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ESTIMATION OF CARRYING CAPACITY OF LARGE DIAMETER BORED PILES BEARING ON HARD GNEISSIC ROCKS

by

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1.0 Introduction

Piles and other deep foundations carry part of their load in end bearing and the rest of the load in skin friction. In the case of small diameter bored piles taken down to rock, the load carried in end bearing was considered to be much more than the load carried in skin friction and hence such piles were referred to as end bearing piles. These piles have been used in this country since the 1960's.

Today, with the construction of several tower type structures, there has been a shift towards the use of large diameter bored piles. (Large diameter piles are arbitrarily defined by BS 8004 as piles having a diameter of more than 600 mm.) It is found that for the economical design of such piles, it is necessary to take into consideration the load carrying capacity both by skin friction and by end bearing.

Many of the large diameter piles that have been constructed in this country have been founded on hard gneissic rocks. Unfortunately there is very little published information about the end bearing capacity of such piles on rock as well as about their skin friction. Therefore, there is a need to provide Design Engineers with guidelines making use of the data already available. This paper has been prepared as a step in this direction.

It should be noted that although the methods of analysis outlined in this paper can also be applied to small diameter piles, the latter are essentially end bearing piles for which the contribution from skin friction can be ignored.

2.0 Bearing Capacity of a Rock Mass

2.1 General

When discussing the allowable bearing pressure of hard igneous and gneissic rocks in sound condition, the British Code of Practice on Foundations, i.e. BS 8004, states that 'only in very rare cases is the full capacity of the rock likely to be called upon, and the possibility of foundation failure is very remote'. However, the code also goes on to state that the carrying capacity of a rock mass could be considerably reduced as a result of discontinuities within the rock mass.

In a similar manner, Tomlinson (1987) states that where piles are end bearing on a strong intact rock, the concept of a safety factor against ultimate failure does not apply, since it is likely that the pile itself will fail as a structural unit before shearing failure of the rock beneath the toe occurs.

However, in today's context, such assertions are no longer valid with better grades of concrete now in the market. The analysis of results of pile load tests have shown that there are many instances in which the failure of the rock mass beneath the pile toe has occurred. Therefore, it is necessary to study more closely the factors affecting the strength of a rock mass.

Whilst the end bearing capacity of a pile founded on rock is related to the crushing strength of rock cores, other factors which affect the bearing capacity are

(i) the quality of the rock mass; and
(ii) the inclination of the rock surface at the pile location.

Quality of the rock mass

The quality of a rock mass is given by the Rock Quality Designation (RQD) which can be usually obtained during a site investigation program.

A classification of the quality of the rock mass based on RQD values has been published in the Soil Mechanics Design Manual of the US Dept. of Navy and this is indicated in Table 1.

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Table 1

RQD (%)	Rock Mass Quality
90 - 100	Excellent
75 - 90	Good
50 - 75	Fair
25 - 50	Poor
0 - 25	Very Poor

For design purposes, engineers would like the quantification of the above; and as such two parameters which have been used are

(i) spacing of discontinuities in a rock mass;

(ii) Mass Factor (j) which is related to the discontinuity spacing in the rock mass.

A relationship between these quantities and the RQD as given by Tomlinson (1987) is shown in Table 2.

Table 2

RQD (%)	Fracture Frequency per metre	Mass Factor (j)
0 - 25	> 15	0.2
25 - 50	15 - 8	0.2
50 - 75	8 - 5	0.2 - 0.5
75 - 90	5 - 1	0.5 - 0.8
90 -100	< 1	0.8 - 1.0

2.2 Inclination of rock surface

The inclination of the rock surface at the pile location is another factor which causes a reduction in the allowable bearing capacity. Whitaker (1970) states that, if the rock slope exceeds about 45^0, then seating of the piles is difficult.

Tennekoon (1994) reports of a case study where a pile was able to carry only one half of its design load because of a pile slipping along the rock as a result of being founded on a rock slope which, at that location was inclined at an angle of 59^0 to the horizontal.

In such cases, preference should be given to socketing the pile well into the rock mass rather than adjusting the design value for allowable bearing capacity.

2.3 Bearing capacity based on the strength of rock cores

BS 8004 gives a set of charts for the determination of the allowable bearing capacity of rock based on the crushing strength of rock cores and the spacing of discontinuities in the rock mass. The chart corresponding to igneous and metamorphic rocks is reproduced as Fig. 1 with the additional parameter of RQD also included using the results of Table 2.

Several crushing tests carried out on rock cores at the University of Moratuwa have shown that Sri Lankan gneisses have crushing strengths in the range of 25-90 N/mm2 with an average crushing strength of about 50 N/mm2. Therefore, it would appear that in such rocks, with RQD's of over 75%, an allowable bearing capacity of 10 N/mm2 is possible.

One of the major problems associated with this method is that in the case histories reported in this paper, the rocks consist of biotite gneisses from which it is difficult to obtain good quality rock cores because

(a) coring has been done with a double tube core barrel (rather than a triple tube core barrel); and

(b) the size of rock cores obtained are usually small, being of the order of about 50 mm diameter.

In such circumstances, the best cores can be obtained using the triple-tube core barrels. But unfortunately, most drilling contractors have only the double tube core barrels and, when coring with such equipment, the biotite gneissic rock has to be cored to a considerable depth before good core recoveries with high RQD values can be obtained.

Subsequently, when piling, the piles do not need to be taken down to such great depths because

(i) of the difficulties, cost increases, and time delays involved in drilling in hard rocks;

(ii) the high allowable bearing stresses associated with the good quality gneisses may not be required.

Therefore, information is required on the bearing capacity of the rock mass when the piles are founded at a higher level where the bearing stratum is still rock, but the RQD's may be less than 75%.

2.4 Bearing capacity based on SPT values

One of the methods which could be made use of to determine the bearing capacity of rock, when rock cores cannot be obtained at founding level, is to use empirical methods correlating the ultimate bearing

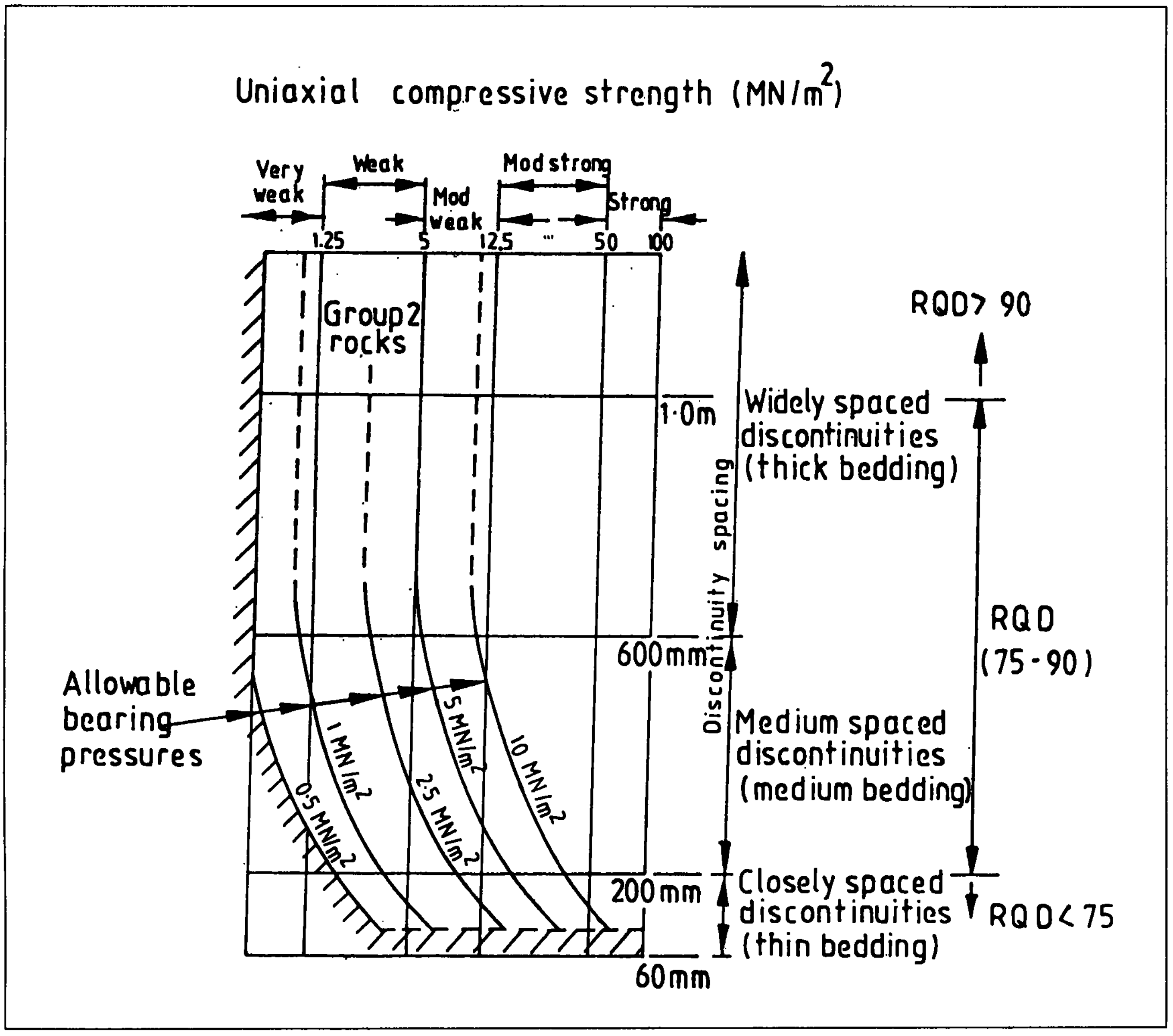

Fig: 1 Allowable bearing pressure for igneous and metamorphic rock [Ref. BS 8004 (1986)]

capacity (qult) to the SPT No. (N). This is a practice followed in certain countries as indicated below.

(i) Hobbs and Healy (1979) report that in soft rocks such as chalk found in UK,

qult = $0.2 \times N$ N/mm^2 Eq. (1)

(ii) The Standards adopted by the Building Control Division, PWD of Singapore is

qult = $40 \times N$ kN/m^2 with $40 \times N \ngtr 11{,}000$ kN/m^2. Eq.(2)

i.e. qult = $0.04 \times N$ N/mm^2 with $0.04 \times N \ngtr 11$ N/mm^2. Eq.(3)

A factor of safety of 3 is then applied to obtain the allowable carrying capacity in end bearing.

It should be noted that, if the SPT test is to be carried out in weathered rock, the SPT tube should be replaced with the SPT cone.

It is recommended that, in the first instance, the Standards adopted by the PWD of Singapore be used, with a limitation of an ultimate bearing capacity of 10 N/mm^2 when N > 250. The corresponding maximum allowable bearing capacity would then be 3.3 N/mm^2.

2.5 Bearing capacity based on pile load tests

The direct method of determining the bearing capacity of a rock mass would be by carrying out a pile load test in which

(a) the pile is loaded almost upto failure; and
(b) the load at the pile toe is measured.

The first of these conditions require that Preliminary Pile Load Tests be done. Unfortunately, this practice is still not adopted in this country.

The second of these conditions requires a load cell to be attached to the base of the pile, but this, too, is not practised.

However, the practice adopted in this country is to carry out Proof Load Tests on Working Piles to check the adequacy of the pile to carry a design working load. Measurements are made of the load applied at the pile head and the displacements at the pile head. Tennekoon (1995) has proposed a method of analysis by which the loads and settlements at the base may be estimated from the results of a Pile Load Test. This method has been applied to 8 case histories in which the piles have not failed, i.e. they have conformed to the specifications set for the adequacy of the pile to carry working load. These results are shown in Table 3, and they are again shown as Fig.2, where the estimated bearing pressure at the toe is plotted against the compression of rock at the toe.

These results show that allowable bearing capacities in the range of 3.0 - 6.5 N/mm² can be easily applied to these rocks and that, under these pressures, the compression at the toe would not be more than 6mm. (No attempt is made to extend these results to other types of hard rock in the absence of information on such rocks.)

3.0 Carrying Capacity in Skin Friction

3.1 General

The uncertainties in the methods of predicting allowable or ultimate loads on piles are reflected in the very little information that is available to designers in the various codes of practice which cover piling. This is especially true in the case of skin friction where the method of pile installation or construction could have a substantial influence on the magnitude of the skin friction load.

The theoretical basis for the computation of skin friction load is by the use of the ultimate skin friction coefficient (f_u) which is written as

$$f_u = c_a + k_s \ \sigma_v \tan \delta \qquad \text{Eq. (4)}$$

where

c_a = adhesion between the soil and pile and is related to the cohesion of the soil.

δ = angle of friction between pile and soil and is related to the friction angle of the soil;

k_s = coefficient of lateral earth pressure on shaft and is related to both k_s and the method of pile installation or construction;

Table 3

Site	Max.(W/A) N/mm²		Settlement (mm)		
	$(W_{Total}/A)^*$	$(W_b/A)^+$	Comp. of pile⁺	Comp. of rock⁺	Total
Hotel Project, Kandy	6.50	5.07	1.24	3.71	4.95
Vauxhall Towers, Colombo 02	8.14	V.Small	-	V.Small	4.45
Bloemendhal Flats, Colombo 01 (Pile A1-1b)	6.85	6.85	1.97	2.34	4.31
Bloemendhal Flats, Colombo 01 (Pile A4-2e)	6.85	4.11	3.72	2.88	6.60
Nawam Mawatha Towers, Colombo 02	7.14	3.97	3.77	1.82	5.59
Galle Road Towers, Colombo 03	10.61	V.Small	-	V.Small	6.85
Towers Flats, Rajagiriya	7.15	3.07	2.74	4.61	7.35
Bank Building, Colombo 03	7.55	3.52	4.82	1.42	6.24

* Loaded upto 1.5 x Working Load
\+ Estimated

Results of Analysis of (Load-Settlement) Data

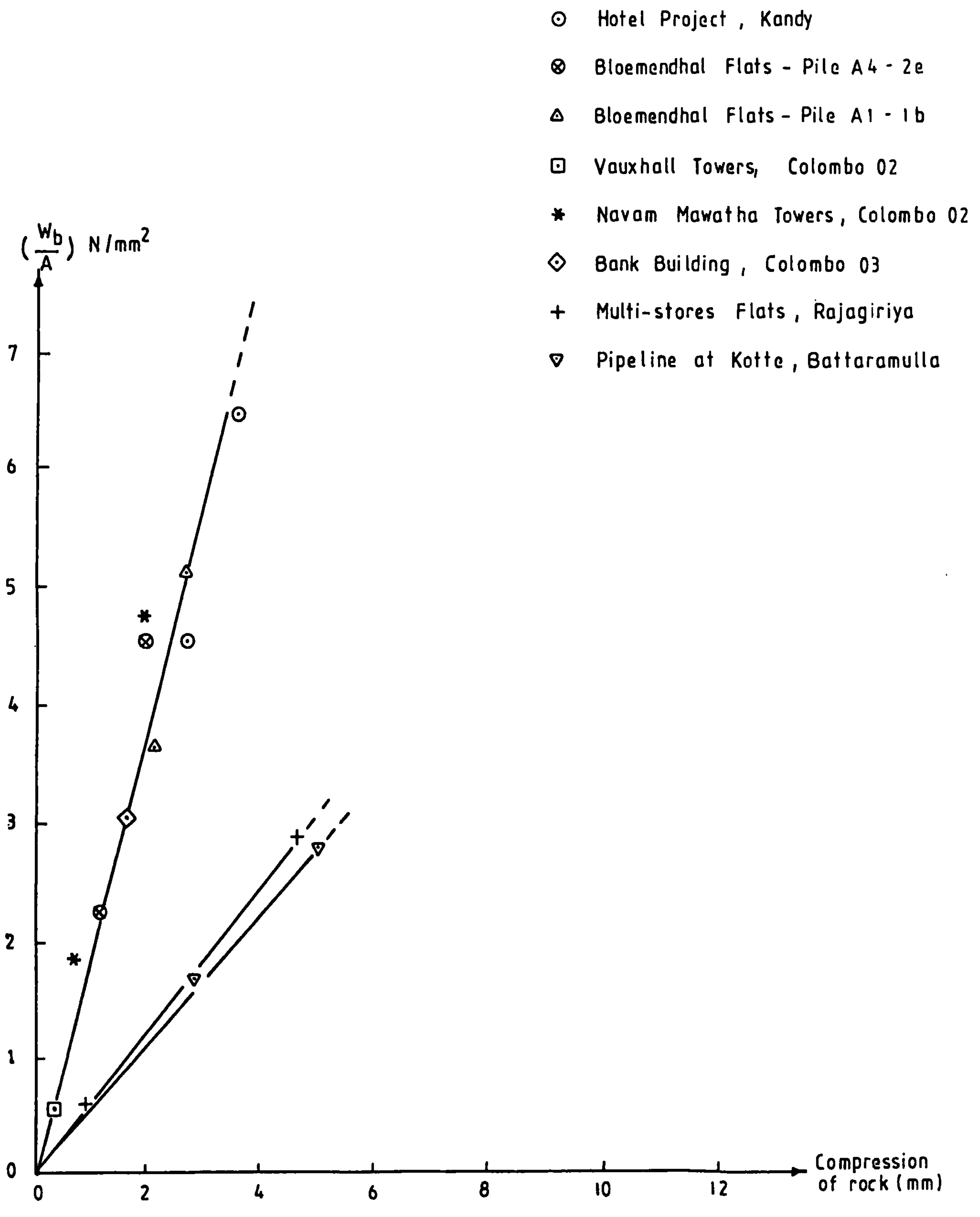

Fig. 2
Relationship between bearing pressure on rock and compression of rock

σ_v = vertical stress at position where skin friction is being computed.

Of the above, σ_v is the only soil parameter which could be ascertained to some degree of accuracy. Therefore, in the present state of knowledge, a semi-empirical method would appear to be the best approach for Design Engineers.

3.2 Skin Friction Coefficient based on SPT values

Some of the empirical relationships that are being presently used in sandy soils are indicated below.

Meyerhof (1976) has proposed that, for low or non-displacement piles such as bored piles, f_u in sandy soils can be estimated from

$$f_u = 1.0 \times N \ \text{kN/m}^2 \qquad \text{Eq.(5)}$$

The Building Control Division, PWD of Singapore uses the relationship

$$f_u = 2 \times N \ \text{kN/m}^2 \text{ and } f_u \ngtr 200 \ \text{kN/m}^2. \qquad \text{Eq.(6)}$$

The Canadian Foundation Engineering Manual of 1992 recommends the values given in Table-4.

Table 4

Nature of Soil	SPT No. (N)	Maximum value of f_u (kN/m^2)
Loose to Medium Dense Sand	< 20	35
Medium Dense to Dense Sand	20 - 50	80
Very Dense Sand	> 50	150

3.3 Skin Friction Coefficient based on pile load tests

Using the method proposed by Tennekoon (1995), the component of load carried by skin friction in a Pile Load Test has been estimated for several cases. These show that a significant part of the pile load is carried by skin friction, and in some cases where the piles are long, there is no load to be transferred by end bearing. This analysis shows that more than 90% of the skin friction load is mobilised by as little as 4 mm of movement measured at the pile head.

In the case histories that have been studied, the skin friction has been provided by

(i) sandy soils in many instances;

(ii) residual soils and highly weathered rock in all cases.

Whilst the former would correspond to a cohesionless material, the latter consists of a (c, ϕ) material whose properties vary considerably by the weathering process. In all these cases, the SPT No. (N) is available, and therefore, the ultimate coefficient of skin friction has been postulated as

$$f_u = k \times N \ \text{kN/m}^2 \text{ and } f_u \ngtr 150 \ \text{kN/m}^2 \qquad \text{Eq.(7)}$$

The value of k obtained to give the skin friction load estimated from the Pile Load Test is given in Table 5.

Table 5

Site	k
Vauxhall Town, Colombo 02	2.7
Navam Mawatha Towers, Colombo 02	1.3
Galle Road Towers, Colombo 03	6.5
Bank Building, Colombo 03	1.3

Therefore, it is recommended, that in sandy soils and residual soils the relationship

$$f_u = 1.3 \times N \ \text{kN/m}^2 \text{ and } f_u \ngtr 150 \ \text{kN/m}^2 \qquad \text{Eq.(8)}$$

be used for estimating the ultimate skin friction load; and that a Factor of Safety of 2 be used in evaluating the allowable load. However, the initial assumption would have to be validated by a load test.

4.0 Design Parameters

4.1 General

When discussing the design of large diameter bored piles in stiff clay, Burland, Butler & Dunican(1966) state that, where straight shafted piles are concerned (as opposed to under-reamed piles), it is quite common to employ a load factor of 2 in the design of single piles. They further go on to state that the British Code of Practice on Foundations suggests that even lower load factors be employed if pile load tests are used for design purposes. From these, they proceed to recommend that, when deciding on the allowable load carrying capacity of a pile, it should be taken as the lesser of the following values:

(i) $Q_{allowable} \leq Q_{ult.\,pile} / 2 \qquad$ Eq.(9)

where $Q_{ult.pile} = Q_{ult.skin\ friction} + Q_{ult.base}$ Eq.(10)

(ii) $Q_{allowable} \leq Q_{ult.skin\ friction} + Q_{ult.base}/2$ Eq.(11)

4.2 Design recommendations

Following a similar procedure, it is recommended that when large diameter bored piles are bearing on rock, the carrying capacity of the pile be determined as the lesser of the following values:

(i) $Q_{allowable} \leq Q_{ult.\ pile}/2$ Eq.(9)

(ii) $Q_{allowable} = (Q_{allowable\ in\ end\ bearing}) + (Q_{ult.\ skin\ friction}\ /F)$ Eq.(12)

It is recommended that, in estimating Q allowable in end bearing, the following procedures be adopted:

(a) if piles are bearing on rock in which cores at founding level can be obtained, then $q_{allowable}$ be determined from Fig.1.

(b)if SPT values at founding level are available, then

$q_{ult.} = 0.04 \times N\ N/mm^2$ with $0.04 N \ngtr 10\ N/mm^2$

and a factor of safety of 3 be then applied to obtain $q_{allowable}$.

(c) if ultimate load in end bearing is determined from a Pile Load Test, then the allowable load in end bearing be determined using a factor of safety of 2.

It is also recommended that in sandy soils and residual soils, when estimating the component of load that can be carried in skin friction from Equation (12),

then F = 2 if skin friction is estimated from SPT results using $f_u = 1.3 \times N\ kN/m^2$ and $f_u \ngtr 150\ kN/m^2$; and f_u 150 kN/m^2;

and F = 1.5 if skin friction is estimated from results of Pile Load Tests

5.0 Closure

The results of several pile load tests have been analysed, and recommendations for pile design have been given in Sec. 4.2.

6.0 Acknowledgements

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