Contribution of the standard penetration test SPT to the design of pile foundations in sand– Practical recommendations

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ABSTRACT

The objective of this article is to present original research work results on the contribution of the standard penetration test (SPT) to the design and analysis of pile foundations, on the basis of the interpretation of a database of full-scale pile loading tests undertaken in sandy soils, with the aim to provide practical recommendations to the designers. The article encompasses three parts, the first one presents a detailed comparative study of 10 currently used methods of design on the basis of the SPT, assessed according to their predictive capability of the axial bearing capacity, within a database consisting of 46 axial pile loading tests carried out in 27 sites in the United Arab Emirates. The second part presents some experimental findings related to the critical (or creep) vertical load, whereas the third part highlights the concept of the reference pile settlement and its application to the limit state design of pile foundations.

Keywords: Full-scale test; Load-transfer curves; Settlement; Pile; SPT test; Sand.

1. INTRODUCTION

Ultimate limit state design (ULSD) of pile foundations requires the verification of the pile bearing capacity, which may be carried out as prescribed by the modern geotechnical standards by geotechnical calculation based on the pile/soil properties, or by a full-scale pile loading test. This test is often used to estimate the pile bearing capacity Q_l, the critical load (also called the creep load) Q_c, as well as the pile settlement under working loads. However, this pragmatic approach of design avoiding all the uncertainties related to any theoretical model of the pile/soil interaction, is relatively expensive and usually undertaken within the scope of an important project of pile foundations. The test pile may be fully instrumented to separately measure the pile base resistance Q_p as well as the skin friction load Q_s, the pile bearing capacity Q_l being the sum of Q_p and Q_s.

However, in most of the cases the test pile is simply instrumented, and the test is limited to estimating Q_l by using some design criteria, the direct measure being practically impossible. There is a multitude of criteria leading sometimes to non-negligible scatter in predicting Q_l (Hirany and Kulhawy, 1988). A possible explanation of this usual discrepancy is the empirical nature of many of these criteria, established on the basis of an observational work of the pile tests involving a local geotechnical context and a specific experimental procedure. One of the aims of this paper is to contribute to compare between these criteria according to a ranking procedure with respect to the experimental values of the pile bearing capacity in sand.

The second aim of this paper is to use the best ranked criterion as a reference of comparison between 10 currently used SPT based-methods of bearing capacity, which led to ranking them according to their predictive capability within a database of 46 axial pile loading tests carried out in 27 sandy soils.

2. BRIEF LITERATURE REVIEW

As previously mentioned, a vertical loading test on a simply instrumented pile cannot directly measure the pile bearing capacity Q_l. The load-settlement curve is nonetheless interpreted via a design criterion to estimate Q_l. The most commonly used criteria are summarized in table 1and may be classified either into theoretical (1 to 4), or empirical (5 to 11), or conventional criteria (12).

The pile bearing capacity Q_l is the sum of the limit load Q_p at the pile base (or end bearing capacity) and the limit load Q_s due to skin friction stresses. For a circular, uniform shaped pile:

$$Q_l = Q_p + Q_s = q_l \frac{\pi.B^2}{4} + \pi.B \int_0^D q_s(z).dz \qquad (1)$$

B, D, q_l and $q_s(z)$ are respectively the pile diameter, the pile embedded length, the end bearing resistance and the limit skin friction stress at the depth z along the pile.

The usual SPT-based methods of the pile bearing capacity may be classified with regard to their nature into two categories. The first one contains the empirical methods derived from the interpretation of databases of pile loading tests in correlation with the N_{spt} value. According to these methods, the end bearing resistance is assumed to be proportional to an equivalent value noted by N_{spt}^e around the pile base in such a way as:

$$q_l = K_s.N_{spt}^e \qquad (2)$$

The limit skin friction stress at a given depth z is proportional to the $N_{spt}(z)$ value at this depth:

$$q_s(z) = n_s.N_{spt}(z) \qquad (3)$$

K_s and n_s have the unit of stress and are respectively called the "tip resistance factor" and the "skin friction factor". All the methods 1 to 9 in table 1 are empirical.

The second category encompasses semi-empirical methods based on a theoretical model of the end bearing capacity as well as of the limit skin friction, calibrated by some geotechnical correlations. To this category belongs the Hansen-Burland's method (Bouafia and Derbala, 2002b).

3. DESCRIPTION OF THE DATABASE

A database was built from axial loading tests carried out in sites located in the United Arab Emirates, with a size of 46 static loading tests in 27 sites. As shown in Figure 1, the sites studied are located in the coastal cities of the Arabian Gulf, and cover the main capitals of the Emirates, particularly Dubai, Sharjah, Ajman, and Um-Al-Quwain (Bouafia & Derbala, 2002a).

Figure 1. Location of the experimental sites (denoted by black dots) (Bouafia, 2004)

The Arabian Gulf was subject to an important process of carbonate sedimentation, and the soil deposits in the UAE are mostly Pleistocene or recent in age. Shallow layer of the soil is composed of fine-grained to medium-grained, slightly silty to silty sand with traces of gravel, gypsum, shell fragments and cemented sand. It is classified as Sw, Sp or Sm sand according to the USCS classification system. Groundwater is always encountered in these sites, at depths up to 6.50 m below the ground level. Site investigations are often carried out by using the SPT test, with a typical configuration consisting of a hammer with a mass of 63.5 kg and a fall height of 0.76 m. The length of the drive shoe is characterized by outer and inner diameters of 75 and 35 mm respectively. In figure 2 is illustrated an example of a borehole log showing the typical lithology of the sites studied.

Test piles are cast in-situ reinforced concrete piles, bored with either the casing method or continuous flying auger method. Sometimes, bentonite mud is used to maintain the pile borehole. Margins of the pile diameter B and the slenderness ratio (Length/Diameter) are respectively 0.45-1.10 m and 10-36.7, where the concrete compressive strength usually varies between 20 and 40 MPa.

Table 1. Compilation of some usual design criteria to estimate Q_l (Bouafia et al, 2022)

N°	Criterion	Procedure
1	Hyperbolic fitting	Curve $Q=f(v_0)$ to be fitted by the hyperbolic function: $Q = \frac{v_0}{a + bv_0}$ Q: vertical load v_0: Pile top settlement $Q_l=1/b$ and $K_{v0}=1/a$: Pile vertical stiffness
2	Exponential fitting (Van-Der-Veen criterion)	Curve $Q=f(v_0)$ to be fitted by the exponential function: $Q = Q_l\left(1 - \exp\left(-\frac{K_{v0}}{Q_l} v_0\right)\right)$ Q_l : Bearing capacity and K_{v0} : Pile vertical stiffness
3	Zero secant stiffness (Decourt's criterion)	Draw the curve of the secant stiffness $K_v=Q/v_0$ as function of Q Last points fluctuating around a line to be fitted by a straight line Q_l is the abscissa of the intercept of this line with the axis of Q
4	Parabola-Rectangle fitting	Curve $Q=f(v_0)$ to be fitted by the PARECT function: $Q = 2Q_l \frac{v_0}{v_0^R}\left(1 - \frac{1}{2}\frac{v_0}{v_0^R}\right)$ Q_l : Bearing capacity and v_0^R : Reference settlement of the pile
5	Tangente-1 (Butler & Hoy Criterion)	Draw the initial tangent of the curve $Q=f(v_0)$ Draw the tangent whose slope is 7 kN/mm Q_l is the intercept of these tangents
6	80% criterion (Hansen's criterion)	Draw the curve $v_o^{1/2}/Q = f(v_o)$ The linear shape of this curve is to be fitted by a straight line: $a+b.v_0$ $Q_l=0.5/(a.b)^{1/2}$
7	90% criterion (Hansen's criterion)	Q_l is the load causing a settlement double of that corresponding to a 90% of this load
		(Use hereafter the units mm and kN) Draw the curve $Q=f(v_o)$ and the line of the elastic pile compression whose

8	Davisson's offset limit (Davisson's criterion)	slope is $\pi E_p B^2/4/L$ Draw a straight line parallel to the first one and passing by the point (3.75+B/120, 0) Q_l is the intercept of the curve Q-v_o with this line
9	Tangente-2 (Fuller & Hoy criterion)	Draw a straight line whose slope is 7 kN/mm. This line is to be shifted horizontally till it will be tangent to the curve $Q=f(v_0)$ Q_l is the ordinate of this point of tangent
10	Logarithmic criterion (De-Beer Criterion)	Draw the curve $Log_{10}(Q)$ as function of $Log_{10}(v_o)$ First points fluctuating around a line to be fitted by a straight line Last points fluctuating around a line to be fitted by a straight line Q_l is the intercept of these lines
11	Limit settlement (FDOT criterion)	Draw the curve $Q=f(v_o)$ and the line of the elastic pile compression whose slope is $\pi E_p B^2/4/L$ Draw a straight line parallel to the first one and shifted horizontally by B/30 (B in mm) Q_l is the intercept of the curve $Q=f(v_o)$ with this line
12	Conventional criterion	Q_l corresponds to a settlement v_0 equal to B/10

Table 2. Summary of the different SPT-based methods of computation of Q_l

No.	References	K_s (kPa)	n_s (kPa)	N_{spt}^e and zone of influence	Pile installation
1	Bazarra & Kurkur (1986)	135 if B≤0.5 m 270xB else (B in m)	0.67 if B≤0.5 m 1.34xB else (B in m)	Average within [D-3.75xB, D+B]	bored
2	Décourt (1982)	400 in sand 250 in residual silty sand	$q_s=10x(N_{spt}/3+1)$ (in kPa)		bored
3	Lopes & Laprovitera (1988)	98.4 in sand 87.0 in silty sand	1.62 in sand 1.94 in silty sand	Average within [D-B, D+B]	bored
4	Meyerhof (1976) CFEM (2006)	120 400	1 2	-Average within [D-8B, D+3B] -Correction due to depth effect	bored driven
5	Shioi and Fukui (1982)	100 300	1 2		bored driven
6	Aoki & Velloso (1975)	286 in sand 228 in silty sand	2.00 in sand 2.28 in silty sand	Average of the three N_{spt} values closest to pile base	bored
7	PHRI Standard (1980)	400	2	- average of N_1 and N_2 -correction due to ground water effect	driven
8	Reese et O'Neill (1989)	60 if B=0.52-1.27 76/B if B>1.27 m (B in m)	3.3		bored
9	Robert (1997)	115 190	1.90 1.90	-correction due to depth effect	bored driven
10	Hansen-Burland (1973)	$q_l= N_q. \sigma_v'(D)$ D: Pile embedded length N_q = Hansen's bearing capacity factor $N_q =f(\varphi)$ φ derived from φ-N_{spt} chart $q_s = K_0.\sigma_v'.tg\delta$ (Burland's β formula) $K_0 =(1-\sin\varphi).(OCR)^{1/2}$ (OCR=1) intermediate roughness of pile shaft: $\delta= 0.75x\varphi$		-Average within [D-B/2, D+2B] - correction due to ground water effect	

Test piles are usually simply instrumented at the top with 4 dial gauges, with usual accuracy of 1/100 mm for the settlement reading. The load is applied in increments by a hydraulic jack and a pump assembly fitted with a pressure gauge, against a weighted platform.

The experimental program consists of a series of increments of vertical load in two cycles, the duration of each increment being controlled by a required rate of settlement less than 0.25 mm/hour. The loading magnitude of the first cycle is equal to the design load (or nominal load), whereas in the second cycle the magnitude reaches 1.5 to 2 times the design load (ASTM, 1994).

GEOCONSULT
Borehole Log

Contract	: 2725, Plot No. 676, Al Majaz, Sharjah
Borehole No	: 1 Sheet (1)
Ground Level	:
Borehole Dia	: 150mm
Type of Boring	: Percussion Drilling
Date	: 2/4/97

Bore-hole Depth	Samples		S.P.T.				Core Recov %	RQD %	Description of Strata	Layer Thk.	Legend
	No	Type	150mm	150mm	150mm	'N'					
0.0									Existing Ground Surface		
	1	D									
-0.5											
-1.0	2		5	4	4	8			Loose brown to grey fine to medium slightly SILTY SAND with shell fragments	2.0	
-1.5											
-2.0	3		3	4	6	10					
-2.5											
-3.0	4		2	4	5	9			Loose grey fine to medium SILTY SAND with shell fragments	2.0	
-3.5											
-4.0	5		13	17	19	36					
-4.5											
-5.0	6		13	21	30	51					
-5.5											
-6.0	7		18	29	40	69					
-6.5									Dense to very dense grey fine SILTY SAND with cemented fragments	4.5	
-7.0	8		32	>50 for 6"							
-7.5											
-8.0	9		36	>50 for 6"							
-8.5											
-9.0	10		15	11	16	27					
-9.5									Medium dense to dense grey very SILTY SAND	1.5	
-10.0	11		17	19	23	42					

Contd. on Sheet (2)

Remarks:

Ground Water Level: 1.26m

Sample/Key
S.P.T. Standard Penetration Test
U Undisturbed Sample
D Disturbed Sample
C Core Sample

Figure 2. Typical Borehole Log and SPT values profile

4. COMPARATIVE STUDY AND RANKING OF THE METHODS

4.1. Selection of the reference bearing capacity

Comparison of predicted values of the pile bearing capacity Q_l by the SPT-based methods summarized in table 2 should be made with respect to a "reference value" derived from the interpretation of the load-settlement curve of the test pile by using a design criterion.

Almeida and Liu (2019) presented a comparative study of two CPT-Based methods and conventionally adopted the criterion of Butler and Hoy as a reference value (criterion 5 in table 1). Stuedlein et al (2014) used a database of 36 loading tests on augered cast-in-place (ACIP) piles in cohesionless soils, and conventionally adopted the Davisson's criterion (criterion 8 in table 1) as a reference value. Duzceer and Saglamer (2002) conducted a comparative study of several design criteria by using a database of 14 driven and 10 bored piles in different soil conditions in Turkey, Saudi Arabia and Kazakhstan, and conventionally adopted the reference value as that predicted by the FHWA method.

In order to select the most reliable design criterion among those formulated in table 1, it was decided to undertake an assessment of these criteria applied to a small sized worldwide database of vertical loading tests on fully instrumented piles in sand, the Q_l being directly measured. The experimental piles were instrumented by different techniques of measurement of the axial load distribution along the pile, as well as the vertical pressure at the pile base.

Table 3 summarizes the main features of the pile/soil configurations used to select the design criterion. Pile loading tests were carried out on 10 instrumented piles and installed through different techniques into 8 sites composed of sandy soils. It was found that the empirical criteria, namely those numbered from 5 to 11 in table 1, are in most cases not applicable, which is confirmed by previous studies (Bouafia et al, 2022, Miad and Mahious, 2021). These empirical criteria are based in graphical procedures, which are not applicable in several cases due to margins of the experimental values obtained during the pile loading test.

The selective study was then focused on the theoretical criteria (1 through 4) and the conventional criterion (12). The "bearing capacity ratio" λ is defined as follows:

$$\lambda = \frac{Q_l^{crit}}{Q_l^{exp}} \qquad (4)$$

where Q_l^{crit} and Q_l^{exp} are respectively the predicted bearing capacity according to a given criterion from the table 1 and the measured one from the loading test. Each criterion was ranked with respect to three features: the accuracy, the precision, and the "quality of prediction" which are respectively quantified by: the arithmetic average λ_m, the standard deviation SD_λ, and the relative frequency of values between 0.8 and 1.2 denoted hereafter by QP. Within the margin of 0.9-1.2 of λ_m, 0.13-0.26 of SD_λ, and 50-90% of QP for the different criteria, it was found that the PARECT criterion has the best overall rank (λ_m =0.91, SD_λ =0.13 and QP=90%) and will hereafter be used as a reference value of Q_l denoted hereafter by Q_l^{PAR}.

Table 3. Main features of the pile/soil configurations (Bouafia, 2022)

Site	Location	Diameter (m)	D/B	Pile material	Installation
S_1	Loon Beach (France)	0.42	19.0	Reinforced concrete	bored
S_2	Pigeon river (USA)	0.36	23.1	Steel closed ended	driven
S_3	Pigeon river (USA)	0.36	23.1	Steel open ended	driven
S_4	North Osaka bay (Japan)	1.50	29.3	Not precised	Not precised
S_5	Sao-Carlos (Brazil)	0.35	30.3	Reinforced concrete	bored
		0.40	26.5	Reinforced concrete	bored
		0.50	21.2	Reinforced concrete	bored
S_6	Univ. Texas (USA)	0.273	33.5	Steel closed ended	driven
S_7	Montevilliers (France)	0.41	36.6	Precast reinforced concrete	screwed
S_8	Surra South (Kuwait)	0.10	22.5	Reinforced concrete	bored

4.2. Comparative study of the methods of bearing capacity

Methods of pile bearing capacity summarized in table 2 were applied to each test pile. Since in most cases, the location of piles is not specified with respect to the SPT boreholes, it was decided to carry out the calculation in each borehole which takes into account the spatial variability of the soil properties, involving so 107 cases to be computed by each method.

Automated computational procedure was then carried out by writing a Fortran computer program called VERBEAR (**VER**tical **BEAR**ing capacity) (Bouafia & Derbala, 2002b).

It is to be noted the method 7 of table 2 is not applicable because it is limited to the driven piles whereas all the piles studied are bored.

Comparison between the predictions of the pile bearing capacity was carried out on the basis of the "bearing capacity ratio" λ defined as follows:

$$\lambda = \frac{Q_l^{Pred}}{Q_l^{PAR}} \quad (5)$$

where Q_l^{pred} and Q_l^{par} are respectively the predicted bearing capacity according to a given method and that estimated by the PARECT criterion. The statistical analysis of this parameter considered as a random variable, with a size of 107 values for each method, led to constructing histograms which were fitted by the Gauss's probability function and characterized by the mean value μ and the standard deviation σ (See Figure 3).

The procedure of ranking the studied methods on the basis of their predictive capability was based on three criteria: the accuracy, the precision, and the quality of prediction respectively quantified by μ, σ, and QP. The last criterion is the defined as the probability that λ is within 0.8 and 1.2, and involves some tolerance about the under- and over- predictions of the bearing capacity. An ideally accurate method has μ=1, whereas an ideally precise method has σ=0. Moreover, an ideal quality of prediction is characterized by QP=100%.

As summarized in table 4, the methods of bearing capacity differ notably with respect to their accuracy, precision, and the quality of prediction. Shioi & Fukui's method was remarkably found the most accurate, the most precise, and having the highest quality of prediction. It is therefore the best ranked and may be recommended as a simple and practical approach to estimate the pile bearing capacity based on the SPT test. The formulas of Q_p and Q_s may easily be computed using respectively equations (2) and (3) by using the values of K_s and n_s recommended by Shioi-Fukui and compiled in table 2.

5. ANALYSIS OF THE CRITICAL LOAD

The concept of bearing capacity is somewhat ambiguous, because the mobilization threshold of the bearing capacity, in terms of pile displacement, is not specified. Moreover, it depends significantly on a multitude of parameters, particularly the loading rate.

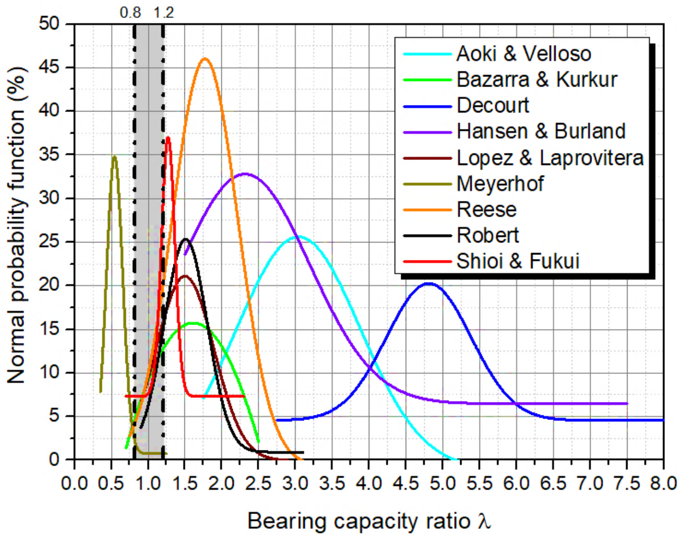

Figure 3. Gauss's probability density functions of the methods

Among the recent concepts postulated by experimental studies of the bearing capacity of piles, is that of the "critical load" (or creep load), the load-settlement curve being characterized by an initial linear portion up to a value called the critical force Q_c. This force corresponds also to the threshold of pile settlement instability in time. Consequently, exceeding Q_c leads to a considerable increase in settlement rate and settlements are no longer stabilized in time.

Figure 4 illustrates a typical experimental diagram of stabilization of the pile settlements during a vertical load test, where Q_c conventionally corresponds to the abscissa of the intercept of two straight lines, showing a significant increase in settlement rate beyond the critical load.

The concept of critical load is inspired by the "creep pressure" measured in the pressuremeter test (PMT), beyond which the radial displacements of the PMT borehole are no longer proportional to the applied pressure and diverge towards failure.

Table 4. Ranking of the methods according to accuracy, precision and quality of prediction

Method	μ	Rank of accuracy	σ	Rank of precision	QP (%)	Rank of quality of prediction	Sum of ranks	Overall ranking
Shioi	1.27	1	0.09	1	21.6	1	3	1st
Lopez-Laprovitera	1.50	3	0.39	4	18.5	2	9	2nd
Robert	1.51	4	0.29	3	13.7	3	10	3rd
Meyerhof	0.54	2	0.11	2	0.0	8	12	4th
Reese	1.77	6	0.45	5	8.7	4	15	5th
Bazarra	1.61	5	2.22	9	7.0	5	19	6th
Hansen-Burland	2.31	7	0.88	8	6.0	6	21	7th
Aoki	3.04	8	0.84	7	1.0	7	22	8th
Décourt	4.81	9	0.57	6	0.0	8	23	9th

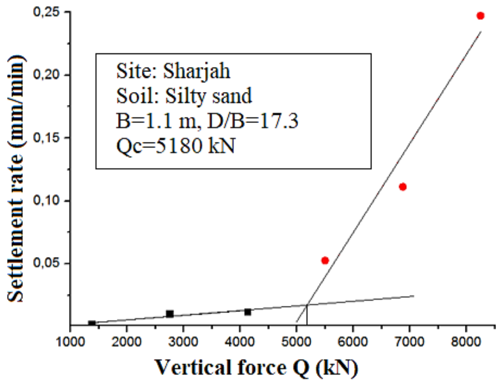

Figure 4. Typical diagram of stabilization of pile settlements

As illustrated by figure 5, Q_c is remarkably proportional to the bearing capacity predicted by the PARECT criterion adopted to interpret the experimental load-settlement curves, and corresponds to about 77% the pile bearing capacity.

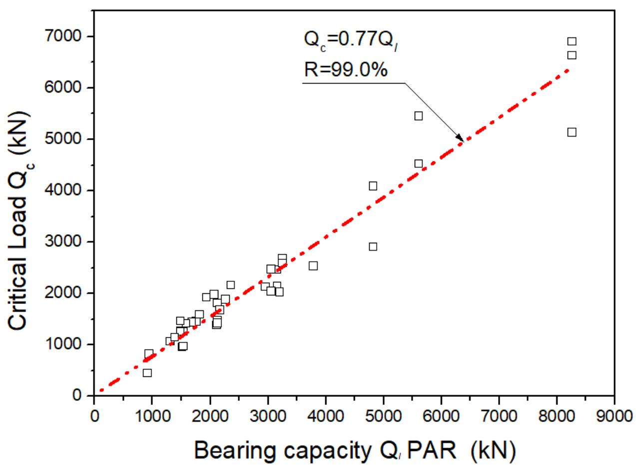

Figure 5. Variation of Q_c vs Q_l^{PAR}

It is interesting to highlight a fundamental feature of the PARECT function, as described in table 1 and illustrated in figure 6. The elastic-perfectly plastic function is characterized by an initial linear portion up to a settlement v_0^c corresponding to Q_l/K_{v0}, which is equal to half the reference settlement v_0^R, corresponding to the bearing capacity. Replacing v_0 by v_0^c into the equation of PARECT gives Q=0.75Q_l which corresponds practically to Q_c. Consequently, the critical load Q_c may be considered as the limit of the linear load-settlement response of the pile. The critical force Q_c is a key parameter used in the serviceability limit state (SLS) design, where the working loads should be limited to a fraction of Q_c in order to keep the pile working within the domain of linear behaviour.

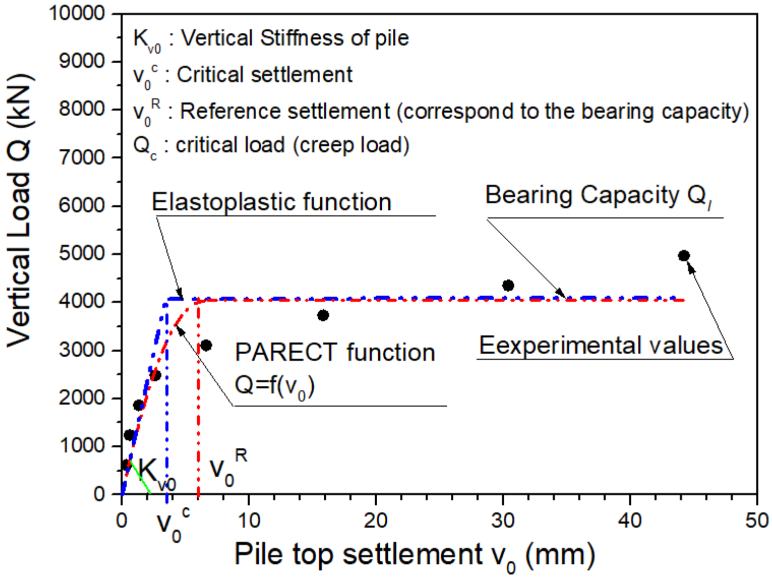

Figure 6. General scheme of the PARECT function

In the case of unavailability of the pile loading test, the critical load may be estimated as a linear combination of the limit loads Q_p and Q_s as follows:

$$Q_c = \alpha Q_p + \beta Q_s \tag{6}$$

According to the French standard NF P94-262 accompanying the Eurocode 7, coefficients α and β depend on the mode of pile installation. When using the French methods based on the PMT or the CPT tests, this standard prescribes that α=β=0.7 for driven or jacked piles and α=0.5 and β=0.7 for bored piles (Bouafia, 2017).

A multiple linear regression analysis was undertaken by using the least squares technique on the basis of equation (6) in order to formulate Q_c as bilinear function of Q_p and Q_s computed by Shioi and Fukui's method applied to the UAE test piles, which led to the values α=0.24 and β=1.41 with a regression coefficient R equal to 97.2% indicating an excellent quality of fitting (See figure 7). Consequently, in absence of the pile loading test, equation (6) offers a simple formula to estimate Q_c by using the values of Q_p and Q_s already computed by Shioi and Fukui's method.

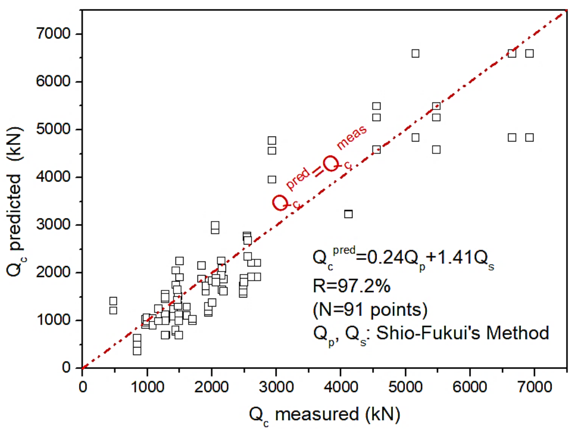

Figure 7. Comparison of predicted and measured values of Q_c

6. ANALYSIS OF THE REFERENCE SETTLEMENT

The PARECT function is formulated as follows:

$$Q = 2Q_l \frac{v_0}{v_0^R}\left(1 - \frac{1}{2}\frac{v_0}{v_0^R}\right) \quad (7)$$

Where Q and v_0 are respectively the vertical applied load and the corresponding pile top settlement. As shown in figure 5, the reference settlement v_0^R is defined as the threshold of full mobilization of the bearing capacity Q_l. According to equation (7), it is given by:

$$v_0^R = 2\,\frac{Q_l}{K_{v0}} \quad (8)$$

where K_{v0} is the initial vertical pile stiffness. Statistical analysis of the normalized reference settlement v_0^R/B led to a mean value of 0.61% and a 95% confidence interval of this mean value of 0.39-0.83%, which is very small compared to the conventional value of 10% as prescribed by the conventional criterion of bearing capacity. Moreover, as it was already mentioned, since the critical settlement v_0^c is half the value de v_0^R, it may then be characterized by a mean value of 0.3%. This value may be considered as an admissible settlement for pile foundations because it corresponds to the critical load Q_c according to PARECT function. For comparison purposes, a similar study carried out on a database of 67 pile loading tests located in Algeria, showed that v_0^R/B of reinforced concrete piles bored in a variety of soil conditions is characterized by a mean value of 0.92% whereas the 95% confidence interval of this mean value is 0.43-1.40%, indicating the same order of magnitude (Bouafia et al, 2022).

Based on these considerations, it is possible to suggest a schematic normalized load-settlement of reinforced concrete bored piles in sandy sands, as depicted in figure 8, where it can be noticed three domains: Domain A corresponding to the linear pile response up to an admissible settlement of 0.3% of B, Domain B within which a non-linear elastoplastic response is exhibited up to 0.6% of B, and domain C where an ultimate limit state (ULS) is reached by exceedance of the pile bearing capacity. This latter is recommended to be computed by the Shioi and Fukui's method.

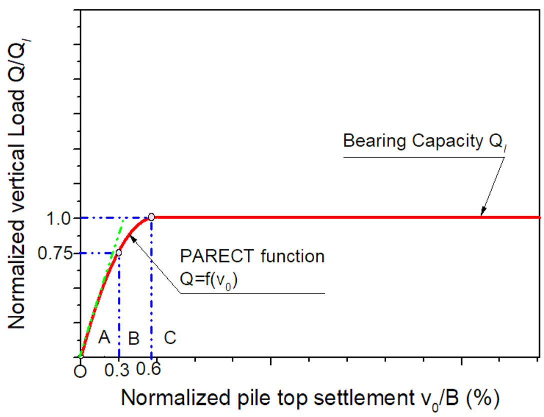

Figure 8. Normalized load-settlement curve in sandy soils

7. CONCLUDING REMARKS

Based on a detailed interpretation of the experimental data of axial pile loading tests carried out in the United Arab Emirates, this paper highlights some practical aspects of the pile design under axial loads in correlation with the standard penetration test (SPT) data.

Comparative study of 10 currently used methods of pile bearing capacity based on the SPT test was carried out by evaluating their predictive capability within a database consisting of 46 axial pile loading tests carried out in 27 sites in the United Arab Emirates. The reference bearing capacity was that given by the PARECT criterion, which was best ranked among 12 usual design criteria. The Shioi and Fukui's method was the best ranked with respect to the accuracy, the precision, and the quality of prediction and then recommended as a practical tool of estimation of the bearing capacity of reinforced concrete bored piles in sandy soils.

The second part of the paper focused on the concept of critical load used in the serviceability limit states (SLS) pile design and showed it may be estimated by 77% of the bearing capacity. Moreover, a simple formula was proposed to estimate the critical load as function of the limit loads Q_p and Q_s computed by Shioi and Fukui's method.

In the last part, the critical and reference settlements were empirically evaluated respectively as 0.3% and 0.6 % of the pile diameter, the first one being considered as the admissible pile settlement whereas the second one being the threshold of full mobilization of the bearing capacity. A normalized load-settlement curve was then proposed as a practical chart to describe the different domains of the pile response under an axial load.

8. ACKNOWLEDGEMENTS

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