

Figure 1 - Typical Cone Test Graph with Estimated Soil Type

### INTERPRETATION OF STATIC CONE PENETRATION TESTS BY USE OF THE FRICTION RATIO

Extensive research has indicated that the ratio of local side friction to cone end resistance ("friction ratio") assists in identifying the soil type. The results of various research studies (Meigh 1987) have been produced in graphical form and a modified version for British soils is presented in Figure 2, where the soil type is given as a function of cone end resistance and friction ratio.

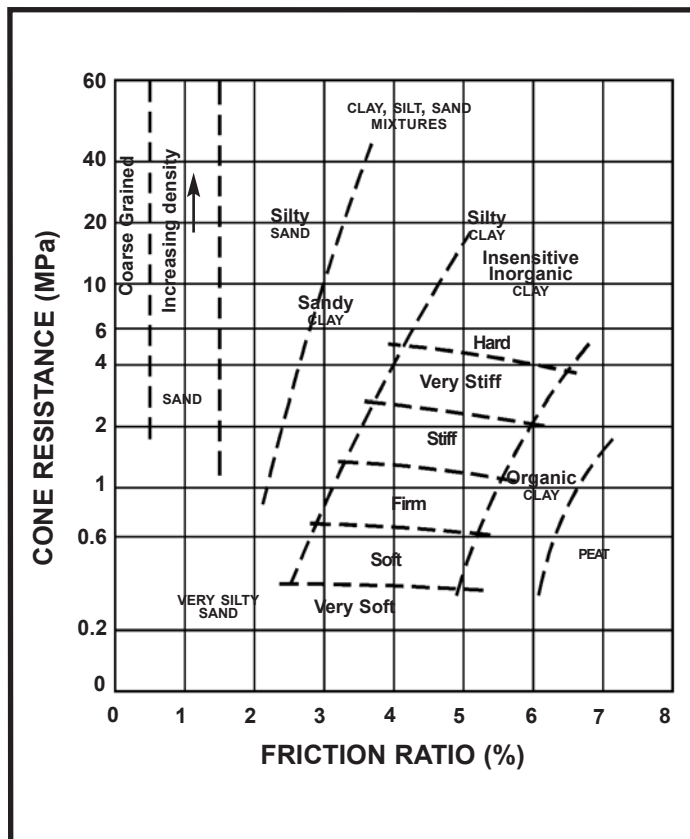


Figure 2 - Guide for Estimating Soil Type

### THE USE OF PORE PRESSURE READINGS

The additional measurement of pore pressure with the piezocone assists in identifying the soil types.

Variations in pore pressure reflect changes in stratification that cannot always be determined with  $q_c$  or  $f_s$ . For instance, in Figure 1, changes in pore pressure response in the clay layers indicate the permeable seams or lenses that can greatly influence the drainage characteristics of the stratum.

Furthermore, the excess pore pressure ratio could give an indication of the stress history of the soil.

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## GRANULAR SOILS

### Correlation of $q_c$ and $D_r$ and $\phi$

Table 1, below provides a guide for the relation between cone end resistance and the angle of internal friction in fine sand.

It should be noted that the guide table applies to unaged, uncemented sands up to about 10 to 15 metres depth.

A more recent correlation between  $q_c$  and  $D_r$  is presented in Figure 3, which takes account of the effect of the effective stress (Lunne and Christophersen, 1983).

## COHESIVE SOILS

Bearing capacity theory indicates that, in simple terms for  $\phi = 0$ , the cone resistance  $q_c$  should be related to overburden pressure  $P_o$  and the undrained shear strength  $S_u$  in the following way (Sanglerat et al 1972):

$$q_c = S_u N_k + P_o \quad (1)$$

where  $N_k$  is a bearing capacity factor or "cone factor". However, in some circumstances, Fugro use a modified expression in which the effect of overburden pressure is included in the cone factor.

$$q_c = S_u N'_k + P_o \quad (2)$$

To use either equation, the cone factor must be determined empirically, or be known from correlations based on previous investigations in the same clay. The value of the cone factor depends on the stress-strain properties of the clay and is frequently found to lie in the range 15 to 20, although it should be noted that values outside this range have been observed.

Table 1 - Correlation of Cone resistance and angle of internal friction

Cone Resistance ( $q_c$ ) (MPa)	Compaction of Fine Sand	SPT (N)	Relative Density $D_r$ (%)	Angle of Internal Friction (degrees)
<2	very loose	<4	<20	<30
2-4	loose	4-10	20-40	30-35
4-12	medium dense	10-30	40-60	35-40
12-20	dense	30-50	60-80	40-45
>20	very dense	>50	80-100	>45

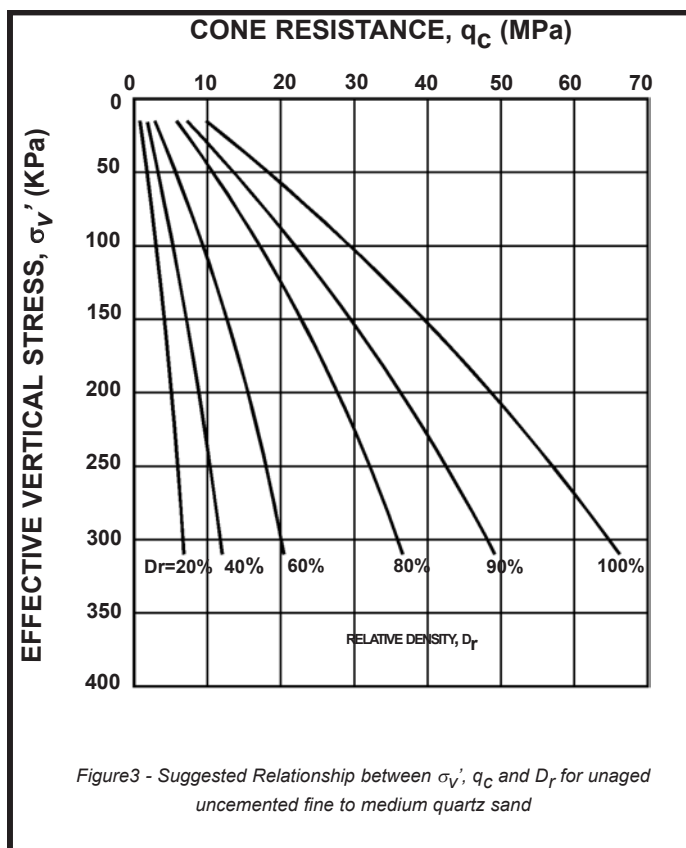


Figure 3 - Suggested Relationship between  $\sigma_v'$ ,  $q_c$  and  $D_r$  for unaged uncemented fine to medium quartz sand

## CORRELATION OF $q_c$ WITH SPT-'N' VALUE

Many comparative studies have been carried out by various researchers and it is known that the particle size distribution has an important influence on the correlation.

It is generally accepted that the conversion factors given below may be used. (ESOPT1, 1982 and Meigh, 1987). In the following  $q_c$  is in MPa.

Soil Description	$q_c / N$
Silts, sandy silts and slightly cohesive silt-sand mixtures	0.2 - 0.3
Fine to medium sands and slightly silty sands sand mixtures	0.4 - 0.5
Coarse sands and sands with some gravel	0.6 - 1.0
Sandy gravels and gravel	1.1 - 1.8

The SPT N-value can be influenced by many factors such as the quality of the equipment, the performance of the test, the depth of test and the groundwater (CIRIA News, No4, 1986; BRE Report 1979). "For foundation design purposes, direct use of the more repeatable CPT results is preferred to conversion to SPT.

## Fugro Engineering Services Limited

Fugro House  
Hithercroft Road  
Wallingford  
Oxfordshire OX10 9RB  
Tel: +44 870 4021 400  
Fax: +44 870 4021 499  
Email: wallingford@fes.co.uk

Armstrong House, Unit 43  
Number One Industrial Estate  
Medomsley Road, Consett  
Co. Durham DH8 6TW  
Tel: +44 1207 581120  
Fax: +44 1207 581609  
Email: consett@fes.co.uk