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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., $I_c = \frac{\Delta \rho}{\Delta q} \times \frac{1}{B^{0.7}}$.

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.

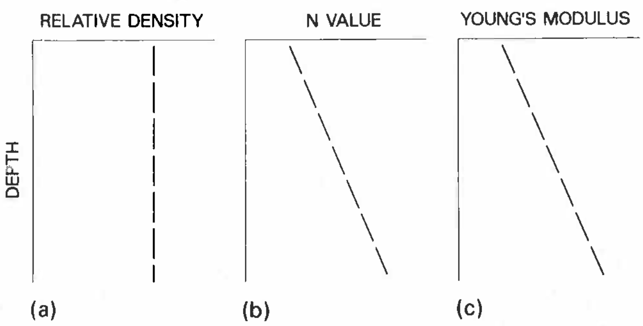

Fig 1 Sands and gravels: basic relationships

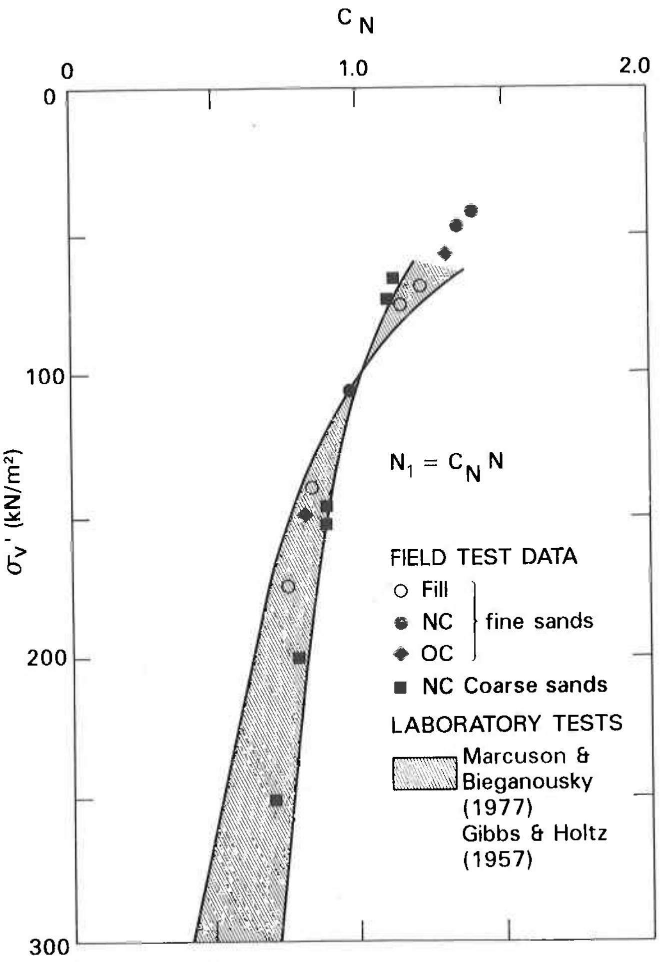

Fig 2 Correction for overburden pressure (after Skempton, 1986)

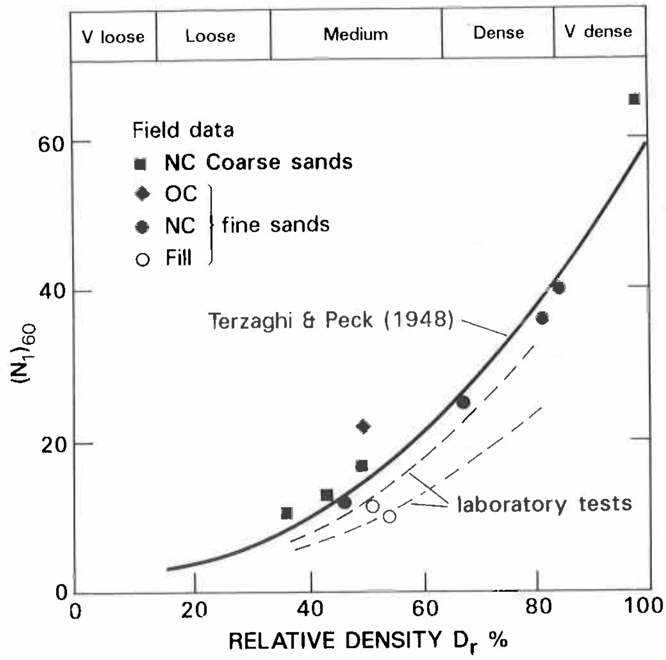

Fig 3 Effect of relative density, based on field data (after Skempton, 1986)

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.

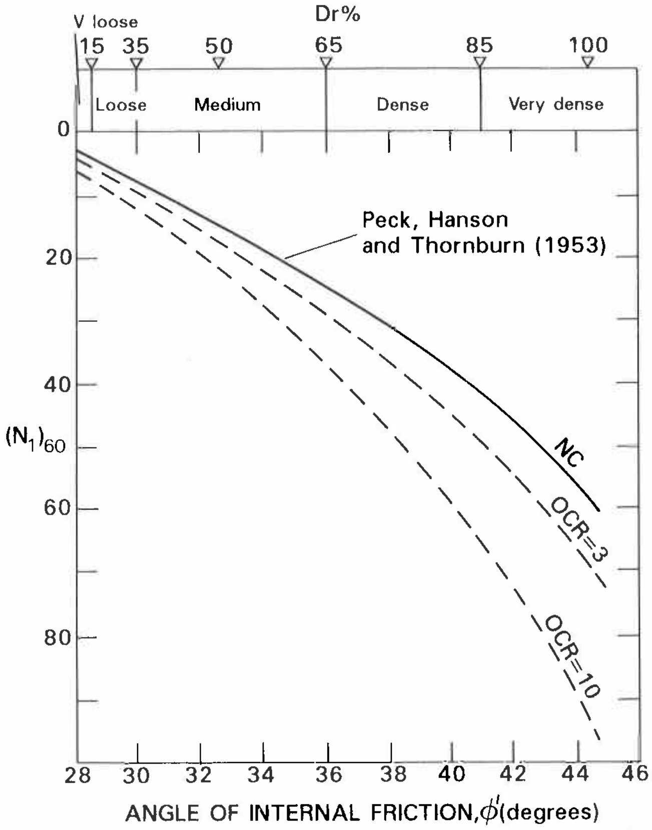

Fig 4 Effect of overconsolidation ratio on the relationship between $(N_1)_{60}$ and angle of friction ϕ'.

Bolton (1986) collected together data for 17 different sands. Figure 5 shows the variation of $\phi'-\phi'_{cv}$ 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 ϕ'_{cv} varied from about 33° for the quartz sands to 37° for sands containing a significant proportion of feldspar.

Taking a value of ϕ'_{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 ϕ'_{cv} = 33°, then a relationship between $(N_1)_{60}$ and ϕ' 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 ϕ'_{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 ϕ' from Peck Hanson and Thornburn, replotted from Figure 4.

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

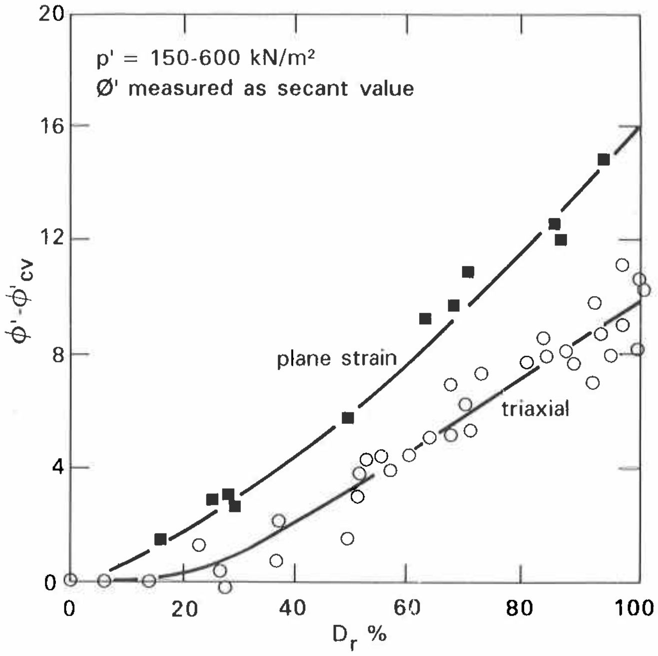


Fig 5 Variation of $\phi'-\phi'_{cv}$ with relative density (after Bolton 1986)

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 ϕ' 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 ϕ'.

The correlations are clearly very sensitive to the value of ϕ'_{cv} appropriate to the material in question. Values of ϕ'_{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 ϕ'_{cv} with particle shape is plotted in Figure 7 from this data for triaxial testing. Typical ϕ'_{cv} values may be summarised as follows:

uniformly graded	quartz	feldspar
rounded	30°	
sub rounded sub angular	32°	
angular very angular	34° 36°	 39°
well graded		
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 ϕ'. 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

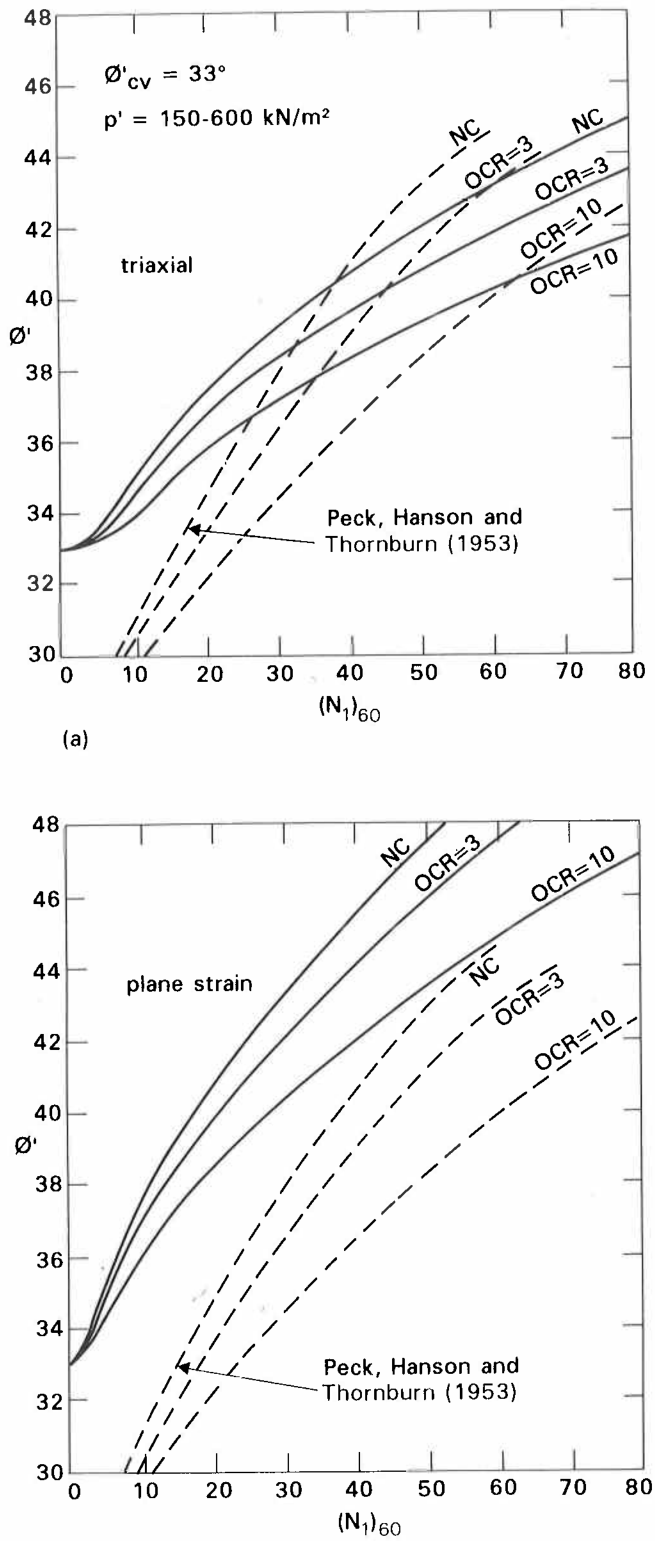


Fig 6 Relationship between $(N_1)_{60}$ and Ø' for materials with $Ø'_{cv}$ = 33°

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 $Ø'_{cv}$ 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

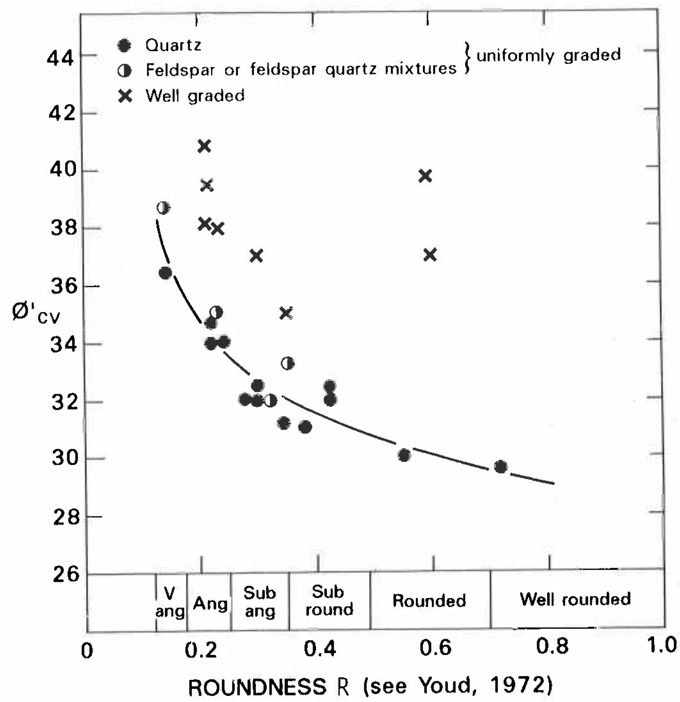


Fig 7 Relationship between particle shape and $Ø'_{cv}$ based on triaxial tests

proportional to the bearing capacity factor, N_q, and thus is uniquely related to Ø'.

This possibility is explored in Figure 8a where the $(N_1)_{60}$ v. Ø' curve for normally consolidated materials under triaxial loading and having $Ø'_{cv}$ = 33° has been replotted from Figure 6a. Also shown is a curve AA of N_q against Ø' 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 $Ø'_{cv}$ = 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 $Ø'-Ø'_{cv}$ 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 $Ø'_{cv}$.

Evidently the value of $Ø'_{cv}$ 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

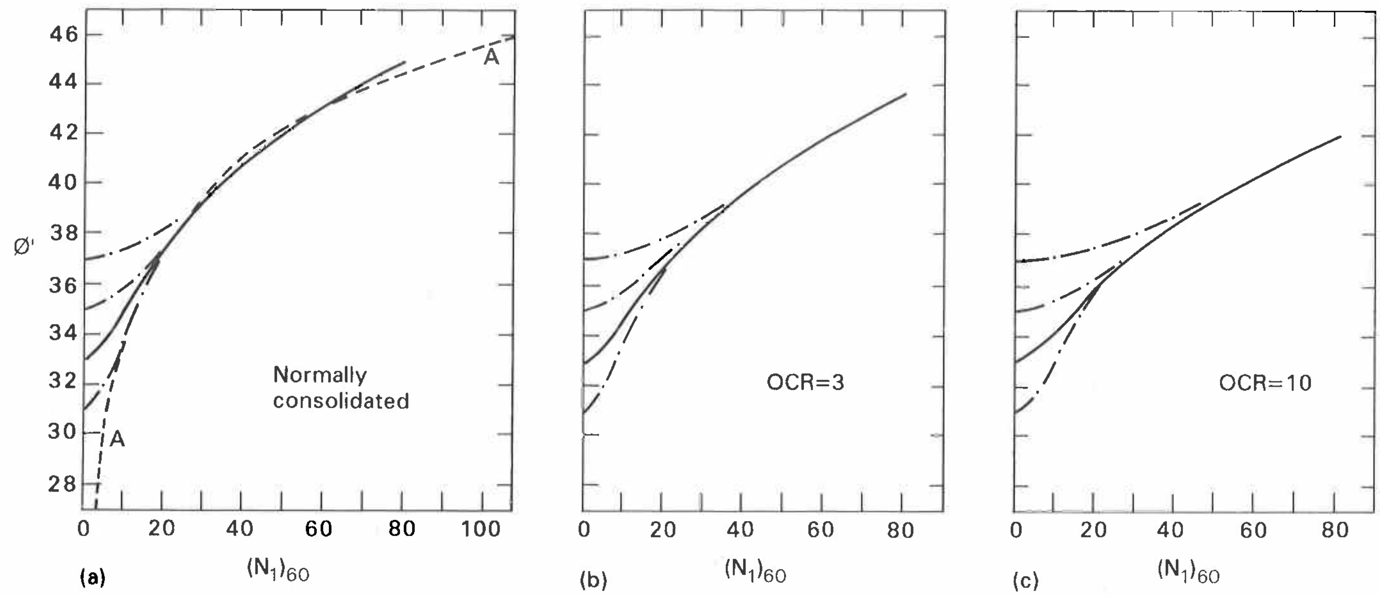


Fig 8 Variation of ϕ' and $(N_1)_{60}$ with ϕ'_{cv} and OCR

75%. No strength values were quoted, but a value of ϕ'_{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 ϕ'_{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 ϕ'_{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

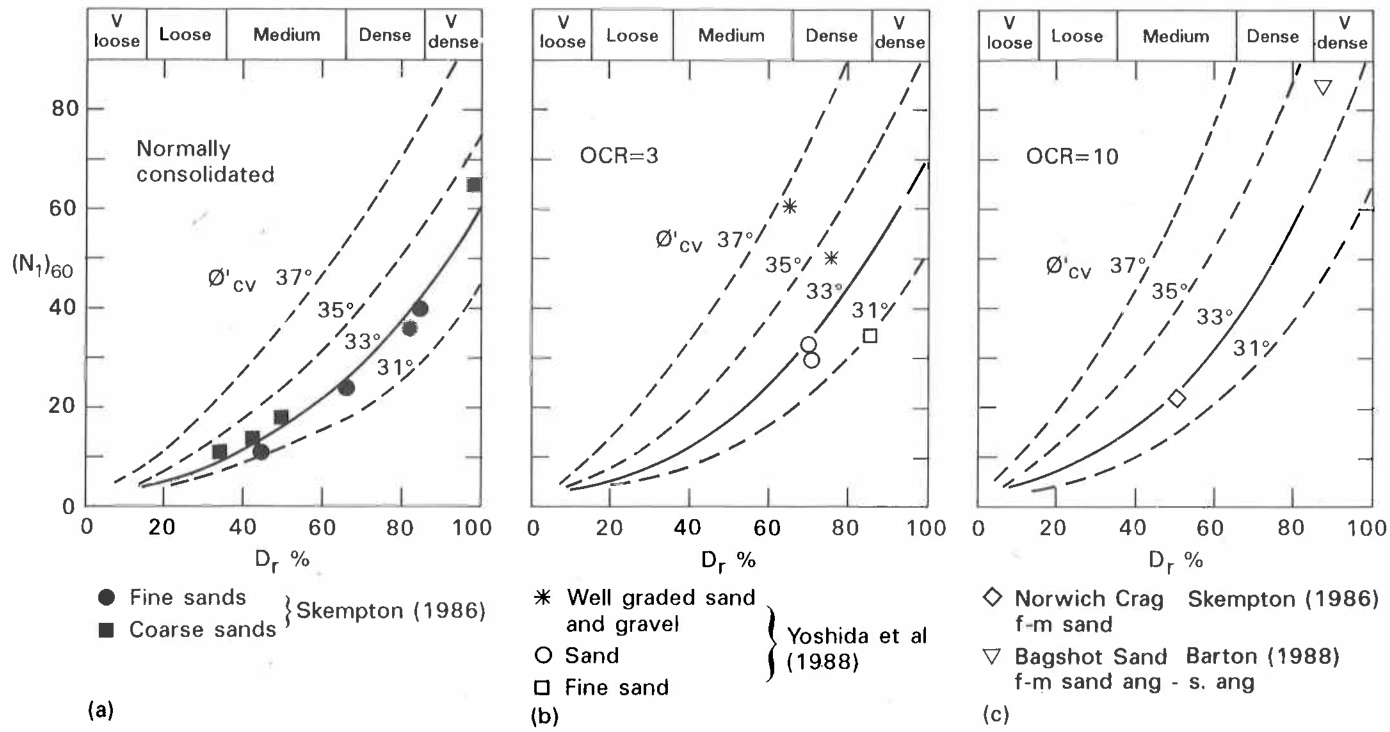


Fig 9 Variation of $(N_1)_{60}$ and relative density with ϕ'_{cv} and OCR

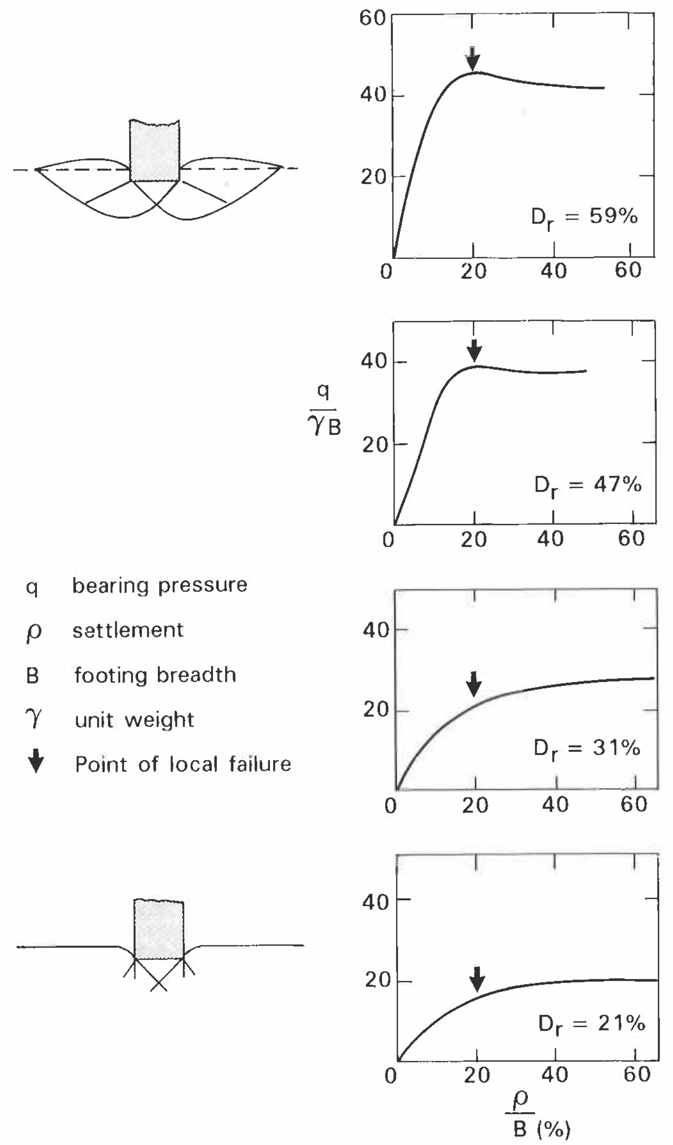


Fig 10 Behaviour of model footing on sand at various relative densities (after Vesic, 1973. B = 38mm)

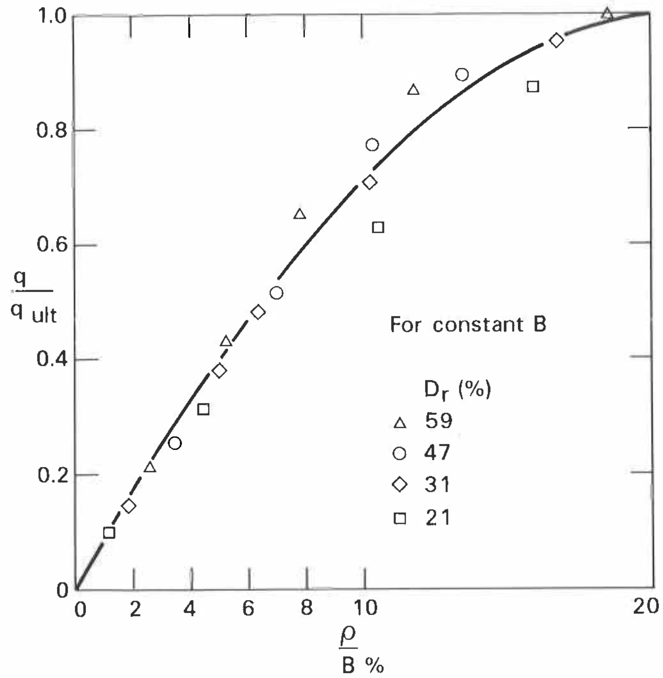


Fig 11 Normalised plot of settlement against bearing stress, for model footing tests by Vesic.

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.

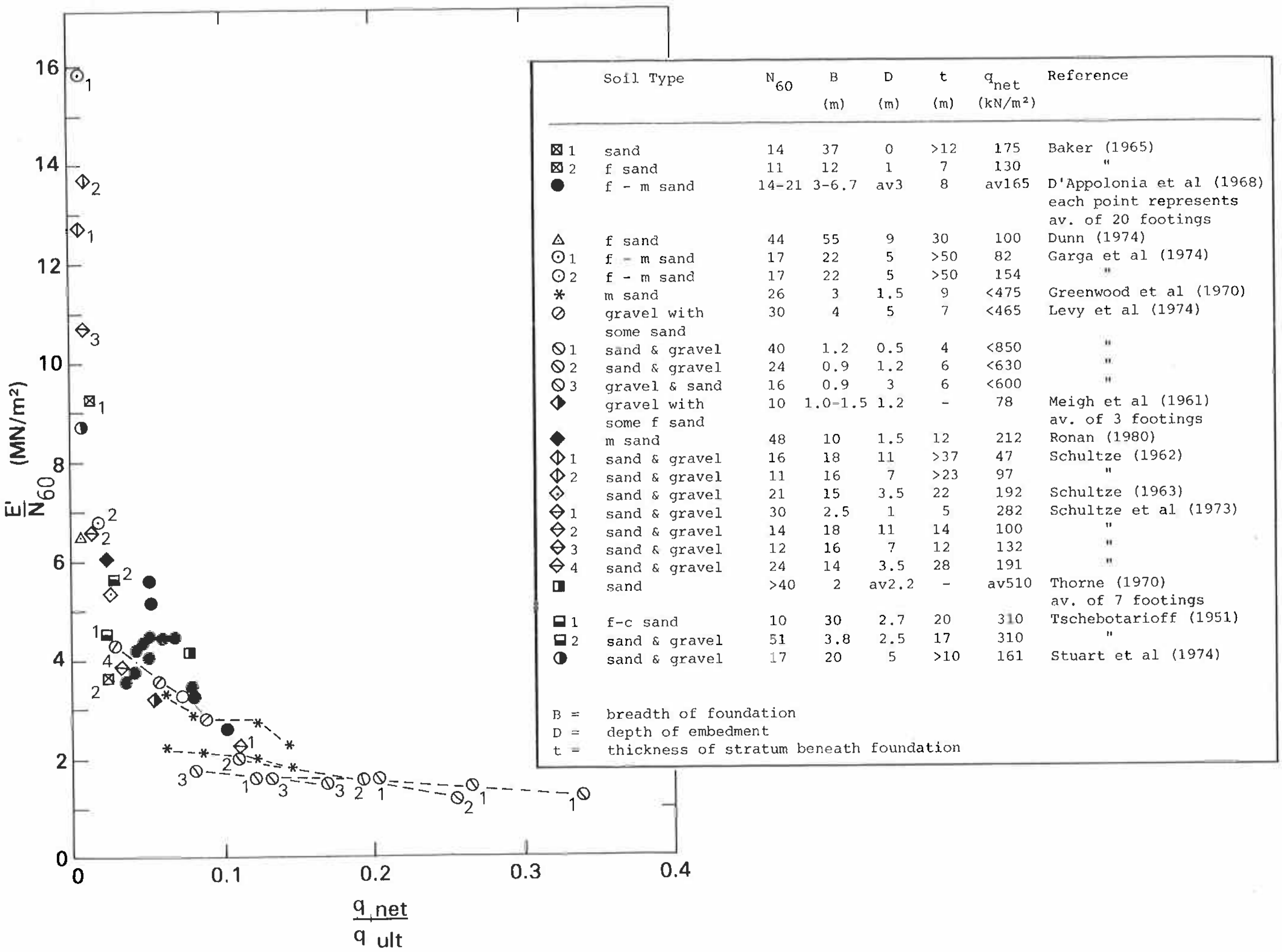
This suggests that q/q_{ult} is an indirect measure of shear strain.

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	"
◆ (half)	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 Ø' 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}.

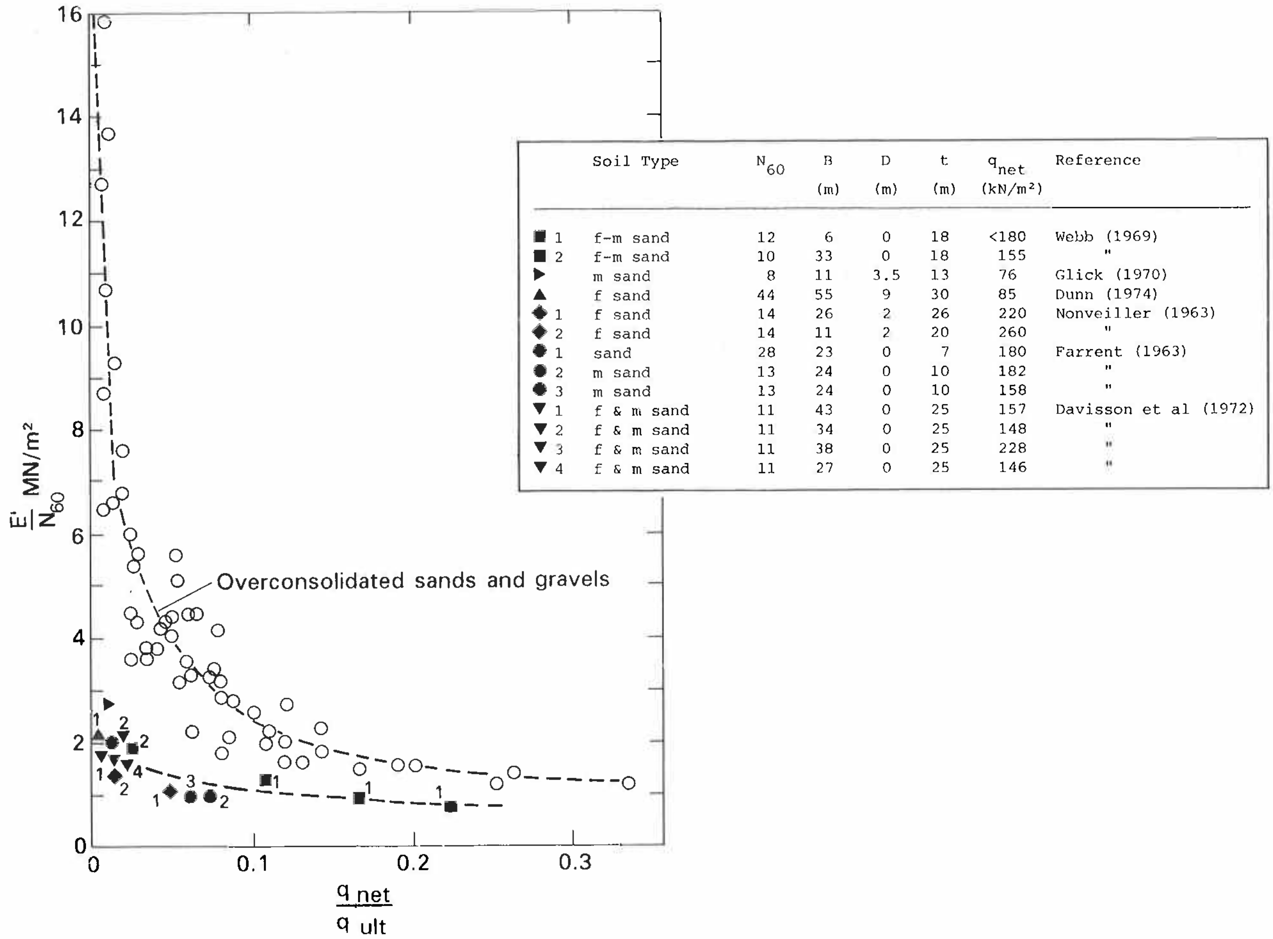
It is evident that the relationship between E' and N_{60} is strongly strain dependent. It is little wonder that the search for simple relationships between E' and N in the past has proved so frustrating in the absence of consideration of strain or degree of loading.

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 $G_o = \frac{E'_o}{2(1+\upsilon)}$ and for Japanese SPT procedures it is reasonable to assume $N_{60} = 1.1\ N$ (Skempton 1986). Thus assuming $\upsilon = 0.25$, the relationship $G_o/N = 7$ MN/m² becomes $E'_o/N_{60} = 16$MN/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).

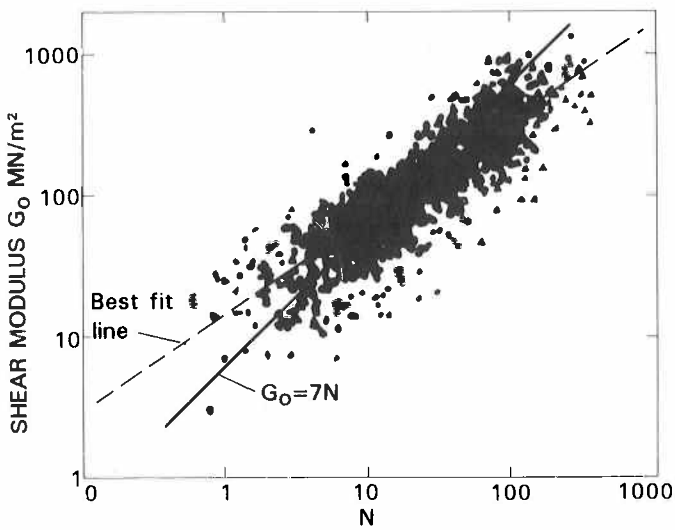


Fig 14 Relationship between small strain shear modulus G_o and N values (after Imai and Tonouchi, 1982)