M. A. STROUD, B.A., Ph.D., A.M.I.C.E.
Ove Arup & Partners

F. G. BUTLER, B.Sc., M.I.C.E.
Ove Arup & Partners

The Standard Penetration Test and the Engineering Properties of Glacial Materials

SYNOPSIS

The process of obtaining reliable engineering data for glacial materials by conventional means is beset with sampling difficulties and problems of disturbance. This paper demonstrates that the Standard Penetration Test can be a reliable and valuable means of estimating the properties of glacial materials in situ with the considerable advantages of cheapness and simplicity.

Data from sixteen sites covering a wide range of glacial deposits in the U.K. are considered. It is shown that simple correlations relate N values to undrained shear strength and compressibility. The correlations, which depend only on Plasticity Index, are consistent with those shown to exist for a wide variety of insensitive clays and weak rocks.

The value of a quasi-elastic approach to the estimation of settlement of structures is discussed. Settlement records for a wide range of large structures and foundation materials are examined including six for structures on glacial deposits. A simple correlation between N value and the vertical drained elastic modulus is indicated. The correlation is shown to depend on plasticity index and on the factor of safety against shear failure implied by the foundation bearing stress. A design curve is proposed unifying the approach for cohesive and cohesionless materials and a comparison of settlement prediction is made with the Skempton and Bjerrum method.

INTRODUCTION

To the frustration of practising soils engineers glacial materials invariably defeat the convenient classification of either cohesionless or cohesive. Traditionally 'cohesive' techniques involving U102 open drive samplers are specified for routine site investigations in the hope that at least some samples will be retained and will give meaningful undrained shear strengths when tested in the laboratory. Even the term Boulder Clay to some extent betrays the optimism of the engineer. While boulders and clay may certainly be present the mechanical properties will frequently be primarily influenced by intermediate particle sizes.

Where samples are not retained or where the scatter of shear strength results turns interpretation into pure conjecture, the engineer relies heavily on visual inspection of the material together with Atterberg limits, moisture contents and grading analyses. While these tests are an important means of bridging the 'cohesive' - 'cohesionless' gap, the jump from, say, Atterberg limits to predicting long term settlements is still a big one and depends heavily on the experi-

The main problems in attempting to obtain samples in glacial materials are often the high cohesionless content preventing recovery, and the presence of the larger granular particles in the more cohesive materials which during driving are displaced into the sample and cause considerable disturbance. The importance of careful handling and preparation of test specimens in the laboratory is emphasised by McKinlay Tomlinson and Anderson (1974), who conclude that while 102mm diameter open drive samples can provide consistent data, a generous sampling programme is essential to provide sufficient numbers of good specimens for testing.

Nevertheless, where the recovery rate is low this may not always be feasible or economical.

For glacial materials, therefore, the case for in-situ testing is a strong one. Plate bearing tests can provide useful data, but for most applications their use is prohibitively costly. Static cone penetrometers have found favour in a number of countries, but the wide variety of

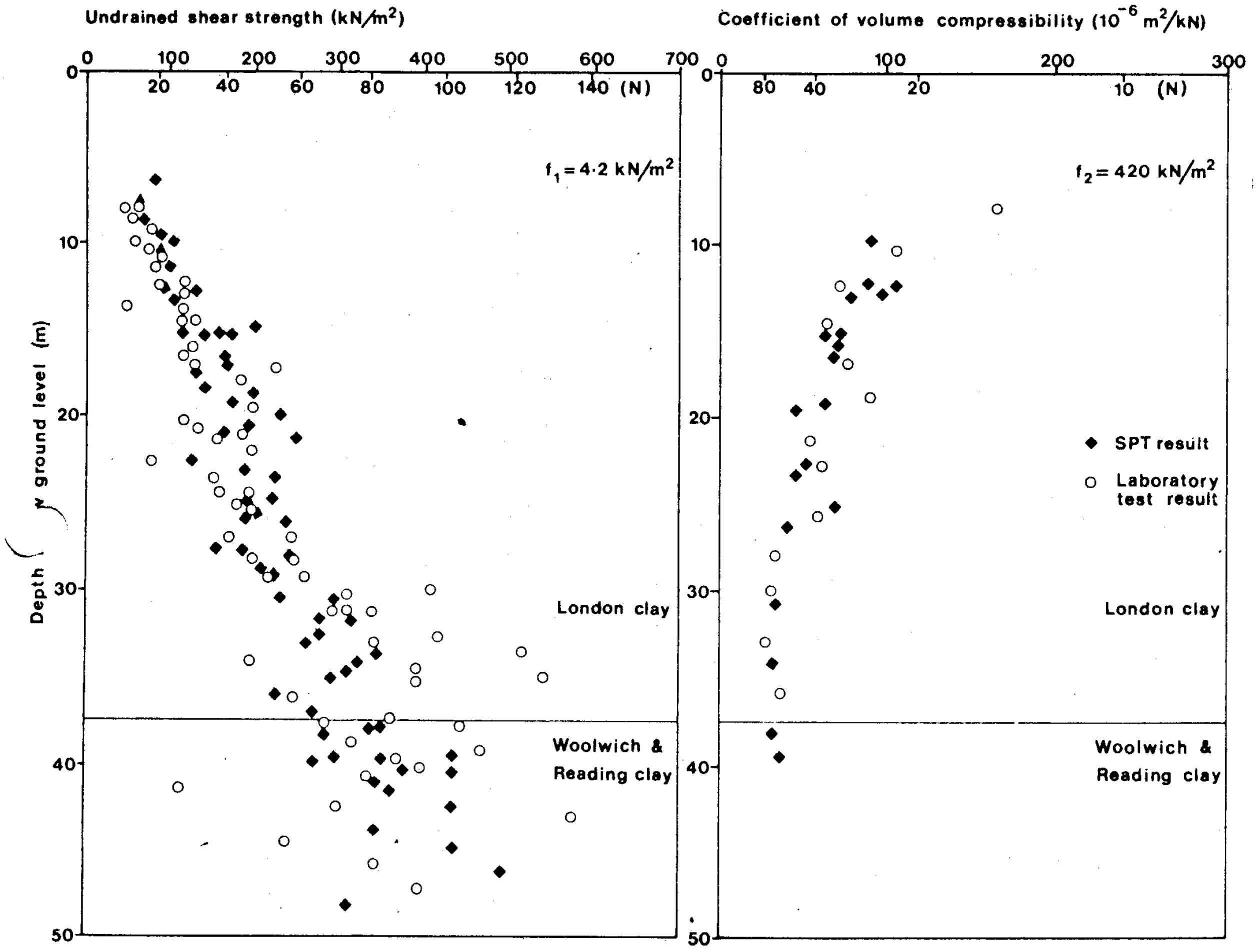

Fig. 1 Comparison of N values with undrained shear strength (c) and coefficient of volume compressibility (m_v) for a typical site on London Clay.

...ises caution (Sanglerat, 1972). Moreover, the strength of many glacial deposits is high and beyond the economic and physical capabilities of many penetrometers.

The Standard Penetration Test on the other hand is cheap, robust and simple to operate. Its application to cohesionless soils is widespread and routine, although the interpretation is still a matter of some debate (Sutherland, 1974). In cohesive soils its application has not been widespread, although the fact that N values are in some measure related to shear strengths is recognised (Terzaghi & Peck, 1948; Sowers, 1954; de Mello, 1971). Recently, Stroud (1974) has shown that for a wide variety of insensitive clays and weak rocks a simple correlation appears to exist between N values and the mass in-situ undrained shear strength (c). The correlation is of the form

$$c = f_1 \times N$$

Down to at least 50m below ground level f_1 appears to be essentially independent of depth least 200mm.

As an example of the quality of the correlation Fig. 1a shows typical data for a site on London clay. The results of the SPTs are superimposed on a plot of undrained shear strength obtained from triaxial tests on 102mm specimens. The horizontal scale of N values has been adjusted to give the best fit with the triaxial results and a value of $f_1 = 4.2$ kN/m² was obtained. The value of f_1 is found to increase with decreasing plasticity index and varies from about 4.0 kN/m² in materials of high plasticity to about 6.0 kN/m² in materials of medium to low plasticity.

As may be expected a similar correlation appears to exist for the coefficient of volume compressibility, of the form

$$m_v = \frac{1}{f_2 N}$$

Fig. 1b shows the values of m_v obtained from oedometer tests, with the N values super-

f_2 of 420 kN/m^2 was obtained for this site. Generally f_2 was found to increase from about 400 kN/m^2 for highly plastic materials to over 600 kN/m^2 for materials of medium to low plasticity.

The objects of this present study are first to investigate in more detail how these correlations apply to glacial materials and then to examine how the SPT can be used to unify the approach to the estimation of settlement.

THE STANDARD PENETRATION TEST

Much has been written about the importance of standardisation in the use of the SPT (Fletcher, 1965; Ireland, Moretto & Vargas, 1970; Serota & Lowther, 1973) and no further comment is necessary here. In the tests described below the standard Raymond splitspoon sampler was used having a 50mm external diameter and a 35mm internal diameter. In each case, the automatic trip monkey developed by Pilcon Engineering was employed to ensure the free fall of the 63.5 kg hammer weight through 760mm.

The borings were advanced by "Shell & Auger" techniques and were generally of 200mm or 150mm diameter. For each SPT the number of blows required to penetrate six successive intervals of 75mm was counted and the last four summed to give the N value. In hard ground, where full penetration was not achieved, the test was stopped at 50 blows and the actual penetration noted. The number of blows for 300mm penetration was then obtained by extrapolation.

Sampling and testing in each borehole was generally at 1m intervals with SPTs alternating where possible with 102mm diameter open

Table 1 Materials Tested and Locations of Sites

Material	Description	Ref. No.	Site Location	Average Liquid Limit %	Average Plasticity Index %	f_1 (kN/m^2)	f_2 (kN/m^2)
Boulder Clay	Stiff to very stiff brown-grey sandy silty CLAY with some gravel.	1	Chester	40	25	5.4	-
	Firm to very stiff dark brown silty CLAY with rounded gravel, cobbles and occasional boulders and lenses of weathered mudstone.	2	Haltwhistle	30	15	5.0	-
	Firm to very stiff red brown sandy silty CLAY with some subrounded gravel and occasional cobbles.	3	Moffat	26	12	5.5	-
	Firm to very stiff red brown and grey very sandy CLAY with rounded gravel of sandstone and dolerite and occasional cobbles.	4	Bathgate	30	15	6.0	-
	Stiff brown silty CLAY with occasional sand lenses and unsorted gravel sized rock fragments.	5	Macclesfield	38	23	4.5	450
	Stiff red brown sandy silty CLAY with a little scattered gravel; frequent thin bands of clayey silty fine and medium sand.	6	Queensferry	34	18	5.0	350
	Very stiff grey very silty CLAY with chalk fragments and occasional gravel and fine sandy partings.	7	Chelmsford	38	20	4.8	600
	Stiff grey and brown silty CLAY.	8	Northampton	33	15	5.5	-
	Stiff to hard red brown sandy CLAY with angular and subrounded gravel and cobbles.	9	Carlyle	31	17	5.0	-
	Firm to stiff grey and brown mottled silty CLAY with a little sand and gravel.	10a	South Shields	37	19	5.0	-
	Firm to stiff grey and brown mottled CLAY with a little sand.	10b	South Shields	67	40	5.0	-
	Stiff brown sandy silty CLAY laminated in parts with gravel size fragments of coal sandstone and quartzite.	11	Manchester	40	20	5.5	500
Laminated Clay	Firm grey brown laminated silty CLAY with partings of fine sand and silt.	10c	South Shields	60	35	5.0	500
	Firm grey brown laminated silty CLAY.	13	Middlesbrough	64	38	4.4	440
	Firm grey poorly laminated silty CLAY with occasional stones.	14	Northampton (brewery)	45	23	5.0	-
	Stiff grey laminated CLAY with silt partings and occasional fine sand in the laminations.	15	Northampton (bridge)	38	22	5.0	-
Sunnybrook Till*	Medium stiff grey CLAY TILL containing a sprinkling of small stones and pebbles.	12	Toronto	28	12	7.0	750

*Crawford and Burn (1962)

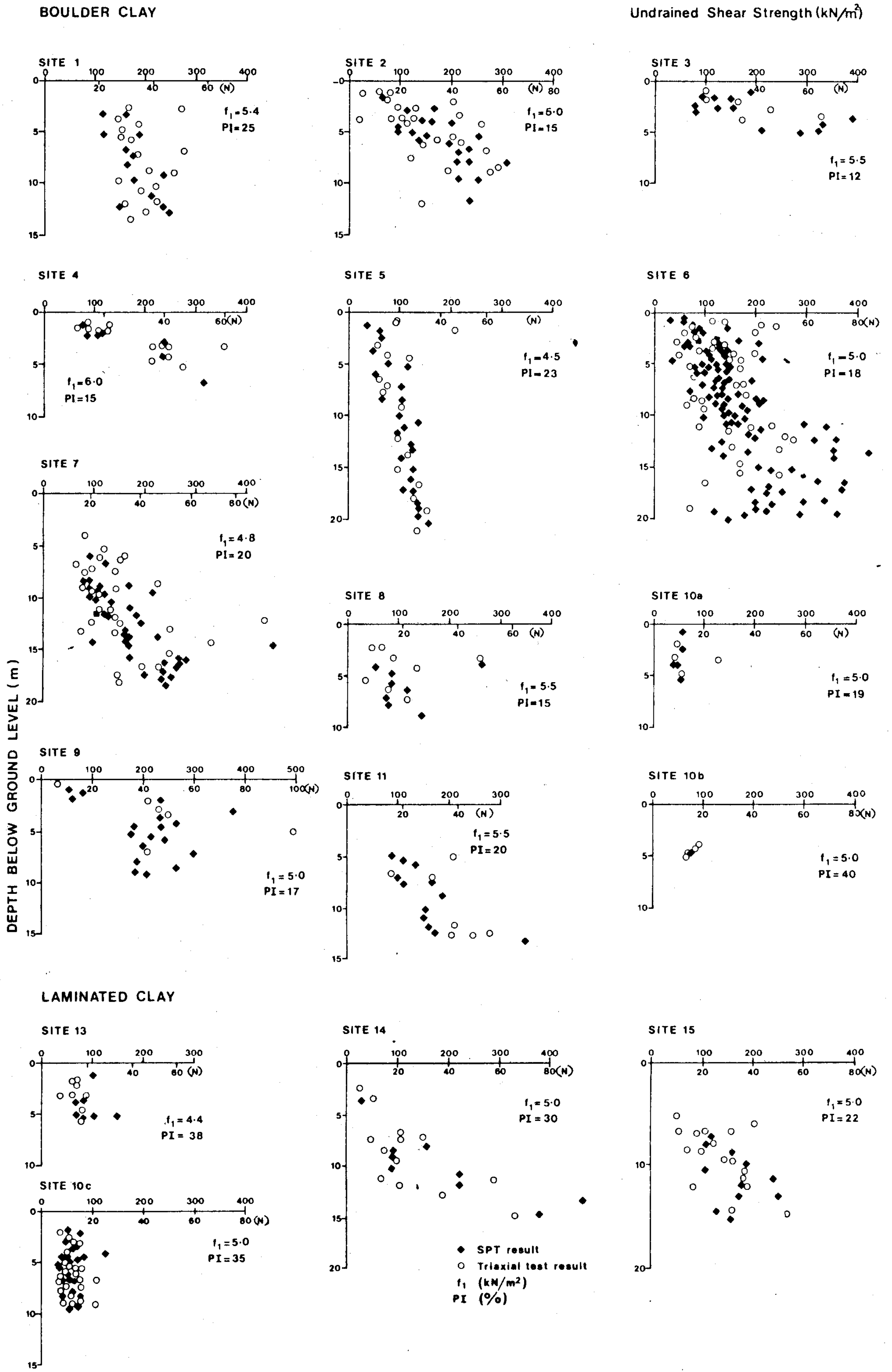

Fig. 2 Comparison of N values with [illegible]

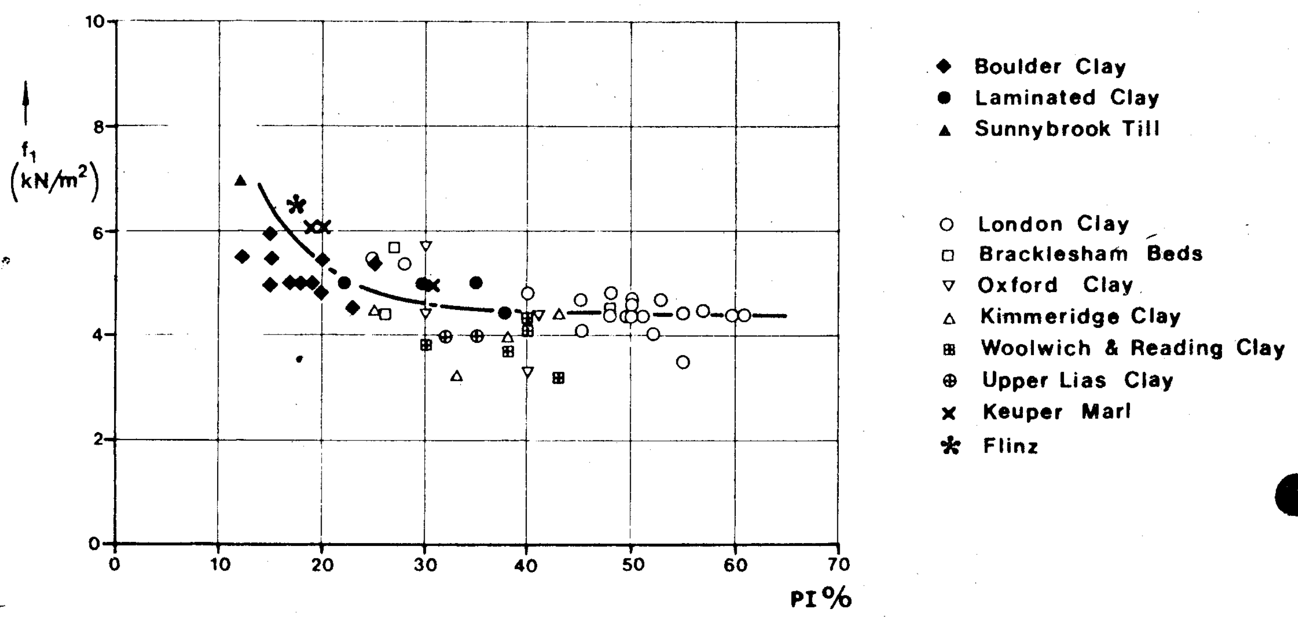

Fig. 3 The variation of $f_1 = c/N$ with Plasticity Index.

drive samples.

N VALUES AND UNDRAINED SHEAR STRENGTH

In Fig. 2 the results of undrained triaxial tests on 102mm diameter samples are presented for twelve sites on 'boulder clay' and four sites on 'glacial laminated clay'. The character and composition of the deposits varies considerably from one site to another reflecting the wide distribution of sites across the country. Brief descriptions of the deposit at each site are given in Table 1.

Superimposed on the plots of Fig. 2 are the N values with the horizontal scale adjusted to give the best fit with the triaxial data in each case. As may be observed agreement is generally good despite the variety of strength/depth profiles. In each case the scatter of N values is not greater than for the shear strength data and in a number of cases is actually less. The value of f_1 varies from 4.4 to 6.0 kN/m^2. Fig. 3 shows the values of f_1 plotted against plasticity index for each of the glacial deposits together with data previously presented (Stroud 1974) for other insensitive clays and weak rocks. An additional point is derived from data given by Crawford and Burn (1962) for Sunnybrook Till, a massive silty till of Wisconsin age underlying much of Toronto. The results for glacial deposits thus confirm for materials of medium plasticity the trend of increasing f_1 with decreasing plasticity index.

N VALUES AND COMPRESSIBILITY

Fig. 4 shows values of the coefficient of volume compressibility m_v estimated from oedometer tests carried out on samples from a number of sites. The values of m_v have been calculated in each case for a pressure increment of 100 kN/m^2 in excess of the effective overburden pressure at the sample depth. Also shown in Fig. 4 are the N values plotted on a reciprocal scale which has been adjusted to give the best fit of N value results with oedometer results. Values of $f_2 = \frac{1}{m_v N}$ vary in the range 440 to 600 kN/m^2 with the exception of site 6 for which a value of 350 kN/m^2 was obtained. In this last case the values of m_v estimated from oedometer tests were considerably higher than the generally stiff to very stiff consistency of the samples would indicate, and it is concluded that the material has been significantly disturbed during sampling. (Applying Schmertmann's construction (Schmertmann, 1953) in an attempt to eliminate some of the effect of disturbance gives a value of f_2 of about 650 kN/m^2.)

In Fig. 5 the values of f_2 have been plotted against plasticity index together with the data for other overconsolidated clays. A trend of increasing f_2 with decreasing plasticity index is indicated. The line drawn in Fig. 5 is the mean line of Fig. 3 replotted on the assumption that $m_v c = \frac{1}{100}$ (Skempton and Henkel, 1957),

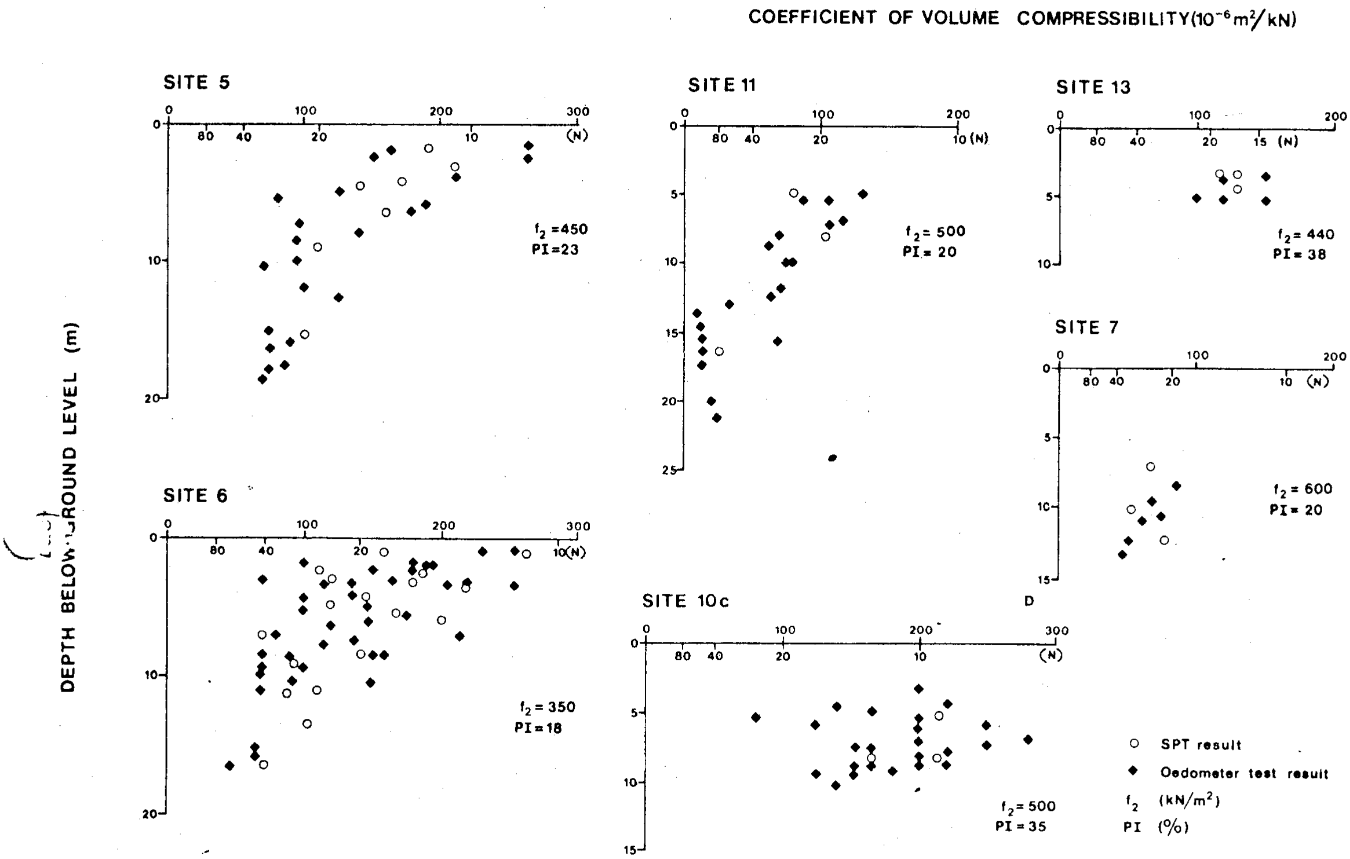

Fig. 4 Comparison of N values with m_V for individual sites.

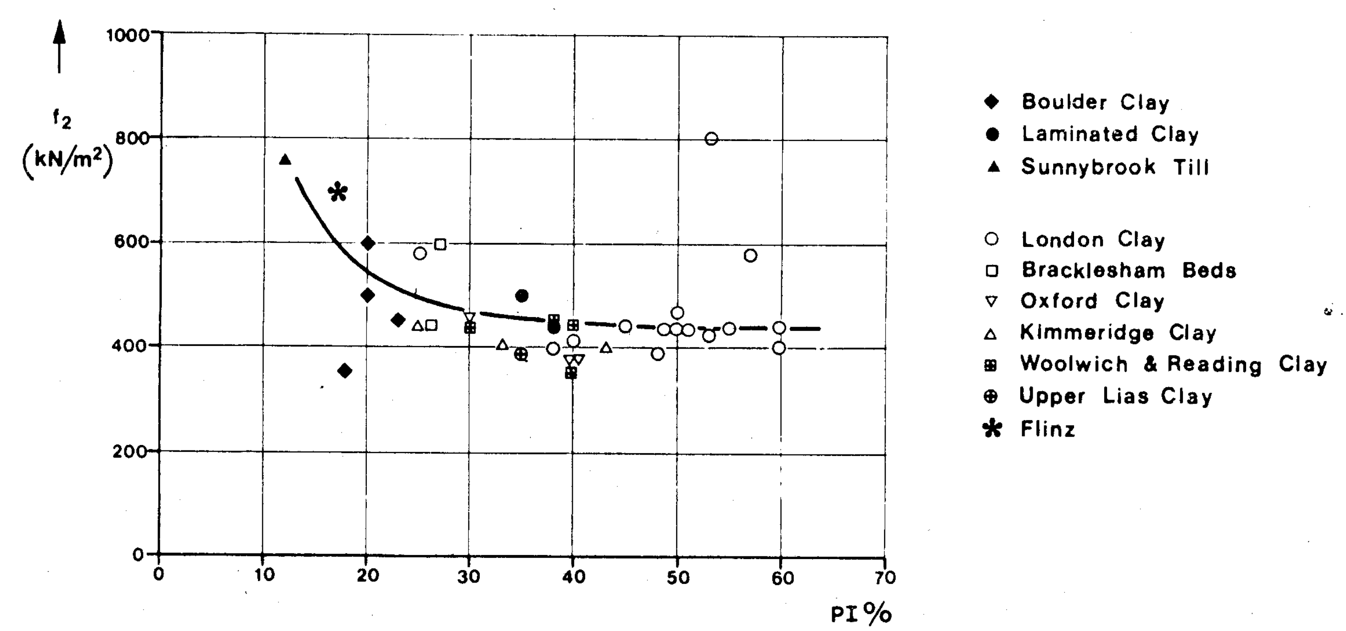

Fig. 5 The variation of $f_2 = 1/m_v N$ with Plasticity Index.

i.e. $\dfrac{1}{m_v N} = 100 \dfrac{c}{N}$

or $f_2 = 100\ f_1$

N VALUES AND SETTLEMENT

Traditionally, estimation of the settlement of structures on glacial materials has been achieved by first considering immediate settlement and then where appropriate adding on a consolidation settlement computed using values of m_v obtained in the laboratory. Such estimates are nearly always excessive due at least in part to inflated values of m_v resulting from sample disturbance.

Recently, however, the importance of the quasi-elastic approach to settlement calculating has been recognised for a wide variety of soils. Elastic moduli have been used for many years to determine the total settlements of structures on sand and rock. Butler (1974) and Burland and ...oth (1974) have now shown that the approach can equally well be applied to overconsolidated clays. In particular, Butler (1974) examined the case histories of 29 large structures on London Clay, Woolwich and Reading Clay and Gault Clay and concluded that final settlements could be adequately predicted by a single computation using elastic analysis, for which Poisson's ratio of 0.1 was assumed and a vertical 'drained' elastic modulus E_v' given by

$$E_v' = 130c.$$

Now $c = f_1 N$, therefore

$$\frac{E_v'}{N} = 130\ f_1$$

i.e. the ratio of drained elastic modulus to the N value is constant for a given overconsolidated clay.

At the lower end of the plasticity scale, Parry (1971) has considered the case records of some 24 structures founded on free draining sand and concludes that for these materials settlement prediction can usefully be made assuming a constant relationship between compressibility and N. Assuming a Poisson's ratio of 0.3 and a rigidity factor of 0.8 the relationship becomes

$$\frac{E_v'}{N} = 3600\ \text{kN/m}^2.$$

The possibility of unifying settlement prediction for overconsolidated materials by using an elastic approach and assuming E_v'/N is constant for a material of given plasticity immediately suggests itself and is of particular importance for glacial materials.

In Fig. 6 and Table 2 data are presented for a wide variety of overconsolidated materials of different plasticity. With the exception of the plate tests on Devonian Marl all data relate to the measured settlement of large scale structures. Values of E_v'/N have been obtained using conventional elastic settlement analysis based on factors derived by Steinbrenner (1934). Where records relate to deposits consisting of more than one definable layer of different geological age and consistency a superposition approach has been employed as described by Butler (1974). In cases where N increases with depth, the value of N at a depth of about 0.7 B has been taken to be representative where B is the width of the foundation (Parry, 1971). Poisson's ratio has been taken as 0.1 to 0.2 for cohesive materials and 0.3 for the cohesionless materials.

Of particular interest are the data relating to six structures on various types of glacial deposit. In general they confirm the trend of increasing E_v'/N with decreasing plasticity indicated by the data for other overconsolidated materials. The rate of increase is most pronounced in passing from very silty sandy clays and clayey silty sands to free draining sands.

Degree of loading

The striking exception to the general trend is given by the CN Tower founded on Edmonton Till. De Jong and Harris (1971) compare the settlement data for this structure with that of the Avord Arms, a similar multi-storey office block also founded on Edmonton Till and conclude that the elastic modulus at stresses above overburden pressure decreases with increasing footing pressure. Elastic moduli of 490 MN/m^2 for the CN Tower and 240 MN/m^2 for the Avord Arms were associated with net bearing pressures of the order of 330 kN/m^2 and 950 kN/m^2.

Confirmation of this effect is obtained by considering settlement data given by Levy and Morton (1974) for the Arts and Commerce Building, University of Birmingham, founded on glacial gravel and sand. Values of E_v' have been estimated as the net bearing pressure q increased with construction, and the values are summarised in Table 3.

In order to make a more direct comparison of the settlement behaviour of structures on different materials, the degree of loading may be assessed by considering the actual net bearing pressure as a proportion of the ultimate bearing capacity q_{ul}, i.e. by considering the ratio q/q_{ul}. In the discussion that follows the ultimate bearing capacity q_{ul} has been taken as

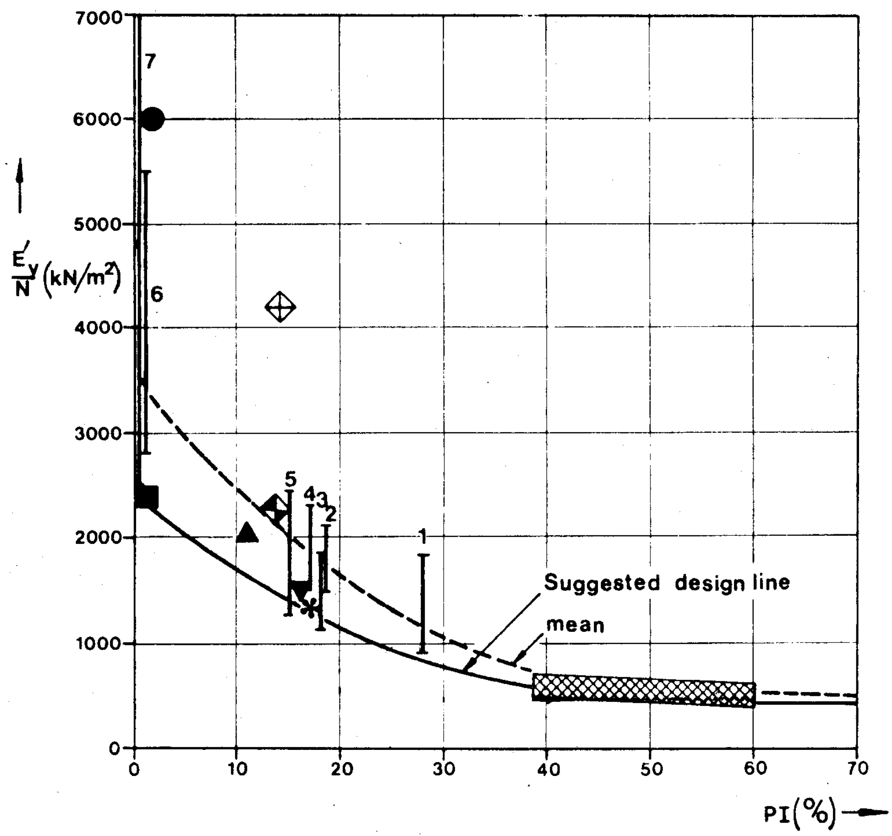

Fig. 6 The variation of E_V'/N with Plasticity Index for overconsolidated materials where E_V' is the vertical 'drained' elastic modulus.

Table 2 Case Histories

Symbol	Material	Type of Structure	B (m)	D (m)	q (kN/m^2)	N	t (m)	ϕ' c'(kN/m^2)	Reference	Remarks
■	Glacial gravel & sand	Arts & Commerce Bldg (12 storeys) Univ. of Birmingham	4	5	400	37	7	37.5^o	Levy & Morton (1974)	
●	Glacial sand	Ashby Institute Bldg (13 storeys) Queens Univ. Belfast	18	3	131	21 *50	10 *20	33^o	Stuart & Graham (1974)	
▲	Sunnybrook till	Mt. Sinai Hospital (11 storeys) Toronto	16	5	85	12 32	4 9	*30^o	Crawford & Burn (1962)	
▼	Moraine clay	Two Sugar Silos	20	5	250	35 100	7 13	*30^o *30	Vefling (1974)	assuming PI = 16% and f_1 = 5.0
◆	Edmonton till	Avord Arms (27 storeys)	5	5	950	106	23	40^o	De Jong & Harris (1971)	
⊕	Edmonton till	C N Tower Bldg (26 storeys)	6	7	330	116	21	40^o	De Jong & Harris (1971)	
★	Flinz	Nuclear reactor	31	7.5	375	50	>73	30^o 35	Breth & Chambosse (1974)	
▩	London clay	27 large structures			see	reference		20^o 15	Butler (1974)	
1	Frankfurt clay	6 large structures	14-43	6-13	130-270	32	>85	20^o 20	Breth & Amann (1974)	assuming f_1 = 5.0
2	Keuper Marl	2 large structures			see	reference			Lord & Nash (1974)	
3	Keuper Marl	Multi storey car park	3-12	2	120-200	50	>40		Ove Arup & Partners (unpublished)	
4	Keuper Marl	3 road bridges	3-6	1-6	160-260	*75	>10		Chandler, Birch & Davis (1968)	
5	Devonian Marl	Plate tests	0.3-0.6		see	reference			Hobbs & Dixon (1974)	
6	Free draining sands	24 structures			see	reference			Parry (1971)	
7	Preloaded sands	8 footings			see	reference			D'Appolonia (1971)	

B = foundation width
D = foundation depth
q = net bearing pressure

t = thickness of compressible layer below foundations
* = value assumed

TABLE 3 The decrease in E_V' with increase in q Arts & Commerce Building, University of Birmingham.

q (kN/m²)	100	200	300	400
E_V' (MN/m²)	137	110	92	85

that for local shear failure beneath the foundation, estimated in the manner described by Lambe and Whitman (1969) using bearing capacity factors derived by Peck, Hansen and Thornburn (1953). In all cases the ultimate bearing capacity is that appropriate to the fully drained case. Where it has been appropriate to consider a cohesion intercept c', this value has been factored by $\frac{2}{3}$ (Terzaghi, 1943; Lambe & Whitman, 1969).

Fig. 7 shows the data for the two structures on Edmonton Till and the data from Table 3 for the glacial gravel and sand at Birmingham University replotted in the form of E_V'/N against q/q_{ul}. Despite the widely differing bearing pressures and material types a similar trend of behaviour is indicated.

Also plotted in Fig. 7 are points for other structures previously considered in Fig. 6. A striking pattern of decreasing E_V'/N with increasing q/q_{ul} is obtained, which includes data for both free draining and cohesive materials.

Free draining materials and materials of low plasticity appear to be associated with very high factors of safety against shear failure with values of q/q_{ul} generally well below 0.15. For this portion of the curve the rate of change of E_V'/N with loading is most severe. This sensitivity of E_V' to the degree of loading may explain at least in part the difficulties well known to be associated with settlement prediction in these materials.

Fig. 7 demonstrates the importance of taking into consideration the level of stress when designing foundations to a settlement criterion. The methods proposed by D'Appolonia (1970) and Parry (1971) for sands, for example, may give satisfactory predictions of settlement for bearing stresses low in relation to the ultimate capacity. For values of q/q_{ul} greater than about 0.10, however, Fig. 7 indicates their methods may overestimate the elastic modulus and therefore underestimate the settlement.

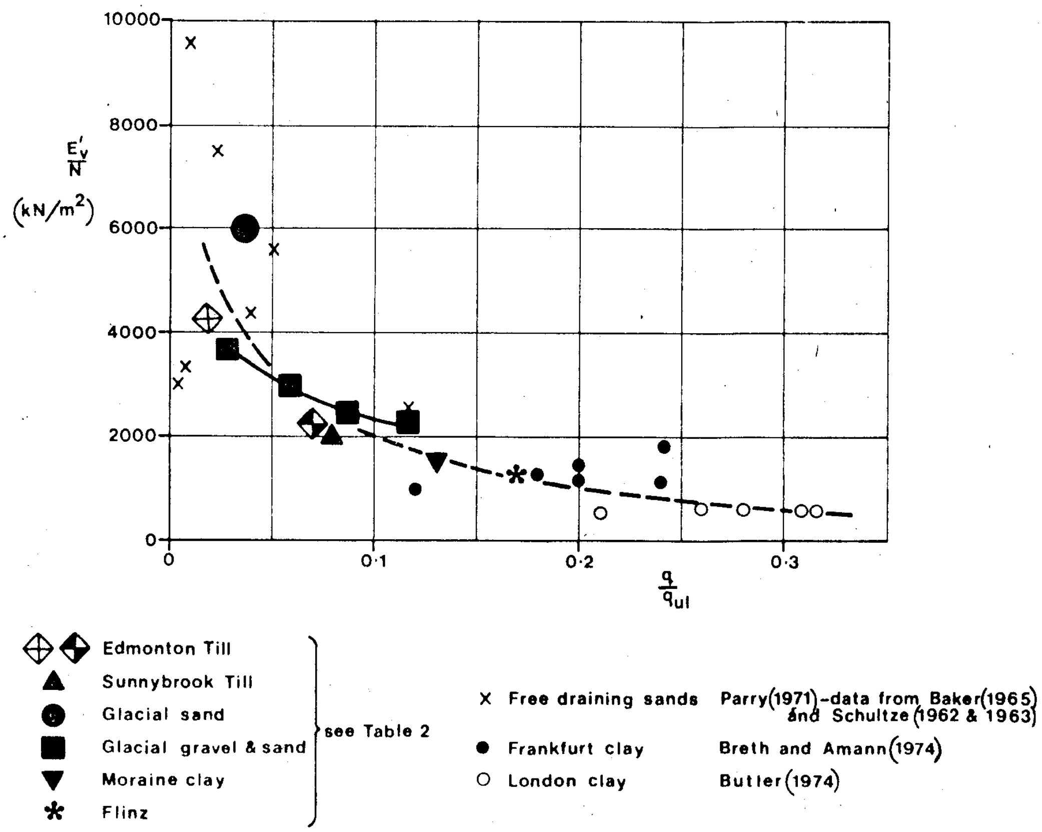

Fig. 7 The variation of E_V'/N with degree of loading q/q_{ul}.

For materials of higher plasticity Fig. 7 indicates that the factors of safety commonly used against shear failure become progressively lower. Whether there is a significant variation of E_v'/N with q/q_{ul} for any one material is not clear. For the level of loading generally used for shallow foundations on London Clay for example the value of E_v'/N appears to be approximately constant. Certainly at low values of q/q_{ul} there is no evidence to suggest that the value of E_v'/N is anywhere near as high as for cohesionless materials.

The band of data in Fig. 7, therefore, should not be interpreted as indicating that a unique curve of E_v'/N against q/q_{ul} exists for all materials. Rather it may be assumed to represent an upper bound on the value of Ev'/N for particular values of q/q_{ul}.

...gn procedure

A unified design procedure for shallow foundations is proposed as follows involving first the settlement calculation and then a check on the degree of loading.

A suggested design line is shown on Fig. 6 that forms a lower bound to most of the data and includes the 1.5 'factor of safety' for free draining sands recommended by Parry (1971). Having determined E_v' from Fig. 6 according to the appropriate values of PI and N, the shape factors and depth corrections should be applied in the usual way together with values of Poisson's ratio appropriate to a drained situation, which in current practice are assumed to range from a...t 0.1 - 0.2 for clays to about 0.2 - 0.3 for s...s. Once a design has been formulated Fig. 7 should be used to check that the appropriate value of q/q_{ul} has not been exceeded.

Comparison of prediction

It is of interest to compare settlement predictions for glacial materials made by the conventional Skempton and Bjerrum method with those suggested by the quasi-elastic approach. Skempton and Bjerrum (1957) suggested that the total settlement of a foundation ρ_t could be conveniently estimated by considering separately the immediate 'undrained' settlement ρ_i and the consolidation settlement $\rho_c = \mu \rho_{oed}$ where ρ_{oed} is the settlement estimated from oedometer tests and μ is a constant depending on footing shape, depth of compressible layer and pore pressure coefficient A. Thus

$$\rho_t = \rho_i + \mu \rho_{oed}$$

Consider a circular flexible footing radius R resting on a deep compressible layer loaded with ... q ... R ... mediate settlement at the centre of the foundation is given by

$$\rho_i = 2(1-\nu^2)\frac{qR}{E_u} \text{ where } \nu = 0.5$$

i.e. $\rho_i = 1.5\frac{qR}{E_u}$.

Now for overconsolidated clay evidence suggests that a reasonable estimate of the undrained elastic modulus may be made by taking $E_u = 220$ c (Butler, 1974).

But $\frac{1}{m_v c} = 100$ (Figs. 3 and 5)

Therefore $E_u = \frac{2.2}{m_v}$

Therefore $\rho_i = 0.68\ qRm_v$

Also it may be shown that $\rho_{oed} = 2\ qRm_v$

Therefore $\rho_t = (0.68 + 2\mu)\ qRm_v$

By the quasi-elastic method, the total settlement ρ is given by

$$\rho = \frac{2(1-\nu'^2)\,qR}{E_v'}$$

Therefore $\frac{\rho_t}{\rho} = \frac{(0.34 + \mu)\ m_v E_v'}{(1-\nu'^2)}$

This expression has been evaluated in Table 4 for two materials: London Clay, and a typical glacial deposit for which the plasticity index is 15. For this latter material a value of $\mu = 0.25$ has been assumed corresponding to A = 0, the low end of the range for heavily overconsolidated sandy clays (Skempton and Bjerrum, 1957). Values of $m_v E_v'$ have been obtained from Fig. 5 and from Fig. 6 using the suggested design line.

It is apparent from Table 4 that while the Skempton and Bjerrum approach is in reasonable agreement for clays of high plasticity, for glacial materials it is likely to lead to thirty percent higher estimates of settlement than the elastic approach which is itself based on a conservative interpretation of the field data.

SUMMARY AND CONCLUSIONS

1. The simple correlation $c = f_1 \times N$ linking undrained shear strength and N values for insensitive clays and weak rocks is shown to apply to the more cohesive glacial materials. The value of f_1 increases with decreasing plasticity index and for the range of glacial materials tested varies from about 4.5 kN/m^2 to about 6.0 kN/m^2.

TABLE 4

	PI(%)	ν'	μ	$\frac{E_v'}{N}$ (kN/m2)	$\frac{1}{m_v N}$ (kN/m2)	$m_v E_v'$	$\frac{\rho_t}{\rho}$
London Clay	50	0.1	0.5	550	440	~~1.3~~ $\frac{550}{440} = 1.25$	1.1
Typical Glacial Material	15	0.2	0.25	1500	700	2.1	1.3

2. A similar correlation of the form $m_v = 1/f_2N$ is also confirmed for the coefficient of volume compressibility. The value of f_2 was found to increase from about 450 kN/m^2 for materials of medium plasticity to over 600 kN/m^2 for materials with a PI less than about 20.

3. The application of elastic theory to the estimation of settlement of structures is examined. It is demonstrated that the use of the Standard Penetration Test can rationalise and unify the approach to settlement prediction for a wide variety of overconsolidated materials. The ratio of the vertical drained modulus of elasticity E_v' to the N value is estimated for a large number of structures on a wide variety of soil types, and a simple design relationship is proposed linking E_v'/N to plasticity index.

4. A significant decrease of E_v'/N with increasing net bearing stress q is demonstrated for cohesionless materials and materials of low plasticity. A relationship between E_v'/N and q/q_{ul} is indicated where q_{ul} is the ultimate bearing capacity estimated for local shear failure beneath the foundation. The relationship indicates an upper bound to the degree of loading admissible for a given value of E_v'/N.

5. The usefulness of SPTs in correlating data for a wide variety of soil types is thus demonstrated and is of particular importance when considering glacial materials. For the more cohesive glacial deposits the traditional approach to design involving sampling in the field and testing in the laboratory may be possible but generally leads to excessive estimates of settlement. As the proportion of granular constituents increases so also do the problems of sample disturbance. A large number of samples need to be taken to obtain enough intact specimens for testing. Often it is not possible to recover samples at all. In all these cases the Standard Penetration Test is capable of providing valuable data relating to the in-situ properties.

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