Pile Capacity Prediction For In Situ Tests

G.Togliani
Senior Geologist Consultant, Switzerland

ABSTRACT: The limited budget generally available for jobs of small size do not allow the traditional check of the pile capacity with static loading tests. A design method based on cone penetration tests (i.e. CPT, CPTU, SCPT), which gave good results for many years, has been used recently for several Pile Prediction Events [Orlando (2002), Merville (2003), Porto (2004)]. The static loading tests realized during these events, have shown that a good agreement exist between the predictions and the results of such tests.

1 INTRODUCTION

During is professional career, the author, initially as Managing Director of a company specialized in site investigations (borings and in situ tests) and in the execution of both drilled and driven piles and later as Geotechnical Consultant, has been engaged in prevalence on job of small size (50 to 150 piles). In these context, especially while working with private companies, the available budget for the preliminary investigations and the deep foundations is very often limited and almost never sufficient to allow static loading tests (SLT) to be performed. Obviously a conservative over design of the piles must however be avoided and, in the same time, the piles must work satisfactorily. Thus the choice of the design method of the ultimate resistance of the piles is of fundamental importance. The design method proposed, is based on the results of cone penetration tests (CPT), combining in particular the method elaborated by Bustamante & Gianeselli (shaft and toe capacity, 1982) and the method suggested by Gambini (taper capacity, 1986) which were fit in a single formula updated in function of the gained experiences. With reference to the calculation of the allowable load that, in absence of static loading tests, represent, according to the author, the crucial point of every design since it determines the necessary numbers of piles, the used safety factor is 2 for the ultimate shaft resistance and 3 for the ultimate toe resistance (this difference is explained with the fact that the toe resistance is entirely activated only after noticeable movements). In twenty five years of professional experience, hundred of kilometer of piles (most were driven concrete prefabricated cylindrical or cone shaped piles shorter than 25 m), have been successfully designed with this method, but having very rare chances to verify with static loading tests, what was the degree of accuracy of the predictions. In these cases the only possibility of verification remains the measure of the final sets per blow and the use of driving formulae [Gates (Poulos & Davis,1980) for the author] to estimate the pile capacity, method that can always be questioned. For this reason the author, when invited, has taken part with enthusiasm in several "Pile Prediction Events", namely: Orlando (2002), Merville (2003) and Porto (2004), even if the soils and the type of piles were different from his experience and practice.

2 REFERENCE EQUATIONS

The following formulae (units kN, m) are concerning the predicted capacity of driven displacement piles:

Shaft : $R_S = \Sigma[(\pi d_{aver.} h_i q_s)]$ (1)

Taper: $R_C = \Sigma[\pi/4(d^2_{top} - d^2_{bottom}) q_c (k_2\, d_{aver.}/d_{toe})]$ (2)

Toe: $R_T = (\pi/4\, d^2_{toe} q_b)$ (3)

Ultimate: $R_U = R_S + R_C + R_T$ (4)

Allowable: $R_A = (R_S + R_C)/2 + (R_T/3)$ (5)

where: h_i=layer thickness; $d_{aver.} = (d_{top} + d_{bottom})/2$=pile average diameter; d_{toe}=pile toe diameter; q_c= tip resistance; $q_s = k_1 q_c^{0.5}$=pile friction unit; $q_b = k_3 q_{ctoe}$=pile toe unit; q_{ctoe} is measured from +8 d_{toe} to -4 $_{dtoe}$;

k_1 =0.75→1.2; k_2=1→1.3; k_3=0.3→0.8 (trend from cohesionless to cohesive soils for all "k" values)

The design of the pile capacity is often based on preliminary investigations carried out by others, mostly limited to borings with in situ tests like SPT (however executed with closed cone tip instead of the traditional sampler), or / and dynamic cased penetration tests like DPSH (Dynamic Penetrometer Super Heavy). From these tests realistic q_c values can be obtained combining the well known Dutch's Formula with a coefficient, derived from the experience of the author, that converts the calculated dynamic resistance into static cone resistance (units: kg, cm).

SPT $\quad q_c=\alpha\{[M^2H]/[Ae(m+M_1)]\}$ (6)

DPSH $\quad q_c=\alpha(MH/Ae)$ (7)

where: M=hammer weight; m=rod weight; M_1= hammer + anvil weight; A= cone area; e=set per blow; α=0.2→1.2 (from peat to coarse or dense gravel).

The reference equations have been applied to the three prediction events that are presented in the following sections, introducing, whereas considered as advisable, some modification.

3 ORLANDO, 2002 (ASCE GEOINSTITUTE'S DEEP FOUNDATION CONFERENCE)

The general ground condition and the characteristics of the pile used, are listed in Table 1. Neither a plan with the location of the piles and the in situ tests, nor information concerning the type and the size of toe plate, was provided. Also no soil laboratory results were furnished to the participants.

Table 1. General Information (Piles and Soil)

Pile Type and Length:	Driven, 13.7m
Material and Diameter:	Steel Pipe (closed toe), 0.324m
Driving Date:	11.26.2001
Cast in situ Date:	-.-
SLT Date:	02.15.2002
Soil (Origin):	Alluvial (?)
Soil (Type):	Fine Sand to Silty Sand (clay lenses)
OCR:	OC (from -2m to -5m)

The competitors have had at their disposal two borings with numerous SPT and three CPTU in only one of which, used as reference (CPTU-M, Fig.1), the pore pressure was measured in u_2 position. The static loading test had to be unfortunately abandoned because, at a load little slightly exceeding 1200 kN, one of the four corners of the reaction frame lifted off and so the comparisons between predicted and measured pile capacity has been possible only through the reconstruction, necessarily approximate, that Fellenius et al. (2004) did. The predicted capacity is also highlighted (Fig.2).

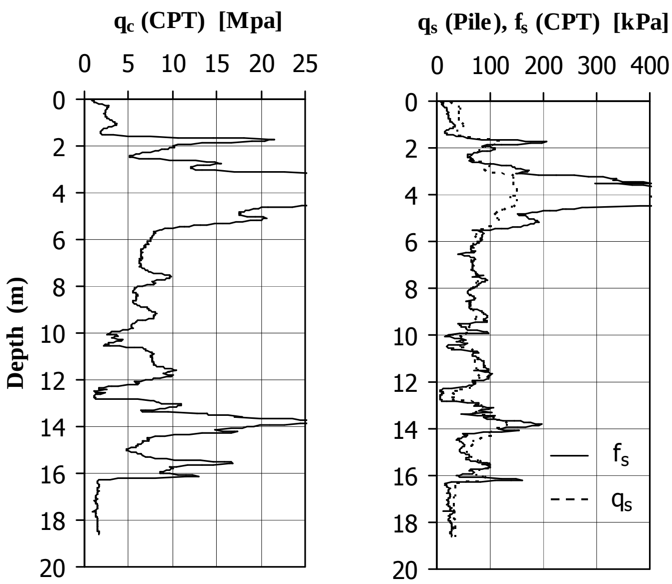

Figure 1. CPTU-M histograms (with q_s values)

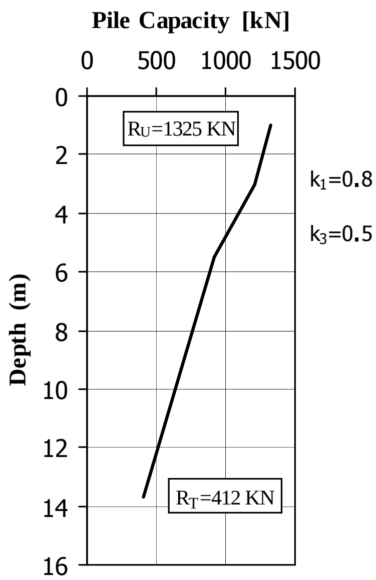

Figure 2. Predicted capacity

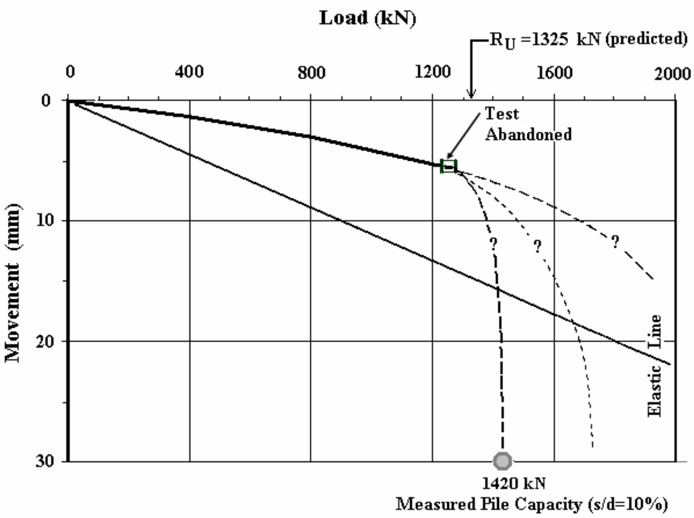

Figure 3. SLT: predicted and measured capacity

Between the suggested load–movement curves (Fig.3), the worst one was chosen because considered as more consistent with the entry data (CPTU-M and pile characteristic). Considering this assumption as valid, the prediction turns out to be good.

4 MERVILLE, (FRENCH NATIONAL DRIVEN VIBRATORY PROGRAM, 2003)

The second competition was carried out in France for a research project involving piles driven by impact or vibration (M.T. Ma et al., 2003). In this case, sufficient results from laboratory analysis and a series of in situ tests (SPT, CPT, SCPT, PMT) were provided to the participants.

Table 2. General Information (Piles and Soil)

Pile Type and Length:	Driven (IHC S70), 9.4m
Material and Diameter:	Steel Pipe (open ended), 0.508/0.487m
Driving Date:	04.08.2003
Soil Plug Height:	7.27 m
Cast in situ Date:	-.-
SLT Date:	05.19.2003
Soil (Origin):	Marine Deposit (Ypresian)
Soil (Type):	OC Clay (50% Smectit, CF=93%)
GWT, w_n, γ	(-1.5/-1.9m), 32.3%, 18.5 (kN/m^3)
Atterberg Limits:	LL=69.2%, IP=40.5%
Vs, G	150→225 (m/sec), 40→90 (MPa)

For the design the CPT PS-1 and PS-2 were chosen (Fig.4).

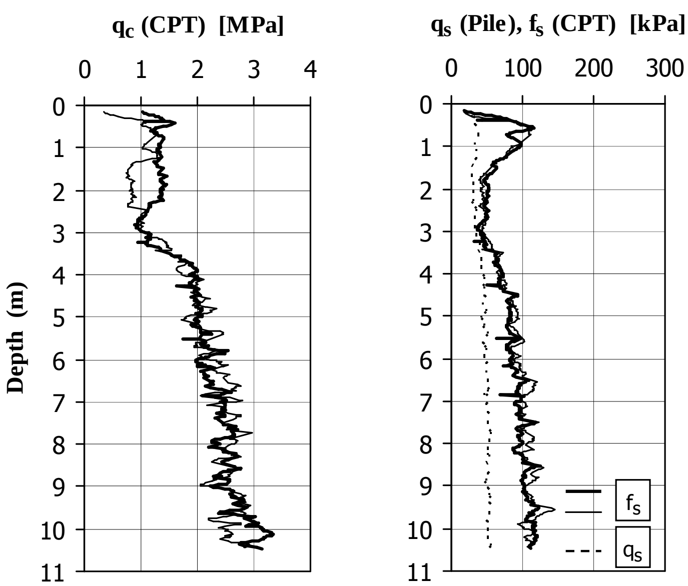

Figure 4. CPT histograms (with q_s values)

The calculated capacity is showed in the following graph (Fig.5). The comparison between the predicted and the measured capacity of the pile (see Fig.6), seems again to be good.

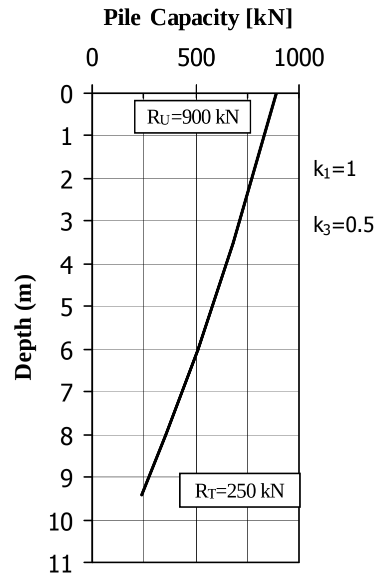

Figure 5. Predicted capacity

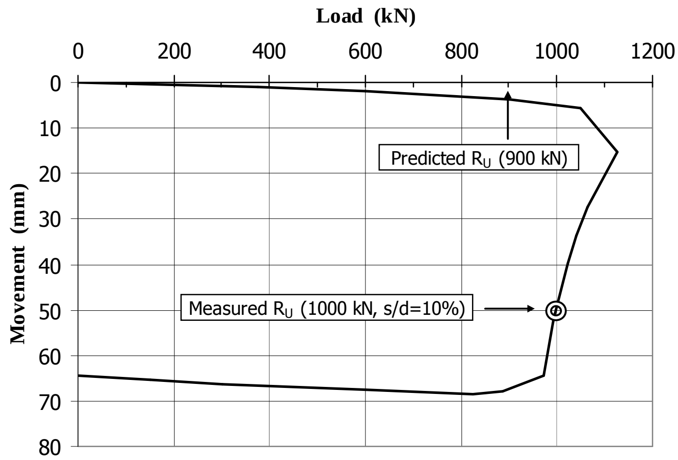

Figure 6. SLT: predicted and measured capacity

5 PORTO (CEFEUP/ISC'2-EXPERIMENTAL SITE, 2004)

The last prediction has been done for the Second International Conference on Site Characterization (Porto, Portugal, 2004). The predictions have been commented by Viana de Fonseca et al. (2007). The characteristics of the piles that were executed are listed in the following Table.

Table 3. General Information (Piles)

Pile Type:	Driven (C1)	Bored (E9)	Bored (T1, CFA)
Material:	Concrete	Concrete	Concrete
Shape:	Square	Round	Round
Diameter (m):	0.35	0.60	0.60
Length (m):	6.0	6.0	6.0
Driving Date:	Sept.2003	-.-	-.-
Cast in situ Date:	-.-	Aug.2003	Aug.2003
SLT Date:	Jan.2004	Jan.2004	Jan.2004

Also in this case, both laboratory analysis and a series of in situ tests (SPT, CPT, SCPT, DMT, PMT), were provided to the competitors. The deriving subsoil properties are summarized in Table 4.

Table 4. General Information (Soil)

Soil (Origin):	Residual (saprolite from granite)
Soil (Type):	Sandy clayey silt with some gravel
OCR:	OC (relicte structure ?)
GWT, w_n , γ	-10/-12 m, 16/22.5 %, 16.6/20 kN/m^3
FC, CF	38.5/47.2 %, 3.3/9.5 %
Atterberg Limits:	LL=32/44 %, IP=5/17 %
Vs, G	250→300 (m/sec), 100→200 (MPa)

Due to lack of experience with residual soils, the author used for the determination of the pile friction unit, the method of Takesue et al., proposed by Mayne & Schneider (2001) for a bored pile in the Coweta County (USA) executed in saprolitic soils with characteristics similar to those of Porto.
In Figure 7 are represented the CPT2 (T1), 8 (E9) and 5 (C1) but only for the last one is also shown the pile friction unit derived from the Takesue's equation:

$$q_{s1}=\{f_s[(\Delta u/1250)+0.76]\} \qquad (8)$$

where: $\Delta u=u_2-u_0$; u_2=water pressure (just behind the cone tip); u_0=hydrostatic pressure; f_s=friction sleeve

Previously the author has never used f_s to calculate directly the pile friction unit because most CPT at his disposal were mechanical and consequently with unreliable f_s values (Togliani & Beatrizotti, 2004). In this case, looking to the Takesue formula, he has tried to use f_s (CPTU are electrical) modifying in the following way the reference equation (1):

$$q_s=\{q_c^{0.5}[0.65+(R_f/8)]\}/2+f_s/2 \qquad (9)$$

where: R_f= friction ratio=$(f_s/q_c)100$

The last equation (9) is also inserted in the prediction obtaining results very similar to those of q_{s1} (see Fig.7). The full q_{s1} values were applied, as already written, only to the driven pile (C1) being consider to the author's judgment, differently to Mayne & Schneider (2001), as excessive for the bored piles (T1, E9). For these last a reduced pile friction unit ($0.6q_{s1}$) was chosen on the basis of Dutch experiences (Brouwer, 2002). Due to small length of the driven pile (C1), the author has preferred to limit to 0.3 his k_3 value. Then, k_3 is respectively decreased to 0.25 (T1) and to 0.15 (E9) also for the difference of the boring and the cast in place methods. The predicted capacity of the various piles is presented in Figure 8 while the comparison with the measured capacities is shown in Figure 9. The result has been considered respectively sufficient (T1), good (E9) and excellent (C1).

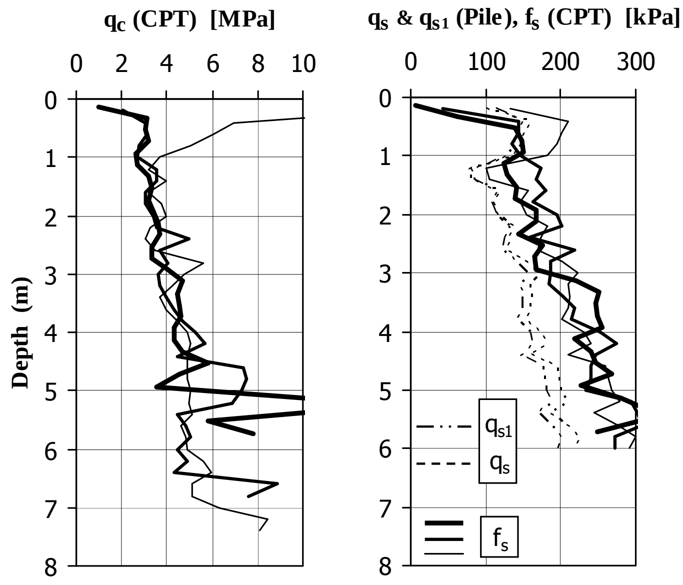

Figure 7. CPT histograms (with q_s & q_{s1} values)

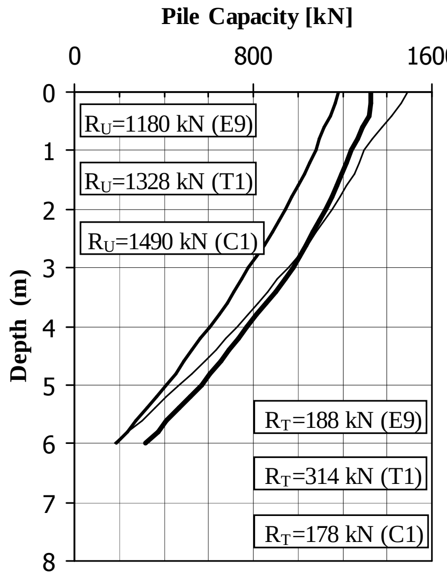

Figure 8. Predicted capacities

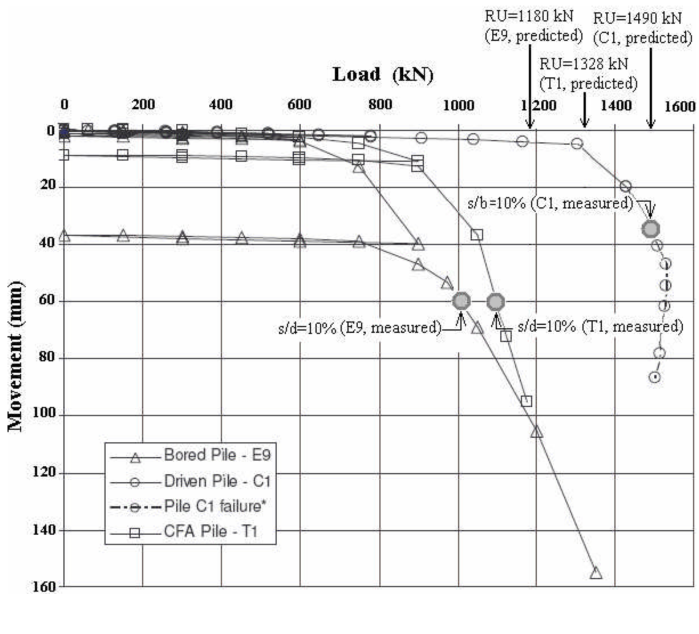

Figure 9. SLT: predicted and measured capacities

The toe of E9 (bored pile) was instrumented with a 350 mm diameter flat-jack load cell, placed between two steel plates connected to the bottom of rebar cage. This cell allowed to split the capacity of the pile (R_U) into shaft (R_S) and toe (R_T) resistance, highlighted in the following graph (Fig.10).

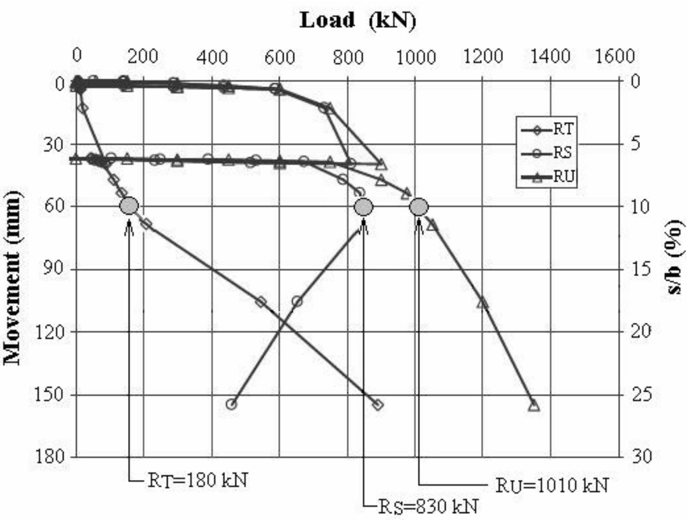

Figure 10. Pile E9 (measured R_S, R_T and R_U)

The extraction of this pile some time after the SLT, has allowed to measure the real section of the toe (0.525 m) an then to correct the R_T value (previously calculated on the basis of the theoretical section). It should be noted that the value derived for the toe capacity, based on the load cell, is questionable because the validity of the assumed conversion from cell pressure to load, is dubious.

6 PILE CAPACITY (R_U) AND ALLOWABLE LOAD (R_A): COMPARISONS

Table 5 summarizes the predicted and the observed capacities of the piles for Orlando and Merville. About, must be finally remembered that the predicted capacity must be considered as Class A activities because submitted to the promoters earlier of the static loading tests.

Table 5. Comparisons:

		Orlando	Merville
		(kN)	(kN)
Prediction:	R_S	913	650
	R_T	412	250
	R_U	1325	900
	R_A	593	408
SLT:	R_U (s/d=10%)	1420	1000
	$R_A = R_U/2$	710	500
R_U (predicted) / R_U (SLT)		0.93	0.90
R_A (predicted) / R_A (SLT)		0.84	0.82

The Porto results are instead synthesized in Table 6.

Table 6. Comparisons: Porto

Pile Type:		C1	E9	T1
		(kN)	(kN)	(kN)
Prediction:	R_S	1312	992	1014
	R_T	178	188	314
	R_U	1490	1180	1328
	R_A	715	559	612
SLT:	R_U (s/d=10%)	1500	1010	1100
	$R_A = R_U/2$	750	505	550
R_U (predicted) / R_U (SLT):		0.99	1.17	1.21
R_A (predicted) / R_A (SLT):		0.95	1.11	1.11

7 REFERENCE EQUATIONS: UPDATING

The predictions improve assigning new values to k_1 and k_3 coefficients according to:

$$k_1=\{[1.2[0.8+(R_f/8)\}\beta \qquad \text{if } R_f < 1 \qquad (10)$$

$$k_1=\{[1.1[0.4+LN(R_f)\}\beta \qquad \text{if } R_f \geq 1 \qquad (11)$$

where: β=1 (displacement driven piles), 0.6 (non displacement driven and CFA), 0.5 (bored)

$$k_3=\{\lambda +[0.01(L_{pile}/d_{toe})]\} \qquad (12)$$

where: λ=0.2 (all driven piles), 0.1 (all bored piles)

The slenderness ratio (L_{pile}/d_{toe}), has been introduced in the above equation to take in account the scale effect despite this assumption is not consistent with the recent findings of Randolph (2003), White et al. (2005) and Lehane et al. (2005), which affirm that "q_b/q_c" is anyway a constant and independent from diameter and length (at least for driven piles in sand). In this case the scale effect is still justified by the different type of piles employed (driven and drilled) and also by the uncertainties to date present in the tools that should establish the theoretical behavior of the piles (the SLT are executed with different methods and length of time, the interpretation of the data obtained with extensometers or load cells or other is often a subjective question as the elaboration of CPT data, besides frequently realized not near the tested pile, etc.). The following tables show the refinement of the predictions obtained with the proposed updating.

Table 7. Comparisons:

		Orlando	Merville
		(kN)	(kN)
Prediction:	R_S	1061	770
	R_T	488	195
	R_U	1549	965
	R_A	693	450
SLT:	R_U (s/d=10%)	1420	1000
	$R_A = R_U/2$	710	500
R_U (predicted) / R_U (SLT)		1.09	0.97
R_A (predicted) / R_A (SLT)		0.98	0.90

Table 8. Comparisons: Porto

Pile Type:		C1	E9	T1
		(kN)	(kN)	(kN)
Prediction: R_S		1185	770	915
	R_T	241	265	282
	R_U	1426	1035	1197
	R_A	673	473	552
SLT:	R_U (s/d=10%)	1500	1010	1100
	$R_A = R_U /2$	750	505	550
R_U (predicted) / R_U (SLT):		0.97	1.02	1.09
R_A (predicted) / R_A (SLT):		0.90	0.94	1.00

8 CONCLUSION

The proposed empirical design method provides acceptable values of capacity (R_U) and allowable load (R_A), as these are included in the tolerances ordinarily admitted (± 20%). This final result seems encouraging, especially considering that only almost a third of the totally presented predictions has produced sufficiently approached R_U values (± 20%) and that, between these, roughly the half properly evaluated the shaft and the toe resistance. Moreover, the last updating of the reference equation has produced further improvements (the approximation is reduced to ± 10%).

9 ACKNOWLEDGEMENTS

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