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Interpretation of load tests on bored piles in the city of Matera

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Abstract. This paper briefly reviews several calculation methods to evaluate the bearing capacity expressed in terms of undrained strength (c_u) of piles bored in clay for their entire length and of piles whose tip is embedded into weak rock. The scope of the paper is to compare the results obtained with those from full-scale pile tests. These tests were carried out within the city of Matera which is well studied from a geotechnical point of view and for which there are statistically significant data on the geomechanical properties of the Subapennine Blue clays and the underlying Gravina Calcarenites. This represents the first attempt to show, on the basis of laboratory and field data, the influence of variability of the above mentioned soils and rocks on the real behaviour of deep foundations.

For piles completely bored into Matera clay, the calculation in terms of total stress are able to interpret sufficiently well the bearing capacity of the piles. For piles having their toe embedded in calcarenite, the variability of the strength of the weak rock presents greater uncertainties in the calculation of base and soft resistance.

1. Introduction

The design of foundations on piles depends on numerous calculation models adaptable to many different situations. However, given the uncertainties present in every phase of correct design including the characterisation of the soils and rocks and the choice of the calculation model, not to mention the influence of drilling and construction procedures, the verification of the full scale behaviour of deep foundations assumes a fundamental role.

In situations in which knowledge of the mechanical behaviour of the soils and rocks involved is considerable, including an evaluation of the intrinsic variability, tests on instrumented piles to evaluate the ultimate load bearing capacity are useful for testing calculation models and eventually calibrating them to take account of the local situation.

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This is the case of the soils and rocks of the city of Matera where piles are normally utilized in the foundations of buildings. These piles are generally drilled for their entire length in Blue Clays: In cases where Calcarenites are found at reasonably shallow depth (max 15–18 m) piles have their toe embedded for 2–3 diameters into the calcarenitic bed. Piles are constructed in clay by means of an auger and in calcarenites by means of an heavy duty auger or a drilling bucket.
No temporary support
(casing or bentonite) is used during boring.

2. The Geology of Matera City

The city of Matera covers an area of 18.5 km^2 astride the western edge of the Murge and the eastern edge of the Fossa Bradanica. The rocks and soils that outcrop in the area are described below, starting from the lowest in stratigraphic terms and then those belonging to the Murge Chain, as shown in Figure 1.

The oldest formation is the so called Altamura Limestone (Upper Cretaceous) belonging to the Murge Limestone. This formation constitutes the bedrock of the territory of Matera as a whole. The Altamura Limestone consists of limestone and dolomitic limestone which are mainly fine grained, also stratified in banks greater than a metre thick and variously fractured. They are arranged in a monocline with a prevalent direction of N 80° E and dipping to the north-west at a very small angle.

The Gravina Calcarenite (Calabrian) does intersect the Altamura Limestone, with slight angular discordance, through a conglomerate less than 2 m thick. These calcarenites constitute the geological environment in which the so called "Sassi" developed, that is the traditional habitations excavated in this weak rock. The car-

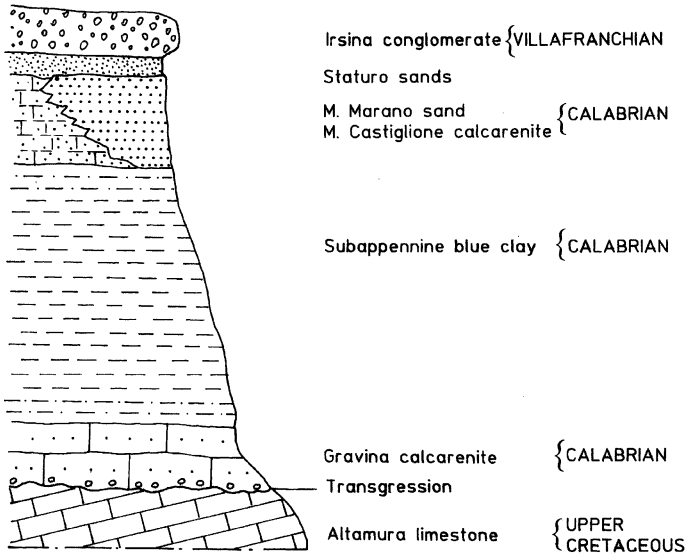


Figure 1. Stratigraphy of soils and rocks outcropping in Matera territory.

bonate content of these calcarenites varies between 85% and 98%. The non carbonate fraction consists essentially of quartz, feldspars, iron hydroxides, hematite, micas, kaolinite, montmorillonite, goethite and illite (Grassi, 1974).

In the north–north east of the inhabited area numerous trench excavations are visible from which parallelepiped shaped bricks were extracted for use in construction, as is also the case in Puglia. These Calcarenites also outcrop to the west–south west in a rapidly expanding zone of the city.

The Gravina Calcarenite is a carbonaceus detritic-organogenic weak rock with calcareous cement, with variable grain size distribution and degree of cementation. In some sections it presents sub-vertical fractures at average intervals of several metres, and rare faults.

The Subapennine Blue Clay (Calabrian) formation is in stratigraphical continuity to the Gravina Calcarenites. This can be lithologically defined as marly clays and/or clay silts mainly grey–blue in colour (yellowish in the surficial part) frequently enriched by sandy layers, especially in the upper part. These layers are mostly intact but can be fairly fissured. They outcrop with varying thickness in vast areas of the city. The mineralogical data of these clays are rather variable. The overall content of phyllosilicates varies from 29% to 71% with average values around 50%. The quartz content varies from 11% to 35% with on average value of 18%. Among clay minerals illite is prevalent with values varying from 25% to 72% the average value being 54%. The montmorillonite content varies from 8% to 56% with an average value of 36%. Lastly, the kaolinite content varies from 5% to 20% the average being perhaps 10%.

The Subapennine Blue Clays pass gradually to the Monte Marano Sands (Calabrian) which lie above. These have quartz grains, mainly yellow in colour, medium fine grained, with rare polygonal pebbles. In some zones the sands pass laterally into an mass which resembles soft rock, a sort of bench that is mainly pinkish in colour, consisting of crushed fossil remains and calcareous grains elements, and fairly well cemented (Monte Castiglione Calcarenites). The overall thickness of the Monte Castiglione Calcarenites and Monte Marano Sands does not exceed 10 m.

Moving upwards, slightly silty reddish quartz–mica sands, poorly sorted including pebbles in places (Staturo Sands, Villafranchian) follow. They do not exceed 2 m in thickness. These sands close the stratigraphic series together with the Irsina Conglomerate (Villafranchian) consisting of polygonal heterometric pebbles, fairly well rounded in a coarse to medium sandy matrix, pinkish-yellow in colour.

The Subapennine Clays and the Gravina Calcarenites together make up approximately 75% of the outcropping formations present in the territory of the city of Matera (Baldassarre, 1990).

3. Geotechnical Properties of the Blue Clays and Gravina Calcarenites

As this paper concerns the behaviour of piles in Blue clays alone or in Gravina Calcarenites, the description here is limited to the geomechanical characteristics of

these formations,on the basis of several data collected from boreholes performed in the city of Matera.

3.1. SUBAPENNINE BLUE CLAYS

From the point of view of grain size distribution (AGI 1990) these clays can be defined in general as silty clays or as clays with silt even if at times the silt fraction can exceed 50%. However, it is possible in the areas near the surface to find thin layers of coarser material (fine and medium size sands). In the Casagrande Chart the clays are located between high plasticity inorganic clays and to a lesser extent medium plasticity with a few samples being characterised by low plasticity as indicated in Figure 2. The Activity values lead to the classification of most of the samples analysed (around 85%) as inactive. The rest of the data indicate normally active clays.

The values of unit weight are around 19.5 kN/m^3, while the natural water content is generally around 26%. These clays are overconsolidated by erosion and their OCR exceeds values of 7 at the surface to ever lower values as the vertical geostatic tension increases. In general, the coefficient of lateral pressure at rest k_0 (Cherubini et al., 1989) is approximately equal to unity, from tests on samples taken at depths less than 20 m.

As for the strength parameters, it can be stated that the effective cohesion varies from 20 to 37 kPa, the effective friction angle varies from 19° to 24°, and the undrained cohesion, for the depths investigated (from 0 to 25 m), varies from 87 to 342 kPa. Preliminary statistical studies of the geotechnical index parameters have made it possible to evaluate with a certain degree of accuracy, given the numerous samples examined, the variability of geotechnical properties of this formation. The mean, and standard deviation values of some properties of Blue clays are listed in

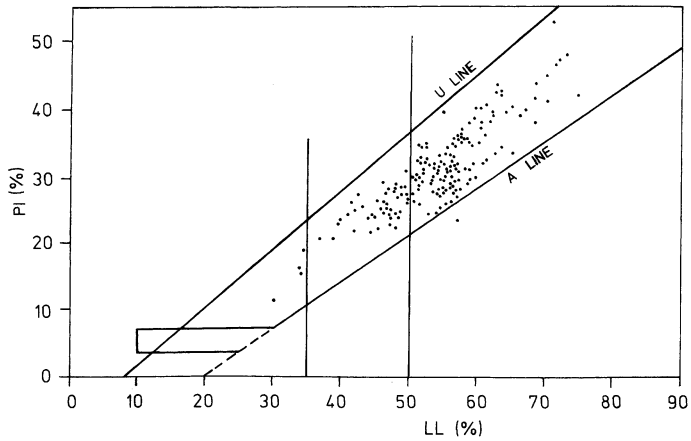


Figure 2. Plasticity chart of Subapennine Blue Clays tested in Matera territory. The A line delineates the boundaries between clays (above the line) and silts and organic soils (below the line). The U line defines the upper limit of the correlation between PL and LL (Budhu, 2000).

Table 1. Basic properties of Matera Blue Clays

	Liquid limit, LL (%)	Plastic limit, PL (%)	Bulk unit weight, γ (kN/m^3)	Natural water content, W (%)	Undrained shear strength, c_u (kPa)
Mean	54.2	23.1	19.5	25.9	178
Standard deviation	7.6	3.7	0.66	3.41	76
Coefficient of variation (%)	14.1	16.2	3.4	12.9	42.7
Number of tests	181	181	579	579	89

Table 1 with the variation coefficients and the number of samples examined for each property. The values of the variation coefficients reported are in line with those found by Cherubini (1997b) for numerous clays.

For depth intervals of 4 m., the mean and standard deviation values of the undrained shear strength of Matera blue clays are plotted in Figure 3. It can be seen that, in agreement with the overconsolidated state of the clay, the mean values of c_u are fairly constant with depth, as are the values of standard deviation. This diagram is of fundamental importance in the interpretation of the load tests on piles completely bored into the clay mass. In Figure 3(b) some specific values of c_u from two different boreholes are reported.

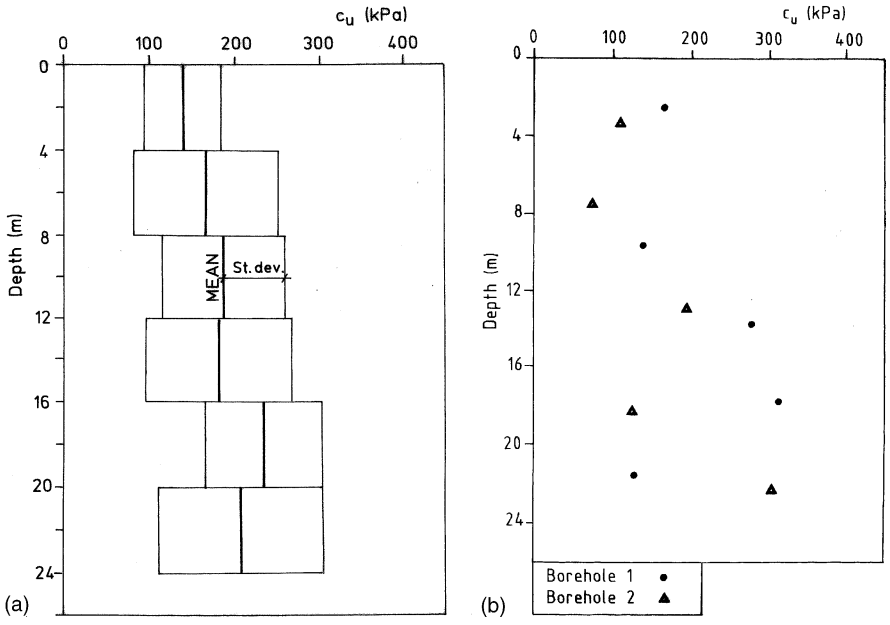


Figure 3. (a) Mean and standard deviation values of undrained shear strength of Matera Blue Clays for bands of 4 m. (b) Some values of undrained shear strength of Matera Blue Clays for samples of two different boreholes.

Table 2. Basic properties of Gravina Calcarenites

	Specific gravity, G_s	Dry unit weight, γ_d (kN/m^3)	Porosity, n (%)	Unconfined compression strength, σ_c (MPa)
Mean	2.73	16.1	40.8	2.51
Median	2.72	15.6	42.2	2.45
Standard deviation	0.019	2.0	7.1	1.13
Coefficient of variation (%)	0.6	12	17	45
Number of tests	28	28	28	28

3.2. GRAVINA CALCARENITES

This section contains the geotechnical data from laboratory tests on samples of Gravina Calcarenites from the territory of Matera. These rocks can be classified as "weak rocks" (Sciotti 1990). Without going into detail of the various classifications (which can be found in Dobereiner and De Freitas, 1986), it can be stated that rocks that show resistance to unconfined compression in the range of 0.6–12.5 MPa can be defined as weak (Gannon et al., 1999). These rocks, outcropping in different zones of Southern Italy, show very variable values of unconfined compression tests (from 0.8 to 9.9 MPa) as stated by Cotecchia et al. (1985).

The mean, median and standard deviation values as well as the coefficients of variation of some of the physical and geomechanical properties of the samples taken in Matera town are reported in Table 2.

As can be seen from Table 2, while the specific gravity (G_s) variability is extremely low, significant variability was found in the dry unit weight and the porosity n. From the results obtained for the resistance to unconfined compression σ_c it can be observed that the variation coefficient is extremely high (45%). This is evidence of the significant variability connected to the varying degrees of diagenesis that these rocks show even in restricted spaces. This variation coefficient is in good agreement with the values (from 35% to 53%) found for analogous calcarenite rocks (Evangelista and Pellegrino, 1990; Cherubini and Giasi, 1994).

4. The Ultimate Bearing Capacity of Piles Embedded in Blue Clays

In these clays the foundations of buildings often consist of bored reinforced concrete piles in groups of 2 or 3, having diameters of 40–60 cm and connected to a plinth.

Load tests (all of them controlled by one of the authors of this paper) carried out on three piles 0.6 m in diameter and 9, 10 and 11 m in length, have made it possible to elaborate the results obtained, comparing test results with those given by theoretical approachs (AGI, 1984; Mandolini, 1995).

The distance between the tested piles was some metres. Incremental loading tests were performed, holding each load value constant for 20 min before increasing to a higher value, measuring the settlements of the head of piles at the end of each loading phase.

To this end, a total stress approach appears best adapted for the times and methods of the tests performed since it would not be expected that drainage would take place during a 20 min loading increment in clay soils. Baldassarre et al. (1993) found that for analogous clays, even if with greater pile lengths and for diameters of 1.2 and 1.5 m, the total stress approach proved to be able to interpret the failure behaviour.

The calculation of the bearing capacity was therefore carried out in terms of total stresses adopting the α method for evaluating the shaft resistance, that is

$$P_{\text{lim}} = P_{\text{p}} + P_{\text{l}} - W = A_{\text{p}}(c_{\text{up}}N_{\text{c}} + \gamma L) + \alpha c_{\text{us}}\pi DL - W \tag{1}$$

where P_{p} is the toe bearing capacity, P_{l} is the shaft resistance capacity, A_{p} is the area of the pile base, c_{up} is the undrained shear strength for base resistance, c_{us} is the undrained shear strength for shaft resistance, N_{c} is the bearing capacity factor, assumed equal to 9, γ is the bulk unit weight of soil (19.5 kN/m^3), L is the length of the piles, equal to 9, 10 and 11 m, D is the diameter of the pile, equal to 0.6 m, W is the pile weight.

The value of c_{up} appropriate for base resistance can be taken as 190 kPa, as this is the mean strength for 8–12 m (Figure 3(a)) where the piles were founded (9–11 m). The value of c_{us} appropriate for shaft resistance can be taken as 175 kPa, as this represents a mean value over the length of a pile.

There are various possible approaches to the evaluation of the coefficient of adhesion α, (ERTC3, 1999; Lancellotta and Calavera, 1999). The proposal of Coduto (1994) correlates α to the undrained shear strength according to the following expression obtained by means of statistical analysis on numerous experimental data

$$\alpha = 0.32 + 250c_{\text{u}}^{-1.5} \tag{2}$$

This is valid for bored piles with $c_{\text{u}} \geqslant 51$ kPa; substituting for c_{u} an average value equal to 175 kPa, assumed to be practically constant for the length of the pile gives:

$$\alpha = 0.428 \tag{3}$$

The expressions of Sladen (1992), and Kolk and Van der Velde (1996) obtained from theoretical relations are functions of the relationship between undrained shear strength and effective vertical stress, that is (see also Patrizi and Burland, 2001):

$$\alpha = a\left(\frac{\sigma_{\text{vo}}}{c_{\text{u}}}\right)^{m} \tag{4}$$

where $a = 0.5$ and $m = 0.45$ (Sladen, 1992), $a = 0.55$ and $m = 0.3$ (Kolk and Van der Velde, 1996). Experience by the authors also suggests, $a = 0.55$ and $m = 0.36$. The application of these relations in our case is less easy with respect to Coduto's expression, but they are very useful in the case of normally consolidated clay masses.

Thus applying Equation (1) with the value of adhesion factor ($\alpha = 0.428$) and the mean values of c_{u}, the limit bearing capacity can be determined as indicated in Table 3.

Table 3. Comparison between bearing ultimate capacity obtained from calculations and using load test interpretation methods

		$L = 9$ m	$L = 10$ m	$L = 11$ m
Calculated P_{lim} (kN)	P^{-}_{lim}	1321	1473	1634
	Mean	1625	1800	1987
	P^{+}_{lim}	1940	2139	2354
Measured P_{lim} (kN)	Dalerci	1530	1710	1630
	Chin	1670	2230	1680

It has been demonstrated, by probabilistic studies relative both to cohesive soils and to cohesionless soils, that the variability of the mechanical characteristics of the soils make themselves felt in a way that affects both the base and shaft resistance (Cherubini 1991, 1994, 1997a). Without going into detail here a statistical "lower" value and a statistical "upper" value of the base and the shaft resistance for the piles of three different lengths were calculated assuming both an upper value of c_u (given by the mean value plus the standard deviation) and a lower value (given by the mean value minus the standard deviation). The standard deviation for c_u is assumed to be equal to 60 kPa, considering that the sample standard deviation effectively measured is higher, but is affected by various sources of errors. Because in the calculation model "intrinsic" variability must be considered, the value of measured standard deviation must be opportunely diminished.

In the case of the shaft resistance calculation the variance is reduced (Cherubini 1994), taking into account the scale of fluctuation, δ assumed by literature to be equal to 1 m (Cherubini 1997b), according to the relation

$$\Gamma^2 = \frac{\delta}{L} \tag{5}$$

where L is the length of the pile.

The lower, upper and mean values of P_{lim} are reported as such with dashed lines in the curve load-settlement of the three tested piles (Figure 4 (a)–(c)). The values of the final load calculated from the test results using the methods of Dalerci (1991) and Chin (1970, 1972) are also reported in Table 3. The method proposed by Chin assumes that the variation of settlement with respect to load can be interpreted by an hyperbolic curve near the collapse, whereas the Dalerci method starts from the study of the axial deformability of a pile on the basis of a constitutive model of a soil that is ideally elastic then perfectly plastic.

Evaluations made using other methods, for example Brinch Hansen's 90% (1963) method, give similar results. Nearly all these values fall (except for the value equal to 1630) within the field of variability indicated ($P^{+}_{lim} - P^{-}_{lim}$). Furthermore, these results, despite not being exhaustive given the scarce number of tests available, seem to confirm the role of the variability of the resistance properties of these clays in the evaluation of the ultimate bearing capacity of piles.

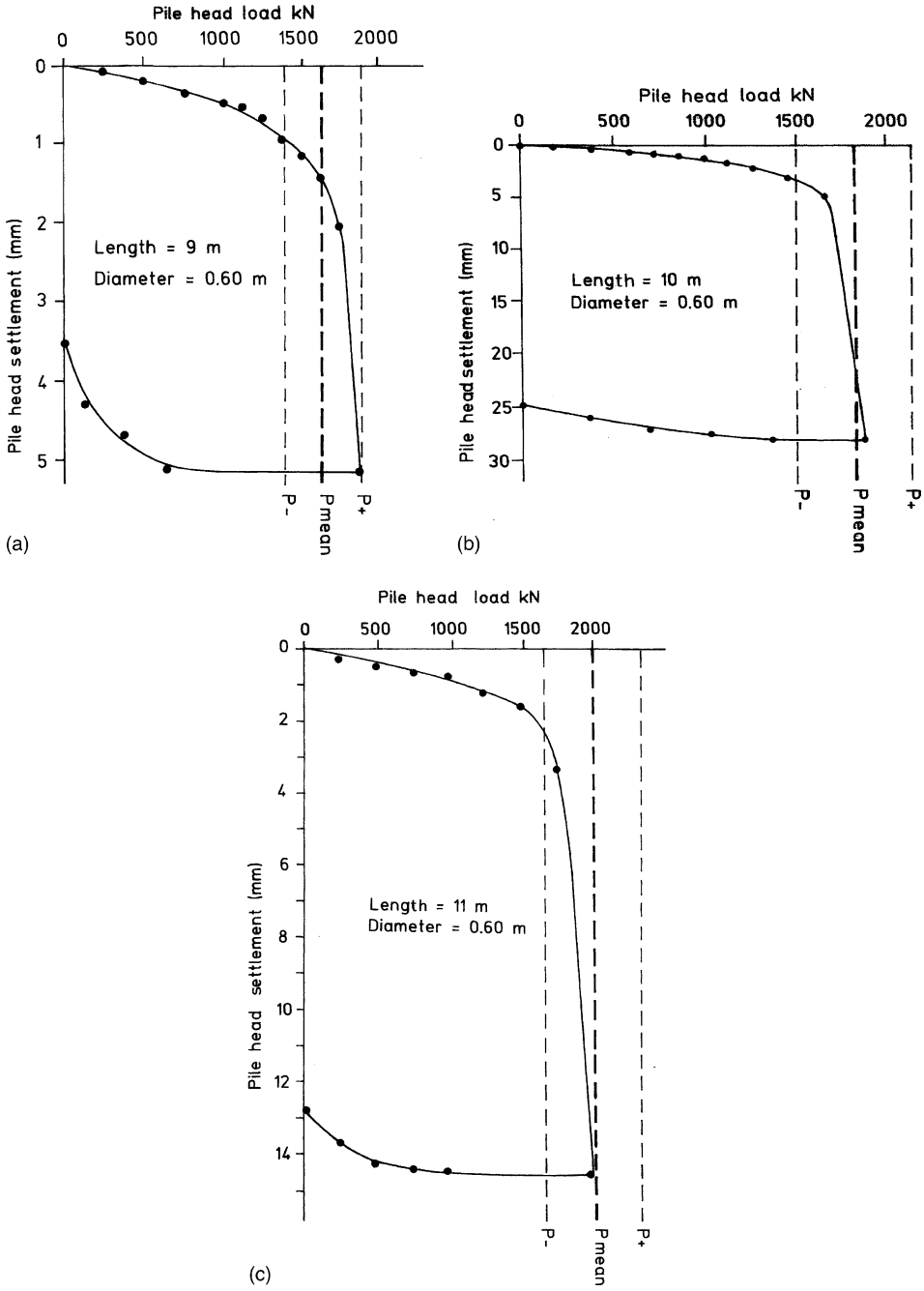


Figure 4. (a)–(c) Load-settlement curves for the piles subjected to tests.

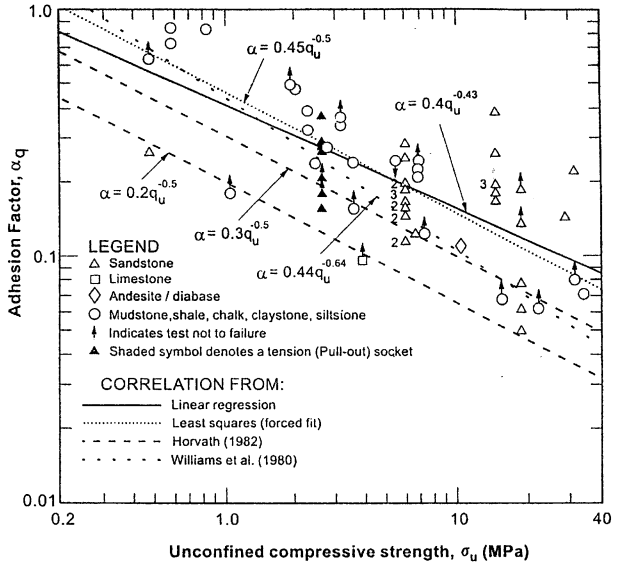


Figure 5. Relation between unconfined compression strength and adhesion factor for rocks of classes R1–R3. (from Seidel and Collingwood 2001, with minor modifications). The triangles reported concerns measured values for sandstones.

5. The Allowable Bearing Capacity of Piles Partially Embedded in Rock

Similar to the findings for piles in clay, the bearing capacity of a pile embedded into rock can be expressed as follows:

$$P_{\text{lim}} = A_1 \sigma_c \alpha_q + A_p q_{\max} \tag{6}$$

where A_1 is the area of the pile shaft embedded in the rock, A_p is the area of the pile base, α_q is the adhesion factor (distinguished here from the symbol α adopted for clay), σ_c is the unconfined compression strength of either rock and concrete (whichever is weaker), which in this case is rock, $q_{\max}$ is the unit base resistance.

A wide ranging study into the shaft resistance of deep structures embedded in rock has been carried out by Seidel and Collingwood (2001). Figure 5 contains the existing relations between the adhesion factor α_q and the unconfined compression strength of a rock sample, for rocks of roughness type R1–R3 as indicated in Table 4 which include the conditions of Gravina Calcarenites shaft roughness. The adhesion coefficient is defined as follows:

Table 4. Shaft roughness classifications (after Pells et al. 1980)

Roughness class	Description
R1	Straight, smooth-sided socket; grooves or indentations less than 1 mm deep
R2	Grooves 1–14 mm deep, >2 mm wide, spacing 50–200 mm
R3	Grooves 4–10 mm deep, >5 mm wide, spacing 50–200 mm
R4	Grooves or undulations >10 mm deep, >10 mm wide, spacing 50–200 mm

$$\alpha_q = \frac{f_{su}}{\sigma_c} \tag{7}$$

where f_{su} is the ultimate socket shaft resistance.

A general decrease in values of α_q can be observed with the increase of σ_c even if the dispersion of the data reported in the graph is noted. For the specific case of sandstones, the data are quite dispersed and do not cover the interval of unconfined compression strength of σ_c of interest for this paper (2.5 MPa). However, the following expression can be initially assumed to represent best fit conditions of shaft roughness R1 to R3 (Seidel and Collingwood, 2001):

$$\alpha_q = 0.4 \cdot \sigma_c^{-0.43} \tag{8}$$

As shown in Figure 5 this represents average behaviour for the rocks investigated.

As for the toe resistance, reference is made to the paper of Zhang and Einstein (1998), who, on the basis of previous models and documented load tests proposed three expressions that correlate the maximum unit resistance at the base of the pile with the unconfined compression strength:

$$\text{Lower bound} \quad q_{max} = 3.0\ (\sigma_c)^{0.5} \tag{9}$$

$$\text{Upper bound} \quad q_{max} = 6.6\ (\sigma_c)^{0.5} \tag{10}$$

$$\text{Mean} \quad q_{max} = 4.8\ (\sigma_c)^{0.5} \tag{11}$$

As can be seen from Figure 6, (Zhang and Einstein, 1998 modified) the two "extreme" relations include most of the data measured by various authors concerning unconfined compression strength and maximum unit resistance at the base of a pile.

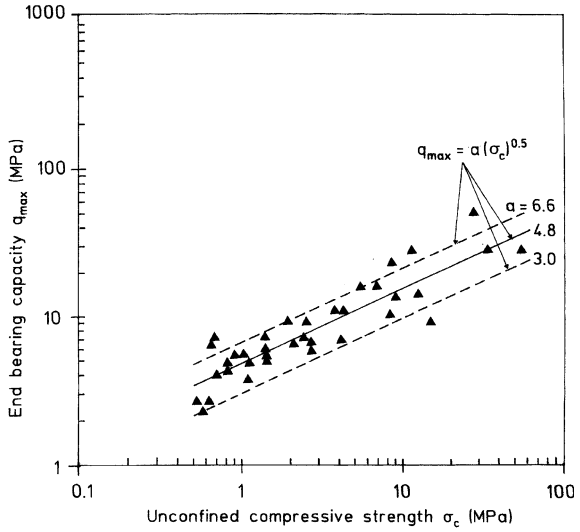


Figure 6. Design curves for q_{max} versus σ_c (from Zhang and Einstein (1998), simplified).

It should also be remembered that these expressions are valid for bored piles with a ratio of embedded depth over diameter greater than 3.

In the following a comparison between the calculated and measured values of bearing capacity in piles formed in calcarenite according to the "CASE" (CND, 1991) method, usually adopted in Italy, is developed. In the case in question, three non destructive tests were carried out on piles embedded into calcarenite in order to evaluate their load bearing capacity. The method that goes by the name of "CASE" is based on the assumption of the ideal plastic behaviour of the soil having viscous damping.

In the CASE method the pile is loaded in a dynamic way by means of a mass of 20–40 kN so that it applies an impact of controlled amplitude and duration controlled by the height of the fall and on the nature of the cushion placed at the head of the pile. The following parameters are measured for every impact (Chiesura and Heritier, 1989):

- Dynamic force in the head $f(t)$
- Acceleration of the head $a(t)$
- Transitory Settlement $s(t)$

The value of the static load of the pile used for null residual settlement is estimated using the expression:

$$P = R - D \tag{12}$$

where R is the reaction force of the pile-soil system, D is the impact force in the damping phase.

These assume values given by the following relations:

$$R = \frac{F(t_1) + V(t_1) \cdot I}{2} + \frac{F(t_2) - V(t_2) \cdot I}{2} \tag{13}$$

$$D = J \cdot [F(t_1) + I \cdot V(t_1) - R] \tag{14}$$

in which F is the maximum force transmitted to the pile at the times t_1 and t_2, V is the velocity reached by the pile base between the direct impulse t_1 and reflex impulse t_2, I is the impedance $= E{\cdot}A/C$, E is the modulus of (dynamic) elasticity $= 34{,}000{,}000$ kN/m^2, A is the cross sectional area of the pile, C is the velocity of the impact wave, J is the damping factor assumed equal to 0.20, where t_1 and t_2 indicate two different times when force and velocity are measured in the direct(t_1) and in the reflected(t_2) impulse.

All the piles subjected to tests were bored piles 50 cm in diameter and 10.5–9.7 m in length with the final 2 m embedded into Calcarenites, with the upper part passing through the Blue clays.

The tests provide the results in terms of allowable bearing capacity reported in Table 5.

Table 5. Values of the allowable bearing capacity of piles of different length L partially embedded into calcarenites, obtained using the "CASE" method

$L = 10.5$ m	$P_{all} = 1023$ kN $\pm$ 82
$L = 9.7$ m	$P_{all} = 973$ kN $\pm$ 78
$L = 9.7$ m	$P_{all} = 1017$ kN $\pm$ 81

The following calculations are performed in order to compare measured and calculated values of allowable bearing capacity.

From the diagram in Figure 5 using expression by Equation 8 α_q can be calculated by substituting the mean value of σ_c equal to 2.51 MPa (Table 2) found for Calcarenite:

$$\alpha_q = 0.4 \cdot 2.51^{-0.43} = 0.27$$

For which the average unit shaft resistance is:

$$f_{su} = \sigma_c \cdot \alpha_q = 2.51 \cdot 0.27 = 0.67\,\text{Mpa}$$

considering the measured embedding depth equal to 2 m, the value of the ultimate shaft resistance is:

$$P_l f_{su} \pi D L = 0.67 \cdot 0.5 \cdot 2.0 \cdot 3.14 = 2100\,\text{kN}$$

Passing on to the base resistance also using the mean value of σ_c and Equation 11 gives:

$$q_{max} = 4.8(\sigma_c)^{0.5} = 4.8 \cdot (2.51)^{0.5} = 7.6\,\text{MPa}$$

and multiplying by the area of the base gives the ultimate base resistance as:

$$P_p = 0.196 \cdot 7.6 = 1490\ \text{kN}$$

The total load, ignoring the contribution of shaft resistance due to the overlying clay and the self weight of pile, that with sufficient approximation can be considered to balance each other:

$$P_{lim} = P_p + P_l = 1490 + 2100 = 3590\,\text{kN}$$

Dividing by a safety coefficient equal to 2.5 (the CASE method has been calibrated on $F_s = 2.5$) gives, for the "mean" allowable bearing capacity in question:

$$P_{all} = \frac{3590}{2.5} 1436\,\text{kN}$$

a value which is comparable with the results obtained from full scale tests (Table 5), being higher than them by perhaps 40%.

Obviously it is also possible to take into account the variability of the ground, defining, as for the clays, statistical "minimum" and "maximum" values for the allowable bearing capacity obtained considering a "lower" value of σ_c (the mean minus the standard deviation) and an "upper" value (the mean plus the standard deviation). The standard deviation is reduced for the 2 m encased part relative to the

shaft resistance according to relation (5) valid also in this case where δ is the scale of fluctuation assumed also in this case to be equal to 1 m while L is 2 m, thus giving $\Gamma = 0.71$.

Carrying out the calculations and considering only the variability of the unconfined compression strength σ_c gives:

$$P_{all}^{+} = 1726\,\text{kN}$$

$$P_{all}^{-} = 1107\,\text{kN}$$

Also considering relations (9) and (10) that indicate the upper and lower values of the base resistance gives:

$$P_{all}^{2+} = 2015\,\text{kN}$$

$$P_{all}^{2-} = 940\,\text{kN}$$

Considering again from Figure 5 the lower relation for adhesion coefficient that is $\alpha_q = 0.2\sigma_c^{-0.5}$ a (P_{all}^{3-}) value is obtained equal to 835 kN.

Finally it can be stated that the results of the calculations are acceptable compared to the results of tests, not considering the relations of the upper bound for the ultimate unit base and shaft resistance.

In particular the considerations of unconfined compression strength variability alone allows the definition of a sufficient range of values of P_{all}; consequently P_{all}^{-} constitutes a value that could be assumed for design.

6. Conclusions

From the examination of several tests on bored piles carried out in the town of Matera, comparing the results obtained from full scale measurements with those derived from calculations of bearing capacity it can be concluded that:

- For piles completely bored into Matera clay the calculation models in terms of total stress, are able to interpret sufficiently well the bearing capacity of the piles deduced from incremental loading tests. The differences between calculated and measured values can be explained by various uncertainty factors; the variability of the undrained shear strength along the length of the pile seems to be a very important factor capable of explaining a significant part of the dispersion of bearing capacity values effectively measured.
- For piles having the toe embedded in calcarenite, the dispersions and uncertainty connected to the various factors present in the relations for the calculation of the base and shaft resistance are certainly greater than those for clays; in addition the variability of unconfined compression strength for Gravina calcarenites is rather high in correspondence with the extremely variable degree of diagenesis for such weak rock. However even in this case, the comparison between the measured and calculated load data shows sufficient agreement.

– These are preliminary results. More numerous data on full scale tests on piles are necessary for an efficient comparison between calculated and measured values of bearing capacity of piles in the two stratigraphic situations generally found in Matera city. However, the results are encouraging for further and more advanced analyses capable of improving the knowledge of the true behavior of these piles.

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