

A Brief Guide to Design of Bored Piles under Axial Compression – A Malaysian Approach

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ABSTRACT: This paper presents a summary of design methodologies commonly used in Malaysia for bored piles under axial compression. Since in Malaysia the design of bored piles in soil is usually based on the Standard Penetration Tests (SPT), that are extensively carried out at site, the empirical equations correlating the value of the ultimate shaft resistance (f_{su}) and the ultimate base resistance (f_{bu}) to SPT 'N' values are suggested. The load-transfer method to predict the load-settlement and load distribution of a pile is briefly described. Rock socket piles are also very common where competent founding bedrock is found within the reachable depth. Design approaches for rock socket piles in Malaysia are also presented.

1. Introduction

Bored piles are commonly used in Malaysia as foundation to support heavily loaded structures like high-rise buildings and bridges in view of its low noise, low vibration, and flexibility of sizes to suite different loading conditions and subsoil conditions. This paper presents a summary of design methodologies commonly practised in Malaysia for bored piles under axial compression.

2. Geotechnical Capacity of Bored Piles

2.1 Factor of Safety

The Factors of Safety (FOS) normally used in static evaluation of bored pile geotechnical capacity are partial FOS on shaft (F_s) and base (F_b) respectively; and global FOS (F_g) on total capacity. The lower geotechnical capacity obtained from both methods is adopted as allowable geotechnical capacity

$$Q_{ag} = \frac{Q_{su}}{F_s} + \frac{Q_{bu}}{F_b} \quad (\text{eq.1})$$

$$Q_{ag} = \frac{Q_{su} + Q_{bu}}{F_g} \quad (\text{eq.2})$$

Note: Use the lower of Q_{ag} obtained from eq. 1 and eq. 2 above.

Where:

Q_{ag} = Allowable geotechnical capacity (have not included down drag force, if any)

Q_{su} = Ultimate shaft capacity = $\sum_i (f_{su} \times A_s)$

i = Number of soil layers

Q_{bu} = Ultimate base capacity = $f_{bu} A_b$

f_s = Unit shaft resistance for each layer of embedded soil

f_b = Unit base resistance for the bearing layer of soil

A_s = Pile shaft area

A_b	=	Pile base area
F_s	=	Partial Factor of Safety for Shaft Resistance = 1.5
F_b	=	Partial Factor of Safety for Base Resistance > 3.0
F_g	=	Global Factor of Safety for Total Resistance (Base + Shaft) = 2.0

In general, the contribution of base resistance in bored piles shall be ignored due to difficulty of proper base cleaning especially in wet hole (with drilling fluid). The contribution of base resistance can only be used if it is constructed in dry hole, proper inspection of the base can be carried out or base grouting is implemented.

2.1 Design of Geotechnical Capacity in Soil

The design of bored pile geotechnical capacity commonly used can be divided into two major categories namely:

- a) Semi-empirical Method
- b) Simplified Soil Mechanics Method

2.1.1 Semi-empirical Method

Bored piles are constructed in tropical residual soils that generally have complex soil characteristics. The complexity of these founding medium with significant changes in ground properties over short distance and friable nature of the materials make undisturbed sampling and laboratory strength and stiffness testing of the material difficult. Furthermore current theoretically based formulae also do not consider the effects of soil disturbance, stress relief and partial reestablishment of ground stresses that occur during the construction of bored piles; therefore, the sophistication involved in using such formulae may not be necessary.

Semi-empirical correlations have been extensively developed relating both shaft resistance and base resistance of bored piles to N-values from Standard Penetration Tests (SPT'N' values). In the correlations established, the SPT'N' values generally refer to uncorrected values before pile installation.

The commonly used correlations for bored piles are as follows:

$$f_{su} = K_{su} \times \text{SPT'N'} \text{ (in kPa)}$$

$$f_{bu} = K_{bu} \times \text{SPT'N'} \text{ (in kPa)}$$

Where:

- | | | |
|----------|---|--|
| K_{su} | = | Ultimate shaft resistance factor |
| K_{bu} | = | Ultimate base resistance factor |
| SPT'N' | = | Standard Penetration Tests blow counts (blows/300mm) |

For shaft resistance, Tan *et al.* (1998), from the results of 13 nos. of fully instrumented bored piles in residual soils, presents K_{su} of 2.6 but limiting the f_{su} values to 200kPa. Toh *et al.* (1989) also reported that the average K_{su} obtained varies from 5 at SPT'N' 20 to as low as 1.5 at SPT'N'=220. Chang & Broms (1991) suggests that K_{su} of 2 for bored piles in residual soils of Singapore with SPT'N'<150.

For base resistance, K_{bu} values reported by many researchers varies significantly indicating difficulty in obtaining proper and consistent base cleaning during construction of bored piles. It is very dangerous if the base resistance is relied upon when the proper cleaning of the base cannot be assured. From back-analyses of test piles, Chang & Broms (1991) shows that K_{bu} equals to 30 to 45 and Toh *et al.* (1989) reports that K_{bu} falls between 27 and 60 as obtained from the two piles that were tested to failure.

Lower values of K_{bu} between 7 and 10 were reported by Tan *et al.* (1998). The relatively low K_{bu} values are most probably due to soft toe effect which is very much dependent on the workmanship and pile geometry. This is even more pronouncing in long pile. Furthermore, a relatively larger base movement is required to mobilise the maximum base resistance as compared to the displacement needed to fully mobilise shaft resistance. The base displacement of approximately 5% to 10% of the pile diameter is generally required to mobilise the ultimate base resistance provided that the base is properly cleaned and checked.

In view of the large movement required to mobilise the base resistance of bored piles and difficulty in base cleaning, the authors strongly recommend to ignore the base contribution in the bored pile design unless proper base cleaning can be assured and verified.

2.1.2 Simplified Soil Mechanics Methods

Generally the simplified soil mechanics methods for bored pile design can be classified into fine grained soils (e.g. clays, silts) and coarse grained soils (e.g. sands and gravels).

Fine Grained Soils

The ultimate shaft resistance (f_{su}) of bored piles in fine grained soils can be estimated based on the semi-empirical undrained method as follows:

$$f_{su} = \alpha \times S_u$$

Where :

α = adhesion factor
 S_u = undrained shear strength (kPa)

Whitaker & Cooke (1966) reports that the α value lies in the range of 0.3 to 0.6 for stiff over-consolidated clays, while Tomlinson (1994) and Reese & O'Neill (1988) report α values in the range of 0.4 to 0.9. The α values for residual soils of Malaysia are also within this range. Where soft clay is encountered, a preliminary α value of 0.8 to 1.0 is usually adopted together with the corrected undrained shear strength from the vane shear test. This method is useful if the bored piles are to be constructed on soft clay near river or at coastal area. The value of α to be used shall be verified by preliminary pile load test.

In the case where bored piles are subjected to significant variations in stress levels after installation (e.g. excavation for basement, rise in groundwater table) the use of the effective stress method is more representative as compared to undrained method. This is because the effective stress can take account of the effects of effective stress change on the K_{se} values to be used. The value of ultimate shaft resistance may be estimated from the following expression:

$$f_{su} = K_{se} \times \sigma_v' \times \tan \phi'$$

Where :

K_{se} = Effective Stress Shaft Resistance Factor = [can be assumed as K_0]
 σ_v' = Vertical Effective Stress (kPa)
 ϕ' = Effective Angle of Friction (degree) of fine grained soils.

However, this method is not popular in Malaysia and limited case histories of back-analysed K_{se} values are available for practical usage of the design engineer.

Although the theoretical ultimate base resistance for bored pile in fine grained soil can be related to undrained shear strength as follows;

$$f_{bu} = N_c \times S_u$$

Where:

N_c = bearing capacity factor

it is not recommended to include base resistance in the calculation of the bored pile geotechnical capacity due to difficulty and uncertainty in base cleaning.

Coarse Grained Soils

The ultimate shaft resistance (f_{su}) of bored piles in coarse grained soils can be expressed in terms of effective stresses as follows:

$$f_{su} = \beta \times \sigma_v'$$

Where:

β = shaft resistance factor for coarse grained soils.

The β values can be obtained from back-analyses of pile load tests. The typical β values of bored piles in loose sand and dense sand are 0.15 to 0.3 and 0.25 to 0.6 respectively based on Davies & Chan (1981).

Although the theoretical ultimate base resistance for bored pile in coarse grained soil can be related to plasticity theories, it is not recommended to be included in the calculation of the bored pile geotechnical capacity due to difficulty and uncertainty in base cleaning.

2.2 Design of Geotechnical Capacity in Rock

The three major rock formations, namely sedimentary, igneous and metamorphic rocks, are commonly encountered in Malaysia. When designing structures over these formations using bored pile, the design approaches could vary significantly depending on the formations and the local experience established on a particular formation.

In Malaysia, bored pile design in rocks is heavily based on semi-empirical method. Generally, the design rock socket friction is the function of surface roughness of rock socket, unconfined compressive strength of intact rock, confining stiffness around the socket in relation to fractures of rock mass and socket diameter, and the geometry ratio of socket length-to-diameter. Roughness is important factor in rock socket pile design as it has significant effect on the normal contact stress at the socket interface during shearing. The normal contact stress increases due to dilation resulting increase of socket friction. The level of dilation is mostly governed by the socket roughness. The second factor on the intact rock strength governs the ability of the irregular asperity of the socket interface transferring the shear force, otherwise shearing through the irregular asperity will occur due to highly concentrated shear forces from the socket. The third factor will govern the overall performance of strength and stiffness of the rock socket in jointed or fractured rock mass and the last factor is controlled by the profile of socket friction distribution. It is very complicated to quantify all these aspects in the rock socket pile design. Therefore, based on the conservative approach and local experience, some semi-empirical methods have evolved to facilitate the quick socket design with considerations to all these aspects. In most cases, roughness of socket is qualitatively considered as a result of lacking of systematic assessing method. Whereas the other three factors can be quantified through strength tests on the rock cores and point load tests on the recovered fragments, the RQD values of the core samples and some analytical method on assessing the socket friction distribution. It is also customary to perform working load test to verify the rock socket design using such semi-empirical method. Safety factor of two is the common requirements for rock socket pile design. Table 1 summarises the typical design socket friction values for various rock formations in Malaysia.

Table 1 Summary of Rock Socket Friction Design Values

Rock Formation	Working Rock Socket Friction*	Source
Limestone	300kPa for RQD <25% 600kPa for RQD =25 – 70% 1000kPa for RQD >70% The above design values are subject to 0.05x minimum of { q_{uc} , f_{cu} } whichever is smaller.	Neoh (1998)
Sandstone	$0.10 \times q_{uc}$	Thorne (1977)
Shale	$0.05 \times q_{uc}$	Thorne (1977)
Granite	1000 – 1500kPa for $q_{uc} > 30\text{N/mm}^2$	-

* Note: Lower range to Grade III and higher range for Grade II or better

Another more systematic approach developed by Rosenberg & Journeaux (1976), Horvath (1978) and Williams & Pells (1981) is also used in Malaysia. The following simple expression is used to compute the rock socket friction with consideration of the strength of intact rock and the rock mass effect due to discontinuities.

$$f_s = \alpha \times \beta \times q_{uc}$$

Where:

q_{uc} is the unconfined compressive strength of intact rock

α is the reduction factor with respect to q_{uc} (Figure 1)

β is the reduction factor with respect to the rock mass effect (Figure 2)

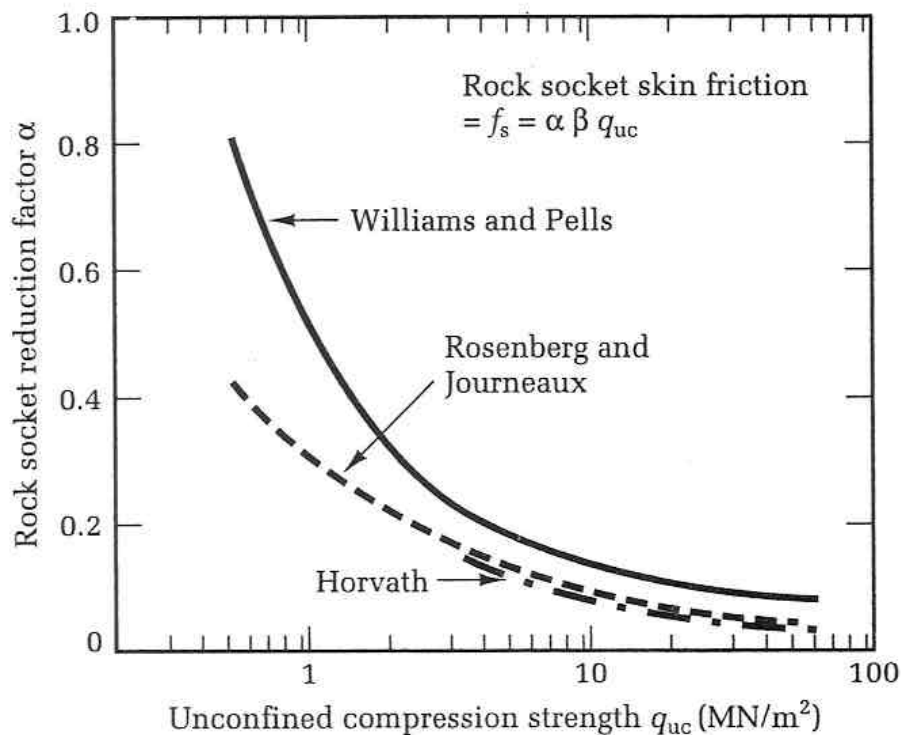


Figure 1 Rock Socket Reduction Factor, α , w.r.t. Unconfined Compressive Strength (after Tomlinson, 1995)

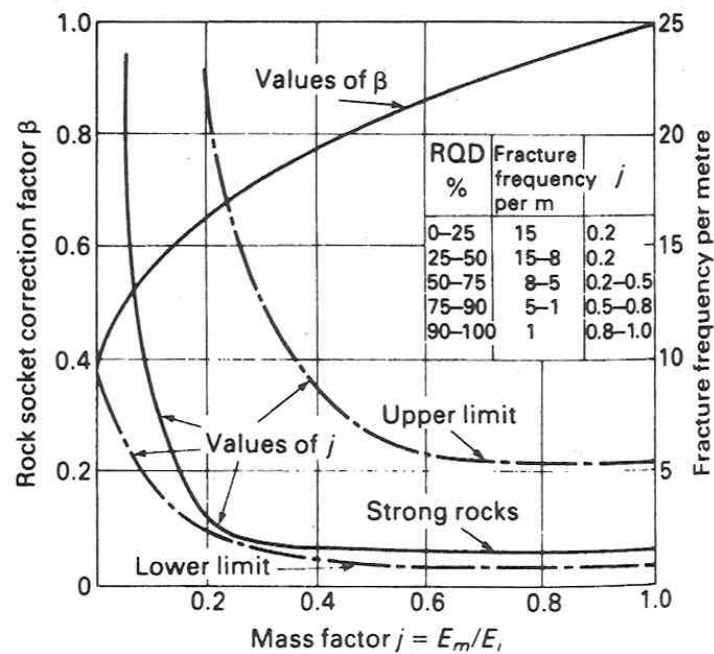


Figure 2 Rock Socket Reduction Factor, β , w.r.t. Rock Mass Discontinuity (after Tomlinson, 1995)

During borehole exploration, statistics of q_{uc} can be established for different weathering grade of bedrock and the rock fracture can be assessed through the Rock Quality Designation on the rock core recovered or by interpretation of pressuremeter modulus in the rock mass against the elastic modulus of intact rock, which is equivalent to mass factor j , which is the ratio of elastic modulus of rock mass to that of intact rock, as in Figure 2. Alternatively, Figure 3 can provide some indications of the modulus ratio of the rock mass. In the some cases, at very small cost, point load test equipment is used to assess and verify the rock strength on the recovered rock fragment during bored pile drilling after proper calibration with borehole results.

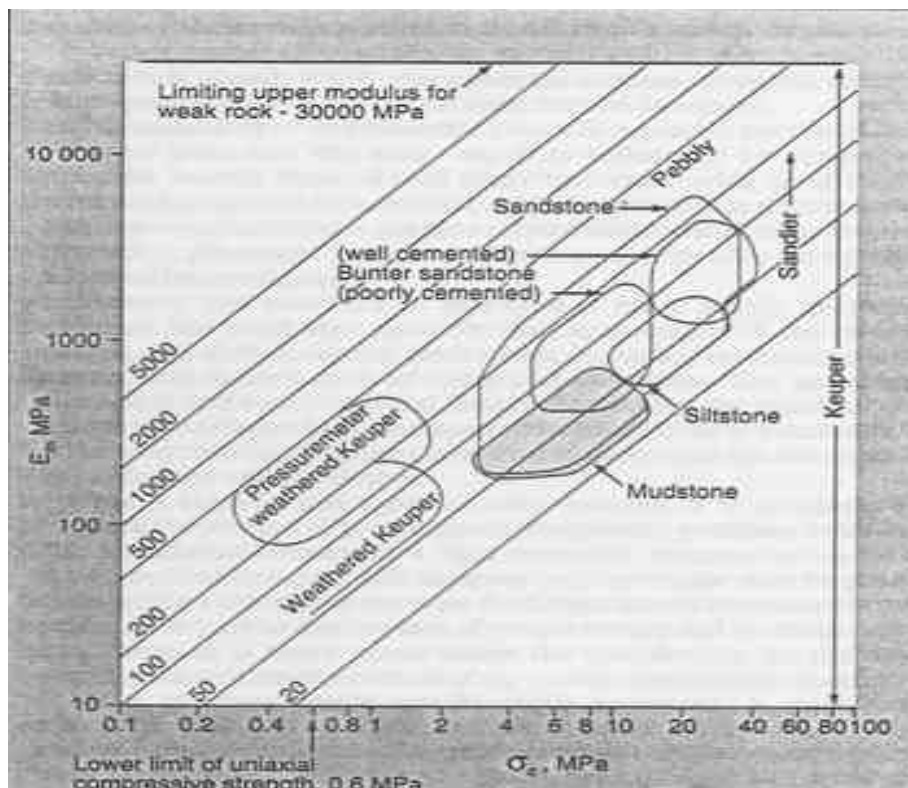


Figure 3 Modulus Ratio Ranges (after Hobbs, 1974)

Due to difficulties on quantification of socket roughness, the effect of roughness has not been explicitly addressed in the above approach, but rather implicitly included in the α factor with certain socket construction method. Based on the works by Kulhawy & Phoon (1993), in which is an extension of the above mentioned model by modifying the friction reduction factor with respect to different socket roughness as shown in the following expression and Figure 4, Seidel & Haberfield (1995) have further developed the theoretical methodology and a computer program, "Rocket" for rock socket design. However, it has not gained wide acceptance in Malaysia as a result of requiring special measuring equipment for the socket roughness for the input of the said computer program. Nevertheless, Figure 4 does provide useful reference on limestone, sandstone, shale, mudstone and clay to account for the socket roughness. The parameter, ψ , is used to represent the socket roughness.

$$\alpha = \psi \times (q_{uc}/2p_a)^{-1/2}$$

Where:

ψ : Indicator of socket roughness

p_a : Atmospheric pressure for normalisation

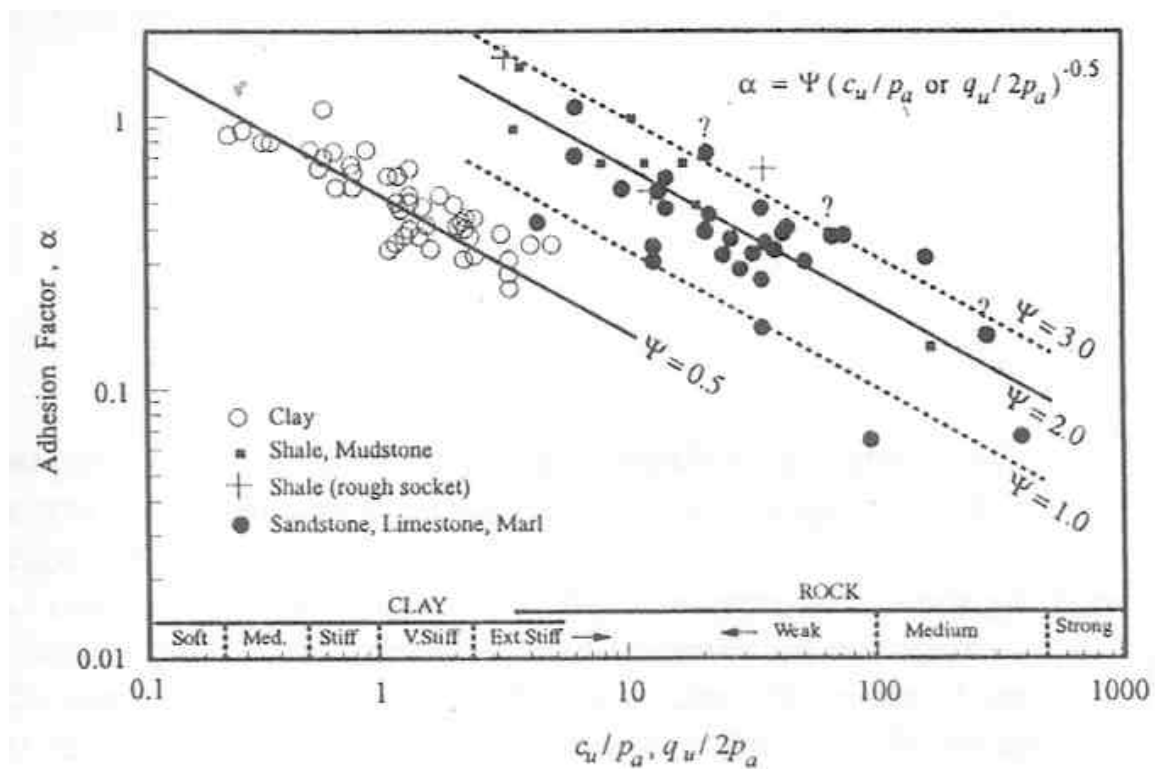


Figure 4 Relation between Socket Roughness, Socket Reduction Factor and Normalised Rock Strength (after Kulhawy & Phoon,1993)

It is also important to optimise rock socket design with due consideration of the load transfer behaviour of the socket. Figure 5 shows the analytical results of the socket load transfer behaviour for modulus ratio, E_p/E_r ranging from 0.25 to 1000. As shown in the figure, it is obvious that there is really no reason to extend the socket beyond 5 times the pile diameter for $E_p/E_r = 0.25$ (very competent intact rock) as no load will be transferred below this socket length.

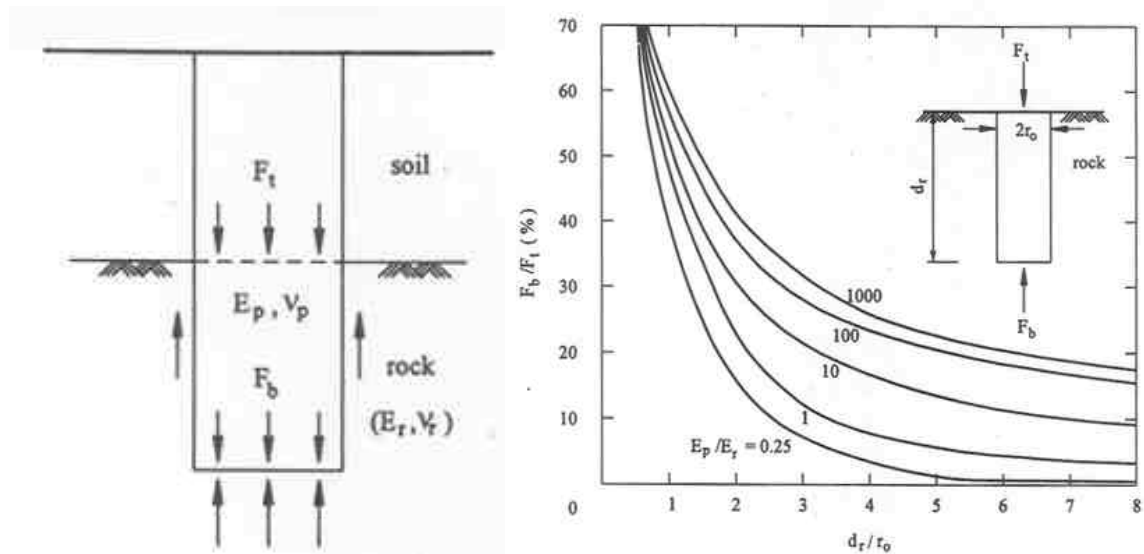


Figure 5 Distribution of Socket Resistance w.r.t. Socket Length and Modulus Ratio (after Pells & Tuner, 1979)

Sometimes, the borehole is a dry hole and at shallow depth, then base resistance will be considered if the base cleaning and inspection of the base condition can be carried out. Very often, the movement to mobilise the base resistance is few folds higher than that to mobilise the socket friction despite the ultimate base resistance could be very high. As such, with consideration of compatibility of the pile movement in mobilising both the socket and base, appropriate mobilising factors to both the socket and base shall be applied to the foundation design after verification from the fully instrumented pile load test. Such mobilising factor shall be at least 3, but finally subjected to verification by instrumented load test prior to production of working piles if there is large number of piles for value engineering. The assessment of ultimate end bearing capacity of bored pile in rock can be carried using the following expression.

$$Q_{ub} = cN_c + \gamma BN_\gamma/2 + \gamma DN_q$$

Where:

- c : Cohesion
- B : Pile diameter
- D : Depth of pile base below rock surface
- γ : Effective density of rock mass
- N_c, N_γ & N_q : Bearing capacity factors related to friction angle, ϕ (Table 2, for circular case, multipliers of 1.2 & 0.7 shall be applied to N_c & N_γ respectively)
- N_c : $2N_\phi^{1/2}(N_\phi+1)$
- N_γ : $N_\phi^{1/2}(N_\phi^2-1)$
- N_q : N_ϕ^2
- N_ϕ : $\tan^2(45^\circ+\phi/2)$

Table 2 Typical Friction Angle for Intact Rock (Wyllie, 1991)

Classification	Type	Friction Angle
Low Friction	Schist (with high mica content), Shale	20° - 27°
Medium Friction	Sandstone, Siltstone, Gneiss	27° - 34°
High Friction	Granite	34° - 40°

If the pile length is significant, the contribution of the shaft resistance in the soil embedment above the rock socket shall also be considered in the overall pile resistance assessment. In most cases for rock socket pile, the settlement performance is usually governed by the elastic shortening of the pile shaft. The socket displacement is usually insignificant. However, load transfer analyses would provide the overall settlement performance.

Construction method is another important aspect to be considered in the bored pile design on rock. In Malaysia, there are two most common methods in forming the rock socket, namely rock coring with rock cutting bits and chiselling by mechanical impact. Both methods have their own merits and need skilful operator to form a proper rock socket. In general, rock coring method will form a smoother, but intact, socket surface. Whereas chiselling method will form relatively rougher socket, but could be more fracture due to disturbance to the inherent discontinuities in bedrock. Chiselling is usually used as a supplementary technique in drilling through hard rock.

There are also other inherent problems associated with some of the aforementioned rock formations such as:

- a. Limestone: Existence of erratic karst features will need further consideration in the foundation pile design. Downgrading of pile capacity for piles founded on these karst features or install the pile at deeper depth to penetrate these features or treatment to strengthen them can be considered depending on the cost-benefit analyses of the viable options. Another problem in limestone formation is the existence of slime made of very loose sand or soft silty clay immediately above the bedrock, which can cause frequent cave-in and pose difficulties in cleaning up the rock socket. Chan & Hong (1985) presented the problems of pile construction over limestone. European Foundations (1998) presented the problems encountered in pile construction in Kuala Lumpur limestone. Gue (1999) presented some solutions to overcome the abovementioned problems and the construction controls.
- b. Degradable sedimentary formations: These formations easily subject to rapid degradation in terms of strength and stiffness as a result of stress relief and ingress of drilling fluid. Slow progress in drilling operation due to inefficient coring method or inter-layered hard and soft rocks and delay in concreting the piles are the usual causes of such softening. The solutions to these problems are to use powerful drilling equipments and avoid delay in concreting.
- c. Granite: Core boulders are common features in this formation. This feature can be easily observed from the outcrops or along river. Therefore, it is important to identify proper founding stratum for the foundation piles during the subsurface investigation. This can be overcome by careful assessment of the weathering profile interpreted from the deep boring exploratory holes.

2.3 Verification of Bored Piles Capacity

For the verification of bored pile capacity, maintained load test is the normal mean specified by most practicing engineers. In certain cases where detailed interaction behaviours between the pile and the foundation formations are of the interest of the designer for design refinement and value engineering, full scale instrumented test pile equipped with multi-level strain gauges, extensometers and occasionally Osterberg load cell and polyfoam soft toe are constructed and tested depending the objective of the verification. Conventional static maintained load test is the most common verification pile test adopted by the design engineers in Malaysia. Quick maintain load test has also gained wide acceptance for the test piles in founding materials, which are not subject to excessive creep or time dependent movement under loading. Otherwise, conventional long holding period at various test load intervals will be used to confirm the time dependent movement of pile. Other indirect tests, such as high strain dynamic pile and statnamic pile tests, have been occasionally used to verify the design.

3.0 Structural requirements of Bored Pile

Following are some brief guidelines for structural design of bored piles:

- a) Allowable structural capacity of bored piles (BS8004, Clause 7.4.4.3.1)

$$\text{Allowable structural capacity of bored piles} = 0.25 \times f_{cu} \times A_c$$

Where:

f_{cu} = concrete cube strength at 28 days (Grade 30 to 35 is most common)

A_c = cross-sectional area of the pile

b) Cover for reinforcement (BS8004, Clause 2.4.5)

Cover for reinforcement = (40mm + values in Table 3.4, BS8110: Part 1)

For example, bored piles (concrete G35) in non-aggressive soil shall required minimum cover of (40mm + 35mm) = 75mm

c) Reinforcement (BS8110: Part 1)

For bored piles in compression only, the structural capacity is derived from the concrete strength alone and some nominal reinforcement is sometimes provided to prevent damage during construction. However, for bored piles supporting bridges where there will be bending moment and shear force acting on the piles, then the bored piles can be designed like beam. Length of the reinforcement can be curtailed until the influence depth of the flexural effect. However, for ease of construction, minimum steels are usually provided right to the bottom of the bored pile to support the upper steel cage during concrete casting. In few cases, hanging the steel cage without the lower supporting steel reinforcements has also been successfully carried out with extra care.

4. Prediction of Bored Pile Settlement

In order to optimise the design of bored pile, it is important to be able to correctly predict both bearing capacity and settlement of pile under different loading. In view of this, a simple load-transfer method (Coyle & Reese, 1966) can be utilised to predict the load-settlement and load distribution of a pile. However, to obtain reasonably reliable prediction of load-settlement characteristics of pile using this method will require sufficient good quality database of load-transfer curves and parameters from fully instrumented test piles tested in similar ground condition to be available for a better correlation with soil properties and pile geometry. Tan et al. (1998) suggests load-transfer parameters obtained from the testing of full-scale instrumented bored piles in residual soil of Malaysia. The necessary correlations to SPT'N' values are also reported.

4.1 Load Transfer Curves for Shaft

The development of shaft resistance is dependent on the relative settlement between the subsoil and the pile shaft and can be expressed as follows:

$$Q_s = \sum_i [f_s(z_s) \cdot A_s]$$

Where:

Q_s = Total Shaft Capacity of the Pile (kN)

$f_s(z_s)$ = Unit shaft resistance for each layer of soil with relative displacement of z_s . (kPa)

i = Number of soil layer.

z_s = Shaft displacement (mm)

A_s = Pile shaft area at each soil layer

Figure 6 shows a typical load transfer curve for shaft. Shaft displacement, z_s is the relative displacement between the pile/soil interface at the mid-depth of each soil stratum.

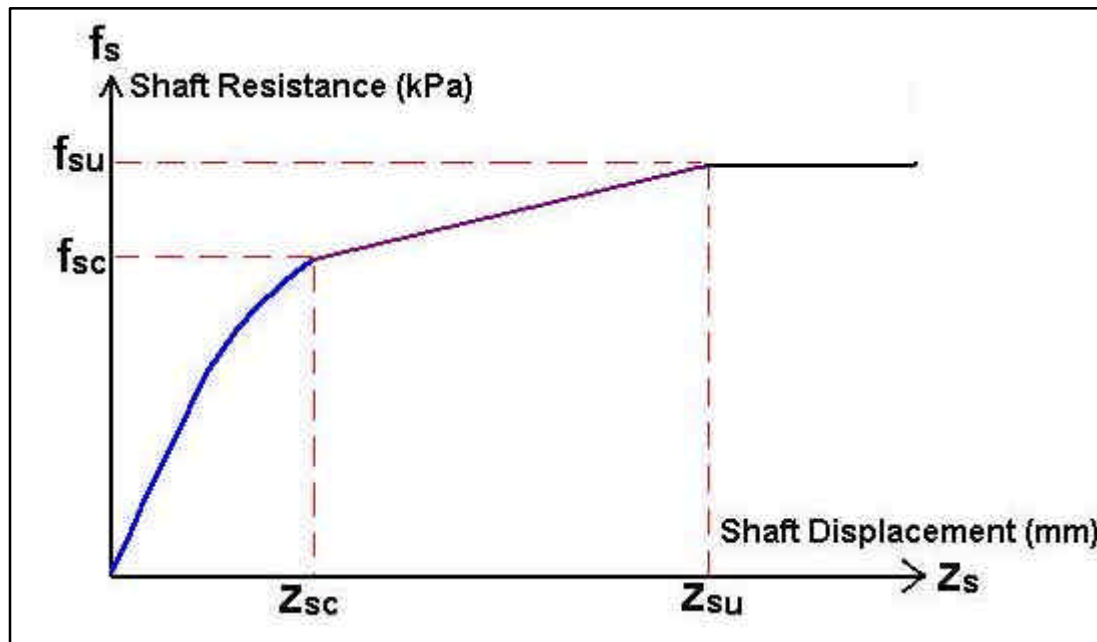


Figure 6 Typical Load Transfer Curve for Shaft

Where:

- f_{sc} = Critical shaft resistance corresponding to critical shaft displacement (kPa)
- z_{sc} = Critical shaft displacement (mm)
- f_{su} = Ultimate shaft resistance corresponding to ultimate shaft displacement (kPa)
- z_{su} = Ultimate shaft displacement (mm)

The measured load transfer curves obtained from 13 nos. of instrumented test piles are normalised against critical shaft resistance (f_{sc}) and critical shaft displacement (z_{sc}). The normalised load transfer curve is shown in Figure 7.

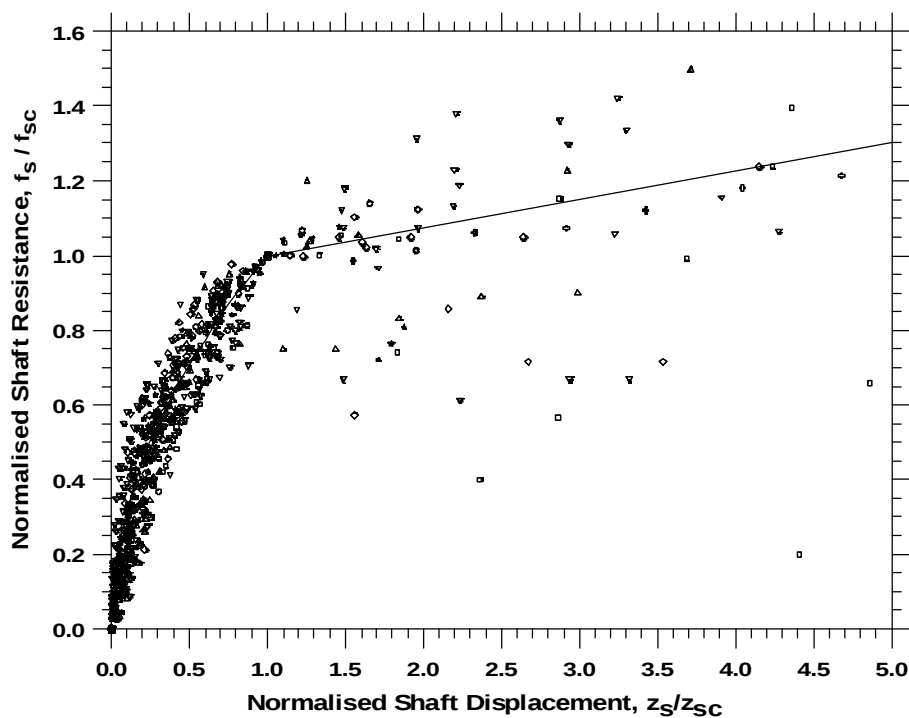


Figure 7 Normalised Load Transfer Curves for Shaft (after Tan et al., 1998)

The best-fit curve obtained to model the load-displacement characteristic of the shaft resistance is as follows:

$$(f_s/f_{sc}) = (z_s/z_{sc})^{1/2} ; \text{ for } (z_s/z_{sc}) < 1.0$$

$$(f_s/f_{sc}) = 1 + \frac{3}{50} (z_s/z_{sc}) ; \text{ for } 1.0 < (z_s/z_{sc}) < 5.0 ; \text{ and}$$

$$(f_s/f_{sc}) = 1.3 ; \text{ for } (z_s/z_{sc}) > 5.0$$

and

$$f_{sc} = 2 \times \text{SPT}'N' \text{ (kPa)} \leq 150 \text{ kPa}$$

z_{sc} = can be obtained from Figure 8.

There are many factors that have influence on the value of critical shaft displacement (z_{sc}) of bored pile and they are drilling method (dry or wet), type of drilling fluid, type of soil, spatial variation of soil properties (stiffness and strength), drilling and concreting duration, drilling tools and also diameter of piles. Tan et al. (1998) selected two key factors, namely the pile diameter and soil strength (via SPT' N' values), that can be easily quantified to evaluate their relationship with z_{sc} and are presented in Figure 8. In general, the critical shaft displacement increases with the increase of pile diameter or decrease in SPT' N' values.

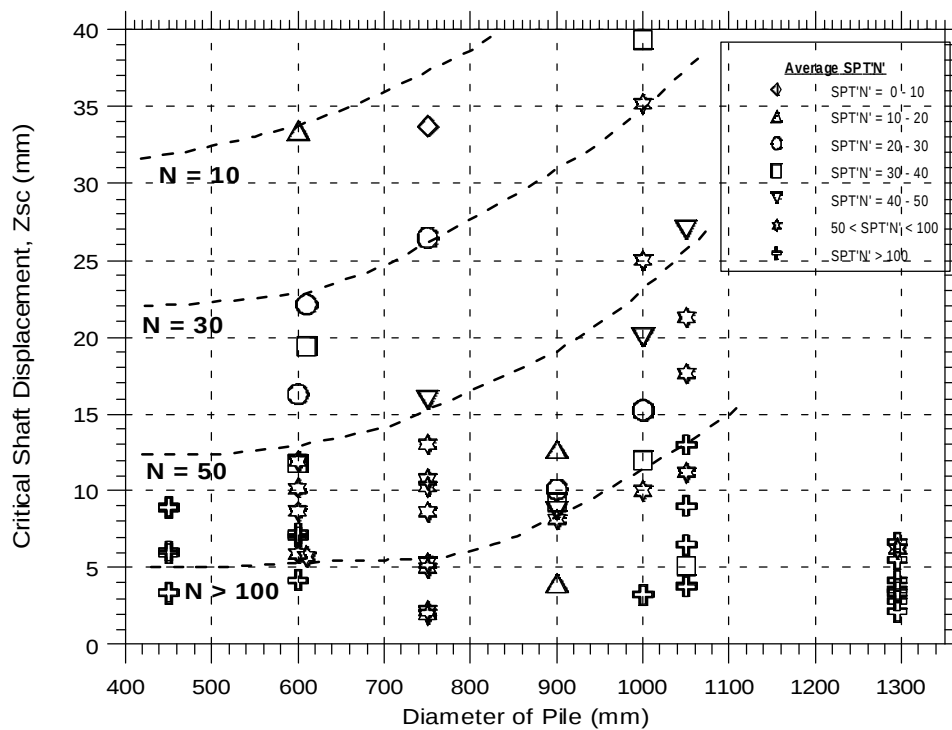


Figure 8 Relationship of z_{sc} with pile diameter & SPT' N' (after Tan et al., 1998)

4.2 Load Transfer Curves for Base

Similar to shaft resistance, the load transfer curves for base can be normalised and presented in Figure 9.

The best-fit curve obtained to model the load-displacement characteristic of the base resistance is as follows:

$$(f_b/f_{bc}) = (Z_b/Z_{bc})^{2/3}$$

Where:

f_{bc} = Critical base resistance corresponding to critical base displacement (kPa)

Z_{bc} = Critical base displacement (mm)

Note: From the field tests, the $f_{bc}=f_{bu}$.

$$f_{bc} = (7 \text{ to } 10) \times \text{SPT}'N' \text{ (kPa)}$$

$$Z_{bc} = 5\% \text{ of pile diameter.}$$

Note: When using the value above, proper base cleaning using cleaning bucket shall be carried out at site.

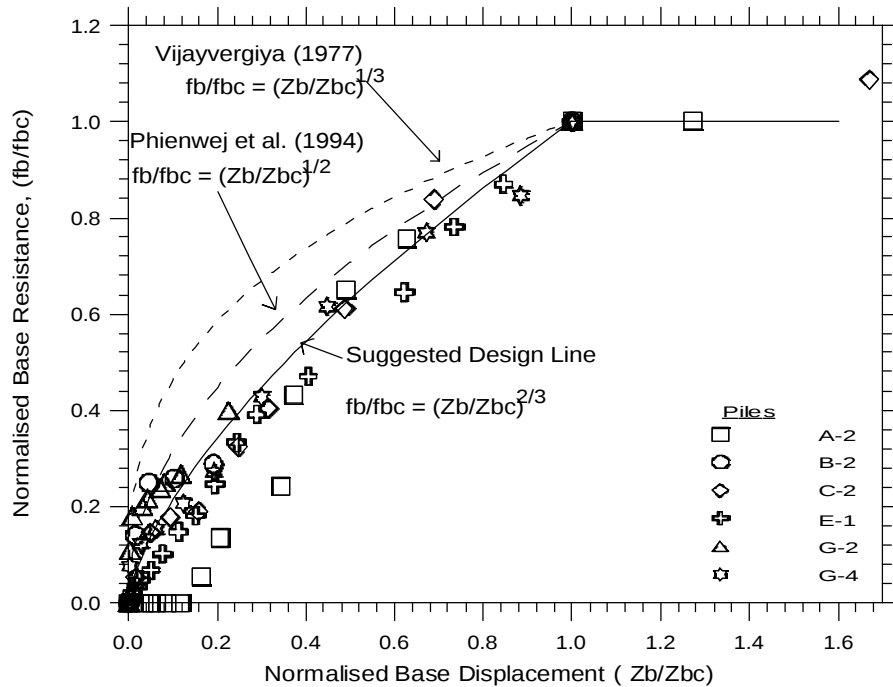


Figure 9 Normalised Base Resistance and Displacement (after Tan et al., 1998)

5. Conclusions

From the above elaborations, the following conclusions can be drawn for the bored pile design practice in Malaysia:

1. For the design of bored piles in soil, the two common methods, namely semi-empirical and simplified soil mechanics methods are commonly used to determine the ultimate pile capacity.
2. For the safety margin of pile capacity, partial safety factor of 1.5 and 3.0 for shaft and base resistances respectively and global safety factor of 2.0 applied to overall ultimate pile capacity (sum of ultimate shaft and base resistances) are used.
3. The use of load transfer method is important to optimise the pile design for value engineering and also provide settlement performance.
4. For rock socket pile design, design approach and charts with consideration of socket roughness, rock strength, rock mass stiffness and socket geometry are presented and discussed.
5. In most scenarios, base resistance of bored pile is usually ignored due to uncertainties in cleaning. Unless for the case of dry hole and inspection of the base is possible, then base resistance can be considered with appropriate mobilising factor.
6. Instrumentation test pile is used for design optimisation and value engineering if there are sufficient pile points for the project to justify the testing cost.

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