Dynamic and Bi-Directional Load Testing: Case Histories at Lugano-Paradiso (Southern Switzerland)

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ABSTRACT: Two buildings with similar characteristics have been built (Novotel, 2010-2012) and are underway for construction in contiguous land plots at Lugano-Paradiso. The geo-lithological context, the same for both areas, has required a deep foundations solution with bored piles (D.=1.2m, L.=30-32m) whose capacity has been preliminarily checked, in first case, with dynamic loading tests and in the other with a bi-directional O-Cell test. The comparison between the results of these loading tests made it possible to check not only their mutual credibility but also that of the methods used to predict the pile capacity. The usefulness of seismic tomography for identifying the thickness of the different soils units in the whole area, is also emphasized. Finally, using DMT (I_D) data, it was possible to make a prediction of the pile capacity 100 days after concreting.

1 INTRODUCTION

The contiguous land plots of interest are shown in Figure 1 below, where the edging in red encloses the area currently under construction before the demolition of existing buildings

Figure 1. Site Map

The Novotel and future buildings have similar characteristics (two underground and nine above ground floors) and for both, the geo-lithological representative stratigraphy (GWT is at 3 m depth) can be generalized in this way:

0-3m Fill
3-11m Silty clay often organic locally with silty gravel layers (fluvio-lacustrine)
11-20m Silty clay locally with some gravel (glacio-lacustrine, NC to LOC)
20-50m Glacial “Varves” (OC) and Till (localized pockets)

This geo-lithological context forced the use of deep foundations on bored piles and, for Novotel, an analytical method has been employed for their design and the input data were provided processing the results of a traditional site investigation [continuous core sample drillings with punctual in situ tests (DMT and SPT) and identification laboratory analyses carried out on the extracted cores].

The preliminary execution of three bored piles under polymer suspension (D=1.2m, L=32m) inspected with Dynamic Load Testing 24-26 days after concreting, allowed the verification of the reliability of the predicted piles capacity and to prepare their final design.

For the construction site started a few months ago, the traditional site investigation, integrated with in situ continuous tests (CPTu and DMT), was preceded by geophysical surveys (seismic tomography to have coverage over the entire building area using the boreholes and in situ test as interpretation keys and electrical tomography to check the possible presence of pollutants due to previous activities).

In this case the piles capacity prediction was based on both direct (CPTu and DMT) and analytical methods with the prediction reliability being verified by performing a bored pile (D=1.2 m, L=30 m), again under polymer suspension, equipped both with Osterberg Cell and cables with thermal sensor (TIP), a combination used for the first time in Switzerland. The bi-directional loading test was performed 22 days after concreting.

2 IN SITU TESTS PROCESSING RESULTS

Regarding SPT, it is important to note that these tests are executed during cased borings with continuous sampling, using, in place of the standard sampler, a conical point of the same section therefore allowing the test to extend beyond the standard 0.45m in order to bypass the drilling remolded zone.

The absence of any energy measurement and the randomness of the correction factors, then justifies the choice to transform the obtained reliable N_{30} values at first into dynamic resistance using the Dutch formula and then into an equivalent static resistance via a coefficient depending on soils lithology and compactness (both known through the boring) according to the procedure suggested by Togliani et al. (2002 and 2015).

CPTu to DMT and DMT to CPTu mutual conversions are governed by Togliani et al. (2015) correlations.

The plot of Figure 2 combines the measured CPTu (up to depth of 12.20m) and DMT parameters (p_0, p_1) together with q_c and f_s values obtained through DMT and again with q_c values derived from SPT.

This graph allows a check of the consistency between measured and virtual values (e.g. $p_0 \approx u_2$ and $q_c \approx p_1$ in organic or soft soils, etc.).

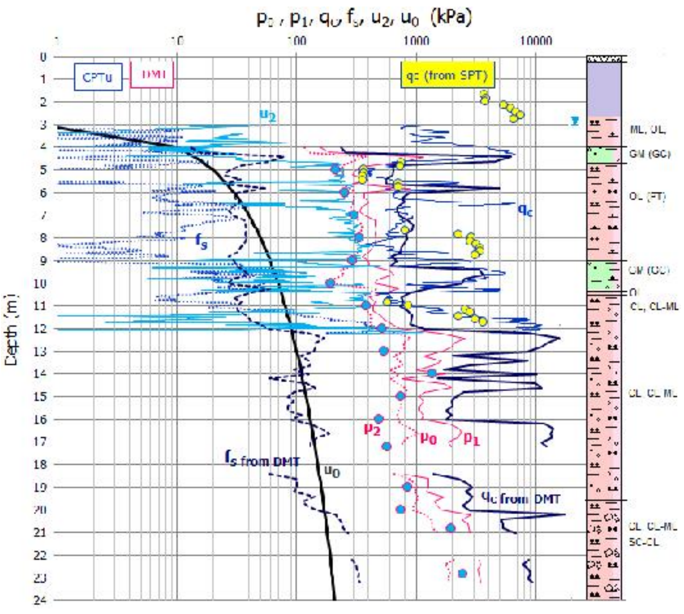

Figure 2. Synoptic Plot

The subsoil geotechnical characterization is represented in Figure 3 and Figure 4 that gathers the parameters needed both for analytical pile capacity (beta method) and load-displacement curves predictions.

With reference to G_0, it seems interesting the comparisons between the values obtained processing the DMT data and those derived by seismic tomography: in regard to moduli at small strain please note that E_0 is considered equal to $2.4G_0$ $[2(1+\nu)G_0]$.

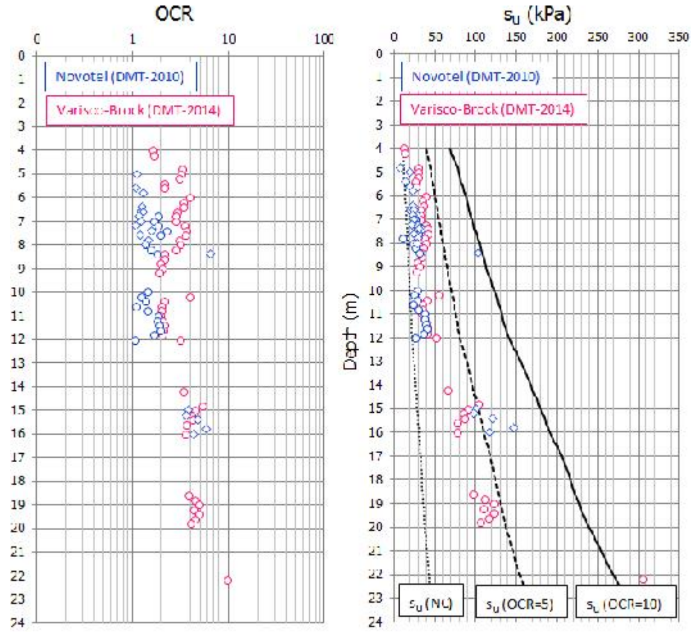

Figure 3. OCR, s_u

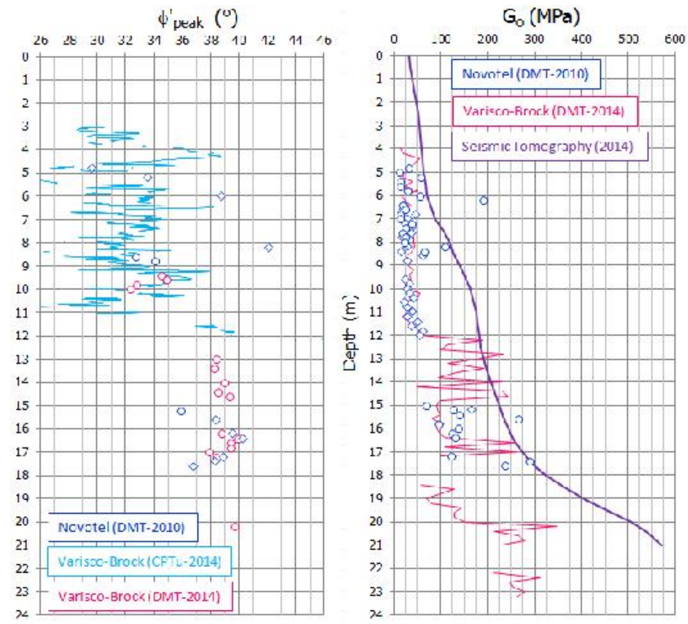

Figure 4. ϕ'_{peak}, G_0

The execution of four seismic tomographic profiles, two longitudinal and two transverse, covering in practice the entire area of interest, has allowed us to link the information resulting from boreholes, in situ tests and laboratory analyses, identifying the stratigraphic boundaries among the soil units detected, marked respectively by the following shear waves velocities: 0.15 km/sec (Fill/Fluvio-Lacustrine), 0.25 km/sec (Fluvio-Lacustrine/Glacio-Lacustrine NC/LOC), 0.40 km/sec (Glacio-Lacustrine NC or LOC/Glacio-Lacustrine OC and/or Till).

The seismic section in Figure illustrates the previous point.

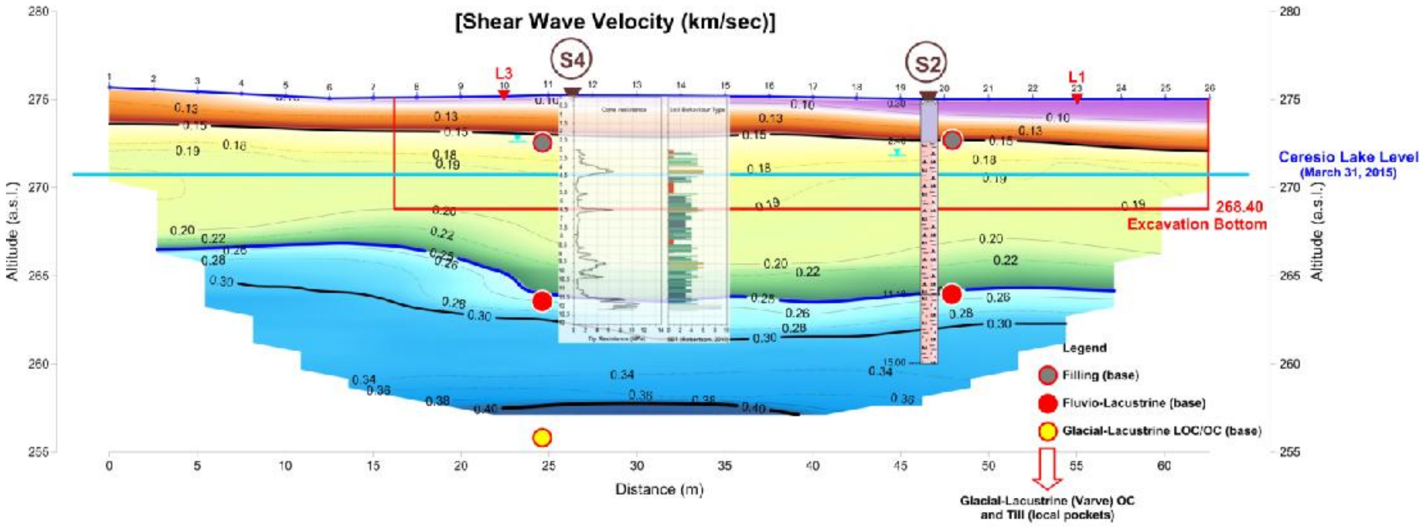

Figure 4. Seismic Tomographic Section

3 LABORATORY ANALYSIS RESULTS

The results, summarized in the Table 1, complete the geomechanical characterization of the subsoil.

Their analysis, among other things, reveal that the carbonates content may influence the measured plasticity because IP values, generally lower than 15 despite the sometimes considerable clay content, are an evident indication of a significant presence of "calcareous mud" in the clay fraction with consequent alteration of its usual behaviour.

It is notheworty that, in the glaciolacustrine LOC/OC deposits, the natural water content (W_n) is often close to plastic limits (W_p), an indication of overconsolidation also confirmed by corresponding high unit weight values.

Table 1. Lab. Analyses

Depth	W_n	W_l	W_p	I_P	γ	Clay	Silt	Sand	Gravel	CaCO3
(m)		(%)			(kN/m3)			(%)		
7.9	46.0	63	31	32	17.9	2.3	76.2	19.6	1.9	1
13.6	20.9	31	18	13	20.0	25.2	70.5	14.3		27
15.3	19.8	30	19	11	20.8	30.7	65.2	4.1		25
15.9	13.7	20	14	6	22.5	14.7	68.9	16.4		20
20.4	10.5	23	14	9	23.7	18.9	44.6	22.3	14.2	22
20.9	11.6	28	15	13	22.1	30.2	66.5	3.3		29
24.2	19.7	29	16	13	20.6	39.1	60.6	0.3		34
29.1	18.7	26	16	10	21.1	33.3	66.2	0.5		36

4 TEST PILES (NOVOTEL - 2010)

In 2010 the Author had not yet developed the correlations of mutual conversion among in situ tests nor the method of piles capacity prediction using DMT and therefore, to this end, the beta method has been employed.

The values chosen for the beta coefficient, given that we have to deal with a bored pile, were respectively equal to 0.3 (fill), 0.2 (organic soils), 0.4 (LOC Varves), 0.9 (Till pockets), 0.5 (OC Varves), while the values used for the toe coefficient (N_t) was deliberately low (equal to 10), being due to the pile toe precisely placed in the glacio-lacustrine clayey soils, overconsolidated but easy to remould when drilling.

The dynamic load testing carried out with the equipment shown below (Fig. 5) has provided the results summarized in Table 2.

Figure 5. Dynamic Load Testing Equipment

The predicted and measured (CAPWAP) piles resistances are illustrated and compared in Figure 6 noting that those predicted are respectively overestimated (shaft) and underestimate (toe).

However, in regard to the shaft resistance, surely the most relevant, it should be specified that the same has not been fully activated in the dynamic tests.

Table 2. PDA results

Pile Driving Analyser	Pile 7	Pile 9	Pile 41
Days after concreting	25	26	24
Pile type		Bored	
Pile diameter (m)		1.2	
Embedded Length (m)		32	
Hammer weight (kN)		160	
Fall Height (m)		1.5	
Theoretical Energy (kN-m)		240	
Measured Energy (kN-m)	141	140	130
Total permanent set (mm)	5	6	8
Shaft resistance (kN)	8600*	7200*	8170*
Toe Resistance (kN)	3700*	5400*	4270*
CAPWAP Match Quality	4.31	2.24	6.61

**Resistances not fully activated*

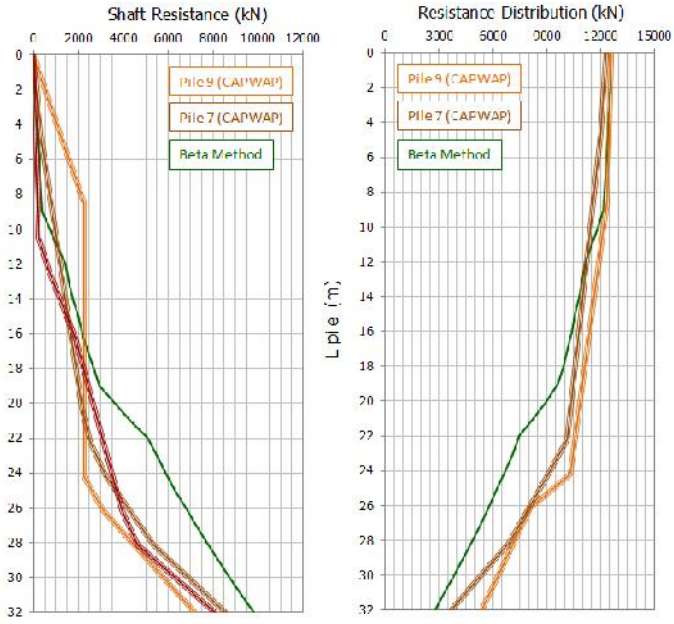

Figure 6. Resistance Curves

5 TEST PILE (VARISCO&BROCK - 2015)

The test pile capacity was obtained using the Author's methods both for DMT (Togliani et al., 2015) and CPTu (Niazi et al., 2013), the latter using the DMT to CPTu conversion already mentioned.

All these methods use the general resistance equations stated below:

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

Toe $R_{toe}= [(\pi/4 d^2_{base} q_b)]$ (2)

where: h_i= layer thickness; $d_{aver.}=(d_{top}+d_{bottom})/2$; d_{base}= toe diameter

The DMT method has been partially updated as hereinafter specified:

If $I_D>1.8$ $q_s= \beta(p_0^{0.68} K_D^{0.3} I_D^{0.4})$ (3)

If $0.6<I_D<1.8$ $q_s= \beta(p_0^{0.66} K_D^{0.3})$ (4)

otherwise $q_s= \beta(p_0^{0.55} K_D^{0.1} I_D^{0.4})$ (5)

If $p_1 \approx q_c$ $q_b = p_{1base} [\lambda+(0.005 L_{pile}/d_{base})]$ (6)

otherwise $q_b=p_{1base}$ (7)

with p_{1base} measured from $+8d_{base}$ to $-4d_{base}$

Also the CPTu method was updated as follows:

If $f_s<20$ kPa $q_s=\beta q_c^{0.4}$ (8)

If $R_f>1.5$ $q_s =\beta\{q_c^{0.52}[1.1(0.4+LN(R_f)]\}$ (9)

If $1<R_f<1.5$ $q_s =\beta\{q_c^{0.51}[0.8+(1–R_f)/8]\}$ (10)

otherwise $q_s =\beta\{q_c^{0.53}[0.8+(1.1–R_f)/8]\}$ (11)

and, again $q_b=q_{c,base} [\lambda+(0.005 L_{pile}/d_{base})]$ (12)

with $q_{c, base}$ measured from $+8d_{base}$ to $-4d_{base}$

Finally the selected values for β and λ, dependent on piles type, are summarized in the following Table 2.

Table 2. Pile Type Factors

Pile Type	β	λ
Driven/Jacked	1.00	0.30
Drill Displacement (e.g. Omega)	0.90	0.25
Pipe (Open End)	0.70	0.20
HP	0.65	0.15
CFA, Bored (Polymer)	0.60	0.10
Bored (Bentonite, Cased)	0.50	0.10

It is necessary to add that using the DMT a depth only of 23.20m was reached, then a few meters inside the OC varves.

As consequence it was assumed, also on the basis of previous experiences in neighbourhood, that the final values of p_0 and p_1 remain unchanged up to 35m depth and therefore this is valid also for q_c and f_s values derived from them.

Obviously the beta method was again used, this time considering a similar stratigraphic sequence but ignoring the till pockets, reducing the pile length to 30m and increasing the N_t coefficient from 10 to 15 to take into account the Novotel test piles experience (therefore the toe resistance changes from the previous 2781 kN to 3918 kN).

The unit skin friction ($f_{p, bored}$) and the shaft piles resistances obtained by aforementioned methods, are then compared with those measured (Figure 7).

In the left graph (unit skin friction), you can see that the paths followed by bidirectional (mean net values) and dynamic load testing measurements are quite different for a depth greater than 12m and not always aligned with the soils strength characteristics highlighted by in situ tests .

This behaviour is not always explainable but despite this, the predicted and measured shaft resistances equally reach, at the end, comparable values (right graph).

The image in Figure 8 illustrates some Osterberg Cell assembly detail and, among other things, the yellow cables used for the thermal profiling are also visible, covered by Duba Pile Control report, whose graphical results are presented in Figure 9.

It is important to note the pile section increase up to a depth of 6m and the reduction below the O-Cell (the concrete volume used was 35 m^3, slightly above the theoretical).

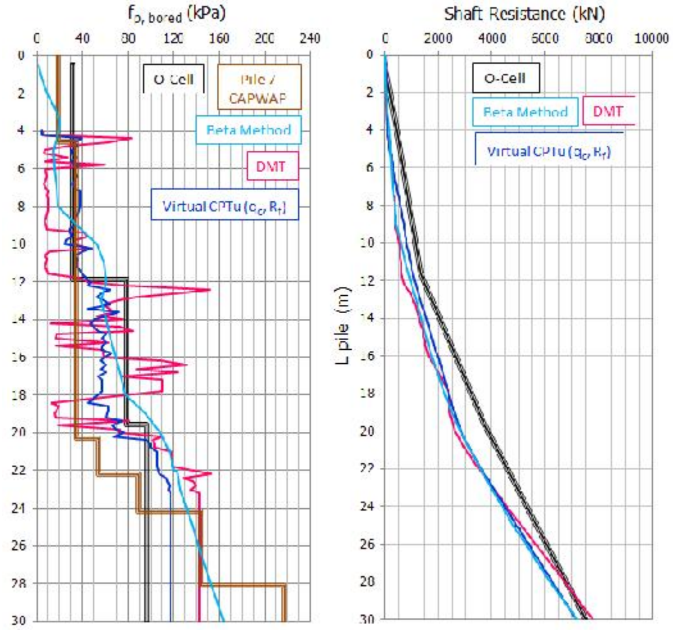

Figure 7. Unit Skin Friction and Shaft Resistances Curves

Figure 8. O-Cell and TIP cables

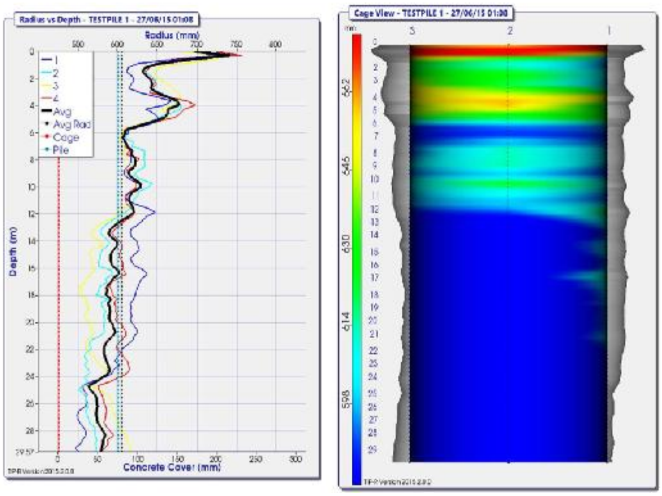

Figure 9. Thermal Integrity Profile

Figure 10 illustrates a schematic section of the test pile as built (Loadtest-Fugro report) and alongside, to make it more comprehensible, the stratigraphy of a nearby borehole (unfortunately the closest (S. 2), was only 15m long].

Figure 11 shows the Load-Movement (upward/ downward) from the bi-directional test results while Figure 12 plots the Strain Gage Load Distribution.

Increments of 400 kN with hourly holds were chosen as load criterion while, during unloading, the decrements were 1000 kN with ten minutes.

It is evident, analysing Figure 11, that the pile bottom was not sufficiently clean so that the toe resistance has begun to engage only after a displacement of 30 mm.

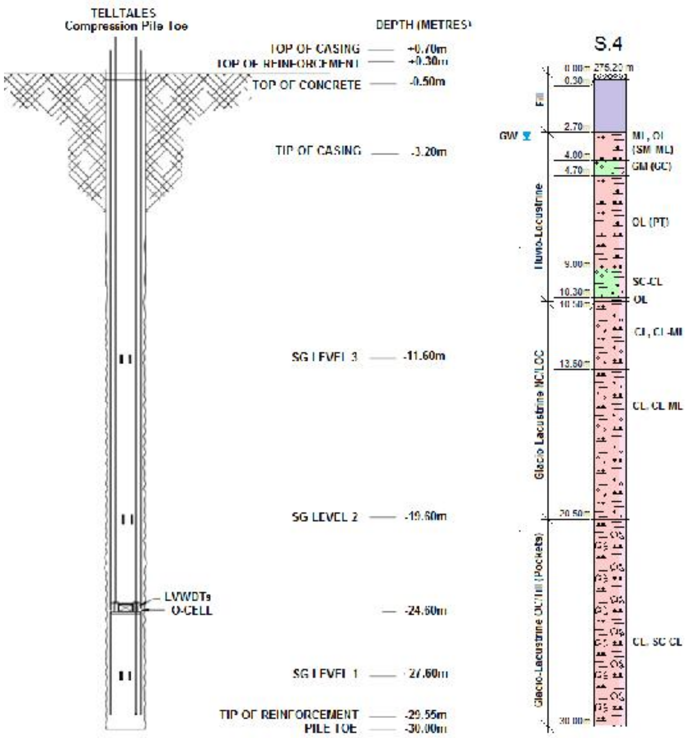

Figure 10. Pile Schematic Section

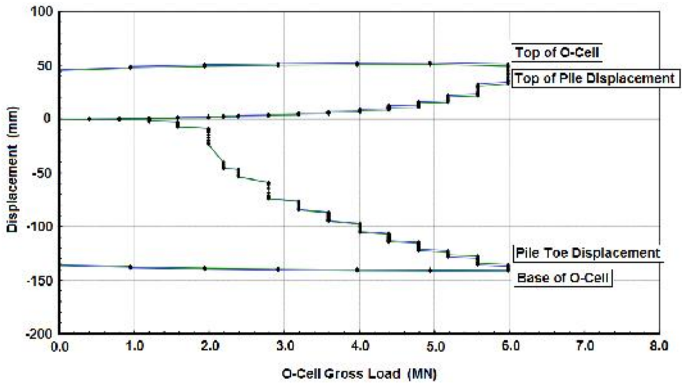

Figure 11. Upward and Downward Load-Movements Curves

This shortcoming probably due to a residual thickness of settled material (7 hours are elapsed between the drilling end and the concreting beginning), explains the remarkable gap between the Equivalent Top Load and the Cemset curves which are usually

closer together, compatibly with the fact that the latter expresses a long term prediction (Figure 13).
On the matter it should be remembered that the construction of a Cemset curve comes out from the combined use of Cemsolve, a method of analysis that evaluates the significant components contributing to the overall modelled pile behaviour using only the pile displacement recorded during the static loading test and of Timeset, a program that can model with respect to time the test results (England, 1993-2009).

Figure 13 presents the CAPWAP load-movement curve of pile 7, the closer to test pile among measured ones at Novotel, revealing a much stiffer behaviour even compared to Cemset curve, not justified by the greater length of the pile (32m).

Figure 13 also shows the simulated load- movement curves referred to DMT pile predicted capacities, which seem reasonably close, except for that of the toe resistance, of course having not considered when calculating, the eventuality of an insufficient cleaning of the hole bottom.

These curves were made using the elastic continuum theory, as presented by Randolph & Wroth (1978-1979) and Poulos (1979) and described in Mayne & Schneider (2001).

The modulus decay with the strain increase was modelled using the Fahey & Carter (1993) equation (modified hyperbola) but it must be remembered that the operational values chosen for the soils elastic moduli at mid-length (E_{sM}), and at pile base (E_{SL}), are not exactly those obtainable from the specific graph of Figure 4: in fact, at least in the Author's opinion, they are lower for a bored pile as a consequence of the decompression due to soil removal.

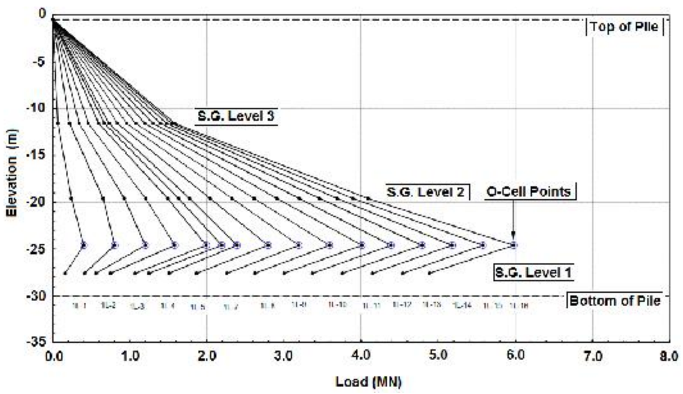

Figure 12. Strain Gage Load Distribution Curves

With regard to the ultimate pile resistance determination taking as reference the equivalent top load curve, it has not obeyed the Eurocodes specification (the load corresponding at a pile head movement equal to 10% of its diameter), but, according to Fellenius (2015), it was considered the resistance value at the intersection with the same curve of a line parallel to the elastic compression line starting from a value corresponding to a displacement of 30 mm on the toe curve.

The pile service capacity was then derived with the same procedure but starting on the toe curve from a displacement of 5mm.

On this subject, the Author suggests as an alternative, the choice of the reference pile capacity corresponding to a displacement of 30 mm on the top curve and of the service capacity at a displacement of 8 mm on the same curve (anyway the net displacement would be less than 6mm).

The same service capacity could be obtained penalizing the reference capacity with a safety factor calculated as specified in Figure 14 (FS=1.6).

This criterion could be useful when the static loading tests are carried out on non-instrumented piles as frequently happens.

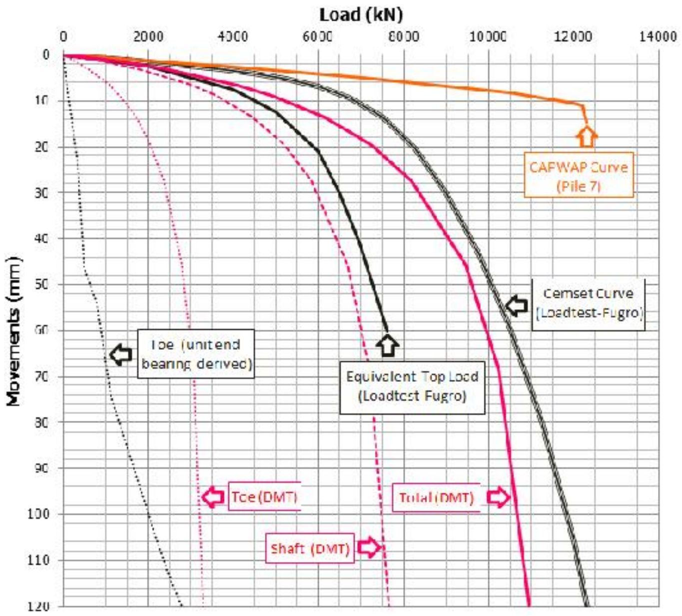

Figure 13. Load-Movements Curves

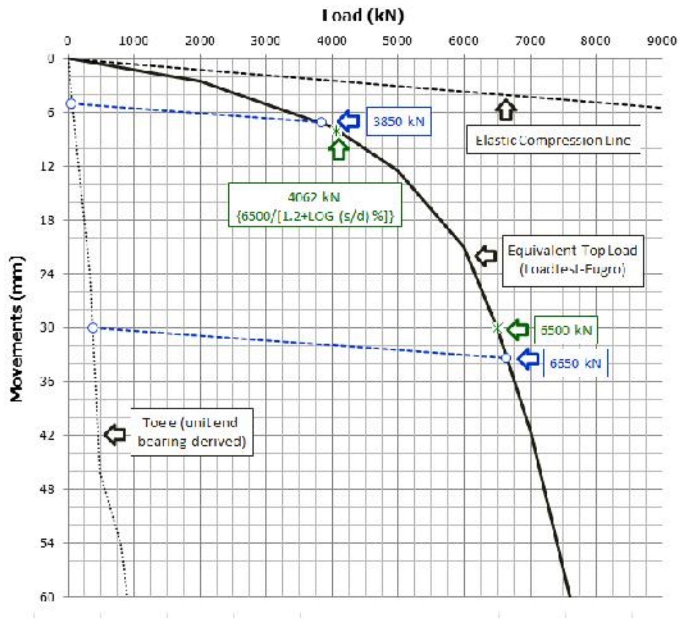

Figure 14. Service and Ultimate Resistances

6 TEST PILE SET UP

To evaluate the pile capacity increase versus time, it was decided to take as a reference the net shaft resistance (5530 kN) of the pile section above the O-Cell because its length equals in practice that of DMT, whose data are necessary to put into practice the method suggested for this purpose by Togliani et al. (2015).

At first this method has been applied not taking into account the adjustment factor ($\leq$ 1) that is dependent on pile and loading test types employed, obtaining a Final Multiplier parameter (FM) of 1.26 and therefore a capacity to 100 days (Q_{100}), considered as reference, equal to 6968 kN.

Then, to calibrate the adjustment factor, the use of the Augustesen et al. method (2006) is made, adopting for $\Delta_{10,}$ the gradient of the line, the mean value of 0.13 suggested as function of t_0 (100 days) and loading condition (unstaged), resulting in a pile capacity of 6070 kN.

Lastly a pile capacity of 6081 kN is obtained using an adjustment factor of 0.86 for $I_D \leq 0.6$ and equal to unity for $I_D > 0.6$ therefore consistent with the previous factor of 0.75 but referred to a bored pile under bentonite suspension subjected to a staged conventional static loading test (Northwestern University, Evanston, 1989).

The Table 3 and the Figure 15 below illustrate the above information.

Table 3. Q_{100} by DMT

PL	UT	I_D	RLT	α	SLT	TF	VLT	ΣVLT	FM	Q_{SLT} Meas.	Q_{100} Pred.
m	m		m		Days		m	m		kN	kN
24.2	12.6	≤0.1		3.0	22	0.68	0.00	0.00			
		≤0.3	3.8	2.0			4.62	4.62			
		≤0.6	8.8	1.5			9.27	13.89			
	2.2	≤0.8	2.2	1.2			2.41	16.30			
	4.8	≤1.8	4.8	1.1			5.03	21.33			
	4.6	>1.8	4.6	1.05			4.68	26.01	1.10	5530	6081

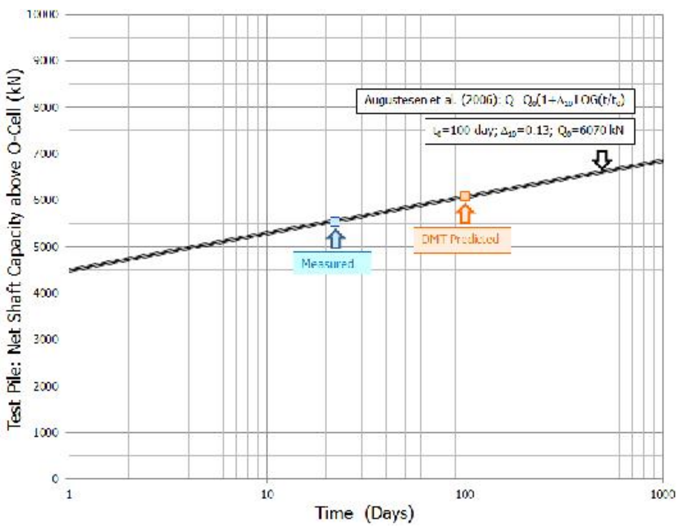

Figure 15. Test Pile Set Up

7 CONCLUSIONS

According to the previous considerations it can be argued that:

a) the dynamic and the bi-directional loading tests have led to final results all in all comparable and in agreement with those obtained using pile capacity prediction methods both analytical (beta) and directly based on in situ tests (DMT, CPTu);

b) this consistency was certainly helped both by the presumable fairly regular shaped of all the test piles for which the concrete consumption was slightly higher than theoretical as shown by the thermal integrity profile of one of them and by the similar elapsed time interval between the pile concreting and the execution of the loading tests;

c) the proposed DMT method, despite the limited use of the test for this purpose, proves that it could serve to reduce the excessive disparity in results which usually corrupts the piles capacity prediction;

d) the possibility to obtain virtual in situ tests (CPTu in this case) can be profitably used as a further and valid method to predict the piles capacity;

e) the mean unit skin friction values obtained both from CAPWAP and from O-Cell results analyses, do not always follow the layers resistances shown by in situ tests, making the shape of the resulting load distribution curves less reliable;

f) the pile ultimate resistance is better determined not by obeying the Eurocodes, but at a maximum settlement of 30mm: the availability of an instrumented pile results will direct the choice, as well as that of the service load, on the Fellenius criterion while, if this were not so, it could take recourse to that suggested by the Author;

g) the interaction between the Togliani et al. (2015) and Augustesen et al. (2006) methods leads to a fairly credible value of $Q_{100,}$ the pile capacity that should be used as reference in the opinion of the Author.

8 AKNOWLEDGMENTS

The Author wishes to thank particularly the building's owners and their Project Managers Arch. G. Lopes (Novotel) and Ing. R. Guerra (Varisco & Brock), for having allowed to make public both the site investigation and the pile loading tests results.

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