Experimental and numerical analysis of axially loaded slender piles implementing a hypoplastic model

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Abstract. Multiple studies have been carried out to estimate uplift pile capacity, within three main categories: i) the experimental method, which is very costly; ii) a simpler method using load transfer mechanisms; and iii) the numerical finite element method (FEM). The predictive capacity of numerical simulation depends mainly on the constitutive model used to model the soil-structure interface at which the load transfers from the pile to the surrounding soil. A reliable constitutive model capable of capturing the main features of soil behaviour based on easily obtained parameters is thus required. The hypoplasticity model is a promising constitutive model in this respect, offering a particular class of rate non-linear constitutive model in which the stress increment is expressed in a tensorial equation as a function of strain increment, actual stress, and void ratio. The hypoplastic model thus requires only eight material parameters (critical friction angle $\phi_c$, $e_{c0}$, $e_{s0}$, $e_{d0}$ (critical, maximum, and minimum void ratios, respectively), granular stiffness $k_s$, and the model constants $n$, $\alpha$, and $\beta$), with two additional parameters used to simulate the pile-soil interface ($k_e$ and $d_e$), which represent the surface roughness and the thickness of the shear zone, respectively. The performance of the model was verified in this work by comparing the numerical results with the experimental results for a single pile model under pull-out force with different surface roughnesses and confining pressures. The resulting good agreement between the numerical and the experimental results confirmed the ability of the hypoplastic model to simulate soil behaviour under complex conditions.

1. Introduction

The bearing capacity of tension piles is governed by the complex pile-soil interface behaviour, which can be considered to be created by two mechanisms: the friction between the soil and the pile and the bonding between them [1]. Accurate modelling of soil-structure interface is very important in terms of creating realistic and less conservative pile design, as this interface area is also a potential failure surface. The hypoplasticity model was thus used in this study to simulate the interface area and the surrounding soil.

The first version of the hypoplasticity model was developed by Kolymbas in 1985 at the University of Karlsruhe; it was created to describe the mechanical behaviour of granular materials by using a single non-linear tensorial function of the rate type [2].

For the pile-soil interface, a hypoplastic sand interface model is used, based on the enhanced model by Stutz et al. [3]. This model uses reduced stress and strain tensors and incorporates them in plane stress to the interface. This model assumes that a fully rough interface can be modelled as simple shear behaviour, where the soil deformation at the interface being the same as the deformation of the continuum soil surrounding the interface [4].
2. Hypoplasticity Model Theory

The general equation for the hypoplastic model is in the form [2]

\[ T_s^0 = F(T_s, e, D). \]  

(1)

where D is the stretching tensor and the symmetric part of velocity gradient grad v, the spin tensor \( \omega \) is the anti-symmetric part of the velocity gradient, and \( T_s^0 \) is the objective co-rotational (Jaumann) stress rate

\[ T_s^0 = \dot{T}_s - \omega T_s + \omega T_s. \]  

(2)

where \( \dot{T}_s \) is the derivation of the stress tensor with respect to time; and F is the tensor valued function. The function F can thus be decomposed into two parts [5]:

\[ T_s^0 = A(T_s, e, D) + B(e, T_s)\|D\| \]  

(3)

where \( A(T_s, e, D) \) is the linear part at D, representing soil hypoelastic behaviour, while \( B(e, T_s)\|D\| \) represents the non-linear part at D. The operators A and B are factorised by introducing the \( f_d \) and \( f_e \) parameters, which are both functions of void ratio (e) to permit easier determination of soil parameters. Thus,

\[ T_s^0 = f_b(tr T_s, e)f_e(tr T_s, e)[L(T_s, D) + f_d(tr T_s, e)N(T_s)\|D\|]. \]  

(4)

L and N are fourth and second order tensors respectively; L represents the linear part of the constitutive model with respect to D; N is the non-linear part at D; \( \|D\| \) represents the Euclidian norm \( \sqrt{tr D^2} \); \( tr \) represents the trace of \( T_s \); \( \bar{T}_s \) is the dimensionless stress ratio tensor \( \frac{T_s}{tr T_s} \); and \( f_b, f_e, \) and \( f_d \) represent the pycnotropy and baratropy scalar factors described by Kolymbas [5]. Pycnotropy refers to the effect of density and represented by \( (f_e, f_d) \) factors, which allows the simulation of strain softening, while baratropy is the effect of confining pressure on the friction angle and the critical void ratio and displayed by \( f_b \) factor.

\[ f_d = (\frac{e - e_d}{e_e - e_d})^\beta. \]  

(5)

This pycnotropy factor controls the peak friction angle, the dilative behaviour, and the transition to the critical state.

\[ f_e = (\frac{e}{e_c})^\alpha. \]  

(6)

This factor controls the relationship of the void ratio to soil stiffness, where a decrease in the void ratio results in an increase in the \( f_e \) factor, which leads to an increase in soil stiffness as the soil becomes denser for the same stress level.

The soil stiffness depends on two parameters \( (h_s, n) \), which are obtained from the oedometric test for a given soil sample in the loosest state possible according to the following equations [6]:

\[ n = \frac{\ln\left(\frac{c_1 c_2}{c_2 c_1}\right)}{\ln\left(\frac{p_2}{p_3}\right)} \]  

(7)

\[ h_s = 3ps\left(\frac{e}{c_c}\right)^{1/n} \]  

(8)

where \( c_c \) is the compression index obtained at two values of \( ps \) with their corresponding e values, as in Figure 1:

\[ c_c = \frac{\Delta e}{\Delta \log ps}. \]  

(9)

The \( \beta \) parameter controls the shear stiffness and influences the size of the response envelope [8]. According to [9], this parameter can be assumed to be 1 for natural soil. In the hypoplasticity model, the parameter \( \alpha \) controls the evolution of soil behaviour towards the critical state and can be obtained from the results of the consolidated drained triaxial test (CD) for a dense soil sample [8]:
Figure 1. Determination of the (hs and n) parameters from the oedometer test [7].

\[
\alpha = \frac{\ln \left( \frac{(2 + K_p)^2 + a^2 K_p (K_p - 1 - \tan \nu_p)}{a(2+K_p)(5K_p - 2)\sqrt{4 + 2(1 + \tan \nu_p)}} \right)}{\ln \left( \frac{e_e - e_d}{e_e - e_d} \right)}
\]  

where \( K_p \) is the passive earth pressure, which may be defined as

\[
K_p = \frac{T_1}{T_2} = \frac{(1 + \sin \phi_p)}{(1 - \sin \phi_p)}
\]  

(10)

\[
\sin \phi_p = \frac{(\tau_1 - \tau_2)}{(\tau_1 + \tau_2)}
\]  

(11)

The dilatancy angle at peak is

\[
\tan \nu_p = \left( - \frac{D_1 + 2D_2}{D_1} \right)
\]  

(12)

The baratropy parameter is obtained as follows:

\[
f_b = \frac{h_s}{n} \left( \frac{1 + e_{i0}}{e_{c0}} \right)^{\beta} \left( \frac{\tau_1}{h_s} \right)^{3 - n} \left( 3 + a^2 - \sqrt{3} a \frac{e_{i0} - e_{d0}}{e_{c0} - e_{d0}} \right)^{-1}
\]  

(13)

This factor is introduced to consider the increase in soil stiffness with the increase in mean stress.

The equation of the hypoplasticity model used in this study was developed by Von Wolffersdorff [9] to overcome the limitations in the original version:

\[
T_s^0 = f_c f_b \frac{1}{1 + \nu c} \left[ F^2 D + a^2 \text{tr}(T_s^0) \right] T_s^0 + f_d a F(T_s^0 + T_s') ||D||
\]  

(14)

This equation is thus a modification of the Gudehus and Bauer equation with implementations of the Matsouka/Nakai failure condition [10]:

\[
a = \sqrt{(3 - \sin \phi_c)}
\]  

(15)

\[
\frac{\sqrt{2} \sin \phi_c}{2} \frac{\sqrt{2}}{2} \sin \phi_c
\]  

(16)

where \( \phi_c \) is the critical friction angle. Based on the Matsuoka-Nakai failure condition, the stress coefficient \( F \) is

\[
F = \frac{1}{\sqrt{2}} \tan^2 \psi + \frac{2 - \tan^2 \psi}{2 + 2 \tan \psi \cos 3 \phi} - \frac{1}{2} \tan \psi.
\]  

(17)

and the factors (a) and (F) determine the critical state surface in the stress space.

\[
\tan \psi = \sqrt{3} \frac{\left| T_s^0 \right|}{\left| T_s^0 \right|}
\]  

(18)
\[
\cos 3\phi = -\sqrt{6} \frac{\text{tr}(T_s^3)}{|\text{tr}(T_s^3)|^2} \tag{19}
\]

\[
\hat{T}_s = (\hat{T}_s - \frac{1}{3} 1) \tag{20}
\]

When there is no occurrence of rotation in the principle planes, the \( T_s^0 \) then corresponds to the time rate of the stress tensor \( \dot{T}_s \).

### 3. Interface Hypoplasticity Model Theory

The same tensorial equations used in the hypoplastic model can be used with some changes for the interface hypoplasticity model. The stress tensor \( T \) generates the stresses \( \sigma_{11} = \sigma_n, \sigma_{12} = \tau_x \) and \( \sigma_{13} = \tau_z \), where \( \sigma_n \) is the normal interface stress (compression negative), which is the mean effective stress in the y-direction, while \( \tau_x, \tau_z \) are the shear stresses in the 2D contact plane. The assumed normal stresses are \( \sigma_{22} = \sigma_{33} = \sigma_p \), this being the in-plane stress, while the out of plane shear stress \( \sigma_{23} \) is assumed to be zero. The final stress tensor thus takes the form [4]

\[
T' = \begin{bmatrix}
\sigma_{11} & \sigma_{12} & \sigma_{13} \\
\sigma_{12} & \sigma_{22} & \sigma_{23} \\
\sigma_{13} & \sigma_{23} & \sigma_{33}
\end{bmatrix} \equiv \begin{bmatrix}
\sigma_n & \tau_x & \tau_z \\
\tau_x & \sigma_p & 0 \\
\tau_z & 0 & \sigma_p
\end{bmatrix} \tag{24}
\]

By using Voigt notation, the reduced stress tensor used in the interface hypoplastic model can take the form [3]

\[
T = \begin{bmatrix}
\sigma_n \\
\tau_p \\
\tau_x \\
\tau_z
\end{bmatrix} \tag{25}
\]

\( T' \) is the full stress tensor, while \( T \) is the reduced stress tensor. In the same manner, \( D' \) is the full stretching tensor and \( D \) is the reduced one.

\[
D = \begin{bmatrix}
\dot{\varepsilon}_n \\
0 \\
\frac{\dot{\gamma}_x}{2} \\
\frac{\dot{\gamma}_z}{2}
\end{bmatrix} \tag{26}
\]

It is assumed that under uni-axial shearing, \( \tau_x \) and \( \frac{\dot{\gamma}_z}{2} = 0 \). The modifications required for the interface hypoplastic model cause the following changes to the original hypoplasticity model to be implemented:

1 - The Lode angle \( \cos 3\theta = 0 \).

2 - The Matsuoka-Nakai failure condition \( F \) is

\[
F = \sqrt{1 - \frac{9}{4} \frac{(\hat{\tau}_x^2 + \hat{\tau}_z^2)}{\frac{\sqrt{3}}{2} (\hat{\tau}_x^2 + \hat{\tau}_z^2)}} \tag{27}
\]

where \( \hat{\tau}_x = \frac{\tau_x}{3\sigma} \) and \( \hat{\tau}_z = \frac{\tau_z}{3\sigma} \).

\[
3 - a = 3 \sqrt{\frac{2\tan^2\psi}{\sqrt{3}}} - 8 - \frac{\sqrt{3}}{2\sqrt{2}} \tag{28}
\]

The tensor of the relative stress is thus
\[
\hat{T}_s = \frac{\hat{T}_x}{tr(T)} = \begin{bmatrix}
\frac{1}{3} & \hat{\tau}_x & \hat{\tau}_z \\
\hat{\tau}_x & 1 & 0 \\
\hat{\tau}_z & 0 & \frac{1}{3}
\end{bmatrix}
\]

(29)

Modifications to the hypoplastic interface model to include the surface roughness can be achieved using several different approaches. The approach suggested by Gutjahr (2003) as stated in [10] is the one used in this research because of the obtained better results. This approach suggests that the surface roughness can be represented by

a- Introducing the parameter \( k_r \), defined as

\[
k_r = 0.25 \log R_n + 1.05 \leq 1
\]

(30)
or

\[
k_r = \tan \varphi_{int} / \tan \varphi_c \leq 1
\]

(31)

where \( R_n \) is the normalised roughness, which depends on the surface roughness \( R \) and the mean grain size \( D_{50} \), while \( \varphi_{int} \) is the interface friction angle.

b- Modification of the scalar parameter \( a \) to \( a_r \) where

\[
a_r = \frac{1}{k_r \varphi_c}
\]

(32)

c- Modification of the pycnotropy factor \( f_d \) to \( f_{dr} \) where

\[
f_{dr} = \left( \frac{e - e_d}{e_c - e_d} \right)^{ak_r^2}
\]

(33)

d- Modification of the baratropy factor \( f_s \) to \( f_{sr} \) where

\[
f_{sr} = \frac{f_s}{n} \left( \frac{e}{e_c} \right)^{\frac{\beta}{n}} \left( 1 + \frac{e_i}{e_c} \frac{tr(T)}{h_s} \right)^{1-n} \left[ 3 + a_r^2 - a_r \sqrt{3} \left( \frac{e_{i0} - e_{d0}}{e_{c0} - e_{d0}} \right)^{ak_r^2} \right]^{-1}
\]

(34)

4. Experimental work

The soil used in this study was a coarse sandy soil obtained from Al-Basrah in the south of Iraq. This sand has an average diameter \( D_{50} = 0.58 \) mm, as obtained from the grain size distribution curve and can be classified as (SP) according to the USCS as shown in Figure 2, according to [11]. The other soil properties are displayed in Table 1. The Direct Shear test is used to determine the soil shear strength parameters (\( \varphi, c \)).
The solution to the boundary value problem requires the determination of the material parameters and the initial values of the state variables. The hypoplastic model has eight material parameters \( (\Psi_c, e_{c0}, e_{i0}, e_{d0}, n, hs, \alpha, \beta) \), as seen in Table 2. Details of the calibration of these model parameters is presented in [12].

5. Single Pile Model Test

Experimental work was implemented to study the displacement-uplift capacity of a single pile model. The results were then compared to the numerical results to test the performance of the hypoplasticity model. The model consisted of a steel box of 70×70 cm in plane, and 60 cm depth and 5 mm in thickness. A horizontal try (1m long and 20cm wide) was attached to the steel box and supplied with a pulley and steel rope placed at the furthest end from the box to supply the pile pull-out force. An electrical motor and a gear box were fixed at the side of the steel box above the horizontal tray to control the speed at which the pile was pulled out from the soil, an AC-drive with a range of 2 to 50 Hz) was attached with the electrical motor. Preliminary tests were performed to determine the relationship between the pull-out speed of pile and the readings of the AC-drive, as shown in Figure 3.

| Specific Gravity (G.S) | \( \gamma_{dmax} \) | \( \gamma_{dmin} \) | \( \gamma_{dused} \) | R.D | Cu | Cc | \( \varphi \) | C |
|------------------------|---------------------|---------------------|---------------------|-----|----|----|-----------|----|
| 2.63                   | 17.9 [kn/m3]        | 16.3 [kn/m3]        | 17.5 [kn/m3]        | 76.7% | 3.24 | 1.3 | 43.50 [KPa] | 1   |

Table 2. Properties of the hypoplasticity model

| \( e_{d0} \) | \( e_{c0} \) | \( e_{i0} \) | \( \alpha \) | \( \beta \) | \( n \) | \( h_s \) | \( \varphi_c \) |
|-------------|-------------|-------------|-------------|-------------|-------|-------|-------------|
| 0.441       | 0.582       | 0.667       | 0.37        | 1           | 0.269 | 18.54 [Gpa] | 31° |

The model pile used was formed of aluminium alloy of hollow square section (1.4×1.4cm) and 2 mm thickness with an embedded length of 68 cm. These dimensions were chosen to ensure that the pile would behave as a long flexible pile, based on the equation [13]:

\[
\frac{L}{d} > 1.5 \sqrt{\frac{E_p}{G}}
\]  

(35)
where L is the pile length, d is the equivalent pile diameter, $E_p$ is the pile’s modulus of elasticity, and G is the soil shear modulus.

The pile was instrumented with two linear variable differential transformers (LVDT) of 25 mm in length and 0.01 mm in accuracy, which were used to measure the axial displacement at the top and the tip of the pile; these were affixed using magnetic holders, as in Figure 4. A load cell (1,000 kg) of S-beam (SS-300) type was fixed at the pile head at one end and used to measure the applied load as the pile was pulled out of the soil. A steel rope was attached to the other end of the load cell and fixed to the shaft, being operated by the electrical motor. All these devices were linked to a data acquisition system for data recording using the LabVIEW application.

Preparation of the soil bed was performed using the traveling pluviation technique (raining), in which the specified amount of sand for each sand layer is poured from a specially designed frame to achieve the required relative density (77%). After the metal box was filled with soil, three different confining pressures were placed at the top of the soil (9.7 (low), 16.2 (medium), and 21.6 kPa (high)). Three piles were used with three surface roughness values (rough R, medium M, and smooth S), generated by covering the pile surface with differing sandpaper types (P60, P220, and no sandpaper, respectively).

6. Numerical Work

The hypoplasticity model was used to simulate the load-displacement relationship using the ABAQUS FEA. In order to use the hypoplasticity model in ABAQUS, a user defined subroutine is required. So a hypoplastic UMAT, a 3-D soil model available in the soilmodel.info project by Masin, was used to simulate the soil surrounding the pile in the model test, while the hypoplastic interface constitutive model developed by Hans Stutz was used to simulate the pile-soil interface by using a FRIC subroutine which is also available at soilmodel.info. This model is based on the hypoplasticity model and simulates the interface soil with the same characteristics as the rest of the continuum soil [14].

The hollow aluminium pile was simulated as an elastic material with only two parameters (E=68800 MPa, and $\nu$=0.32), with boundary conditions as shown in Figure 5. All the points at the base of the soil were fixed in all degrees of freedom (x, y and z), while the vertical edges of the soil were fixed only in the x and z direction, and being free in the y direction.

![Figure 3. Relationship between the AC-drive reading and pull out speed rate.](image-url)
Three steps were required for the analysis. The first was the “geostatic step”, is a single increment step in which a body force of 17.5kN/m³ is applied to the soil sample. The second step was the “loading step”, which is also a single increment step, in which a predefined pressure (the same pressure used in the experimental work) was applied at the top surface of the soil in triangular form to simulate geostatic stress. In both steps, a static general command was invoked, since the soil was in a dry condition. The third step was the “shearing step”, with featured a time increment of 1,500 seconds with a 1mm/min strain rate. The shearing was presented as constant displacement control by applying 25 mm displacement in the tension direction at the pile head. An eight-node continuum three-dimensional brick element (C3D8R) with reduced integration, available in ABAQUS, was used to simulate both the soil sample and the pile.

7. Results and Discussion

Figure 6 shows the experimental and the numerical load-displacement relationships at the top of the rough, medium, and smooth pile surfaces under constant confining pressure (21.6kPa). Figure 7 shows the same relationship for a rough pile surface under three confining pressures (9.7, 16.2, and 21.6 kPa).

Examining Figure 6 shows an increase in the pull-out force with an increase in surface roughness. This increase is in agreement with the work of [15], [16] and [17], who stated that the most important factor that affects the pile uplift capacity is the interface friction angle, which increases with the increase in pile surface roughness. Pile loading induces radial effective stress (normal to the pile surface) due to dilation phenomena at the pile-soil interface, which increases with the increase in the confining pressure.

Another explanation was offered by [16], which stated that when the surface of the pile is rough enough, interlocking between the soil and the pile leads to an extra load needed to overcome the surface resistance, leading to less displacement (settlement). As the tensile force continues to increase, the interface soil dilates due to the development of shear strain, which increases the lateral stress (normal to the pile surface) and thus increases the shaft resistance. With the continuation of load increments,
continuous displacement (settlement) occurs, forcing the soil adjacent to the pile to reach a critical state and causing the pile to pull out rapidly.

Figure 7 shows that the pull-out resistance increases with the increase in the confining pressure, but this increase is not linear. Such increase in the pull-out force can be explained by the increase in the confining pressure, which reduces the soil dilation and increases the friction between the soil particles and the pile interface, leading to an increase in the overall pull-out resistance of the pile [18].

The ultimate pull-out force (Tult.) equals the load when the pile is plunged or the load when the displacement increases rapidly under the same load, as defined by [19] and this is the criterion that will be used in this research, as all tests have a unique plunge point. Table 3 shows the experimental and numerical ultimate uplift force at the top of the pile.

The surface roughness considered as the major factor affecting the pull-out force, with an increase percentage of about 50% when the pile used changes from smooth to rough, while the confining pressure increases the pull-out force by only 30% when the confining pressure above the soil surface changes from 9.7 to 21.6 kPa.

Good agreement was obtained between the experimental and the numerical results, showing the ability of the hypoplasticity model to simulate the load-displacement behaviour under different conditions, as shown in Table 3.
Figure 6. Load-displacement relation for a-rough, b-medium roughness, and c-smooth pile under a confining pressure 21.6 kPa.
**Figure 7.** Load-displacement for a rough pile under three confinement pressures a-21.6, b-16.2, and c-9.7 kPa, respectively.

**Table 3.** Experimental and numerical results for ultimate uplift force at the top of the pile.

| Ultimate uplift force (kN) | Results          |
|----------------------------|------------------|
|                            | (6.a) | (6.b) | (6.c) | (7.d) | (7.e) | (7.f) |
| Numerical                  | 0.49  | 0.37  | 0.23  | 0.49  | 0.41  | 0.34  |
| Experimental               | 0.47  | 0.36  | 0.24  | 0.47  | 0.42  | 0.33  |

**8. Conclusion**

This study investigated the ability of the hypoplasticity model to simulate the load-displacement behaviour of a single pile under tension loading with different surface roughness values and different confining pressures. The pile pull out force increased with the increase in the surface roughness values and with the increase in the confining pressure at the top of the soil surface. The roughness of the pile surface was the main factor affecting the tensile force, with a percentage of increase close to 50% when the pile was changed from smooth to rough, while as the confining pressure increased from 9.7 to 21.6 kPa, the pile pull out load increased by about 30%. Good agreement was obtained between the experimental and the numerical simulation in ABAQUS FEA using the hypoplasticity model, confirming the ability of the model to capture load-displacement behaviours under various conditions.
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