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Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation

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Granted that good site control is exercised in carrying out the standard penetration test the energy delivered to the sampler, and therefore the blow count obtained in any given sand deposit at a particular effective overburden pressure, can still vary to a significant extent depending on the method of releasing the hammer, on the type of anvil and on the length of rods (if less than 10 m). For consistency it is essential to correct the observed blow count N to the value which would have been measured using a specified rod energy. A recommended value, which should be recognized internationally, is 60% of the free-fall energy of the standard hammer weight and drop. The corrected blow count is then designated as N_{60} and the normalized value $(N_1)_{60}$ at unit effective pressure (1 kg/cm^2 or 100 kPa) may be regarded as a basic characteristic of the sand. Factors controlling the rod energy ratio are examined in detail and methods of deriving N_{60} values are developed. An examination of selected field and laboratory data shows that the relation between blow count, effective overburden pressure σ_v' (kg/cm^2) and relative density D_r is given to a close approximation by an equation of the form proposed by Meyerhof: $N_{60} = (a + b\sigma_v')D_r^2$ or $(N_1)_{60} = (a + b)D_r^2$ where a and b are constants for a particular sand within the range $0{\cdot}35 < D_r < 0{\cdot}85$ and $0{\cdot}5$ kg/cm$^2 < \sigma_v' < 2{\cdot}5$ kg/cm^2. The parameters a and b, values for which are given for all the cases studied, tend to increase with increasing grain size, with increasing age of the deposit and with increasing overconsolidation ratio. The long-standing apparent discrepancy between field and laboratory tests is resolved when the effects of differing rod energy ratios and of 'ageing' are taken into account. Also, the Terzaghi–Peck limits of blow count for various grades of relative density, as enumerated by Gibbs and Holtz, are shown to be good average values for normally consolidated natural sand deposits, provided that the blow counts are corrected to $(N_1)_{60}$ values.

Pourvu que le chantier soit bien contrôlé lors de 'standard penetration test' l'énergie transmise à l'appareil de prise de chantillon et donc le nombre de coups donnés dans un dépôt de sable à une pression effective de terrain de couverture spécifique peuvent encore varier de façon importante selon la méthode de déclanchement du marteau, le type d'enclume et la longueur des tiges (si moins de 10 m). Pour être correct il est essentiel de corriger le nombre de coups observé N pour obtenir la valeur qui aurait été mesurée si une énergie de tige spécifique avait été employée. Une valeur recommandée, qui devrait être acceptée sur le plan international, est 60% de l'énergie de chute libre du poids normal du marteau. On appelle alors le nombre de coups corrigé N_{60} et on peut considérer la valeur normalisée $(N_1)_{60}$ à la pression effective spécifique (1 kg/cm^2 ou 100 kPa) comme une caractéristique fondamentale du sable. L'article examine de façon détaillée les facteurs qui contrôlent le rapport de l'énergie de la tige et présente des méthodes pour calculer des valeurs N_{60}. Des données sélectionnées obtenues in situ et en laboratoire indiquent que le rapport entre le nombre de coups, la pression effective du terrain de couverture σ_v' (kg/cm^2) et la densité relative D_r est donné à peu près par une équation de la forme proposée par Meyerhof: $N_{60} = (a + b\sigma_v')D_r^2$ ou bien $(N_1)_{60} = (a + b)D_r^2$ où a et b sont des constantes pour un sable spécifique dans l'intervalle $0{\cdot}35 < D_r < 0{\cdot}85$ et $0{\cdot}5$ kg/cm$^2 < \sigma_v' < 2{\cdot}5$ kg/cm^2. Les paramètres a et b, dont les valeurs sont données pour tous les cas étudiés, tendent à s'accroître au fur et à mesure que le diamètre des grains, l'âge du depot et le rapport de surconsolidation augmentent. Le désaccord apparent trouvé depuis longtemps entre les essais in situ et en laboratoire disparait lorsqu'on tient compte des effets des rapports différents de l'énergie de la tige et du 'vieillissement'. On démontre aussi que les limites du nombre de coups données par Terzaghi et Peck pour différentes fourchettes de densités relatives, comme mentionnées par Gibbs et Holtz, représentent de bonnes valeurs moyennes pour des dépôts naturels de sable normalement consolidés, à condition que les nombres de coups soient corrigés pour obtenir les valeurs $(N_1)_{60}$.

KEYWORDS: field tests; liquefaction; relative density; sands.

INTRODUCTION

In connection with investigations of the River Indus alluvium, for the Kalabagh Dam project in Pakistan, the Author has carried out an ancillary study of the standard penetration test (SPT) with regard to the influence of various test procedures and the relationship between blow count, relative density and overburden pressure. Much use is made of recent research involving measurements of rod energy ratios and field investigations in Japan.

Discussion on this Paper closes on 1 January 1987. For further details see inside back cover.

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The 'dynamic efficiency' of the hammer–anvil system is introduced as a new parameter. Some of the confusion surrounding comparisons between laboratory and field tests is dispelled, and the effects of particle size, 'ageing' and over-consolidation are quantified.

INFLUENCE OF TEST PROCEDURES

General considerations

To obtain reliable results the SPT should be carried out under controlled conditions. These include

(*a*) the use of the wash boring technique or rotary drilling with a tricone drill bit and mud flush, water or mud in the borehole to be maintained up to groundwater level
(*b*) tests to be made from the bottom of boreholes not less than 65 mm ($2\frac{1}{2}$ in) or greater than 150 mm (6 in) and preferably not more than 100 mm (4 in) diameter, with the casing (if used) not advanced below the bottom of the borehole
(*c*) the blow count (N value) to be determined between 6 in and 18 in penetration, the first 6 in being regarded as a possible zone of disturbance from drilling operations.

Even with good site control there are still two major variables, depending on the method of releasing the hammer and the type of anvil. These together control the energy E_r delivered into the rod stem which can be expressed as a ratio of the theoretical free-fall energy of the hammer, E^x. Thus

$$E_r = ER_r E^x \tag{1}$$

where ER_r is the 'rod energy ratio' and

$$E^x = \frac{1}{2}\frac{w}{g}v^2 \tag{2}$$

For free fall

$$v = (2gh)^{1/2} \tag{3}$$

so

$$E^x = wh = 140 \times \tfrac{30}{12} = 350 \text{ ft lb} \tag{4}$$

for the standard weight (140 lb) and height of drop (30 in) of the hammer.

However, owing to frictional losses, the hammer velocity at impact is less than the free-fall velocity. The actual hammer energy E_h can be expressed as

$$E_h = ER_v E^x \tag{5}$$

where ER_v is the 'velocity energy ratio'.

There is a further loss of energy on impact. This can be expressed by the dynamic efficiency η_d where

$$E_r = \eta_d E_h \tag{6}$$

Thus finally

$$ER_r = \eta_d ER_v \tag{7}$$

If the rod stem has a length of 10 m or more the sampler receives the full rod energy E_r. With shorter rods the sampler receives less than E_r and a correction has to be made for this effect: see later.

Velocity energy ratio

In general practice four methods of releasing the hammer are used

(*a*) a trigger mechanism, such as the Japanese 'Tombi'
(*b*) a trip hammer, such as the Pilcon or Dando hammers
(*c*) manual (lifting) and release of the rope passing over the crown sheave of the drilling rig
(*d*) the 'slip-rope' method of rapidly slackening the rope on the winch cathead: it is usual to have two turns of rope on the cathead for lifting the hammer, sometimes three turns and rarely one turn, and it is these turns of rope which have to be cast off to release the hammer.

Measurements of the impact velocity of a Borros trip hammer, similar in type to the Dando hammer, show (Kovacs, 1979) that $ER_v = 0{\cdot}99$. Trigger release mechanisms also impart very nearly a free fall.

In method (*c*) and still more in method (*d*) there is some retardation even though the rope may seem to have been completely freed. Frydman (1970) briefly reported alternate tests in boreholes, in Israel, performed either with trigger release or by the two-turn slip-rope method, using the same hammer. The results, plotted in Fig. 1, show an N ratio of about 1·4 or $ER_v \approx 0{\cdot}7$. Comparative tests by Douglas (1982) in a sand fill at San Diego are also plotted in Fig. 1, the points being site averages at various depths from tests with a trip release or with a two-turn slip-rope. They show on average a slightly lower ER_v value of about 0·66 or an N ratio of 1·5. Diameters of the cathead in both cases were probably about 8 in, a typical dimension in American and American-influenced practice.

A thorough investigation at Niigata (Yoshimi & Tokimatsu, 1983) compared a donut hammer released by the Tombi trigger and by a two-turn slip-rope. The results, together with similar tests

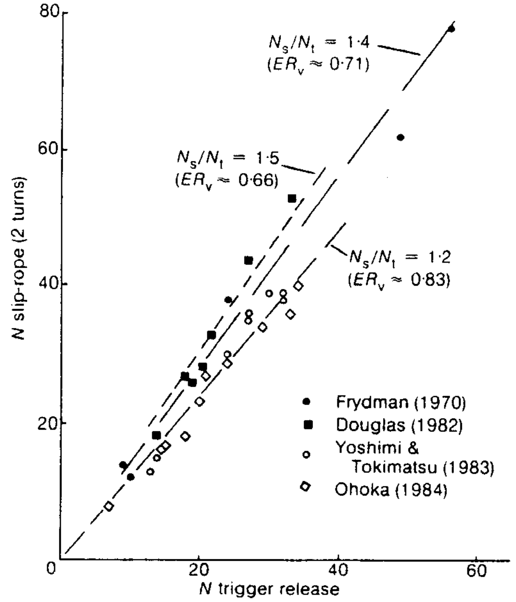

Fig. 1

by Ohoka (1984) are shown in Fig. 1, the average N ratio being 1·2 or $ER_v = 0{\cdot}83$.

The higher velocity ratio of the Japanese slip-rope method, compared with American practice, is explained by the smaller cathead diameter of 130 mm (5 in) used in the Niigata tests (100–130 mm is usual in Japan) and partly by the thinner manila rope; 12–17 mm diameter compared with 19–25 mm in America.

Shi-Ming (1982) compared the traditional Chinese manual operation of donut hammers with trip release hammers of the same form and found

$$N(\text{manual}) \approx 1{\cdot}15N(\text{trip})$$

or

$$ER_v \approx 0{\cdot}87$$

This is largely a measure of the retarding effect of the crown sheave. The value is in agreement with special tests (used sometimes in Japan) in which the rope is thrown sideways completely off the cathead, when on average $ER_v = 0{\cdot}88$ (tests summarized in Seed, Tokimatsu, Harder & Chung (1984)).

Direct measurements of hammer velocity at the instant of impact (Kovacs, Evans & Griffith, 1977; Kovacs & Salomone, 1982) confirm the N ratios of around 1·4 found by comparative tests with trigger release and the American two-turn slip-rope and also provide information on ER_v for one turn and three turns of rope; see Fig. 2 where the foregoing data are assembled, with manual release and 'special throw' velocity ratios being counted as zero turns.

Experience at Kalabagh has shown no practical difference in average N values obtained by manual release and one turn of rope on a small (80 mm) cathead. This fact is taken into account in drawing the uppermost curve in Fig. 2.

Rod energy ratio

By means of dynamic load cells inserted in the rod stem it is possible to determine the energy E_r in the rods (Schmertmann & Palacios, 1979) and therefore the rod energy ratio. These researchers showed that the blow count in a given sand is

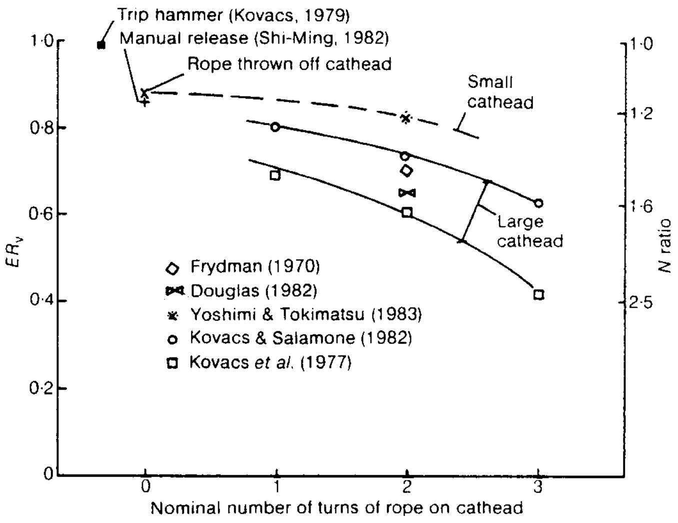

Fig. 2. Velocity–energy ratio

Table 1. Blow count N and rod energy ratio ER_r in two series of field tests

	S hammer, AW rods	F hammer, N rods
N	8·8	14·2
ER_r	0·52	0·31

inversely proportional to ER_r. For example, five or six tests in adjacent borings, at depths between 10 ft and 30 ft, gave the average results for two different hammers and drill rods given in Table 1 and to a close approximation

$$14{\cdot}2 \times 0{\cdot}31 = 8{\cdot}8 \times 0{\cdot}52$$

In another example (Robertson, Campanella & Wightman, 1983) alternate tests with two different hammers were carried out in the same borehole, using the same rig and a two-turn slip-rope release. The results, Fig. 3(a), show considerable differences both in N values and rod energy ratios, but if the N values are normalized to a single energy ratio (chosen here as $ER_r = 0{\cdot}55$) the results, Fig. 3(b), fall into a consistent pattern.

A collection of rod energy ratios for the two commonly used American hammers, donut and safety, is presented in Table 2, in all cases with the two-turn slip-rope release. The lower efficiency of the donut hammer, Fig. 4(b), is to be attributed to the heavy anvil associated with this type of hammer in America, compared with the small anvil of the safety hammer.

Similar data from Japan for donut hammers operated by the Tombi and slip-rope methods are given in Table 3. The higher rod energy ratios, compared with the relevant values in Table 2, are due partly to the more efficient Japanese slip-rope technique (i.e. a higher ER_v) and also to the lightweight anvils used in Japan.

Trip hammers, Fig. 4(c), despite having virtually a free fall, impart a rod energy ratio of only about 0·6 (Table 4) because of a low dynamic efficiency which is due to their heavy anvils (see next section). Comparative tests by Serota & Lowther (1973) show no significant difference between N values obtained in tests with a Pilcon hammer and the basically similar Dando trip hammer.

In other tests Serota and Lowther found almost identical N values with a Pilcon hammer and the original British type of SPT hammer, Fig. 4(a), released by one turn of rope on a small

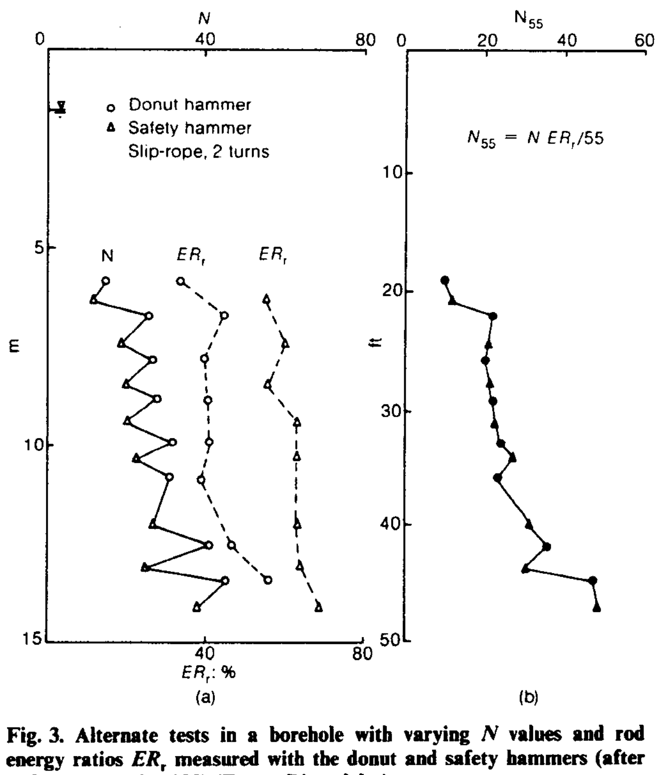

Fig. 3. Alternate tests in a borehole with varying N values and rod energy ratios ER_r measured with the donut and safety hammers (after Robertson *et al.*, 1983) (Fraser River delta)

Table 2. American average rod energy ratios for two types of hammer and anvil, with the two-turn slip-rope release method

Donut		Safety		Notes	Reference
ER_r: %	Number of tests	ER_r: %	Number of tests		
53	4	72	9	'Laboratory' tests	Kovacs & Salomone (1982)
48	8	52	9	Various field rigs	Kovacs & Salomone (1982)
		55	24	See Fig. 5	Schmertmann & Palacios (1979)
		52	5		Schmertmann & Palacios (1979)
48	23			For $N = 15$–45	Robertson *et al.* (1983)
43	8	62	8	See Fig. 3	Robertson *et al.* (1983)
45		55		Typical field values	

Table 3. Japan: rod energy ratios for the donut-type hammer and small anvil with the Tombi and two-turn slip-rope release methods

ER_r: %		N ratio	Notes	Reference
Tombi	Slip-rope			
80–90	63–72	(1·25)	N ratio deduced	Nishizawa, Fuyuki & Uto (1982)*
80	67	(1·2)	from ER_r values	Kovacs & Salomone (1982)
76			Mean of 10 tests	Kovacs, Yokel, Salomone & Holtz (1984)†
		1·2	See Fig. 1	
78	65	1·2	Typical values	

* Summarized by Tokimatsu & Yoshimi (1983).
† Quoting Decker (1983).

Table 4. Rod energy ratios for trip hammers

Hammer	ER_r: %	Reference
Pilcon type	58	Liang (1983)
Pilcon	62	Douglas & Strutznsky (1984)
Pilcon	55	Kovacs *et al.* (1984)*

* Quoting Decker (1983).

Table 5. Energy ratios and dynamic efficiency

	Release			Hammer system			ER_r: %
	Type	Cathead	ER_v: %	Hammer	Anvil weight: kg	η_d	
Waterways Experiment Station	Trip	—	100	Vicksburg	0	0·83	83
Japan	Tombi	—	100	Donut	2	0·78	78
Japan	Slip-rope (2 turns)	Small	83	Donut	2	0·78	65
USA	Slip-rope (2 turns)	Large	70	Safety	2·5	0·79	55
UK	Slip-rope (1 turn)	Small	85	Old standard	3	0·71	60
USA	Slip-rope (2 turns)	Large	70	Donut	≈12	0·64	45
UK	Trip	—	100	Pilcon	19	0·60	60

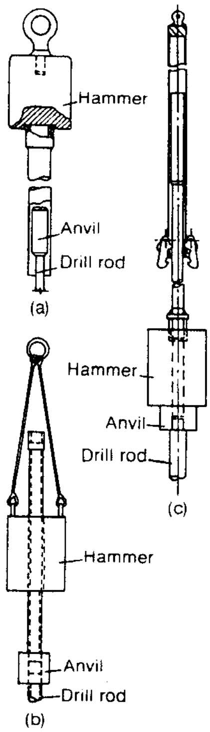

Fig. 4. SPT hammers: (a) old standard; (b) donut; (c) trip

(100 mm) cathead. The latter system therefore gives a rod energy ratio of about 60%.

Dynamic efficiency

If the velocity and rod energy ratios are known the dynamic efficiency η_d can be deduced from equation (7)

$$ER_r = \eta_d ER_v$$

Values of η_d found in this way are listed in Table 5. They lie between 0·83 and 0·6, and appear to depend principally on the anvil weight. No doubt other factors are involved but until further information is available the assumption can be made that η_d is about 0·7–0·8 for light-weight anvils and between 0·6 and 0·7 for heavy anvils.

Standard rod energy ratio

It is clearly necessary to normalize the N values measured by any particular method to some standard rod energy ratio. In the USA ratios of 50% or 55% have been suggested, but a better value is 60% as proposed by Seed *et al.* (1984).

N values measured with a known or estimated ER_r value can be normalized to this standard by the conversion

$$N_{60} = N \frac{ER_r}{60} \tag{8}$$

The foregoing data are assembled in Table 6 along with conversion factors for normalizing to $ER_r = 60\%$, the factors being rounded off to the nearest 5%.

Effect of rod length

Values of ER_r in Table 6 are for rod lengths of 10 m or more. Wave equation studies (Schmertmann & Palacios, 1979) indicate that the theoretical maximum ratio thereafter decreases with decreasing rod length as shown in Fig. 5. Test results, also plotted in Fig. 5, follow this trend. Appropriate correction factors are given in Table 7. The weight or stiffness of the rod stem, of a given length, appears to have little effect (Brown, 1977; Matsumoto & Matsubara, 1982).

Sampler without liners

The modern American sampler is equipped with liners. Often these are omitted, giving an internal diameter of $1\frac{1}{2}$ in instead of the standard $1\frac{3}{8}$ in. Comparative tests (summarized in Seed *et al.* (1984)) show on average that the sampler with liners requires about 20% more blows per foot penetration than does a sampler with the

Table 6. Summary of rod energy ratios

	Hammer	Release	ER_r: %	$ER_r/60$
Japan	Donut	Tombi	78	1·3
	Donut	2 turns of rope	65	1·1
China	Pilcon type	Trip	60	1·0
	Donut	Manual	55	0·9
USA	Safety	2 turns of rope	55	0·9
	Donut	2 turns of rope	45	0·75
UK	Pilcon, Dando,	Trip	60	1·0
	old standard	2 turns of rope	50	0·8

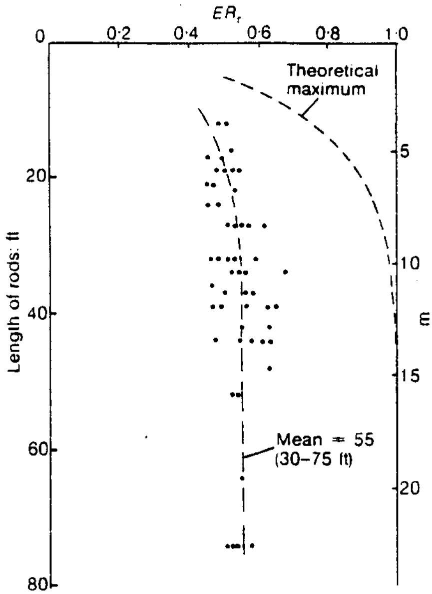

Fig. 5. Effect of rod length for a safety hammer with two-turn slip-rope (after Schmertmann & Palacios, 1979)

liners omitted: rather fewer in loose sands and rather more in dense sands.

Effect of borehole diameter

In its original form the SPT was carried out from the bottom of 2½ in or 4 in dia. wash borings. The best modern practice still adheres to these dimensions. In Japan, for example, tests are mostly made from 66 mm (2·6 in) or 86 mm (3·4 in) boreholes and virtually never in holes larger than 115 mm (Yoshimi & Tokimatsu, 1983). However, in many countries 150 mm (6 in) test boreholes are common, and even 200 mm (8 in) boreholes are permitted (Nixon, 1982).

The effect of testing from relatively large boreholes in cohesive soils is probably negligible, but in sands there are indications that appreciably lower N values may result (Lake, 1974; Sanglerat & Sanglerat, 1982). More information is urgently required on this aspect of the subject but, for the present, minimum (and therefore conservative) correction factors to allow for the effect of testing in over-large boreholes may be suggested, as given in Table 7.

Illustrative examples

Consider six tests each with a measured N value of 20 and the same rod length of more than 10 m

(*a*) in the USA with the original standard sampler, a 4 in borehole, donut hammer and 12 kg anvil, and two turns of rope on a large diameter cathead: $N_{60} = 20 \times 0{\cdot}75 = 15$

(*b*) as in (*a*) but using a modern American sampler without liners in a 6 in borehole: $N_{60} = 20 \times 0{\cdot}75 \times 1{\cdot}2 \times 1{\cdot}05 = 19$

(*c*) in Pakistan with a standard sampler, 100 mm borehole, donut hammer with 7 kg anvil ($\eta_d = 0{\cdot}7$) and manual release ($ER_v = 0{\cdot}85$): $N_{60} = 20 \times 1{\cdot}0 = 20$

(*d*) in the UK with a standard sampler, 6 in borehole and a Pilcon or Dando hammer: $N_{60} = 20 \times 1{\cdot}0 \times 1{\cdot}05 = 21$

(*e*) in Japan with a standard sampler, 86 mm borehole, donut hammer with 2 kg anvil and two turns of rope on a small diameter cathead: $N_{60} = 20 \times 1{\cdot}1 = 22$

(*f*) as in (*e*) but with a Tombi trigger release: $N_{60} = 20 \times 1{\cdot}3 = 26$

Table 7. Approximate corrections to measured N values

Rod length:	>10 m	1·0
	6–10 m	0·95
	4–6 m	0·85
	3–4 m	0·75
Standard sampler		1·0
US sampler without liners		1·2
Borehole diameter:	65–115 mm	1·0
	150 mm	1·05
	200 mm	1·15

Finally, if in any of these tests the rod length had been 5 m (i.e. the test carried out at a depth of about 4 m below ground level), instead of 10 m or more, N_{60} would be 15% lower. For instance in case (*c*)

$$N_{60} = 0{\cdot}85 \times 20 \times 1{\cdot}0 = 17$$

Now the difference between $N_{60} = 17$ and $N_{60} = 22$, for example, could mean the difference between liquefaction or no danger of liquefaction in a sand subjected to a cyclic stress ratio of 0·2 in a 7½ magnitude earthquake (Seed, Idriss & Arango, 1983). The importance of making proper corrections to measured N values is therefore apparent, but it will also be noted that, as a fortunate consequence of compensating effects, several of the modern test procedures, e.g. cases (*b*), (*d*) and (*e*), give rather similar results.

LABORATORY TESTS ON NORMALLY CONSOLIDATED SANDS

Full-scale laboratory tests have been made on two sands (GHC and GHF) at the US Bureau of Reclamation (Gibbs & Holtz, 1957) and on three sands (PR, SCS and RBM) at the Waterways

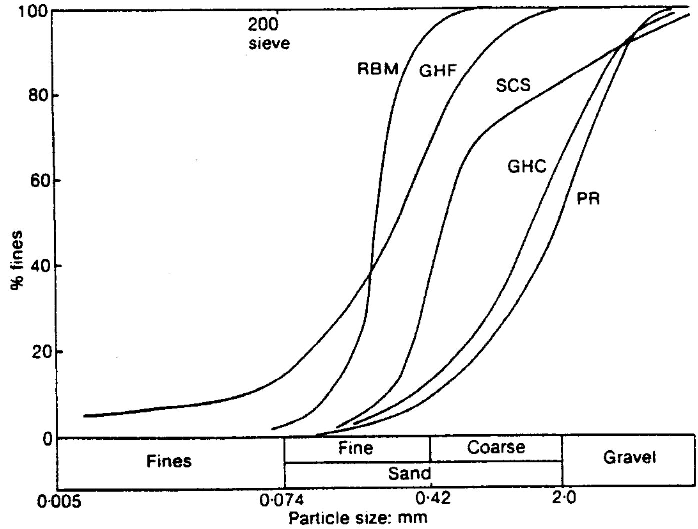

Fig. 6. Grading curves of sands in laboratory tests

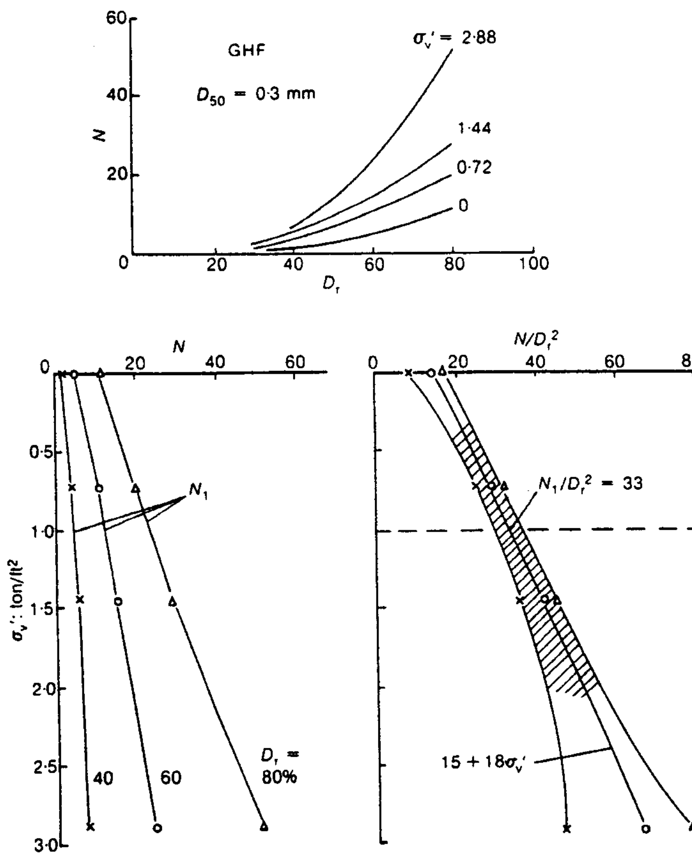

Fig. 7. Laboratory tests

Experiment Station (WES) (Bieganousky & Marcuson, 1976; Bieganousky & Marcuson, 1977). Grading curves are plotted in Fig. 6, the division between 'fine' and 'coarse' sands being taken at $D_{50} = 0{\cdot}4$ mm as in Japanese practice (Japanese Society of Soil Mechanics and Foundation Engineering, 1979).

The tests, as illustrated in Fig. 7, show in any particular sand for effective overburden pressures less than 2 ton/ft^2 and for relative densities in the range 40–80%:

(*a*) the blow count N increases almost linearly with overburden pressure σ_v' at a constant relative density D_r

(*b*) at a constant overburden pressure, N increases roughly as D_r^2: thus to a first approximation, as pointed out by Meyerhof (1957)

$$\frac{N}{D_r^2} = a + b\sigma_v' \tag{9}$$

but when the whole set of tests is considered it is seen that

(*c*) at a given relative density and overburden pressure, N is higher for sands with a larger mean grain size (D_{50}).

Average values of the parameters a and b, within the limited range of σ_v' and D_r mentioned, are given in Table 8.

It is convenient to characterize a sand by the blow count N_1 normalized to an overburden pressure of 1 ton/ft^2 (which can be taken also as 1 kg/cm^2 or 100 kPa) or, more generally, by the parameter N_1/D_r^2 where, as in equation (9), D_r is expressed as a ratio, not a percentage. In the tests N_1 is found simply by interpolation (see Fig. 7). Values of N_1 and N_1/D_r^2 are listed in Table 8 for each sand at relative densities of 40%, 60% and 80%.

At the WES the maximum void ratio e_{max} was determined by pouring air-dry sand through a 1 in dia. spout with a free fall of 1 in, and the minimum void ratio e_{min} was found by vibrating dry sand on a shaking table for eight minutes. The relative density is then calculated from the usual expression

$$D_r = \frac{e_{max} - \bar{e}}{e_{max} - e_{min}} \tag{10}$$

where $\bar{e}$ is the void ratio of the sand under test.

An American sampler was used without liners, and a short rod typically about 8 ft long, employing a hydraulically operated Vicksburg trip hammer falling directly on the head of the rod stem. Measurements of the rod energy ratio of this system for the WES by Schmertmann in 1979 gave $ER_r = 0{\cdot}83$, but factors of about 0·65 and 1·20 respectively have to be applied to allow for 8 ft rods and the larger than standard internal diameter of the sampler. The appropriate conversion factor to obtain N_{60} values is therefore 1·1.

The normalized values $(N_1)_{60}$ and $(N_1)_{60}/D_r^2$ are given in Table 8 together with the corresponding corrected parameters a and b.

Insufficient information is available to allow an estimate to be made of the rod energy ratio applicable to the US Bureau of Reclamation tests.

Table 8. Laboratory tests

Sand	Tested	D_{50}: mm	UC*	Fines: %	D_r:	N_1	$\frac{N_1}{D_r^2}$	$\frac{N}{D_r^2}$	$\frac{ER_r}{60}$	$(N_1)_{60}$	$\frac{(N_1)_{60}}{D_r^2}$	$\frac{N_{60}}{D_r^2}$
PR	Wet	2·0	5·3	0	0·4	7·5	47		1·1†	8	52	
					0·6	19	53	$30 + 22\sigma_v'$		21	58	$33 + 24\sigma_v'$
					0·8	37	58			41	64	
GHC	Dry and moist	1·5	5·5	0	0·4	6·5	40					
					0·6	14·5	40	$18 + 22\sigma_v'$				
					0·8	25	39					
SCS	Wet	0·51	2·5	4	0·4	7	44		1·1†	7·5	48	
					0·6	16	44	$21 + 24\sigma_v'$		18	48	$23 + 26\sigma_v'$
					0·8	29	45			32	49	
RBM	Wet	0·23	1·8	2	0·4	5·5	34		1·1†	6	37	
					0·6	12	33	$16 + 17\sigma_v'$		13	36	$17 + 19\sigma_v'$
					0·8	21	33			23	36	
GHF	Dry	0·3	7	14	0·4	4·5	28					
					0·6	12	33	$15 + 18\sigma_v'$				
					0·8	23	36					

* Uniformity coefficient.

† Includes a correction for no liners.

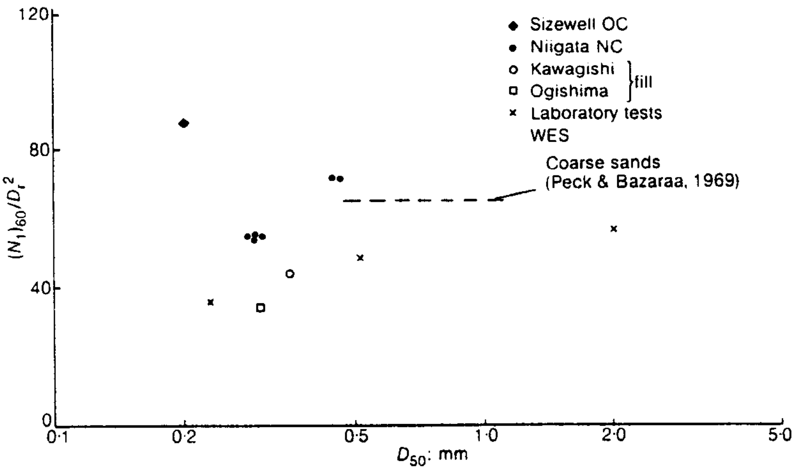

Fig. 8. Effect of particle size, ageing and overconsolidation

However, the results of these tests are broadly compatible with those at the WES.

Average values of $(N_1)_{60}/D_r^2$ for the three WES sands are plotted against D_{50} in Fig. 8. The tendency to increase with increasing grain size is clearly seen; it is probably related to a similar trend in ϕ (at a given relative density).

For sands RBM and SCS the relationship between $(N_1)_{60}$ and D_r, and the increase in N_{60}/D_r^2 with effective overburden pressure, are shown in Figs 9 and 10. These curves can be taken as typifying laboratory tests on fine and medium–coarse normally consolidated sands.

The effect of overconsolidation is considered later.

FIELD DATA ON NORMALLY CONSOLIDATED SANDS

Investigations in Japan during the past few years have yielded reliable measurements of relative density and grain size coupled with well-defined SPT procedures.

Ogishima Island

Samples from Ogishima Island, in a fine sand fill (Saito, 1977), were taken with a compressed air

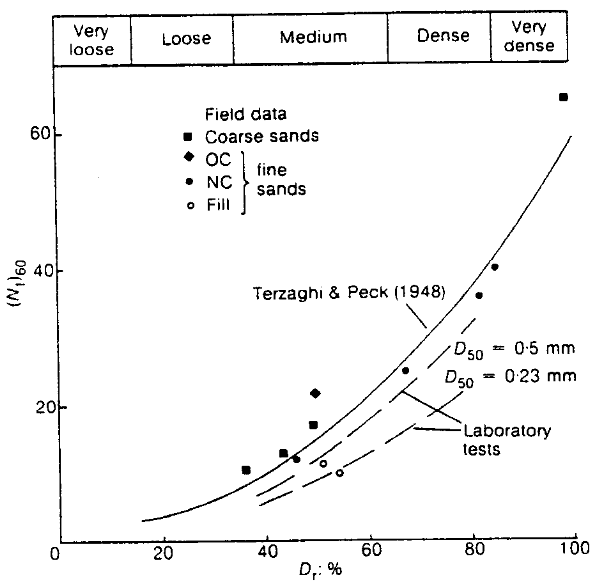

Fig. 9. Effect of relative density

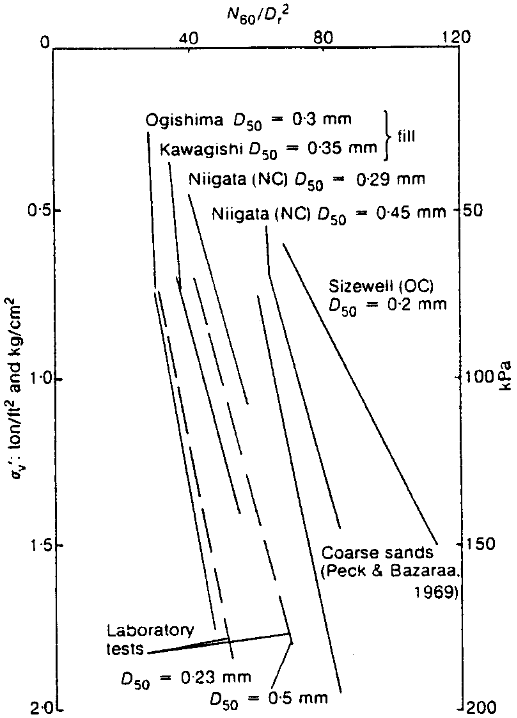

Fig. 10. Effect of overburden pressure

sampler of the Bishop type but equipped with a piston. The penetration and hence the sample volume can therefore be measured, and from the dry weight of the whole sample its average void ratio is known.

Maximum and minimum void ratios were determined respectively by the tilting test and by tapping a 120 cm^3 container filled with dry sand under a load of 1 kg/cm^2.

The SPT procedure followed usual Japanese practice: a standard sampler, 66–86 mm dia. boreholes stabilized with drilling mud and casing, a donut hammer with a lightweight (2 kg) anvil, released by a two-turn 15 mm manila slip-rope on a 120 mm cathead. The tests were made in 1974 soon after most of the reclamation work had been completed.

The relative density, below water level, remains almost constant at 52–55% and the N values increase steadily from 8 to 14, at depths from about 5 m to 15 m: see Fig. 11 where each point is the average of several tests.

The $ER_r/60$ ratio is well established at around 1·1, but rod length corrections are applicable to the two uppermost points in Fig. 11. The results conform to a linear relationship

$$\frac{N_{60}}{D_r^2} = 17 + 17\sigma_v' \qquad (\sigma_v' \text{ in kg/cm}^2)$$

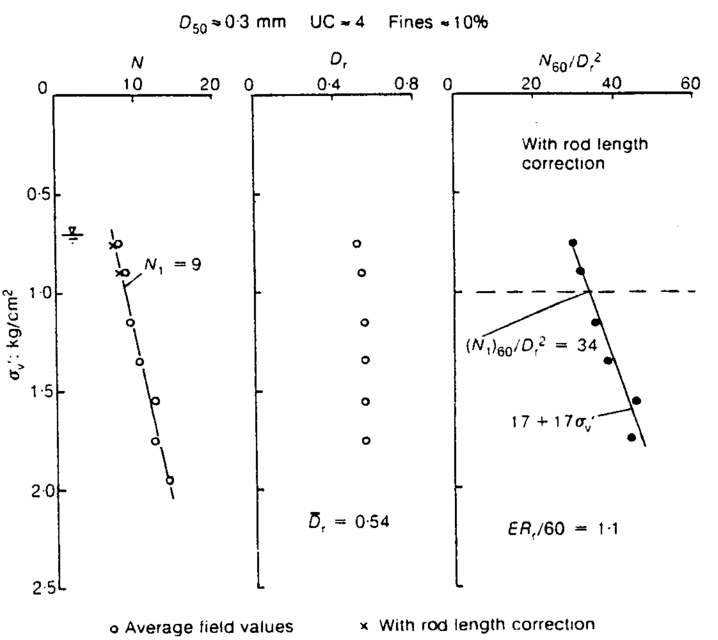

Fig. 11. Results from Ogishima Island, sand fill

Hence

$$(N_1)_{60}/D_r^2 = 34$$

$$(N_1)_{60} = 10$$

for $D_r = 0.54$.

The results, plotted in Figs 8–10, are similar to those from laboratory tests on sand RBM, which has a comparable D_{50} grain size but a smaller content of fines. However, an exact comparison is not possible since different methods were used for determining the limiting void ratios, tests which unfortunately are not internationally standardized.

Kawagishi-cho

The tests at Kawagishi-cho were made in land reclaimed between 1930 and 1950 (approximately) by depositing about 15 m of sand in a natural inlet close to the mouth of the Shinano River in Niigata. Recent investigations (Ishihara & Koga, 1981) include sampling by means of a large (200 mm) diameter sampler (Ishihara & Silver, 1977) and a 76 mm piston sampler of the Osterberg type (Ishihara, Silver & Kitagawa, 1979) in two closely spaced boreholes adjacent to the SPT borehole.

Both types of sample were allowed to drain for 24 hours. Small brass tubes (50 mm dia. × 100 mm long) were then pushed into the large diameter samples and these, and also the Osterberg samples, were frozen on site to prevent disturbance during transport to the laboratory. The SPT procedure conformed to modern Japanese methods using the two-turn slip-rope release, as described previously for the Ogishima site. The N values, plotted in Fig. 12, are results of individual tests.

Natural void ratios were determined from the dry density. To measure the maximum void ratio oven-dry sand was carefully spooned into a mould 60 mm in diameter and 40 mm deep with a negligible height of drop. The procedure for determining the minimum void ratio consisted of placing the sand in a water-filled mould 52 mm in diameter and 80 mm high, applying a load of 1 kg/cm² and vibrating on a shaking table with a single amplitude displacement of 7 mm at a frequency of 17 Hz for three minutes.

Each relative density point in Fig. 12 is the

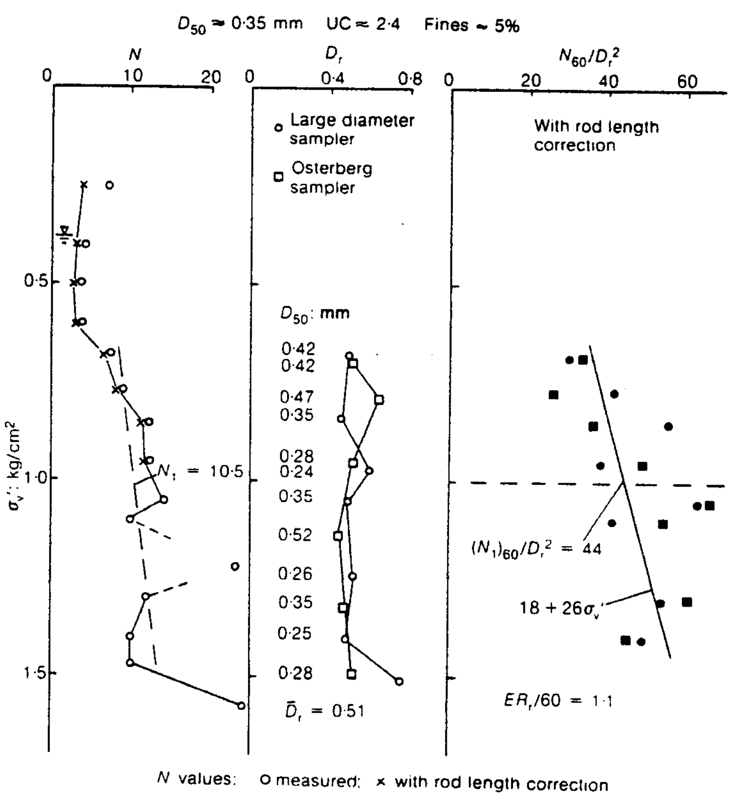

Fig. 12. Results from the Kawagishi-cho site, sand fill

mean from three or four specimens from the samples. There is a considerable scatter, but average values from both types of sampler, for depths between 5 mm and 14 mm, are almost identical at 51%. Between these depths no obvious trend exists in D_r or in the mean particle size. It is therefore reasonable to treat this body of sand as having broadly uniform properties, and the results correspond approximately to the equation

$$\frac{N_{60}}{D_r^2} = 18 + 26\sigma_v' \qquad (\sigma_v' \text{ in kg/cm}^2)$$

Hence

$$(N_1)_{60}/D_r^2 = 44$$

$$(N_1)_{60} = 11\cdot5$$

for $D_r = 0\cdot51$.

These values, plotted in Figs 8–10, are higher than those for the Ogishima sand fill. The difference may be attributed partly to a somewhat larger grain size at Kawagishi and probably also to the fact that the fill had been deposited at least 25 years before being tested.

Niigata station

At a site near Niigata railway station SPTs were made at six equally spaced boreholes on the circumference of a 4 m dia. circle, three by the slip-rope method with two turns of 15 mm manila rope on a 130 mm cathead and three by the Tombi release method, using in all cases a standard sampler, 66 mm boreholes stabilized with drilling mud and a donut hammer with a 3 kg anvil (Yoshimi & Tokimatsu, 1983). Below 3 m there is a fine sand, becoming dense at a depth around 6 m. At the centre of the test circle a large block sample was obtained in the dense sand by the in situ freezing method (Yoshimi, Tokimatsu, Kaneko & Makihara, 1984).

On specimens cut out from the block and hand trimmed to size, cyclic triaxial tests and density measurements were made, and on the dried sand maximum and minimum density tests were carried out in accordance with the new Japanese standard (Japanese Society for Soil Mechanics and Foundation Engineering, 1979). For the minimum density, sand was poured through a small (12 mm) diameter funnel, with effectively zero height of fall, into a mould 60 mm in diameter and 40 mm high. For the maximum density the sand was placed in the same mould in ten layers, the mould being tapped 100 times with a wooden hammer after placing each layer. Comparative tests on a standard (Toyoura) sand showed only minor differences in e_{max} and e_{min} determined by these procedures and the methods adopted before 1979 by Ishihara and co-workers.

Average results from several specimens at each of three depths at the station, between 9·25 m and 9·8 m, are given in Table 9 together with the relevant N values, each of the latter being the mean of three tests by Tombi and three by slip-rope release. The results can be taken as having a high reliability. It may be added that the triaxial tests show $\phi' = 41\cdot4°$ at the maximum stress ratio.

Niigata, south bank site

Large diameter and Osterberg samples were taken in a medium dense fine sand of Pleistocene age near the south bank of Shinano river at the Shoma Bridge, about 1·2 km from Niigata station (Ishihara & Koga, 1981). The techniques of sampling and testing were as described for the Kawagishi site. Results are given in Table 10 along with N values derived by interpolation from tests in an adjacent borehole.

Table 9. Niigata station*

Mean depth: m	σ_v': kg/cm²	N		N_{60}		$\bar{e}$	e_{max}	e_{min}	D_r	D_{50}: mm
		Tombi	Slip-rope	Tombi	Slip-rope					
9·1	1·05	30	39	39	43					
9·25						0·84	1·22	0·76	0·82	0·29
9·6						0·82	1·20	0·75	0·87	0·28
9·8						0·84	1·20	0·78	0·86	0·30
9·9	1·11	32	39	42	43					
9·5	1·08	31	39	$\underbrace{40\cdot5 \quad 43}_{41\cdot6}$		0·83	1·21	0·76	0·85	0·29

* $ER_r/60 = 1\cdot3$ (Tombi) and $ER_r/60 = 1\cdot1$ (slip-rope); rod length correction, 1·0; at $\sigma_v' = 1\cdot08$ kg/cm²
$C_N = 2/(1 + \sigma_v') = 0\cdot96$; $N_{60}/D_r^2 = 57\cdot5$; $(N_1)_{60} = 40$; $(N_1)_{60}/D_r^2 = 55$.

Table 10. Niigata, south bank*

Depth: m	σ_v': kg/cm²	N	N_{60}	$\bar{e}$	e_{max}	e_{min}	D_r	$\frac{N_{60}}{D_r^2}$	$(N_1)_{60}$†	$\frac{(N_1)_{60}}{D_r^2}$	D_{50}: mm
4·2	0·43	13									
4·45	0·45	18‡	18	0·80	1·10	0·65	0·67	40	25	55	0·28
4·9	0·50	27‡	27	0·72	1·08	0·64	0·82	40	36	54	0·29
5·2	0·52	33									

* $ER_r/60 = 1·1$; rod length correction, 0·9.
† C_N taken as $2/(1 + \sigma_v')$.
‡ By interpolation.

Niigata, road site

The same techniques were used to investigate an ancient alluvial deposit of the River Shinano at a site beside a road situated on an old flood bank of the river, about 1·5 km upstream of the Showa Bridge. Under 4 m of fill, dating from the early years of this century, the alluvial sand has a relative density typically around 50% and the grain size tends to increase with depth.

Two groups of results from depths of about 8–9 m and 13–14 m are given in Table 11.

The grading and mean grain size of the 8–9 m samples is closely similar to the sand at Niigata station and the south bank, although the relative densities range from 45% to 85% at the three sites. Nevertheless the parameter $(N_1)_{60}/D_r^2$ is constant, equal to 55, and the sands can be taken as belonging geotechnically to a single group. The results, plotted in Fig. 13, lead to the relationship

$$\frac{N_{60}}{D_r^2} = 27 + 28\sigma_v' \qquad (\sigma_v' \text{ in kg/cm}^2)$$

The alluvium is normally consolidated, and the mutually consistent characteristics of the deposits at the south bank and station sites strongly indicate that they also are in this condition.

Comparisons with the Kawagishi fill, of rather similar grain size and tested by identical procedures, clearly show the effects of ageing (Figs 8–10).

Ageing of sands

Direct evidence of the increased resistance of sands which have long been under consolidation pressure is provided by cyclic triaxial tests on samples from the station site (Yoshimi *et al.*, 1984). The stress ratio required to cause a double-amplitude strain of 5%, at ten cycles, is found to be at least 60% greater in the undisturbed samples than in freshly reconstituted samples at the same density. Similar results have been found for alluvial sand in Niigata with a relative density around 52% (Ishihara, 1985) but, significantly, a smaller difference is measured in the Kawagishi fill.

There is little doubt that the effect is time controlled. Seed (1979) reported a 25% increase in resistance to the development of initial liquefaction in laboratory samples tested after 100 days

Table 11. Niigata road site

Depth: m	σ_v': kg/cm²	N	Rod length correction	N_{60}	$\bar{e}$	e_{max}	e_{min}	D_r	$\frac{N_{60}}{D_r^2}$	C_N	$(N_1)_{60}$	$\frac{(N_1)_{60}}{D_r^2}$	D_{50}: mm	UC
8·1	0·98	12	0·98	12·9										
8·4	1·00				0·87	1·15	0·59						0·30	1·8
8·4	1·00				0·84	1·15	0·61						0·28	1·6
9·1	1·07	10	0·99	10·9										
9·2	1·09				0·96	1·16	0·67						0·34	1·4
9·3	1·09				1·01	1·18	0·68						0·30	1·8
	1·04			11·9	0·92	1·16	0·64	0·46	56	0·98	11·7	55	0·30	1·7
12·8	1·40				0·77	0·99	0·55						0·48	1·7
12·8	1·40				0·77	1·01	0·56						0·48	1·8
13·1	1·45	17	1·00	18·7										
13·8	1·50				0·79	1·07	0·58						0·39	1·8
13·8	1·50				0·86	1·05	0·56						0·65	1·9
14·3	1·55	20	1·00	22·0										
	1·47			20·3	0·80	1·03	0·56	0·49	84	0·85	17·3	72	0·45	1·8

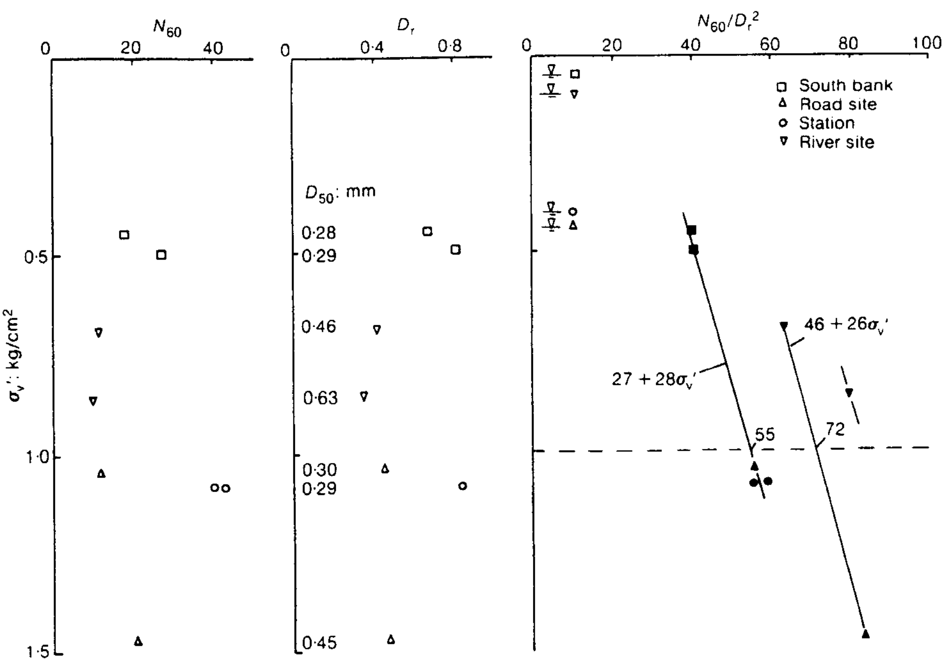

Fig. 13. Niigata sand strata

consolidation and briefly presented some field data indicating an increase of 50–100% for consolidation periods of the order of 100–1000 years.

Laboratory tests by Daramola (1980) indicate that the mechanics of the process is essentially an increase in stiffness. In drained triaxial tests on a river sand, with $D_{50} = 0{\cdot}3$ mm, he found that the secant modulus increased by 60% and 100% respectively in tests made after 30 days and 150 days consolidation under 400 kPa, compared with the results of freshly prepared samples, all at virtually the same relative density of 67%. The strains to failure were correspondingly reduced, from 5·5%, to 3·5% and 2·5% at 30 days and 150 days, but interestingly the angle of internal friction was scarcely changed (at $\phi' = 39$–$39{\cdot}5°$), the interparticle bonds presumably having been broken on the approach to failure.

Niigata, river site

Situated close to the road site, the Niigata river site had been in the Shinano flood plain until 1955, when the area was reclaimed by dumping about 4 m of fill. From borings adjacent to an SPT borehole large diameter samples were taken (Ishihara, Silver & Kitagawa, 1978) and tested by the techniques described for the south bank and river sites. The alluvial sand here is somewhat coarser, and at depths around 7 m compares with the deeper sand at the road site, both having $D_{50} \approx 0{\cdot}45$ mm and relative densities around 45%. If the data are combined (Fig. 13) they give

$$\frac{N_{60}}{D_r^2} = 46 + 26\sigma_v'$$

$$(N_1)_{60}/D_r^2 = 72$$

The coarser sand ($D_{50} \approx 0{\cdot}65$ mm) at depths around 9 m at the river site has a still higher value of $(N_1)_{60}/D_r^2 = 84$, but the relative density is only 36% and comparisons on the basis of N/D_r^2 may not be exact.

The results are given in Table 12.

Dense coarse sands

Peck & Bazaraa (1969) plotted a large number of N values against σ_v' for very dense coarse sands tested below groundwater level, and they drew a line for $D_r = 100\%$. This is expressed by the equation

$$\frac{N}{D_r^2} = 60 + 25\sigma_v'$$

for $\sigma_v' > 0{\cdot}75$ ton/ft².

It can be assumed that most of the tests will have been made with the original standard sampler and hammer using a two-turn slip-rope release on a large diameter cathead. The appropriate value of $ER_r/60$ is therefore about 0·75: see

Table 12. Niigata river site*

Depth: m	σ_v': kg/cm²	N	Rod length correction	N_{60}	$\bar{e}$	e_{max}	e_{min}	D_r	$\frac{N_{60}}{D_r^2}$	C_N	$(N_1)_{60}$	$\frac{(N_1)_{60}}{D_r^2}$	D_{50}: mm	UC	Fines
6·5	0·65	11	0·95	11·5	0·84	1·06	0·53						0·46	2·0	0
7·0	0·69	9	0·96	9·5									0·45	2·1	1
7·5	0·73	13	0·965	13·8	0·79	0·97	0·55								
	0·69			11·6	0·81	1·02	0·54	0·425	64	1·12	13·0	72	0·46	2·0	1
8·5	0·82	10	0·98	10·8	0·84	0·99	0·55						0·55	2·7	1
9·0	0·86	9	0·985	9·7									0·70	3·0	3
9·5	0·90	9	0·99	9·8	0·80	1·00	0·48								
	0·86			10·1	0·82	0·99	0·51	0·355	80	1·05†	10·6	84	0·63	2·8	2

* $ER_r/60 = 1·1$.
† Assuming $C_N = 3/(2 + \sigma_v')$.

the section on illustrative examples earlier. Thus, in round numbers

$$\frac{N_{60}}{D_r^2} = 45 + 20\sigma_v'$$

$$(N_1)_{60}/D_r^2 = 65$$

This result, although approximate, gives a useful check on SPT results in coarse-grained sands.

EFFECTS OF OVERCONSOLIDATION

General considerations

It is reasonable to suppose, and model tests confirm (Clayton, Hababa & Simons, 1985), that penetration resistance in a given sand is controlled by the mean effective stress

$$\bar{\sigma}' = \tfrac{1}{3}(\sigma_v' + 2\sigma_h')$$

or

$$\bar{\sigma}' = \frac{\sigma_v'}{3}(1 + 2K_0) \qquad (11)$$

where

$$K_0 = \sigma_h'/\sigma_v' \qquad (12)$$

Thus if K_{ONC} is the in situ stress ratio in a normally consolidated sand, with an overconsolidation ratio OCR of 1·0, the same sand when overconsolidated (OCR > 1) will have an increased N value given by the expression

$$\frac{N}{D_r^2} = a + C_{oc}b\sigma_v' \qquad (13)$$

where

$$C_{oc} = \frac{1 + 2K_0}{1 + 2K_{ONC}} \qquad (14)$$

An analysis of tests on many soils has shown (Mayne & Kulhawy, 1982) to a first approximation that

$$K_{ONC} = 1 - \sin\phi' \qquad (15)$$

$$K_0 = K_{ONC}(\mathrm{OCR})^{\sin\phi'} \qquad (16)$$

Values of K_0 and C_{oc} are given in Table 13 from which it can be seen that quite a modest degree of overconsolidation may be expected to increase the coefficient of σ_v' in equation (13) by 20–40%, and for heavy overconsolidation the increase could be at least 100%, corresponding to K_0 values in excess of 1·0.

Laboratory tests

Some experimental results on sand RBM (Bieganousky & Marcuson, 1976) illustrate the effect of increasing OCR from 1 to 3 (Fig. 14). For $\sigma_v' < 2$ ton/ft² the tests show on average

Table 13. Values of K_0 and C_{oc} from equations (15) and (16)

OCR	$\phi' = 32°$		$\phi' = 36°$		$\phi' = 40°$	
	K_0	C_{oc}	K_0	C_{oc}	K_0	C_{oc}
1	0·47	1·00	0·41	1·00	0·36	1·00
2	0·68	1·22	0·62	1·23	0·56	1·23
3	0·84	1·38	0·78	1·41	0·73	1·43
4	0·98	1·53	0·93	1·57	0·87	1·59
6	1·21	1·76	1·17	1·84	1·14	1·91
10	1·59	2·15	1·58	2·28	1·58	2·42

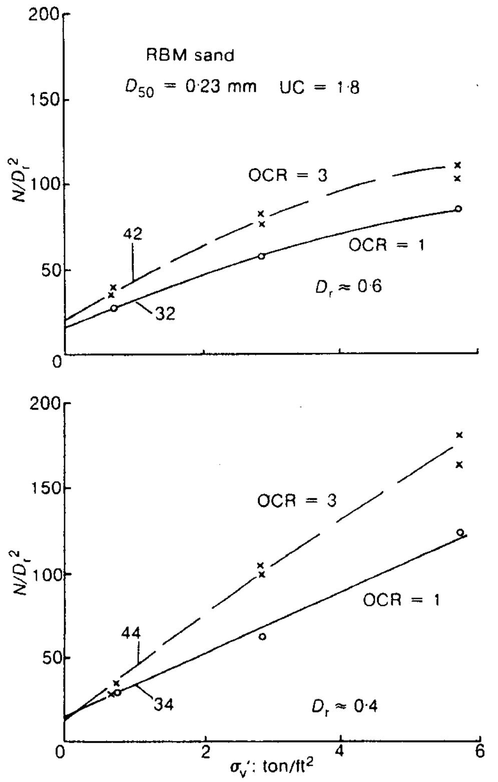

Fig. 14. Effect of overconsolidation

$$\frac{N}{D_r^2} = 16 + 17\sigma_v'$$

for OCR = 1 and

$$\frac{N}{D_r^2} = 16 + 27\sigma_v'$$

for OCR = 3. From Table 13, for sand with $\phi' = 32–36°$ (as in the tests) and OCR = 3, the coefficient C_{oc} is about 1·4; thus

$$bC_{oc} = 17 \times 1{\cdot}4 = 24$$

which is close to the measured value of 27.

Sizewell

Tests have been reported (Meigh & Nixon, 1961) on a fine uncemented sand of the Norwich Crag, at Sizewell in Suffolk. This deposit was formed in Lower Pleistocene times and has been heavily overconsolidated by the weight of an ice sheet of the Anglian glaciation, more than 250 000 years ago. In situ densities were determined at three levels in a test pit 34 ft deep, giving an average relative density of about 50%, and the SPT N values measured in two adjacent borings show a fairly uniform increase with depth (Fig. 15).

The tests were carried out in 6 in dia. boreholes using a standard sampler and the original British standard SPT hammer (Fig. 4(a)), released by two turns of 22 mm rope on a 5 in dia. cathead. From tests with this type of hammer and anvil (Serota & Lowther, 1973) the dynamic efficiency can be estimated at about 0·7 (Table 5) and the velocity energy ratio for the method of release would be around 70% (see Fig. 2). Thus $ER_r \approx 50\%$ and $ER_r/60 \approx 0{\cdot}8$, to which a minimum correction of 5% has to be added for the 6 in borehole effect.

All the tests were made above the water-table. The effective overburden pressure is therefore

$$\sigma_v' = \sigma_v - u_c \tag{17}$$

where u_c is the capillary tension. In sand with $D_{10} = 0{\cdot}1$ mm, as at Sizewell, the capillary rise h_c is of the order of 80 cm (Terzaghi, 1942). Hence

$$u_c = -\gamma_w h_c = -0{\cdot}08 \text{ ton/ft}^2$$

The N_{60} values (corrected where necessary for rod length) and effective overburden pressures

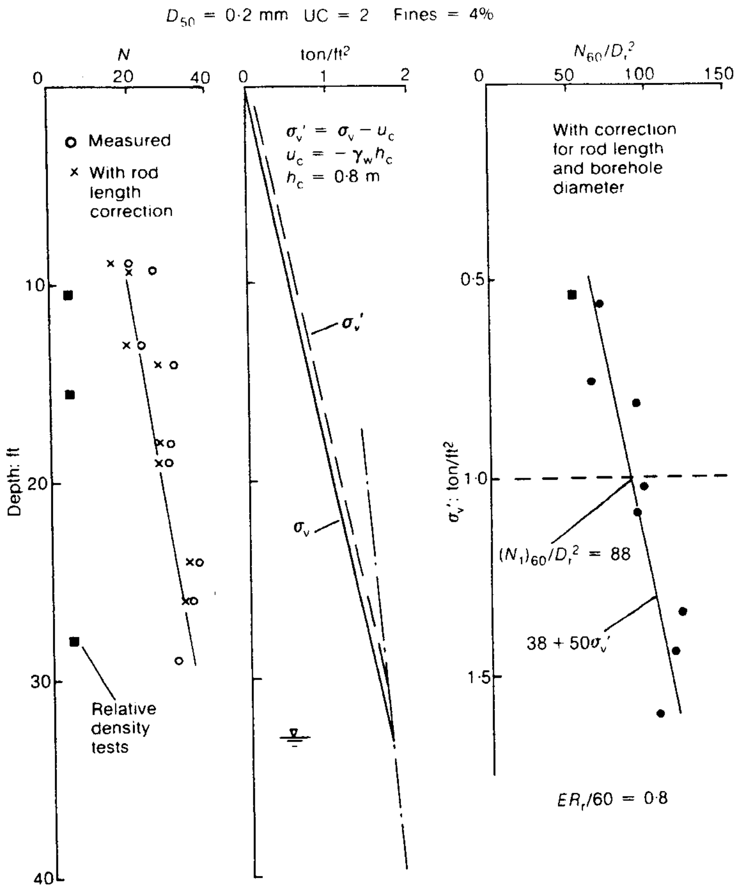

Fig. 15. Sizewell: Lower Pleistocene sand

can now be calculated, and the results (Fig. 15) give

$$\frac{N_{60}}{D_r^{2}} = 38 + 50\sigma_v'$$

Thus

$$(N_1)_{60}/D_r^{2} = 88$$

and for $D_r = 0{\cdot}5$

$$(N_1)_{60} = 22.$$

Since D_r is known only approximately these values must be subject to appreciable variation, but when compared with the normally consolidated Niigata sand of similar grain size they clearly indicate a much greater penetration resistance. This is due largely to the higher coefficient of σ_v' (50 compared with 28), which can be attributed to overconsolidation, and in part to the great age of the Sizewell sand.

EFFECT OF OVERBURDEN PRESSURE

In the field and laboratory tests described each of the sands (with one exception) is sufficiently uniform with regard to grain size and relative density to be treated as a unit, and the blow count at $\sigma_v' = 1$ ton/ft^2 is found by direct interpolation. In general, however, it is necessary to be able to estimate the N_1 value for any particular test, and this is done by means of the formula

$$N_1 = C_N N \tag{18}$$

Now if

$$\frac{N}{D_r^{2}} = a + b\sigma_v'$$

then

$$C_N = \frac{a/b + 1}{a/b + \sigma_v'} \tag{19}$$

From the data given in this Paper it will be seen that a/b varies roughly from 1·0 for fine sands of medium relative density to 2·0 for dense coarse sands when normally consolidated. The corresponding limits for C_N (plotted in Fig. 16) are

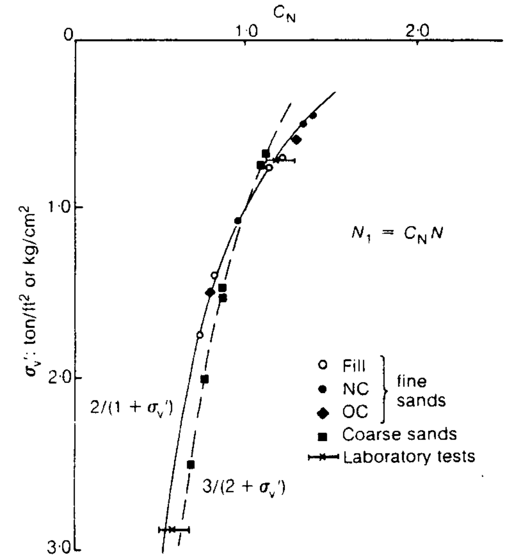

Fig. 16

$$C_N = \frac{2}{1 + \sigma_v'} \tag{20}$$

$$C_N = \frac{3}{2 + \sigma_v'} \tag{21}$$

These expressions are numerically similar to curves derived by Seed *et al.* (1983) from the WES laboratory tests for sands with $D_r = 0{\cdot}4$–$0{\cdot}6$ and $D_r = 0{\cdot}6$–$0{\cdot}8$ respectively, and equation (21) differs little from the well-known formula of Peck, Hanson & Thornburn (1974)

$$C_N = 0{\cdot}77 \log\left(\frac{20}{\sigma_v'}\right) \tag{22}$$

For detailed comparisons, see Table 14.

Table 14. Normally consolidated sands: values of C_N

σ_v': ton/ft^2	$\frac{2}{1+\sigma_v'}$	From Seed *et al.* (1983)		$\frac{3}{2+\sigma_v'}$	$0{\cdot}77 \log\left(\frac{20}{\sigma_v'}\right)$
		D_r = 40–60%	D_r = 60–80%		
0·5	1·33	1·36	1·36	1·20	1·23
0·75	1·14	1·14	1·14	1·09	1·10
1·0	1·00	1·00	1·00	1·00	1·00
1·5	0·80	0·80	0·84	0·86	0·87
2·0	0·68	0·69	0·74	0·75	0·77
2·5	0·57	0·60	0·67	0·67	0·69
3·0	0·50	0·54	0·61	0·60	0·63

For overconsolidated fine sands the a/b ratio lies between about 0·6 and 0·8. Thus to a first approximation

$$C_N = \frac{1{\cdot}7}{0{\cdot}7 + \sigma_v'} \quad (23)$$

TERZAGHI AND PECK'S CLASSIFICATION

Terzaghi & Peck (1948) gave the first classification of relative density in terms of the SPT; see Table 15. Values of D_r were assigned to this classification by Gibbs & Holtz (1957) who pointed out that the resulting N–D_r relationship corresponded, more or less, to their laboratory tests at an overburden pressure of 40 lb/in^2 or nearly 3 ton/ft^2. Clearly something was amiss, for the field experience from which Terzaghi and Peck formulated the classification had been derived from tests at the normal depths for ordinary foundations corresponding to an overburden pressure typically around 0·75 ton/ft^2 (Peck & Bazaraa, 1969).

Now at $\sigma_v' = 0{\cdot}75$ ton/ft^2 the coefficient $C_N \approx 1{\cdot}1$ and an appropriate rod energy ratio for the original American test procedure can be taken as about 0·45. Hence the $(N_1)_{60}$ values in Table 15 are determined. A plot of these against the D_r values assigned by Gibbs and Holtz, see Fig. 9, gives about the best single line that could be drawn through the field data, falling midway between points for normally consolidated fine and coarse sand deposits. For $D_r > 0{\cdot}35$ the correlation can be expressed to a good approximation by the parameter

$$(N_1)_{60}/D_r^2 = 60$$

which from the data already given in this Paper can be recognized as highly typical.

Thus, despite repeated criticisms, there is nothing wrong with the N and D_r values; indeed they could not have been better chosen. Equally there is nothing wrong with the laboratory tests; they apply to cases where the effects of ageing are minimal.

Finally it is interesting to note a statistical analysis of more than 50 field tests in sands by Schultze & Menzenbach (1961). Their data, although presented as a logarithmic relationship, fit closely to the equation

$$\frac{N}{D_r^2} = 17 + 22\sigma_v'$$

in the range $\sigma_v' = 0{\cdot}5$–$1{\cdot}5$ kg/cm^2 and for $D_r = 0{\cdot}4$–$0{\cdot}9$. Professor Schultze later questioned the accuracy of the relative density measurements, but if the SPT procedure had a moderately high efficiency (e.g. $ER_r/60 = 1{\cdot}1$) the results would conform quite well with recent tests on normally consolidated sands.

SUMMARIZING REMARKS AND CONCLUSIONS

Experience shows that wide variations in N values can occur as between different sands, even at a given overburden pressure and constant relative density. Part of the variation is arbitrary, being due simply to the use of different test procedures. This effect can be eliminated by normalizing the results to a standard rod energy ratio.

The variations which remain are intrinsic to the nature of the sands being tested and fall into a consistent pattern when the effects of ageing, particle size and overconsolidation are taken into account.

Tables 8 and 16, and Figs 8–10 and 16, summarize the results of laboratory and field tests. The main conclusions are as follows.

Measured N values must be normalized to a standard rod energy ratio, and $ER_r = 60\%$ is a suitable standard.

The relationship between blow count, relative

Table 15. Terzaghi and Peck's classification*

D_r	Classification	$N(\sigma_v' = 0{\cdot}75)$	N_1	$(N_1)_{60}$	$(N_1)_{60}/D_r^2$
	Very loose				
0·15		4	4·4	3	—
	Loose				
0·35		10	11	8	65
0·5	Medium	(18)	20	15	60
0·65		30	33	25	59
	Dense				
0·85		50	55	42	58
	Very dense				
1·00		(70)	77	58	58

* $C_N = 1{\cdot}1$; $ER_r/60 = 0{\cdot}75$.

Table 16. Summary of field data

Site	Type*	D_{50}: mm	UC	Fines	$\bar{e}$	e_{max}	e_{min}	D_r	σ_v': kg/cm²	N_{60}	$\frac{N_{60}}{D_r^2}$	C_N	$(N_1)_{60}$	$\frac{(N_1)_{60}}{D_r^2}$
Ogishima Island	Fill	0·3	4	10	0·80	1·08	0·57	0·54	0·75 1·00 1·75	9 10 13·5	30 34 } $17 + 17\sigma_v'$ 47	1·14 1·00 0·73	10	34
Kawagishi-cho	Fill	0·35	2·4	†	0·86	1·08	0·63	0·51	0·7 1·0 1·4	9·5 11·5 14	36 44 } $18 + 26\sigma_v'$ 54	1·22 1·00 0·82	11·5	44
Niigata station	NC	0·29	1·8	2	0·83	1·21	0·76	0·85	1·08	42	57	0·96	40	55
Niigata, south bank		0·28	2·4	†	0·80	1·10	0·65	0·67	0·45	18	40 } $27 + 28\sigma_v'$	1·39	25	55 } 55
Niigata, south bank		0·29	1·5	†	0·72	1·08	0·64	0·82	0·50	27	40	1·34	36	54
Niigata, road site		0·30	1·7	†	0·92	1·16	0·64	0·46	1·04	12	56	0·98	12	55
Niigata, road site	NC	0·45	1·8	†	0·80	1·03	0·56	0·49	1·47	20	84 } $46 + 26\sigma_v'$	0·85	17	72 } 72
Niigata, river site		0·46	2·0	1	0·81	1·02	0·57	0·43	0·69	11·5	64	1·12	13	72
Niigata, river site	NC	0·63	2·8	2	0·82	0·99	0·51	0·36	0·86	10	80	1·05‡	10·5	84
Sizewell	OC	0·2	2·0	4	0·72	0·94	0·50	0·50	0·60 1·00 1·50	17 22 28	68 68 } $38 + 50\sigma_v'$ 113	1·29 1·0 0·78	22	88

* NC, normally consolidated; OC, overconsolidated.
† Fines less than 5%.
‡ Assuming $C_N = 3/(2 + \sigma_v')$.

density and effective overburden pressure in a given sand can be represented by the expression

$$\frac{N_{60}}{D_r^2} = a + b\sigma_v'$$

(σ_v' in ton/ft^2 or kg/cm^2 or kPa/100).

The parameters a and b are nearly constant for $0{\cdot}35 < D_r < 0{\cdot}85$ and $0{\cdot}5$ kg/cm$^2 < \sigma_v' < 2{\cdot}5$ kg/cm^2.

A sand can conveniently be characterized by the parameters $(N_1)_{60}$ and $(N_1)_{60}/D_r^2$ where $(N_1)_{60}$ is the normalized blow count at $\sigma_v' = 1$ kg/cm ^{2}.

N_1 can be evaluated from the equation $N_1 = C_N N$ where, in normally consolidated sands, C_N ranges from $C_N = 2/(1 + \sigma_v')$ for fine sands of medium density to $C_N = 3/(2 + \sigma_v')$ for dense coarse sands.

For normally consolidated natural sand deposits the best average correlation between blow count and relative density is

$D_r =$ 0	15	35	50	65	85	100%
	Very loose	Loose	Medium	Dense	Very dense	
$(N_1)_{60} =$ 0	3	8	15	25	42	58

This is derived from the Terzaghi–Peck classification. For $D_r > 0{\cdot}35$ it corresponds to

$$(N_1)_{60}/D_r^2 \approx 60$$

For fine sands the N values should be reduced in the ratio 55/60, and for coarse sands the N values should be increased in the ratio 65/60. The second case corresponds to data given by Peck & Bazaraa (1969). The tendency for N values to increase with increasing particle size is probably related to the same trend in ϕ, for a given relative density.

There is evidence that the resistance of sand to deformation is greater the longer the period of consolidation. This 'ageing' effect is reflected in higher blow counts, and appears to cause an increase in the parameter a. Typical results for normally consolidated fine sands are given in Table 17.

Table 17. Effect of ageing

	Age: years	$(N_1)_{60}/D_r^2$
Laboratory tests	10^{-2}	35
Recent fills	10	40
Natural deposits	$>10^2$	55

Overconsolidation increases the coefficient b by the factor

$$\frac{1 + 2K_0}{1 + 2K_{ONC}}$$

where K_0 and K_{ONC} are respectively the in situ stress ratios for the overconsolidated and normally consolidated sand.

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