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SOIL PLUGGING OF OPEN-ENDED STEEL PILES DURING IMPACT DRIVING

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ABSTRACT

During the driving process of an open-ended steel pile, soil is allowed to enter the pile's core until partial or complete plugging occurs. Once this phenomenon is activated, soil resistance to driving (SRD) increases, which can prevent further soil introduction inside the pile, and in turn halt installation works under similar conditions. In this paper, we demonstrate the relationship between the plugging phenomenon, pile geometry, and driving acceleration by comparing literature with some conducted experiments (Paikowsky, 1990; Magroun, 2012), which show that the risk of plugging decreases mainly with an increase in both pile diameter and driving acceleration. A verification study of plugging occurrence in terms of depth is conducted on an open-ended driven pile at a Moroccan port terminal crossing predominantly sandy layers before anchoring in a lower marl layer. The pile's bearing capacity is evaluated in both plugged and unplugged cases using the SRD method following the processes of Toolan and Fox (1977) and Alm and Hamre (2001), and based on cone penetration test (CPT) results. The pile is thus found unlikely to plug during driving. As a general rule, we found that the plugging phenomenon must be controlled to avoid any interruption in the pile driving process.

Keywords: open-ended pile, pile driving, plugging, bearing capacity, soil resistance to driving SRD, CPT.

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INTRODUCTION

Open-ended piles are the most popular type of foundation for offshore construction. Their advantages are several: high bearing capacity, light weight, excellent maneuverability, cost and installation time advantages over bored piles. However, the open-ended pile driving operation causes soil to penetrate inside the pile as it is driven. At some point, a cylinder of soil forms inside said pile, mobilizing sufficient friction on the pile's inner wall to prevent any upward movement of the soil within, therefore leaving no space for further soil intrusion inside the pile. The pile becomes plugged.

SETTINGS AND METHODS

Plugging Effect as a Failure Mechanism

When installing an open pile, one of the following mechanisms can occur, as shown in Figure-1:

- Unplugged penetration: when the soil column inside the pile remains stationary (b.3).
- Plugged pile: when the soil column inside the pile moves downwards along with the pile (b.1).
- Partially plugged pile: When the soil column moves downwards but slower than the pile itself (b.2).
- The plugging degree is measured by the incremental filling ratio IFR, formulated by equation (1):

$$\mathrm{IFR} = \frac{\delta h}{\delta L} \tag{1}$$

where δh defines the variation in the length of the soil column inside the pile during a penetration increment and δL indicates the variation in the embedded length of the pile that occurs during a penetration increment.

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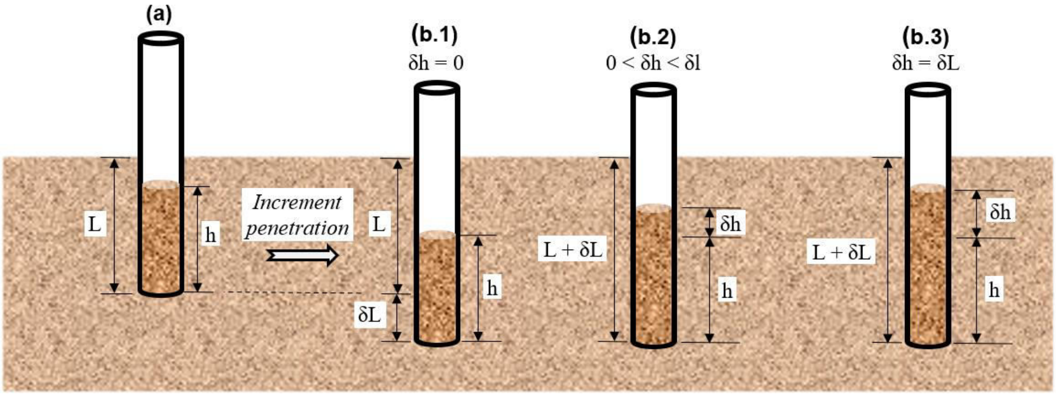

Figure-1. Failure mechanism of open piles. (a) Initial state: initial embed length "L" and soil penetration into the pile "h", (b) after penetration increment, (b.1) total plugging (IFR = 0), (b.2) partial plugging (0 < IFR < 1) and (b.3) no plugging (IFR = 1) (adapted from Randolph and Gouvernec (2011)).

Plugging Condition

According to the static vertical equilibrium to which the soil column penetrating the pile is subjected, plugging occurs when, as illustrated in Figure-2, the following condition is verified:

$$Q_{sf,i} > Q_{bf,p} - W_p \qquad (2)$$

where:

- $Q_{sf,i}$: the total friction of the soil column inside the pile.
- W_p: The weight of the soil column.
- $Q_{bf,p}$: Tip resistance of the soil column expressed as $Q_{bf,p} = A_p . q_{bf,p}$.
- $q_{bf,p}$: the unit tip resistance offered by the soil plug.
- A_p: cross-sectional area of the soil column inside the pile expressed as $0.25\pi(D_i^2)$.

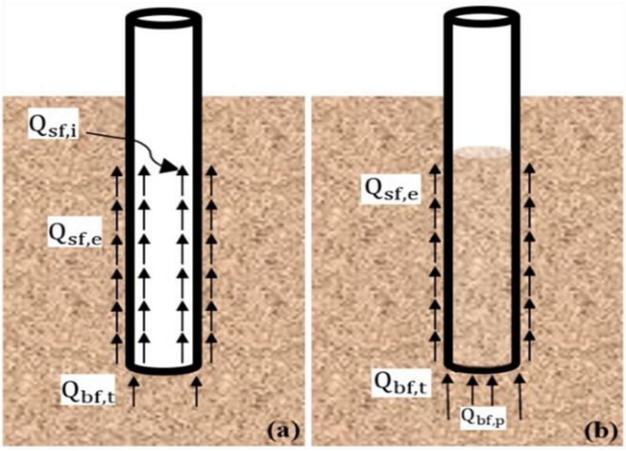

Figure-2. Forces acting on the piles in the unplugged (a) and plugged (b) conditions.

Soil Resistance to Driving

Soil resistance to driving (or SRD) is equal to the sum of lateral friction along the length of the pile on one hand, and tip resistance on the other. It must account for not only the soil's physical and mechanical properties but also any potential plugging at the tip. The works of Toolan & Fox (1977), Stevens & al. (1982) and Alme & Hamre (2001) offer calculation methods that distinguish between the case of plugged and unplugged piles as shown in equations (3) for a plugged pile and (4) for an unplugged pile:

$$SRD = \sum(q_{sf,e} * A_e) + (q_{bf,t} * A_t) + (q_{bf,p} * A_p) \qquad (3)$$

$$SRD = \sum(q_{sf,e} * A_e) + (q_{sf,i} * A_i)) + (q_{bf,t} * A_t) \qquad (4)$$

where:

- $q_{bf,t}$: unit tip resistance provided by the pile ring.
- $q_{bf,p}$: unit tip resistance provided by the soil plug.
- $q_{sf,e}, q_{sf,i}$: Unit friction resistance outside and inside the pile (see Figure 2).
- A_p : area of the plug tip: $0.25\pi(D_i^2)$

- A_t: area of the pile tip expressed as: $0.25\pi(D_e{}^2 - D_i^2)$
- A_e, A_i: Outer and inner lateral surfaces of the pile.
- D_i and D_e: internal and external diameter of the pile.
- Both Toolan & Fox and Alm & Hamre models estimate lateral friction and tip resistance as shown in Table-1.

Table-1. Tip and frictional resistance formulas according to Toolan & Fox (1977) and Alm & Hamre (2001).

Parameter	Toolan & Fox model	Alm & Hamre model
Friction resistance	$\frac{q_c}{300}$	$f_{s,res} + (f_{si} - f_{s,res})e^{k(d-p)}$, where $f_{si} = K\sigma'_{v0}\tan\delta_{cv}$ and $f_{s,res} = 0.2f_{si}$
Tip resistance	qc	$0.15q_c\left(\frac{q_c}{\sigma'_{v_0}}\right)^{0.2}$
Setting parameters	q_c: cone penetration resistance, obtained from a CPT test.	$K\sigma'_{v0} = 0.0132q_c\left(\frac{\sigma'_{v0}}{100}\right)^{0.13}$ (Jardine & Chow (1996)), and $k = \frac{1}{80}\sqrt{\frac{q_c}{\sigma'_{v_0}}}$ where: • f_{si} and $f_{s,res}$: initial and residual friction resistance. • K: coefficient of lateral earth pressure. • k: frictional degradation form coefficient. • d: depth of soil layer. • p: pile penetration. • δ_{cv}: Interface friction angle at constant volume or critical state.

PARAMETRIC ANALYSIS

Impact of Pile Diameter

Analytical study: Let's consider an elementary volume of soil with a specific weight γ', delimited by the pile's inner lateral surface and located between coordinates z and z + dz. With the following simplifying assumptions:

- The vertical stress σ is constant over a straight section perpendicular to the z axis.
- The friction force is proportional to the vertical stress, i.e., $\tau = k.\tan\varphi.\sigma$ where k is the lateral pressure coefficient, and φ is the soil-pile friction angle.

We establish the following equilibrium formula:

$$(\sigma + d\sigma).A = \sigma.A + \tau.P.dz + \gamma'.A.dz$$

where A and P are respectively the area section and perimeter of the cylindrical soil column (which are constant).
We therefore have:

$$\frac{d\sigma}{dz} = \frac{\tau.P}{A} + \gamma'$$

and thus:

$$\frac{D\sigma}{dz} - \frac{4.k.tan\varphi.\sigma}{D} - \gamma' = 0$$

where $\tau = k.\tan\varphi.\sigma$ and $A = 0{,}25\pi D^2$, D being the internal diameter of the pile.
If we set a factor β as follows:

$$\beta = \frac{4.k.\tan\varphi}{D}$$

we get:

$$\frac{d\sigma}{dz} - \beta.\sigma - \gamma' = 0$$

The solution to this differential equation that satisfies the boundary condition σ(z = 0) = 0 is:

$$\sigma = \frac{\gamma'}{\beta}\left(e^{\beta.z} - 1\right) \quad (5)$$

The exponential term in z shows that significant stresses develop along the soil column inside the pile and become increasingly localised near the tip of the pile, where the "plug" is formed. The longer the pile, the greater the increase in stress, depending in fact on the L/D ratio. Figure-3 shows the variation of σ and τ as a function of depth for: $\gamma' = 10\ KN/m^3$, $D = 1{,}2m$, $k.\tan\delta = 0.25$.

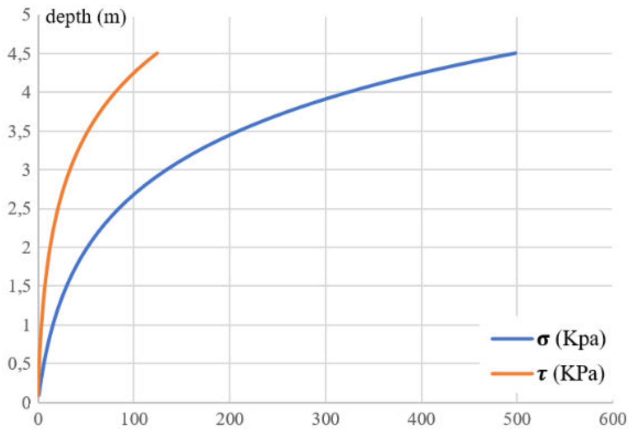

Figure-3. Evolution of stresses inside the pile in terms of depth.

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For example, if we set a length L of 10 meters, Figure-4 illustrates this effect for different diameters values (D_1 = 1m; D_2 = 1.2m; D_3 = 1.4m; D_4 = 1.6 m; D_5 = 1.8m; D_6 = 2m).

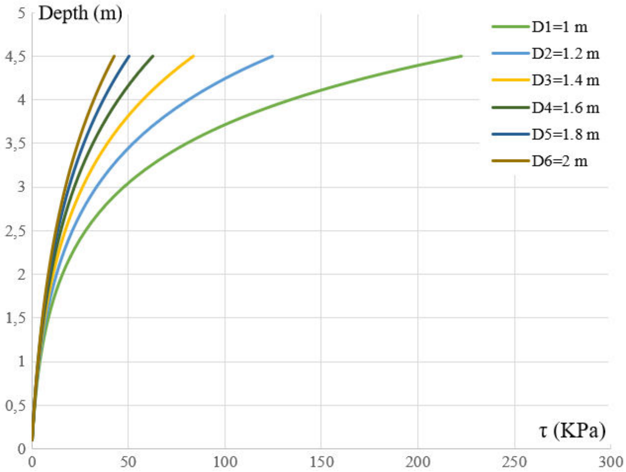

Figure-4. Evolution of tangential stresses inside the pile as a function of depth for different diameters.

Experimental approach: Driving tests carried out by Paikowsky (1990) on scale model piles with plug formation showed that plugging can occur from the start of driving for very small diameters (B = 10mm). However, for piles with a diameter of around 150mm, plugging does not occur (as shown in Figure-5). These results confirm those of the analytical study.

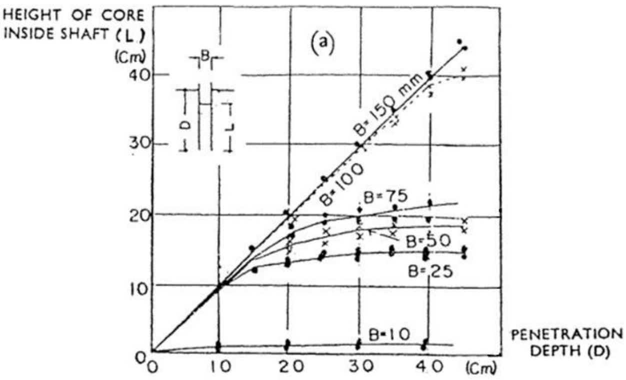

Figure-5. Length of soil column inside the pile's shaft L versus penetration for several diameter values (Paikowsky, 1990).

Effect of Installation Method (Dynamic or Static)

Analytical study: The influence of the soil column's driving inertia inside the pile on the plugging phenomena was studied by asserting the following hypotheses:

- The inertial force in the soil column is constant.
- The damping forces are negligible compared with the other forces.
- The unit friction fs along the soil-pile interface is assumed to be constant.
- A distinction is made between the static and dynamic cases of pile driving.
- In case of a static pile driving: Considering a soil column inside the pile of which density is γ and dimensions are shown in Figure-6:

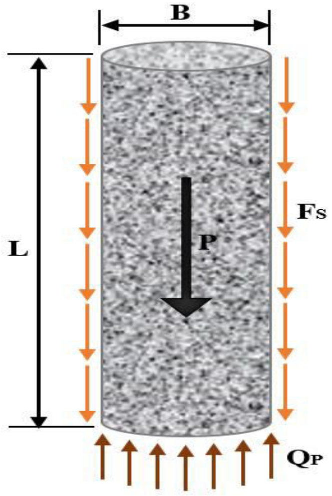

Figure-6. Soil column - static case.

Applied forces to the column are:

- Weight of soil column: $P = 0.25\gamma.L.\pi.D^2$
- Lateral friction: $F_s = L.\pi.D.f_s$, where fs is the unit friction along the soil-pile interface.
- Tip resistance: $Q_p = 0.25q.\pi.D^2$, where q is the unit tip bearing capacity.

When equilibrium is established, we have:

$$P + F_s = Q_p$$

and finally:

$$L = \frac{q}{\gamma + \frac{4}{D} * f_s}$$

We can already deduce that a wider pile (large diameter) favours greater lengths of soil inside the pile's shaft and therefore lesser risk of plug forming, which confirms previously obtained results.

- In case of a dynamic pile-driving: When a pile is driven, a resultant inertia force F_i opposes the movement and is added to the previous forces. It is given by:

$$F_i = L.\frac{\pi.D^2}{4}.\frac{\gamma}{g}.a$$

where a is the acceleration of the pile during driving.
When equilibrium is established, we have:

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$$P + F_s = Q_p + F_i$$

We finally get:

$$L = \frac{q.D}{\left[\gamma.D\left(1-\frac{a}{g}\right)+4.f_S\right]}$$

This formula shows that for a given soil resistance "q" and a fixed pile diameter D, the height of the soil column within the pile's shaft increases with the driving acceleration "a" and thus the risk of a plug forming decreases. Figure-7 illustrates this increase for the following values: D = 1.5m, γ = 20kN/m^3, q = 15MPa and f_s = 120kPa.

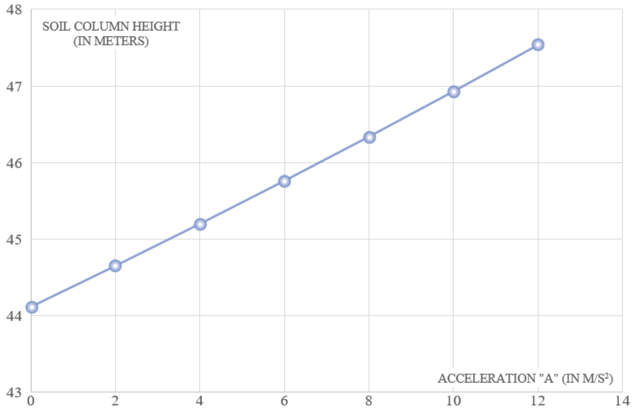

Figure-7. Influence of inertia forces on the height of soil column.

Figure-8 summarises the improvement in soil column length with diameter, acceleration, and penetration mode for: γ=20kN/m3 q=15MPa et fs=120kPa:

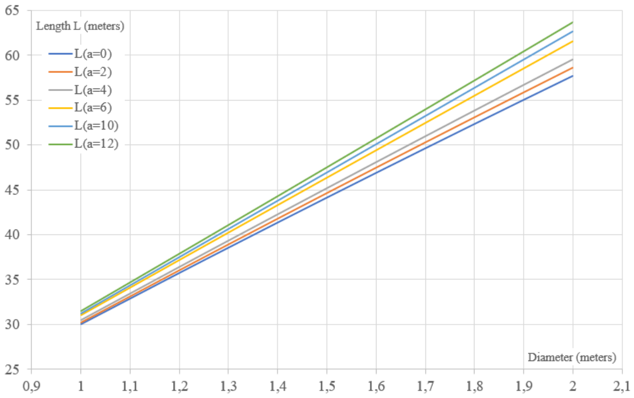

Figure-8. Influence of diameter, inertia forces and penetration mode on the height of the soil column in the pile's shaft.

Experimental approach: Static and dynamic pile driving tests conducted by Paikowsky (1990) on scale model piles showed that static piles have a greater tendency to plug. On the other hand, dynamic driven piles show abrupt changes in SRD, fluctuations in filling rate, and a succession of dense and less dense zones in the soil column within.

In addition, an experimental program was carried out by Margoun (2012) with the participation of several companies operating in offshore works, with the aim of examining the effect of driving energy (directly linked to hammer drop height) on plug forming. Tests were carried out on model piles having a 7-centimener diameter, a 2-millimeter thickness, and a 2.55-meter length. These piles were driven using an 8.3-kilogram hammer with two different drop heights of h = 0.45m and h = 1.4m. Figure-9 shows the evolution of the pile filling ratio versus penetration.

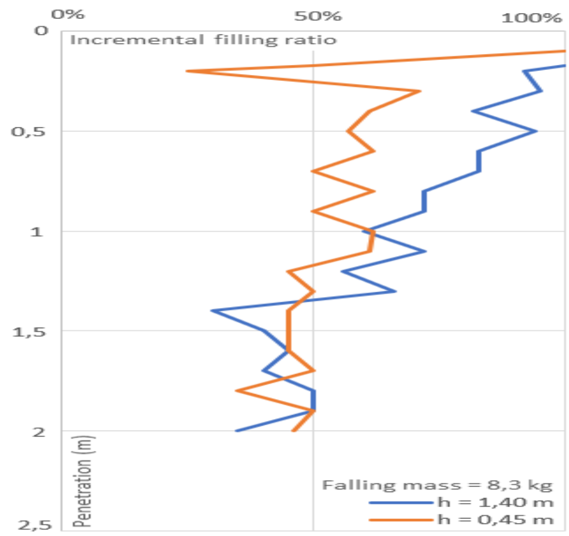

Figure-9. Evolution of the pile filling ratio with the drop height of the hammer.

From the obtained results, we can assume that higher hammer drops (and thus, the energy of the blow) reduce plugging risk.

CHECKING PLUG FORMATION: A CASE STUDY

The open pile plugging has an influence on its resistance at the tip. It behaves like a closed pile and the energy required to drive it will be greater than in the unplugged case, hence the need to check it. In this section, we examine the case of an offshore steel pile on the quay of a container terminal in a Moroccan port, of which the characteristics are shown in Table-2.

Table-2. Some characteristics of the studied pile.

D_e (m)	D_i (m)	L(m)	Natural ground surface (mhz)	End-of-driving depth(m)

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1.422	1.397	40.3	3.2	32

Soil profile and results of the CPT tests at several depth values are shown in Table-3.

Table-3. Soil characteristics around the studied pile area.

From (m)	to (m)	Soil type	q_c (MPa)
0	-16,7	Embankment	4
-16,7	-20,45	Loose sand	2
-20,45	-23,55	Moderately dense sand	8
-23,33	-25,7	Dense and	18
-25,7	-28,2	Very dense gravel	65
-28,2	-35,2	Dense sand	18
-35,2	-36,2	Green Marl	12
-36,2	-63,2	Gray Marl	19,5

The distribution of cone penetration resistance q_c in terms of depth is shown in Figure-9.

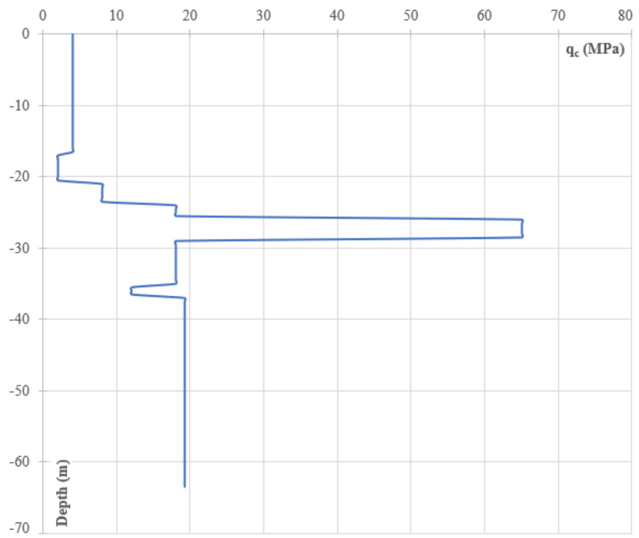

Figure-10. Distribution of cone penetration resistance q_c with depth.

Plugging risk assessment is based on the comparison between tip force and internal lateral friction force, which are both calculated using the Alm and Hamre (2001) method. The results of this calculus are plotted in Figure-11.

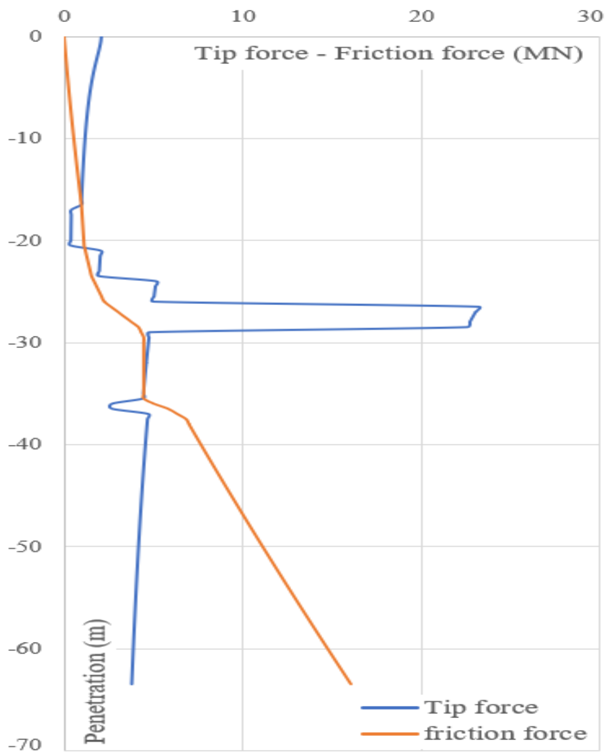

Figure-11. Tip force and frictional force with the pile penetration using the Alm and Hamre (2001) method.

It can be seen that from -17mzh down to -20.5mzh depths, the tip force is slightly less than the internal lateral friction (maximum difference of 0.73MN at -20.5mzh). Therefore, local plugging risk on this soil layer exists. From -20.5mzh to -35mzh depths though, plugging risk disappears as tip force becomes greater than the internal friction. However, from this point downwards, internal friction increasingly exceeds tip force, indicating substantial plugging. Our study was limited to -32mzh in depth. Thus, our study area is not affected by the plugging phenomenon, especially since the accelerated pile driving would prevent this from forming.

Soil resistance to driving was calculated for both the "plugged" and "unplugged" cases using the Alm & Hamre and Toolan & Fox models. Results are shown in Figure-12.

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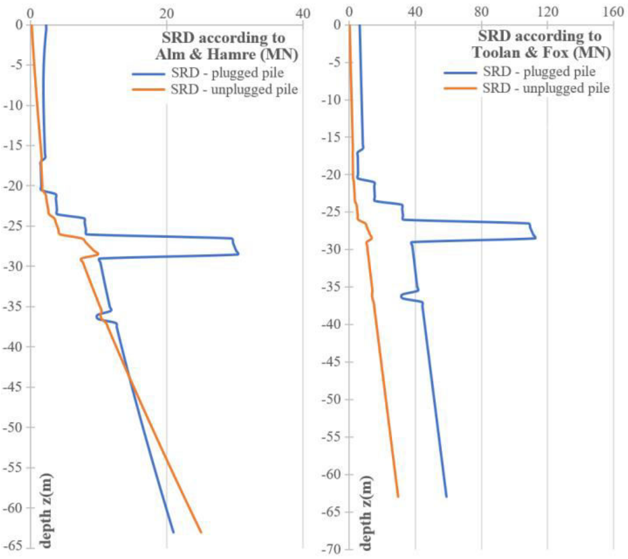

Figure-12. SRD evolution with depth according to Alm & Hamre (left) and to Toulan & Fox (right) for plugged and unplugged piles.

Figure-12 shows that the soil's resistance to driving following the Alm and Hamre (2001) method in the plugged case is generally greater than the unplugged one up to a depth of -44mzh, where the lateral friction outweighs tip resistance. Also, SRD distributions for the plugged and unplugged cases have the same shapes for both tip friction and lateral friction. As for the Toulan and Fox model (1977), the SRD curves for the plugged and unplugged cases are well separated and the resistance of the plugged case is clearly greater than that of the Alme & Hamre model.

Finally, the SRD values (bearing capacity) calculated previously by the Alm & Hamre and Toolan & Fox methods can be used to predict the number of blows necessary per meter of pile driving. This prediction is made using the Smith (1960) model based on the wave equation.

Simulation results of the two models and of the dynamic pile-driving test are shown in Figure-13.

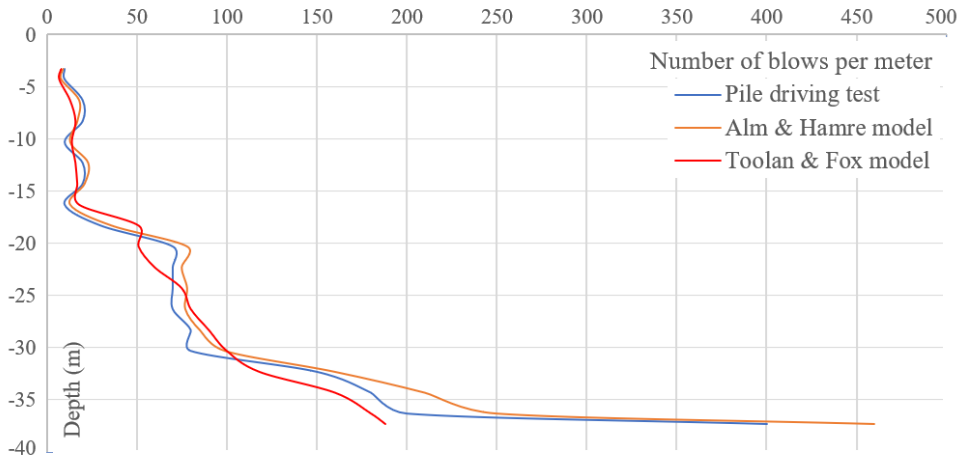

Figure-13. Number of blows per meter necessary for depth reach.

For the studied pile, Figure-13 shows that in the embankment layer (from 0 to approximately 16 meters deep), the two models (Alm & Hamre and Toolan & Fox) give results similar to those recorded in the pile-driving report. From this depth downward, the Toolan & Fox model deviates significantly from the actual results. From the depth of 35 meters, driving becomes difficult (significant increase in the number of blows) with the Alm & Hamre model corresponding exactly to the actual results.

The significant increase in the number of blows per meter from depth -35m (beginning of the marl layer) confirms previously predicted pile plugging occurrence (see Figure-11).

DISCUSSIONS

Soil plugging is explained by the vault effect. As the pile penetrates the soil, the latter becomes more compact, leading to an increase in horizontal stresses and in frictional forces between the pile and the soil column. This plugging is likely to occur when the pile is installed using static methods. Dynamic driving and vibro-driving methods, on the other hand, produce continuous cyclical shearing of the soil, preventing the continuous development of the vault effect (cycles of forming, destruction, and reforming).

The Alm & Hamre model takes into account soil fatigue due to friction by including soil degradation in their formula. That is, the resistance of the plugged case of the Toolan & Fox model is clearly greater than that of the Alme & Hamre one.

Drive refusal of the pile at the marl layer level is rather caused by the plugging phenomenon than by the substratum resistance since the pile penetrated a more resistant layer (very dense gravel of q_c=65 MPa at 25.7 meters deep).

CONCLUSIONS

The main outtake of this study is that plugging risk mainly decreases with an increase in pile diameter and driving acceleration.

Even if the risk of plugging diminishes with driving, checking its absence is important in order to avoid premature pile drive refusal. To do this, it is best to use the Alm & Hamre model, which takes into account soil fatigue due to friction when estimating the SRD.

The Alm and Hamre model gives results that are more representative of reality than the Toolan and Fox model. It follows the same evolution as the driving results graph, with a few deviations due to the choice of simulation parameters.

The significant increase in the number of blows per meter down from 35 meters deep (beginning of the marl layer) confirms the previously predicted pile plugging. Thus, the initially formulated plugging condition was confirmed by our case study.

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