Pile driveability prediction method based on CPT results

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ABSTRACT: The presented method is based on the assumption that bearing capacity of a pile can be computed with reasonable accuracy by the Danish formula and that this capacity is equivalent to a capacity calculated based on the results of a CPT test. The method allows driveability prediction of a single pile and also the influence of group of piles can be taken into account. For the purpose of analysis a calculation file was generated allowing for the comparison of calculation results with the results of field-tests. A comparative analysis of a number of square 0.4x0.4 m piles in clays and sands is carried out. The comparison shows reasonable agreement between the predicted and actual depth driving profiles.

1 INTRODUCTION

A number of methods have been presented where the focus is concentrated on the evaluation of pile capacity (Abu-Farsakh and Titi 2004; Jardine et al. 2005). Such an approach can be insufficient for driven piles. When the problem of driveability is ignored, a risk of uneconomical design arises. Even more serious can be consequences in case of such construction as an offshore windmill with monopile foundation, where ignoring the driveability can result in unsafe design.

The experience of the author indicates that there is a group of factors determining the possibility of driving a pile to the assumed level. The most obvious are the type and strength of the soil to be penetrated, pile length and its material and last but not least the driving energy. When considering cohesionless soils, a very important factor is the number of piles for the unit area. Significance of some additional elements was emphasized in the literature. These are friction fatigue (Colliat et al. 1995; Heerema 1978) and point of the transmission energy onto the pile, i.e. if the pile is driven by the so called bottom- or top-hammering (Choe and Juvkam-Wold 2002). In the method proposed below the two latter factors are disregarded.

2 BACKGROUND

The problem of driveability was most often analyzed in case of oil platforms (Dutt et al. 1995), though some analyses were also done for onshore piles (Goble et al. 1979) including concrete piles (Alm and Hamre 2001; Hussein et al. 2006). Most methods are based on the wave equation as given by Smith (1960). Usually they require relatively much effort and special software (Goble et al. 1997). Many of the methods do not allow for the analysis of the impact of other, previously driven piles on the soil strength and thus on the driveability of the next piles. To the author’s knowledge, only a few papers discuss the issue of the influence of the group effects (Henke 2008).

3 SINGLE PILE ANALYSIS

3.1. *Basic equations*

Starting points for the method are Eq. (1) and Eq. (2). The first equation is the so-called Danish formula for the evaluation of bearing capacity of a pile based on the driving characteristic. The Danish formula is considered as one of three most reliable dynamic formulae (Poulos and Davis 1980). In Eq. (2) the bearing capacity of a pile is estimated on the basis of CPT results.

$$\mathrm{R}(N_{20}) = \frac{\eta \cdot \mathrm{G} \cdot \mathrm{H}}{\mathrm{s}_{20} + 0.5 \cdot \sqrt{\dfrac{2 \cdot \eta \cdot \mathrm{G} \cdot \mathrm{H} \cdot \mathrm{L}}{\mathrm{E} \cdot \mathrm{A}}}} \tag{1}$$

$$\mathrm{R}(\mathrm{C}PT) = \mathrm{A} \cdot \mathrm{q_b} + \int_0^{l} \mathrm{U} \cdot \mathrm{q_s}\, dz \tag{2}$$

where $R(N_{20})$ = pile capacity based on the number of blows necessary for 0.2m of pile penetration; A = pile base area; η = efficiency of transfer of the potential energy of hammer; G = nominal hammer weight; H = hammer stroke; s_{20} = pile set averaged over 0.2m; L = total pile length; E = elasticity modulus of pile-cushion system; R(CPT) = pile bearing capacity based on CPT results; q_b = unit pile base resistance; q_s = unit pile shaft resistance; U = pile shaft circumference; l = pile embedment; z = depth coordinate.

It is assumed that pile capacities given by both formulae are equivalent and their right sides can be therefore equated. After a simple transformation Eq. (3) and Eq. (4) are obtained.

$$s_p = \frac{\eta \cdot G \cdot H}{A \cdot q_b + \int_0^l U \cdot q_s dz} - 0.5 \cdot \sqrt{\frac{2 \cdot \eta \cdot G \cdot H \cdot L}{E \cdot A}} \quad (3)$$

$$N_{20} = \frac{0.2m}{s_p} \quad (4)$$

where N_{20} = number of blows needed for 0.2 m of pile penetration.

3.2. *Unit resistances of pile base and shaft*

Pile base unit resistance and pile shaft unit resistance are calculated based on the Eq. (5) and Eq. (6) respectively

$$q_b = k_b \cdot q_{cb} \quad (5)$$

$$q_s = k_s \cdot q_c \quad (6)$$

where k_b = base resistance coefficient; k_s = shaft resistance coefficient see Table 1; q_{cb} = cone resistance averaged between 1,5D below and 1,5D above pile base; q_c = cone resistance at depth in question for pile shaft capacity evaluation; D = pile diameter.

Table 1. Pile unit resistance coefficients

Soil type	Base coefficient, k_b	Shaft coefficient, k_s
Cohesive	0.7	0.02
Cohesionless	0.4	0.005

Since an accurate calculation of pile capacity by dynamic formula is actually impossible, minimal and maximal pile capacities are calculated. It is recommended that the minimal capacity is 0.9 and the maximal 1.1 of the results obtained based on Table 1. In this manner a number of blows for pile driveability is acquired as a range.

3.3. *Additional assumptions*

To evaluate the capacity of a pile by Eq. (1) some additional recommendations should be given. Very important but uncertain factors are hammer efficiency and the elastic modulus of pile-cushion system. The indications of Table 2 and Table 3 can be taken to obtain reliable results. When using a dolly, all numbers in Table 2 should be reduced by 0.2. The latter number is a rough estimation, because the loss of energy change with the penetration of the dolly.

Table 2. Efficiency of hammers

Hammer type	Drop height, H_m (m)	Hammer efficiency, η_m
Free fall hydraulic	≤0.4	1.0
	0.4 to 0.6	0.9
	>0.6	0.8
Hydraulic with accelerator	≤0.3	1.3
	0.3 to 0.5	1.2
	>0.5	1.0

Table 3. Moduli of elasticity of pile-cushion system

Pile is made of a number of elements	Reinforcement ratio of pile cross section, ξ_a (%)	Modulus of elasticity, E_p (GPa)
NO	≤2	30
	2÷4	35
	>4	38
YES	≤2	20
	2÷4	25
	>4	28

It should be noted that the number of elements contributes to the elasticity of pile-cushion system, such as the Young modulus of the concrete, reinforcement ratio, Young modulus of the cushion and the fact whether a pile is coupled of a number of sub-elements or not. The recommendations given in Table 3 refers to the concrete of grade 50MPa made from basalt aggregate.

4 INFLUENCE OF GROUP OF PILES

Beside checking the driveability of a single pile the method also allows for driveability evaluation of a pile including the influence of previously driven piles. It is considered this can be only problem in cohesionless soils. In clays it is assumed that installation of a pile does not alter the strength of the soil. It was in fact observed by the author that the driving profiles didn’t differ between successively driven piles within one foundation. This idealization can be also theoretically justified by the fact that while the pile is being driven the clay is loaded in undrained conditions and thus there is no volume change of the soil. Instead, the soil in the space taken by the pile element moves to the sides or upwards.

In sands an assumption is adopted that the zone of the soil influenced by any pile being driven equals to a vertical cylinder with a radius of $10D_p$ and the axis at the pile axis. This has been roughly confirmed by unpublished inclinometer measurements done by the author during the installation of piles in a number of soil conditions. The influence zone is assumed around the pile to be analyzed.

The total influence zone is divided into two subzones i.e. zone 1 and zone 2, which are respectively represented by a cylinder with a radius of $5D_p$ and a hollow cylinder taking the rest of the whole influence zone. The soil strength change is characterized by a rise in cone resistance q_c in the zone 1. The grade of the change depends on the number of piles previously driven in the subzones. The change is induced by the reduction of void ratio, which in comes from displacement of the soil from the space taken by the piles and from vibrations during the installation.

The first step of the calculation is the assessment of the voids volume in zone 1. In order to evaluate the void ratios of soils along the line of CPT, the definition of density index I_D Eq. (7) is transformed to Eq. (8).

$$\mathrm{I_D} = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \tag{7}$$

$$\mathrm{e} = e_{\max} \cdot I_D \cdot \left(e_{\max} - e_{\min}\right) \tag{8}$$

where e_{min}; e_{max} = minimal and maximal void ratio respectively. A relation between q_c and I_D according to Eq. (9) is used to obtain the Eq. (10) i.e. the relation between the in situ void ratio e and cone resistance q_c.

$$\mathrm{I_D} = 0.709 \cdot \log(q_c) - 0.165 \tag{9}$$

$$\mathrm{e} = 0.709 \cdot e_{\max} \cdot \log(q_c) - 0.165 \cdot \left(e_{\max} - e_{\min}\right) \tag{10}$$

The Eq. (9) is the formula from the Polish Code (2002), however, any formula can be used which is considered as reliable for the region in question. The range of minimum e_{min} and maximum e_{max} volume ratios as dependent on the type of soil is proposed (see Table 4). This is a rough assessment based on the literature (Kim et al. 2002; Konrad 1998). Ideally, the values of $e_{min;}$ e_{max} should be measured in the laboratory for each site.

Table 4. Void ratio of different types of soil

Soil type	Coefficient of uniformity, c_u	Range of void ratio, $e_{min} \div e_{max}$
Fine and silty sand	≤ 2	0.55÷0.80
	(2;4)	0.50÷0.85
	≥4	0.40÷0.85
Medium and coarse sand	≤ 2	0.55÷0.80
	(2;4)	0.50÷0.80
	≥4	0.40÷0.80
Gravel and sand-gravel mix	≤ 2	0.55÷0.70
	(2;4)	0.50÷0.70
	≥4	0.40÷0.70

Taking the volume V_1 of the zone 1, volume of voids within it can be calculated by Eq. (11)

$$\mathrm{V_{e1}} = e \cdot V_1 \tag{11}$$

Next, the change of the void ratio of the soil in the zone 1 caused by the impact of the piles already installed is evaluated. It is assumed that the piles which have been previously driven induced two phenomena, i.e. the rise of void ratio by the rearrangement of soil particles and displacement of soil towards the region outside the influence zone. Only the first problem is of interest, because the second one by definition does not alter the strength of the soil within the zone. In order to assess the rise of the void ratio equation Eq. (12) and Eq. (13) are proposed.

$$\kappa_1 = 0.15 \cdot \sqrt{n_{p1}} + 0.7 \cdot \frac{V_{p1}}{V_{e1}} \tag{12}$$

$$\kappa_2 = 0.5 \cdot \frac{V_{p2}}{V_{e2}} \tag{13}$$

where V_1 and V_2 = total volume of the zones 1 and 2 respectively, V_{e1} and V_{e2} = volumes of voids in the zones, V_{p1} and V_{p2} = total volumes of piles previously driven in the zones, n_{p1} = number of piles in the zone 1. The influence factor κ_1 defines the impact of piles installed earlier in the zone 1 on the void ratio in the same zone. It is assumed that the change of the void volume in the zone 1 equals to the 70% of volume of piles introduced to the zone. Additional assumption is adopted that the soil densifies also by dynamic impact (see first part of Eq. (12)). The second factor κ_2 is used to calculate the effect of piles driven in the zone 2 on the void ratio in the zone 1. Here it is assumed that half of the volume of piles introduced into the zone 2 reduce the volume of voids in the zone 1. The factors κ_1 and κ_2 are put into use in Eq. (14) to Eq. (16).

$$\Delta \mathrm{V_{e1}} = \kappa_1 \cdot V_{e1} + \kappa_2 \cdot V_{e2} \tag{14}$$

$$\mathrm{V_{e1red}} = V_{e1} - \Delta V_{e1} \tag{15}$$

$$\mathrm{e_{red}} = \frac{V_{e1red}}{V_1} \tag{16}$$

where V_{e1red} = reduced volume of voids in the first zone after driving all the piles in question; e_{red} = reduced void ratio in the zone.

In following calculation Eq. (7) is again used to assess the density index I_{Da} of the densified soil in the zone. Upon rearrangement of Eq. (9) to the form of Eq. (17) the cone resistance q_{ca} of the altered soil can be calculated.

$$q_{ca} = 10^{1.41 \cdot (I_{Da} + 0.165)} \qquad (17)$$

Then the calculation of driveability is made by formulae Eq. (3) and Eq. (4) and with the use of Table 1 to Table 3 analogously as for a single pile but using the q_{ca}. Note in practical application all parameters determined by Eq. (7) to Eq. (17) should be computed for vertical regions with the heights equal to the step of CPT, i.e. 0.02m for electrical cone and 0.2m for mechanical cone.

5 VALIDATION OF THE METHOD FOR SINGLE PILES

The actual and calculated driving profiles of four piles at three sites in Poland are shown below (see Fig. 1 to Fig. 4).

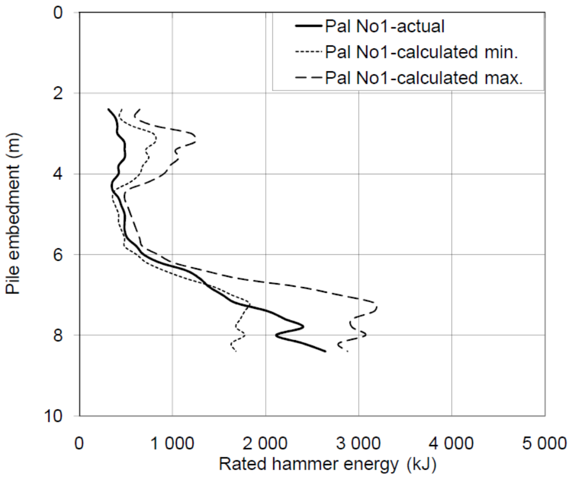

Fig. 1 Driving profiles of pile No1

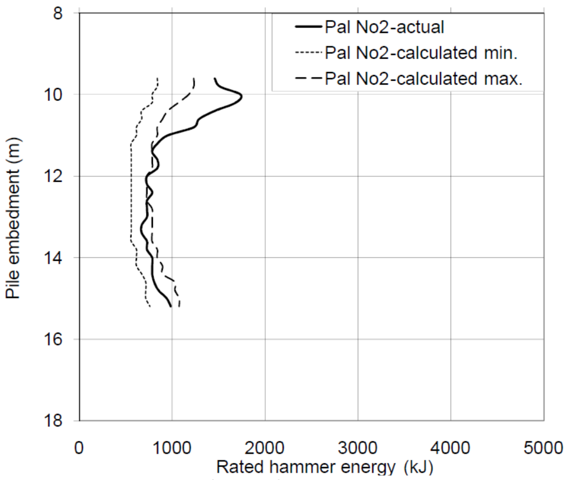

Fig. 2 Driving profiles of pile No2

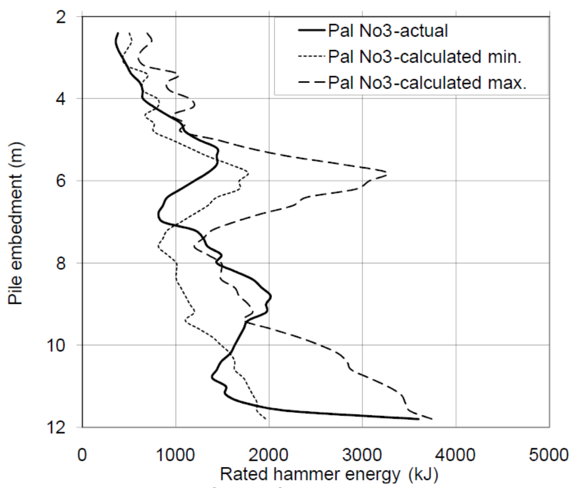

Fig. 3 Driving profiles of pile No3

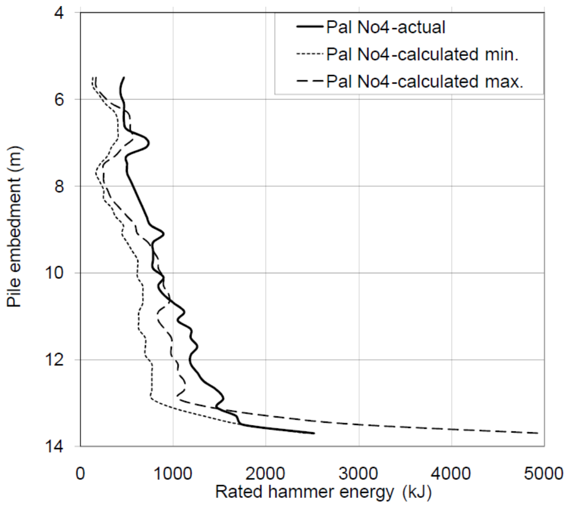

Fig. 4 Driving profiles of pile No4

The rated energy is taken as energy necessary to achieve the penetration of a pile of 0.2m. It is assumed, that the actual energy is potential energy of the hammer multiplied by the efficiency factor given in Table 2. Basic information regarding the piles are given in Table 5.

Table 5. Basic information about piles No1 to No4

Pile No	Soil con-ditions (*)	Hammer weight, G (kN)	Hammer type (**)	Pile cross-section, D (m)	Pile length, L (m)
1	A	60	2	0.4x0.4	11
2	B	70	1	0.4x0.4	14
3	C	60	2	0.4x0.4	13
4	D	70	2	0.4x0.4	15

(*) A – sands along pile shaft and below base after driving;
B – clays along pile shaft and below base after drving;
C – layered along pile shaft, clays below base after driving;
D – layered along pile shaft, sands below base after driving
(**) 1 – free fall hydraulic; 2 – hydraulic with accelarator

Presented results indicate a satisfactory correlation between the measured and calculated driving profiles. At some depths the acual profile falls outside the calculated range. This is probably due to a number of factors, such as inadequate assessment of the efficiency of the hammer and the elasticity of the pile-cushion system. It should be noted that both parameters vary during driving. The changes of the first one are likely to be more complex than suggested by the indications given in Table 2. The second parameter in turns is influenced by the wear of the wooden cushion during driving, the humidity of the concrete, and also its age and cracking. The latter factor is in practice unpredictable since it varies strongly with the advance of works, especially during hard driving. It is possible that at least some of the discrepancies between actual and predicted driving profiles arise from the fact that the CPTs were carried out at a distance between 10m and 15m from the analyzed piles. It should be also mentioned that the Danish formula is a very simple equation which does not take into account such important parameters like viscosity and damping of the soil along the shaft and below the pile base.

In the light of the comments given above the method should be applied with care, especially in stiff or soft cohesive soils. In practice it is recommended to use the approach mainly to assess the maximal depth, to which a pile can be driven. A criterion of refuse used usually by the author for concrete piles is 50 blows by hammer falling from 0.9m. The hammer weights of rigs most often used in Poland are 60 to 70kN, what gives the rated energy of about 2150 to 3150 kJ. From the driving profiles of piles No1, No3 and No4, which have been driven to refuse, it is evident that the maximal depth of driving is predicted with reasonable accuracy. The author analyzed over 50 piles with the lengths between 10 and 25m using the presented approach. All the mentioned piles had been driven to refuse and the maximum depth of driving was predicted with the accuracy of about 10% of the total pile length.

6 VALIDATION OF THE METHOD FOR A GROUP OF PILES

At present there is no reliable data to validate the approach for a group of piles. Two factors hamper the collection of necessary information. First, at a typical site in Poland the minimal and maximal void ratios of sands are very rarely measured. Second, the first piles driven at any site are the piles used for the purpose of static or dynamic tests. It is often the case that they have lengths, cross sections and age other than construction piles which are installed next. Preferably site data for the validation of the method should be collated in controlled conditions, what is not easy at a real site. Nevertheless from the measurements gathered so far some comparison of the actual and calculated driving profiles can be made (see Fig. 5). For the sake of clarity the calculated profiles are here drawn as profiles of the rated energy based directly on the indications of Table 1 instead of previously used ranges of maximal and minimal rated energy.

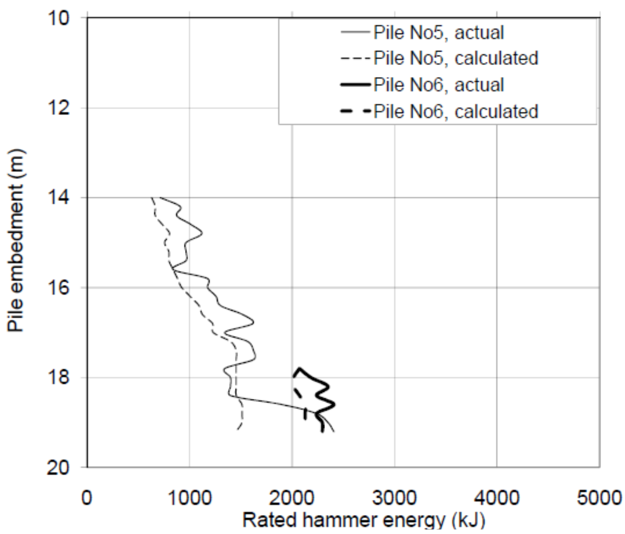

Fig. 5 Driving profiles of piles in group

Basic information about piles presented on Fig. 5 are given below (see Table 6).

Table 6. Basic information about group of piles

Pile No	Soil con-ditions	Hammer weight, G (kN)	Hammer type	Pile cross-section, D (m)	Pile length, L (m)
5	A	60	1	0.4x0.4	19
6	A	60	1	0.4x0.4	19

The pile No5 was a construction pile and during its driving a total number of 1 and 0 piles were already in place in the zones 1 and 2 respectively. Pile No6 was also a construction pile and before its installation a total number of 3 and 7 piles were in place in the zones 1 and 2 respectively. From the Fig. 5 it can be seen that the general driving profiles are predicted with reasonable accuracy although the information about the pile No6 is sparse to the depth of 17.7m. In addition to that it is apparent that the actual profiles of piles No5 and No6 are very similar in the depth range 18.5 to 19.5m. From other profiles at the same site it was evident the driving profiles of piles installed at a distance of 5 to 6m from each other differ strongly, what suggests significant spatial variability of soils.

7 CONCLUSIONS

An important task when designing driven piles is their driveability. Disregarding the problem can result in uneconomical or unsafe design. In the paper a method allowing for a simple analysis of driveablity of single piles and a group of piles is presented.

In case of a single pile the number of blows necessary to immerse a pile by 0.2m is obtained. The method has been verified for single piles in various soil conditions. The correlation between the calculated and measured driving profiles is satisfactory. Nevertheless the method has never been tested for very long piles in soft soils. In some cases analyzed by the author discrepancies between actual and predicted driving profiles in stiff clays were significant. For piles driven into sands driving profiles are obtained which are reliable. One of the important reasons for the limited credibility is very simple equations used in the method i.e. the Danish formula. In a majority of cases however the maximal depth, to which a pile can be driven, was predicted with reasonable accuracy.

Through for the lack of reliable data definite validation of the proposed method for a group of piles has to be postponed. The data collected so far allows for a preliminary evaluation. A comparison made in the paper shows reasonable agreement between actual and predicted driving profile. Again some discrepancies between both profiles suggest that for the time being practical application of the method should be limited to the prediction of the depth of refuse rather than of the full driving profiles.

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