rocscience

RSPile

Bored Pile

Theory Manual

Table of Contents

1. Introduction

The purpose of this manual is to describe the theoretical background of the Bored feature in **RSPile**. Bored is devoted for calculation of axial carrying capacity of bored reinforced concrete piles. The feature is available under the “Bored” tab in “Project Settings” in RSPile.

1.1. Capacities

The calculated capacity consists of ultimate skin resistance, ultimate end bearing resistance and total ultimate pile capacity.

1.2. Special Design Considerations

The following design considerations are mutually exclusive. They cannot both be used at the same time:

1.2.1. Scour

There are two types of scours that can be considered: long-term and short-term scour. For short-term scour, the shear stress is simply reduced to zero. This occurs due to erosion around the pile. The effect overburden pressure, σ_v’, is not affected due to the presence of the soil around the pile area. No end bearing can be placed above this level.

For long-term scour, the effective overburden stress is reduced to zero until the scour consideration depth. This is due to soil eroding over a large area, reducing the effective overburden stress. The program stacks the effects of both scour types, where long-term scour is first considered then local scour is applied below the long-term scour depth.

1.2.2. Soft Comprehensible Soils / Negative Skin Friction

A depth of soft compressible soil at the top of the soil profile can be specified. For ultimate calculations, the shaft resistance from the soft soil layer can be considered as soft compressible soil or as negative skin friction.

If the soil at the top of the shaft is a soft soil, the skin friction for the layer is not included in the ultimate skin friction capacity. If negative skin friction is considered, the skin friction from the soft soil layer will be considered negative and is subtracted from the total skin friction for ultimate capacity computations.

1.3. Ultimate Pile Carrying Capacity

Pile load carrying capacity Q_{ult} in compression (downward vertical), T_{ult} in tension (uplift).

According to the traditional literature and most of the standard codes, the ultimate load carrying capacity of a pile is calculated as:

$$Q_{ult} = Q_{sult} + Q_{bult}$$

$$Q_{sult} = \sum_{i}^{n} A_{si}\, f_{sult} \qquad A_{si} = L_i * \pi D$$

$$Q_{bult} = \ q_{bult}\, A_b \qquad A_b = \frac{D^2\pi}{4}$$

where

Q_{bult} is the ultimate base capacity and Q_{sult} is the ultimate frictional capacity

q_{bult} and f_{sult} are the ultimate unit base resistance (end bearing capacity) and skin resistance (frictional resistance) respectively,

A_b and A_{si} are the area of the base at pile tip and the area of the surface of the shaft respectively, with i denoting the segment number if the shaft is segmented.

If the pile is large and have a significant weight the weight should be deducted from the capacity above to get the net pile capacity:

$$Q_{ult(net)} = Q_{sult} + Q_{bult} - W_p$$

This is not included in the “Bored” feature in **RSPile**.

Many methods are available for estimating the pile resistance. The ultimate skin resistance and ultimate end bearing of a bored pile are determined by theoretical and empirical methods. The choice of the method is dependent on soil type, construction method, soil parameters, availabilities of field tests, and local experience.

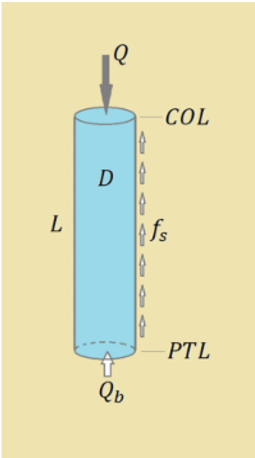

Figure 1.0: A sketch showing the load and resistance components of a pile

The uplift capacity T_{ult} (tension on pile head) may be assumed as the ultimate skin resistance in compression Q_{sult} reduced by a suitable factor. Negative skin friction is turned positive and W_p is added instead of subtraction.

In all methods below, the pile is divided into segments based on changes in cross section, soil or rock layers and ground water level.

2. Skin Resistance in Cohesionless Soil

2.1. Traditional K_s tan∆ Method

The basic method to estimate the skin resistance of a bored pile in cohesionless soil (c = 0) is to determine the vertical effective stress at the middle of each segment length and then get the effective lateral pressure on the pile surface (perpendicular to the pile surface) by multiplying the vertical effective stress by the coefficient of lateral earth pressure K_s.

$f_{sult} = K_s \sigma'_{v0} \tan \delta$

Where K_s is found from its relation to K_o, the coefficient of lateral earth pressure at rest. The wall friction angle δ is found empirically or through testing as a ratio of the angle of internal friction of the cohesionless soil ϕ'. The ratios of K_s/K_0 and δ/ϕ' that may be used for design are listed in Table 1 and Table 2. Of course, σ'_{v0} is the effective vertical stress calculated at mid depth of each segment.

The friction angle may be found in laboratory or from field SPT N values as (Peck et al., 1974):

$\phi' = 27.1+0.3N_{60}-0.00054[N_{60}]^2$

Where N_{60} is taken as the average number of the N values within the elevation of ϕ.

Meanwhile, K_o may be estimated as 1-sinϕ' for normally consolidated sands and (1-sinϕ') $OCR^{\sin\phi'} \leq K_p$ for overconsolidated sands (dense sands) with OCR being the overconsolidation ratio, where K_p is the coefficient of lateral earth pressure for the simple passive state $\tan^2(45+ \phi'/2)$.

The higher values of K_s in overconsolidated sands should be used very carefully.

If everything is done perfectly at site the K_s/K_o ratio may be taken as unity except if casing is permanently left in place, see the Table 1.

Table 1.0

Pile Type	K/K_o	Construction method (Bored piles)	K/K_o
Jetted piles	½ ~ 2/3	Dry construction with minimal sidewall disturbance and prompt concreting	1.0
Drilled shaft, cast-in-place	2/3 ~ 1	Slurry construction – good workmanship	1.0
		Slurry construction – poor workmanship	2/3
		Casing under water	5/6
References	(Kulhawy, 1984)	(Reese and O'Neill, 1989)	

Table 2.0

Pile Material	δ/ϕ'	Construction method (Bored piles)	δ/ϕ'
Rough concrete, cast-in-place	1.0	Open hole or temporary casing	1.0
		Slurry method – minimal slurry cake	1.0
		Slurry method – heavy slurry cake	0.8
		Permanent casing	0.7
References	(Kulhawy, 1984)	(Reese and O'Neill, 1989)	

2.2. Beta Method

The idea of the Beta method is to replace the $K_s \tan\delta$ term in skin resistance equation with a parameter that may vary with depth or used as a constant for the soil/pile type. The method can be put in this form:

$f_{sult} = \beta\sigma'_{v0}$

where σ'_{v0} is the average effective vertical stress at the middle of the segment length of the pile.

If the maximum preconsolidation stress σ'_p is known, then:

$$\beta \approx (1 - sin\phi')\left(\frac{\sigma'_p}{\sigma'_v}\right)^{sin\phi\prime} tan\phi' \quad \leq \quad K_p tan\phi'$$

The important thing in this method is to estimate an accurate value for β. For sands, the following values may be used (Source: Canadian Foundation Engineering Manual, 4th ed):

Table 3.0: Beta values recommended by CFEM 2006

Soil Type	Cast-in-Place Piles	Driven Piles
Silt	0.2 – 0.30	0.3 – 0.5
Loose sand	0.2 – 0.4	0.3 – 0.8
Medium sand	0.3 – 0.5	0.6 – 1.0
Dense sand	0.4 – 0.6	0.8 – 1.2
Gravel	0.4 – 0.7	0.8 – 1.5

Use values given for driven if driven casing is permanently left in the ground.

3. Skin Resistance in Cohesive Soils

The skin resistance of bored piles in cohesive soils may be different between the short term and the long term. The reason for this is due to the undrained behavior of clayey soils during the short period after the installation of the pile and construction of the structure. In long terms, the behavior turns into drained behavior. For short term capacity, undrained shear strength is used while in long term the effective stresses are calculated in a way like that of cohesionless soils.

3.1. The Alpha Method

In this method, the ultimate skin resistance (shaft adhesion) is calculated as:

$f_{sult} = \alpha s_u$

where s_u is the undrained shear strength of the cohesive soil.

The factor α (also called the adhesion factor) is correlated to the undrained shear strength of the soil, (Kulhawy and Jackson, 1989)

$$\alpha = 0.21 + 0.26\frac{p_a}{s_u}\ (\alpha \leq 1)$$

Where p_a is the atmospheric pressure used for normalization. This equation is good for estimating the adhesion factor in different clays. FHWA 2018, suggests the following equation:

$$\alpha = 0.30 + \frac{0.17}{\left(\frac{s_u(CIUC)}{p_a}\right)}$$

Where CIUC refers to undrained isotopically consolidated triaxial compression test. Choice of the adequate parameter must follow local experience. If no experience is available more conservative values shall be used unless a trial pile is tested.

In highly fissured clays a value of 0.3 is recommended. A rapid value of α = 0.4 to 0.45 is usually satisfactory in most cases. A graph presented in Fig. 2 may be used as well although it is more recommended for boulder clays.

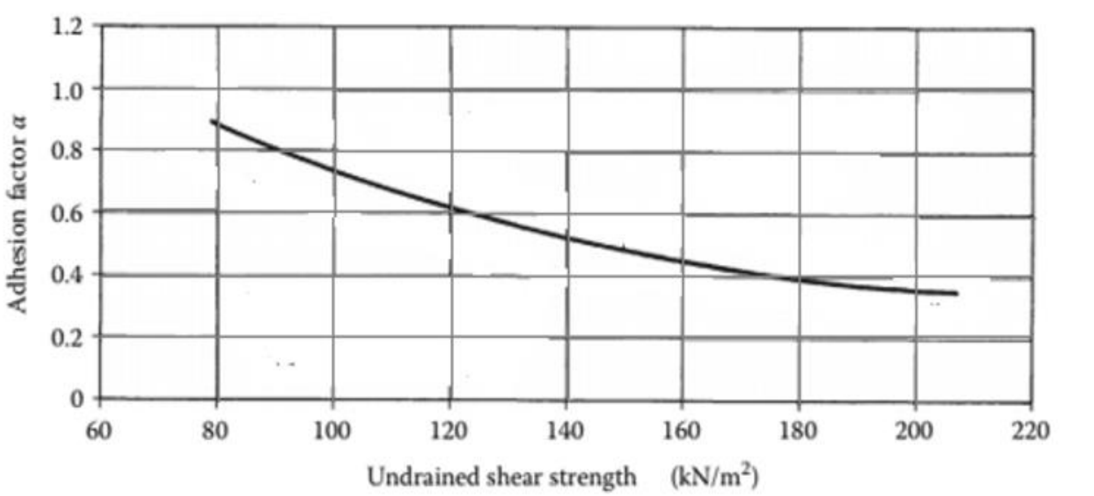

Figure 2.0: Adhesion factor vs. undrained shear strength for boulder clays and glacial tills (see Tomlinson and Woodward, 2008)

3.2. The Beta Method Again

The Beta method is widely acceptable for soft to firm clays for long term ultimate skin resistance, but the values chosen for β are different than those discussed for cohesionless soils.

In cohesive soils, β may be variable with depth and is given as:

β = 0.25 to 0.32 (recommended by CFEM)

Some authors suggested β = (1-sinϕ)tanϕ, i.e. a range of 0.3 to 0.35.

β may vary with pile embedment in the clay layer and it get affected by the overconsolidation ratio, so when stiff layer is encountered the following may be used:

$\beta = 0.4\sqrt{OCR}(L+20)/(2L+20)$ $(L\ in\ m)$ (see Guo, 2013)

4. Skin Resistance in Weak Rocks

Skin resistance of bored piles in weak rocks has been researched for a long time. The literature in the subject is tremendous. Several methods will be presented in this course which are known as widely accepted by design engineers. In all the methods for sands, clays, or rocks the skin resistance is capped by 5% of the concrete cylinder strength or 4% of the cube strength.

4.1. Williams and Pells Method

This method takes into consideration the joints of the rock mass represented by the Rock Quality Designation RQD of that rock (Williams and Pells, 1981).

$f_{sult} = \alpha.\beta.q_{uc}$

where α is an adhesion factor of the intact rock recommended in the graph in Fig. 3, and β is a reduction factor that is related to the mass continuity factor j as shown in Fig. 4. The mass continuity factor in return, is related to the number of joints in a unit distance or in other words the spacing between the joints and it can be directly estimated from the RQD of the rock, see Table 4.

The adhesion factor may be approximated by the practical fit line for the curve in Fig. 3 as (Wyllie, 1999),

$\alpha = 0.5*q_{uc}^{-0.5}$ (q_{uc} in MPa)

So, the rock may be divided into layers based on RQD and q_{uc} values and the skin resistance is calculated accordingly.

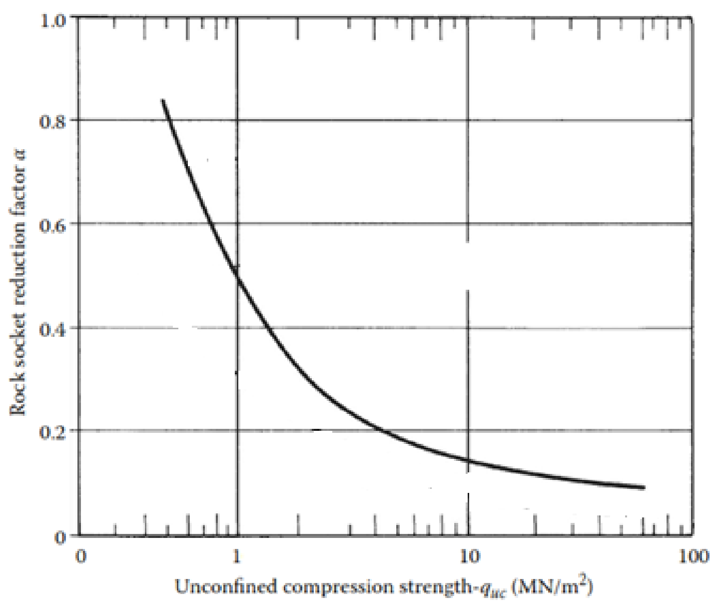

Figure 3.0: The adhesion factor of intact weak rocks (mudstone, shale, sandstone, etc.) (Williams and Pells, 1981)

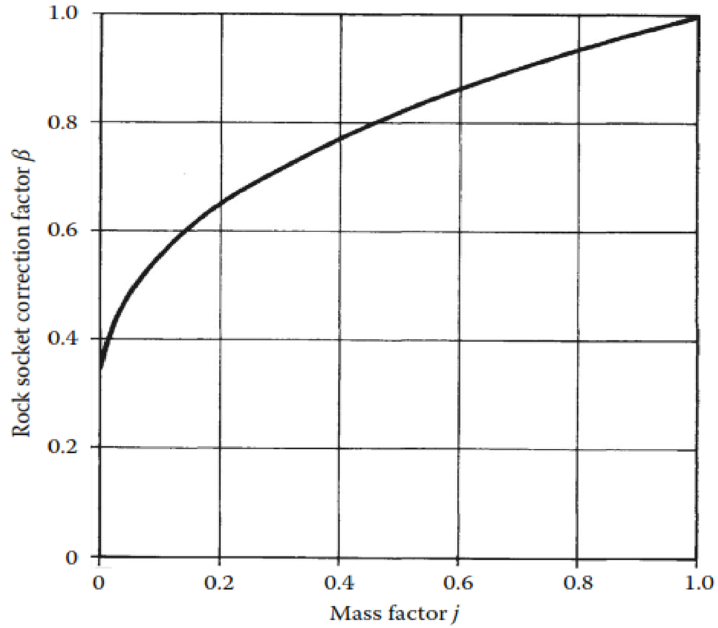

Figure 4.0: The correction factor β in Williams and Pells method

Table 4.0: Mass factor j values corresponding to RQD

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

4.2. Other Methods for Rocks and Stiff Clays using the Adhesion Factor

Table 5 lists some other equations presented by several authors (references can be found in CIRIA R181) for the skin resistance in weak rocks.

Table 5.0: Skin resistance methods from different authors

Pile design method	Formula for ultimate unit shaft resistance τ_{su}, based on uniaxial compressive strength σ_c, in MPA	$\tau_{su}/(\sigma_c/2)$
1. Rosenberg and Journeaux (1976)	$0.375(\sigma_c)^{0.515}$	$1.64[\sigma_c/(2p_a)]^{-0.485}$
2. Horvath (1978)	$0.33(\sigma_c)^{0.5}$	$1.47[\sigma_c/(2p_a)]^{-0.5}$
3. Horvath and Kenney (1979)	$0.2 - 0.25(\sigma_c)^{0.5}$	$0.89[\sigma_c/(2p_a)]^{-0.5}$ to $1.12[\sigma_c/(2p_a)]^{-0.5}$
4. Meigh and Wolski (1979)	$0.22(\sigma_c)^{0.6}$	$0.84[\sigma_c/(2p_a)]^{-0.4}$
5. Williams and Pells (1981)	$\alpha\beta(\sigma_c)$	$2\alpha\beta$
6. Rowe and Armitage (1987)	$0.45(\sigma_c)^{0.5}$	$2.01[\sigma_c/(2p_a)]^{-0.5}$

σ_c being another denotation for the unconfined strength and the reason it is shown as divided by 2 is to be able to use s_u the undrained shear strength of the clay if needed.

Kulhawy and Phoon unified the presentation of the adhesion factor for stiff clays and rocks in one equation:

$\alpha = \chi(\sigma_c/2P_a)^{-0.5}$

and $f_{sult} = \alpha\ s_u$ or $\alpha\ (\sigma_c/2)$

P_a is the atmospheric pressure to normalize the values can be used as 100kPa or equivalent.

The values chosen for χ can be found in the graphs in Fig. 5, below:

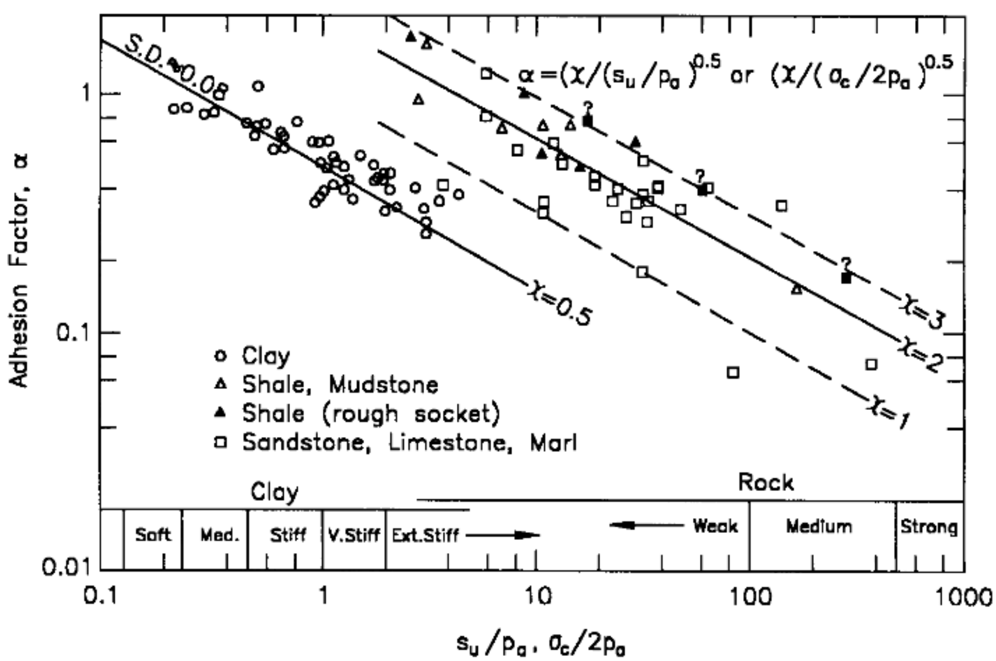

Figure 5.0: Design values of adhesion factor from unified equation of Kulhawy and Phoon, see CIRIA R181

Recommended in BS 8004:2015:

In a new version of BS 8004, issued in 2015, a suggestion for estimating the skin resistance was included. The suggested equation is:

$f_{sult} = P_a.\alpha.(q_{uc}/P_a)^k$

Where Pa is 100kPa and for soft rocks, α = 1.0 to 1.29 and k = 0.57 to 0.61

Recommended by FHWA 2018

The FHWA equation for side resistance in rocks (from O'Neill and Reese, 1999 cited in FHWA) is

$f_{sult} = 0.65\alpha_E P_a(q_{uc}/P_a)^{0.5}$

where α_E is a correction factor depends on RQD value as shown in Table 6.

Table 6.0: FHWA recommended values of α_E

RQD (%)	**Joint Modification Factor, α_E**	
	Closed joints	**Open or gouge-filled joints**
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

5. End Bearing Resistance in Cohesionless Soils

The unit end bearing resistance of a pile tipped in a cohesionless soil may be calculated from the general bearing capacity equation using only the N_q term as the N_γ term will be negligible, and the N_c term is zero (c = 0). Note that in piles of rectangular section a shape factor may apply as the following equation is for circular or squared sections.

$q_{bult} = N_q.\ \sigma'_v$

where σ'_v is the calculated effective overburden pressure at the pile toe level and N_q may be found from various literature one of most common is from Berezantzef presented by Polous and Davis 1980, see Fig. 6.

It is advised to decrease the angle of internal friction by 2 to 3 degrees before entering the graph.

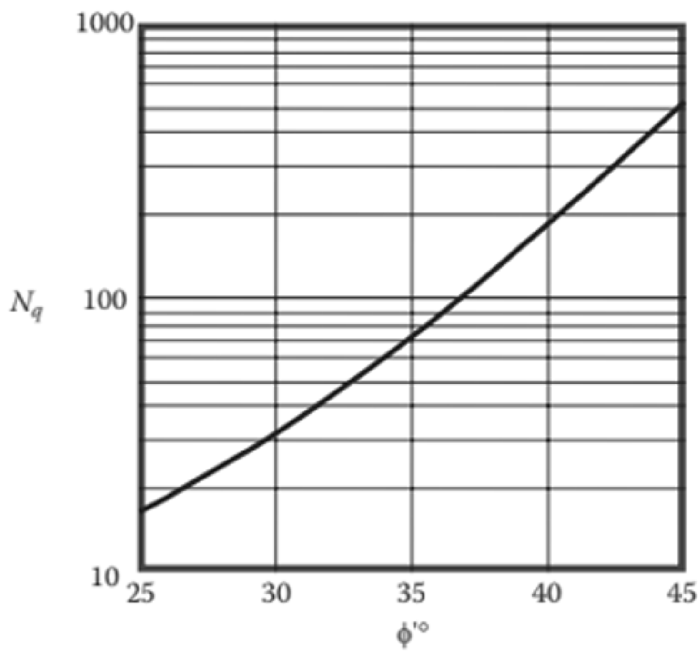

Figure 6.0: The bearing capacity factor, N_q from Berezantzef

If the friction angle is calculated from SPT test, use the equation:

$\phi' = 27.1+0.3N_{60}-0.00054[N_{60}]^2$

Where N_{60} is taken as the average number of the N values within the elevation of the specific layer where the friction angle is calculated for to two times the shortest dimension of the pile below the base of the layer. Other equations for ϕ may be used.

The Canadian Foundation Engineering Manual CFEM recommends using the values given in Table 7.

Table 7.0: Values of N_q recommended by CFEM

Soil Type	Cast-in-Place Piles	Driven Piles
Silt	10 – 30	20 – 40
Loose sand	20 – 30	30 – 80
Medium sand	30 – 60	50 – 120
Dense sand	50 – 100	100 – 120
Gravel	80 – 150	150 – 300

Limiting values are suggested for the unit end bearing resistance calculated in overconsolidated sands (dense sands) and to be used in design, see Fig. 7. **RSPile** does not calculate limits but allows the user to limit the skin friction and end bearing resistances.

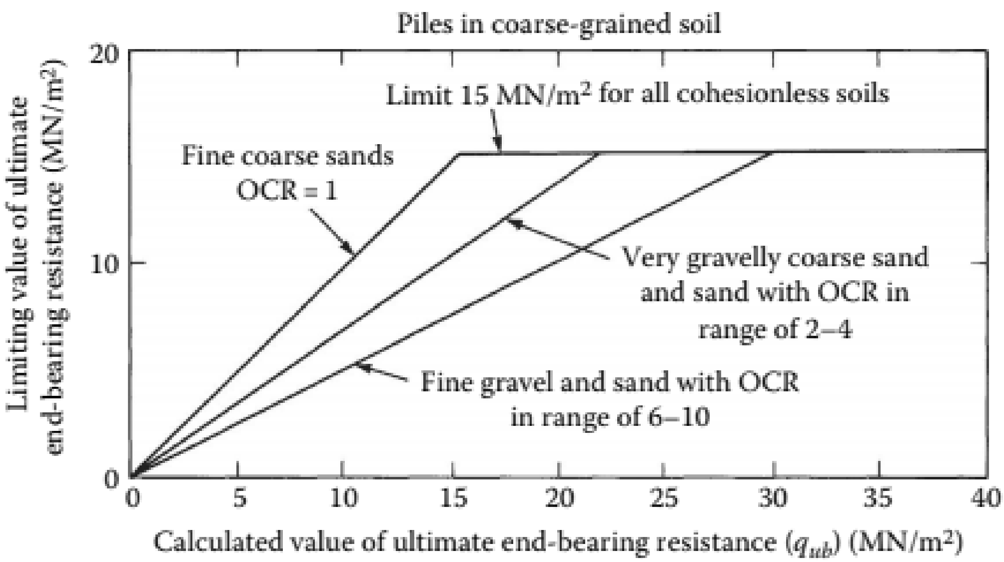

Figure 7.0: Limiting design values for the unit end bearing resistance in bored piles in dense and very dense cohesionless soils (see Tomlinson and Woodward, 2008)

6. End Bearing Resistance in Cohesive Soils

If effective stress analysis is followed the same equations used for cohesionless soils may be followed. Anyhow, most of the codes and literature recommends using the N_c term of the bearing capacity equation and ignore the rest of the terms.

$q_{bult} = N_c.\ s_u$

where s_u is the undrained shear strength of the clay below the pile tip, and N_c is the bearing capacity factor which may be found varying in literature. A general value is 9 for deep long piles compared to their cross-sectional dimensions. But this value of 9 is reduced or increased based on the literature.

CFEM recommends that a value of N_c of 9 may be used for piles having a diameter up to 500mm and to be decreased for larger diameters.

D < 500mm use $N_c = 9$

500mm < D < 1000mm use $N_c = 7$

And use 6 for larger than 1m piles.

In the recent edition of BS 8004: 2015, two factors are multiplied to correct N_c value of 9.

$N_c = 9k_1k_2$

Where k_1 is the shape factor

$$k_1 = \frac{2}{3}\left(1 + \frac{L}{6B}\right)$$

L is length of the pile embedment in the clay layer (the bearing stratum) and B is the least width of the pile or pile diameter.

While k_2 is a material factor dependent on the undrained cohesion of the clay, see Table 8.

Table 8.0: Values of k_2 (BS8004)

Undrained shear strength of soil, c_u (kPa)	**k_2**	**9 x k_2**
≤ 25	0.72	6.5
50	0.89	8
≥ 100	1.0	9

7. End Bearing Resistance in Weak Rocks

Tomlinson and Woodward, 2015, suggested the following:

Where the joints are spaced widely, that is at 600 mm or more apart, or where the joints are tightly closed and remain closed after pile driving, the base resistance may be calculated from the following equation:

$q_{bult} = 2\ N_{\varphi}\ q_{uc}$

where q_{uc} is the uniaxial compressive strength of the rock and the bearing capacity factor

$N_{\varphi} = \tan^2(45+\varphi/2)$

For (strong) sandstone, which typically has φ values between 40° and 45°, end bearing at failure is stated to be between 9 and 12 times q_{uc}. As the laboratory assessment of q_{uc} is likely to be considerably less than the in-situ strength, a reasonable characteristic value in this case would be $3q_{uc}$ to $4.5q_{uc}$.

For ultimate base resistance of bored piles terminated in weak mudstones, siltstones, and sandstones, the φ values as recommended by Wyllie are in the range of 27°–34° giving Nφ values from 2.7 to 3.4.

It should be noted that values obtained for the ultimate end bearing resistance are assuming the failure is at a settlement of around 10% to 20% of the base least width or pile diameter. For which, another method suggested by Zhang and Einstein (cited in Guo 2013) is presented as:

$q_{bult} = 15P_a(q_{uc}/P_a)^{0.5}$

For closely spaced open joints or for lower base settlement, lesser value for the ultimate end bearing should be used and may in sometimes reach only the unconfined strength itself.

Based on local experiences and field tests, the designer may choose a factor to be multiplied by the unconfined compressive strength of the rock to get an ultimate unit end bearing resistance for the pile. This will be available in **RSPile** as a user-defined method.

8. SPT Methods

Methods to calculate ultimate skin resistance and ultimate end bearing resistance from SPT N_{60} field records are old.

Generally, it can be put in the form (kN/m^2):

f_{sult} = a + bN

q_{bult} = c + dN

where a, b, c, and d are empirical factors found by correlations with field pile load tests.

CIRIA R143 presents some of these values from different publications.

Table 9 lists some cases for a and b. But CIRIA R143 recommends using zero for a and 2.0 to 2.3 for the parameter b in clays of unknown local experience and 2.7 to 3.3 in London clay.

Table 9.0: Values suggested for the a and b parameters

Soil Type	a	b	Remarks	Reference
Cohesionless	0	1.0	--	Findlay (1984) Shioi and Fukui (1982)
	0	3.3	--	Wright and Reese (1979)
Cohesive	0	5.0*	--	Shioi and Fukui (1982)
	10	3.3	piles cast under bentonite $0 \geq N \geq$ $f_s \ngtr 170\ kPa$	Decourt (1982)
Chalk	-125	12.5	$30 \geq N \geq 15$ $f_s \ngtr 250\ kPa$	Fletcher and Mizon (1984)

For c and d CIRIA R143 suggests zero for c and the following values for d:

For Sands 100,

For clays 45 to 75 with a maximum value of 100,

For weak rocks 45 to 180,

And for Chacks 225.

For obtaining suitable values for a, b, c, and d to get ultimate resistances in psf, multiple the suggested values above by 20.9.

It is recommended to use AASHTO 1998 method for SPT based pile capacity in cohesionless soils or weak rocks. The following summarizes the method:

For the ultimate base resistance (kN/m^2),

q_{bult} = 57.54 N with 75 as an upper limit to be used for N (in the new AASHTO 2007)

$f_{sult} = 2.87\ N$ for $N \leq 53$

$f_{sult} = 2.11\ (N-53)+148.7$ for $N > 53$

All in kN/m^2. To transform the above equations to get resistances in psf, multiply the equations by 20.9.

An upper limit for N may be taken as 120 to 200 depending on local experience. For low SPT N values, such as for loose sands, lower skin resistance shall be taken and the factor 2.87 may go down between 1 and 2. The user will be the one who sets his limits.

Averaging N for skin resistance is based on the average over the length segment in question while for end bearing resistance, the software calculates the average N through a depth of 2D below the tip and 1D above the tip where D is the diameter/breadth of the pile.

9. References

1. ACI 336.3R, Report on Design and Construction of Drilled Piers, 2014.
2. Tomlinson, M.J. and Woodward, J., Pile Design and Construction Practice, 5^{th} ed., 2008.
3. Geo Publication No. 1/2006, Foundation Design and Construction, Geotechnical Engineering Office, Civil Engineering and Development Department, The Government of the Hong Kong, Special Administrative Region, 2006.
4. Das, B.M., Principles of Foundation Engineering. 5^{th} ed., by Thomson Books/Cole, 2004.
5. Naval Facilities Command, Soil Mechanics Design Manual DM7, 1986.
6. BS 8004: 2015, Code of Practice for Foundations.
7. ICE Specifications for Piling and Embedded Retaining Walls, 2^{nd} ed., 2007, Thomas Telford. UK
8. BS EN 1536:2000, Execution of special geotechnical work- Bored piles.
9. Wyllie, D.C., Foundations on Rock, 2^{nd} ed., 1999.
10. Engineering and Design- Rock Foundations, EM 1110-1-2908, US Army Corps of Engineers, Nov. 1994.
11. Kulhawy, F. H. 1984. Limiting tip and side resistance: Fact or fallacy? Analysis and design of pile foundations. Proc ASCE Geotech Eng Symp, 80–98.
12. CIRIA R181, Pile Foundations in Weak Rocks. 1999.
13. CIRIA R143, The standard Penetration Test (SPT): Methods and Use. 1995.
14. **FHWA** Publication NHI 18-024, 2018, Drilled Shafts: Construction Procedures and Design Methods.
15. Canadian Foundation Engineering Manual **CFEM** 4^{th} ed. Canadian Geotechnical Society 2006.
16. Guo, W.D (2013), Theory and Practice of Pile Foundations, CRC pres.
17. Polous H.G and Davis E.H, Pile Foundation Analysis and Design, John Wiley, 1980.