

Reliability-based pile design in sandy soils by CPT measurements

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ABSTRACT: Field tests, as static cone penetration tests (CPT), are widely used in granular soil characterization because they are largely correlated by empirical charts and graphics to the main physical and mechanical soil properties (i.e. D_R , ϕ , lithology etc.) but seldom the uncertainties concerned with such estimations are taken into account. In this paper results from analyses on spatial variability distribution of measured parameters from CPTs performed at Scanzano Jonico site are employed for assessing pile bearing capacity by means of direct and indirect methods. Although those formulations are widely accepted by literature and used in practice, they are not discussed in terms of resulting reliability level of the pile bearing capacity. In this paper, Bustamante and Gianeselli's formulation have been adopted as direct method, for estimating pile bearing capacity from q_c profile at Scanzano Jonico site. As far as the reliability-based design of limit pile resistance is concerned, the statistical properties of design variables are based on the variability study carried out on such sands in the companion paper. Finally results are discussed in terms of reliability level and economical advantages in usage of direct instead of indirect methods for pile design in sands.

1 INTRODUCTION

Field tests are commonly used to characterize cohesionless soils. Over the years equipment for cone penetration tests have been developed and the usage of measurements from this type of testing has been introduced in direct and indirect methods for foundation design.

Especially with respect to pile design in sandy soils, according to indirect designing methods several correlations are provided between q_c measurements and sand mechanical parameters as D_R (Baldi et al., 1986), ϕ' (Robertson and Campanella, 1983; Kulhawy and Mayne, 1990), overconsolidation ratio OCR (Mayne, 2001) and lateral stress coefficient K_0 (Mayne, 2005). Such correlations can introduce a large amount of model errors which can lead to uncertainties in pile design.

Direct methods, instead, estimate side and tip resistance of piles from q_c and f_s profiles. Such methods have been widely applied and compared each other by researchers as Briaud (1988), Robertson et al. (1988) and Eslami and Fellenius (1997), among others.

However, up to now, no comparison between direct and indirect methods are available in terms of reliability level of predictions.

Therefore, in this paper, indirect and direct pile design methods are applied for the case of Scanzano Jonico site. The study employs a reliability-based design approach and takes into account model errors in empirical correlations and design formulations, measurement uncertainties in CPTs and the natural soil variability, in order to assess and discuss the differences in the reliability levels of direct and indirect methods.

2 CPT FOR PILE DESIGN

Two main approaches for the application of cone data to pile design are available; indirect and direct methods. Indirect CPT methods employ theoretical formulations of failure mechanisms where soil parameters, such as the friction angle, the relative density, the coefficient of lateral earth pressure are used. Such methods introduce significant uncertainties owing to the calculation of intermediate parameters and to the introduction of empirical factors whose uncertainty cannot be quantified.

On the other hand the direct CPT methods more or less equate the measured cone resistance to the pile unit resistance. Commonly, direct methods (such as Schmertmann and Nottingham's method (Schmertmann, 1978, Nottingham, 1975), the European method (DeRuiter and Berigen, 1979), the French Method (Bustamante and Gianceselli, 1982) and Meyerhof method (Meyerhof, 1983) in sands proportion the toe and the shaft resistance to the cone resistance. In particular, to relate the cone resistance to the pile unit toe resistance, direct CPT methods determine the arithmetic average of the CPT data over an "influence zone" about the toe. Therefore, before averaging, it is a common practice to manually filter and smooth the data, generally by simply removing the peaks and troughs from the records by means of application of "subjective rules", that is, derived by experience and engineering judgement.

2.1 From CPTs to soil parameters

Whenever pile design is carried out in sandy soils, the following expression is commonly used for the ultimate bearing capacity Q_{tot} of a pile with length L and diameter d :

$$Q_{tot} = \pi d L \cdot f_s + \frac{\pi d^2}{4} \cdot q'_{toe} \quad (1)$$

where f_s is the unit side friction resistance and q'_{toe} is the net unit toe-bearing resistance. The unit side friction resistance is more clearly defined than the toe bearing because of the sharp break in the load-settlement curve. The analysis of side friction is based on the principle of sliding friction, especially in the case of sandy soils. A general expression for side friction is:

$$f_s = \sigma'_h \tan \phi_f \quad (2)$$

where σ'_h is the horizontal effective stress and ϕ_f is the soil-foundation interface friction angle. Researchers have used laboratory tests to correlate $\tan \phi_f$ with the ratio between ϕ_f and the effective friction angle of the soil, ϕ' . Table 1 gives typical values of ϕ_f/ϕ' .

Table 1 Approximate ϕ_f/ϕ' values for the interface between deep foundations and soil (Adapted from Kulhawy et al., 1983, Kulhawy, 1991).

Foundation Type	ϕ_f/ϕ'
Rough concrete	1.0
Smooth concrete (i.e. precast pile)	0.8-1.0
Drilled shaft built using dry method or with temporary casing and good construction techniques	1.0
Drilled shaft built with slurry method (higher values correspond to more careful construction methods)	0.8-1.0

Moreover, it is well known that the ratio between the horizontal and vertical effective stresses is defined as the coefficient of lateral earth pressure, K :

$$K = \frac{\sigma'_h}{\sigma'_v} \quad (3)$$

Deep foundation construction induces significant changes in the surrounding soil behaviour, thus the coefficient of lateral earth pressure is generally not equal to the coefficient of lateral earth pressure in the ground before construction, K_0 . The ratio K/K_0 depends on many factors as summarized by

Coduto (2001). Kulhawy et al. (1983) suggested typical K/K_0 ratios presented in Table 2 . Combining Eq. (2) and Eq. (3),

$$f_s = K_0 \sigma'_v \left(\frac{K}{K_0} \right) \left(\frac{\phi_f}{\phi'} \right) \quad (4)$$

Table 2 Approximate ratio of the coefficient of lateral earth pressure after construction to that before construction (Adapted from Kulhawy et al., 1983, Kulhawy, 1991).

Foundation Type and Method of Construction	K/K_0
Drilled shaft-built using dry method with minimal sidewall disturbance and prompt concreting	0.9-1.0
Drilled shaft-slurry construction with good workmanship	0.9-1.0
Drilled shaft-slurry construction with poor workmanship	0.6-0.7
Drilled shaft-casing method below water table	0.7-0.9

The most common method of assessing K_0 is by empirical correlations with other soil properties. Several such correlations have been developed, including the following by Mayne and Kulhawy (1982), which is based on laboratory tests performed on 170 soils that ranged from clay to gravel:

$$K_0 = (1 - \sin \phi') \text{OCR}^{\sin \phi'} \quad (5)$$

where OCR is the overconsolidation ratio. The latter can be determined from CPTs by means of the expression arranged by Mayne (2001):

$$\text{OCR} = \left[\frac{0.192 \cdot (q_t / p_a)^{0.22}}{(1 - \sin \phi') \cdot (\sigma'_{v0} / p_a)^{0.31}} \right]^{(\sin \phi' - 0.27)} \quad (6)$$

in which q_t is the stress-normalized cone tip stress according to Eq. (7):

$$q_t = \frac{\frac{q_c}{p_a}}{\left(\frac{\sigma'_{v0}}{p_a} \right)^{0.5}} \quad (7)$$

Where $p_a=100\text{kPa}$. Finally, Kulhawy and Mayne (1990) from 702 calibration chamber tests conducted on 26 clean quartz to siliceous sands, derived an expression for ϕ' , that is:

$$\phi' = 17.6^\circ + 11 \cdot \log(q_t) \quad (8)$$

The normalization to square root of the effective overburden stress l accounts for sand compressibility and grain crushing effects (Mayne, 2001). As far as the evaluation of toe bearing resistance of piles in sandy soils is concerned, it may be expressed using the formula suggested by Kulhawy et al. (1983):

$$q'_{\text{toe}} = d\gamma N_\gamma + \sigma'_{\text{vt}} N_q \quad (9)$$

in which d is the pile diameter, γ is the unit weight of soil immediately beneath pile toe, N_γ and N_q are the bearing capacity factors and σ'_{vt} is the vertical effective stress at pile toe elevation. In the case of piles, the bearing capacity factors depend on both shear strength and compressibility, because any of the three modes of failure (general, local or punching shear) could govern failure. According to Vesic (1977), the bearing capacity factors N_γ and N_q are computed as follows:

$$\begin{aligned} N_\gamma &= 0.6(N_q - 1)\tan\phi' \\ N_q &= \frac{(1 + 2K_0)N_\sigma}{2} \\ N_\sigma &= \frac{3}{3 - \sin\phi'} e^{\frac{(90-\phi')\pi}{180}} \tan^2\left(45 + \frac{\phi'}{2}\right) I_r^{\frac{4\sin\phi'}{3(1+\sin\phi')}} \\ I_r &= \frac{E}{2(1+\nu)(\sigma'_{\text{vt}} \tan\phi')} \end{aligned} \quad (10)$$

where I_r is the rigidity index which defines the compressibility effects, E is the modulus of the elasticity of soil in vicinity of toe, ν is Poisson's ratio of soil in vicinity of toe, N_σ is a bearing capacity factor and K_0 is the coefficient of lateral earth pressure at rest.

As far as E estimation is concerned, Schmertmann (1978) and Robertson and Campanella (1989) developed empirical correlations between E and the measured cone resistance q_c from cone resistance tests, summarised in Table 3 for sandy soils.

Table 3 E values from CPT results (After Schmertmann, 1978, Robertson and Campanella, 1989).

Soil Type	E/q_c
Normally consolidated silty or clayey sands	1.5
Overconsolidated silty or clayey sands	3

2.2 From CPTs to pile bearing capacity

When CPTs are performed it could be easier and faster to use directly q_c and f_s profiles for estimating a single pile bearing capacity numerous authors proposed direct methods developed based on full scale pile loading tests. Among others, one of the most extensive experimental effort has been provided by Bustamante and Gianselli (1982), who proposed a direct method based on the interpretation of a series of 197 full scale static pile loading tests.

They investigated 48 sites containing soils made up of varied material as clay, silt, sand, gravel, weathered rock but also mud, peat, more or less weathered chalk and marl. The method relies on q_c measurement from which values of both the unit side resistance q_s and the unit toe resistance q_t of a pile can be drawn, according to the following expression:

$$Q_{\text{tot}} = q_t \cdot A_t + q_s \cdot A_s = q_{\text{ca}} \cdot k_c \cdot \frac{\pi d^2}{4} + \pi d \cdot \frac{1}{\alpha_0} \int_0^L q_s(z) \cdot dz \quad (11)$$

where q_{ca} is the equivalent cone resistance at the level of pile toe (kN/m^2), k_c is the penetrometer bearing capacity factor, d is the pile diameter (m), $q_c(z)$ is the cone resistance along the pile length (kN/m^2).

As far as the calculation of q_{ca} is concerned, the steps are here summarised:

1. the curve of the cone resistance q_c is smoothened to q'_c so as to reduce the local irregularities of the raw curve;
2. starting from the smoothed curve, q_{ca} is calculated which is the mean value of q'_c between the values $-a$ and $+a$, where $a=1.5 D$ (Fig.1);
3. finally q'_{ca} is calculated after eliminating the values higher than $1.3q_{ca}$ under the pile point, whereas the values higher than $1.3q_{ca}$ and lower than $0.7q_{ca}$ are eliminated over the pile point.

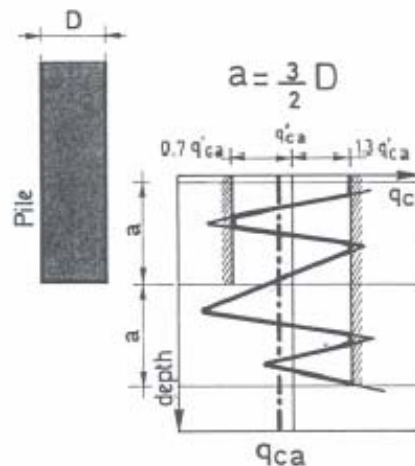


Fig.1 Evaluation of q'_{ca} according to Bustamante' and Ganeselli's method. (After Bustamante and Ganeselli, 1982).

The bearing capacity factor k_c is provided by the same authors, derived from full-scale loading tests (Table 4). These values depend on the nature of the soil and on different pile placement techniques, which are divided into two groups:

- Group I: Plain bored piles; mud bored piles; Micropiles grouted under low pressure (Type I); cased bored piles; hollow auger bored piles; piers; barrettes.
- Group II: Cast screwed piles; driven precast piles; prestressed tubular piles; driven cast piles; Micropiles or small diameter piles grouted under high pressure, with diameter $d < 25\text{cm}$ (Type II); driven grouted piles (low pressure grouting); driven metal piles; driven rammed piles; jacked concrete piles; high pressure grouted piles of large diameter.

The limit unit skin friction q_s is calculated by dividing the cone resistance q_c corresponding to the given level by a coefficient α , which takes into account the nature of the soil and the pile production and placement methods. As a matter of fact, the researchers define three categories of piles:

▪ **Category I:**

- A) Plain bored piles; mud bored piles; hollow auger bored piles; cast screwed piles; type I Micropiles; piers; barrettes.
- B) Cased bored piles (concrete or metal shaft); driven cast piles.

▪ **Category II:**

- A) Driven precast piles; prestressed tubular piles; jacked concrete piles.
- B) Driven metal piles; jacked metal piles.

▪ **Category III:**

- A) Driven grouted piles; driven rammed piles.
- B) High pressure grouted piles with diameter greater than 25cm; Type II Micropiles.

Table 4 Values of bearing capacity factors k_c for the calculation of limit tip resistance (Adapted after Bustamante and Ganeselli, 1982).

Nature of soil	$q_c(10^5\text{Pa})$	Factors k_c	
		Group I	Group II
Silt and loose sands	≤ 50	0.4	0.5
Moderately compact sand and gravel	$50 \div 120$	0.4	0.5

Compact to very compact sand and gravel	>120	0.3	0.4
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they also suggested α values given in Table 5 according to the nature of the soil and the category of piles. As can be noted from Table 5, only q_s maximum values are provided for piles belonging to Category III.

Table 5 Values of coefficient α and maximum values for unit limit skin friction q_s .

Nature of soil	q_c (10^5 Pa)	Coefficient α				Maximum value of q_s (10^5 Pa)					
		CATEGORY				CATEGORY					
		I(A)	I(B)	II(A)	II(B)	I(A)	I(B)	II(A)	II(B)	III(A)	III(B)
Silt and loose sand	≤ 50	60	150	60	120	0.35	0.35	0.35	0.35	0.8	-
Moderately compact sand and gravel	$50 \div 120$	100	200	100	200	0.8	0.35	0.8	0.8	1.2	≥ 2
Compact to very compact sand and gravel	>120	150	300	150	200	1.2	0.8	1.2	1.2	1.5	≥ 2

3 RELIABILITY ANALYSIS IN GEOTECHNICAL DESIGN

With respect to geotechnical design, four uncertainty factors can be distinguished (Honjo, 2004):

- Physical uncertainty, called aleatory, which is related to the spatial fluctuation of soil properties owing to natural and continuous processes of deposition, diagenesis and erosion of soil and rock deposits.
- Statistical uncertainty, called also epistemic. It relates to the number of experimental samples from both laboratory and field tests and to the volume sampled with respect to the soil/rock volume involved in construction. It also concerns measurement errors affecting experimental procedures.
- Model uncertainty, which belongs to the epistemic uncertainties. It refers to the approximations introduced by both theoretical and empirical formulations used for the geotechnical designing. Moreover, such uncertainty is introduced by correlations between measured indexes and mechanical design variables.
- Gross errors which are commonly disregarded.

With respect to the uncertainties affecting soil mechanical parameters from CPTs, they are briefly discussed in the companion paper (Vessia and Cherubini, 2007). Phoon and Kulhawy (1999a) by means of an extensive review of several variability analyses, quantified in terms of coefficients of variation, the contribution of different sources of uncertainties on soil mechanical properties from field and laboratory tests and they proposed an additive variability model (Phoon and Kulhawy, 1999b) in which the total uncertainty is derived as follows:

$$\text{CoV}^2(R_p) = \Gamma^2(L) \cdot \text{CoV}_\omega^2(P_m) + \text{CoV}_m^2(P_m) + \text{CoV}_M^2(M_t) \quad (12)$$

in which $\text{CoV}(R_p)$ is the total coefficient of variation of the design variable/resistance, $\text{CoV}_\omega(P_m)$ is the coefficient of variation of the inherent variability of measured properties, $\Gamma^2(L)$ is the variance reduction function (related to spatial measurements) which depends on the length of the construction along which the measurements are averaged, that is L , $\text{CoV}_m(P_m)$ is the coefficient of variation of measurement uncertainty of the measured property and $\text{CoV}_M(M_t)$ is the coefficient of variation of transformation model uncertainty.

As far as measurement errors are concerned, an estimate of mechanical cone testing errors is carried out by Kulhawy and Trautmann (1996) as indicated in Table 6.

Table 6 Estimated coefficients of variation for mechanical cone testing MCPT (After Kulhawy and Trautmann, 1996).

test	CoV _{eqp}		CoV _{prc}		CoV _{rnd}	
	q _c	f _s	q _c	f _s	q _c	f _s
MCPT	0.05	0.05	0.10	0.15	0.10	0.15

In the case of q_c, the uncertainty due to equipment and procedure, according to the additive coefficient of variation model leads to CoV_m(P_m) equals to 7%. This result comes from the expression:

$$\text{CoV}^2(q_{\text{cmis}}) = \sqrt{\text{CoV}_{\text{eqp}}^2(q_c) + \text{CoV}_{\text{prc}}^2(q_c)} \quad (13)$$

This value shall be summed up to the inherent variability of q_c, for the case studied.

Hence, the point in the total resulting variability of the estimated pile bearing capacity from CPTs is the quantification of model uncertainty. It is clear that it differs from direct to indirect methods. As a matter of fact, in direct methods only the design model uncertainty shall be taken into account; whereas both correlation model and design model uncertainties shall be considered when indirect methods are employed. As Phoon and Kulhawy (1999b) pointed out, the amount of transformation uncertainty concerning with the estimation of internal friction angle ϕ' from q_c is 2.8° in terms of standard deviation, according to Kulhawy's and Mayne's formulation (Eq. 8).

In the foregoing study, all the sources of uncertainties discussed are taken under consideration in order to estimate the level of reliability of the bearing capacity of bored piles installed in Scanzano Jonico sands according to direct and indirect design methods from CPT measurements.

4 PILE DESIGN IN SANDS BY RELIABILITY APPROACH: THE CASE STUDIED

Sandy deposits set in Scanzano Jonico, were described in the companion paper (Vessia and Cherubini, 2007). Scanzano Jonico is set along Ionian coast in Basilicata region. Cherubini and Lupo (2002) studied physical and mechanical properties of these sand mixtures by means of laboratory and field tests. Main features are summarized in Table 7.

Table 7 Main features of sands in Scanzano Jonico site.

Soil type	Unit weight (kN/m ³)	Natural water content (%)	Relative density (%)	Internal friction angle (°)
Sand mixtures (0÷7m)	17.8-19.7	22.8-33.4	10-20	<30
Sand mixtures (7÷14m)			60-80	36-44

In this site two type of sand mixtures can be distinguished: the upper one rich in clay and the lower one rich in silt. In order to recognize the interface between the two strata, two CPTs are analysed by means of intraclass correlation coefficient RI, that is CPT4 and CPT5 and the interface has been found at 7m depth.

Table 8 Mean values ($\hat{\mu}$), variance values ($\hat{\sigma}$), scale of fluctuation (δ) and square root of variance reduction function values (Γ) of mechanical properties of the upper sand layer (up to 7m) used in the reliability-based pile design.

Variables	$\hat{\mu}$	$\hat{\sigma}$	δ	Γ
Friction angle ϕ'	32°	2.8°	2.8°	1.3m
Lateral earth pressure coefficient K ₀	0.5	0.02	-	-

Unit weight (kN/m ³)	18	-	-	-
Poisson's ratio ν	0.2	-	-	-
K/K_0	0.65	-	-	-
ϕ_f/ϕ'	0.9	-	-	-
Cone tip stress: q_c	1.89 (MPa)	0.89MPa	0.13MPa	1.3m
Coefficient α (I B)	150	-	-	-

Table 9. Mean values ($\hat{\mu}$), variance values ($\hat{\sigma}$), scale of fluctuation (δ) and square root of variance reduction function values (Γ) of mechanical properties of the lower sand layer (beneath 7m) used in the reliability-based pile design.

Variables	$\hat{\mu}$	$\hat{\sigma}$	δ	Γ
Friction angle ϕ'				
$L_{pile}=10m$	39°	0.6°	2.8°	0.5
$L_{pile}=12m$				0.41
Lateral earth pressure coefficient K_0	0.43	0.01	-	-
Unit weight (kN/m ³)	19	-	-	-
Poisson's ratio ν	0.2	-	-	-
Bearing capacity factor N_γ	17	2.5	-	-
Bearing capacity factor N_q	35	4.4	-	-
K/K_0	0.65	-	-	-
ϕ_f/ϕ'	0.9	-	-	-
Cone tip stress: q_c				
$L_{pile}=10m$	10.48 MPa	1.5MPa	0.73MPa	0.5m
$L_{pile}=12m$	10.83 MPa		0.76MPa	0.42
Equivalent cone resistance q'_{ca} :	D=0.4m	D=0.6m	D=0.4m	D=0.6m
$L_{pile}=10m$	11.02 MPa	11.12 MPa	1.87MPa	1.55MPa
$L_{pile}=12m$	12.18 MPa	12.00 MPa	0.73MPa	0.69MPa
Penetrometer bearing capacity factor k_c (Group I)	0.4	-	-	-
Coefficient α (I B)	200	-	-	-

Moreover q_c profiles (from CPT4 and CPT5) have been studied in terms of spatial variability in order to carry out a reliability based design of pile foundations. The most commonly used techniques of setting piles, in this area, is the bored concrete piles because of the high density of these sand mixtures. Two design methods are investigated: (1) the Bustamante and Ganeselli method (1982) which uses q_c measurements according to Eq. (11); (2) the indirect method that employs empirical correlations between q_c and mechanical properties of sandy soils according to Eqs. (1)-(10).

The reliability analysis has been undertaken by means of STRUREL program (1996) where first order reliability method is employed for calculating the reliability index β . In this study, β measures the reliability level of the pile bearing capacity according to direct and indirect method. Table 8-9 report the values of mechanical variables used in this study for the two sandy layers. The q_c measures are referred to CPT4.

Four combinations of pile geometry have been investigated: two diameters equal to 40 and 60 mm and two lengths equal to 10 and 12m. Mean values and variances are derived for mechanical variables from variability study presented in the companion paper (Vessia and Cherubini, 2007). Variances have been reduced by means of the variance reduction function Γ^2 , according to Vanmarcke (1977). This means that the larger the length (or the volume) over which the property is averaged, the higher the fluctuation of soil property tends to be cancelled in the spatial averaging.

Starting from Vanmarcke's formulation, Babu and Shastry (2004) suggested a corrected formulation for the variance reduction function which has been used in this study, that is:

$$\Gamma^2 = \frac{\delta}{L} \left[1 - \exp\left(-\frac{L}{\delta}\right) \right], \quad \frac{L}{\delta} > 0 \quad (14)$$

where L is the pile length (10 and 12m respectively) and δ is the scale of fluctuation of q_c recorded along L .

Results from reliability analyses are plotted in Fig.2, where resulting reliability index values from Bustamante and GIANESSELLI's method (called direct method), *dotted lines*, are compared with results from indirect method (Eqs. 1-10), *solid lines*.

At first, it is worth noticing that as β values increase the indirect method passes from overestimating to underestimating the bearing capacity of the pile with respect to the direct method. This means that indirect method, at a fixed reliability level, gives higher or lower bearing capacity values for piles. This is true for different pile geometries. This trend, represented by cross points on the plot, occurs at different load values, without an evident trend. Moreover, it can be seen that the cross points in three out of four cases analysed belong to β values ranging from 3.5 to 5.5 (β values suggested by Eurocode for structures of reliability class 2 fall within this interval).

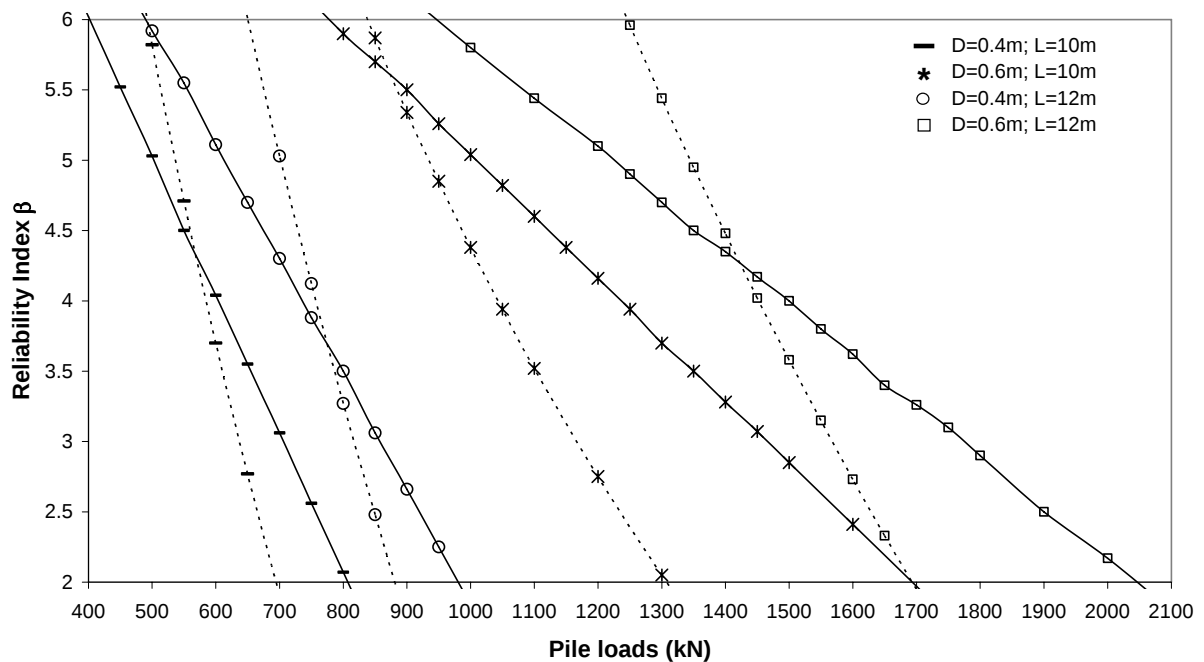


Fig.2 Trends of reliability index values vs. pile loads. **Solid lines** correspond to **indirect method** results; **dotted lines** relate to **direct method** results.

However, the gap between the predictions from the two methods is not constant for different pile dimensions. It increases over and under the cross point thus it cannot be estimated in advance in any of the considered cases. Therefore, both direct and indirect methods give similar trends to the curves the slopes are different: they increase with the increase of geometrical dimensions of the pile.

Such occurrence can be explained as resulting from different formulations of the two methods and different number of random variables taken into account. Finally, the indirect method predicts higher β values corresponding to a fixed load value, whereas apparently less reliable than the direct one. This is not always true but it is apparently not predictable.

5 CONCLUSIONS

The study carried out is based on field characterization of sandy deposits in the South of Italy at Scanzano Jonico site. These sands are also characterized in terms of inherent variability structure in the companion paper. This paper deals with reliability level of pile bearing capacity estimation, by means of cone tip stress from CPTs. Two types of methods for pile design are compared: the direct which proportions pile bearing capacity to cone tip stress (Bustamante and Gianeselli's method is here employed) and the indirect which uses soil mechanical properties drawn from empirical correlations to q_c measurements.

The analyses carried out by FORM have pointed out that indirect method gives higher pile bearing capacity values than the direct one but such a behaviour varies according to the variation of pile dimensions and to the loads magnitude.

An evident reason for such behaviour cannot be suggested because of the apparently lack of trend in the results. However it can be suggested that different formulations introduce different number of random variables which give higher or lower mean value to pile bearing capacity according to different formula and on the other hand differently affect the overall reliability level of pile bearing capacity.

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