Deformation modulus determined by pile load test

Module de déformation déterminé par essai de chargement de pieux

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SYNOPSIS In this Paper are shown the results of several field load tests carried out on bored piles of diameter D = 90 cm, D = 120 cm and D = 150 cm.Using the load settlement curves the ultimate bearing capacity of the single pile was determined by Van der Veen's,Mazurkiewicz's and bilinear approximation method. On the basis of the results of cone penetration test (CPT) performed in the vicinity of the tested piles, the point resistance P_p and the side resistance P_s were calculated. The values of the modulus of deformation of soil E_s were also determined from field load test results.Correlations between unit skin friction and modulus of deformation on one hand, and cone penetration,on another, have also been discussed.

INTRODUCTION

The analysis of bearing capacity of piles consists of separating the total resistance of the pile in the poing resistance P_p and the side resistance P_s. The static methods are commonly used to determine the ultimate axial bearing capacity of pile:

$$P_u = p_p + P_s = P_pA_p + f_sA_s \tag{1}$$

where:

P_p = ultimate unit point resistance
A_p = area of pile point
f_s = ultimate unit skin friction
A_s = area of pile shaft

Using the cone penetration test results, the ultimate bearing capacity of a pile can be calculated by the following expression (Mohan et al, 1963):

$$P_u = R_pA_p + \frac{R_{pav}\ A_s}{50} \tag{2}$$

where R_p and R_{pav} are the cone resistance around the pile toe and the average penetration resistance over the length of the pile, respectively.

Bustamante and Gianeselli (1982) proposed a modified method for predicting the bearing capacity of pile:

$$P_u = R_{pc}\ K_c\ \frac{D^2\Pi}{4} + \sum_i \frac{R_{ps}}{K_s}\ D\Pi\ell_i \tag{3}$$

where D is the diameter of a pile, K_c and K_s are dimensionles coefficients, R_{pc} is the penetration resistance around the pile point and R_{ps} is the penetration resistance along the pile shaft.

Duncan and Chang (1970) showed that the nonlinear stress--strain curve for soils may be approximated in hyperbolic form. For piles, the relationship between the load P and settlement ρ is given by:

$$P = \frac{\rho}{a+b\rho} \tag{4}$$

where a and b are constants determined from field load tests.The ultimate bearing capacity of a pile can be determined as an asympthotic value of function when settlement ρ tends to infinity:

$$P_u = \frac{1}{b} \tag{5}$$

When the ratio $\frac{\rho}{P}$ has the form of the bilinear relationship, it is possible to define four parameters a, b, a_1 and b_1 from field load tests data.In this case the friction resistance of a pile can be determined by the following expression:

$$P_s = \frac{\rho_c^2\ b_1}{(a_1+b_1\rho_c)^2} \tag{6}$$

where ρ_c is assumed to indicate the full mobilization of skin resistance.

Side friction can also be determined on the basis of penetration test results. Meyerhof (1956, 1976) suggested the relation:

$$f_s = \frac{N}{50} \quad \text{or} \quad f_s = \frac{R_p}{100} \tag{7}$$

where N is the number of blows per foot in standard penetration tests in clays. Mohan et al (1963) proposed the relation:

$$f_s = \frac{R_p}{50} \tag{8}$$

where R_p is the cone resistance in the static penetration tests.

Bustamante and Gianeselli (1982) suggested dimmensionless coefficients K_s varying between 30 and 300.

The modulus of deformation E can be determined from the expression:

$$E = \frac{pD}{\rho}\mu_o\mu_1 \tag{9}$$

where D is the diameter of a pile, p is the contact pressure, ρ is the displacement of a pile, μ_o and μ_1 are dimensionless coefficients related to the shape of the loaded area, to the depth of the foundation and the thickness of the compressible layer.

The values of the soil modulus are often correlate with the cone resistance in the penetration test. Mitchell and Gardner (1975) have summarized some of the available correlations between R_p and E. In most cases this is a linear relationship given in the form:

$$E = \alpha R_p \tag{10}$$

where α is the coefficient of correlation. This coefficient was found to be α=1,5-4 for sands and α=3-12 for clays (Meyerhof, 1956; Thomas, 1968; Trofimenkov, 1974).

SOIL MODULUS FROM FIELD LOAD TEST

Because many uncertainties may be associated with laboratory test, it is desirable to determine the deformation modulus from a full-scale pile-loading test.

As shown by Poulos and Davis (1980), the pile-settlement behaviour can be analysed by Mindlin´s equations.

The displacement of a single pile is given by:

$$\rho = \frac{PI}{E_s D} \tag{11}$$

in which P = total applied load on pile, E_s = modulus of deformation of soil and D = pile diameter.

The dimensionless coefficient I depends on the ratio $\frac{L}{D}$, on the pile stiffness, on the Poisson´s ratio of soil and on the correction factor for base modulus.

RESULTS OF PILE LOAD TESTS

In this analysis the results of 4 large diameter bored piles have been considered. Test pile 1 had a diameter D=0,90 m and the length L=16,0 m. In Fig. 1 is shown the load settlement curve, obtained by field load test.

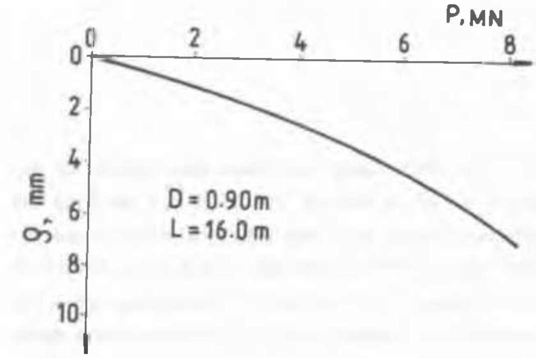

Fig.1 Load settlement curve; Pile 1

According to the cone penetration test, the average value of cone resistances R_p, from the surface to the depth of 7 m, is R_p = 3 MN/m² (CL-CI clay), in the layer between 7 m and 12 m the value of R_p is 5 MN/m² (CL-CI clay), and in the layer below the depth of 12 m (CI-CH clay) the average point resistance is R_p = 18 MN/m².

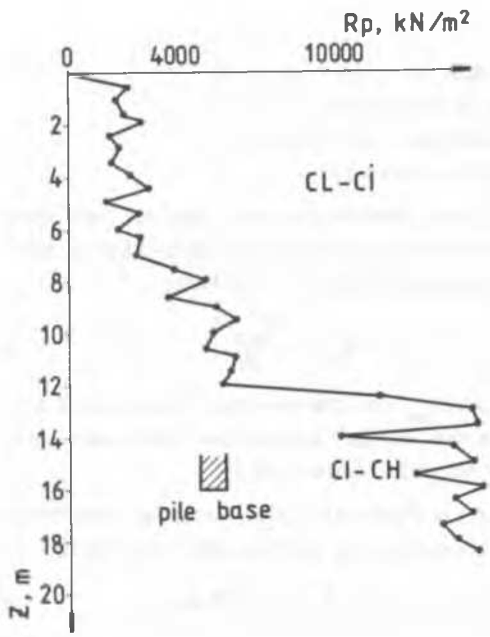

Fig.2 Penetration test results; Pile 2

Test pile 2 had a diameter D = 1,20 m and the length L = 18 m. In Fig. 3 is shown the load settlement curve, obtained by field load test.

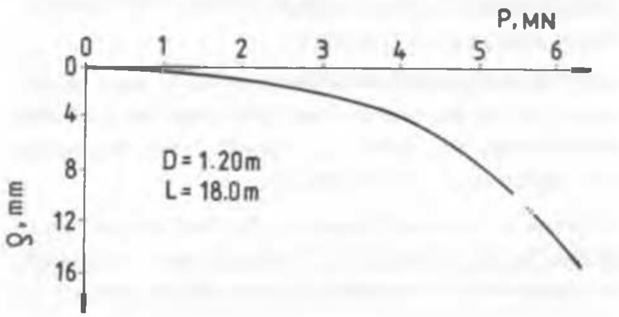

Fig.3 Load settlement curve; Pile 2

According to the cone penetration test results, the average value of cone resistance R_p, from the surface to the depth of 10 m, is R_p = 4 MN/m² (silty clay), in the layer between 10 m and 18 m the cone resistance increases, having the average value R_p = 8 MN/m², and the layer below the depth of 18 m the average point resistance is R_p = 10 MN/m².

Test pile 3 had a diameter D = 1,20 m and the length L = 22 m. In Fig. 4 is shown the load settlement curve, deduced from field load test.

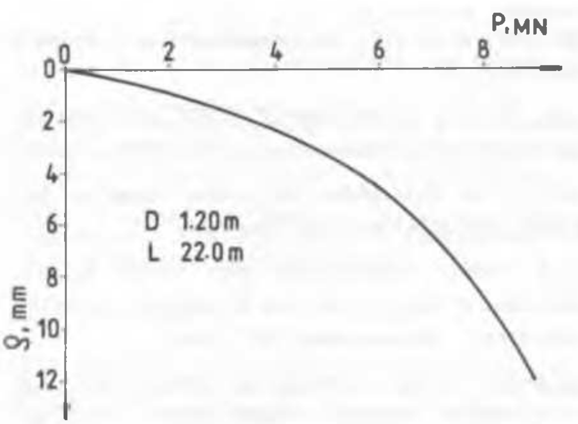

Fig.4 Load settlement curve; Pile 3

According to the static penetration test, performed in the civinity of a test pile, clay layer to the depth of 7 m has the cone penetration resistance R_p = 1,5 MN/m², the sand layer from 7 m to 9 m is with R_p = 5 MN/m², clay layer 13 m thick has R_p = 15 MN/m² and sand layer below pile base has R_p = 15 MN/m².

Test pile 4 and a diameter D = 1,5 m and the length L = 15 m. The load settlement curve, obtained by field load test, is shown in Fig. 5.

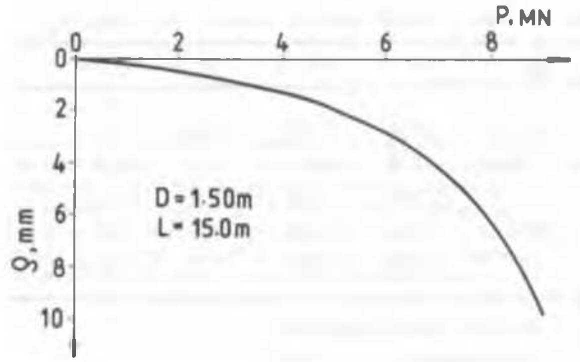

Fig.5 Load settlement curve; Pile 4

The point resistance of layer from 4 m to 10 m (silty) clay) is R_p = 3 MN/m², from 10 m to 19 m (fine to medium sand) is R_p = 12 MN/m² and below pile base (sandy gravel) is R_p = 14 MN/m², a shown in Fig. 6.

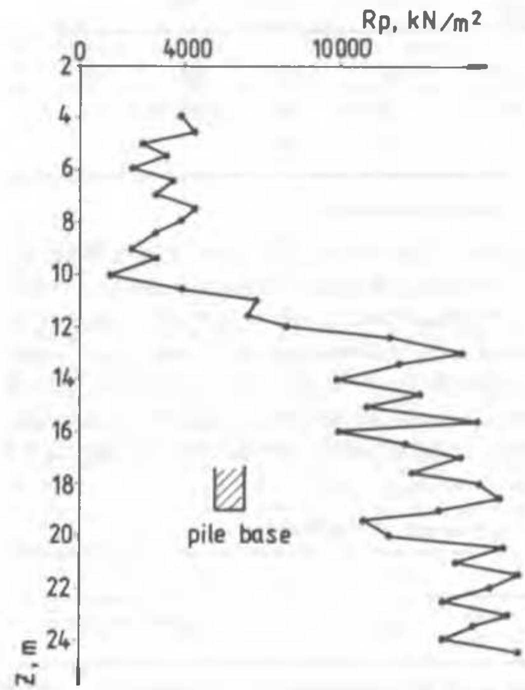

Fig.6 Penetration test results; Pile 4

ANALYSIS OF THE RESULTS

The ultimate bearing capacity of piles was calculated with the equations (2) and (3). Using the data obtained from field load test, the ultimate bearing capacity was determined by Van der Veen's and by Mazurkiewicz's procedure and also by bilinear approximation.The obtained values are presented in Table 1.

Table 1. The Ultimate Bearing Capacity of Piles, P_u

P_u MN	Pile 1	Pile 2	Pile 3	Pile 4
Eq.(2)	17,4	19,2	33,2	36,6
Eq.(3)	8,6	7,1	11,3	11,0
VV*	15,0	6,0	12,0	9,0
M**	13,0	5,6	12,5	9,3
B***	25,0	8,3	12,0	11,4

VV* = Van der Veen's procedure
M** = Mazurkiewicz's procedure
B*** = Bilinear approximation

The unit skin friction f_s was calculated using the equations (2) and (3). The obtained values are presented in Table 2.

Table 2. The Unit Skin Friction of Piles, f_s

f_s $\frac{KN}{m^2}$	Pile 1	Pile 2	Pile 3	Pile 4
Eq.(2)	132	116	196	168
Eq.(3)	89	39	54	51
B*	51	52		

B* = Bilinear approximation

On the basis of the registred values of settlement during field load tests and using the elastic solution for settlement, the equations of the type $E_s = \alpha R_k$ (where R_k is a function of pile stiffness factor K) and $E_s = \frac{E_p}{K}$ (where E_p is the modulus of elasticity of concrete have been obtained. The solution of these equations is carried out graphically. The values of the deformation modulus E are given in Table 3.

Table 3. Modulus of deformation E_s

	Pile 1	Pile 2	Pile 3	Pile 4
E_s, $\frac{MN}{m^2}$	190	190	166	270

In these cases the coefficient of correlation α, defined in eq.(10), varies between the limits α=10-19.

CONCLUSIONS

On the basis of the obtained results one may cay that the ultimate bearing capacity of piles can be determined using the penetration test results (Eq.3). However, the values obtained by Eq. (2) or by bilinear approximation are too high.

Coefficients K_s suggested by Mohan et al.is equal to 50, whereas the values deduced from field load tekst of piles varied between the limits K_s = 80-180 (for the average point resistance R_p = 7-10 MN/m²).

The values of the coefficient α, for the calculation of the modulus of deformation in terms of cone resistance, are higher than those usually used in the practice.

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