Pile Capacity Prediction (Class C): DMT vs. CPTu

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ABSTRACT: The DMT results are rarely used to predict the capacity of vertically loaded piles. This is in contrast to the plethora of papers that, for the same purpose, utilize direct or indirect CPTu-based methods, despite the fact that the basic measured DMT parameters (p_0, p_1) are well suited for calculation of shaft and toe pile resistance. Among the international pile capacity prediction events of the past quarter century DMT and CPTu were both performed only in Evanston (1989) and Porto (2004) and the Authors demonstrate that new DMT developed equations and an updated existing CPTu method (Togliani, 2008), give pile capacities equivalent to those measured. Load-movement curves by elastic continuum theory formulas and a 100-day reference setup capacity using DMT (I_D) and CPTu (f_s, I_c, FC) data, are also proposed.

1 EVANSTON EVENT (1989)

At the 1989 Foundation Engineering Congress at Northwestern University, in Evanston, Illinois, USA, a series of static load tests were performed on both driven piles (closed-end steel pipe and HP 360 steel H-piles) and bored piles (slurry or cased) as part of a prediction event held in conjunction with the conference (Finno, 1989).

The piles had equal length (15.24 m) and similar diameter (0.45/0.48 m), and were installed through a 7 m thick, variable density hydraulically-placed sand fill, ending in a soft to medium consistency, lacustrine clay deposit ("Chicago clay").

The ground water table was at a depth of 4.5 m.

The static loading tests were performed 2, 5, and 43 weeks after pile installation; the piles were also instrumented to verify the distribution of load along the shaft during the various loading steps.

Soil characterization was performed by various laboratory tests and in-situ tests, including, flat plate dilatometer (DMT) and piezocone penetration tests (CPTu).

The DMT and CPTu results are summarized in Fig.1, where the abscissa scale is reduced to allow for a better view of the variation of the basic parameters with depth.

As can be seen, high values of f_s were measured, thus justifying the use of a prehole to facilitate the penetration of the closed-end pipe piles.

The values of q_c in the clay layer lie unexpectedly between the p_0 and p_1 values, even though they are measured at a lower stress level (to properly correlate, the pore pressure u_2 was taken into account, thus correcting q_c to q_t).

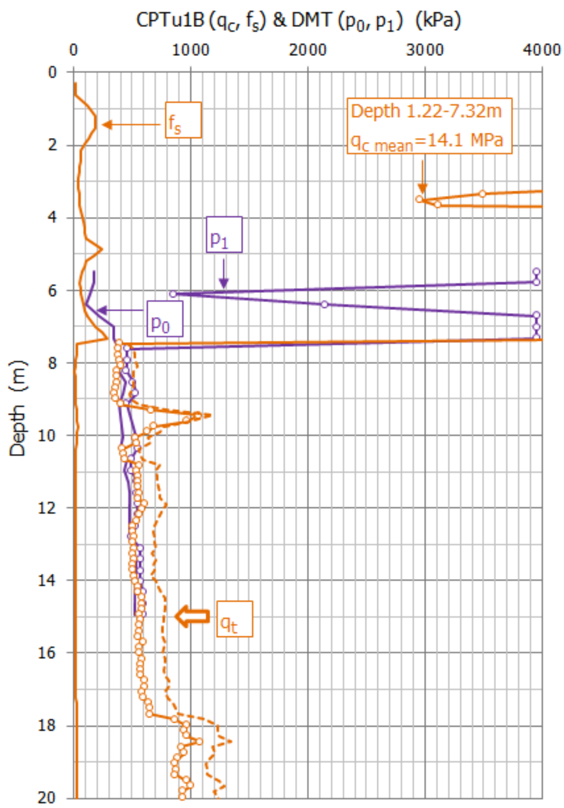

Fig.1. DMT and CPTu basic plots

The DMT testing did not begin until about the 5.5 m depth.

As will be discussed later, it was observed that both of the pile capacity curves calculated from DMT and CPTu (Figs 6 and 7) are similar for the full depth, therefore, justifying the assumption that the measured mean DMT values could also be extended for the upper soil portion where no testing was performed.

The soil behavior type is shown in Fig. 2 (material index I_D from DMT) and Fig. 3 (fines content, FC, and soil behavior type, I_c from CPTu).

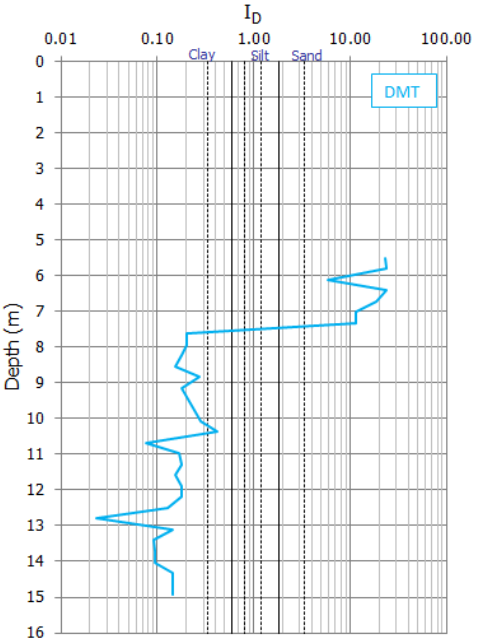

Fig.2. I_D plot (DMT)

FC is calculated by the equations proposed by Robertson, Idriss et al. and Yi, while the Ku's criterion (I_c boundary of < or > 2.67) is utilized for the sand-like and clay-like subdivision.

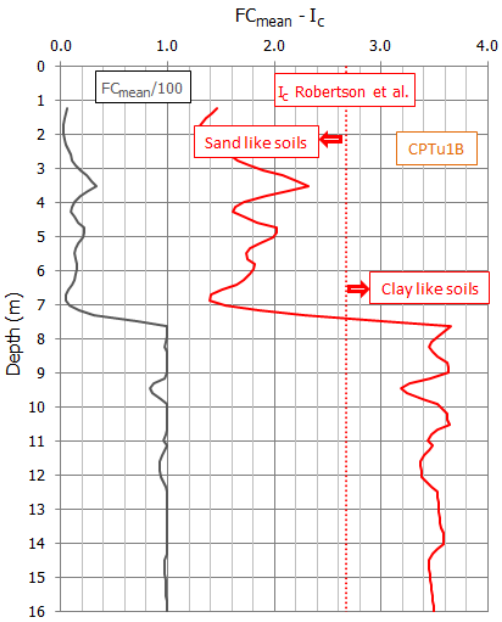

Fig.3. FC and I_c plots (CPTu1B)

To complete the series of parameters necessary for the pile capacity prediction, reference is made to Fig.4 (DMT's Horizontal Stress Index, K_D - red squares refer to anomalous values) and to Fig. 5 (friction ratio, R_f).

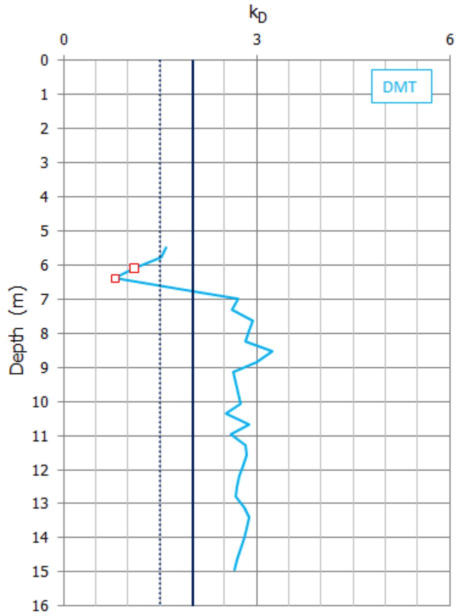

Fig.4. K_D plot (DMT)

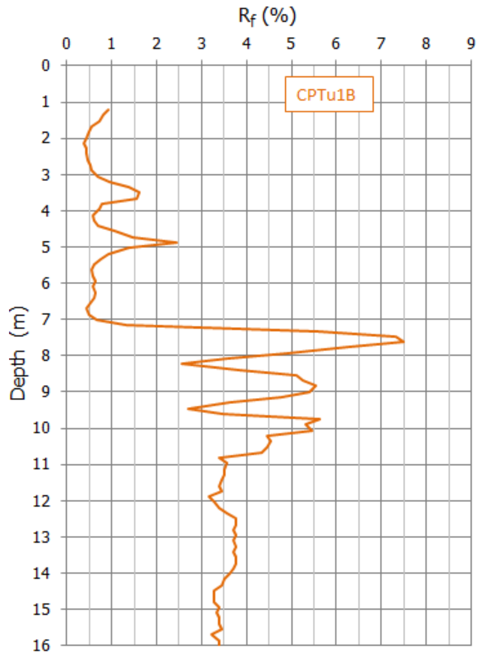

Fig.5. R_f plot (CPTu)

For both DMT and CPT the pile unit side friction (q_s) has been assessed for the case of the displacement pile (closed-end steel pipe) and was then used as reference for the "low" displacement pile (HP360).

The reference equations are:

1.1 *DMT*

If $I_D > 0.6$ either $q_s = \beta p_0^{0.89}$ (1)

or $q_s = \beta p_0^{0.68} K_D^{0.1} I_D^{0.4}$ (2)

If $I_D \leq 0.6$ $q_s = \beta p_0^{0.6} K_D^{0.1} I_D^{0.4}$ (3)

1.2 *CPTu*

If $R_f \leq 1$ $\quad q_s = \beta\{[q_c^{0.5} 1.2(0.8 + R_f/8)]\}$ (4)

If $R_f > 1$ $\quad q_s = \beta\{q_c^{0.5}[1.1(0.4 + LN(R_f)]\}$ (5)

If $R_f > 1$, $q_c < 1$ MPa and FC > 90%

$q_s = \beta\{q_c^{0.5}[q_c(0.4 + LN(R_f)]\}$ (6)

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

FC % < 0.074 mm (estimated from CPTu)

Equation 6 is an update of Togliani method (Mayne et al., 2013).

The unit toe bearing capacity was calculated with the following equation:

$q_b = q_c[\lambda + (0.005\ L_{pile}/d_{toe})]$ (7)

where q_c is measured from +8 d_{toe} to -4d_{toe}.

Equation (7) belongs to the CPTu Togliani method and also used without modification for the toe resistance evaluated by DMT considering that p_1 is nearly equal to q_c (Fig. 1).

Table 1 presents the suggested values of the coefficients for use in the former equations based on the pile type to be performed.

Table 1. Coefficients values.

Pile Type	β	λ
Precast Driven & Jacked	1.00	0.30
DD (Bauer, Omega, etc.)	0.90	0.25
Pipe (Open Ended)	0.70	0.20
HP	0.65	0.20
CFA	0.55	0.15
Bored	0.50	0.10

In calculating the capacity of the closed-end pile, the β-value in the upper sandy layer was reduced by 50% (0.5 instead of 1.0) for consideration of both the stress relief due to the pre-bore hole performed and the slightly larger diameter of the base of pile (0.48 m versus 0.46 m for the shaft) due to a slightly oversized plate at the toe. In the pile capacity estimate it was thought that, even marginally, this oversized plate would also produce a reduction in the unit friction in the clay section from β = 1.00 to β = 0.95.

As can be seen in Fig.6, the capacity calculated for the pipe pile by DMT and CPTu are nearly the same, with the two calculated curves being similar to the load distribution with depth measured in the load test five weeks after the pile installation.

For the HP360 pile, the calculated pile capacity diverges from the trend of the load distribution at 5 weeks in the clay soils, however converges in the upper 3 meters in the sandy stratum with the measured 5 week values in the pile head (Fig. 7).

The use of the two intermediate parameters (I_D and K_D) and one of the derived parameters (constrained modulus, M), allows a credible profile rebuilding of G_0, then of E_0.

It is then possible, in accordance with Fahey and Carter (1993), to calculate a modulus degradation curve and subsequently a pile load-movement curve using the elastic continuum method (Randolph & Wroth, 1978-1979, Fleming et al., 1985).

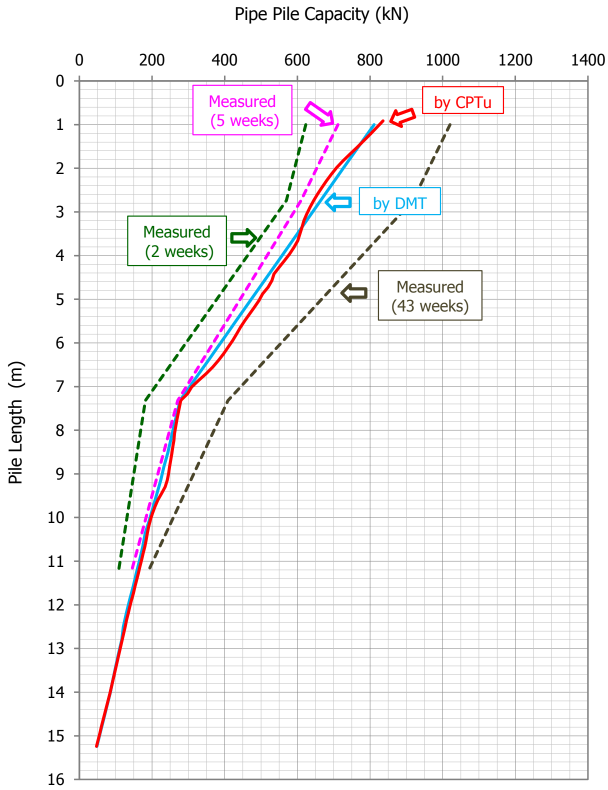

Fig.6. Pipe Pile Capacity

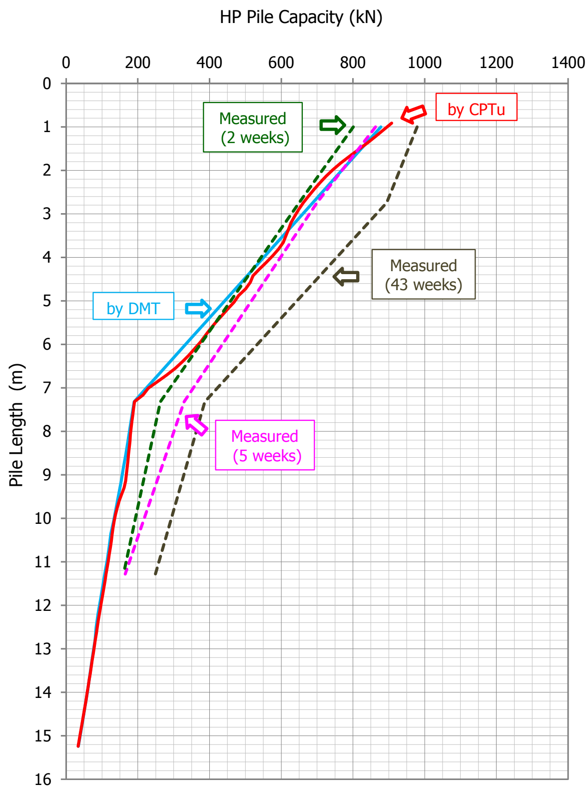

Fig.7. HP360 Pile Capacity

Fig. 8 presents the equation for the calculation of E_0 from DMT and the results from the tests; it does not, however, coincide with the values used for the settlement curve prediction likely because the driving of piles changes the site conditions, causing in this specific case, an improvement to E_0 which is a function of the soil displacement (the opposite occurs in case of bored piles).

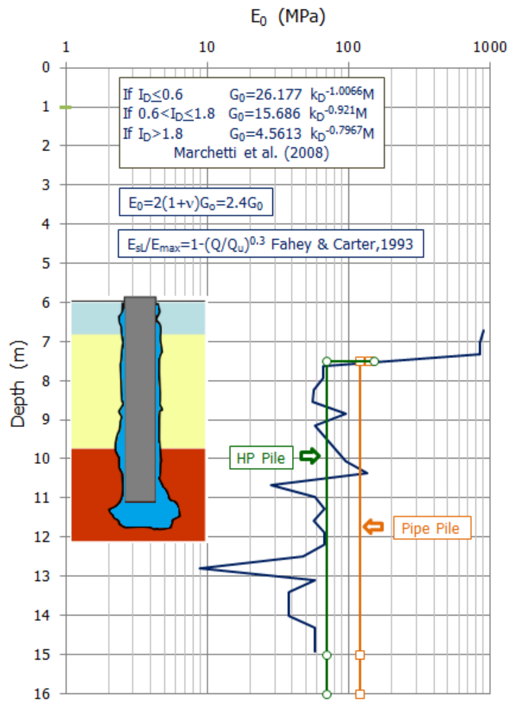

Fig.8. E_0 values by DMT

Again, the vertical displacement (w_t) of an axially compressed pile is expressed in the Fig.9 below (Mayne & Schneider, 2001).

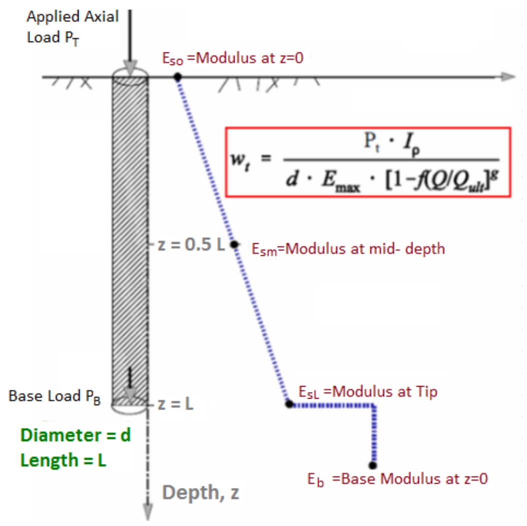

Fig.9. Load-Movement Prediction Method

The calculated and measured load-movement curves are shown in Fig.10; the anomalous measured behavior of the pipe pile results appears clearly, as movements caused by the load test after 2 and 5 weeks (up to 700 kN) do not fall into line with those of the load test after 43 weeks from installation.

This may be due to the dissipation of excess of pore pressures caused by driving and/or strength gain due to soil setup. The same phenomenon is not visible for the HP pile in which the driving causes less pile displacement.

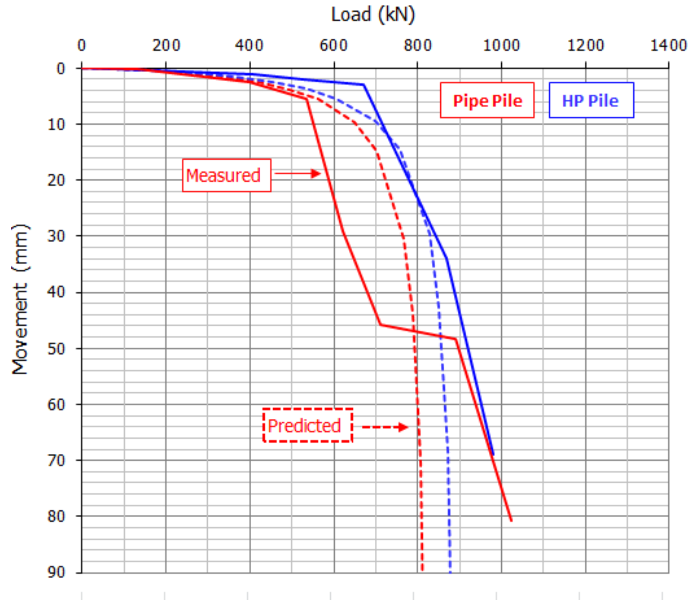

Fig.10. Load-Movement curves by DMT

Up to now, no comment has been made on the bored piles, which were an integral part of Evanston test program; due to the impossibility of defining the actual, in-place pile diameter from the presented data, which depends on the drilling technique, on soil type (particularly for the section in clay), and also on the professional skill of the driller.

As a confirmation of this, the load test results have indicate a capacity much higher than that of the driven piles when, which under the same conditions regarding diameter and length, the contrary should be true.

However such tests, performed similarly as for the driven pile after 2, 5 and 43 weeks after installation, have contributed to complete the evolutionary picture of the capacity function of time, thus making it possible to draw a worthy series of considerations of interest for the writers.

All of the piles show capacity increase with time due to soil setup; by use of the Augustesen et al. (2006) equation, it was in fact possible to construct a complete graph of the capacity function of time for each type of pile (Fig.11).

In the case of the bored piles, the interpolating straight line does not take into account the measured results at 5 weeks which appear anomalous, as there being no certain explanation for the possible reduced capacity at 43 weeks (possible restoration of original excess pore pressure and subsequent softening?), however, in the case of the driven piles the interpolating lines connect almost perfectly all the measured values.

By reviewing the graph in Fig.11, after calculating the capacity of the driven piles with either DMT and CPTu data, it can be seen that the reference time is 60 days for both the pipe pile and the HP pile using DMT, and 70 and 95 days respectively using CPTu.

The principles of this proposal, using the experiences of Hotstream and Schneider (2012) on the interdependence between the f_s value and the phenomena of soil setup, are based on the presupposition that the increase in the capacity with the passing of time is bound to a virtual increase of the length of pile, mainly as a function of the thickness and characteristics of the soil having clayey behavior.

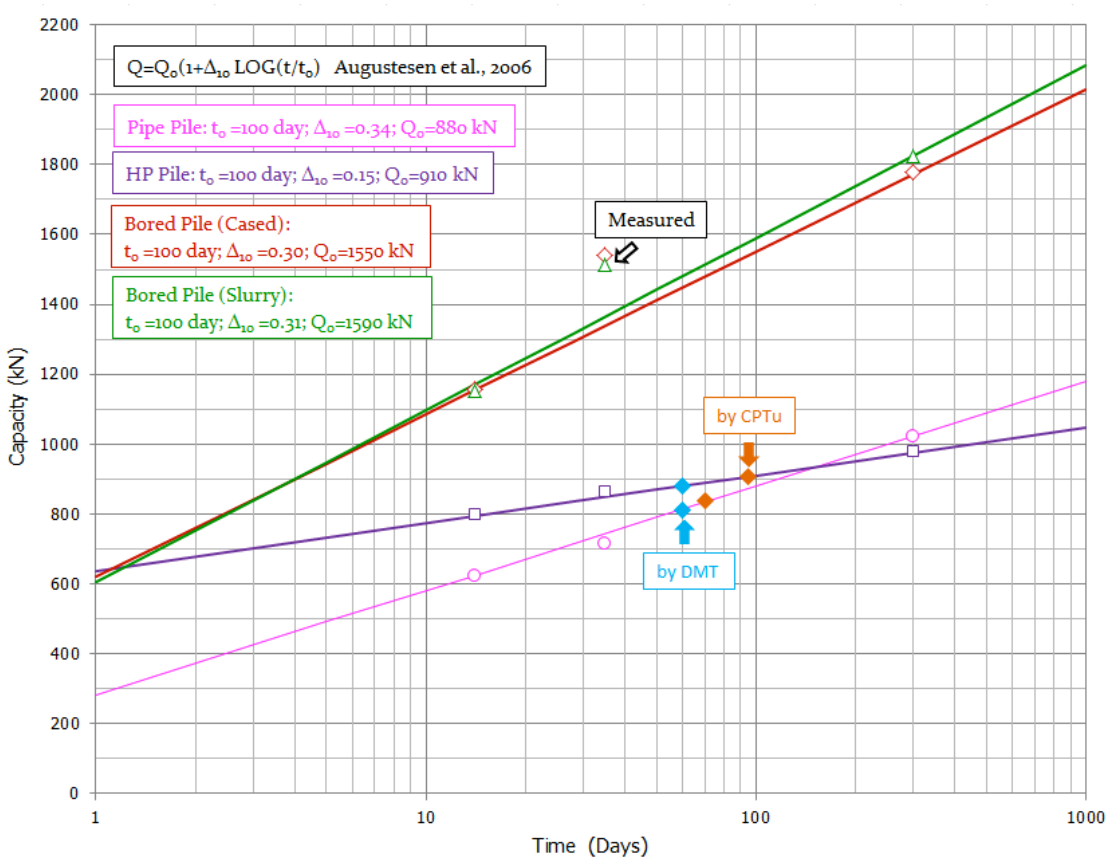

Fig.11. Pile Capacity vs Time

The possibility of being able to anchor a capacity prediction at a time after the pile installation is important because, besides allowing for a credible comparison between various methods of calculation (Togliani and Reuter, 2014), it also allows for an estimate regarding the capacity to be assigned to the pile in the initial design phase.

The identification of the capacity at 100 days is, however, the larger objective, not only for the implication in the cost reduction that usually derives from it, but also as this datum serves in fact as a comparison term to judge the validity of the method proposed by the authors to predict from a DMT or a CPTu in-situ test the pile capacity at 100 days.

This can be performed once the result of load test, static or dynamic (with static tests generally carried out 3 or 4 weeks after the end of driving) are available, as an alternative, and/or integration into, and/or control comparison to the Svinkin & Skov, Bullock et al., Augustesen et al, methods.

The first version of the proposed method, intended to be a simple attempt by the authors as presented in their previous paper (CPT'14,) has been modified by inserting a "time factor" into some of the terms.

The calculation sequence is as follows:

- subdivide the soils into unit layers (UT), which for CPTu is obtained from I_c and FC_{mean} and also, for clayey soils only, from f_s while for DMT the subdivisions are based solely from I_D, as shown in Table 2;
- a virtual multiplier "α" (Table 2) which is bigger than unity for each identified layer, whose real thickness (RLT) is therefore increased by α, creating a unitary virtual length (VLT);
- an adjustment coefficient "χ" which depends on the type of analyzed pile, the value of FC_{mean} and the kind of load test from which the capacity was derived;

- a time factor that is equal to "0.9 - T/100”, where T is the elapsed time between initial pile installation and the load test;
- a final multiplier (FM), which depends on the summation of the virtual lengths (ΣVLT), the pile real length (PL), the virtual thickness (CVL), and the real thickness, (CRL) of the soil with clayey behavior, is then applied to the capacity measured with the load test (Q_{SLT}).

Table 2. DMT/CPTu Subdivisions

ID	Ic	FC/100	f_s (kPa)	α
≤ 0.1	≥ 2.67	≥ 0.6	≤ 10	3.00
≤ 0.2			≤ 20	2.00
≤ 0.6			≤ 30	1.50
≤ 0.8			> 30	1.20
≤ 1.8				1.10
>1.8				1.05

The equations for virtual pile length, final multiplier and the capacity at 100 days are:

$$VLT = \chi RLT \alpha (0.9\text{-}T/100) \quad (8)$$

$$FM = (\Sigma VLT/PL)^{CVL/CRL} \quad (9)$$

$$Q_{100} = FM\ Q_{SLT} \quad (10)$$

The example shown in Table 3, which uses the pile capacity prediction at 100 days by DMT from pile load test results at 2 and 5 week, shows that the application of the proposed method is relatively simple.

The next example (Table 4) based on the results from the Evanston site, offers the opportunity for the following considerations:

- the difference between the predicted and measured capacity is generally less than ±10%, which is acceptable;
- the precision of the prediction is decidedly remarkable for both driven piles in the case of a chosen Q_{SLT} at 35 days;
- the adjustment coefficient "χ" seems to change in accordance with the degree of soil disturbance due to the pile installation; the same also for the slope Δ_{10} of the interpolating setup line.

We can see the same effect also in the value of the one day cut off point (Fig.11): for the steel pipe pile the value is about half that for the HP360 pile, which is nearly the same as the bored piles; however, where Δ_{10} for the bored piles is double the result of the HP 360 and is comparable with the result for the steel pipe pile.

Table 5 (from Augustesen et al., 2006) shows that the Δ_{10} values in Table 4 are systematically out of

Table 3. Q_{100} by DMT (Pipe Pile CE)

PL	UT	ID	RLT	α	SLT	TF	VLT	Σ VLT	FM	Q_{SLT} Meas.	Q_{100} Pred.	Q_{100} Refer.	Q_{100} Pred./Refer.
(m)	(m)		(m)		(days)					(kN)	(kN)	(kN	
15.24	7.92	≤ 0.1	1.52	3.00	14	0.76	2.98	2.98					
		≤ 0.3	6.10	2.00			8.78	11.76					
		≤ 0.6	0.30	1.50			0.35	12.11					
		≤ 0.8	0.00	1.20			0.00	12.11					
	0.00	≤ 1.8	0.00	1.10			0.00	12.11					
	7.32	> 1.8	7.32	1.05			7.60	19.70	1.48	623	922	880	1.048
15.24	7.92	≤ 0.1	1.52	3.00	35	0.55	2.36	2.36					
		≤ 0.3	6.10	2.00			7.59	9.96					
		≤ 0.6	0.30	1.50			0.32	10.27					
		≤ 0.8	0.00	1.20			0.00	10.27					
		≤ 1.8	0.00	1.10			0.00	10.27					
	7.32	> 1.8	7.32	1.05			7.52	17.79	1.22	713	872	880	0.991

Table 4. Comparisons

Pile Type	Pile Diameter (m)	Pile Length (m)	Clay like soil DMT (m)	CPTu (m)	FC_{mean} (%)	χ DMT	CPTu	Δ_{10}	Q_{100} Refer. (kN)	Q_{100} (DMT) Pred. (kN) 14 (days)	35	Q_{100} (CPTu) Pred. (kN) 14 (days)	35
Pipe CE	0.46	15.24	7.92	7.92	97	0.85	0.90	0.34	880	922	872	876	861
Bored	0.61/0.46	15.24	7.92	7.92	97	0.75	0.80	0.30	1550	1473	1667	1426	1661
Bored	0.46	15.24	7.92	7.92	97	0.75	0.80	0.31	1590	1473	1667	1426	1661
HP	0.46	15.24	7.92	7.92	97	0.70	0.75	0.15	910	958	908	935	889

range, which may be because the piles were installed partially in soils not of natural origin.

Table 5. Δ_{10} Mean Values and Confidence Interval

Loading	Time t_0 (days)	Δ_{10} mean	Lower Limit (95%)	Upper Limit (95%)
Staged	100	0.24	0.20	0.29

This irregular behavior may be tied, especially in the sands of hydraulic fill, also to a phenomenon of a chemical nature (increased shaft friction due to oxidation of the steel), as is evidenced by the example photograph of Fig.12 which shows oxidation, below and above GWT, on the exterior of a steel water well after dewatering.

Fig.12. Steel Oxidation

The above considerations point out that, for the moment, the proposed method obviously needs further refinement.

2 PORTO EVENT (2004)

At the experimental field site at the University of Porto, in Porto, Portugal, piles of different types and diameters were installed during the months of August and September 2003 for a "Pile Prediction Event" competition opened to the participants of ISC'2 held the following year.

The characteristic of the piles were:

- C1 [prefabricated concrete driven pile, 0.35 cm square with length of 6 m];
- E9 [bored pile with temporary casing, 0.6 m diameter with a length of 6 m, instrumented with strain gauges and load cell];
- T1 [CFA pile with diameter and length equal to E9 pile, instrumented with strain gauges].

The arrangements of the test piles and of the anchor piles used for the performance of the load tests, carried out over 4 months after installation (January 2004), the same piles after the extraction, and the grain size analysis of the soils in the area, are all shown in Figs 13, 14, 15, which were taken from the book written on the subject by Viana de Fonseca et al. (2008). The soils are predominately residual sands formed by the weathering of the underlying granite bedrock. The water table was present at about 9 meters depth.

Fig.13. Piles layout

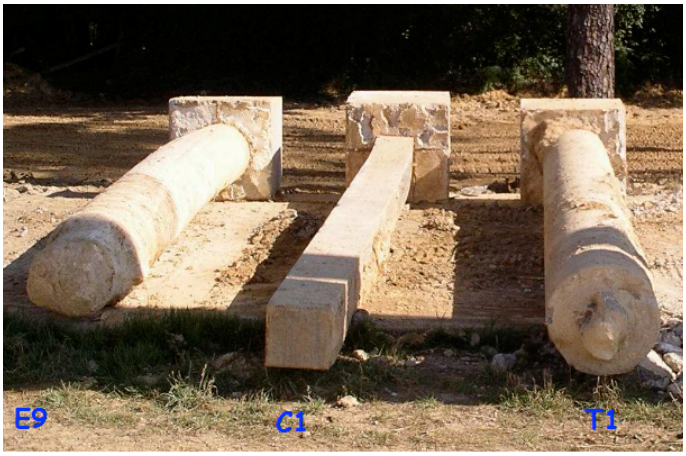

Fig.14. Piles after extraction

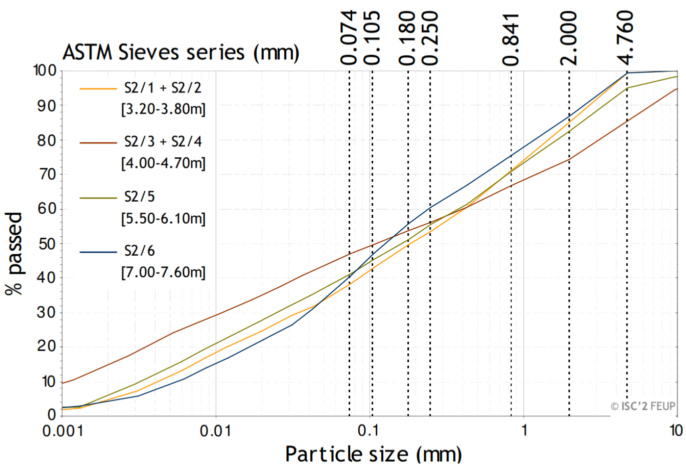

Fig.15. Grain size distribution curves

A series of in-situ tests were performed for the characterization of the residual sands, among them CPTu and DMT, which were both carried out before and after the installation of the piles.

The trend with the depth of the basic parameters chosen for the calculation of the pile capacity is shown in Fig.16.

In the same Fig it can be seen that in the graph the values of f_s are on average above 200 kPa, probably because the residual soils retain the original relic structure; as a consequence, the R_f values belongs more to a silty clay then to a sandy silt (Fig.17); this is also indicated by the values of K_D, which identify overconsolidated soils (Fig. 18).

The behavior of the residual soils, on the contrary, is correctly identified in both the values of FC_{mean} and I_c from CPTu (Fig.19) and I_D from DMT (Fig.20).

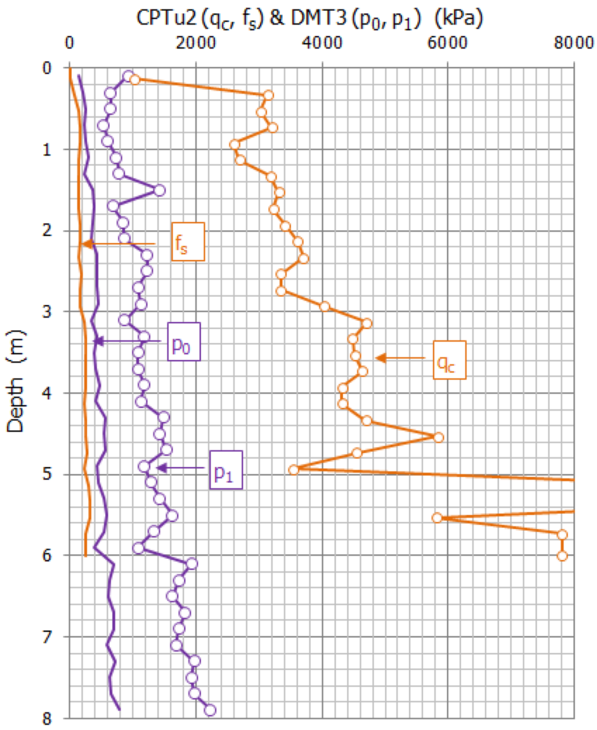

Fig.16. DMT and CPTu basic plots

The capacity of the piles from the CPTu have been calculated with the same equations used for Evanston; from DMT, however, considering the special ground features (p_0 and p_1 are certainly more sensible than q_c and f_s) and the remarkable differences which exists between p_1 and q_c, the following equations have been used:

- DMT (I_D is always > 0.6):

$$q_s = \beta p_0^{0.83} \quad (11)$$

$$q_b = p_{1,base}\,[\lambda + (0.05\,L_{pile}/d_{toe})] \quad (12)$$

where p_1 is measured from +8 d_{toe} to -4d_{toe}.

(Note, the reference capacity is for the driven pile C1, for the bored piles E9 and T1 reference should be made to values in Table 1.)

The capacity calculated from the two in-situ tests compare very well also for the Porto site (Fig.21), with values very near to those measured when, in the load-movements curves, the capacity is chosen for a settlement of pile head equal to 10% of pile diameter (Fig.23).

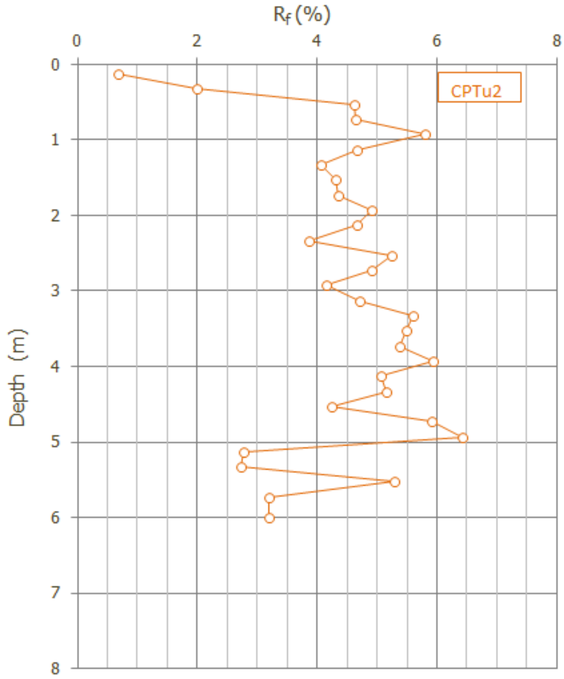

Fig.17. R_f plot

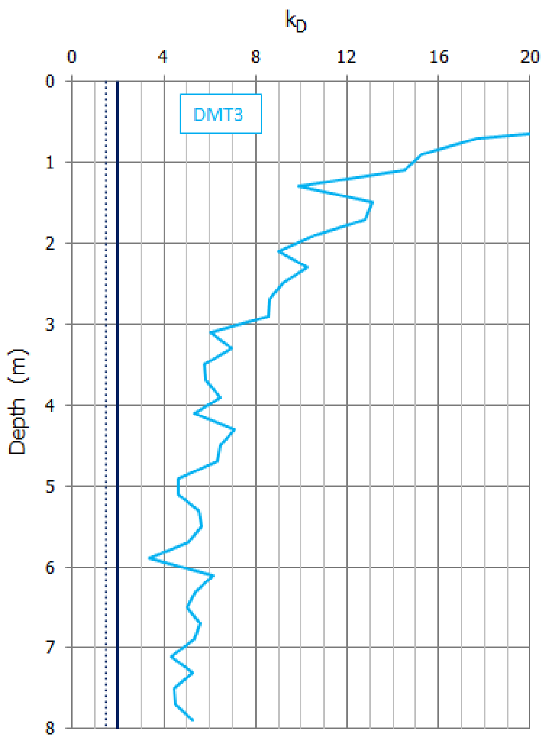

Fig.18. k_D plot

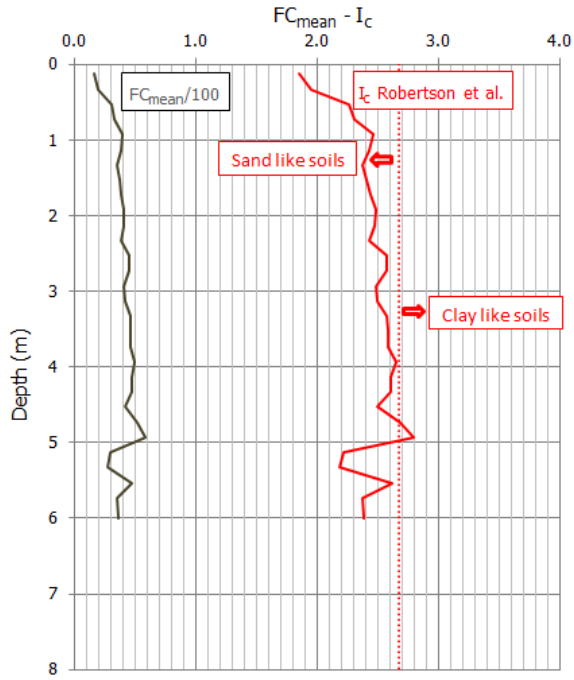

Fig19. FC, I_c plots (CPTu2)

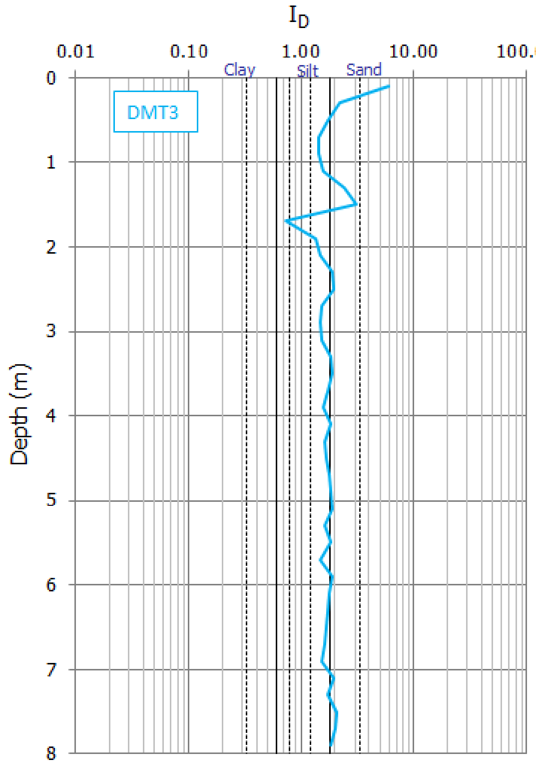

Fig.20. I_D plot

Is necessary to add that changes to E9 and T1 by the control instrumentation inserted into piles has not taken into account, except for what provided into the square at bottom of Fig.21.

The values of E_0 from DMT and those employed for the load-movement simulation, using the same criterion adopted for Evanston, are presented in Fig. 22.

Setup has not taken place; the dynamic load test carried out on pile C2 (hidden in Fig.13 by pile E7) a few hours after driving, gives similar results to the ones of the static load test on pile C1 (with same characteristics) performed four month after driving (Fig. 24, Viana da Fonseca et al., 2008).

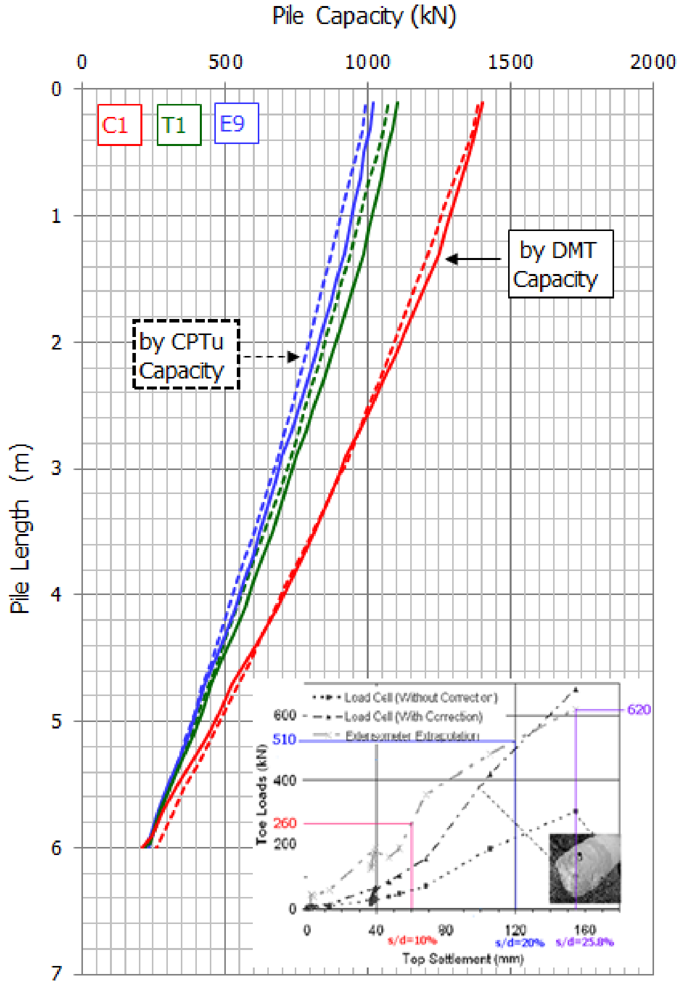

Fig.21. Piles capacity

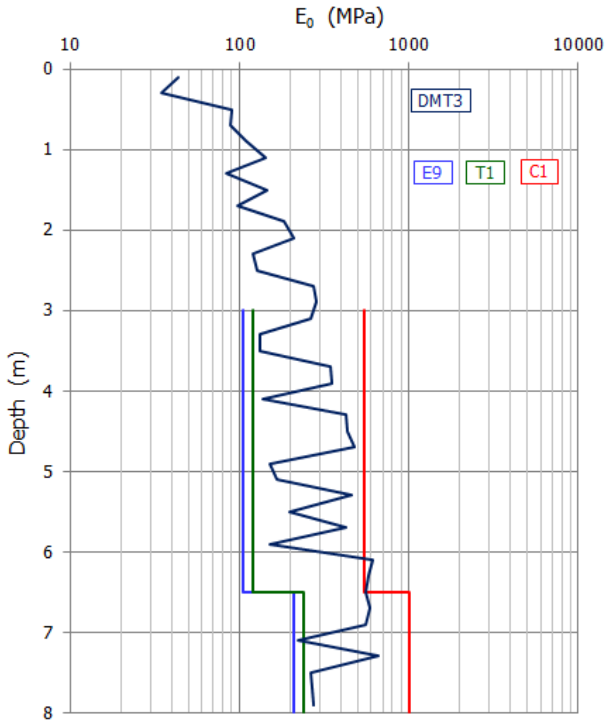

Fig. 22. E_0 values by DMT

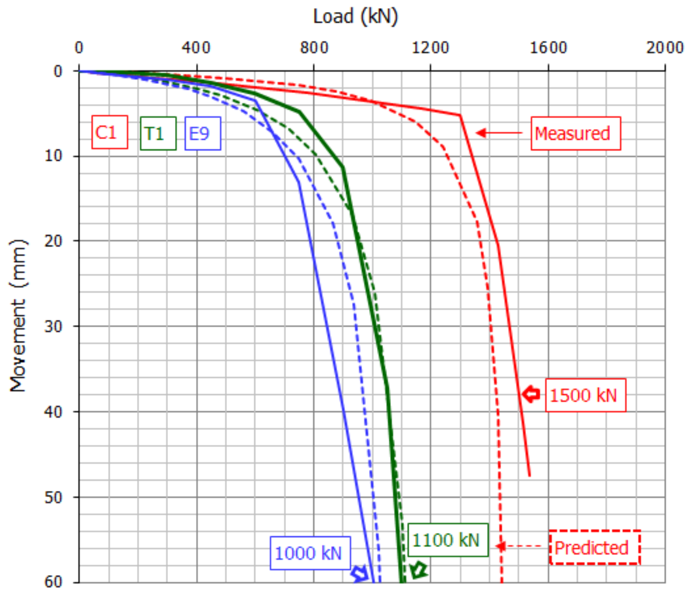

Fig.23. Load-Movement curves

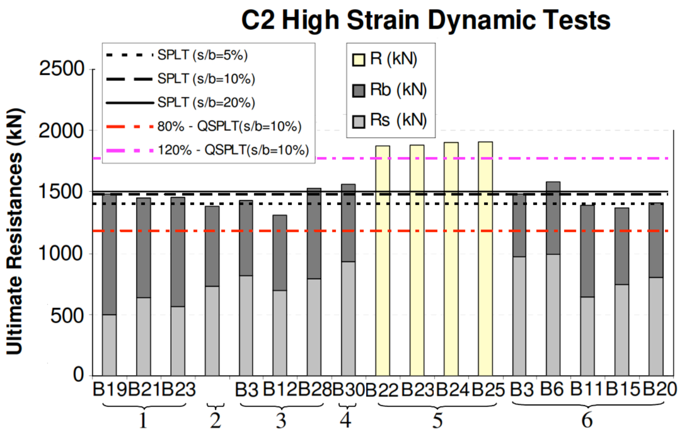

Fig.24. Dynamic Loading Test

3 CONCLUSIONS

The values of p_0 and p_1 from the dilatometer, together with the intermediate parameters I_D and K_D, have allowed the development of some equations that have proved to be effective to predict the pile capacity in the experimental sites of Evanston and Porto, and to simulate the load-movement curves with sufficient approximation.

In the example cases, the obtained DMT capacities superimpose very nearly on those derived from CPTu, a worthy note because it reveals that the combined use of the two described methods can improve the prediction quality (an afflicted point in our profession).

However the availability of case histories with DMT, CPTu and pile load tests is still too limited to evaluate the real potentiality of this combination.

The situation could radically change if more reliable correlations between the two in-situ tests could be found, deepening the Roberston (2009) studies, to be used with access to a data bank of "CPTu tests - pile load tests" to utilize available results for further analysis.

Only for Evanston site where the lower section of piles are in clay and the results of load tests at 2, 5, 43 weeks after the pile execution are known, it was possible to predict the capacity of piles after 100 days (as recommended by the authors to be used as the reference time when evaluating soil setup).

4 REFERENCES

Augustesen, A.H. 2006. The Effects of Time on Soil Behavior and Pile Capacity. *DCE Thesis No. 4, Aalborg University Department of Civil Engineering Division of Water & Soil.*

Fahey, M. and Carter, J. 1993. A finite element study of the pressuremeter using a nonlinear elastic plastic model. *Canadian Geotechnical Journal 30(2), 348-362*

Finno, R. J. et al. 1989. Predicted and Observed Axial Behavior of Pile. *ASCE.Geotechnical Special Publication No.23.*

Fleming, W.G.K., et al. 1985. Piling Engineering. *Surrey University Press , Wiley &Sons, New York.*

Hotstream, J.N., and Schneider, J.A. 2012. Piezocone sleeve friction setup in low plasticity clays of Green Bay, Wisconsin, USA. *In Coutinho and Mayne (eds.), Proceedings of ISC'4, Brazil,* Taylor & Francis

Ku, C.S. et al. 2010.Reliability of CPT Ic as an index for mechanical behaviour classification of soils.*Geotecnique 60*

Marchetti S. et al. 2008. In Situ Tests by Seismic Dilatometer (SDMT). *GSP No. 270*, 2008

Mayne, P. W. and Niazi, F.S. 2013. Cone Penetration Test Based Direct Methods for Evaluating Static Axial Capacity of Single Piles. *Springer*

Mayne, P. W. and Schneider, J.A. 2001. Evaluating Axial Drilled Shaft Response by Seismic Cone. Foundation and Ground Improvement, *GSP 113, ASCE*, Reston/VA: 655-669.

Randolph, M.F. and Wroth C.P.1978. Analysis of deformation of vertically loaded piles. *Journal of the Geotechnical Engineering Division ASCE.* Vol.104 (GT12),1465-1488

Randolph, M.F. and Wroth, C.P: 1979. A simple approach to pile design and the evaluation of pile tests. Behavior of Deep Foundation, *STP 670, ASTM*, 484-499

Robertson, P.K. 2009 "CPT-DMT Correlations," *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, Vol. 135, No. 11, November, 2009, pp. 1762-1771

Robertson, P.K. 2012 " Guide to Cone Penetration Testing," Gregg.

Svinkin, M.R. and Skov, R.2002.Set-up Effect of Cohesive Soils in Pile Capacity. Vulcanhammer.net. *Online Report*

Togliani, G. and Reuter, G.R.2014.CPT/CPTu Pile Capacity Predicton Methods: Question Time. *Proceedings CPT'14. Las Vegas*, May 12-14, 2014.

Togliani, G. 2008. Pile Capacity Prediction for in Situ Tests. *Proceedings ISC-3. Taiwan*, April 1-4, 2008. 1187-1192. Taylor & Francis Group, London, UK

Viana de Fonseca A., and Santos, J. 2008. International Prediction Event Behavior of Bored, CFA and Driven piles. ISC'2 Experimental Site. *FEUP*

Yi, F. 2010. Case study of CPT application to evaluate seismic settlement in dry sand. In Robertson and Mayne (eds.), *Proceedings of the 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA*.