Pile Capacity Prediction using CPT - Case History

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ABSTRACT: The behavior of tapered precast centrifuged concrete piles, used for a small piled raft, that penetrate lacustrine (often organic) soils to end into dense alluvial sandy gravelly soils, has been investigated. The predicted capacity, derived directly from q_c and f_s values (CPT), has been checked at first by the Gates formula combined with the Svinkin & Skov solution to quantify the set-up improvement and subsequently with a Static Loading Test, obtaining concordant and reliable results.

1 INRODUCTION

Generally the Author's task is to choose on the base of the available data (often alone CPT or DPSH), the type, the characteristics and the dimensions of the precast centrifuged concrete piles best suited to the particular deep foundations.

This preliminary design serves to prepare an offer and then, if the job is acquired, the consultant is often engaged to verify if the predicted pile capacities are in agreement with the in situ observations (almost always penetration logs, rarely PDA and exceptionally Static Loading Tests).

Over the years the Author working at first with LCPC [Bustamante et al.,1982 (shaft and toe resistance)] and Gambini [Gambini, 1990 (taper resistance)] formulas, has developed a method, using directly the CPT results, to predict the capacity of driven displacement piles adapting it, subsequently, also to small displacement, bored and CFA piles (Togliani, 2008).

2 SOIL CONDITIONS

In the locality of Terranegra (Legnago, Italy), the soil is initially formed of a sandy gravelly cover followed by a soft lacustrine deposit (clayey sandy silt sometimes with organic lenses) and, at a depth of 17 m, by a dense alluvial deposit (sand, gravelly sand).The site investigation consisted of two mechanical CPT with essentially similar results.The geotechnical characteristics of the crossed soils are based on the worst (CPT2) penetration resistances, represented in Figure 1 below.

The pile installation was carried out in a 5.5 deep excavation protected by sheet pile and by dewatering the initial 2.5 m deep groundwater table.

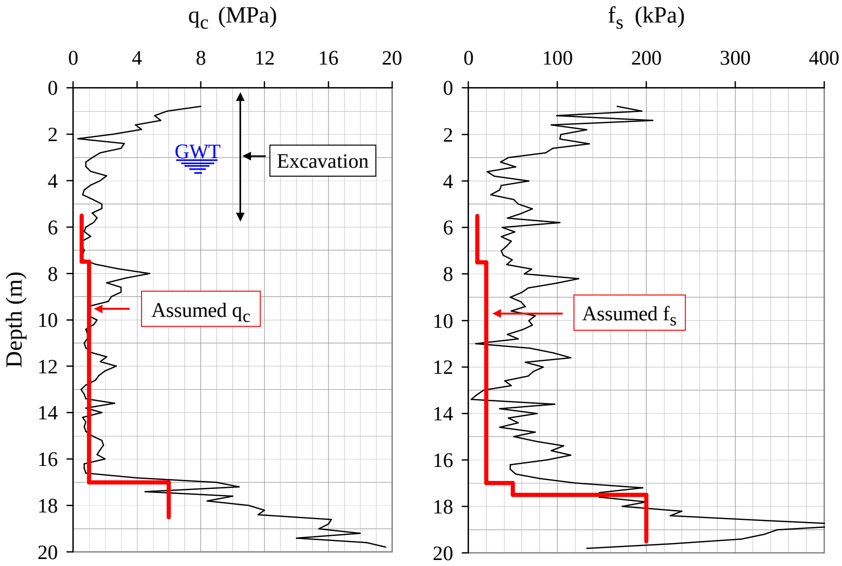

Figure 1. CPT2 Histograms

The q_c and f_s schematizations (red line) have been necessary because the excavation has modified the overburden pressure and therefore the CPT results, that exactly depend on overburden stress, must be realigned to the new condition.

The decrease of the "q_c mean" has been valued around 30% while that of the f_s twice over, considering that the mechanical friction jack measures, at the connection with the cone, also a part of the base resistance (q_c) and for this reasons is often much larger than f_s derived by an electrical cone (Lo Presti et al., 2009; Togliani & Beatrizotti, 2004).

3 PILES DESIGN

For the planned small piled raft (8m x 22m), the required pile service load was $\leq$ 300 kN. Considering that the selected tapered precast centrifuged concrete piles cannot penetrate without damages more than 1-1.5 meter in the dense alluvial deposit and that the intermediate length between 12 and 14 meters is not standard, it has been verified, first, that the capacity of the pile of shorter length is sufficient employing the method explained below (Togliani,2008), especially referred to these displacement piles.

3.1 *Shaft Resistance (R_S)*

$R_f = (f_s/q_c)100<1 \quad R_S= (\pi d_{mean}\ l)\ q_c^{0.5}\{1.2[0.8+(R_f/8)]\}$ (1)

$R_f \geq 2 \quad R_S =(\pi d_{mean}\ l)\ q_c^{0.5}[1.1[0.4+LN(R_f)]$ (2)

$1 < R_f < 2 \quad R_S =(\pi d_{mean}\ l)\ q_c^{0.5}\{1.2[0.8+(R_f/8)]+1.1[0.4+LN(R_f)]\}/2$ (3)

3.2 *Taper Resistance (R_C)*

$q_c \leq 3$ MPa $\quad R_C= 0.785\ (d_{sup}^2-d_{inf}^2)1.2\ q_c\ (d_{mean}/d_{base})$ (4)

$q_c > 3$MPa $\quad R_C= 0.785\ (d_{sup}^2-d_{inf}^2)\ q_c\ (d_{mean}/d_{base})$ (5)

3.3 *Toe Resistance (R_T ; q_c as the mean of the values between+8d and -4d)*

$q_c \leq 15$ MPa $\quad R_T=0.785\ d_{base}{}^2 q_c[0.2+(0.01\ L_{pile}/d_{base})]$ (6)

The obtained result is summarized in Table 1 (Tab.1)

Table 1. Pile Length 12 m : Capacity at s/b=10% (R_L)

Depth (m)	Pile Length (m)	q_c (kPa)	f_s (kPa)	R_f (%)	f_p (kPa)	Φ Pile (m)	Shaft (kN)	Taper (kN)	Toe (kN)	R_L (kN)
5.5	0.0					0.420				
6.0	0.5					0.413	0	0		0
7.5	2.0	550	16	3.0	39	0.390	73	16		88
10.5	5.0	1000	20	2.0	38	0.345	132	48		268
12.5	7.0	1000	20	2.0	38	0.315	79	26		372
14.5	9.0	1000	20	2.0	38	0.285	72	21		465
17.0	11.5	1000	20	2.0	38	0.248	79	21		565
17.5	12.0	6000	50	0.8	84	0.240	32	17	116	731

Seeing that the proportions among the different resistances (R_{Shaft} = 465 kN, R_{Taper} = 149 kN and R_{Toe}=116 kN) clearly advantages R_{Shaft}, that being R_{Shaft} slightly more than 1.5 time the requested service load ($Q_{Service}$ =300 kN) correspondingly will have small settlements and, finally, that $R_{Allowable}$ is larger than $Q_{Service}$ (R_{Shaft}/2 + R_{Taper} / 2.5 + R_{Toe} /3=331 kN), the pile of 12 m length was chosen and with this suggestion the job was acquired. The above safe factors were selected as depending on the different settlements that mobilize the shaft, taper and toe resistance.

4 PILE PENETRATION LOG

Using a Delmag D16-32, the first driven pile has given these results (Fig.2):

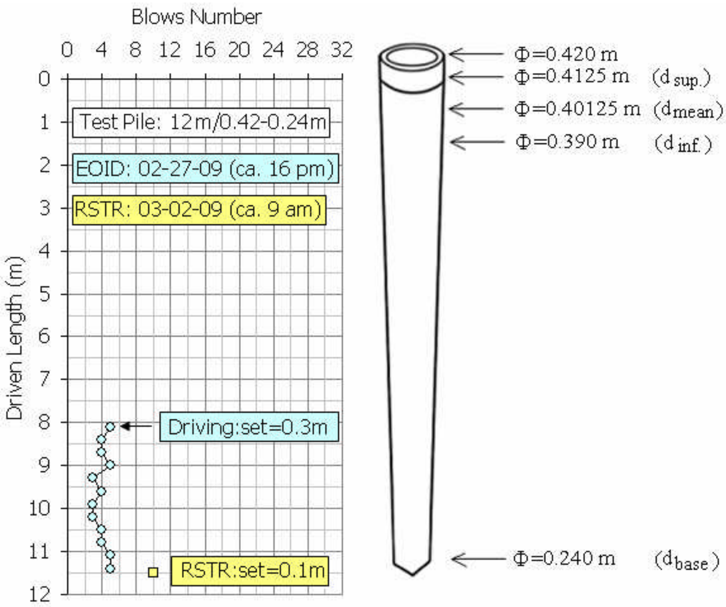

Figure 2. Penetration Log and Pile Sketch

5 PILE CAPACITY PREDICTION INCLUDING SET-UP

The Chief Project Engineer was not satisfied with the result of the penetration test and the March 4 2009 asked for a quick evaluation of the same.

The Author with the help of the Gates formula (WSDOT, 1998) for the calculation of R_U (failure resistance) and of Svinkin & Skov solution (Svinkin & Skov, 2002), experimented in the past as able to evaluate the set-up capacity, two days later sent the prediction on the future behaviour of the pile showed in Figure 3 [respectively 10, 40 and 100 days after the End of Initial Driving (EOID)], having assumed that the hammer transferred energy was equal to the nominal mean energy penalized by an efficiency factor of 0.35 as, on the other hand, written in the same Figure 3.

In detail the used formulas are:

5.1 *Ultimate Resistance [Gates; TE (ft-kips), s (inch)]:*

R_U=27*[(TE)$^{0.5}$]*(1-LOG s)   where TE=Transferred Energy; s=set per blow   (7)

5.2 *Set-up Capacity (Svinkin and Skov):*

R_U(t)/R_U(EOID)-1=B*[LOG(t)+1]   (8)

Other more recent methods (i.e. Bullock et al., 2005) were also examined but not implemented, at least in this phase, for the difficulty in the evaluation of the set-up dimensionless factor A, as no information on soil plasticity was available.

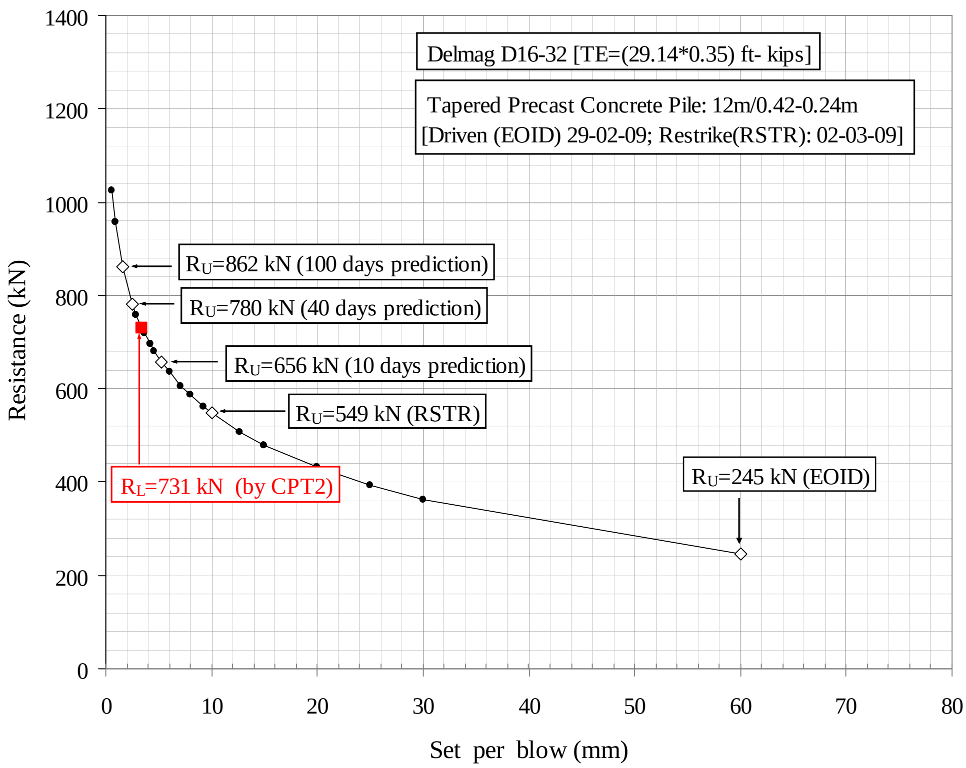

Figure 3. Pile Capacities Prediction

6 STATIC LOADING TEST (SLT)

The Chief Project Engineer was not yet convinced and March 18 2009 a Static Loading Test (up to 1.5 time $Q_{Service}$) on a different pile driven 13 days before with the same EOID final refusal of the preceding analysed pile, was carried out.

The measured load-movement curve is illustrated in the graph below (Fig.4).

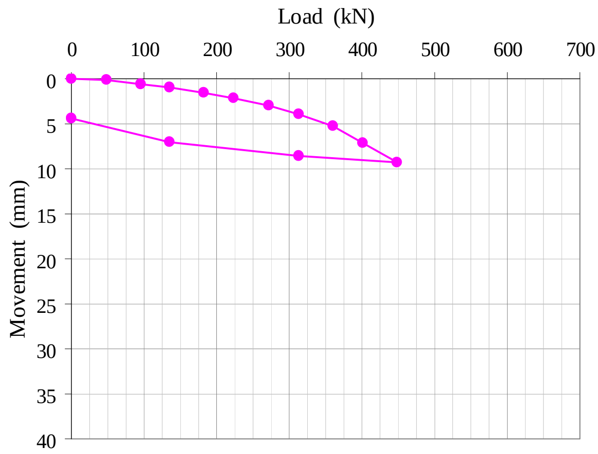

Figure 4. Pile Load-Settlement Curve

7 LOAD-SETTLEMENT CURVE

The test was considered satisfactory but a further comment has been equally asked to link the obtained results to the preceding predictions. To establish the connection is first necessary that the curve above is "extrapolated" to the ultimate resistance (R_U). This can be done, in example, using the elastic continuum method presented by Randolph & Wroth (1978-79) and Poulos (1989), as described in Mayne & Schneider (2001), to evaluate the vertical axial pile movements. To apply this method it is necessary to know the maximum elastic soils moduli that can be deduced by q_c and f_s values employing, in order, the following equations:

7.1 *Shear waves velocity (V_s m/s)*

$V_s= 1.75\ q_c^{0.627}$ (Mayne & Rix,1995) (9)

$V_s= (10.1\ LOG\ q_c-11.4)^{1.67}R_f^{0.3}$ (Mayne & Hegazy,1995) (10)

$V_s = 118.8LN(f_s)+18.5$ (Mayne, 2006b) (11)

7.2 *Shear Modulus (G_0)*

$G_0=\rho V_s^2$ (12)

7.3 *Elastic Modulus (E_0)*

$E_0 =[2(1+ \nu)G_0]$ where $\nu=0.2$ (13)

The choice of the Mayne & Rix formula to obtain the wave shear velocity in the lacustrine deposit (V_s=133 m/sec) and the average result derived by the others two formulas for the alluvial deposit (V_s=312 m/sec), has allowed the practically perfect overlap between the measured and the predicted Load-Movement Curves, as demonstrated by the following Figure (Fig.5).

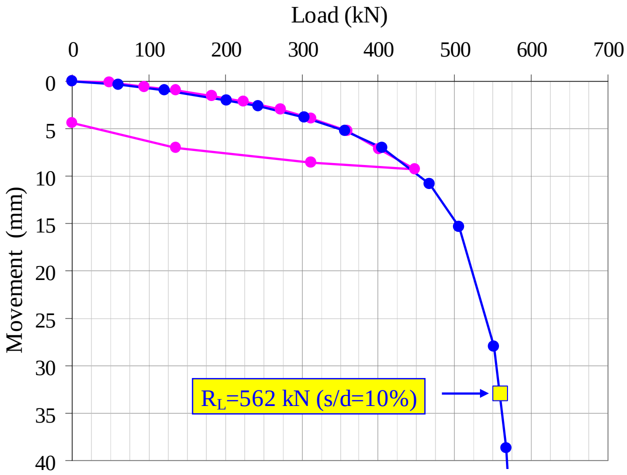

Figure 5. Measured-Predicted Load-Movement Curves

Using the chosen reference failure load (R_U= 607 kN) and the limit resistance (R_L=562 kN at s/d=10%) as results of a restrike procedure, it has been finally possible to obtain other R_U and R_L values at 100 days after EOID, considered as the long term capacities of the tested pile.

Assuming a dimensionless coefficient A=0.4, since A varies from 0.2 for sand to 0.6 for clay and applying then the Bullock et al. solution (2005), it has been possible to also compare other deduced long term set-up pile capacities to the preceding ones (Fig. 6).

The reference equation and the related results are specified below:

7.4 *Long Term Capacities (R_L, R_U)*

$R_t=R_{REF}\,[1+A\,LOG(t/t_{REF})]$ Bullock et al., 2005 (14)

R_L=562[1+0.4LOG(100/13)]=761 kN

R_U=607[1+0.4LOG(100/13)]=822 kN

8 PILE CAPACITIES COMPARISON

The next semi-logarithmic graph clearly highlights both the increase of the pile resistances with the time, function of the excess pore pressure dissipation developed during the pile driving and the pretty near pile capacities obtained with the different methods above proposed and summarily explained.

It is also to note that the numeration of the graph's labels follows the sequence of the done predictions and the intermediate tests.

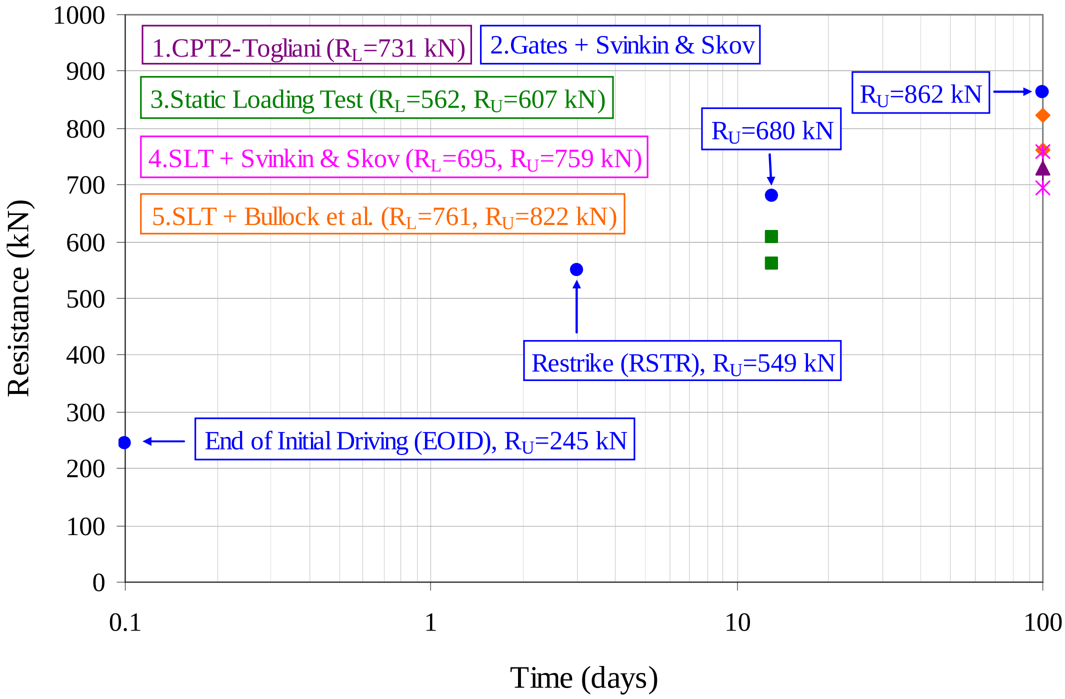

Figure 6. Pile Capacities

9 CONCLUSIONS

The examined case history has demonstrated that:

A critical analysis of the CPT results is essential to select the right q_c and f_s values;

With this “virtuous” choice the method by Togliani (2008) produces realistic pile capacity predictions;

The generally very stigmatized and questionable dynamic formulas are, for the small job, the only economic possibility to verify the predicted pile capacity and give, using Gates with a proper choice of the transferred energy, also reliable results;

The set-up capacity can be correctly predicted with the Svinkin & Skov (2002) solution and, in second order, with the Bullock et al. method (2005);

Reasonable values of the shear wave velocity and consequently of the shear soils moduli can be derived from CPT results;

Reliable Load-Movement Curves can be deduced by the elastic continuum method as described in Mayne & Schneider (2001);

Combining all the cited methods or solutions, it has been possible to derive consistent and realistic pile capacity predictions [i.e. the approximation between the measured and the predicted ultimate resistances (R_U) at 13 days after EOID is only off by 12% and considered excellent since the restrike (RSTR) value of a different pile had been used; still, the approximation of the limit resistances at 100 days (R_L) deviates by $\pm$ 4-5 % from the mean value while for the ultimate resistances (R_U) the deviation is $\pm$ 6-7 %].

10 ACKNOWLEDGMENTS

Job Location: Terranegra di Legnago (Verona, Italy)
Project : Manufatto Regolatore Fiume Bussè
Company: Rossi Tranquillo Nord (Mantova, Italy)

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