PILE LOAD CAPACITY – CALCULATION METHODS

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Abstract: The article is a review of the current problems of the foundation pile capacity calculations. The article considers the main principles of pile capacity calculations presented in Eurocode 7 and other methods with adequate explanations. Two main methods are presented: α – method used to calculate the short-term load capacity of piles in cohesive soils and β – method used to calculate the long- term load capacity of piles in both cohesive and cohesionless soils. Moreover, methods based on cone CPTu result are presented as well as the pile capacity problem based on static tests.

Key words: *pile load capacity calculation, Eurocode 7, α – method and β – method, direct methods based on CPTu data*

1. INTRODUCTION

Piles can be either driven or cast in place. Pile driving is achieved by: impact dynamic forces from hydraulic and diesel hammers; vibration or jacking. Concrete and steel piles are most common. Driven piles which tend to displace a large amount of soil due to the driving process are called full-displacement piles. Cast-in-place (or bored) piles do not cause any soil displacement, therefore, they are non-displacement piles.

Piles may be loaded axially and/or transversely. The limit states necessary to be considered in the design of piles are the following (EN-1997-1, §7.2.(1)P):

- Bearing resistance failure of the pile foundation,
- Insufficient compression resistance of the pile (Fig. 1a),
- Uplift or insufficient tensile resistance of the pile (Fig. 1d),
- Failure in the ground due to transverse loading (Fig. 1f),
- Structural failure of the pile in compression (Fig. 1b), tension (Fig. 1e), bending (Fig. 1g), buckling (Fig. 1c) or shear (Fig. 1h),
- Combined failure in the ground, in the pile foundation and in the structure,
- Excessive settlement, heave or lateral movement,
- Loss of overall stability,
- Unacceptable vibrations.

The pile load capacity on compression (Fig. 1a–1c) is considered in the article, in particular the sufficient compressing resistance case (Fig. 1a).
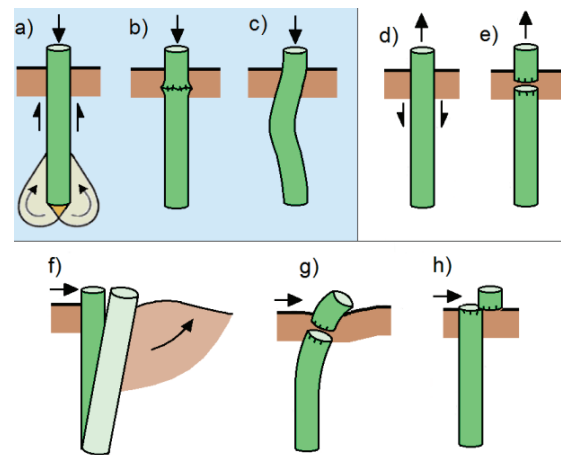

Fig. 1. Piles load capacity:
(a)–(c) on compression, (d), e) on tension,
(f)–(h) on transverse loading

Figure 2a shows the following main parameters used in the pile capacity problem:

- $s(Q)$ – load-settlement top pile data recorded in the in-situ test on compression,
- s_k – characteristic settlement, generally calculated using the assumption on soil behavior as: semi-infinite elastic, isotropic and homogenous area (Boussinesq theory), which gives such larger settlement than the measured one,

- $R_{c;d}$ – design resistance as the capacity parameters determined from designing standards, considered in the present article,
- R_{test} – in-situ static test result on top pile,
- Q_{lim} – limit resistance defined as rapid settlement occurs under sustained or slight increase of the applied load – the pile plunges.

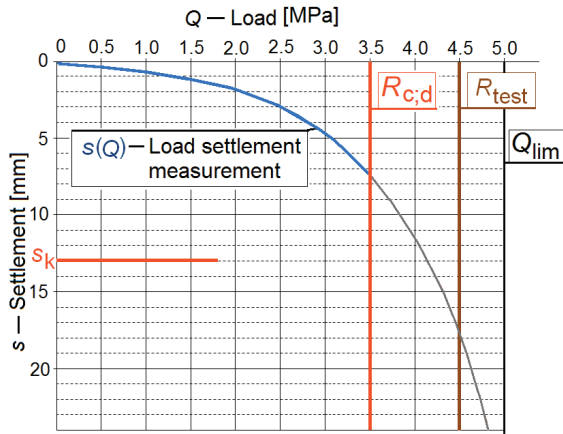Ration of $Q_{lim}/R_{c;d} = \gamma_t$ presents the total safety factor.

Fig. 2a. Capacity parameters:
$R_{c;d}$ – design resistance, R_{test} – static test result,
Q_{lim} – limit resistance, $s(Q)$ – load-settlement curve
from top pile measurement s_k – characteristic settlement

Figure 2b shows typical load/settlement curves for compressive load of the shaft Q_s and the base Q_b load capacity and the total load capacity Q_t characteristic depending on soil layers: (a) for friction pile and (b) for end-bearing pile.
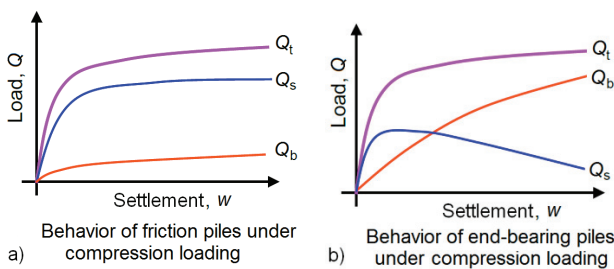

Fig. 2b. Typical load/settlement curves for compressive load tests:
(a) friction pile; (b) end-bearing pile

2. APPROACHES OF PILE DESIGN ACC. TO EN-1997

EN 1997-1 §7.4(1)P states that the design of piles shall be based on one of the following approaches:

(1) The results of static load tests, which have been demonstrated by means of calculations or otherwise, to be consistent with other relevant experience,
(2) Empirical or analytical calculation methods whose validity has been demonstrated by static load tests in comparable situations,
(3) The results of dynamic load tests whose validity has been demonstrated by static load tests in comparable situations,
(4) The observed performance of a comparable piles foundation, provided that this approach is supported by the results of site investigation and ground testing.

Equilibrium equation

The equilibrium equation to be satisfied in the ultimate limit state design of axially loaded piles in compression is

$$F_{c;d} \leq R_{c;d} , \qquad (1)$$

where $F_{c;d}$ is the design axial compression load and $R_{c;d}$ is the pile compressive design resistance.

Design axial load

The design axial compressive load $F_{c;d}$ is obtained by multiplying the representative permanent and variable loads, G and Q by the corresponding partial action factors γ_G and γ_Q

$$F_{c;d} = \gamma_G G_{rep} + \gamma_Q Q_{rep} . \qquad (2)$$

The two sets of recommended partial factors on actions and the effects of actions are provided in Table A3 of Annex A of EN 1997-1.

Characteristic pile resistance

Eurocode 7 describes three procedures for obtaining the characteristic compressive resistance $R_{c,k}$ of a pile:
(a) Directly from static pile load tests with coefficient ξ_1 and ξ_2 for n pile load tests, given in Table A.9 of EN 1997-1 Annex A,
(b) By calculation from profiles of ground test results or by calculation from ground parameters with coefficient given in Table A.10 of EN 1997-1 Annex A,
(c) Directly from dynamic pile load tests with coefficientgiven in Table A.11 of EN 1997-1 Annex A.

In the case of procedures (a) and (b) Eurocode 7 provides correlation factors to convert the measured pile resistances or pile resistances calculated from profiles of test results into characteristic resistances.

Case (c) is referred to as the *alternative procedure* in the Note to EN 1997-1 §7.6.2.3(8), even though it is the most common method in some countries.

Characteristic pile resistance from profiles of ground test results

Part 2 of EN 1997 includes the following Annexes with methods to calculate the compressive resistance, $R_{c,cal}$ of a single pile from profiles of ground test results:

(a) D.6 Example to determine $R_{c;cal}$ based on cone penetration resistance: relating the pile's unit base resistances q_b at different normalised pile settlements, s/D, and the shaft resistance q_s to average cone penetration resistance q_c values. The values in Tables D.3 and D.4 are used to calculate the pile base and shaft resistances in the pile.

(b) D.7 Example to determine $R_{c;cal}$ base on maximum base resistance and shaft resistance from the q_c values obtained from an electrical CPT.

(c) E.3 Example to determine $R_{c;cal}$ based on results of an MPM test.

Characteristic total pile compressive resistance $R_{c;k}$ or the base and shaft resistances $R_{b;k}$ and $R_{s;k}$ may be determined directly by applying correlation factors ξ_3 and ξ_4 to the set of pile resistances calculated from the test profiles. This procedure is referred to as the Model Pile procedure by Frank et al. (2004) to determine $R_{c;cal}$.

Characteristic pile resistance from the ground parameters

The characteristic base and shaft resistances may also be determined directly from the ground parameters using the following equations given in EN 1997-1 §7.6.2.3(8)

$$R_{b;k} = A_b \cdot q_{b;k} \qquad (3)$$

$$R_{s;k} = \sum A_{s;i} \cdot q_{s;i;k} \qquad (4)$$

where

$q_{b;k}$ – characteristics of unit base resistance,

$q_{s;i;k}$ – characteristics of unit shaft resistance in the *i*-th layer.

Design compressive pile resistance

The design compressive resistance of a pile $R_{c;d}$ may be obtained either by treating the pile resistance as a total resistance

$$R_{c;d} = \frac{R_{c;k}}{\gamma_t} \qquad (5)$$

or by separating it into base and shaft components $R_{b;d}$ and $R_{s;k}$, using the relevant partial factors, γ_b and γ_s

$$R_{c;d} = \frac{R_{b;k}}{\gamma_b} + \frac{R_{s;k}}{\gamma_s} . \qquad (6)$$

The combinations of sets of partial factor values that should be used for Design Approach 2 are as follows

DA2.C1: A1 "+" M1 "+" R2

where R2 for base, shaft and total: $\gamma_t = \gamma_b = \gamma_s = 1.1$ in case of compression, and $\gamma_t = 1.15$ in case of shaft in tension.

3. DRAINED AND UNDRAINED LOADING CONDITIONS

Drained loading occurs when soils are loaded slowly, resulting in slightexcess pore pressures that dissipate due to permeability.On the other hand, undrained loading occurs when fine-grained soils are loaded at a high rate, they generate excess pore pressures because these soils have very low permeabilities.

The drained (or long-term) strength parameters of a soil, c' and ϕ' must be used in drained (long-term) analysis of piles.

The undrained (or short-term) strength parameter of a soil, c_u, must be used in undrained (short-term) analysis of piles.

4. ESTIMATING LOAD CAPACITY OF PILES

Pile load carrying capacity depends on various factors, including: (1) pile characteristics such as pile length, cross section, and shape; (2) soil configuration and short and long-term soil properties; and (3) pile installation method. Two widely used methods for pile design will be described:

- α – method used to calculate the short-term load capacity (total stress) of piles in cohesive soils,
- β – method used to calculate the long-term load capacity (effective stress) of piles in both cohesive and cohesionless soils.

Piles resist applied loads through side friction (shaft or skin friction) and end bearing as indicated in Fig. 3. Friction piles resist a significant portion of their loads by the interface friction developed be-

tween their surface and the surrounding soils. On the other hand, end-bearing piles rely on the bearing capacity of the soil underlying their bases. Usually, end-bearing piles are used to transfer most of their loads to a stronger stratum that exists at a reasonable depth.

Design bearing capacity (resistance) can be defined as

$$R_{c,d} = Q_b + Q_s = A_b \cdot q_b + \sum A_{s,i} \cdot q_{s;i;d} . \qquad (7)$$
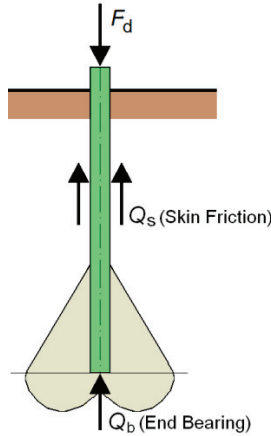

Fig. 3. Pile's side friction (shaftor skin friction)
and end bearing

5. α-METHOD, SHORT-TERM LOAD CAPACITY FOR COHESIVE SOIL

5.1. UNIT SKIN FRICTION $q_s(z)$

The method is based on the undrained shear strength of cohesive soils; thus, it is well suited for short-term pile load capacity calculations. In this method, the skin friction is assumed to be proportional to the undrained shear strength s_u, of the cohesive soil as follows and the interface shear stress q_s between the pile surface and the surrounding soil is determined as

$$q_s(z) = \alpha(z)s_u(z) \qquad (8)$$

where

s_u – undrained shear strength,

α – adhesion coefficient depending on pile material and clay type.

It is usually assumed that ultimate skin friction is independent of the effective stress and depth. In reality, the skin friction is dependent on the effective

stress and undrained shear strength but decreases for long piles.

Niazi and Mayne [24] presented 25 methods of estimating pile unit shaft resistance within α-method and compared them. They showed main differences with respect to parameters: length effect, stress history, I_p, s_u, σ'_v, progressive failure, plugging effect. Belowthe main methods estimating skin friction in claysare shown:

(a) American Petroleum Institute (API, 1984, 1987)

The equation by API (1984, 1987) suggests values for α as a function of c_u as follows

$$\alpha = \begin{cases} 1 - \dfrac{c_u - 25}{90} & \text{for } 25\,\text{kPa} < c_u < 70\,\text{kPa}, \\ 1.0 & \text{for } c_u \leq 25\,\text{kPa}, \\ 0.5 & \text{for } c_u \geq 70\,\text{kPa}. \end{cases} \qquad (9)$$

(b) NAVFAC DM 7.2 (1984). Proposition for α coefficient depends on type of pile (Table 1)

Table 1. α vs. undrained shear strength (NAVFAC DM 7.2)

Pile type	Soil consistency	Undrained shear strength s_u [kPa]	α
Timber and concrete piles	Very soft	0–12	1.00
	Soft	12–24	1.00–0.96
	Medium stiff	24–48	0.96–0.75
	Stiff	48–96	0.75–0.48
	Very stiff	96–192	0.48–0.33
Steel piles	Very soft	0–12	1.00
	Soft	12–24	1.00–0.92
	Medium stiff	24–48	0.92–0.70
	Stiff	48–96	0.70–0.36
	Very stiff	96–192	0.36–0.19

As in the API method, effective stress effects are neglected in the DM 7.2 method.

(c) Equation based on undrained shear strength and effective vertical stress, Kolk and Van der Velde method [18]. Coefficient α is based on the ratio of undrained shear strength and effective stress. A large database of pile skin friction results was analyzed and correlated to obtain α value (Table 2).

(d) Simple rules to obtain coefficient α based on s_u/σ'_v proposed standard DNV-OS-J101-2007

$$\alpha = \begin{cases} \dfrac{1}{2\sqrt{s_u/\sigma'_v}} & \text{or } s_u/\sigma'_v \leq 1, \\ \dfrac{1}{2^4\sqrt{s_u/\sigma'_v}} & \text{or } s_u/\sigma'_v > 1. \end{cases} \qquad (10)$$

Table 2. Skin friction factor dependent on s_u/σ_v'

s_u/σ_v'	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
α	0.95	0.77	0.70	0.65	0.62	0.60	0.56	0.55	0.53	0.52	0.50	0.49	0.48
s_u/σ_v'	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5	3.0	4.0
α	0.47	0.42	0.41	0.41	0.42	0.41	0.41	0.40	0.40	0.40	0.40	0.39	0.39

(e) Mechanism controlling friction fatigue, Randolph [26]

Randolph [26] suggested that progressive failure, which occurs in strain softening soil, was a possible mechanism controlling friction fatigue. The progressive failure from the peak (τ_{peak}) to the residual (τ_{res}) shaft resistance is shown in Fig. 4. Randolph [26] proposed a reduction factor (R_f) which depends on the degree of softening ξ and the pile compressibility K

$$R_f = 1 - (1 - \xi)\left(1 - \frac{1}{2\sqrt{K}}\right)^2 \qquad (11)$$

where

$$\xi = \frac{\tau_{\text{res}}}{\tau_{\text{peak}}}, \quad K = \frac{\pi D L^2 \dfrac{\tau_{\text{peak}}}{(EA)_{\text{pile}}}}{\Delta w_{\text{res}}}, \qquad (12)$$

EA – axial stiffnes of pile,

Δw_{res} – post-peak displacement required to mobilize the residual shaft resistance.

(f) Norwegian Geotechnical Institute, NGI-05

Karlsrud et al. [15], proposed modification of the NGI method by introducing correlation of s_{ud}/σ_{v0}' and I_p with α – coefficient presented by the trend lines shown in Fig. 4. New data are included herein, all previous data have been re-interpreted.
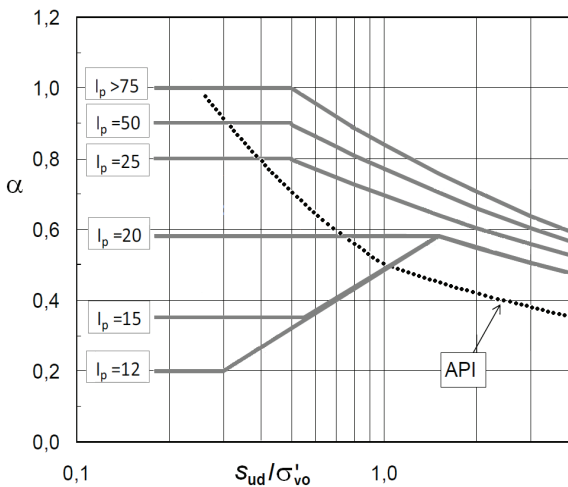

Fig. 4. Measured values of α in relation to normalized strength, for all piles (Karlsrud et al. [15])

Figure 4 presents mobilized values of α versus s_{ud}/σ_{v0}' for all piles discussed in this paper. Studies have shown that the plasticity index has a largeimpact on the mobilized ultimate shaft friction and corresponding α-values.

5.2. UNIT BASE RESISTANCE q_B

For cohesive soils it can be shown, using Terzaghi's bearing capacity equation, that the unit base resistance of the pile is

$$q_b = (s_u)_b N_c \qquad (13)$$

where $(s_u)_b$ is the undrained shear strength of the cohesive soil under the base of the pile, and N_c is the bearing capacity coefficient that can be assumed equal to 9.0 (Skempton [29]).

6. β-METHOD, LONG-TERM LOAD CAPACITY FOR COHESIVE AND COHESIONLESS SOILS

6.1. UNIT SKIN RESISTANCE $q_s(z)$

The method is based on effective stress analysis and is suited for long-term (drained) analyses of pile load capacity. The unit skin resistance q_s, between the pile and the surrounding soil is calculated by multiplying the friction factor, μ, between the pile and soil by σ_h'

$$q_s(z) = \mu \sigma_h' = \mu(z) K(z) \sigma_v'(z) = \beta(z) \sigma_v'(z) \quad (14)$$

where at rest pressure coefficient depends on the installation mode, usually $K = K_0$, with $K_0 = (1 - \sin\phi')(\text{OCR})^{0.5} \le 3$,

OCR – overconsolidation ratio,

σ_v' – vertical effective stress.

Niazi and Mayne [24] presented 15 methods of estimating pile unit shaft resistance within β-method and compared them. They showed main differences

between them with respect to parameters: σ_r', δ, ϕ', OCR, K, σ_v', L, d, s_u, I_D, I_p. The main methods estimating skin frictionare shown below:

(a) according to NAVFAC DM 7.2(1984), $\beta = \mu(z)K(z) = \tan\delta(z)K(z)$, Tables 3 and 4.

Table 3. Pile skin friction angle (δ)

Pile type	Pile-soil interface friction angle (δ)
Steel piles	$20°$
Timber piles	$3/4\ \phi'$
Concrete piles	$3/4\ \phi'$

Table 4. Lateral earth pressure coefficient (K)

Pile type	K (piles under compression)	K (piles under tension)
Driven H-piles	0.5–1.0	0.3–0.5
Driven displacement piles (round and square)	1.0–1.5	0.6–1.0
Driven displacement tapered piles	1.5–2.0	1.0–1.3
Driven jetted piles	0.4–0.9	0.3–0.6
Bored piles (less than 60 cm in diameter)	0.7	0.4

(b) proposition value of $\beta = \mu(z)K(z)$ can be estimated according to the following propositions:

Author	Proposition of β value
McClelland [21] for driven piles	$\beta = 0.15$ to 0.35 for compression, $\beta = 0.10$ to 0.25 for tension (for uplift piles)
Meyerhof [22]	$\beta = 0.15,\ 0.75,\ 1.2$ for $\phi' = 28°,\ 35°,\ 37°$, for driven piles $\beta = 0.1,\ 0.2,\ 0.35$ for $\phi' = 33°,\ 35°,\ 37°$, for bored piles
Kraft and Lyons [19]	$\beta = C\tan(\phi' - 5)$ $C = 0.7$ for compression, $C = 0.5$ for tension (uplift piles)

(c) Average K method

Earth pressure coefficient K can be averaged from K_a, K_p and K_0: $K = (K_0 + K_a\,K_p)/3$ where: $K_0 = (1 - \sin\phi')$, $K_a = \tan^2(45 - \phi'/2)$, $K_p = \tan 2(45 + \phi'/2)$

(d) Karlsrud [16]

Karlsrud [16] proposed to take into account the plasticity index I_p in β-method. Figure 5 shows diagram of β-values from as low as 0.045 for low-plastic NC clays to about 2.0 to very stiff clays with OCR of 40, which is the upper range of available pile data.
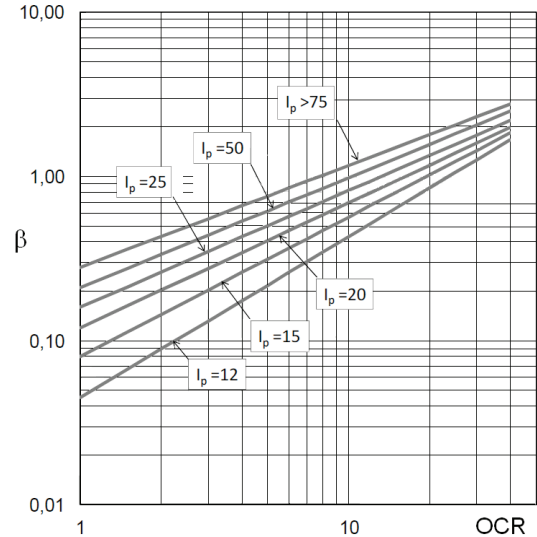

Fig. 5. Chart for determination of β-values dependent on OCR and I_p, Karlsrud [16]

6.2. UNIT BASE RESISTANCE q_b

Using Terzaghi's bearing capacity equation, the unit base resistance at the base of the pile can be calculated

$$q_b = (\sigma_v')_b N_q + c_b' N_c \tag{15}$$

where

$(\sigma_v')_b$ – vertical effective stress at the base of the pile,

c_b' – cohesion of the soil under the base of the pile,

$N_c = (N_q - 1)\cot\phi'$.

Values of bearing capacity factor N_q

(a) Janbu [13] presented equations to estimate capacity coefficients N_q and N_c for various soils
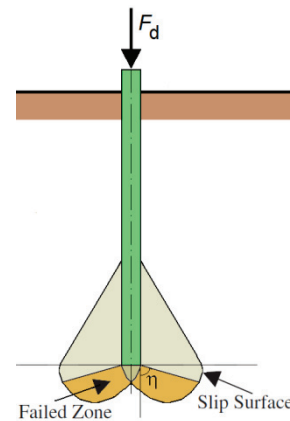

Fig. 6. Shear surface around the base of a pile: definition of the angle η (Janbu [13])

$$N_q (\tan \phi' + \sqrt{1 + \tan^2 \phi'})^2 \exp(2\eta \tan \phi')$$

where η is an angle defining the shape of the shear surface around the tip of a pile as shown in Fig. 6. The angle η ranges from $\pi/3$ for soft clays to 0.58π for dense sands.

(b) Values of bearing capacity factor N_q according to NAVFAC DM 7.2(1984), see Table 5.

friction angle decreases with depth. Hence N_q, which is a function of the friction angle, also would reduce with depth. Variation of other parameters with depth has not been researched thoroughly. The end bearing capacity does not increase at the same rate as the increasing depth. Figure 7 attempts to formulate the end bearing capacity of a pile with regard to relative density (I_D) and vertical effective stress σ'_v (Randolph et al. [17]).

Table 5. Friction angle ϕ' vs. N_q

ϕ' [°]	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q for driven piles	10	15	21	24	29	35	42	50	62	77	86	120	145
N_q for bored piles	5	8	10	12	14	17	21	25	30	38	43	60	72

If water jetting is used, ϕ' should be limited to 28°. This is because water jets tend to loosen the soil. Hence, higher friction angle values are not warranted.

6.3. PARAMETERS THAT AFFECT THE END BEARING CAPACITY

The following parameters affect the end bearing capacity:
(c) Effective stress at pile tip,
(d) Friction angle at pile tip and below (ϕ'),
(e) The dilation angle of soil (ψ),
(f) Shear modulus (G),
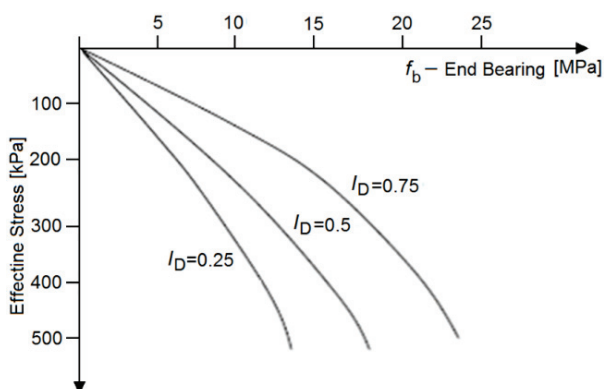(g) Poisson's ratio (v).

Fig. 7. End bearing capacity of a pile
with regard to relative density (I_D) and effective stress
(Randolph et al. [27])

Most of these parameters have been bundled into the bearing capacity factor N_q. It is known that the

6.4. CRITICAL DEPTH FOR SKIN FRICTION (SANDY SOILS)

Skin friction should increase with depth and it becomes a constant at a certain depth. This depth was named a critical depth. The typical experimental variation of skin friction with depth in a pile as evidence for critical depth is shown in Fig. 8.
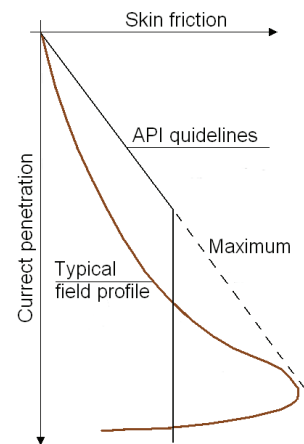

Fig. 8. Variation of skin friction (Randolph et al. [27])

Remarks:
- As one can see, experimental data do not support the old theory with a constant skin friction below the critical depth.
- Skin friction tends to increase with depth and just above the tip of the pile to attain its maximum value. Skin friction would drop rapidly after that.
- Skin friction does not increase linearly with depth as was once believed.
- No satisfactory theory exists at present to explain the field data.

- Due to lack of a better theory, engineers are still using critical depth theory of the past.

Reasons for limiting skin friction

The following reasons have been offered to explain why skin friction does not increase with depth indefinitely, as suggested by the skin friction equation:
1. K value is a function of the soil friction angle (ϕ'). Friction angle tends to decrease with depth. Hence, K value decreases with depth (Kulhawy [20]).
2. Skin friction equation does not hold true at high stress levels due to readjustment of sand particles.
3. Reduction of local shaft friction with increasing pile depth, see Fig. 9 (Rajapakse [28]).
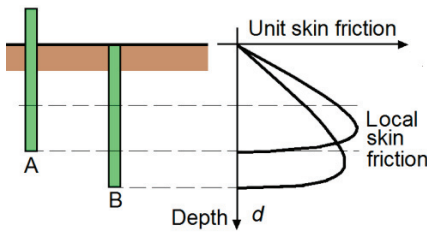

Fig. 9. Example of unit skin friction distribution
(Rajapakse [28])

Let us assume that a pile was driven to a depth of 3 m and unit skin friction was measured at a depth of 1.5 m. Then let us assume that the pile was driven further to a depth of 4.5 m and unit skin friction was measured at the same depth of 1.5 m. It has been reported that unit skin friction at 1.5 m is less in the second case.

NAVFAC DM 7.2 gives maximum value of skin friction and end bearing capacity is achieved after 20 diameters within the bearing zone. The following approximations were assumed for the critical depth:
- Critical depth for loose sand = 10 D (D is the pile diameter or the width),
- Critical depth for medium dense sand = 15 D,
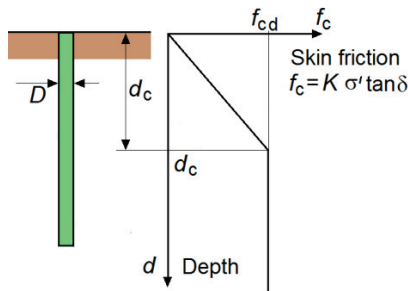- Critical depth for dense sand = 20 D.

Fig. 10. Critical depth: d_c – critical depth, f_{cd} – unit skin friction
at critical depth, $f_{cd} = K\ \sigma'_c \tan\delta$,
σ'_c – effective stress at critical depth

6.5. CRITICAL DEPTH FOR END BEARING CAPACITY (SANDY SOILS)

Pile end bearing capacity in sandy soils is related to effective stress. Experimental data indicate that end bearing capacity does not increase with depth indefinitely. Due to lack of a valid theory, engineers use the same critical depth concept adopted for skin friction as for the end bearing capacity. As shown in Fig. 11, the end bearing capacity was assumed to increase till the critical depth. It is clear that there is a connection between end bearing capacity and skin friction since the same soil properties act in both cases, such as effective stress, friction angle, and relative density. On the other hand, two processes are vastly different in nature.
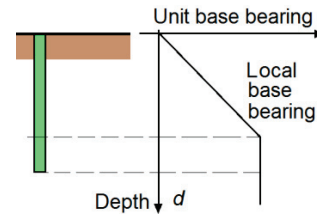

Fig. 11. Unit base bearing capacity and critical depth

The following approximations were assumed for the critical depth within the bearing zone:
- Critical depth for loose sand = 10 D (D is the pile diameter or the width),
- Critical depth for medium dense sand = 15 D,
- Critical depth for dense sand = 20 D.

The critical depth concept is a gross approximation that cannot be supported by experimental evidence.

7. ESTIMATING PILE LOAD CAPACITY BASED ON CPT RESULTS

7.1. INTRODUCTION

Owing to the difficulties and the uncertainties in assessing the pile capacity on the basis of the soil strength-deformation characteristics, the most frequently followed design practice is to refer to the formulae correlating directly the pile capacity components of q_b and q_s to the results of the prevalent in situ tests. Within the domain of these in situ methods, the cone penetration test (CPT) is one of the most frequently used investigation tools for pile load capacity evaluations. Ever since the first use of CPT in geo-

technical investigations, research efforts have advanced the very elementary idea of considering it as mini-pile foundation. This has resulted in plethora of correlative relationships being developed between the CPT readings cone resistance (q_c) or more proper corrected cone resistance (q_t), sleeve friction (f_s), and shoulder pore water pressure (u_2) and the pile capacity components of q_b and q_s.

As commonly reported (e.g., Ardalan et al. [2]; Cai et al. [5], [6]), there are two main approaches to accomplish axial pile capacity analysis from CPT data: (a) *rational* (or *indirect*) *methods* and (b) *direct methods*.

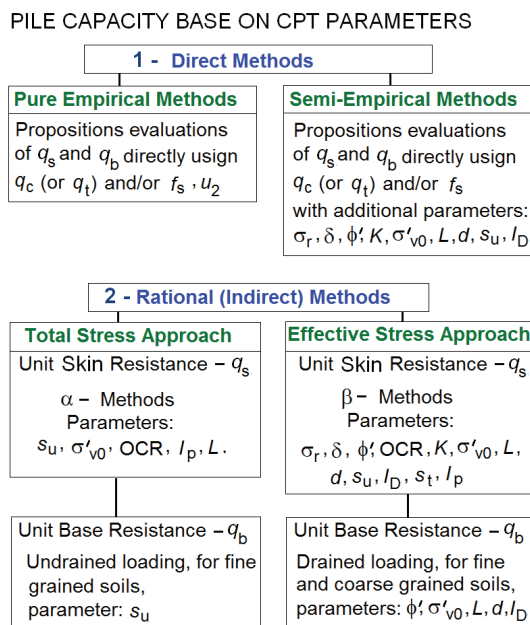PILE CAPACITY BASE ON CPT PARAMETERS

Fig. 12. CPT based evaluations of pile capacity
(Niaziand Mayne [24])

- *Direct CPT methods* – used the similarity of the cone resistance with the pile unit resistances. Some methods may use the cone sleeve friction in determining unit shaft resistance. Several methods modify the resistance values to consider the difference in diameter between the pile and the cone. The influence of mean effective stress, soil compressibility, and rigidity affect the pile and the cone in equal measure, which eliminates the need to supplement the field data with laboratory testing and to calculate intermediate values, such as K, and N_q.
- *Pure empirical methods* – initial formulations were based solely on cone resistance (q_c) derived from mechanical cone penetrometers. Subsequently, with the introduction of the electrical cone penetrometer, the additional channels measuring sleeve friction (f_s), and porewater pressures (u_1 and u_2) were considered.

- *Semi-empirical methods* – with the purely CPT parameters, the additional estimated parameters are taken into account (σ_r, δ, ϕ', K, σ'_{v0}, L, d, s_u, I_D).
- *Indirect CPT methods* – employ soil parameters, such as friction angle and undrained shear strength obtained from cone data to estimate bearing capacity. The indirect methods apply strip-footing bearing capacity theories, and neglect soil compressibility and strain softening. These methods are rarely used in engineering practice.

7.2. DIRECT CPT METHODS

In one viewpoint, the cone penetrometer can be considered as a mini-pile foundation as noted by Ardalan et al. [2] and Eslami and Fellenius (1997). The mean effective stress, compressibility and rigidity of the surrounding soil medium affect the pile and the cone work in a similar manner.This concepthas led to the development of many direct CPT methods.
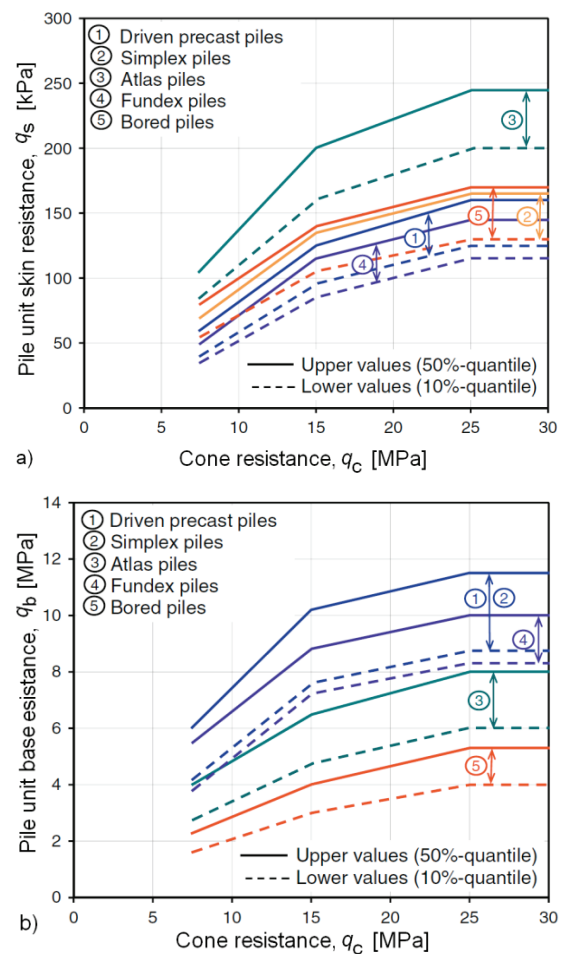

Fig. 13. Upper and lower empirical values
of different piles in coarse grained soils for:
(a) $q_s(z)$; (b) q_b (after Kempfert and Becker [17])

Unit skin resistance $q_s(z)$ and unit base resistance q_b

Based on the load test database up to 1000 load tests on precast concrete, cast-in-place concrete, steel pipe, screw cast-in-place, and micro piles etc., Kempfert and Becker [17] developed correlations for pile $q_s(z)$ and q_b from CPT q_c and s_u. Their results, presented in the form of empirically derived charts with upper and lower bound estimates of $q_s(z)$ and q_b (Fig. 13), have been integrated into the national German recommendations for piles.

Partial embedment reduction factor

White and Bolton [31] studied, the causes of low values of q_b/q_c in sand in contrast with $q_b = q_c$ for steady deep penetration (e.g., cavity expansion solutions and strain path method). They examined a database of 29 load tests on a variety of CE piles (steel pipe piles, Franki piles with enlarged base, and precast square, cylindrical and octagonal concrete piles) and CPT q_c data. The low value of q_b/q_c, which forms basis of the apparent scale effect on the diameter, can be attributed topartial embedment in the underlying hard layer (Fig. 14), whereas, partial mobilization was explained by defining failure according to a plunging criterion. They concluded that any reduction of q_c when estimating q_b of CE piles in sand should be linked to the above factors rather than pile diameter.
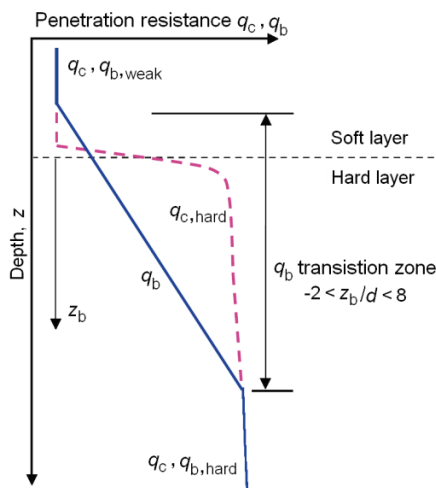

Fig. 14. Partial embedment reduction factor on q_b
(after White and Bolton [31])

8. PILE LOAD CAPACITY DETERMINED FROM STATIC LOAD TESTING

The pile bearing capacity is necessary to verify with the assumption of the design. The most com-
mon practice is by means of a static loading test. The capacity is the total ultimate soil resistance of the pile determined from the measured load-settlement behavior. It can be defined as the load for which rapid settlement occurs under sustained or slight increase of the applied load – the pile plunges. This definition is inadequate, however, because large settlement is required for a pile to plunge and is not obtained in the test. Therefore, the pile capacity or ultimate load must be determined by some definition based on the load-settlement data recorded in the test.

Load-displacement curves obtained from axial load tests on pile foundations exhibit differing shapes and resulting conclusions. There is only a single value of load termed "capacity" that is selected from the entire curve for design purposes. Yet, there are at least 45 different criteria available for defining the "axial capacity" (Hirany and Kulhawy [8]). An example of a load test conducted on a 0.76 m diameter, 16.9 m long drilled shaft installed at Georgia Institute of Technology is shown in Fig. 15.
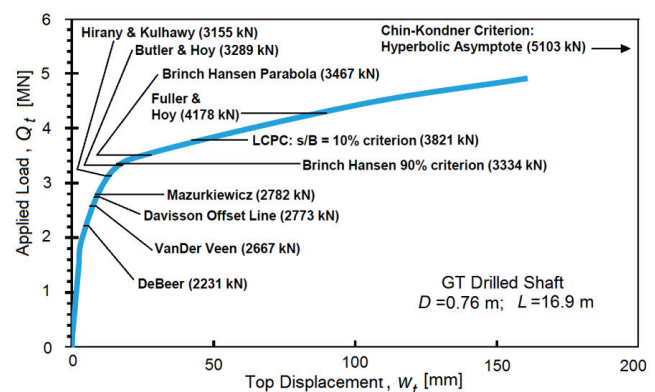

Fig. 15. Comparison of capacity interpretation criteria
from axial pile load tests (Hirany and Kulhawy [8])

When interpreting loading tests, the failure condition can be interpreted in several different ways. Tomlinson [30] lists some of the recognized criteria and list disadvantages and advantages of pile tests in general. The main interpreted failure loads correspond to settlements equal to $0.1d$, where d is the equivalent pile diameter referring to an equivalent circle diameter for square and hexagonal piles. Such definition does not consider the elastic shortening of the pile, which can be substantial for long piles, while it is negligible for short piles. In reality, settlement relates to a movement of superstructure (pile with soil), and it does not relate to the capacity as a soil response to the loads applied to the pile in a static loading test.

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