CPT and pile tests in granitic residual soils

Essais de pieux et CPT dans les sols résiduels de granite

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SYNOPSIS: Cone penetrometer tests and pile tests have been performed in residual granitic soils and results are compared. Two piles have been tested one 600 mm diameter bored pile and other 500 mm driven tube pile. Results are compared with some penetrometer tests previously done at the same site and also with results from bearing capacity formulae and triaxial shear strength characteristics.

1 INTRODUCTION

In the town of Guimarães, Azurem site, there is a number of outcropts of granite.

However, on the right bank of a small river crossing the area there are granitic residual soils and the compact rock is found at variable depths of a few meters up to near twenty meters.

The foundations of a set of buildings were done on piles whose points did not reach the rock since the buildings where of three stores only.

Reinforced concrete piles of 600 mm in diameter where at first bored.

During the boring of the piles it was noticed that although the residual granitic soils are rather impermeable, a good amount of water seeped into the hole and saturated the soil mass. That is, there is parcolation of water in seems across the soil mass.

For this reason the soil taken out from the casing of the pile was soft and it was supposed that softsing of the soil on the pile tip also occurs before fresh concrete is poured into the hole an casing removed.

For this reason the bearing capacity of the pile was under the value estimated on the base of results of static penetrometer tests performed beforehand.

Also due to that softning of the soil during pile instalation a change occured in the type of pile. Driven tube piles of 500 mm in diameter where performed and one of them tested.

A somewhat better behavior was found.

The loads on the piles were applied by a jack reacting against the center base of a large wooden box filled with sand.

Settlements were read in two dial gages installed on an independently supported beam with their points of the top of the pile. The dial gages were in oposite points of the pile head and an average of the readings was taken.

The settlements were checked by an independent system of leveling, reading a milimetric rule attached to the pile head whith a level.

From the larger pieces of soil comming out of the hole during the boring for bored piles at the final depth, some indisturbed samples have been taken and conventional triaxial tests performed on them.

Dinamic penetrometer tests were performed beforehand side by side with static penetrometer tests, and a correlation between them was made, taking results not only from this site but from other sites. For a weight of 15 kg. falling 0.40 m and a dynamic cone of 0.04 m in diameter it was found that for 0,25 m of dynamic penetration the number of blows are nearly equal to the point resistance in kgf/cm2 of the soil measured in the static (dutch) cone.

2 PILE TESTS

The first pile to be tested was bored pile No. 77 of 600 mm in diameter and 7,20 m in length. It is a concrete pile reinforced with 8 bars of 16 mm diameter and with helicoidal bands of 8 mm, spaced 25 m.

The loads were increased by steps og 20 tons first and 10 tons later up to 170 tons design load. At each step the load was kept constant until the dial reading to stabilize and the final load remained during 17 hours on the pile. Then the pile, was unloaded at the same rate up to zero load. The pile was unloaded during about 16 hours, before initiating a new loading cycle up to 190 tons. The residual (permanent) settlement for the first cycle of loading was 35.5 mm (Fig. 1).

With the maximum of 190 tons the pile remained a quarter of hour only. Then the unloading was performed at the same rate up to zero and there remained by half an hour. Due to this second cycle of loading and loading the residual settlement increased up to 47 mm. A third cycle of loading was then initiated up to 170 tons followed by the final unloading. The final permanent settlement was 51 mm.

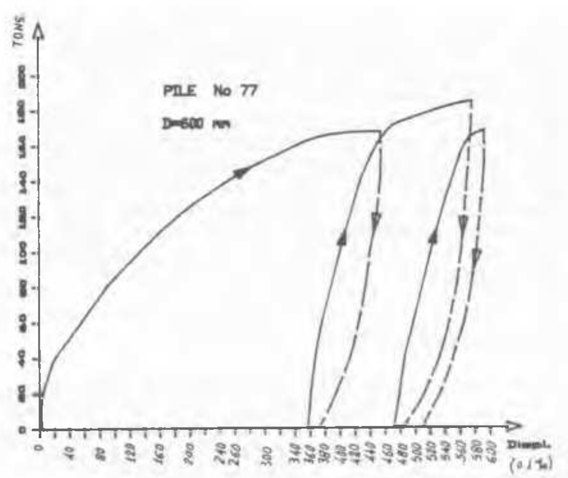

Fig. 1. Load-displacement curves

The second pile tested was pile No. 9, a driven tube pile, of 500 mm in diameter and 7,20 m in length. It is a concrete pile with a similar reinforcement.

In the first cycles the load was increased at the same rate up to 84 tons (design load) which remained on the pile during 21 hours. Then the pile was unloaded and remained with zero load during 1h15min.

The permanent settlement corresponding to this loading cycle was 8.5 mm. Then a second cycle of loading was performed again up to 84 tons. After the unloading in this cycle the permanent settlement increased by 1 mm only. In a third cycle the load was increased up to 120 tons and there remained during 16 hours. Then the pile was unloaded and remained 1h10min at zero load. The permanent accumulated settlement was 26 mm.

A last load cycle was performed up to 84 tons with no permanent settlement increase.

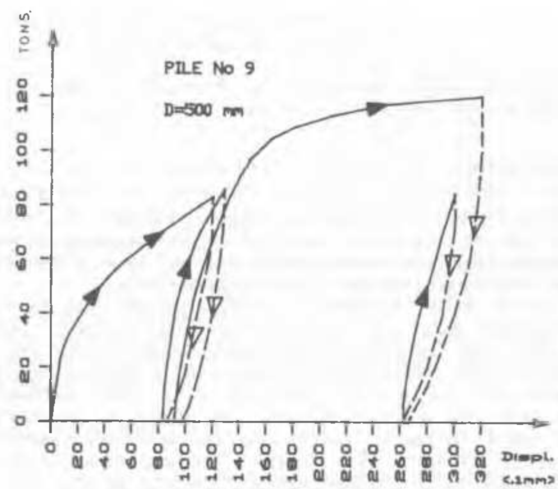

Fig. 2. Load-displacement curves

3 COMMENT ON PILE TEST RESULTS

The settlement for the 600 mm bored pile at "design" load was too large to be acceptable, and the pile with the maximum load of 190 tons was not far from "failure". However, the last cycle of loading at 170 tons showed an increase of permanent settlement of 4 mm only, compared with the 35,5 mm of the first loading cycle at the same load.

The 500 mm driven tube pile performed better. However, it must be said that the "design" loads are not in proportion to the cross section areas of the piles. Should we keep this proportion the "design" load of the 500 mm pile might be 118 t and in that case the maximum permanent settlement would reach about 18 mm or more for the first cycle of loading, which still is also unacceptable.

4 STATIC PENETROMETER TESTS

Dutch cone tests have been performed at the sites of the piles. The cross section area of the cone is 10 sq.cm. Fig. 3 shows the point stresses measured. The measurements were made each 0.25 m of penetration.

It can be seen that up to about 3 meters in depth the strength values are very low and that correponds to the first layer of brown rather clayey soil.

It follows a sharp increase in the point resistance corresponding to the residual granitic soil which under natural conditions is very compact. On the site of the pile No. 77 (600 mm) the point resistance went up to 15,000 kN/m² (15,000 kPa) with oscilations down to 8,000 kPa (80 kgf/cm²).

On the site of pile No. 9 (500 mm) the point resistance reached the maximum of 9,000 kPa (Fig. 4).

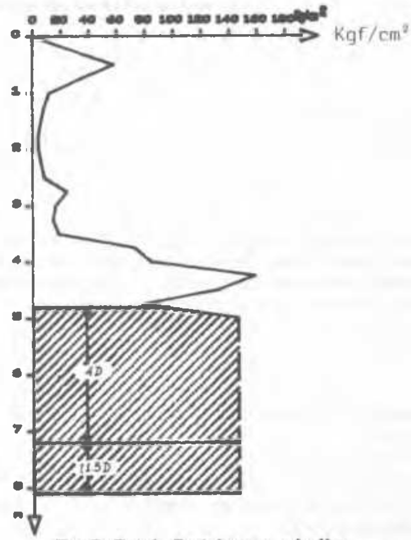

Fig.3 Point Resistances at the site of 600 mm pile.

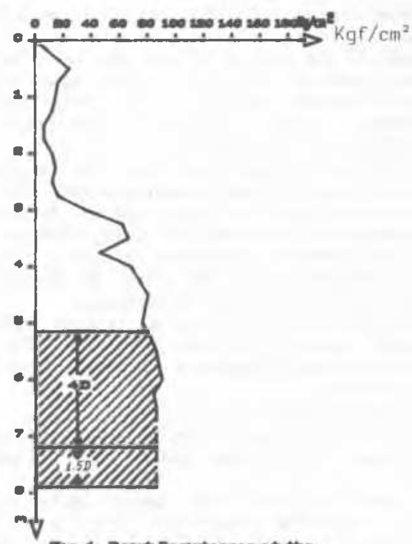

Fig.4 Point Resistances at the site of 500 mm pile.

5 BEARING CAPACITY FROM CONE TESTS

The development of failure of the soil near the point of a pile involves a height of several pile diameters.

If we consider the average values of the point resistance of the cone 1,5 pile diameter under and 4 diameter above the base of the pile for the ultimate point bearing capacity of the 600 mm bored pile would be about 400 tons. However, as refered above the soil was weakened during the boring of the pile due to the action of natural water. Therefore, the cone average resistance must be multiplied by a reduction factor. Since, lateral friction in this gives a little contribution

to the bearing capacity of the pile, we see that the reduction factor would be about 190/400 = 0.475.
For the tube driven pile that reduction factor would be about 0.75.

6 SOIL PARAMETERS

Granulometric curves of the soil are given in Fig. 5.

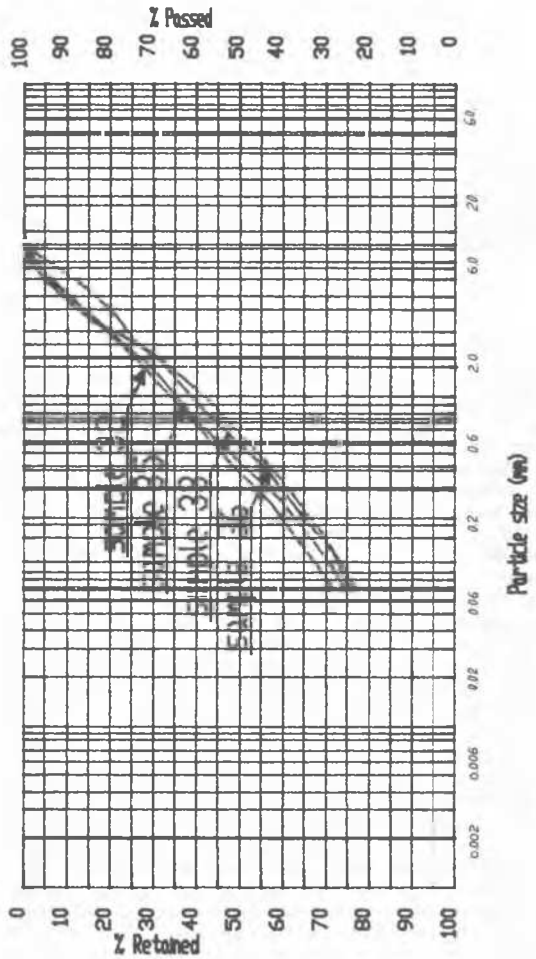

Fig. 5

The average water content of the samples tested was 23.2% with a minimum of 19.9% and a maximum of 25.4%. The average dry density was 15.7 kN/m³ with a minimum of 14.9 kN/m³ and a maximum of 16.6 kN/m³.
The average friction angle measured in triaxial tests 29.8^o and the average cohesion was 27 kPa. The maximum friction angle was 31^o and the minimum 29^o. The maximum cohesion was 36 kPa and the minimum was zero.

7 BEARING CAPACITY OF PILES FROM SOIL LAB SHEAR STRENGTH.

To obtain the bearing capacity of the piles from results of laboratory tests we have used Vesic's bearing capacity formulae (Bowles, 1977)·

$$q_{ult}=cN_c\cdot s_c\cdot i_c\cdot d_c\cdot g_c\cdot b_c\cdot+q\cdot N_q\cdot s_q\cdot d_q\cdot i_q\cdot g_q\cdot b_q+ B/2\cdot\gamma\cdot N_\gamma\cdot s_\gamma\cdot i_\gamma\cdot d_\gamma\cdot g_\gamma\cdot b_\gamma\cdot \qquad (7.1)$$

where

q_{ult} = bearing capacity per unit area of the cross section of the pile.
c = cohesion of soil
N_q = $K_p \exp(\pi \tan\phi)$
K_p = earth pressure coeficient $(1+\sin\phi)/(1-\mathrm{sen}\phi)$
N_c = $(N_q-1)\mathrm{ct}\phi$
N_γ = $2(N_q+1)\tan\phi$
s = shape factors; d = depth factors; i = factor for inclination and excentricity of load; g = factors for slope of the ground and b = factors for the inclination of the base of the foundation. In this case $i=g=b=1$. Also we can forget about the last item in (7.1) since diameter B of the piles are are small compared with depth D.
s_c = $1+(B/L)(N_q/N_c)$.(L=B in the case)
s_q = $1+(B/L)\tan\phi$
d_q = $1+2\tan\phi_1(1-\sin\phi_1)^2$ area tan (D/B)
d_c = $d_q-\dfrac{1-d_q}{N_c\tan\phi}$ (ϕ_1 and ϕ, friction angles of the soil above and under pile points, respectively).

Applying the above formula to the point bearing capacity of the pile of 600 mm in diameter, we obtain

Q_{ult} = 195 tons.

To this number we should add the lateral friction. Since the pile is short and the soil was remoulded during pile installation the contribution of lateral friction must be small.
The application of the same formula to the point bearing capacity of the tube driven pile, will give

Q_{ult} = 136 tons.

8 CONCLUSIONS

Strength of residual soils during boring of piles is reduced mainly if there is water seepage in the hole. Therefore, the average value of the dutch cone resistance of the soil must be multiplied by a reduction factor which was found to be in this case about 0.5.
For driven piles this reduction factor seems to be larger, i.e., strength of the residual soil has a smaller reduction during pile installation.
To fix safe values to these reduction factors much more experimental results are nedeed and from other sites.
The results obtained from laboratory tests and application of the bearing capacity formulas are very good. However, it is noted that the soil tested was obtained during the boring of the piles, therefore weakned, and, even so, we think those good results were obtained by chance.
For these soils the stellment of the piles on reloading under the "ultimate" load, is much smaller than the corresponding settlement for the first cycle of loading.

REFERENCES

Bowles, J.E. (1977). Foundation analysis and design. 2nd ed. MacGraw Hill Book C^o, New York.