DETERMINING THE AXIAL BEARING CAPACITY OF PILE BASED ON COMMON METHODS AND COMPARISON WITH PILE LOAD TEST

ABSTRACT: Determine the bearing capacity of piles is one of the most common problems in the geotechnical practice. In the last decades the CPT method is preferred, although the SPT and correlation with table values are often used as first approximation. FEM methods (using Plaxis for example) are also good alternative to the theoretical ones. This paper presents and compares some methods for estimation the axial bearing capacity of piles based on real geology and full scale load test according ISO DIS 22477-1. The influence of the selected method and data (DPH, CPT and FEM) as well as the technology uncertainties on the bearing capacity are discussed.

INTRODUCTION

Looking at the growing importance of geotechnics (due to urban development) and the execution of the new European standart (БДС - EN 1997-2:2007), it is often required to precisely determine the bearing capacity of piles. The investitures now are more likely to test number of piles. In this formulation it is easier for geotechnical engineers to determine the exact bearing capacity of adjacent piles. This situation leads to better understanding of soil–pile interaction and some technological specifics of pile foundation. It could even contribute to more economical design of such constructions. The article describes, summarises and compares the results of the bearing capacity calculations.

LOCATION AND GEOLOGY

Maritsa valley in the region of Plovdiv, Bulgaria is known for its particularly high permeable and loose sands. The foundation technology for the second half of the 20th century was based on driven piles. Now it is more available to use bore piles instead.

Table 1 Soil group symbol/number according to depth (Zero Pile point)

Depth, m	Soil group symbol/number
0,00 - 1,00	*siCl* / 2
1,00 - 2,00	*Sa* / 3
2,00 - 5,40	*siCl* / 2
5,40 - 8,00	*saCl - Sa* / 4
8,00 - 10,00	*Sa* / 3
10,00 - 20,00	4

Although there is only one borehole in the 1300 m2 area, there is no lack of geological data. The geology layers and depths are shown in table 1 and figure 1. CPT and DPH are also made for this site. Having in mind the local specifics of the geology (heterogeneous soil) it is still not enough for a perfect description of the soil. Based on the different nature of the insitu and laboratory testing it is appropriate for the input data not to be unified. The Table 2 shows all the laboratory and in situ parameters of soil types as well as the values used for the higher FEM models.

Because of investment problems a minimum pile diameter and lenght are chased. The pile has length 12,60 m and diameter of 88 cm, bored with casing. Its cap is 1.8 m under the flat ground surface.

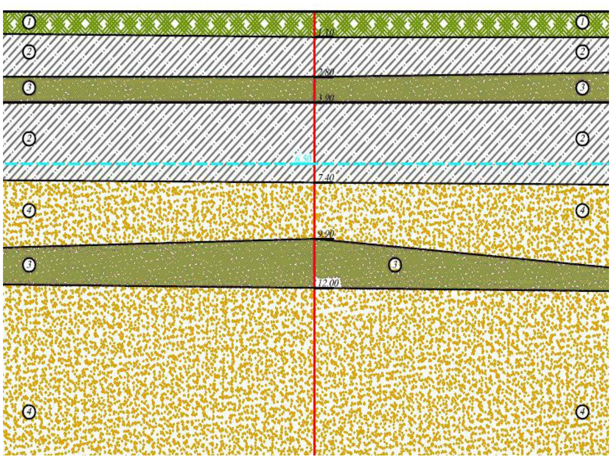

Figure 1 Geology - soil layers

Table 2 Soil parameters in models and calculations

Soil Number			2	3	4	5
Unit weight	γ_n	kN/m^3	17,6	17,4	19,7	19
Unit weight - sat.	γ_r	kN/m^3	17,9	19,1	20,5	19,1
Odometer modulus	E_{oed}	kN/m^2	12000	8000	20000	18000
Modulus at 50% shear strenght	E_{50}	kN/m^2	12000	8000	20000	18000
Unloading reloading modulus	E_{ur}	kN/m^2	48000	24000	60000	80000
Small strain shear modulus	G_0	kN/m^2	80000	10,0E4	12E4	80000
Poasson's ratio	ν'	-	0,35	0,3	0,45	0,45
Poasson's ratio ur.	ν_{ur}	-	0,2	0,2	0,2	0,2
Shear strain at 70% reduction of G/G0	$\gamma_{0.7}$	-	0,0003	0,0001	1,5E-4	0,0005
Referent pressure	*pref*	kPa	100	100	100	100
m parameter	*m*	-	0,7	0,6	0,6	0,8
Rf parameter	*Rf*	-	0,9	0,9	0,9	0,9
Over consolidated r.	*OCR*	-	1	1	1	1
Cohesion	*c*	kPa	18	5	18	36
Frinction angle	φ	°	9	30	31	11
Water content	*w*	%	42,4	18,7	17,6	31,9
Void ratio	*e*	-	1,184	0,81	0,606	0,87
Liquid limit	w_L	%	60,7	-	33,7	42,0
Plastic limit	w_p	%	34	-	19	22
Plasticity index	I_p	%	27	-	14	20
Consistency index	I_c	-	0,76	-	0,42	0,51
Degree of saturation	S_r	%	91,6	61,5	75,6	98,4
Grain size distrib.						
Gr: 200-2 mm		%	1	7	15	0
Sa: 2-0,1 mm		%	4	86	54	2
Si: 0,1-0,005 mm		%	64	6	23	73
Cl: <0,005 mm		%	31	1	8	25
Soil group symbol			*siCl*	*Sa*	*saCl Sa*	*siCl*

AXIAL BEARING CAPACITY ACCORDING DIN (CPT)

Method description

Although this method is now well known in Bulgaria, it is used for preliminary design. This method for determination of the pile bearing capacity is based on DIN 4014/DIN1054:2005. The German code suggests drawing a "load-displacement curve" according to Cone Penetration Test. The base resistance and skin friction are determined as a function of pile cap displacement.

$$R_{c,k} = R_{b,k}(s) + R_{s,k}(s) \tag{1}$$

Index *"k"* after the comma symbol indicates that the quantity has characteristic value (without safety factors).

Calculations

The base resistance is calculated at three different levels of displacement in accordance with qc (CPT). S = 0,02D = 17,6 mm; S = 0,03D = 26,4 mm; S = 0,10D = 88 mm. The CPT result for pile toe is q_c = 28.66 MPa. The next table data are used for cohesionless soils (table 3):

Table 3 Base resistance of piles $q_{b,k}$(in KN) in accordance of q_c (CPT) for cohesionless soils

Displacement	q_c=20 kPa	q_c=25 kPa
0,02D	1400	1750
0,03D	2800	2250
0,10D	3500	4000

The calculations are based on q_c = 25 kPa, as q_c = 28,66 kPa from CPT test. Multiplied by the base area of the pile (0,608 m^2) leads to base resistance: $R_{b(1,76cm)}$ = 1064kN; $R_{b(2,64cm)}$ = 1360 kN; $R_{b(8,80cm)}$ = 2432 kN.

According DIN the skin friction increases with pile displacement until threshold value is reached. That treshold is calculated as:

$$s_{sg} = 0,5.R_{s,k} + 0,5 \leq 3cm, \tag{2}$$

$$s_{sg} = 0,5.2,95 + 0,5 = 1,795cm. \tag{3}$$

The Skin friction is obtained as function of q_c (CPT) for each 10 cm of pile length according to DIN table (Table 4).

Table 4 Skin friction for bored piles $q_{s,k,}$(in KN/m^2) in accordance with q_c(CPT) for cohesionless soils

q_{ck}=0kPa	q_{ck}=5kPa	q_{ck}=10kPa	q_{ck}=15Pa
0	40	80	120

The skin resistance $R_{s,k}$ is calculated with the expression:

$$R_{s,k}(s) = \sum_{0}^{Lpile} (P_{pile}.q_{s,k}.0,10),\ kN\ , \tag{4}$$

where:

P_{pile} is the perimeter of the pile;

L_{pile} - pile length;

And 0,10 is the 10 cm pile slice.

The skin resistance R_{sk} increases from zero to 2950kN. After reaching the threshold displacement value of 1,795 cm $R_{s,k}$ remains constant.The sum of both base and skin resistance are given in table 5. Figure 2 presents graphically the calculated results.

Table 5 Pile bearing capacity, DIN (CPT)

Displacement, mm	R_{bk} kN	R_{sk}, kN	R_{ck} kN
0	0	0	0
17,6	1064	2630	3694
19,75	1035	2950	4085
26,4	1360	2950	4310
88	2436	2950	5382

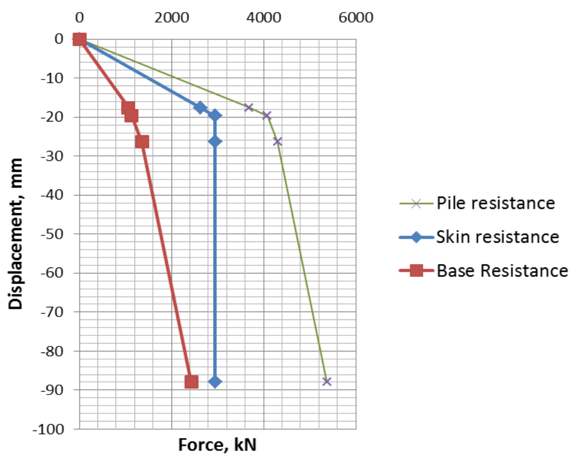

Figure 2 Pile bearing capacity, DIN (CPT)

AXIAL BEARING CAPACITY BASED ON DIN (DPH)

Method description

This method is also based on DIN Standard using correlated value for the qc (CPT). To obtain qc data, correlations between the number of blows N_{10H} and the cone penetration resistance (q_c) for poorly-graded sands as well as for well-graded sand-gravel are used. (Stenzel, 1978). Here N_{10H} shows the number of tip penetration blows of the dynamic penetrometer. The Dynamic penetration test is performed using the equipment described in table 6.

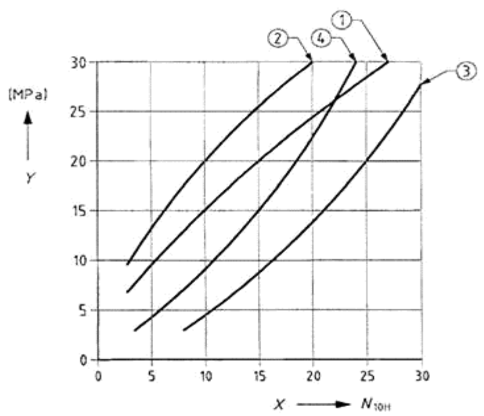

Figure 3 Correlations between N_{10H} and q_c (CPT) (Stenzel, 1978)

Table 6 Technical Probing equipment characteristics DPH

Regulation ref.	1750
Weight of striking mass	50 Kg
Freefall height	0,50 m
Weight of striking system	18 Kg
Diameter of cone tip	43,70 mm
Area of tip base	15 cm²
Rod length	1 m
Weight of rods /mt	6 kg/m
Depth first rod joint	0,90 m
Tip penetration	0,10 m
Number of blow by tip	N(10)
Correlation coeff.	2,034
Coating/Slurries	No
Cone tip angle	90°

The calculations are done analog to the description of DIN (CPT). The calculated values are shown in table 7 and figure 4.

Table 7 Pile bearing capacity, DIN (DPH)

Displacement, mm	R_b kN	R_s, kN	R_c kN
0	0	0	0
15,9	961	2195	3156
17,6	1064	2195	3259
26,4	1360	2195	3555
88	2432	2195	4627

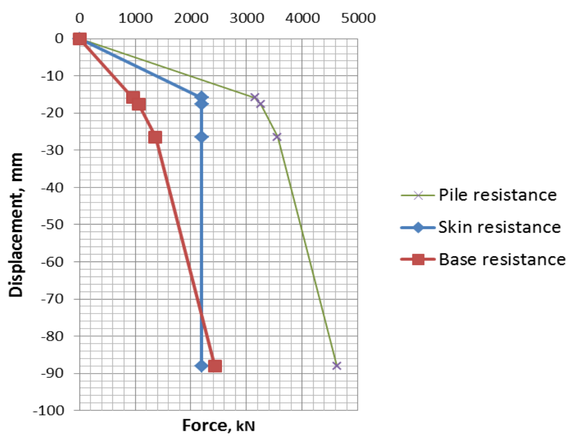

Figure 4 Pile bearing capacity, DIN (DPH)

FEM METHOD (USING PLAXIS)

Input data

The geometry of the model is based on the isotropic half space. It is accomplished by using the axisymmetry in Plaxis 2D.

The material properties are based on the laboratory and insitu tests. Material properties used in the FEM calculations are listed in table 2. The data for friction angle and cohesion are laboratory determined. The deformation moduli are obtained from odometer tests and insitu correlation. For the E_{ur} the well-known rule for 3-5 times E_{50} is also used (Schanz, 1999), whereas the parameter m is calibrated according to other tests on a similar type of soil (Tanev, 2015). The G_0 – module is calculated based on correlations (Benz, 2007) and tests (Kerenchev, 2014). Normalized shear modulus reduction curve and $\gamma_{0,7}$ are calculated according (Benz, 2007).

The boundary conditions and meshing are standard for Plaxis 2D axisymmetric (figure 5).
Force (point load) is used to simulate the stepwise loading procedure. The procedure follows that of the pile

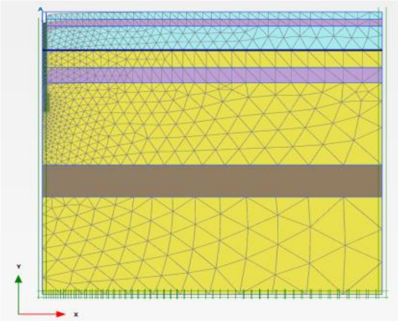

Figure 5 Figure caption of a typical figure

load test (DIN EN ISO 22477-1): Loading – unloading – reloading – unloading.

Calculations and results

The easiest way to obtain a load-displacement curve is by using time multiplier for all step levels. The results for both models - HSM and HS small are presented in figure 6.

COMPARISON AND DISCUSSION

Before the comparison it is important to notice the technology issues when boring the tested pile. It is well known that the base resistances as well as skin friction are technology dependent. The pile skin was well smoothed using the bore case and the pile toe was not cleaned after boring process. That has negative effect on the total bearing capacity.

The results from calculating the bearing capacity using penetrations are more common to show higher values than the load test. It could be because of the scale factor, as the cone has smaller surface than the pile and gravely soil is partly presented. The FEM model tends to underestimate the pile bearing capacity, but having in mind the technology of the tested pile it matches perfectly. HS small model has better performance during

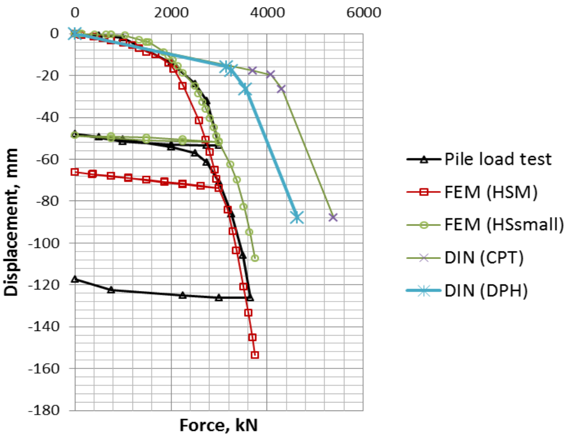

Figure 6 Plaxis 2D axisymmetric compered to DIN and pile load test

the first load cycle, when skin friction and small strain stiffness are presented, whereas the Hardening Soil model matches better the unloading-reloading conditions.

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