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A prediction method for the installation of vibratory driven piles

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Abstract

A calculation method is presented to predict the drivability of vibratory driven piles and sheet-piles. The calculation model incorporates degradation of the soil resistance under cyclic loading. The model is strengthened by an exhaustive preliminary site measurement campaign which gave an insight in the dynamic behaviour of both the vibrated sheet-pile and the surrounding soil. The paper shows the correlation between measured and calculated driving times for several sites.

1. Introduction

In order to install sheet-piles, three basic different driving techniques can be distinguished : impact hammering, vibratory driving and pressing. With regard to drivability, impact driving and pressing of piles is considered possible as long as the driving force - limited to a certain extent by available energy - exceeds the resistant forces (figure 1). Driving is possible as long as :

$$F_{driving} > F_{base} + F_{shaft} + F_{clutch} \qquad (1)$$

During the impact driving, the driving energy is delivered by high forces induced by means of an impact hammer blow per blow. For pressing, the driving potential comes from the reaction force of a kentledge or uplift resistance of already installed sheet-piles.

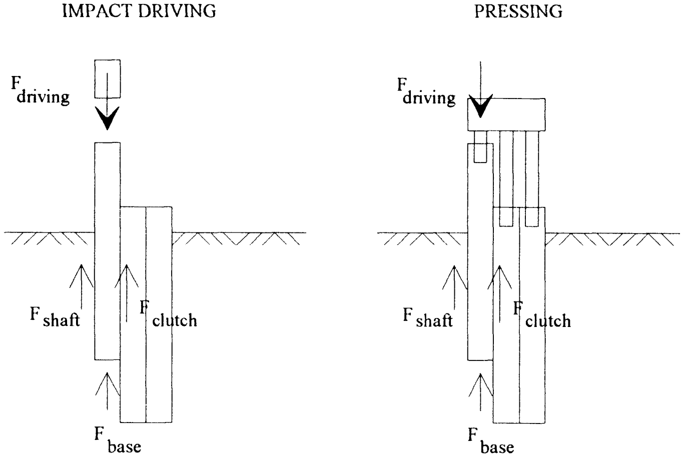

Figure 1. Drivability of impact driven and pressed sheet-piles

Vibratory driving is often used to install steel piles. Unlike impact driving and pressing of steel piles where driving is possible as long as the static soil resistance is overcome by the induced force, installing sheet-piles by vibrating relies mainly upon the reduction of the static soil resistance by vibrating the pile (and thus the soil). As discussed by others [1, 2], the building up of pore pressures, eventually leading to liquefaction, caused by the cyclic movement, leads - especially in saturated sands - to a significant reduction of the static soil resistance. Actually, the pile is not installed into the ground primarily by the vibrating force but rather under its own weight.

2. Theoretical model for the drivability of vibratory driven piles

The drivability of vibratory driven piles has been investigated as part of a European research program. An analytical model has been developed to permit the prediction of the driving speed into the soil.
For each depth the following calculations are made :

2.1. Primary estimate the acceleration amplitude

$$a = \frac{F_c}{M_v} = \frac{m_e . \omega^2}{M_v} \tag{2}$$

where

F_c = centrifugal force of the vibrator
M_v = mass of the vibrating parts (pile, clamps, vibrating part hammer)
m_e = eccentric moment of the vibrator
ω = frequency of the vibrator

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2.2. Define static soil resistance

The static base (q_s) and shaft (τ_s) resistance profiles are derived from Cone Penetration (CPT) tests.

2.3. Calculate liquefied soil resistance

The totally liquefied base and shaft soil resistance are derived from the CPTdata based on an exponential law as expressed respectively in equations (3) and (4) :

$$q_l = q_s \cdot \left[\left(1 - \frac{1}{L}\right) . e^{-\frac{1}{FR}} + \frac{1}{L} \right] \qquad (3)$$

$$\tau_l = \tau_s \cdot \left[\left(1 - \frac{1}{L}\right) . e^{-\frac{1}{FR}} + \frac{1}{L} \right] \qquad (4)$$

where

q_l = liquefied base resistance
τ_l = liquefied shaft resistance
FR = friction ratio as measured in a CPT test (ratio of the mantle friction to the cone resistance)
L = empirical liquefaction factor expressing the loss of resistance attributable to liquefaction (L will be higher for saturated and loose sands and is chosen in the range of 4 to 10)

2.4. Calculate driving soil resistance

The driving base and shaft resistance are derived from the static and the "liquefied" soil resistance depending on the vibration amplitude following an exponential law as expressed respectively in equations (5) and (6).

$$q_d = (q_s - q_l) . e^{-\alpha} + q_l \qquad (5)$$

$$\tau_d = (\tau_s - \tau_l) . e^{-\alpha} + \tau_l \qquad (6)$$

where

q_d = driving base unit resistance
τ_d = driving shaft unit resistance
α = acceleration ratio (= a/g) of the pile

At each depth z the vibratory pile driving resistance is calculated :

$$F_{base} = q_d \cdot \Omega \qquad (7)$$

$$F_{shaft} = y . \int_{z=0}^{z=z} \tau_d . dz \qquad (8)$$

where Ω is the pile section and y is the pile perimeter.

2.5. Recalculate the vibration amplitude

The soil resistance causes a diminution of the vibration amplitude. Equation (2) can be modified as :

$$a = \frac{m_e \cdot \omega^2 - \delta . F_{shaft}}{M_v} \tag{9}$$

where δ is a damping factor expressing the ability of the shaft resistance to dampen the movement of the sheet-pile.

Calculation steps 2 to 5 can be remade until there exist no difference between 2 consecutive values of the vibration amplitude.

2.6. Calculate the driving speed

The vibratory driving speed is obtained by applying Newton's law on the vibratory pile driving process (figure 2).

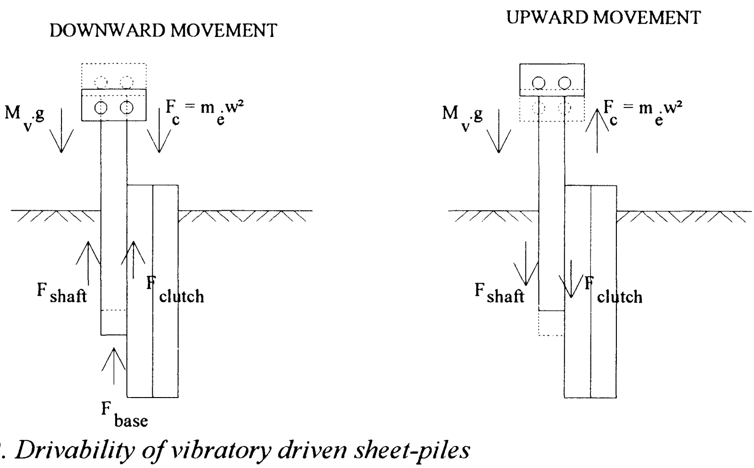

Figure 2. Drivability of vibratory driven sheet-piles

The resultant downward and upward forces are calculated as :

$$F_{\downarrow} = m_e \cdot \omega^2 + M_v \cdot g - F_{shaft} - F_{clutch} - F_{base} \tag{10}$$

$$F_{\uparrow} = m_e \cdot \omega^2 - M_v \cdot g - F_{shaft} - F_{clutch} \tag{11}$$

Finally, the resultant driving speed is calculated

$$v = v_{\downarrow} - v_{\uparrow} = \int \frac{F_{\downarrow}}{M_v} . dt - \int \frac{F_{\uparrow}}{M_v} . dt \tag{12}$$

3. Comparison of predicted and observed driving times

A computer program has been developed based on the general approach explained above. In order to increase the calculation speed some simplifications, which are not addressed in this paper, have been introduced. Although the model presented herein is rather simple, a very good agreement is obtained between the predicted and the recorded driving times on a variety of jobsites with different soil characteristics and different vibratory hammers and piles. The predicted and the observed penetration times are compared in table 1.

From the last two columns in table 1, it can be seen that the measured penetration times correspond well to the predicted in most of the cases. For the measurements 13,14, 17 and 18, an important difference is obtained. This difference can not be easily explained for the measurements 13, 14 and 17. However, it must be noted that in both sites covering those cases an important variation in driving times was observed due to the irregular presence of respectively hard sand lenses and schist layers.

Measurement 18 refers to the installation of a 20,6 m long tubular steel pile with a thickness of 9,5 mm and a diameter of 1 m on a site in Kortrijk (B). Figure 3 shows the subsoil profile at the site as deducted from a CPT soil investigation (10 cm² cone section). The water table was found at -1.8 m.

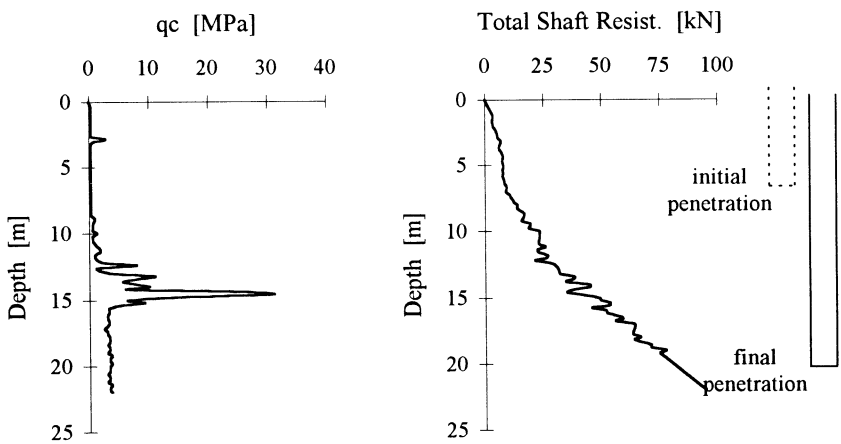

Figure 3. Subsoil profile site at Kortrijk

A preliminary calculation with the above described computer program pointed out that the necessary time to install the pile to a depth of 20 m with a PTC 30HFV vibratory hammer was 13½ minutes. However, it was observed that driving was possible down to a depth of only 11 m.

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Site	Subsoil	Hammer type (f [Hz] / m_e [kg.m])	Pile: Type	Pile: Length [m]	Pile: Installat. Depth [m]	Ref no.	Predicted time [min'sec"] or max. depth	Observed time [min'sec"] or max. depth
Hingene (B)	sand	PTC 50H2 (25/50)	AZ 13	9,8	9	1	1'15"	1'55"
			BZ 17	11,7	10	2	1'45"	2'
			BZ 17	16,1	14	3	3'55"	3'40"
		PTC 30HF (38/26)	BZ 17	11,7	10	4	1'50"	2'20"
			BZ 17	16,1	14	5	6'10"	6'35"
Evergem (B)	sand,clay	PTC 30HFV (38/26)	ZN 41/1670	14	13	6	2'40"	+/- correct (*)
St-Stevens-Woluwe (B)	sand	PTC 50H2 (25/50)	AZ 26	14,5	13	7	3'15"	3'30"
			Larsen 3N	10	8	8	1'30"	1'35"
Watermaal-Bosvoorde (B)	loose sand	PTC 13HF (38/13)	BZ 12 double	8	7	9	35"	32"
			BZ 12 simple	8	7	10	20"	11"
Moerzeke (B)	clay	PTC 15HF (38/15)	AZ 18	11,5		11	driv. to 11,5 m	driv. to 11 m
North Sea	clay,sand	ICE 1412 (22.5/115)	tubular pile (Ø 1,067 m)	28		12	driv. to 9,5 m	driv. to 10 m
			tubular pile (Ø 1,067 m)	27	10	13	21'	12'
Liège (B)	grit schist	ICE 14 RF (38/14)	BZ 17	9,8	7,5	14	3'30"	6'40"
			BZ 17	12	8,2	15	4'20"	4'10"
			PU 25	11,6		16	driv. to 7,6 m	driving to 8 m
		ICE 416 (17/23)	BZ 17	12		17	driv. to 7,5 m	driving to 5 m
Kortrijk a (B)	dense clay	PTC 30HFV (38/26)	tubular pile (Ø 1,01 m)	20,6	20 m	18	13'30"	driv. to 11 m
		PTC 110HD (20/98)	tubular pile (Ø 1,01 m)	20,6	11 to 20	19	5'45"	5'25"
Kortrijk b (B)	clay	PTC 30HFV (38/26)	tubular pile (Ø 1,01 m)	20,6	20 m	20	8'	+/- correct (*)
Limelette (B)	loam	prototype (25/1.4)	U-pile (46 cm^2, 0.82 m)	3		21	driv. to 0,5 m	driv. to 0,5 m

(*) The real penetration time was not measured but does more or less correspond to the predicted time (information contractor)

Table 1. Predicted and observed penetration times

The reason for the difficult driving and the difference between the predicted and the observed penetration speed was explained by measurements taken during the actual driving of the pile.

The pile vibration amplitude was measured by means of a velocity transducer placed at the pile head and a velocity transducer (protected by a cover) at the pile toe.

Figure 4 shows the monitored amplitude of vibration at the pile top and at the pile base upon loss of drivability. The observed frequency was 38 Hz.

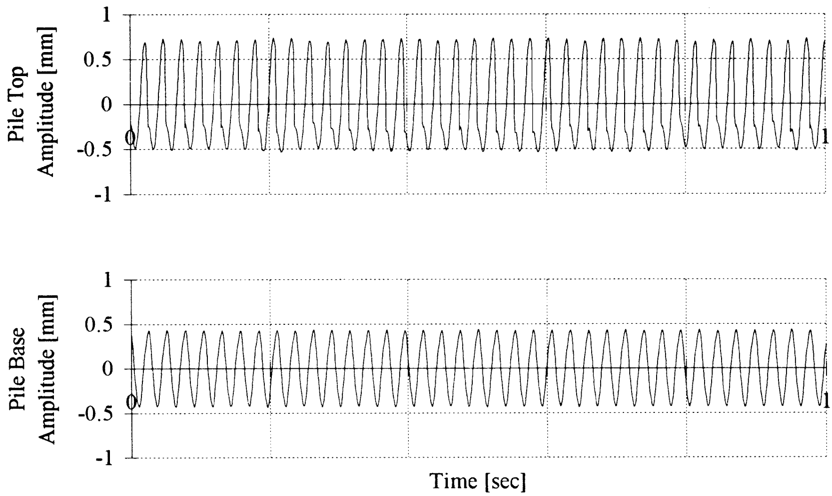

Figure 4. Record from the vibration amplitude upon loss of drivability

From the measurement results, one can observe that :

- the vibration amplitude at the pile top (0.65 mm zero to peak) is considerably less than the nominal vibration amplitude which is,

$$\frac{m_e}{M_{vibr}} = \frac{m_e}{M_{hammer} + M_{pile}} = \frac{26000 kg.mm}{(6500 + 4820) kg} = 2.3 mm$$

- the amplitude at the pile base (0.45 mm zero to peak) is smaller than the amplitude at the pile top (0.65 mm)

It would appear that the pile base amplitude (0.45 mm) is not sufficient to allow the pile to penetrate as the stress-strain behaviour for clayey soils is primarily elastic for small amplitudes. The small vibration amplitude may be attributed to one or more of the following factors :

① An important soil (i.e. clay) mass is sticking to the vibrating pile, leading to a more important vibrating mass, leading to a smaller vibration amplitude.

② The vibratory hammer may not be able to deliver the required energy, and thus not maintain its nominal amplitude or frequency. A characteristic of the PTC variable eccentric hammers is that a lack of power results in a reduction of vibration amplitude (in contrast to a reduction of frequency) [3].

③ A smaller amplitude at the pile toe is obtained due to the elasticity of the pile.

By applying the observed vibration amplitude to the calculation model (figure 5), a much better correlation between the calculated and the observed penetration time was obtained. The pile was placed at the bottom of an excavation at -2,5 m and penetrated another 4,5 m under its own weight. As a result, observed and calculated penetration rates are reported starting at level -7 m. Figure 5 evidences that the difference for the predicted and observed penetration times for the site in Kortrijk was not due to an incorrect estimation of the dynamic soil resistance but due to an incorrect estimation of the vibration amplitude.

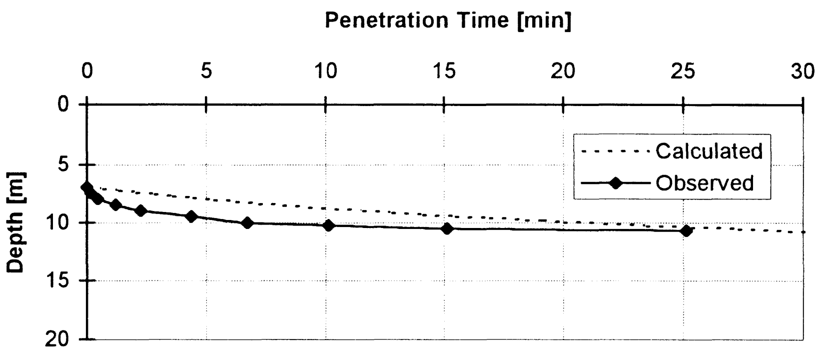

Figure 5. Predicted and observed penetration times site at Kortrijk

4. Evaluation and limitations of the calculation model

Although the model presented herein is rather simple, the calculated driving times show very good agreement with the observed driving times under a variety of subsurface conditions.
We may conclude that the calculation of the dynamic soil resistance may well show very good agreement with physical reality.
Analysing more in detail the phenomenons leading to loss of drivability reveals that the determination of the dynamic soil resistance is not the only part governing the drivability calculation.

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Based on our experience with many different sites [4] we can summarise the phenomenons leading to loss of drivability (table 2). Not all this phenomenons are incorporated in the calculation model, thereby evidencing the limitations of the suggested approach.

With regard to the vibratory hammer : • The cyclic movement reduces the static soil resistance allowing the pile to penetrate under its steady driving forces. Lack of penetration speed can be due to the insufficient weight of the (non-vibrating part of the) vibratory hammer. • An insufficient eccentric moment of the vibratory hammer leads to an deficient vibration amplitude. • An deficient power of the driving powerpack leads to a decrease of the vibration amplitude or of the vibration frequency of the vibratory hammer. • Deficiently serviced hammers will not be able to run at their nominal amplitude and frequency.
With regard to the pile : • Too a heavy pile reduces the vibration amplitude following the equations (1) and (8). On the other hand, a heavy pile is advantageous as appears from equation (10) and (11). A compromise has to be found. • When the pile is too elastic (e.g. too small pile section Ω), the vibration amplitude can be insufficient at the pile base, too much transverse vibration can occur preventing the pile from penetrating, and the pile can be damaged. • Amongst other possible problems, poor quality sheet-piles lead to an important increase in clutch friction resistance.
With regard to the soil : • Too high base resistance leads to bouncing of the vibrated pile (e.g. rock). • Increase of soil resistance due to compaction (e.g. sand) as reported more in detail in [5]. • Insufficient reduction of the dynamic soil resistance (e.g. hard clay). Some references deal with this subject [1], [2]. • Diminution of the pile vibration amplitude due to soil sticking to the vibrating pile (e.g. clay). • Too elastic soil stress-strain behaviour leads to a lack of relative displacement of the pile with regards to the soil (e.g. clay).

Table 2. Phenomenons leading to loss of drivability

The mentioned phenomenons do not cover all the possible drivability problems ; in many cases different from those monitored phenomenons are interfering, as shows the analysis of the results on the site in Kortrijk.

5. Conclusion

A calculation method has been developed for the prediction of vibratory driven piles and sheet-piles. The model allows to calculate the penetration time and to define whether it will be possible or not to drive a pile or sheet-pile to the required depth. The best choice regarding drivability between the available hammers can be made by calculating the driving times for different hammer types in given site conditions.
The comparison of the predicted penetration times with the observed times demonstrates the reliability of the model.

It has been shown that a correct calculation of the dynamic soil resistance during the vibratory driving process is one of the important issues in the determination of the drivability. Further research is required to analyse phenomenons leading to loss of drivability. A more elaborated pile-driving simulation program has been recently developed to allow the calculation of the vibrations in the surrounding soil.

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