On the use of RITA pile technique in Tunisia

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ABSTRACT: This paper discusses the use of RITA pile technique in Tunisian territory, peculiarly in the capital suburbs. More precisely, it presents the performance brought by such a technique in terms of increasing the frictional bearing capacity of a given pile foundation system compared to classical piles. First, an overview about this technique with related projects is exposed. Then, a particular case study highlighting the contribution of frictional contact offered by a single RITA pile to increase the bearing capacity is presented. This includes the results of geotechnical campaign, bearing capacity estimation, in situ loading test data and settlement prediction of a numerical loading test model. Based on the obtained results, the final section is devoted to a reflection about the main causes explaining the important frictional bearing capacity of RITA piles comparing to classical circular piles.

1 RITA pile technique : A brief overview

RITA piles (Recharge-Impulse Technologies and Apparatus) are cast-in-situ piles executed by means of high electrical pulse compaction. In situ, once the drilling operation and installation of steel bars elements are carried on, an injection of a 780 kg/m³ cement mortar accompanied with electrical pulse compaction in the horizontal direction is executed. Figure 1 presents a high voltage station ensuring the transmission of electrical pulses to a conductive electrode which, in turns, transmits the shock wave to the surrounding soil. As a result of this horizontal compaction, the final shape of a RITA pile is therefore similar to Frankie piles (Murthy, 2002). Hence, the diameter of such a pile is greater than the diameter of the initial drilling. In practice, the final diameter could attain 3 times the length of the initial drilling thanks to high electrical pulse compaction. Since its appearance in
Tunisia, Rita pile technique had been used in seven projects. In terms of design, it has been proven that estimated bearing capacities according to analytical and numerical methods are much lower than real bearing capacities obtained from loading tests. Similar experiences in Russia and Germany confirm such a statement. Thus, a particular concern should be pointed to current causes yielding to such an underestimation. In the following case study, only 25% diameter augmentation of a Rita pile will be considered for safety reasons.

2 Case Study

2.1 Site conditions

In Ain Zaghouan region (Tunisia), the investigated site is characterized by superposed silt and clayey layers with poor pressuremeter characteristics (Ménard modulus and limit pressure) up to 60 m depth. As exception, between 25 and 29 m depth, it is noted the presence of a relatively stiff layer clued by a significant increase in the Ménard modulus. Thus, the deep foundation solution was decided to be composed by a raft resting on floating Rita piles group embedded one meter depth in this stiff layer. Indeed, such a floating alternative has been chosen in reference with the high frictional component that a RITA pile can offer. Based on this criterion, there is no need to overpass the stiff layer looking for enhanced bearing capacity at higher depth of embedment. Figures 2 and 3 present, respectively, the profiles of limit pressure and pressuremeter modulus. In the same context, frictions and cohesions estimated by direct shear tests are presented in Table 1.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (m)</th>
<th>(Cohesion, friction) (KPa,°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clayey sand</td>
<td>3</td>
<td>25, 20</td>
</tr>
<tr>
<td>Grey carbonate clay</td>
<td>10</td>
<td>35, 4</td>
</tr>
<tr>
<td>Silty brown clay</td>
<td>2.5</td>
<td>35, 4</td>
</tr>
<tr>
<td>Grey compressible clay</td>
<td>27</td>
<td>30, 4</td>
</tr>
<tr>
<td>Brown silty clay</td>
<td>2</td>
<td>20, 25</td>
</tr>
<tr>
<td>Sandy clay</td>
<td>2</td>
<td>25, 25</td>
</tr>
<tr>
<td>Silty clay</td>
<td>5.5</td>
<td>25, 25</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>4</td>
<td>1, 33</td>
</tr>
</tbody>
</table>

Table 1 - Cohesion and friction angle of crossed layers
2.2- Bearing Capacity

2.2.1- Bearing capacity according to Pressuremeter data

Based on pressuremeter results exposed in Figure 2, the design of the projected group of piles (Final diameter = 0.4m, length = 20.5 m) in terms of bearing capacity and settlement was carried out. Analytically, the bearing capacity of a single pile according the French code DTU.13.2 is evaluated as a function of the limit pressure profile. The two expressions from which one can determine the bearing capacity based on pressuremeter results can be expressed as:

\[ Q_p = k_p P_l A \]

\[ Q_s = P \int_0^h q_s(z) \, dz \]

Qp: Tip bearing capacity term.
Kp: Dimensionless bearing capacity factor depending on soil classification.
Pl: Limit pressure corresponding to the embedment depth.
A: Pile circular section
Qs: Frictional bearing capacity term
P: perimeter of pile section
qs(z): unit frictional variation depending limit pressure profile (Figure 2)

For the ultimate state, the bearing capacity is predicted as: \( Q_{ul} = \frac{f_\alpha P + Q_s}{1.4} \)
Similarly, for service loading, the bearing capacity can be expressed as: \( Q_{sr} = \frac{f_\alpha}{1.4} \)
Based on expressions presented above, Table 2 presents the obtained bearing capacities.

<table>
<thead>
<tr>
<th>Type of loading</th>
<th>Bearing Capacity (Tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service loading</td>
<td>72</td>
</tr>
<tr>
<td>Ultimate State</td>
<td>90</td>
</tr>
</tbody>
</table>

Table 2 - Service and ultimate bearing capacity for a single pile (D=0.4m, L=22m) according to the French code D.T.U.13.2

Table 1 shows that the ultimate state bearing capacity is related to 71% of the bearing capacity \((Q_p + Q_s)\) of a single pile which, in turns, corresponds to 2.5 cm of admissible settlement.

2.2.2- Bearing capacity according to NAVFAC DM 7.2

Based on friction angles and cohesions presented in Table 1, the bearing capacity of the same single pile studied in section 2.1 according to NAVFAC DM 7.2 (1984) standard was evaluated. Analytically, the bearing capacity expression according to this method can be written as:

\[ R_u = R_b + R_s \]

\( R_b \): pile tip resistance
\( R_s \): pile shaft resistance.

Since the surrounding soils in contact with the pile shaft are assumed cohesive (Table 1), the expressions of \( R_b \) and \( R_s \) are as follows:

\[ R_b = 9 \cdot C_u \cdot A_b \]
\[ R_s = \sum_{j=1}^{n} a_j \cdot C_{uj} \cdot A_{s,j} \]

\( C_u \): Undrained shear strength at the pile base
\( A_b \): area of pile base
\( a_j \): Skin friction coefficient or adhesion factor
\( C_{uj} \): Undrained cohesion in the j-th layer
\( A_{s,j} \): Area of pile shaft in the j-th layer
For getting closer to the real behavior of a RITA pile characterized by high frictional contact with surrounded soils, a maximum adhesion factor $\alpha=1$ was considered. Table 3 presents the obtained results.

<table>
<thead>
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<tbody>
<tr>
<td>Service loading</td>
<td>73</td>
</tr>
<tr>
<td>Ultimate state</td>
<td>91</td>
</tr>
</tbody>
</table>

Table 3- Service and ultimate bearing capacity for a single pile (D=0.4m, L=22m) according to NAVFAC standards

As exposed in Table 2, the maximum bearing capacity equals 910 KN. Above this value the surrounding soil (in contact with the studied pile) is supposed to be importantly mobilized in friction and therefore, large settlements will occur. This statement should be confirmed by an in situ loading test.

### 2.3- Loading test

Figures 4 and 5 present the instrumentation of the loading test carried on a single RITA pile (D=0.4m, L=20.5m) and its load-settlement curve, respectively.

![Figure 4- Instrumentation of the loading test](image)
According to load-settlement curve in Figure 6, a vertical force of 120 T yields to 3.88 mm of settlement. Hence, this load can easily be adopted for service design since it corresponds to a largely admissible settlement. However, according to both French and NAVFAC codes, proposed ultimate bearing capacities loadings do not overpass 91 tones. Consequently, one can clearly remark that the bearing capacity of a RITA pile is underestimated comparing to experimental results. Meaning that the behavior of a RITA pile, priory the frictional one, is far to be compared to a classical pile, hence cannot be correctly estimated using standardized codes. Alternatively, as the tip bearing capacity term only depends on the stiffness (deformation modulus) of the embedment layer, the difference between a classical pile and a RITA one having the same base dimensions should not be important herein. In order to confirm these statements, we have built up a numerical model simulating a loading test of a classical pile having the same characteristics of the described RITA pile (D=0.4m, L=20.5m) and in contact with the same surrounding layers. This work will be exposed in the following.

2.4- Numerical modeling of a classical loading test

In axisymmetric conditions, a classical circular pile having the same dimensions of the studied RITA pile (D=0.4m, L=20.5m) and in contact with the same layers described in section 1 had been submitted to a compressive loading test. As we have already presented in Table 1 and Figure 2 the characteristics of each layer in terms of deformation moduli, frictions and cohesions, we just present in Table 4, the deformation parameters of the simulated pile including the compressive strength of its material (concrete), the elastic modulus and the shear one.
Compressive strength (MPa) | 20.68
---|---
Elastic modulus (MPa) | 21525.65
Shear modulus (MPa) | 9040.74

Table 4- Deformation parameters of the simulated circular pile (D=0.4m, L=20.5m)

A rigid plate element between the simulated pile and the applied load had been placed to homogenize the vertical displacement in the whole circular section. Further, an interface element had been employed at the contact between the pile and each surrounding layer to better simulate the frictional behavior. Plaxis tutorial recommends interface coefficients simulating the frictional contact for sands and clays respectively equal to 0.67 and 0.2. However, as our objective is to get the closer possible to a RITA pile behavior, peculiarly in friction, a coefficient of 0.9 was adopted for both clayey and sandy soils. Although previous works on frictional contact claimed that interface use has no important effect in moderate deformations analysis (Sheng et al., 2007), this choice is agreed to fairly asses the behavior of a RITA pile comparing to a classical one having the same in situ conditions. Figures 6 and 7 present the described model and its load-settlement curve, respectively.

As shown in Figure 5, a 90 tones load corresponds to 7.2 mm of settlements. Besides, according to the slope change of the load settlement curve, it seems that frictional contact between pile and surrounding soils has importantly been mobilized for vertical loadings greater than 60 tones. Hence, a maximum bearing capacity of 90 tones should be recommended based on these results.
Elastic settlement relationship proposed by Poulos (1988) for this case study gives similar results. Figure 8 presents therefore the plots corresponding respectively to the loading test of a RITA pile, the elastic load settlement-curve and the numerical plot for an ordinary pile having the same dimensions and in situ conditions of the studied RITA pile.

According to the three plots, it is clear that the performance of a RITA pile in terms of frictional contact is far to be similar to a classical circular pile. Indeed, during its compression, a RITA pile behaves as a contractive sandy soil sample submitted to a compressive loading in triaxial conditions. This can clearly be remarked from the load-settlement slope which slightly decreases during the compression. Meaning that mobilized shear along the contact surface between the RITA pile and
surrounding soils in the range of 0-120 tones is encore much lower than the available shear strength which could be offered by such a surface. However, the frictional contact between a classical pile and the same surrounding layers appears to be importantly mobilized beyond 60 T. This naturally yields to more important settlements for the classical pile than the RITA one.

3 Discussion
According to the obtained results, it has been primarily proven that a given RITA pile offers a more important frictional bearing capacity comparing to a classical one having the same dimensions and in contact with the same surrounding layers. Priory, the main causes contributing to the important frictional bearing capacity of RITA piles can be explained as following:

- Rita piles contribute by their execution technique for improving the surrounding soil parameters including the limit pressure (PL*), deformation modulus, friction angle and cohesion and so the friction bearing capacity term.
- The shape of Rita pile itself offers a longer contact surface with surrounding soil comparing to a classical pile. Therefore, frictional bearing capacity term should be naturally higher.
- Thanks to the curvature shape offered by a RITA pile, the distribution of normal stresses along the contact surface with surrounding soils is lower than the classical pile case for the same applied load. In the same context, the mobilized shear stress along the interface RITA pile-soil is lower than the mobilized shear stress along the interface ordinary pile-soil. Therefore, a RITA pile allows us to go further during a loading test as it needs higher loads than classical piles to mobilize a certain amount of frictional contact.

4 Conclusion
Using electrical horizontal compaction, Rita piles seem to offer a considerable performance in terms of frictional bearing capacity. Further works should axially turns around the collection a rich data base of case studies similar to the one exposed herein and build up in a second step a new set of friction design charts special for such a class of piles.

5 References