Three-dimensional analysis of laterally loaded barrette foundation using Plaxis 3D

Djamila BAHLOUL¹, Belkacem MOUSSAI², M. ASCE

¹PhD student, USTHB University, Algiers – Algeria
²Professor, USTHB University, Algiers – Algeria

ABSTRACT: The aim of this paper is to investigate the effect of soil constitutive models (elastic model, Mohr Coulomb model and Hardening Soil model), nonlinearity of the barrette material as well as the loading direction on the response of laterally loaded barrette foundation. The analyses were performed on documented lateral load test on a barrette having a cross section of 2.8 m by 1.2 m and a length of 11 m embedded in sandy soil deposit. The finite element code Plaxis 3D – 2015 was used to simulate the barrette load test. The results indicate that the response of the barrette to lateral loading is highly affected by the loading direction and nonlinear behavior of the soil and barrette material.

INTRODUCTION

Barrettes are used to carry heavy loads and therefore a single barrette can replace a group of conventional piles, resulting in savings in construction time, quantity of concrete and steel, size of pile caps, etc.

When foundations have to be designed to resist lateral loads and bending moments in a preferential direction in addition to vertical loads, the use of rectangular shapes is more advantageous than circular sections (Ramaswamy and Pertusier, 1986).

The performance of barrette foundation to lateral loadings is not well documented in the literature. Zhang (2003) performed a three dimensional analysis of two laterally loaded barrettes tested in Hong Kong using FLPIER finite element software. He reported that the lateral response of the barrette is influenced by loading direction and the lateral load capacity is the greatest when the loading is along the major axis. Similar results were reported by El Wakil et Nazir (2013) on a small scale model of barrettes tested in laboratory.
There have been a number of studies published which investigate the behavior of piles under lateral loading with the assumption that the pile material is linear (Broms, (1964); Matlock (1970), Poulos (1971), Poulos et al. (2001), Basu and Salgado (2007, 2008), Comodromos et al. (2009) and Choi et al (2013), etc). However, this assumption may lead to unrealistic response of the piles subjected to high rates of loading. Recently, Conte et al. (2012) reported that the assumption of the pile material as linear leads to an overestimation of the lateral capacity of the pile.

In this study, the effect of soil constitutive models (elastic model, Mohr Coulomb model and Hardening Soil model), nonlinearity of the barrette material as well as the loading direction on the response of laterally loaded barrette foundation is numerically investigated using the finite element software Plaxis 3D. The numerical analysis was conducted on documented lateral load test on a barrette reported by Conte et al. (2012).

CONSTITUTIVE MODELS

_The Mohr Coulomb model (MC) _is a linear elastic perfectly plastic model with Mohr-Coulomb failure criterion. The model requires five input parameters namely Young’s modulus ($E$), Poisson’s ratio ($\nu$), cohesion ($c$), friction angle ($\phi$) and dilatancy angle ($\psi$).

_The hardening soil model (HS) _is an advanced model applied for all types of soils and is based on shear and compression hardening. This model supersedes the hyperbolic model of Duncan & Chang (1970) by using the theory of plasticity rather than the theory of elasticity and by including soil dilatancy and yield cap (Schanz et al., 1999). In contrast to Mohr-Coulomb model, the yield surface of a hardening plasticity model is not fixed in principal stress space but it can expand due to plastic straining (Brinkgreve et al., 2012).

The hyperbolic relationships for standard drained triaxial tests tend to yield curves, which can be described by:

\[-\varepsilon_1 = \frac{1}{E_i} \frac{q}{1-q/q_a} \quad \text{for: } q < q_f \quad (1)\]

\[R_f = \frac{q_f}{q_a} \quad (2)\]

Where $E_i$ is the initial tangent Young’s modulus, $q_a$ is the asymptotic value of the shear strength, $q$ is deviatoric stress, $\varepsilon_1$ is vertical strain, $q_f$ is the ultimate deviatoric stress derived from the Mohr-Coulomb failure criterion and $R_f$ is the failure ratio.

$E_i$ is related to $E_{50}$ by:

\[E_i = \frac{2E_{50}}{2-R_f} \quad (3)\]
The confining stress dependent stiffness modulus for primary loading ($E_{50}$), for unloading and reloading ($E_{ur}$) and for oedometer stress-strain conditions ($E_{oed}$) are given by the following equations:

$$E_{50} = E_{50}^{ref} \left(\frac{c \cos \varphi - \sigma_z \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi}\right)^m$$  \hspace{1cm} (4)

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cos \varphi - \sigma_z \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi}\right)^m$$  \hspace{1cm} (5)

$$E_{oed} = E_{oed}^{ref} \left(\frac{c \cos \varphi - \sigma_z}{K_0} \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi}\right)^m$$  \hspace{1cm} (6)

Where $E_{50}^{ref}$, $E_{ur}^{ref}$, $E_{oed}^{ref}$ are reference stiffness modulus corresponding to the reference confining pressure $p^{ref}$, $m$ is the amount of stress dependency, $K_0^{rc}$ is the $K_0$ value for normal consolidation, $c$ and $\varphi$ are strength parameters.

The triaxial modulus controls the shear yield surface and the oedometer modulus controls the cap yield surface.

The shotcrete constitutive model was developed and implemented by Bert Schädlich for Plaxis b.v. in 2012-2014. The work is based on two conference papers (Schädlich & Schweiger 2014, Schädlich et al. 2014), which have been extended by additional details on the implementation of the model.

The constitutive model can account for time dependent strength and stiffness, strain hardening/softening in tension and compression, creep and shrinkage. Parts of the model are based on previous work by Schütz et al. (2011) and Meschke et al. (1996).

The primary objective of this constitutive model was the modelling of shotcrete behavior for tunneling applications, but it can also be used for cast concrete, jet grout and other cement-based materials. This model takes into account the non-linearity of the material behavior (Schädlich, 2014). The input parameters of the shotcrete model are given in Table 1.

The behavior in compression follows an approach proposed by Schütz et al. (2011). The stress-strain curve is divided in four parts (Fig. 1): Part I - quadratic strain hardening, part II - linear strain softening, part III - linear strain softening and part IV - constant residual strength. Due to the time dependency of the involved material parameters, a normalized hardening/softening parameter $H_c = \varepsilon_3^p / \varepsilon_{cp}^p$ is used, with $\varepsilon_3^p$ = minor principal plastic strain (calculated from $F_c$) and $\varepsilon_{cp}^p$ = plastic peak strain in uniaxial compression.
The behaviour in tension is linear elastic until the tensile strength $f_t$ is reached (Fig. 2). Linear strain softening follows, governed by the normalized tension softening parameter $H_t = \varepsilon_{1p} / \varepsilon_{tu}$ with $\varepsilon_{1p}$ = major principal plastic strain (calculated from $F_t$) and $\varepsilon_{tu}$ = plastic ultimate strain in uniaxial tension.

$$f_{ty} = f_t \cdot (1 + (f_{tu} - 1) \cdot H_t)$$
Similar to softening in compression, $\varepsilon_{tu}$ is derived from the fracture energy in tension, $G_t$.

$$\varepsilon_{tu}^{P} = \frac{2 \cdot G_t}{(1 + f_{tu}) \cdot f_t \cdot L_e}$$

Once the residual strength $f_{tu} = f_{tu} \cdot f_t$ is reached, no further softening takes place.

**Fig. 2. Tension softening**

**NUMERICAL ANALYSIS**

Figure 3 presents the layout of the barrette load test reported by Conte et al. (2012). The barrette has a cross section of 2.8 m by 1.2 m and a length of 11 m embedded in a sandy soil deposit (Fig. 4). A cap was connected at the barrette head with a cross-section dimension of 1.5 m by 3.1 m and a thickness of 1.5 m. The barrette load test was simulated using the finite element code Plaxis 3D - 2015. The barrette was modelled as a volume barrette using the nonlinear constitutive model (Schädlich & Schweiger, 2014) with the parameters shown in Tables 2 & 3. The soil was modelled using the Mohr-Coulomb model (MC) (Brinkgreve et al. 2012).

The soil encountered consists of dense sand with interbedded thin layers of gravel and sometimes of silty sand. The standard penetration tests provided values of the SPT blow counts ranging between 30 and 60. The cone penetration tests provided values of $q_c$ from 5 to 15 MPa at depths less than 5 m. Beyond this depth, $q_c$ ranged from 15 to 30 MPa (Conte et al., 2012). The soil parameters were based on these field investigations and were adjusted through a back analysis of the barrette load test results. Table 4 summarizes the soil parameters used in the numerical analysis. The groundwater table is located at a depth of 2.5 m from the ground surface.

Figure 5 shows the FEM model of the barrette load test analysis under 3D condition, composed of ten node volume elements. In order to minimize the effect of
the artificial boundaries on the pile load test results, a mesh of 30m x 50m x 20m has been chosen.

The barrette load test simulation sequence included an initial phase which corresponds to the initial stress condition, followed by a second phase in which the barrette is wished in place. Then, horizontal loads were applied on the barrette head.

Table 2: Properties of the barrette concrete

<table>
<thead>
<tr>
<th>$\gamma_{KN/m^3}$</th>
<th>$E_{28}$ GPa</th>
<th>$v$</th>
<th>$f_{c28}$ MPa</th>
<th>$f_{t28}$ MPa</th>
<th>$\psi$</th>
<th>$f_{cun}$</th>
<th>$\varepsilon_{cp}^p$</th>
<th>$G_{c28}$ KN/m</th>
<th>$f_{tu}$ M Pa</th>
<th>$G_{t28}$ KN/m</th>
<th>$\alpha$</th>
<th>$\phi_{ma}^c$</th>
<th>$\phi_{cun}$</th>
<th>$\varepsilon_{fin}^{shr}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>33.6</td>
<td>0.2</td>
<td>33.2</td>
<td>3.10</td>
<td>0.85</td>
<td>0.05</td>
<td>-1.20E-03</td>
<td>70</td>
<td>0</td>
<td>7</td>
<td>18</td>
<td>37</td>
<td>2</td>
<td>-0.8E-03</td>
</tr>
</tbody>
</table>

Table 3. Properties of the barrette reinforcement steel

<table>
<thead>
<tr>
<th>$E_a$ (MPa)</th>
<th>$\nu_s$</th>
<th>$F_y$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>210000</td>
<td>0.3</td>
<td>430</td>
</tr>
</tbody>
</table>

Table 4. Soil parameters

<table>
<thead>
<tr>
<th>Model</th>
<th>Upper sand layer</th>
<th>Lower sand layer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EL</td>
<td>MC</td>
</tr>
<tr>
<td>$\gamma_{unsat}$ [kN/m$^3$]</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>$\gamma_{sat}$ [kN/m$^3$]</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>-</td>
<td>33</td>
</tr>
<tr>
<td>$\psi$</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>$E$ [MN/m$^2$]</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>$E_{50}^{rel}$ [MN/m$^2$]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$E_{osed}^{rel}$ [MN/m$^2$]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$E_{ur}^{rel}$ [MN/m$^2$]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$\nu_{ur}$</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>$m$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$R_f$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$R_{inter}$</td>
<td>0.67</td>
<td>0.67</td>
</tr>
</tbody>
</table>
Fig. 3. Barrette load test layout

Fig. 4. Soil profile at the site of the barrette load test

Figure 5. Finite element mesh
5 RESULTS AND DISCUSSIONS

Figure 6 shows the measured and predicted load – horizontal displacement curves when the barrette is loaded along the major axis. It can be seen that the numerical results match the barrette load test results quite well. However, when the barrette is modelled using linear model, the horizontal displacements are significantly reduced. Thus, the use of the elastic model lead to an underestimation of the barrette horizontal displacements and the amount of the displacement underestimation increases as the load level increases.

![Figure 6. Measured and predicted load - deflection curves using linear elastic and non-linear behavior of the barrette foundation](image)

**Effect of soil constitutive model**

In order to investigate the effect of soil constitutive model on the lateral response of the barrette foundation, three models were used, namely linear elastic model, Mohr Coulomb model and hardening soil model. The soil parameters used in the numerical analysis are given in Table 4.

The non-linear models (Mohr Coulomb model and hardening soil model) showed quite similar results, but the linear elastic model revealed significantly lower displacements (Fig. 7). The difference in horizontal displacements between the linear elastic model and the non-linear models increases as the applied load increases.

When the non-linear models are used, the horizontal displacement at an applied load of 4.6MN is about 3 times greater than that when the linear elastic model is used, which indicates that the linear elastic model significantly underestimate the lateral response of the barrette foundation.
Effect of load direction

The barrette has a rectangular form. Thus, its response to lateral loading depends on the direction of the applied load. Figure 8 shows the relationship between the lateral load and the horizontal displacement at the barrette head when it is loaded along the major axis (x direction) and the minor axis (y direction).

As expected, the horizontal displacement of the barrette head is greater along the minor axis than that along the major axis. This can be attributed mainly to the difference in the moment of inertia of the barrette along the major and minor axes.
CONCLUSIONS

The comparison between measured and predicted load-horizontal displacement curves reveals that the nonlinear soil models (MC and HS) match the barrette load test quite well compared to the linear elastic model. The nonlinear soil models showed quite similar results.

The linear elastic model revealed higher displacements than the other models and the difference in horizontal displacements between these models increases as the applied load increases.

The resistance of the barrette foundation to lateral loading depends on the direction of the applied load. Its resistance along the major axis is greater than that along the minor axis.

REFERENCES


