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Parametric study of the behaviour of a pile under axial load using Frank and Zhao's trilinear law and the theoretical formula of the conventional limit pressure *pl*

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ABSTRACT: This article deals with the determination of axial deformations and forces along a pile under axial loading, taking into account the interaction between the pile and the surrounding soil, and more specifically with the parametric study of the behavior of an axially loaded pile using Frank and Zhao's model and the analytical expression of the conventional limit pressure ***pl***. Frank and Zhao's (1982) trilinear method of load transfer curves, used in this study, offers an analytical approach for calculating axial strains and forces along the pile. This method, suggested by the French national standard for the application of Eurocode 7 (NF P 94-262) in the calculation of pile settlements, is based on the progressive mobilization of lateral friction and tip pressure, modeled by ***t-z*** (lateral friction) and ***q-z*** (tip pressure) curves, and enables soil-pile interactions to be simulated. The type of soil chosen for this study is clay (coherent soil), which provides better mobilization of lateral friction around the pile. It emerged from this study that the parameters with the greatest influence on pile behavior are the value of the applied axial load N_0, the ratio (***D/L***) and the pressure modulus E_M, in other words, the modulus of deformation ***E*** of the soil. In addition to these three essential parameters, we can add the influence of the Young's modulus E_p of the pile on its behavior. The choice of its value is important, as the compressibility of the pile material is a factor in the calculation of settlement. The results also show that the mechanical characteristics c_u, K_0, γ, α and **ν** have very little influence on settlement, axial force and deformation curves. All these results confirm the empirical relationships often used to calculate pile settlement, which are based directly on the value of the diameter D or on both the diameter D, the applied load N0, the Young's modulus of the pile Ep and its length L.

KEYWORDS: Axial load, Pile settlement, Soil-pile interaction, Load transfer curve method, Parametric study, Pile behavior, Frank and Zhao law.

1 INTRODUCTION

In the study of soil-pile interaction, it is important to be able to appreciate the influence of the various mechanical and geometric parameters of the pile and soil on the behavior of the foundation, specifically on the evolution of axial forces and deformations along the pile. For this purpose, a parametric study will be carried out based on the Frank and Zhao behavior model. This choice is based on the following aspects:

- This model has proven its effectiveness and accuracy in calculating settlements, and is recommended by some current standards, such as the French application of Eurocode 7.
- The model is based on the pressuremeter test, and a study of the mechanical characterization of soils by this test has been carried out by some authors ([1], [2], etc.) and allows us to make a detailed mechanical parametric study of the behavior of a pile under axial load in a coherent or rubbing soil.

Since Frank and Zhao's model is based on the results of pressuremeter tests, the expression of the parameters (p_l and E_M) as a function of the mechanical characteristics of the soil. The parameters studied here that affect the behavior of piles under axial loading in clay are: diameter ***D,*** pile length ***L,*** soil pressure modulus E_M, limit pressure p_lYoung's modulus of the pile E_p, undrained cohesion c_u, resting earth thrust coefficient K_0, soil specific gravity **γ,** rheological coefficient **α** and Poisson's ratio **ν**. In what follows, by means of an analytical calculation, we will study the variations in normal force, deformation and settlement along depth as a function of the various mechanical and geometric parameters of the materials involved. To this end, using the load transfer curve model of [3], we will establish the expressions for settlement $s(z)$normal force $N(z)$ and relative axial deformation $\varepsilon(z)$ as a function of depth z. This is followed by a parametric analysis of the influence of the mechanical and geometric characteristics of the pile (**E_p, L, D**) and the soil (c_u, K_0, γ, α, **ν** and **E_M**) on the evolution of settlement, normal force and deformation.

2 Frank And Zhao's Load Transfer Curve Model (1982)

The transfer curve method is used to calculate the vertical displacement of a pile subjected to axial loading. It is based on the progressive mobilization of the axial friction on the pile shaft τ or the stress under the pile base **q_p** with the relative soil-pile displacement ***s*** (Figure 1.a). According to this method, the soil/deep foundation interaction results in the assimilation of the soil around the foundation to a series of non-linear springs, behaving independently of each other.

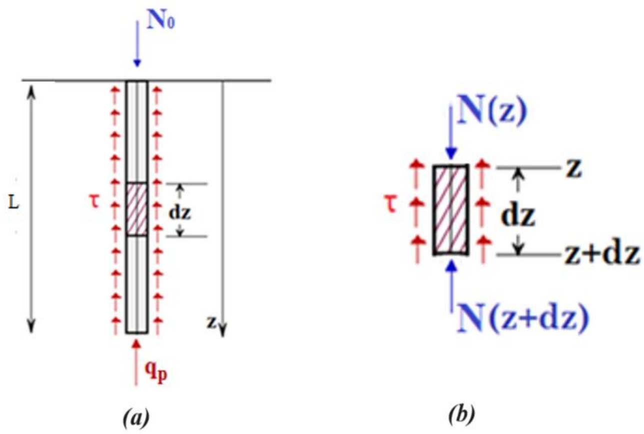

Fig. 1. (a) Pile under axial loading N_0 - (b) Pile section under normal stress N

The (***t-z***) curve method is based on solving the differential equation of a pile in compression, obtained by assuming the equilibrium of an infinitesimal section of a pile of length ***dz*** (Figure 1.b):

$$E_P A_P \frac{d^2 s}{dz^2} - P.\tau(s) = 0 \ [1]$$

Frank and Zhao's (1982) model for axial friction mobilization τ (s_z) and peak stress **q_p (s_p)** is trilinear (Figure 2) and is based on pressuremeter results. The initial slopes k_τ and k_q are given respectively for fine (clays and silts) and granular soils by the following relationships:

For fine soil:

$$k_\tau = \frac{2E_M}{D}; k_q = \frac{11E_M}{D} \ [2]$$

For granular soil:

$$k_\tau = \frac{0{,}8E_M}{D}; k_q = \frac{4{,}8E_M}{D} \ [3]$$

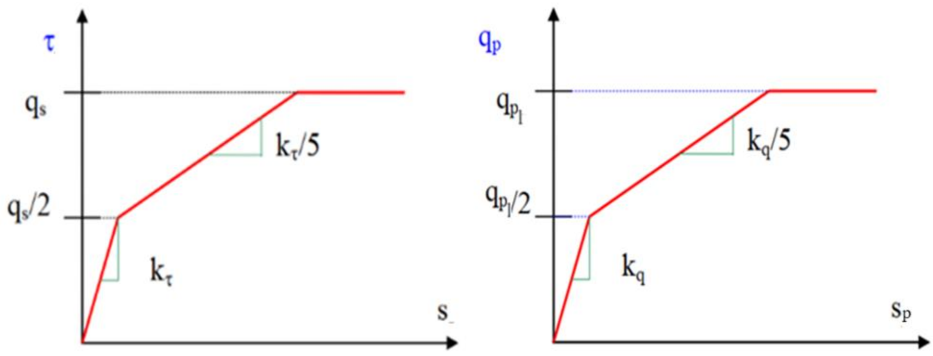

Fig. 2. Law of mobilization for lateral friction and unit peak force (Frank and Zhao 1982)

If the function τ *(s)* is linearized as $\tau = A_0 + B_0 s$, we can deduce the expression for unit friction τ on each phase. The differential equation governing pile deformation can then be solved according to this model. [2] give expressions for settlement $s(z)$ and normal force $N(z)$ as a function of depth *z*. By analyzing an infinitesimal section of pile of length *dz,* we derive the expressions for relative axial deformation $\varepsilon(z)$ and pile head settlement s_0.

$$\varepsilon(z) = \frac{N(z)}{E_P A_P} = -\frac{ds(z)}{dz} \quad [4]$$

- For $\tau \le \frac{q_s}{2}, A_0 = 0, B_0 = k_\tau$ and $\frac{q_s}{2} \le \tau \le q_s, A_0 = \frac{2q_s}{5}, B_0 = \frac{k_\tau}{5}$ with $\mu^2 = \frac{\pi D B_0}{E_P A_P}$

$$\begin{cases} s(z) = \cosh(\mu z)\, s_0 - \frac{\sinh(\mu z)}{\mu E_P A_P} N_0 + \frac{A_0}{B_0}(\cosh(\mu z) - 1) \\ N(z) = -E_P A_P \mu \sinh(\mu z)\, s_0 + \cosh(\mu z)\, N_0 - \frac{E_P A_P A_0 \mu}{B_0} \sinh(\mu z) \\ \varepsilon(z) = -\mu \sinh(\mu z)\, s_0 + \frac{\cosh(\mu z)}{E_P A_P} N_0 - \frac{A_0 \mu}{B_0} \sinh(\mu z) \end{cases} \quad [5]$$

From equation 4 and taking into account the expression for mobilization of the peak force ($q_p = k_q s_p$), we find the expression for pile-head settlement s_0. We can write:

$$\frac{ds(z=L)}{dz} = -\frac{N(z=L)}{E_P A_P} = -\frac{N_p}{E_P A_P} = -\frac{q_p}{E_P} = -\frac{k_q s(z=L)}{E_P} \quad [6]$$

$$\Rightarrow \frac{ds(L)}{dz} = -\frac{k_q}{E_P} s(L) \quad [7]$$

It is easy to see that the settlement at the head of the pile is given by:

$$s_0 = \frac{4N_0}{\pi D} \frac{1 + \frac{k_q \tanh(\mu L)}{\mu E_P}}{D(k_q + \mu E_P \tanh(\mu L))} + \frac{A_0}{B_0}\left(\frac{k_q}{k_q cosh(\mu z) - \mu E_P \sinh(\mu z)} - 1\right) \quad [8]$$

- For $\tau = q_s$, $A_0 = q_s, B_0 = 0$

  The expressions of $s(z), N(z)$ and $\varepsilon(z)$ become:

$$\begin{cases} s(z) = s_0 - \frac{z}{E_P A_P} N_0 + \frac{\pi D q_s}{E_P A_P} \frac{z^2}{2} \\ N(z) = N_0 - \pi D q_s z \\ \varepsilon(z) = \frac{N_0}{E_P A_P} - \frac{\pi D q_s}{E_P A_P} z \end{cases} \quad [9]$$

From equation 4, we find the expression for head settlement s_0.

$$s_0 = \frac{N_0 L}{E_P A_P} + \frac{4N_0}{\pi D^2 k_q} - 4q_s L\left[\frac{L}{2E_P D} + \frac{1}{D k_q}\right] \quad [10]$$

Using Frank and Zhao's model, we show that the evolution of settlement along the pile $s(z)$ depends on the value of the axial limit friction $q_s(z)$. This limiting axial friction can be expressed using Ménard's conventional limiting pressure expression $p_l(z)$.

3 PARAMETRIC STUDY

Since Frank and Zhao's model is based on the results of pressuremeter tests, the expression of the parameters (p_l and E_M**)** as a function of the mechanical characteristics of the soil.

3.1 INFLUENCE OF SOIL MECHANICAL CHARACTERISTICS ON LIMIT PRESSURE

The theoretical formula for p_l governed by equation 11 given by [1], is used to study the influence of soil mechanical parameters. It should be noted that in this theoretical formula, Ménard identifies the elastic modulus of the soil *E* with the ratio $\frac{E_M}{\alpha}$. For coherent soils, the theoretical expression for conventional limit pressure becomes:

$$p_l = \gamma z + c_u . \ln\left[\frac{\frac{E_M}{\alpha(1+\nu)}(\sqrt{2}-1)+c_u}{(1+K_0)\gamma z+c_u}\right] \quad [11]$$

In what follows, we present a complete parametric study with the aim of highlighting the influence of the different variables (c_u, K_0, γ, α, E_M, ν) on the limiting pressure p_l. The values of these parameters are selected from the literature on clayey soils. Each parameter is studied independently in relation to the whole set of parameters to verify its predicted and calculated influence on the limit pressure. The results of the parametric analysis are shown in Figures 3 to 8.

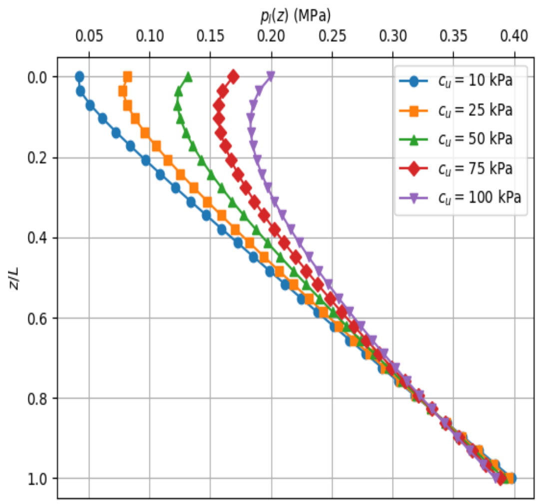

Fig. 3. Evolution of limit pressure at depth for different values of undrained cohesion

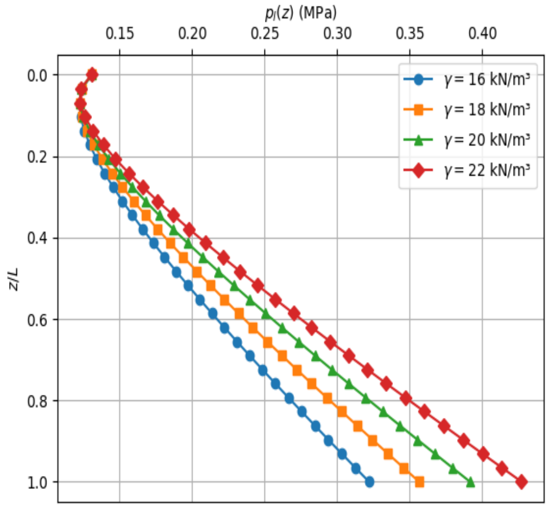

Fig. 4. Evolution of limiting pressure at depth for different density values

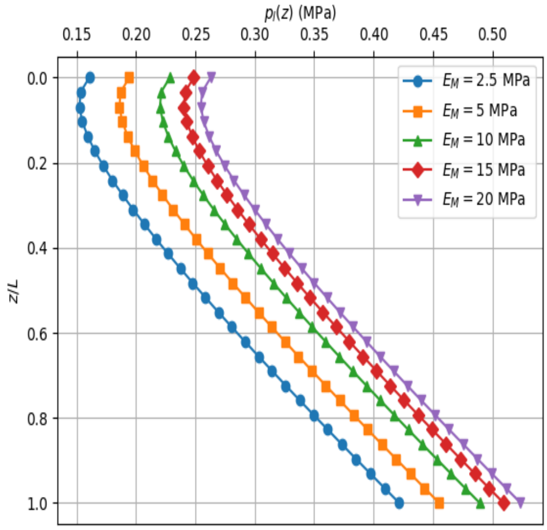

Fig. 5. Depth trends in limiting pressure for different values of pressuremeter modulus

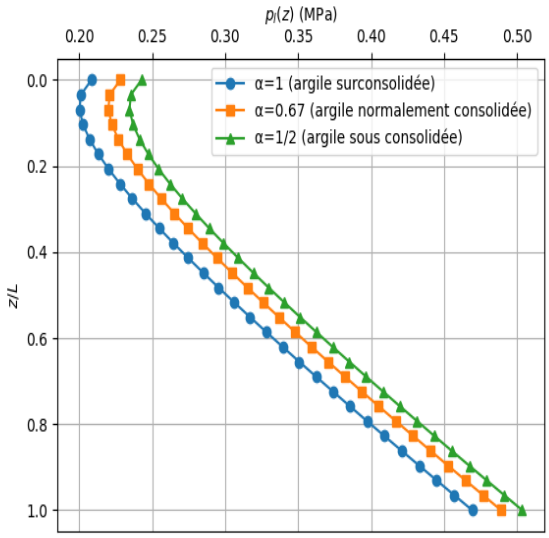

Fig. 6. Depth evolution of limiting pressure for different rheological coefficient values

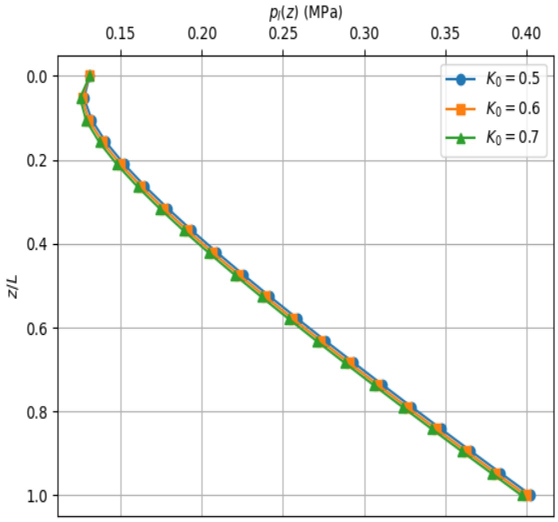

Fig. 7. ***Evolution of limiting pressure for different values of earth pressure coefficient at rest***

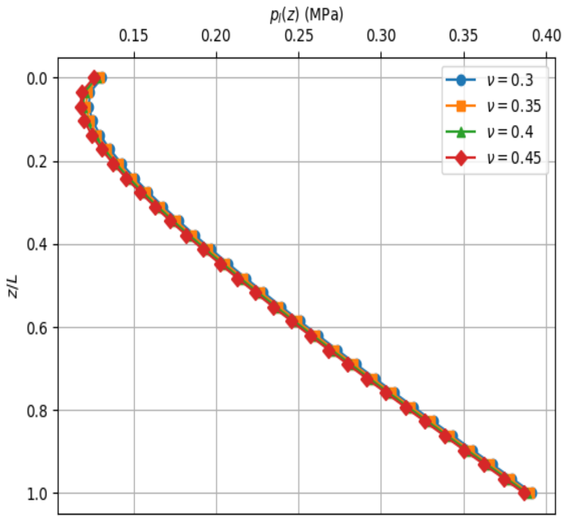

Fig. 8. ***Evolution of limiting pressure at depth for different values of Poisson's ratio***

Undrained cohesion acts as a deformation resistance parameter for clay soil, so as it increases, so does conventional limit pressure. Conventional limit pressure is a function of undrained cohesion as presented in the theoretical formula. We also note that as depth increases, the significant effect of undrained cohesion on limiting pressure decreases. Boundary pressure therefore becomes more influenced by vertical stress (γz). Thus, at greater depths, limiting pressure values for different values of c_u values approach each other (Figure 3). For coherent soil, conventional limit pressure is a function of vertical stress (γz), and as the weight per unit volume increases with depth, this leads to an increase in conventional limit pressure. This variation, described by equation 11, is clearly visible in Figure 4.

A variation is applied to the pressure modulus E_M to evolve the Young's modulus of deformation E. The conventional limit pressure represents the pressure associated with a probe volume that doubles the initial volume. If the soil is stiffer, deformation should be less for a given pressure, requiring a high value of pressure to reach twice the initial volume. In soft soil, on the other hand, deformation should be greater for a given pressure, requiring a low value of pressure to reach double the initial volume. This trend is anticipated by the analysis, as illustrated in Figure 5, where the pressuremeter modulus exerts an increasing influence on the conventional limit pressure. As the pressuremeter modulus increases, so does the conventional limiting pressure.

The rheological coefficient α is correlated with the Ménard pressuremeter test. The pressuremeter test is a short duration test, therefore identifying behavior tending to be undrained. The α coefficient is an adjustment variable that allows the effects of consolidation to be introduced into the use of the pressuremeter modulus and the soil deformation modulus to be evaluated. An analysis of Figure 6 shows that the limiting pressure is greater for underconsolidated soil ($\alpha = 1/2$) than for overconsolidated soil ($\alpha = 1$) or normally consolidated soil ($\alpha = 2/3$). This can be explained by the fact in the formula of p_l that α divides E_M and we have already seen the increasing influence of the pressuremeter modulus E_M on the limit pressure. p_l.

An increase in the resting earth thrust coefficient K_0 should increase the resting horizontal pressure and thus raise the conventional limit pressure. It can be seen that this coefficient exerts a moderate influence on the conventional limit pressure (Figure 7).

Figure 8 shows that the influence of Poisson's ratio ν coefficient on the conventional limit pressure. All in all, we can see that these various parameters act and must act in the formulation of the evolution of settlement, normal force and axial force deformation as a function of depth.

3.2 Influence Of Pile And Soil Characteristics On The Evolution Of Settlement, Normal Force And Deformation With Depth

Table 1 below summarizes the data used in the pile behavior study. These data are the mechanical and geometric characteristics of the pile and of a layer of clay soil used in the formulation of the normal force $N(z)$settlement $s(z)$ and deformation $\varepsilon(z)$ with depth z.

Table 1. Average values of the pile and soil characteristics considered

D (m)	L (m)	c_u (kPa)	γ (kN/m)³	α	K_0	ν	f_{ck} (MPa)	E_p (GPa)	E_M (MPa)
0,8	20	10	16	2/3	0,5	0,3	25	10,49	2,5

These data are representative of a soft clay layer of low consistency. The creep load calculated in accordance with standard NF P 94 - 262 (Deep foundations) is $R_{c;cr;d}$ = 0.6 MPa, it must be ensured in this study that the axial compressive load to be applied N_0 to be applied does not exceed $0{,}7R_{c;cr;d}$. Based on equations 5, 8, 9 and 10, the influence of the various parameters (D, L, c_u, γ, E_M, α, K_0, ν, E_p) on the normal force $N(z)$settlement $s(z)$ and deformation $\varepsilon(z)$ as a function of depth z. The results obtained are shown in Figures 9 to 17.

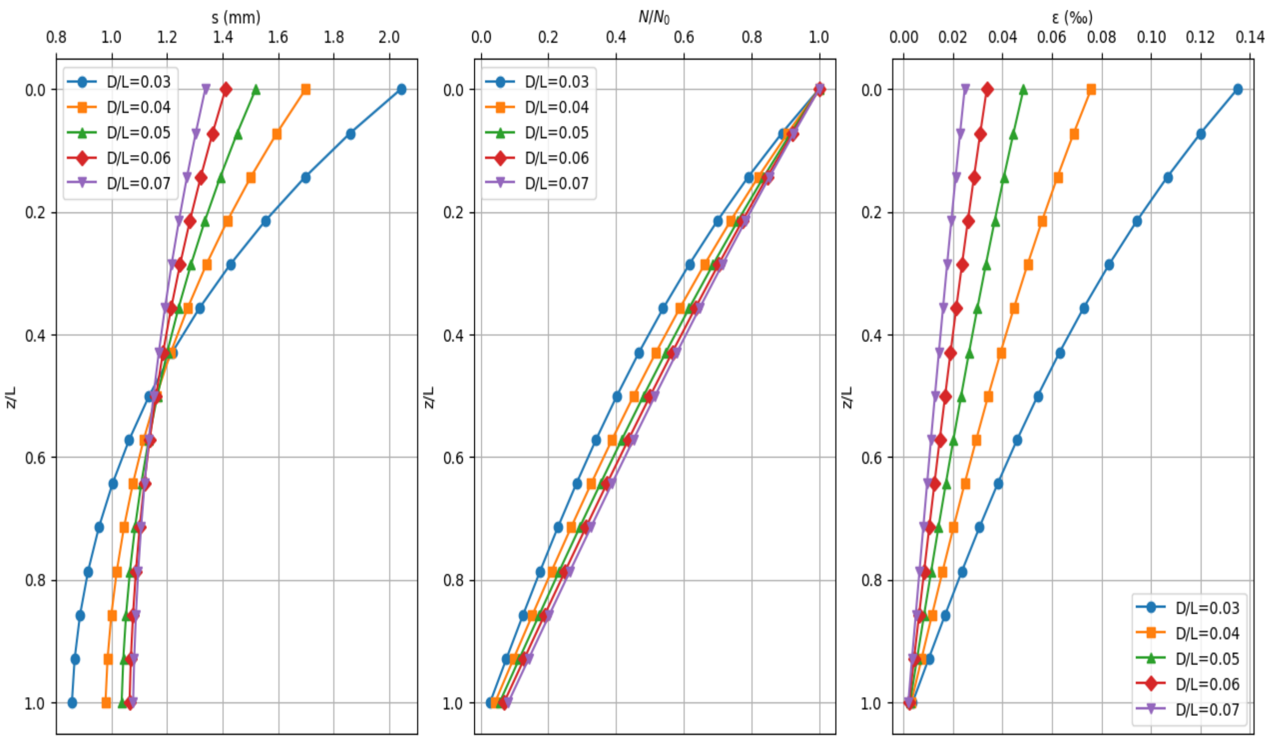

Fig. 9. Influence of D/L ratio on pile behavior under axial load

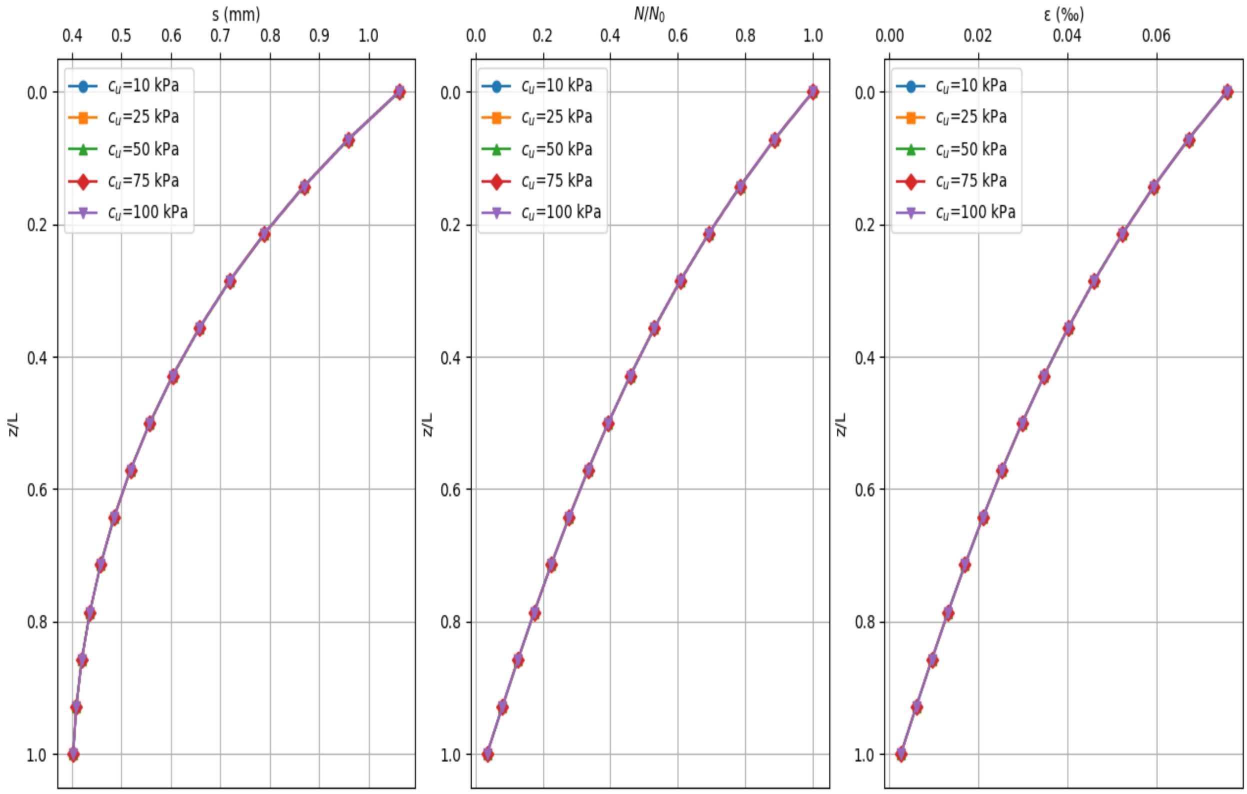

Fig. 10. Influence of undrained cohesion on pile behaviour under axial loading

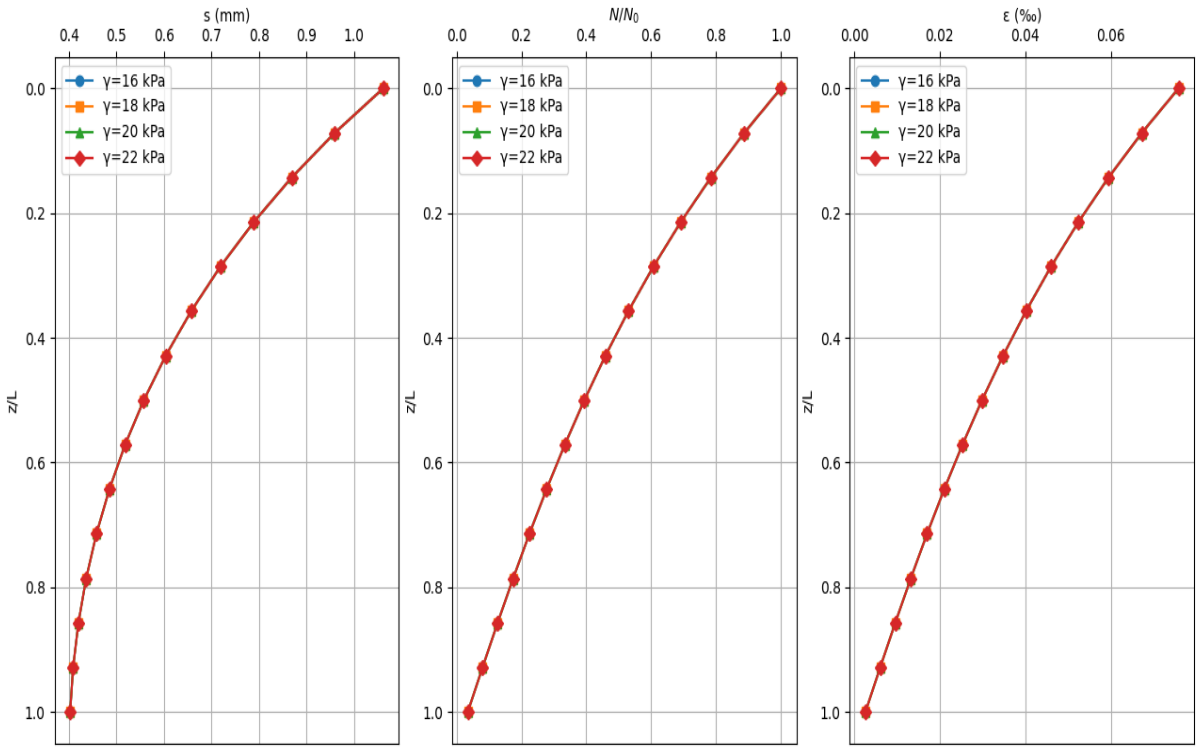

Fig. 11. Influence of volume weight on pile behavior under axial loading

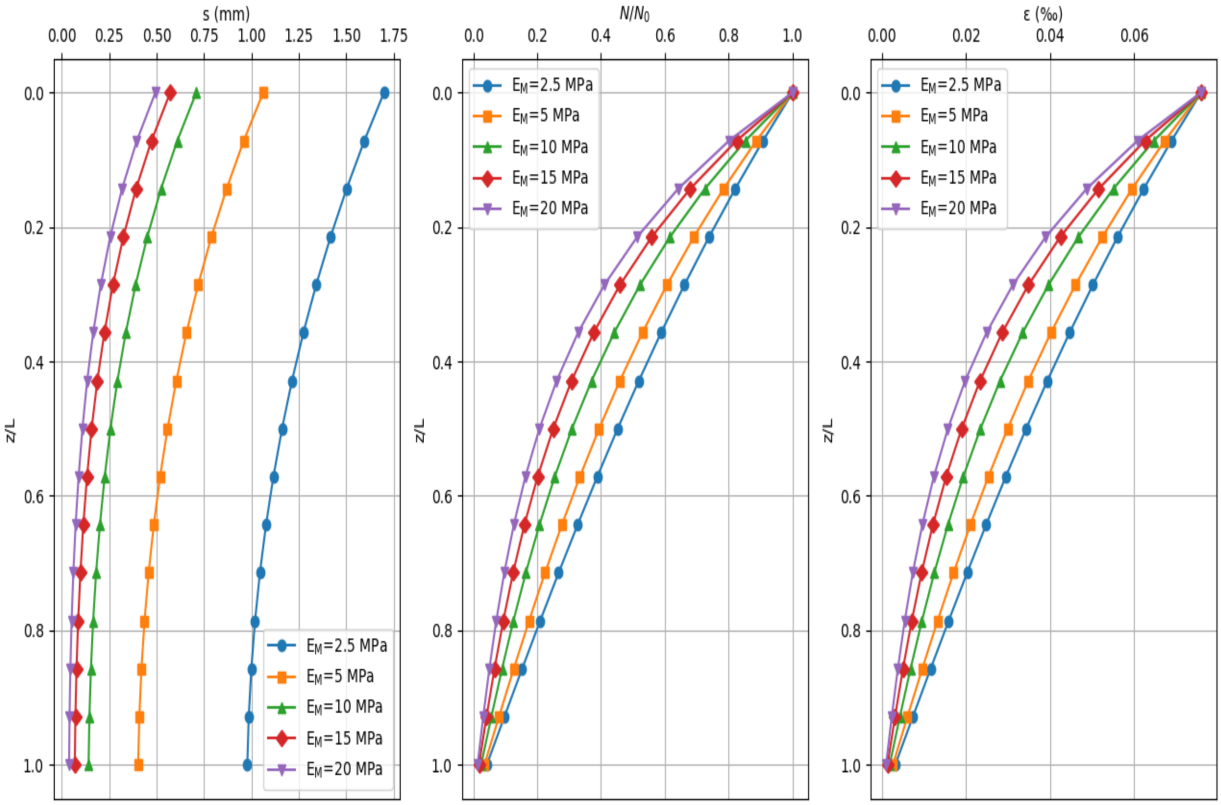

Fig. 12. Influence of pressure modulus on pile behavior under axial load

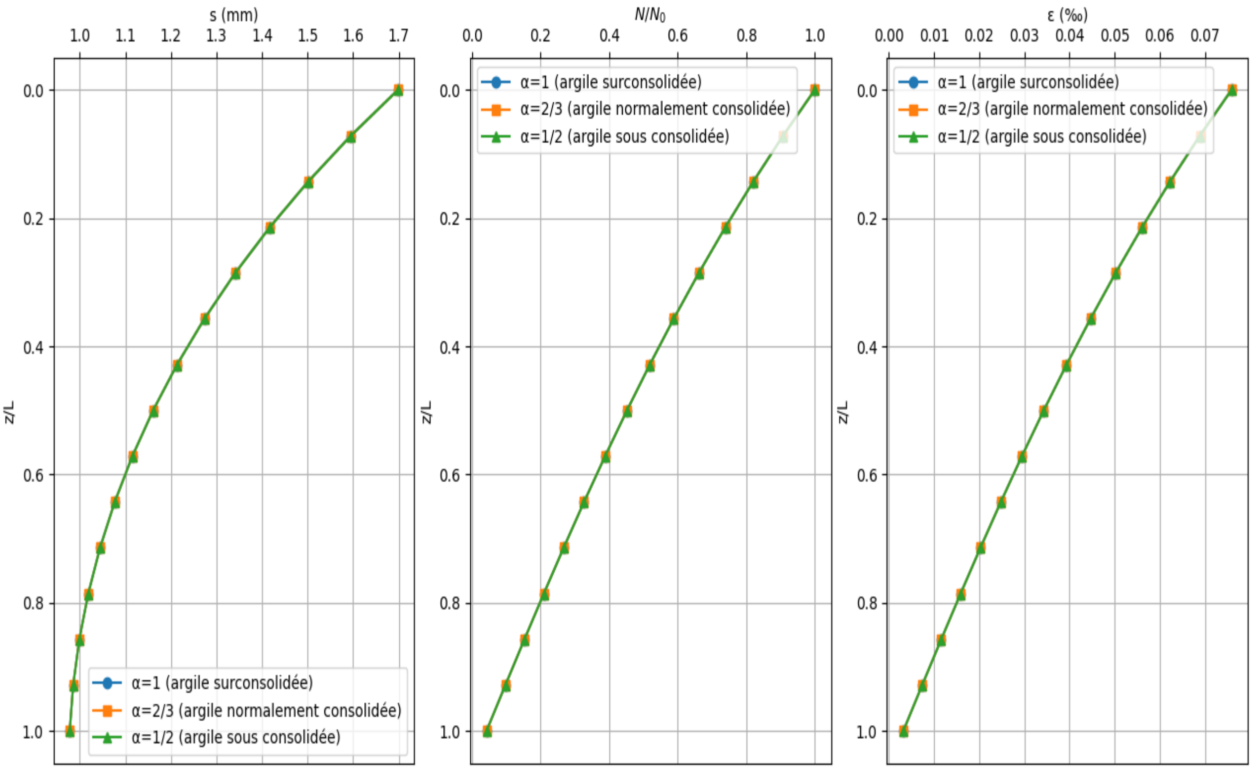

Fig. 13. Influence of rheological coefficient on pile behaviour under axial load

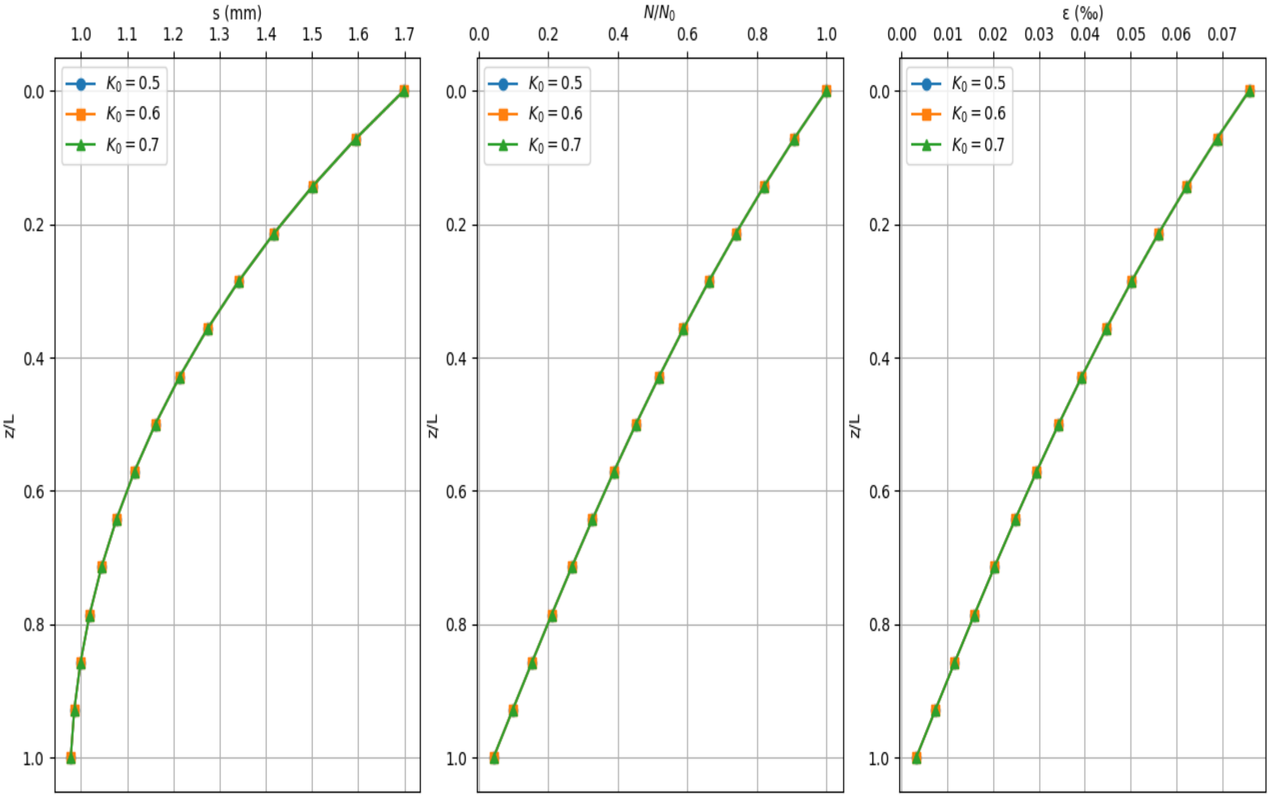

Fig. 14. Influence of the earth pressure coefficient at rest on pile behavior under axial loading

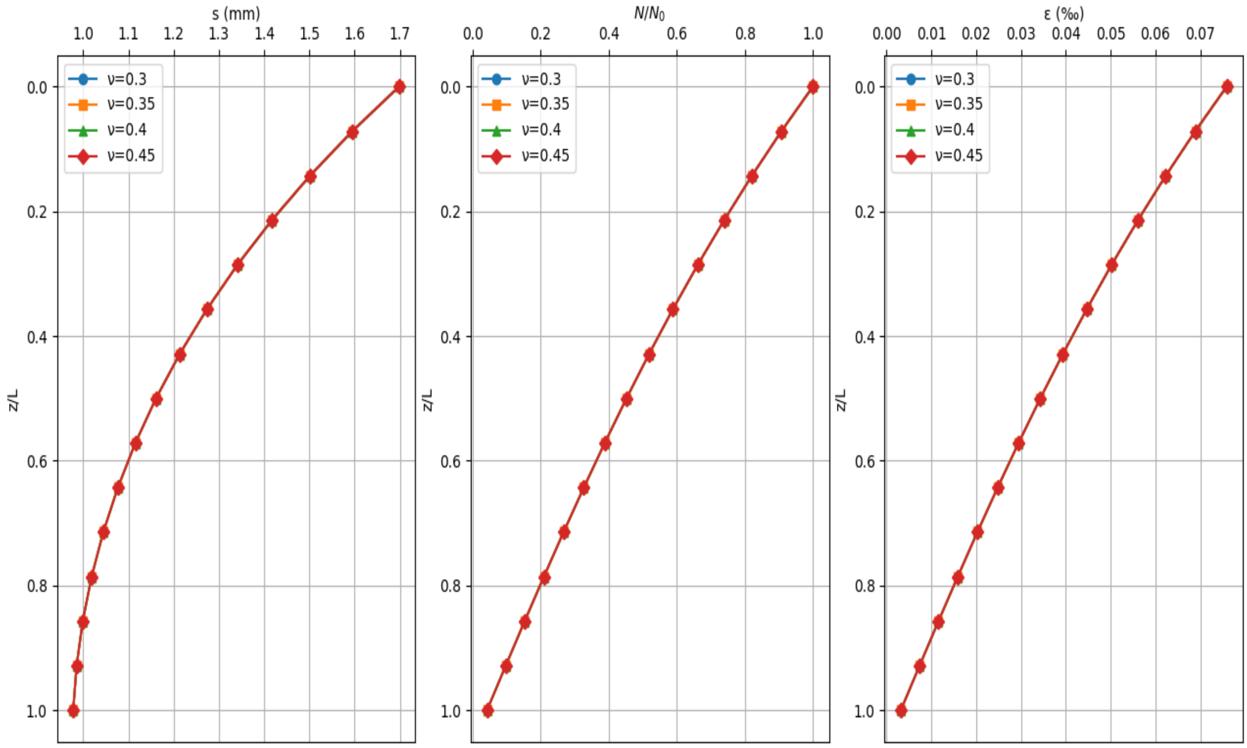

Fig. 15. Influence of Poisson's ratio on pile behavior under axial loading

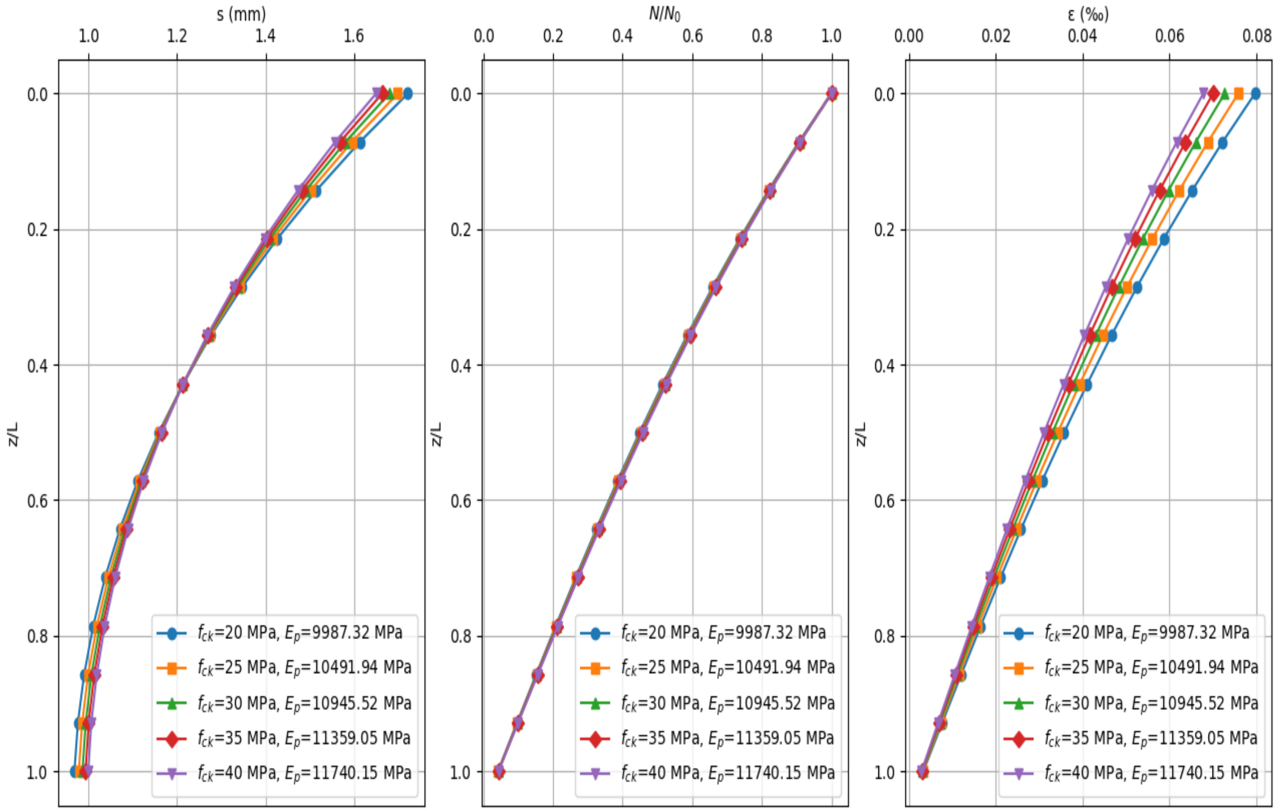

Fig. 16. Influence of Young's modulus on pile behavior under axial load

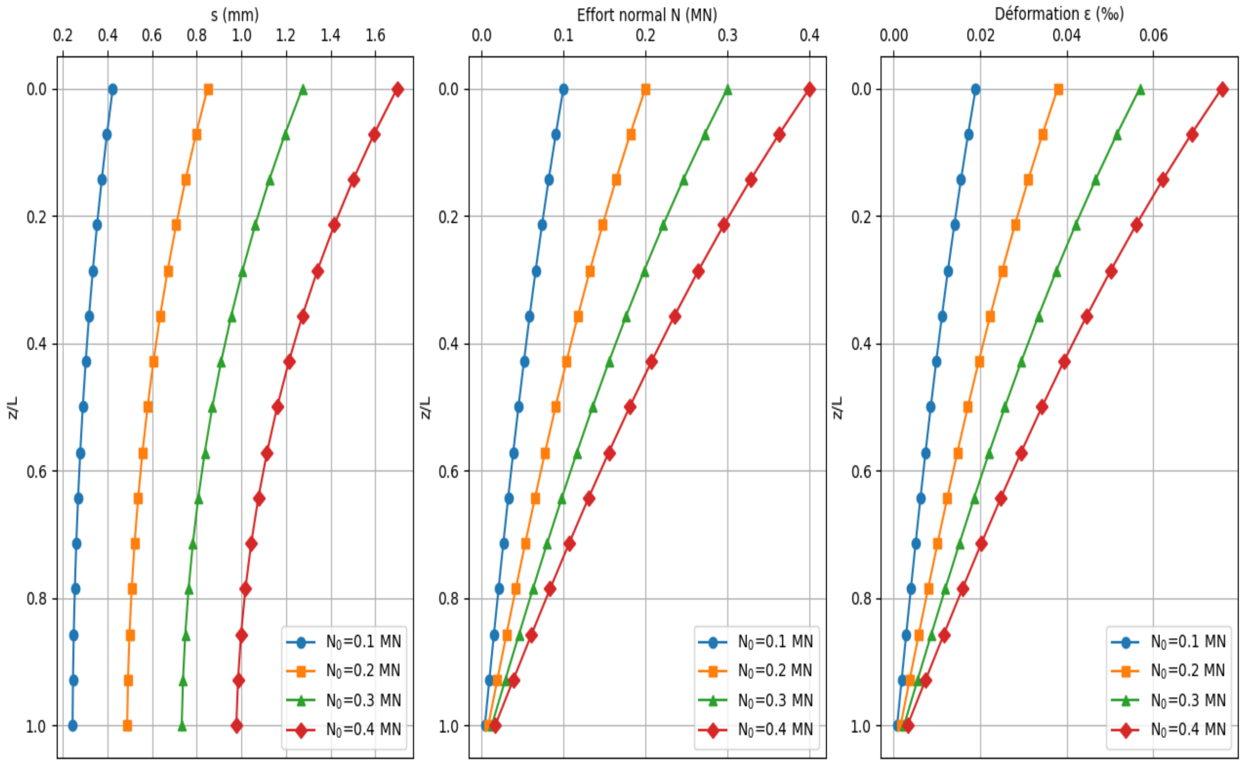

Fig. 17. Influence of pile head load on pile behavior under axial load

From the expressions obtained for pile head settlement (equations 8 and 10), we can see that pile diameter, as it increases, has a reducing influence on settlement. This can be seen in the first graph in Figure 9, where we obtain lower settlement values at the foot of the pile, which is quite logical, since it can be assumed that the deeper you go, the denser the soil mass becomes, so settlement decreases. The results also show that in the upper half-layer of the soil mass, settlement increases as the diameter decreases, while the opposite effect is observed in the inner half-layer. Figure 9 shows the existence of a neutral point insensitive to variations in the ***D/L*** ratio.

The results show that for a given pile, the smaller the diameter, the higher the settlement at the head of the pile, but the lower the settlement at the tip, compared with other diameter values. This can be explained on the one hand by the fact that, in this study, we take into account the deformability or compressibility of the pile; and on the other hand by the fact that, under compression, the shortening of the pile tends to reduce the value of the settlement as it advances in depth. Large-diameter piles, with lower deformability, therefore experience a much smaller variation in settlement with depth.

On the other hand, this finding can be explained by the fact that lateral friction around the pile, when mobilized, increases for a larger diameter, and the greater the friction, the smaller the reduction in settlement at depth. As can be seen from the graph, the decrease in settlement is much less noticeable at greater ***D/L*** ratios. Settlement between pile head and tip shows a decreasing hyperbolic trend, with a greater slope at the tangent of the curve at smaller diameters.

It can be seen that all these curves meet at a point where, for any value of D/L, settlement is the same at that depth; this position is z = L/2, the mid-span of the pile. To sum up, these results confirm that wider foundation elements tend to settle less than narrower ones, and maximum settlement is always at the head of the pile.

On the second graph, we can see a decrease in the normal force N_0 applied at the head as we descend in depth, with a hyperbolic shape. The larger the diameter, the more visible the decrease in normal force. This distinction between the curves shows that diameter has an influence on the evolution of normal force along the length of the pile. This can be explained by the fact that a larger diameter mobilizes more lateral friction, which progressively reduces the axial force transmitted.

A pile with a smaller diameter will experience a more noticeable reduction in normal force at depth. According to Hooke's law, a larger pile cross-section results in lower deformation for an applied force $\boldsymbol{N_0}$. This can be seen in the third graph for pile head deformation values. The evolution of deformation at depth is influenced by both the value of the force ***N (z)*** at depth ***z*** and the value of the diameter considered. It can be seen in this evolution that the effect of the diameter is predominant over that of the value of ***N (z);*** therefore, at any position z, the deformation ε is always lower for a larger diameter. The further you advance towards the tip, the closer the deformation values become. ε is always slightly greater for a smaller ***D/L*** slenderness.

The first graph in Figure 10 shows that, for a given cohesion value, settlement becomes less significant with depth, decreasing hyperbolically towards the minimum value obtained at the tip. This behavior indicates that settlement is proportional to depth, whatever the cohesion value. However, there is no significant change in slope between the different curves, suggesting that the influence of $\boldsymbol{c_u}$ on settlement is very small (almost nil).

The results also show that cohesion has little influence on normal stress. The normal compressive stress of the pile naturally decreases with depth under the effect of lateral friction along the pile. Even though in paragraph III.1, we noted that cohesion influences the value of the limit pressure ***pl*** and therefore the lateral friction around the pile. However, in this case, settlement and normal force values are not affected by lateral friction, hence the zero influence of undrained cohesion on their evolution. This can be understood from the equations, which show that lateral friction comes into play just when the second step of the trilinear law is reached.

For a pile with compressive stiffness $\boldsymbol{E_pA_p}$ constant, the normal force ***N*** and relative deformation ε do not differ according to Hooke's law.

The same analysis carried out on undrained cohesion can be attributed to density (Figure 11). These parameters, which play a part in the formulation of lateral friction around the pile, do not affect the pile's settlement and axial force. This is due to the low influence of these soil mechanical parameters on pile behavior, which becomes zero when the head load is low (when settlement $s(z)$ is lower $\frac{q_s D}{4E_M}$ according to Frank and Zhao).

Figure 12 shows the curves for settlement, normal stress and deformation as a function of depth for different values of pressure modulus.

Analysis of the graphs shown in Figure 12 reveals a linear decrease in settlement with depth, with maximum settlement at the pile head, decreasing hyperbolically until it reaches its peak value. Taking into account the different values of pressure modulus used, we can see that the curves are distinct, with the settlement values at the head, at the foot and at any relative depth ***z/L*** differing for each value of pressure modulus E_M. A lower value of pressure modulus of the soil gives a greater settlement. This first graph shows that the soil pressure modulus is a highly significant parameter in the calculation of pile settlement; the higher its value, the lower the pile settlement.

At a given depth ***z,*** the axial force and deformation values for a given pressure modulus E_M are significantly higher than the force and settlement values obtained for a higher pressure modulus. This shows that the pressuremeter modulus has an influence on the limit pressure and lateral friction around the pile, thus varying the normal force ***N*** and deformation ε. So, the higher the soil's pressure modulus, the greater the decrease in normal stress and deformation with depth ***z.***

The rheological coefficient α (Figure 13) of the soil was taken into account to reflect the state of consolidation of the clay studied, and to see how this parameter influences pile stresses, settlement and deformation. Research has shown that the use of this coefficient has an impact on the evaluation of soil deformation. [4] took this coefficient into account in their empirical formula for calculating the settlement of an insulated footing. The influence of the rheological coefficient is almost negligible in the case of deep foundations.

In the case of the influence of the resting soil buoyancy coefficient K_0, settlement always varies from a maximum value at the head of the pile to its minimum value at the tip in a hyperbolic pattern. As in the case of undrained cohesion c_u and soil density $\gamma$the value of the resting soil buoyancy coefficient K_0 may well influence the evolution curves of normal force, settlement and deformation, but only in the case where lateral friction q_s is engaged, taking into account the net limit pressure p_l^* (Figure 14) These three parameters are in fact the only ones used in Ménard's correlative limit pressure formula, the formula most often used in pile design based on the pressuremeter method. Related to both weight by volume and depth, the effect of this coefficient is more noticeable at greater depths. The resting earth thrust coefficient K_0 is a reducing factor for net limit soil pressure $p_l^*(z)$ and lateral friction q_s, due to the horizontal constraint p_0. A higher value of K_0 corresponds to lower lateral friction and therefore to a more or less significant reduction in normal force at depth. In the case of this study, the effect of K_0 on behavior is not significant, so we can understand that this is due to the fact that this state of pile loading develops fairly low settlements in the range $0 \leq s(z) \leq \frac{q_s D}{4E_M}$. The same analysis can be attributed to deformation, which here respects Hooke's law; the trends are similar to those of normal force for a Young's modulus E_p and pile cross-section A_p considered constant.

Looking at Figure 15, the results show that the influence of the Poisson's ratio v of the soil on settlement, deformation and axial force in the pile is negligible. Poisson's ratio is used in this study to express the soil shear modulus ***G*** from the soil Young's modulus ***E.*** Previous results show that the value given to this modulus does influence pile behavior, but the impact of Poisson's ratio is very small.

In Figure 16, the results show that the value of the Young's modulus of the pile does influence the evolution of settlement at depth. Analysis of the graphs shows that the effect of Young's modulus is similar to that of pile diameter, but the influence is less significant. At pile heads and shallow depths, a much larger pile Young's modulus results in slightly lower settlement. The decrease in settlement at depth is more obvious when the Young's modulus is small. So, for a pile with a low Young's modulus, settlement is highest at the head of the pile, but becomes lowest at the tip, compared with other values of E_p. These results can be explained by the fact that pile settlement is calculated, in this study, taking into account the compressibility of the concrete material. Longitudinal deformation under compressive force influences the value of settlement, as can be seen from the analytical expression of settlement *s (z)* according to the term $-\frac{z}{E_P A_P} N_0$ of the longitudinal deformation of the pile in compression.

In the second graph, the normal force curve shows that the Young's modulus of the pile has no influence on the axial force along the pile. These results are understandable, and generally demonstrate that the Young's modulus of the pile material has zero influence on the transmission of axial force. However, pile material can influence axial force in the sense that the roughness of the interface influences lateral friction, which in turn influences the expression of axial force. However, in this case study, the Young's modulus of the pile has no influence on the evolution of the normal force. The evolution of the deformation at depth is influenced by both the value of the force ***N (z)*** at depth ***z*** and the value of the Young's modulus of the pile. It can be seen from this evolution that, at any position ***z,*** deformation ε is always lower for a higher Young's modulus. The further you advance the point, the closer the deformation values become.

The results can be seen in Figure 17. The variation in the value of the axial force applied at the pile head shows the significant influence of this force value on pile behavior. The settlement curves are distinct, so a higher applied load creates higher settlement at any ***z*** position. The normal force applied at the head of the pile decreases with depth, and its peak values approach each other, showing that the variation in normal force at depth due to the mobilization of lateral friction τ along the shaft is highly significant. The force and deformation curves have similar curves, and the evolution of deformation is based on Hooke's law; the curves are similar to those for normal force, since the pile's extensional stiffness is assumed to be constant.

4 CONCLUSION

This study explores in detail the behavior of a pile subjected to axial loading through the study of axial deformations, normal forces and interactions between pile and surrounding soil; based on the method of load transfer curves, precisely Frank and Zhao's law; recognized effective for predicting the behavior of piles under axial loads. A study of the influence of soil and pile parameters involved in and affecting the behavior of the foundation was carried out. The main factors influencing behavior include the axial load applied at the head N_0, the *D/L* ratio, and the pressiometric modulus E_M i.e. the modulus of deformation *E* of the soil. The Young's modulus E_p of the pile also plays a fairly significant role in pile behavior.

Overall, this study shows that the parameters with the greatest influence on pile behavior are the value of the applied axial load $\boldsymbol{N_0}$, the ratio (***D/L***) and the pressure modulus $\boldsymbol{E_M}$, i.e. the modulus of deformation ***E*** of the soil. In addition to these three parameters, we can add the influence of the Young's modulus $\boldsymbol{E_p}$ of the pile on the behavior of the system. The results also show that the mechanical characteristics c_u, K_0, γ, α and **ν** have very little influence on settlement, axial force and deformation curves.

All these results confirm the empirical relationships often used to calculate pile settlement, such as those of [5] or [6], which are based directly on the value of the pile diameter ***D,*** or that of [7], which is a function of the diameter ***D,*** the applied load $\boldsymbol{N_0}$, the Young's modulus of the pile $\boldsymbol{E_p}$ and its length ***L***.

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