Soil Behavior and Pile Design: Lesson Learned from Recent Prediction Event - Part 2: Unusual NC Soils

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ABSTRACT: Pile design methods generally do not consider that the presence in the soil surrounding the piles of a microstructure related to phenomena such as aging, cementation and weathering, could significantly affect their performance. Analyzing soils classified as NC with reference to the CPT-based SBT classification system recently updated by P.K. Robertson, it is possible to highlight, via the normalized small-strain rigidity index $K^*_{G,}$ the presence of a microstructure and to define his impact on piles design.

1 INTRODUCTION

Tables summarizing soils and piles characteristics of Santa Cruz (2017), the prediction event chosen for the part 2 of the paper, are provided below. The following CPTu plots contain the data used for the piles design.

At first glance the Santa Cruz alluvial layers appear, as stated, normally consolidated (e.g. see the u_2 curve and compatibility between derived and lab. FC values in Figures 1 and 2); however, their K^*_G values run around the upper limit of sedimentary soils, demonstrating that, even if of recent deposition, these mainly sand-like soils have been influenced by some phenomenon that has increased their stiffness (Figure 3).

Pairs of measured values should be used to calculate K^*_G, but being too few those of V_s, it is reasonable to employ derived values since the V_s curves (measured/predicted), are close enough to each other (Figure 1).

Table 1. Santa Cruz: Soil Details

Deposit:	Alluvial (Holocene)
Lithology	Layers of sand, silt and clay
Soil structure from	Chemical Diagenesis?
GWT	-2.2 m
FC	12/53 %

Table 2. Santa Cruz: Pile Details

Pile Type:	Driven (C2) Drill Disp.	Bored (A3) Bentonite	Bored (B2) CFA
Material:	Concrete	Concrete	Concrete
Shape:	Round	Round	Round
Diameter (m):	0.45	0.62	0.45
Length (m):	9.5	9.5	9.5
Cast in situ Date:	March 07	March 08	March 11
SLT Date:	March 25	March 20	March 23

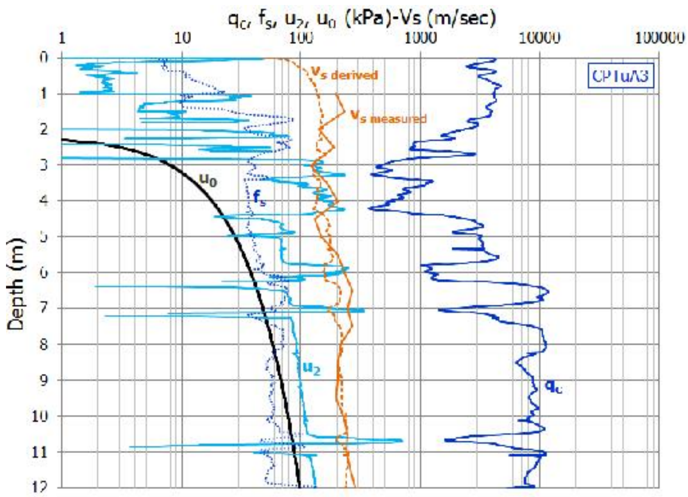

Figure 1. CPTu basic plots

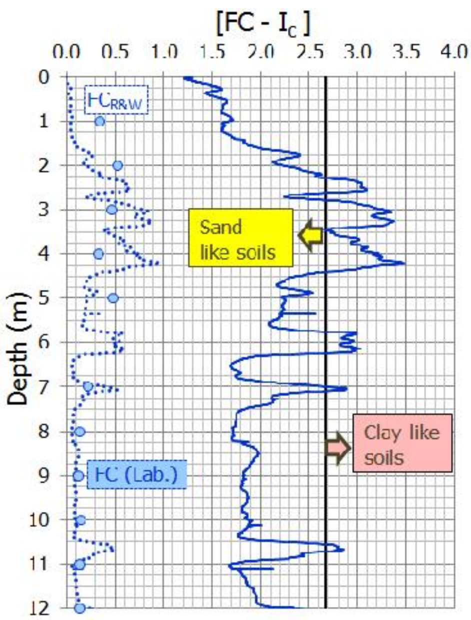

Figure 2. Soil Behaviour

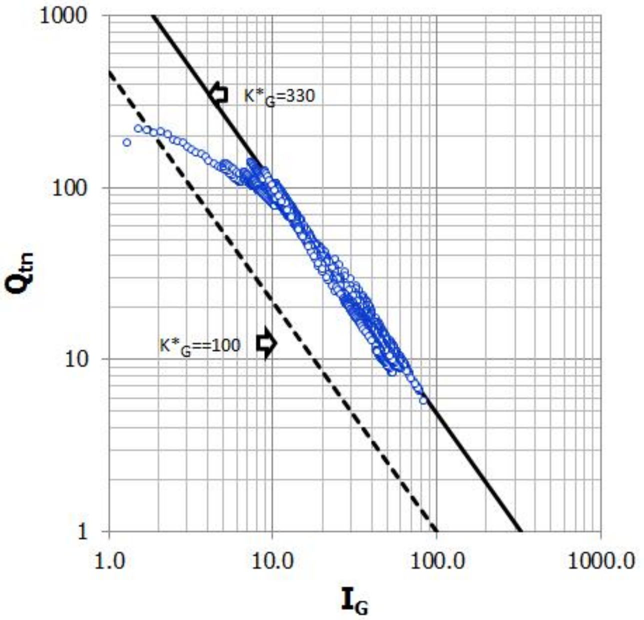

Figure 3. K*G plot

Meaning of parameters appearing in the previous figures is provided in the glossary reported at §8.

2 PILE CAPACITY DESIGN METHODS

To verify their sensivity to the presence of a soil structure, the CPT based direct design methods (LCPC, Eslami&Fellenius, KTRI and Togliani) described by Niazi and Mayne (2013) have been used.

The Author's method, updated just before the prediction, now focuses on the following equations:

2.1 *Unit Friction (q_s)*

$q_s=\beta\, q_c^{0.4}$ if $f_s \leq 20$ (1)
$q_s=\beta\{q_c^{0.52}[(0.4+LN(R_f)]\}$ if $R_f \geq 1.5$ (2)
$q_s=\beta\{q_c^{0.51}[0.8+(1-R_f)/8]\}$ if $1 \leq R_f < 1.5$ (3)
$q_s=\beta\{q_c^{0.53}[0.8+(1.1-Rf)/8]\}$ if $R_f < 1$ (4)

2.2 *Unit Base (q_b)*

$q_b=q_{c\ toe}[\lambda+(0.005Lpile/Dpile)]$ (5)
where $q_{c\ toe}$ goes from +8 dtoe to -4dtoe

Table 3 summarizes the corresponding Pile Type coefficients:

Table 3. β and λ coefficients

Pile Type	β	λ
Driven (precast/jacked)	1.00	0.30
Drill.Displacement.	0.90	0.25
Pipe (Open End)	0.70	0.20
CFA, Bored (cased-cohesionless, bentonite)	0.60	0.15
Bored (bentonite-upper bound)	0.60	0.10
Bored (cased- cohesive)	0.50	0.10
Bored (bentonite lower bound)	0.40	0.05

Capacities derived from the four quoted methods were then compared with the ones derived from the method specified below, including the normalized rigidity index K^*_G.

2.3 *Unit friction (q_s)*

$q_s=\beta q_c^{[0.3+x]}$ (6)
where $x=LOG(K^{*\alpha}_G)$

2.4 *Unit base (q_b): same of § 2.2*

The following table shows the Pile Types coefficients developed for the new method (Class C Predictions), significantly different from the previous ones; specifically for λ, values from two (B2 an C2) to three times (A3) higher are obtained.

A reasonable explanation is the fact that in Santa Cruz the embedment into the bearing layer for the estimate of the toe resistance (from + 8d to -4d), is entirely (C2 and B2 piles) or almost entirely (A3 Pile) occurring in a medium dense to dense sand characterized by fair values of the cone resistance.

Table 4. α, β, λ coefficients

Pile Type	α	β	λ
Drill Displacement - Santa Cruz (C2)	0.10	0.90	0.60
CFA - Santa Cruz (B2)	0.10	0.70	0.40
Bored (Bentonite) - Santa Cruz (A3)	0.10	0.40	0.15

3 LOAD-MOVEMENT PREDICTION

The Chen & Kulhawy criterion (2002), shown in Figure 4, has been used to choose the force/movement distribution for every pile; subsequently the corresponding predicted curve was set via the Ratio Function.

According to this Function, the equation for unit resistance at a given movement in relation to the target resistance and target movement, is expressed by the following equation (B.Fellenius-Red Book):

$r=r_{trg}\,(\delta/\delta_{trg})^{\theta}$ (7)
where r= variable force; r_{trg} =target resistance
δ= var. movement δ_{trg} = movement at r_{trg}
θ=function coefficient (0<θ<1)

The resistances derived with the new design method were considered as the target ones while the corresponding movement was set at s/d=10% to obtain a complete load-movement curve. For Santa Cruz, the cohesionless soil curve of Figure 4 was adopted.

The superposition between measured and predicted load movement diagrams is obtained by adjusting the value of the ϑ coefficient within reasonable limits until a satisfying agreement was obtained; hence the theoretically evaluated curves are of Class C.

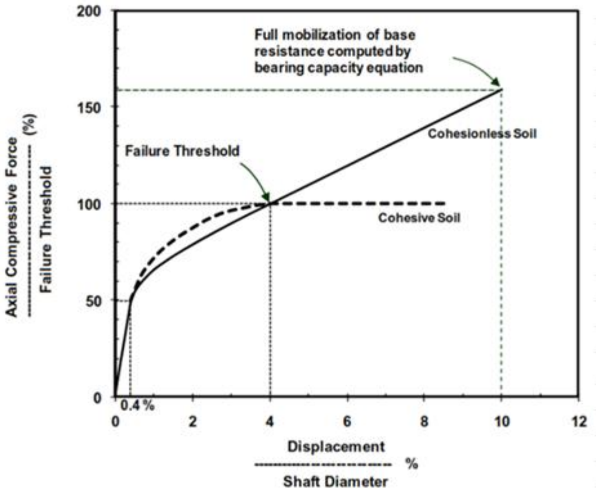

Figure 4. Chen & Kulhawy Criteria

4 SANTA CRUZ PILES DESIGN

The incomplete elimination of the “bentonite cake” during concreting has lowered the shaft resistance of A3 pile (Figure 5); this is why the new method used a β coefficient equal to 0.40 (Table 3 & 4).

In Figure 5, the LCPC capacity is too high, while the Togliani method (2008/2017) with β and λ coefficients averaged among upper and lower bound (Table 3), shows a reduced toe resistance for the reason given at §2; the same, obviously occurs for B2 and C2 piles.

The function coefficient used for the predicted load-movement curve of A3 pile (Figure 6) is equal to 0.4, also in relation to a very likely imperfect toe cleaning.

For B2 and C2 piles (Figure 8 and Figure 10), the ϑ coefficient is equal to 0.32 and 0.25, respectively, showing an increasingly rigid response of the soil may be due to the progressive increase in the soil displacement caused by the different technique of execution.

Surprisingly, for the CFA pile (B2 in Figure 7), the LCPC method provides satisfactory results for both components (cast screwed piles was the choice), while this time both KTRI and Togliani methods lead to a totally inadequate shaft resistance. Very likely, in this case, their design is affected by weak values of f_s due to a structure rearrangement occurred during the CPTu execution.

For the drill displacement pile (C2 in Figure 9), the LCPC method provides a shaft resistance unexpectedly half the one of pile B2 and a third of the actual one, in spite of having chosen, as a guideline, the driven cast pile type effectively carried out.

Also shaft resistances provided by KTRI and Eslami&Fellenius methods are severely inadequate, once again due to reduced f_s values; the same problem affects, albeit less, the Togliani method.

As a last note, related to the load-movement curves, it has to be stressed that the choice of pile capacity at s/d=10%, is purely conventional.

In fact this choice should be made, regardless of the pile diameter at a standard movement of, for example, 30 mm as suggested by Fellenius et al. (2017).

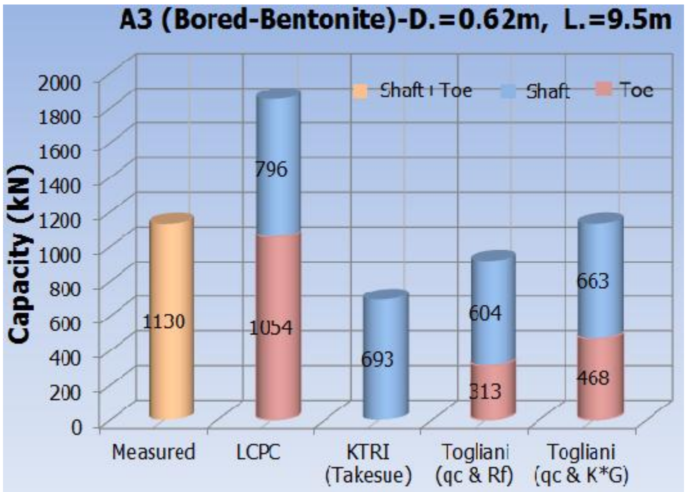

Figure 5. A3: Pile Capacities Comparison

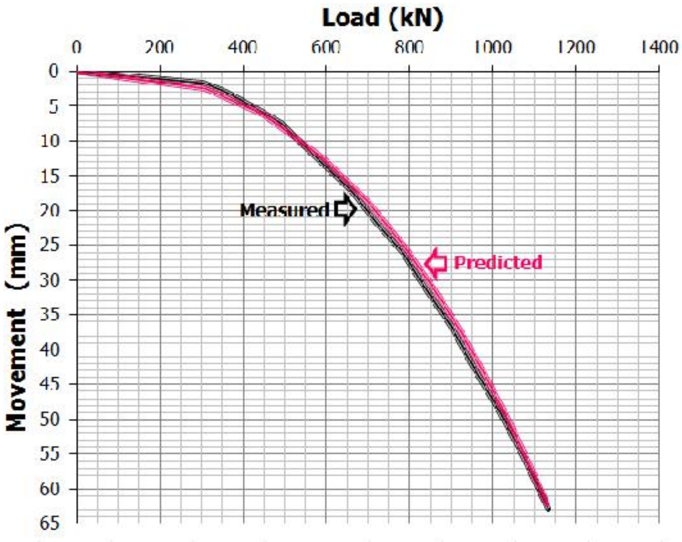

Figure 6. A3: Load-Movement Comparison

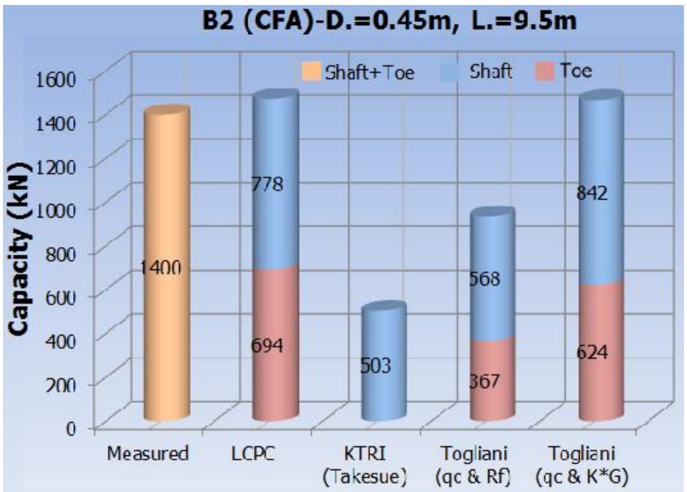

Figure 7. B2: Pile Capacities Comparison

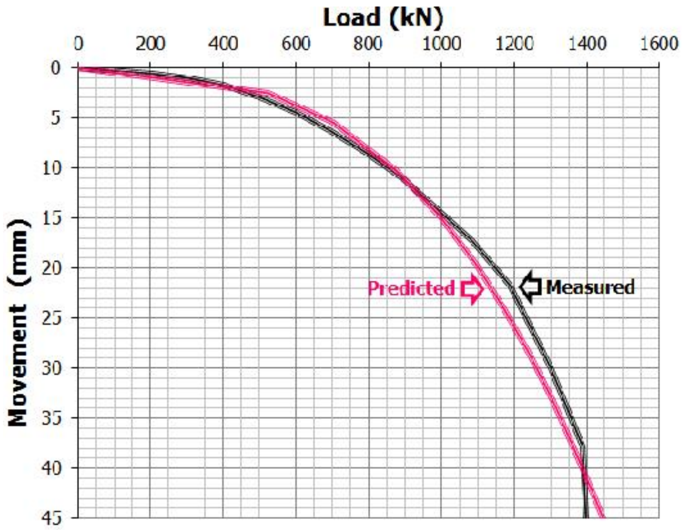

Figure 8. B2: Load-Movement Comparison

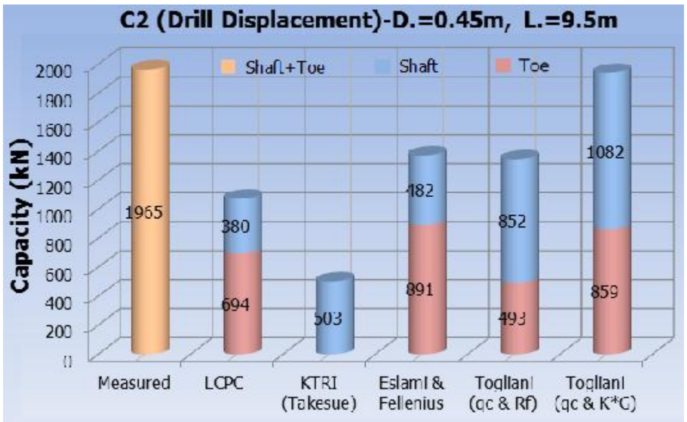

Figure 9. C2: Pile Capacities Comparison

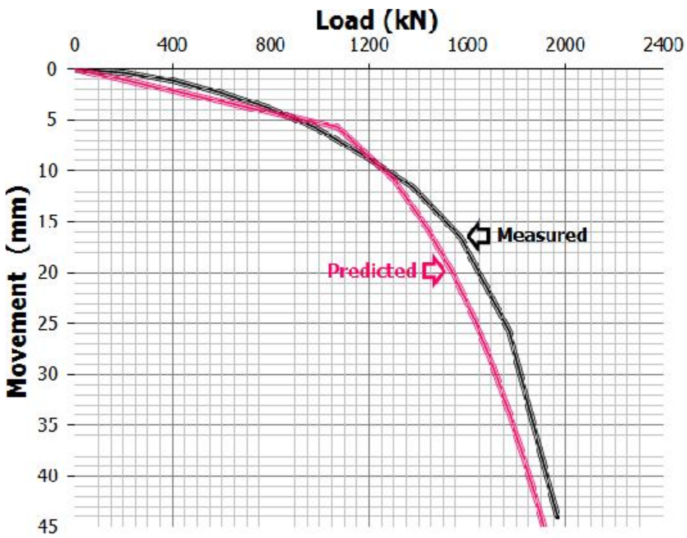

Figure 10. C2: Load-Movement Comparison

5 CONCLUSIONS

The sole cone resistance, despite being the most reliable among the measured CPTu parameters, does not allow to understand the possible dilatant behaviour of a micro-structured soil; consequently, at least in these instances, the LCPC method, providing random results, appears not completely reliable.

Design methods giving higher priority to f_s values (KTRI and Eslami& Fellenius), in this case are negatively affected and therefore not suitable, by f_s values often reduced probably as a result of the break/loosening of interparticle bonds caused by the cone being located in front of the friction sleeve.

The Togliani method (2008/2017), using the coefficients of Table 3 (Class A Prediction), provides too low pile capacities [respectively -24% (A3), -33% (B2) and -32% (C2)] being the most likely reason an inadequate choice of λ values.

Replacing them with the "consistent" values of Table 4 and, accordingly, using toe resistances obtained from the new method the approximation would be much better [-5% (A3), -15% (B2) and -13% (C2), see Figures 5, 7 and 9].

One could then argue that the Togliani method (2008/2017), updated for the λ values and with the proposed q_c and R_f combinations, is able to characterize in a reasonable, balanced way the mechanical behaviour of NC, micro-structured soils.

Finally, the q_c, K^*_G pair, as proposed in the new design method, provides appropriate and promising results, however to be confirmed with additional, future analyses.

6 AKNOWLEDGMENT

Many thanks are addressed to Prof. P.K. Robertson, and to Dr. L. Albert for their critical review and suggestions.

7 REFERENCES

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8 GLOSSARY

z= depth
γ_n =natural unit weight
q_c= cone resistance
f_s= sleeve friction
u_2= pore pressure measured behind the cone
q_t =corrected cone resistance=$q_c+u_2(1-a)$
R_f= friction ratio=$(f_s/q_t)100$
σ_v= vertical stress= $z\gamma n$
u_0=hydrostatic pore pressure
σ'_v=effective vertical stress= σ_v-u_0
σ'_p=effective max. past vertical stress
OCR= overconsolidation ratio=σ'_p / σ'_v
q_{tn}=net corrected cone resistance=$q_t-\sigma_v$
Q_{tn}= normalized cone resistance=$[(q_t- \sigma_v)/P_a][P_a/\sigma'_v]^n$
I_c= classification index (P.K. Robertson)
$FC_{R\&W}$= Fine content (Robertson & Wride)
$I_c >3.5$ FC=1, $1.31<I_c<3.5$ FC=$1.75I_c^{3.25}-3.7$, $I_c<1.31$ FC=0
CF= Clay Fraction
Vs=shear wave velocity
V_s derived-Baldi (1992) mod. Togliani=
$277q_c^{\ 0.12}\sigma'_v{}^{0.18}$ if $\sigma'_v<100$ kPa otherwise = $277\ qc^{\ 0.13}\sigma'_v{}^{0.22}$
G_0= maximum .shear modulus =ρV_s^2
I_G= small-strain rigidity index =(G_0/q_{tn})
K^*_G=normalized small-strain rigidity index=$Q_{tn}^{0.75}I_G$

Ed.: Symbols, definition and reference equations are mainly derived from the CPT Guide (P.K. Robertson, 2015), the Manual of Subsurface Investigation (P.W. Mayne, 2001) and the CPT State of Practice Report (P.W. Mayne, 2007).