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Effects of residual stresses on shaft friction of bored cast *in situ* piles in sand

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Abstract. The existence of residual stresses locked-in prefabricated displacement piles is a well-known problem and has been addressed by a number of researchers in the last decades. This is not the case with cast *in situ* piles: as a consequence of concrete curing, pile-soil interaction starts soon after concrete casting, causing stress changes in terms of both normal and shear stresses. Such circumstance has been confirmed by few experimental evidences, reported in the paper, in saturated or dry soil conditions.

In order to evaluate the influence of residual stresses on the subsequent pile response to axial loading, a broad parametric study has been carried out by means of numerical modelling. Particular focus is given to the effects induced on the shaft friction of floating bored piles embedded in wet and dry sandy soils.

The results have been interpreted with the aim of highlighting errors commonly made if a stress-free pile is assumed when interpreting a specific load test results on instrumented piles and/or arranging general design methods.

Keywords: Bored pile, cast *in situ*, concrete curing, residual stresses, shaft friction, sandy soils, dilatancy

1. Introduction

The behaviour of piles is influenced by the properties of the surrounding soil, which in turn are markedly affected by the installation technique. This is valid for any combination of pile type/soil type, but it is particularly true when dealing with piles embedded in granular soils whose drained response allows for the development of volumetric strains during the construction phase.

The problem is of particular concern if piles axially loaded are under consideration and has triggered research to assess their ultimate bearing capacity. Detailed scrutiny of the latest results on this topic have been provided, among others, by [1–3].

Despite recent advances, a significant margin of error between predictions and measured values still remains (e.g. prediction events: [4, 5]; systematic collection of results of load test on piles installed with different procedures in different soils: Federal Highway Administration, USA; analyses of the experimental evidences on single piles for the assessment of existing design methods and development of new design methods for pile types not covered by current codes and regulations: Laboratoire Central des Ponts et Chaussées, France).

Full scale load test is still considered to be the most reliable tool to investigate the load-settlement behaviour of piles (e.g. [1, 6]). Some very detailed and useful information can be further detected if piles are fully instrumented along the entire length to separately obtain the shaft and base response.

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When dealing with prefabricated displacement piles (driven, jacked) it is customary to take into account residual stresses generated during the installation process and consider them when interpreting the results (e.g. [7–9]).

This does not apply to cast *in situ* piles: apart from soil changes during drilling, once the concrete is placed chemical processes occur, giving rise to contraction or expansion according to external environmental conditions (wet or dry) and to cement type. It follows that a distribution of volumetric strains along the shaft exists at the beginning of a load test, implying that pile-soil interaction starts well before the load is applied to the pile head.

Such aspect has not often been tackled (e.g. [10]); little experimental evidence is available (e.g. [11–13]), and what does exist demonstrates that the order of magnitude of these strains is the same as that which develops during the load test. By neglecting this aspect when arranging general design methods or interpreting a specific load test result gross errors can occur.

To better understand how the residual stresses due to concrete curing process alter the response of pile to axial loading (with particular attention to the shaft resistance of bored cast *in situ* single piles in granular soils), the results of a broad parametric studies carried out by FLAC 2D are presented. Some practical, operational procedures are also suggested.

2. Concrete curing

Cement hydration is an exothermic chemical reaction where the specific volume of products (Calcium, Sulphur, Hydrogen and Calcium Hydroxide) is lower than the initial volume of reactants (clinker and water). At the macroscopic scale, however, the volume of the material sometimes reduces and sometimes swells: this illusory contradiction is nothing but an apparent process.

Drying shrinkage may be defined as the volume reduction that concrete experiences as a consequence of moisture migration when exposed to a lower relative humidity (RH) environment than that initially in its pore system. For workability purposes the amount of water added to the mixture is much higher than that strictly needed for hydration of concrete [14, 15]. Nevertheless, almost half of the water added to the mixture will not take part of the hydration products and, hence, is not chemically bound to the solid phase. Accordingly, during the curing period, if concrete is subjected to a low relative humidity environment (RH <95%), the resulting gradient acts as a driving force for moisture migration out of the material, followed by a volume reduction of the porous material. Obviously the environmental conditions define the severity of the drying process, being more detrimental when there is a combination of dry conditions (low RH) and elevated temperatures. A low RH ambient generates strong gradients near the drying surface, thus increasing the drying rate.

Similarly swelling occurs when there is an increase in moisture content due to water absorption [16]. If concrete curing occurs in saturated water vapour (RH >95%) or in water, the water is sucked up by the pores and the final concrete volume increases.

The magnitude of this volumetric change (contractive or dilative) mainly depends on the concrete mix design, with particular reference to cement and water contents.

Different theoretical models exist to describe unconfined concrete strain evolution during hydration. They will be not discussed in details herein: reference should be made to [17, 18] among the others.

If the concrete is within a confining environment, as is soil for piles, the curing process will obviously determine an interaction, hence giving rise to a change in the stress state of the soil itself.

In literature there is little experimental evidence to quantify such effects as very often strain measurements (for instrumented piles along shaft) start when the load is applied at the pile head and not during the concrete curing, that is shortly after concrete placement. In those few published cases where these measurements are available, it is clear the relevance of the phenomena: Figs. 1 and 2 show data by [12, 13] referred to bored cast *in situ* piles in different subsoil conditions (granular soil and soft clay layer overlying a dense sand layer, respectively) and ground water level (at 4 m from ground surface and at ground surface, respectively).

Viggiani and Vinale [12] reported negative (contraction) measured axial strains for that pile portion above water level and increasingly positive (dilation) values at greater depth. The maximum absolute values are of the order of 200 $\mu\varepsilon$.

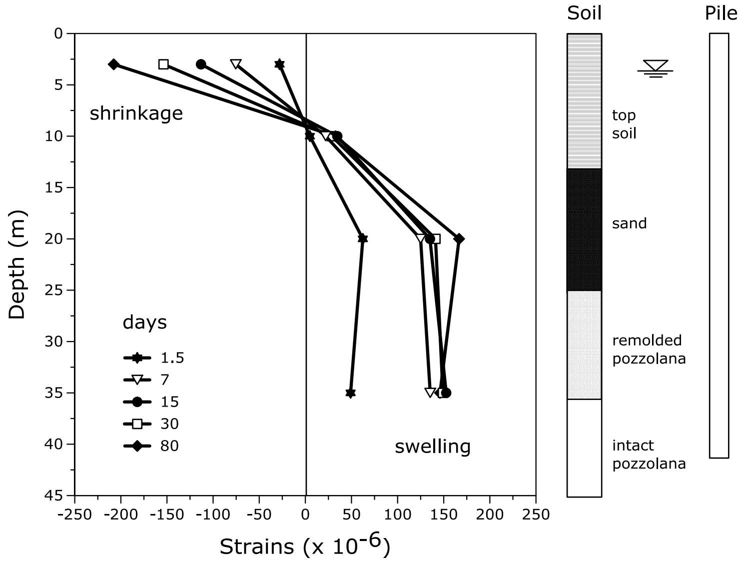

Fig. 1. Measured strains after casting (data from [12]).

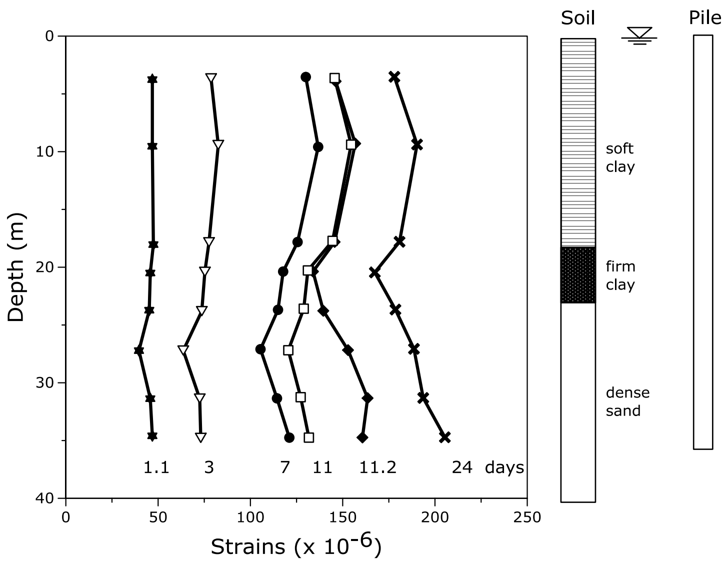

Fig. 2. Measured strains after casting (reproduced from [13]).

Pennington [13] reported positive (dilation) measured axial strains for all the (submerged) pile length, with similar maximum values.

3. Effects of the concrete curing on pile behaviour

According to [9], if excavation is properly executed and high fluidity concrete is then placed, it is possible to assume that concreting can reinstate the effective horizontal stresses existing before drilling. This also applies to the pile base since the weight of the concrete column almost balances the weight of the soil column removed. Therefore, it would be reasonable (and indeed often assumed in practice) to consider a stress-free pile soon after the casting, when concrete is still fluid and thus unable to significantly alter the geostatic state of stress.

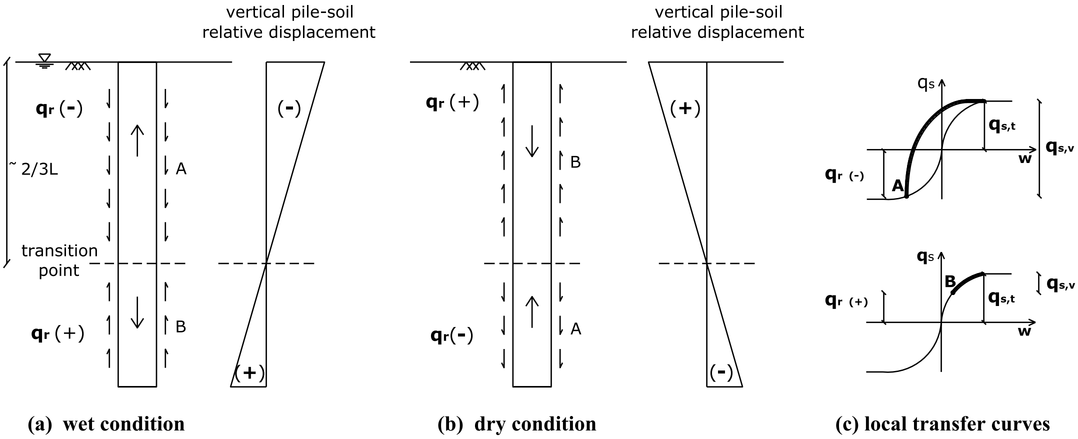

Fig. 3. Effects of concrete curing on a floating pile in (a) wet and (b) dry conditions; (c) "true" and "virtual" values of shaft friction on local transfer curves.

Given these hypotheses, if shrinkage or swelling of the concrete occur, the state of stress around the pile is modified as a consequence of the induced displacement field in radial and vertical directions.

In detail, along the pile shaft radial strains (i.e. increase or reduction of pile diameter) generate a change of horizontal stresses ($\Delta\sigma'_{hr}$), while vertical strains generate shear stresses (q_r) as a consequence of pile displacements (w_r); at the pile base, shrinkage or swelling also determine a change of the normal stresses ($\Delta p'_r$). Similar to what happens when dealing with driven piles or down-drag, such stresses can be defined as residual stresses.

Obviously, according to zero load condition at pile head, the stress changes must guarantee (i) a null resultant and (ii) the compatibility with pile displacements along the depth. It derives that q_r distribution and $\Delta p'_r$ are strictly influenced by:

(i) water table depth respect to the pile head;
(ii) soil properties and stratigraphy;
(iii) pile properties and geometry (slenderness ratio L/D, with L = pile length and D = pile diameter).

In order to qualitatively introduce the main effects produced by concrete curing, the ideal cases of a floating pile embedded in homogeneous wet and dry soils are considered (Fig. 3a and b, respectively).

In wet conditions swelling produces an increase in pile volume: outward radial displacements generate an increase of horizontal stresses ($\Delta\sigma'_{hr} > 0$), while vertical displacements generate pile upward movements (thus negative skin friction) in the upper portion of the pile and downward movements (thus positive skin friction) in the lower portion. If $\Delta p'_r$ is negligible, q_r distribution must guarantee vertical equilibrium: a transition zone (neutral depth) between negative and positive shear stresses has to exist. If, for the sake of simplicity, pile-soil contact behaviour is assumed as rigid-perfectly plastic, any relative displacement different from zero fully mobilizes ultimate shear stresses along the shaft, thus resulting in:

$$\int_0^L q_r(z) \cdot dz = \int_0^L \sigma'_h(z) \cdot \tan\delta \cdot dz = 0 \tag{1}$$

where $\sigma'_h(z)$ is the horizontal effective stress acting at a depth z after concrete curing and δ is the pile-soil friction angle. If $\sigma'_h(z)$ is supposed to vary linearly with z, starting from zero at the ground surface, the neutral depth would lie at $z = 2/3 \cdot L$. Obviously, the neutral depth would differ slightly from this value according to soil heterogeneity, to $\Delta p'_r$ influence and to shear stresses mobilization necessarily varying along the shaft according to different values of pile displacements: it appears reasonable to presume that shear stresses mobilization continuously increases, moving from the transition zone (where $w_r = q_r = 0$) to pile extremities (where maximum values of $|w_r|$ and q_r are attained, Fig. 3a, b).

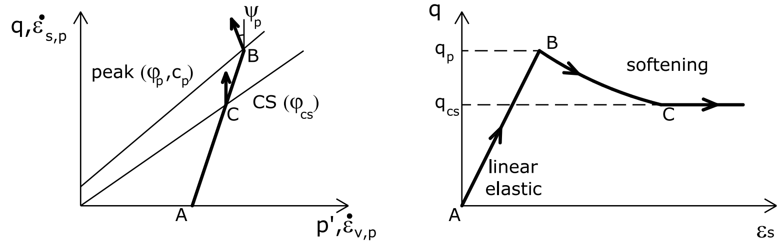

Fig. 4. SS response for conventional drained triaxial compression test.

In dry conditions the same applies (Fig. 3b), with the difference that shrinkage generates a reduction in pile volume: horizontal stresses decrease ($\Delta\sigma'_{hr}$ <0) while shear stress distribution is reversed.

These effects can be modified by different boundary conditions. For instance, end-bearing pile embedded in a saturated soil experiences upward pile-soil relative displacements, therefore a negative skin friction is induced along all the shaft to be equilibrated by the mobilized base load (neutral depth lies close to pile base). Moreover, groundwater table lying at some depth along the pile length determines shrinking or swelling above or below the water table, respectively.

Whatever the case, residual stresses affect pile response to loading. With reference to the local transfer curve sketched in Fig. 3c (for the sake of simplicity assumed as symmetrical with respect to the origin of the axis), points such as A represent the condition reached along the pile shaft when upward pile movements occur, while points such as B represent the condition reached along the pile shaft when downward pile movements occur. In the first case, a larger pile-soil relative displacement is needed to approach the "true" ultimate value ($q_{s,t}$), thus developing a "virtual" larger skin friction ($q_{s,v} > q_{s,t}$). In the other case, smaller pile-soil relative displacement is needed, thus causing a "virtual" smaller skin friction ($q_{s,v} < q_{s,t}$).

Such "virtual" values are those measured when setting to zero the strains along the pile before applying the load (very common in practice).

The problem is very complex and, as stated before, depends on a number of factors. To thoroughly investigate the subject, a broad parametric study has been carried out as described in the next section.

4. Numerical modelling

Numerical axial symmetric analyses dealt with single floating piles, embedded in sandy soils and axially loaded.

Piles were assumed as continuum elements, with an isotropic linear elastic constitutive model. Typical concrete properties were assigned: unit weight $\gamma_{con} = 25\,\text{kN/m}^3$, Young's modulus $E_{con} = 30\,\text{GPa}$ and Poisson ratio, $\nu' = 0.2$. Piles were to some extent "wished in place", in the sense that all the effects produced during drilling and concrete placement were totally neglected, whereas concrete curing was considered.

To simulate the mechanical behaviour of sandy soil the Strain Softening constitutive model (SS) was chosen. It is a relatively simple elasto-plastic model with Mohr-Coulomb failure criterion and a non associated flow rule. The mechanical behaviour is initially linear elastic; after yielding, isotropic non-linear soil softening is assumed, regulated by plastic shear strains. The typical response described using SS for a conventional drained triaxial compression test is shown in Fig. 4.

In addition to the two parameters needed to describe linear elastic behaviour (i.e. shear modulus, G, and Poisson's ratio, ν'), if cohesion, c, is neglected and assuming that both φ and ψ linearly reduce with plastic shear strains from peak to Critical State (CS) conditions, further 4 parameters are needed to describe the plastic behaviour (Table 1): φ_p and φ_{cs} represent the friction angle at peak and at CS conditions, respectively; ψ_p is the angle of dilation at peak; $\varepsilon_{cs}{}^p$ is the plastic shear strain necessary to achieve CS condition (Fig. 5).

SS parameters have a clear physical meaning and can easily be determined. Shear modulus represents an operative value $G < G_0$ (secant stiffness); φ_p and ψ_p can be evaluated by means of Bolton [19] and Rowe [20] theories, respectively: relative density I_D, mean effective stress p' and Lode's angle effects on strength are thus taken into account; φ_{cs} and $\varepsilon_{cs}{}^p$ can be deduced by known literature indications (e.g. [21, 22], respectively).

Table 1
SS parameters

Shear modulus	G
Poisson's ratio	ν'
Angle of friction at peak condition	φ_p
Angle of friction at critical state condition	φ_{cs}
Angle of dilation at peak condition	ψ_p
Plastic shear strains to get CS condition	$\varepsilon_{cs}{}^p$

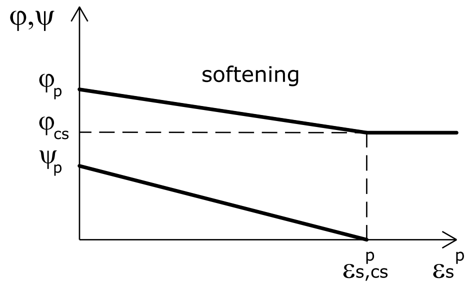

Fig. 5. Linear reduction for angle of friction and dilation.

When dealing with the numerical analyses of pile response to axial loading, one of the most important issues is the modelling of the contact between pile and soil, where the localization of shear strains occurs (shear band). In the present study this was modelled explicitly connecting the pile to the soil by means of *interface elements*. Mascarucci [23] proposed an interface constitutive model (SSI: Strain Softening for Interface), equivalent to SS for continuum elements, whose formulation was derived imposing a mechanical equivalence between SS and SSI predictions when simple shearing is applied. A fictitious thickness t_s was assigned to interface elements in order to represent the shear band thickness: for the cast *in situ*, concrete-soil interface $t_s = (2\div 20)\cdot D_{50}$, with D_{50} = mean particle size, was adopted according to the suggestions of a number of researchers (e.g. [24, 25]).

Figure 6 reports the finite different mesh considered for a generic analysis: lateral boundaries of the mesh have been restrained in the horizontal direction, while the base at a depth $H = 2\cdot L$ was prevented from moving either vertically or horizontally. Model width and mesh density were established by means of sensitivity analyses to guarantee the solution accuracy [23]: particular attention was paid to defining the mesh close to the pile to prevent errors due to stress concentration.

In order to reproduce volumetric strains due to concrete curing process, fictitious thermal swelling or shrinkage was simulated before loading: coupled mechanical and thermal analyses were performed.

The volumetric change ΔV induced by the temperature change ΔT was thus:

$$\Delta V = \alpha(T) V_0 \Delta T \tag{2}$$

where V_0 is pile initial volume at a given reference temperature T_0 and $\alpha(T)$ is the volumetric thermal dilation coefficient assigned to the pile. According to [17], to modify strain rates during concrete curing, $\alpha(T)$ is assumed to vary with temperature. Figure 7a shows the adopted strain evolution in wet (swelling) and dry (shrinkage) conditions: the final values (+300 $\mu\varepsilon$ for swelling and –750 $\mu\varepsilon$ for shrinkage) were selected according to those experimentally determined by [27] on the basis of several laboratory tests on unconfined concrete blocks or pile sections. It is worth noting that such values substantially correspond to only the axial strains reported in Figs. 1 and 2 (about 1/3), but are clearly not coincident as they are measured for piles embedded in the soil, hence with confinement stresses.

As well known, during concrete curing the stiffness increases reaching its conventional final value 28 days after placement: Fig. 7b shows the adopted concrete Young Modulus evolution with temperature, reproducing the time evolution [17].

Soil was considered as temperature-independent.

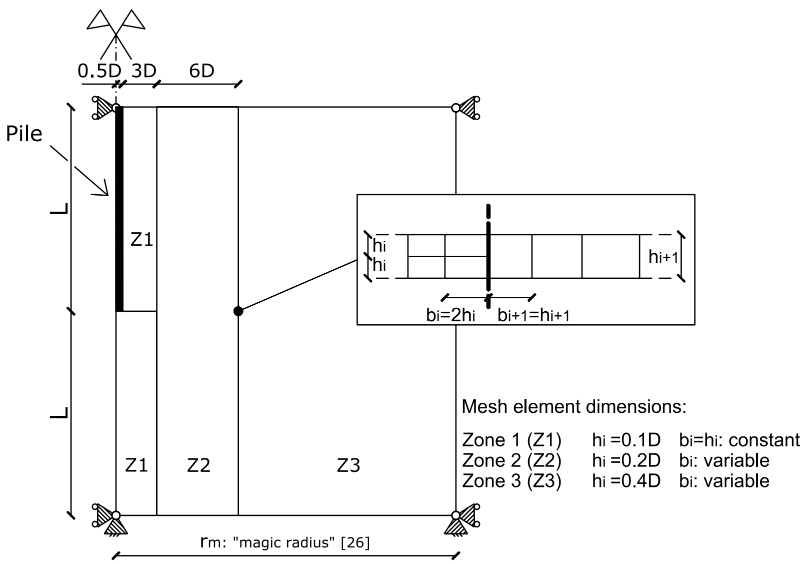

Fig. 6. Finite different mesh adopted for the analyses.

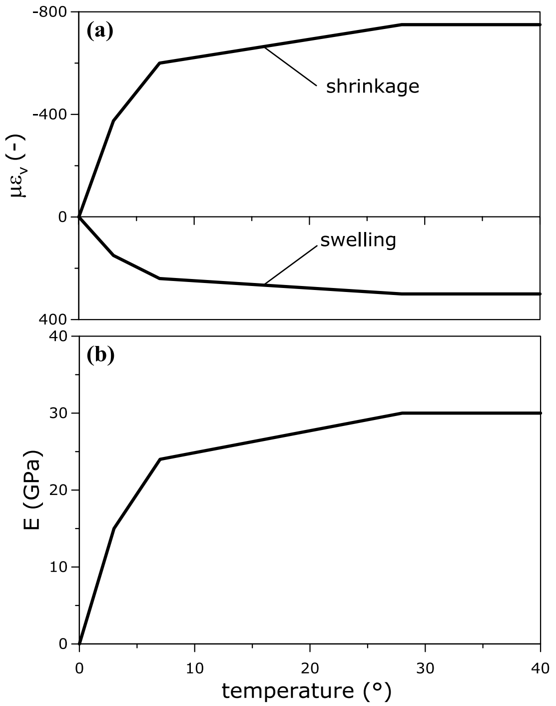

Fig. 7. Concrete curing: (a) volumetric strains and (b) stiffness evolution.

After modelling concrete curing, thermal computation was stopped while mechanical computation continued in order to carry on the axial pile load test. Loading phase was simulated by means of the displacement-control technique up to the full mobilization of the shaft friction: a fictitious vertical displacement velocity of 10^{-7} m/step was applied at the pile head, low enough to make inertial effects negligible and to ensure solution accuracy.

5. Results of the numerical analyses for a single pile

The results of numerical analyses regarding a floating pile embedded in a dense, over-consolidated (OC), fine sand (Fig. 8) are reported. Load test was simulated with and without the concrete curing process, in both saturated and dry conditions, in order to highlight differences.

In Fig. 9 (a) concrete volumetric strains, (b) horizontal stresses and (c) shear stresses acting along the pile shaft soon after concrete curing are presented. Different strains distributions with depth occur depending on the source strain: close to pile extremities, the values are similar to those imposed ($-750\,\mu\varepsilon$ for shrinking and $+300\,\mu\varepsilon$ for swelling); in the middle, due to a larger kinematic constrain, strains are smaller.

Pile
$D = 1$m $L = 40$ m

Geotechnical Model - OC sand
$\gamma_t = 17$ kN/m³; $\gamma'_t = 7$ kN/m³
$K_{0,OC} = K_{0,NC} \cdot OCR^{sen\varphi}$
$I_D = 70$ %
$\varphi_{cs} = 38°$
$\varepsilon^p_{cs} = 30$ %
$t_s = 4$ mm ($D_{50} = 0.2$ mm)
$G_0 = 65 + 2z$; $G = G_0/2$
$\nu' = 0.25$

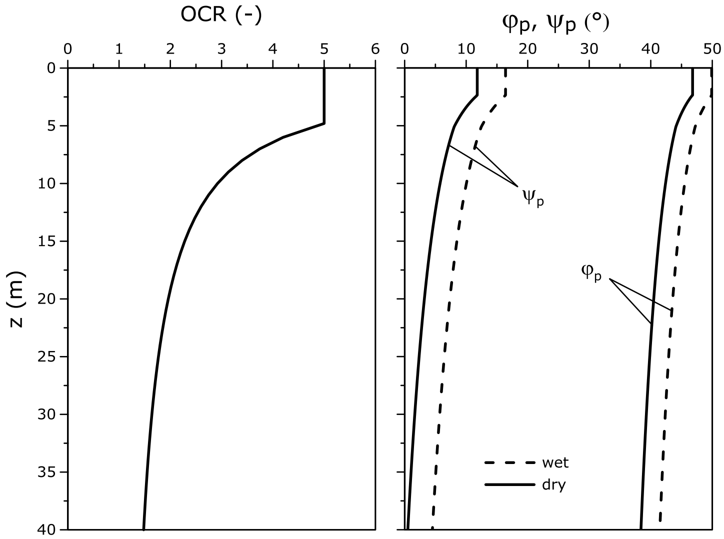

Fig. 8. Data used for the analyses of a single pile.

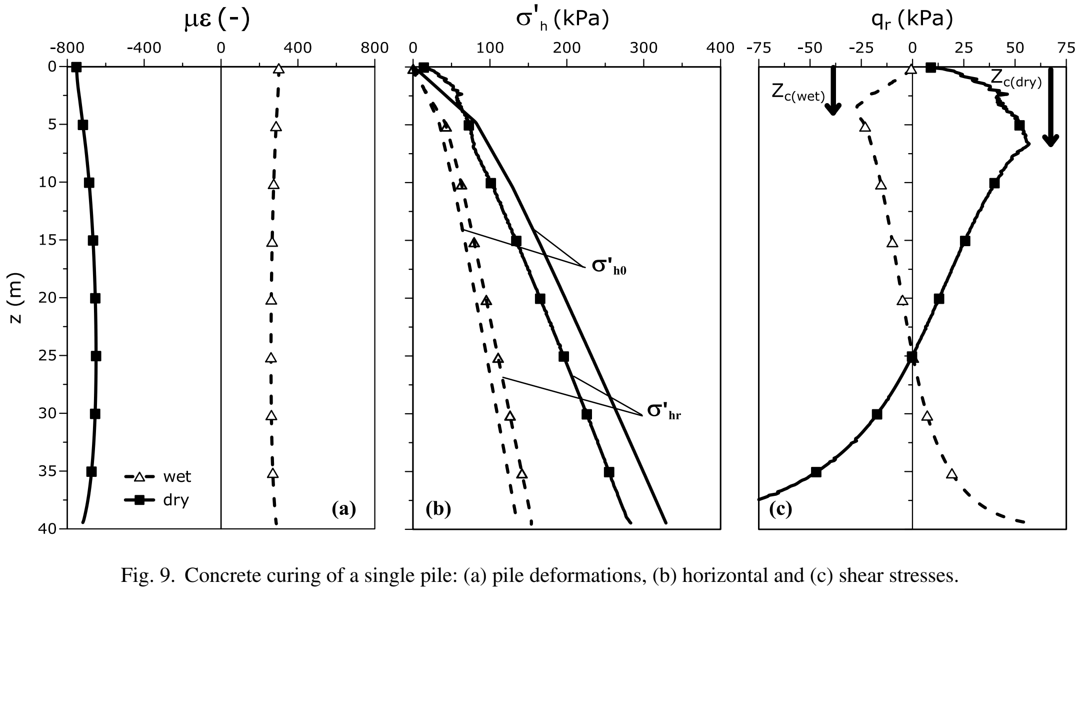

Fig. 9. Concrete curing of a single pile: (a) pile deformations, (b) horizontal and (c) shear stresses.

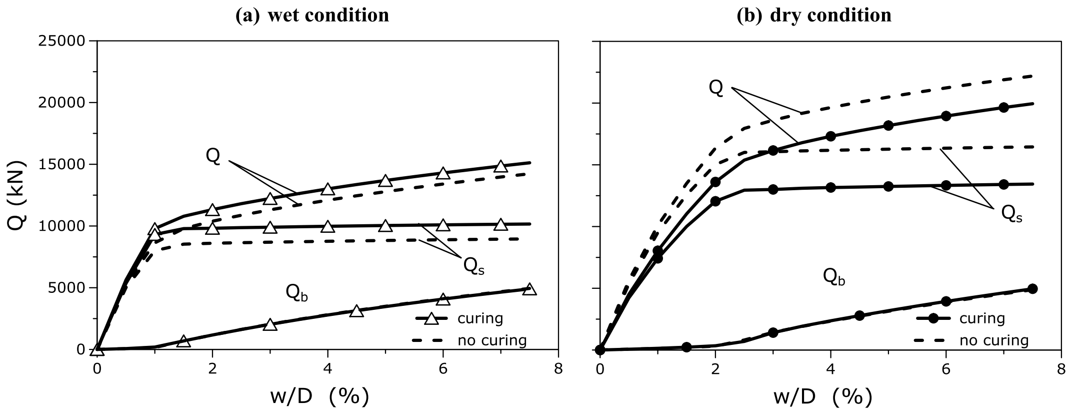

Fig. 10. Single pile: load-displacement curves in (a) wet and (b) dry conditions, with and without concrete curing simulation.

With regard to residual stresses, profiles are coherent with expectations on the basis of the theoretical framework previously depicted (§ 3). The following considerations may be added:

– in dry conditions, change in horizontal stress is larger because concrete shrinkage is bigger in absolute values when compared with concrete swelling;
– in dry conditions, shear stresses along the pile shaft are larger as a consequence of larger vertical pile displacements;
– whatever the hydraulic conditions (dry or saturated), as expected the transition zone is close to $2/3 \cdot L$ ($z \approx 25\,\text{m} = 0.65 \cdot L$);
– in the upper part of the pile, where low confining stresses exist, shear stresses are fully mobilized: the cusp in shear stress distribution represents the depth, Z_c, above which the resistance is mobilized; since in dry conditions pile-soil relative displacements are larger, $Z_{c\,dry} > Z_{c\,wet}$;
– at $z < Z_c$, plastic volumetric strains develop (depending on soil dilatant behaviour), thus contributing to a further increase of $\Delta\sigma'_h$ as it is partially forbidden by the surrounding soil. This becomes evident if the horizontal stress profile in dry environment is considered (Fig. 9b): atop the pile, the reduction of horizontal stresses induced by pile shrinkage is lower than the effect of soil dilatancy, hence an overall horizontal stress increase is observed.

Figures 10a, b show the load displacement curves referred to the pile head (Q), pile shaft (Q_s) and pile base (Q_b) in wet and dry conditions, respectively; the displacement w has been normalized with respect to the pile diameter.

As expected, the sliding mechanism at the pile shaft needs small displacement ($1\text{–}2\% \cdot D$) to fully mobilize Q_s; at such values, Q_b is far from its ultimate value, typically attained at displacements not smaller than $10\% \cdot D$.

Given that in dry conditions Q_s is larger than in wet conditions because of larger geostatic horizontal stresses, it has to be noted that concrete curing alters the values of Q_s mainly as a consequence of the changes of horizontal stresses (residual); in particular, Q_s increases/decreases in wet/dry conditions while Q_b is almost insensitive to concrete curing.

Effects of residual shear stresses on local shaft friction mobilization (Fig. 11a, b) are in agreement with observations discussed above (§ 3): $q_{s,v} > q_{s,t}$ where negative residual shear stresses develop; $q_{s,v} < q_{s,t}$ in the opposite case. These results can be well synthesized in terms of $\beta(z) = q_s(z)/\sigma'_v(z)$, where $\sigma'_v(z)$ is the vertical geostatic effective stress acting at a depth z: in Fig. 11c the ratio between "virtual" and "true" values of β (β_v/β_t) is shown in wet and dry conditions. Comparisons highlight that above the neutral depth in wet conditions the increase of β is nearly 50%; in dry conditions the effect is more remarkable, with a reduction of about 80%. Below the neutral plane the opposite occurs.

6. Results of the parametric study

6.1. Generalities

To better understand the combination of factors for which concrete curing strongly affects the response of pile to axial loading, a broad parametric study has been carried out by changing pile slenderness ratio, *L/D*, relative density, I_D, stress history, OCR, critical state friction angle, φ_{cs}, shear band thickness, t_s (thus mean grain size, D_{50}). Table 2 summarizes the whole spectrum of analyses: a suite of 96 different combinations (48 for wet conditions and 48 for dry conditions) was analyzed. All the analyses were then repeated to consider concrete curing as discussed in § 6.3, thus yielding a total of 192 cases.

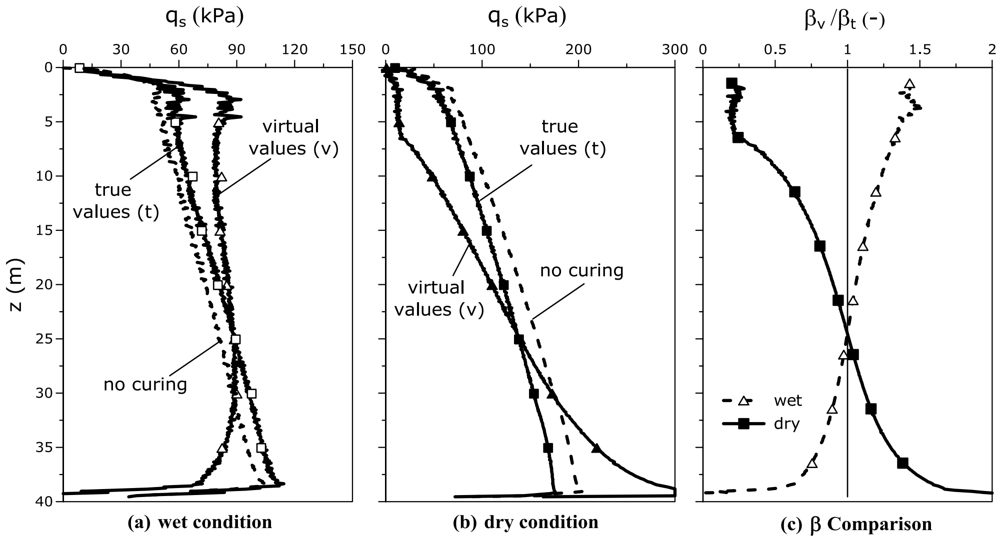

Fig. 11. Single pile: "true" and "virtual" shear stress profiles in (a) wet and (b) dry conditions, with and without concrete curing simulation; (c) comparison between "true" and "virtual" β values.

Table 2
Fundamental parametric analyses features

Parameter	Value	Description
L	15 m	*Long Pile*
	40 m	*Short Pile*
φ_{cs}	32°	$\varphi_{cs} = 32°$
	38°	$\varphi_{cs} = 38°$
I_D	70%	*Dense Sand*
	50%	*Medium Dense Sand*
	30%	*Loose Sand*
K_0	0.47	*NC*_$\varphi_{cs} = 32°$
	0.38	*NC*_$\varphi_{cs} = 38°$
	1*	*OC (max value)*
t_s	4 mm	*Fine Sand*
	40 mm	*Coarse Sand*
G_0	30 + 2z (MPa)	$I_D = 30\%$
	46 + 2z (MPa)	$I_D = 50\%$
	65 + 2z (MPa)	$I_D = 70\%$

*In OC soils, K_0 varies along depth following OCR reduction; in any case $K_0 \leq 1$.

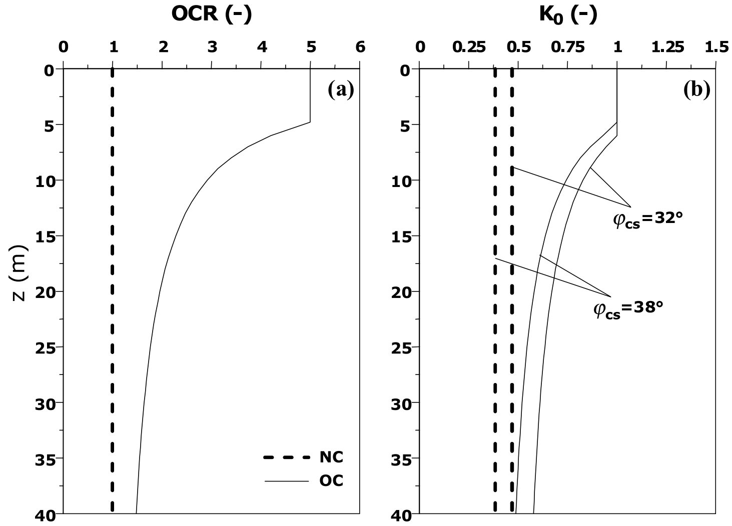

Fig. 12. Parametric analyses: (a) OCR and (b) K_0 profiles for NC and OC models, changing φ_{cs}.

Saturated soil unit weight was set equal to $\gamma_t = 17\,\mathrm{kN/m^3}$. Critical state angle ($\varphi_{cs}$=32–38°) and relative density (I_D = 30–70%) values were selected to fall within typical ranges [21] and kept constant along the depth for each case. Normally consolidated (NC) and over-consolidated (OC) soils were considered; in the latter case, the adopted OCR profile (Fig. 12a) yields to the corresponding K_0 profiles (Fig. 12b; $K_{0,OC} = K_{0,OC} \cdot \mathrm{OCR}^{sen\varphi}$ according to [28], with $\varphi = \varphi_{cs}$ as suggested by [29] and $K_0 \leq 1$ as suggested by [30]).

Peak angles of friction and dilation were evaluated according to Bolton [19] and Rowe [20] theories, thus continuously varying with depth according to p' changes (being I_D constant for each case); linear reduction curves for φ_p and ψ_p (Fig. 5) were considered, attaining CS condition for a plastic shear strains $\varepsilon_{cs}{}^p = 30\%$.

Throughout the analyses pile diameter, D, and soil Poisson's ratio, ν', were kept constant and equal to 1 m and 0.25 (typical value for sandy soils), respectively; the small strain shear modulus G_0 was assumed linearly varying with depth coherently to mean stress and relative density. To overcome the inability of both SS and SSI to take into account the non linear behaviour of the soil, a secant value was adopted ($G = G_0/2$).

Lastly, two shear band thicknesses were simulated: t_s = 4–40 mm (corresponding to D_{50} = 0.2–2 mm if $t_s = 20 \cdot D_{50}$ is supposed).

6.2. *Effects of concrete curing*

From a selection of results, Fig. 13 plots those regarding the longer pile embedded in NC and OC, loose and dense, fine and coarse sands, in wet and dry conditions. This option was made as, from a qualitatively point of view, no differences were observed for the shorter pile.

As it can be seen the horizontal stress changes are almost constant and equal to about ±20% along the major pile length, with the exception of the upper part of the pile where the values become steadily larger, even larger than 100% in dry conditions at a very shallow depth (in the Fig. 13 normalized with respect to pile length).

It follows that the influence of concrete curing is not so remarkable and is almost independent of soil properties. Indeed, in denser soils the changes are very similar (slightly larger) to those obtained in looser soils; the thickness of shear band (in turn dependent on grain size distribution) does not affect the results, being practically superimposed all over the pile length.

Similar results have been obtained in terms of residual shear stresses (Fig. 13c). It must be noted however that larger pile-soil relative displacements develop in dry conditions, thus resulting in larger shear stresses (also due to larger effective geostatic horizontal stresses) acting over a larger portion of the pile ($Z_{c(dry)} > Z_{c(wet)}$; § 5).

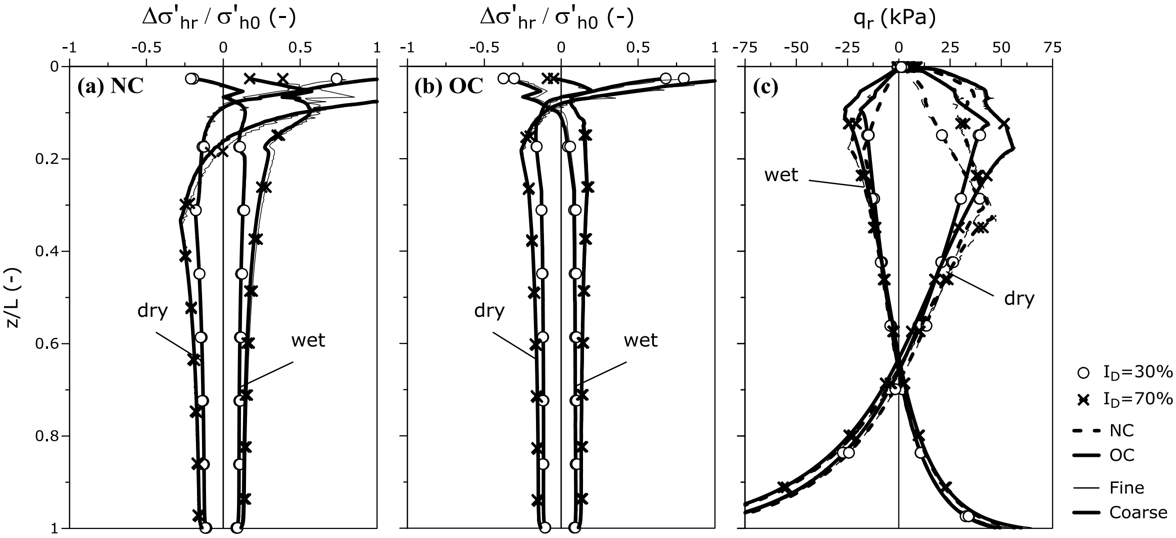

Fig. 13. Concrete curing - results of the analyses for long piles ($L = 40$ m): horizontal stress changes in (a) NC and (b) OC soil; (c) residual shear stresses.

6.3. Pile response to axial loading

To quantify the effects of concrete curing on the pile response to axial loading, two sets of identical analyses were carried out, with and without normal and shear residual stresses. For the same reason as above, only the results referring to the longer pile ($L = 40$ m) are presented herein.

Figure 14 reports the comparison between the "true" β values, with (β_t) and without (β) concrete curing. In particular, Fig. 14a summarizes the results for piles embedded in fine sands ($t_s = 4$ mm), while Fig. 14b those for piles embedded in coarse sand ($t_s = 40$ mm).

As it can be seen the maximum value attained by the ratio β_t/β is of the order of $\pm 20\%$ in the case of fine sand, even smaller in the case of coarse sand. It is worth noting that these values are always valid independently of relative density, stress history and water condition (especially at larger depth). Such a result should not come as a surprise: since residual stresses are not as sensitive to soil properties (Fig. 13), their influence on pile response to axial loading tends to be increasingly important where shaft friction is small, i.e. for piles embedded in loose and fine sand. This finding can be explained by the dilatant behaviour of sandy soil.

During the loading phase, shear stresses tend to concentrate in a thin cylinder of soil (shear band) around the pile, whose thickness t_s mainly depends on D_{50} (§ 4). If dilative strains occur, the thickness of the shear band increases and, being partially forbidden by the surrounding soil, determines larger effective horizontal stresses (and thus shear stresses) acting on the pile shaft at failure. This effect is proved to be strictly governed by t_s: for given conditions, very dense fine sands could not allow for large values of q_s as a consequence of plastic volumetric strains confined in a very thin shear band; on the contrary, loose coarse sands could allow for large values of q_s due to the development of plastic volumetric strains in a relatively thick shear band [23, 31].

Keeping in mind observations related to Fig. 3c, as well as the results showed for the single pile (Fig. 11c), some considerations can be made by looking at the overall numerical findings contained in Fig. 15. In detail, the graphs report the ratio between "virtual" (β_v) and "true" (β_t) values of the coefficient β, already introduced in § 5. Due to their definition, such a ratio represents the error made when interpreting experimental data from a load test on an instrumented pile where the residual shear stresses are neglected and/or the strain sensors are reset to zero before the start of loading. In both the cases, the interpretation of the field data yields to wrong values of q_s, thus of β.

This error is by no means negligible, especially for those situations where the effects of concrete curing play a major role if compared with other factors. From Fig. 15, for instance, it is clear that the ratio β_v/β_t is larger

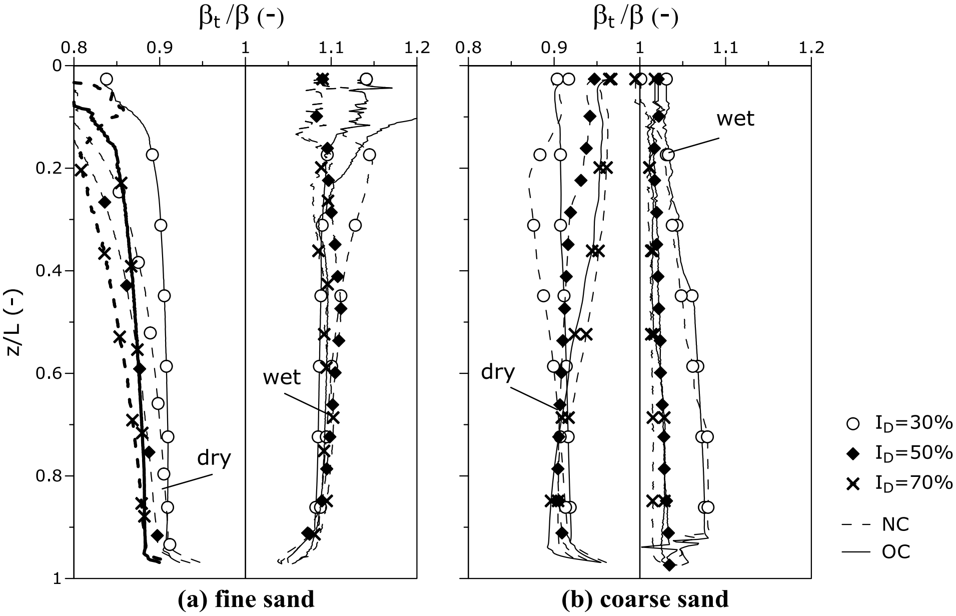

Fig. 14. Load tests - results of the analyses for long piles ($L = 40$ m): β_t/β for (a) fine and (b) coarse sands.

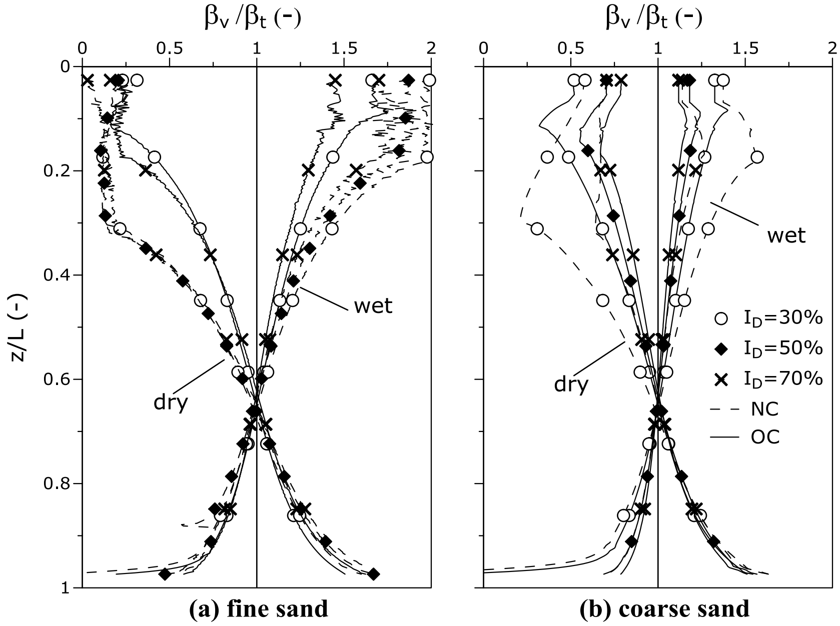

Fig. 15. Load tests - results of the analyses for long piles ($L = 40$ m): β_v/β_t for (a) fine and (b) coarse sands.

when dealing with piles embedded in dry NC loose fine sands. In the same figure it is worth noting that the error varies along the depth as a function of the distribution of pile displacement (Fig. 3a,b): at shallow depth ($z < Z_c$, Fig. 9c), pile displacements due to concrete curing are so large as to mobilize soil strength, hence measured values can virtually increase by 100% or reduce to zero ($\beta_v/\beta_t = 2$ or about 0 depending on the hydraulic conditions); the same does not apply at greater depth ($z > Z_c$) where residual shear stresses are smaller than the ultimate values as they are proportional to the varying pile displacement. Clearly β_v/β_t is equal to 1 at the neutral depth, by definition. Finally, it must be noted that such an error is remarkable over a significant portion of pile length.

7. Conclusions

It is widely accepted that load test to failure is the most widespread and reliable tool for investigating the load-settlement behaviour of piles. This is mainly due to the fact that, for given subsoil conditions, pile response to a vertical load is significantly affected by technological factors.

The authors clearly agree with this point of view yet, although well aware of the limitation of necessarily simplified schemes and constitutive laws for the involved materials as well as the sensitivity of the results to the numerical details (e.g. mesh, interface elements, loading procedure, boundary conditions, etc.), they are of the opinion that the use of numerical tools greatly assists in an improved knowledge and understanding.

Among the several factors influencing the response of a pile to axial loading, in the paper attention was paid to the effects of concrete curing on shaft friction. These activities are part of a wider research on the behaviour of a single pile aimed at detecting the main factors affecting pile-soil interaction along the shaft in sand.

The role played by concrete curing on shaft friction is often neglected when interpreting load test data of instrumented piles. Indeed, the experimental evidence of the existence of residual stresses, not only for driven piles but also for cast *in situ* piles, makes it mandatory to fully understand how they affect the pile response to axial loading.

Given these premises, the broad performed parametric studies, although limited to the theoretical case of a floating pile in homogeneous soils, clearly highlight that soil-pile interaction starts well before the pile *strictu sensu* exists: concrete curing alters the state of the stress in the surrounding soil and, at a generic depth, moves the initial condition from the origin of the presumed local transfer curves to a point whose position mainly depends on environmental conditions (dry or saturated soil).

All these factors further impact the available skin friction at the pile shaft. Although it is not of particular relevance to the global value of shaft resistance, it is very important in terms of skin friction distribution along the pile.

A quite remarkable and practical suggestion can therefore be triggered: full scale load tests on cast *in situ* instrumented piles should be carried out considering the residual stresses developing after concrete placement. This aspect is of a particular relevance if load tests are aimed to develop databases for β values to be used in daily practice: not taking into account these locked-in stresses can lead to significant mistakes, as often indirectly demonstrated by larger or smaller measured values with respect to those expected on the basis of soil properties.

Data interpretation must have a rational basis, properly quantifying also concrete curing effects, before supplying ultimate values for the skin friction believed as "not questionable just because measured"... but maybe not really understood!

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