	1.8	0.88	0.53	Holubec et al (1972)
Olivine	34.0	f-m		ang	0.26	0.20	1.5	1.07	0.63	" " "
Angular gravel										
Furnas gravel	34.0	m-c	q	ang						Casagrande (1965)
Very angular sand										
Crushed quartz	36.4	f	q	v ang	0.12	0.07	2.0	1.15	0.55	Koerner (1970)
Crushed feldspar	38.7	f	f	v ang	0.12	0.07	2.0	1.21	0.49	" "
Crushed feldspar	42.0*	f-m	f	v ang	0.21	0.11	2.0	0.91	0.56	Barden et al (1969)

	ϕ'_{cv}	Particle size	Mineralogy	Particle shape	D_{50} (mm)	D_{10} (mm)	Unif coeff.	e_{max}	e_{min}	Reference
WELL GRADED										
River sand & gravel	35.0	37mm-f sand	f+q	s rnd/s ang	4.8	0.6	8			Holtz et al (1956)
Glacial outwash sand	37.0	f-c		s ang	0.75	0.15	6	0.84	0.41	Hirschfield et al (1964)
Sandy gravel	37.0	76mm-fines		rnd						Casagrande (1965)
San Francisco	38.0	50mm-fines	basalt	ang						Marachi et al (1969)
Furnas Dam	39.0	10mm-fines	quartzite							Casagrande (1965)
San Fransisco	38.0	76mm-6mm		ang						Marsal (1967)
Dredger tailings	40.0	50mm-fines		rnd						Marachi et al (1969)
Granite gneiss	40.8	37mm-4mm		ang						Marsal (1967)
Conglomerate	40.8	37mm-4mm		ang						Marsal (1967)

Key:

f = fine
m = medium
c = coarse
* = plane strain

q = quartz
f = feldspar
s = some

rnd = rounded
s ang = sub angular
s rnd = sub rounded

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NOTATION

a,b	constants relating N_{60}/D_r^2 to σ_v' for normally consolidated granular materials. viz. $N_{60}/D_r^2 = a + b\,\sigma_v'$
A	constant relating ν, ν_u, and f_1, viz. $A = \frac{(1+\nu)}{(1+\nu_u)}.f_1$
B	foundation breadth
c_u	undrained shear strength of clay or rock in the mass
c'	effective cohesion intercept
C_{oc}	factor modifying the constant b for overconsolidated granular materials
C_N	overburden correction factor, converting N to N_1, viz. $N_1 = C_N N$
D	depth of foundation embedment below ground surface
D_r	relative density
D_{10}	particle size at which 10% by weight of a sample is of smaller particles
D_{50}	particle size at which 50% by weight of a sample is of smaller particles
D_{60}	particle size at which 60% by weight of a sample is of smaller particles
e_{max}	maximum voids ratio
e_{min}	minimum voids ratio
E'	drained Young's Modulus of elasticity
E_o'	drained Young's Modulus at very small strains
E_u	undrained Young's Modulus of elasticity
f_1	factor relating c_u and N_{60}, viz. $c_u = f_1 N_{60}$
G	shear modulus of elasticity
G_o	shear modulus at very small strains
I	settlement influence coefficient
K_o	in situ stress ratio σ_h'/σ_v'
K_{oNC}	in situ stress ratio σ_h'/σ_v' for normally consolidated granular materials
N	standard Penetration Test blow count for 300mm penetration
N_1	N value corrected to that appropriate to a vertical effective stress of 100kN/m²
N_{60}	N value resulting from using rod energy equal to 60% of the free-fall energy of the standard hammer weight and drop
$(N_1)_{60}$	N_{60} value normalised to vertical effective stress of 100kN/m²
NC	normally consolidated
N_q, N_γ, N_c	bearing capacity factors
OCR	overconsolidation ratio
p'	mean effective stress
PI	plasticity index
q	average bearing pressure
q_{net}	average net effective bearing pressure
q_{ult}	ultimate bearing capacity in drained loading, at the point of local failure
$(q_{ult})_u$	ultimate bearing capacity in undrained loading
R	roundness, defined as the ratio of the average of the radii of the corners of a sand grain image to the radius of the maximum circle that can be inscribed within the grain image
t	thickness of stratum beneath foundation
Unif.coeff.	uniformity coefficient = D_{60}/D_{10}
z_1	depth of influence, at which the settlement is 25% of the surface settlement
γ	shear strain
μ_1	depth of embedment factor in elastic settlement calculation
μ_2	rigidity factor in elastic settlement calculation
ν	effective Poisson's Ratio
ν_u	undrained Poisson's Ratio
ρ	settlement of foundation
σ	rock compressive strength in the mass. $\sigma = 2c_u$
σ_h'	horizontal effective stress
σ_v'	vertical effective stress
ϕ'	angle of internal friction, measured as a secant value
ϕ'_{cv}	angle of internal friction at the critical state, where shearing takes place with zero volume change
ϕ'_m	mobilised angle of friction