Bearing Capacity Equations of Static Sounding of Pliocene Clay

Equations d'Equilibre Limite de Pénétration Statique dans les Argiles du Pliocene

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SYNOPSIS The general bearing capacity equations for a plane horizontal strip foundation were adopted by numerous authors to interpret results of static sounding. This paper contains the evaluations of usefulness of this equation in determining changes of cone resistance in Pliocene clays. Glaciotectonical processes caused clay to have strong anisotropic properties. In this paper, a statistical evaluation of the properties of clay is presented, and comments are made on the causes of differences between the measured cone resistance and the calculated from bearing capacity equation.

INTRODUCTION

For a considerable area of Poland,particularly within Great Poland Lowland,subsoil for construction are Pliocene clays.These deposits were produced in the process of water sedimentations. The specific properties ofPliocene clays include:clay fraction content exceeding 50% and considerable swelling.Clay fraction belongs to illite-montmorillonite mineralogical type/coefficient of colloidal activity A=1.0/ The distinctive property of these clays is their high anisotropicity.As a result of glaciotectonical processes the structure of clay shows interbeddings of thickness not exceeding a few milimeters.Interbeddings,being continous wave lines,constitute silty sand and silt. Anisotropy of clay properties was one of the causes of failure for a retaining wall erected as part of a wharf near Poznań. In order to examine throughly the properties of subsoil,the Geoproject Company made 19 borings,15 static soundings and 18 dynamic-vane tests on an area of approximately 0.5ha;also 50 samples were taken for laboratory testing.Static sounding were made with a Gouda Penetrometer /RHS-12/. The cone,with an apex angle of 60°,diameter of 36mm and a section area F=10sq.cm,penetrated with a speed 0.025m/sec for every 0.10m.In an assigned program of examination,regular location was assumed according Lumb/1974/ in net of squares of borings and soundings.Shear parameters of clay were estimated on the basis of laboratory investigations and an attempt was made to identify these parameters by means of the value of cone resistance.In identification of cohesion and angle of internal friction we may use the bearing capacity equation and the procedure suggested by Durgunoglu and Mitchell /1973/.The main objects of this report are to verify the usefulness of this equation in evaluating the changes of cone resistance,and later the estimation of shear parameters of clay with attenuation surfaces.

STATISTICAL EVALUATION OF PROPERTIES OF CLAY

The examination of physical properties and shear parameters of clay were carried out in two laboratories.On the basis of a replication test and by means of statistics "F",it was observed that for the same standard samples of clay the variations ofthe clay's physical properties presented in Table I obtained in both laboratories did not differ.It was also recorded that the physical properties of clay do not depend on depth.Thus,the evaluation of each property's variations was based on a mean value and a significance level,which was calculated on basis of "t" distribution.

TABLE I

Statistical Estimation of Properties of Clay Layer

Property	$\bar{x} \pm S_{\bar{x}} \cdot t_{0.05}$		CV %
Specific gravity	19.4 ± 0.4	KN/m^3	4.9
Water content	27.0 ± 1.8	%	23.8
Plastic Limit	26.5 ± 1.8	%	18.1
Liquid Limit	83.4 ± 6.8	%	22.9
Clay content	64.8 ± 6.6	%	25.6
Undrained cohesion	88.5 ±12.5	kPa	39.5
Angle of internal friction	16.5 ± 2.6	deg.	43.2

The value of site coefficients of variation "CV" shows that the layer of clay is characterized by small variability of density,water content and consistence.The obtained result proves that the layer of clay is also characterized by small variability of strength.However,coefficients of variability show that the changeability of cohesion and angle of internal friction is high.This observation supports Lumb's /1971/ opinion that routine examination on standard samples can not be expected to give any useful information on the strength of soils belonging to the group of jointed soils.Also, the relation between cohesion and moisture content of clay has not been found /Fig.2/.Whereas occurence of such relation for other cohesion soils has been reported many times.

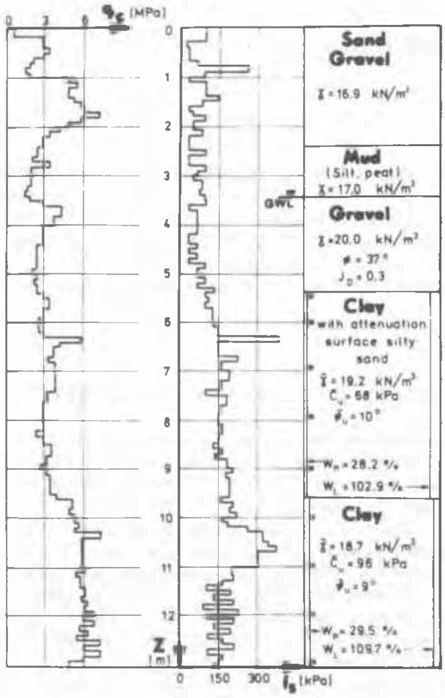

Fig.1 Typical Log of Subsoil with the Static Sounding Diagram

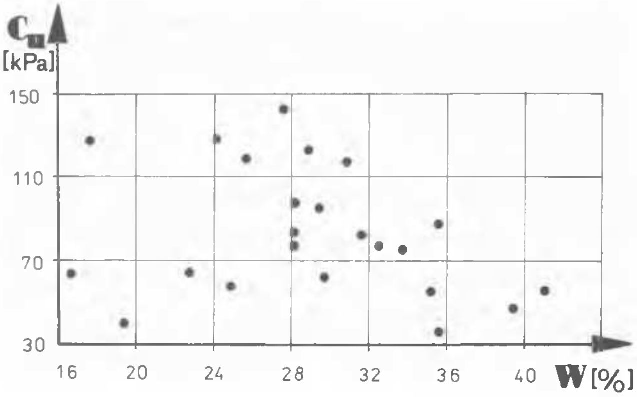

Fig.2 Relationship between Moisture Content and Cohesion

For homogenous layer of marine clay,Lumb /1974/ has determined a linear change of cohesion with depth. For an examined clay such a relation could not be found /Fig.2/.However,the trend of cone resistance to increase with depth has been established.From Fig.3 we can conclude however,that trend component and the standard deviation S_x of the random component can not be estimated from linear approximation. Fig.3 also proves that the static sounding method reflects the changes of the strength of clay depending on depth,which we could expect from changes of density and the consistence of clay

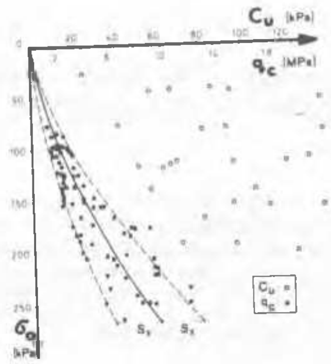

Fig.3 Cone Resistance and Cohesion against Depth /σ_o -overburden Pressure/

The cause for high values of the variability coefficient for cohesion and angle of internal friction is: the influence of the direction of attenuation surfaces, density and the kind of material constitutes the attenuation /Gown,1977 Młynarek,1979/.From Fig.4 we may conclude,that on the attenuation surface,the material can reach a boundary state earlier than the ratio of tangential stress to a vertical one is of maximum value. The possibility of dilatation forming outside shearing plane supports Drescher's /1974/ opinion on this problem. At the same time this fact has significant importance for the formation of failure areas below the cone

Fig.4 Sample Damage State after Direct Shear

ANALYSIS OF STATIC SOUNDING RESULTS

General relation determining cone resistance in a static sounding of cohesion soils is described in the following form /Młynarek,1978/

$$F(q_c, D, H, z, w, \gamma_d, \sigma_o, v_s, c_u, \varphi_u, \delta, \kappa_o, E_q) = 0 \quad /1/$$

where q_c-cone resistance, D-cone diameter, H-cone heigth, z-depth of sounding, w-moisture content γ_d-effective unit weigth of soil, σ_0-overburden pressure, v_s-speed of penetration, c_u-cohesion φ_u-angle of internal friction, δ-angle of friction between cone material and the soil, κ_0-coefficient depending on the degree of consolidation, E_q-modulus of deformation of the grains of soil.

The structure of the soil, the mineralogical composition of the soil and grain size distribution of the soil have also an influence on cone resistance.

Using a dimensional matrix and substituting R= F/U Kezdi, Młynarek/1980/ gave the following structural formula:

$$\frac{q_c}{\gamma_d D} = F_1\left(\frac{D}{H}, \frac{Z\sigma_0}{D\,c_u}, \frac{\delta}{\varphi_u}, \frac{\gamma_d R}{E_q}, w, v_s, \kappa_0\right) \qquad /2/$$

Equation /2/ is analogous to de Beer's /1974/ equation for cohesioless soils. In equation /2/ the shear parameters of soil are presented in evident form. It can be seen from equation/2/ that the bearing capacity equation used to calculate cone resistance is special cases of solving equation/2/.

Bearing capacity equation can be written in the following form:

$$q_f = c_u N_c \xi_c d_c + 1/2\, \gamma_d B N_\gamma d_\gamma \xi_\gamma + \gamma_d z N_q \xi_q d_q \qquad /3/$$

where ξ_c, ξ_γ, ξ_q there are shape factors for cohesion, friction and surchage terms, d_c, d_γ, d_q depth factors for cohesion, friction and surchage terms, B-breadth of foundation. The above mentioned factors and primary capacity factors N_c, N_γ, N_q were objects of investigations of numerous authors. The bearing capacity equation in which mechanism of failure /Fig.5/ and factors are taken to Vesic/Janbu, 1974/ and Durgunoglu /1973/ was adopted to interpret the changes of cone resistance with great success. It must be pointed out that the bearing capacity equation does not consider kinetic conditions, thus comparative analysis between calculated and measured cone resistance concerns constant and determined speed of penetration. Durgunoglu, Mitchell /1973/ expressed the unit cone resistance for cohesion-friction soils as follows:

$$\frac{q_c}{\gamma_d D} = \frac{c_u}{\gamma_d D} N_c \xi_c + N_{\gamma q} \xi_{\gamma q} \qquad /4/$$

at v_s=const D/H=const

$$\text{where } N_c, N_{\gamma q} = f_1, f_2\left(\varphi_u, \frac{\delta}{\varphi_u}, \alpha, \frac{z}{D}\right)$$

$$\xi_c, \xi_{\gamma q} = f_3, f_4\left(\varphi_u, \frac{z}{D}\right) \qquad /5/$$

Interpretation of changes in cone resistance of Pliocene clay was carried out on basis of equation which takes into consideration mechanism of failure and factors suggested by Durgunoglu-Mitchell, Vesic and Brinch Hansen. The detailed forms of equation and abounded information concerning a theoretical basis of sounding process are presented in the report of Durgunoglu and Mitchell/1973/. Process of iteration in Durgunoglu and Mitchell's method was carried out on Odra 1304 computer.

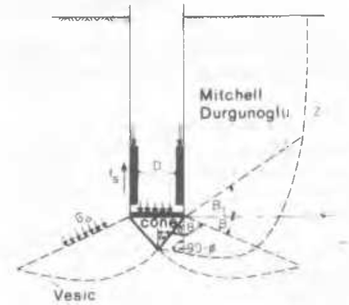

Fig.5 Compression and Shear Zones under Cone

For calculating the values of cone resistance the following basic data were assumed: mean shear parameters for the log shown on Fig.1, angle of friction δ, which was determined in direct shear apparatus. Angle of friction between cone material and clay varied from 2° to 6°. Second calculation was made on basis of mean values from population of shear parameters and compared with mean cone resistances-Fig.3

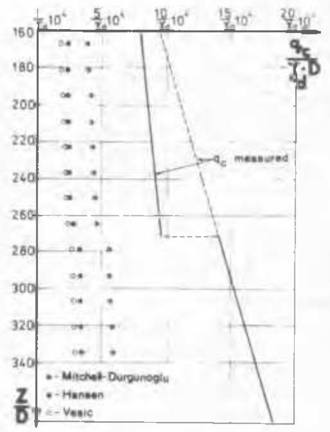

Fig.6 Measured and Calculated Cone Resistance /Log of the Subsoil Fig.1/

On Fig.6 the continuous line presents a generalized value of the measured cone resistance factor according to Janbu's /1974/ suggestion. From this figure we may conclude that measured values of cone resistance are higher than ones calculated from bearing capacity equation. Coefficient "m", which defines the relation between measured and calculated values of cone resistance changed accordingly with depth /Tab.II A better value of this coefficient was obtained on a basis of mean values from population of the values of cohesion, the angle of internal friction and mean values of cone resistance on a given level σ_0 /Fig.3/. The coefficient changed, compared to the one presented in Tab.II, for example, for dimensionless depth z/D=220 m=2.2/after Durgunoglu/, m=2.2/after Vesic/, m=1.4/after Brinch Hansen/.

Table II

The Coefficient " m" of Measured to Calculated Cone Resistance

Calculated Cone Resistance after:	Coefficient "m"			
	Dimensionless depth			
	160	220	280	340
Durgunoglu-Mitchell	3.5	3.7	4.3	4.8
Vesic	4.2	4.2	5.2	5.4
Brinch Hansen	2.1	2.1	2.6	2.8

According to the authors of this paper, two main factors note the differences between calculated and measured cone resistances:

/i/ Shear parameters defined on samples of soil in laboratory investigations are not representative for the layer of soil in situ. Generalizing these parameters in a statistical way requires a much bigger number of samples than for homogenous clays, and does not quaranted satisfactory results.

/ii/ Mechanism of failure below the cone in examined clay differs considerably from the mechanism which has been observed many times in sandy soils or homogenous clays. The possibility of dilatations forming outside the planes of maximum tangential stress shows that constructing classical slip surfaces /Fig.5/ for clays with attenuation surfaces may prove impossible and bearing capacity factors calculated on basis of angle of internal friction differ considerably from the real ones.

Because of noted differences between calculated and measured cone resistance, Durgunoglu and Mitchell's method could not be used to estimate shear parameters of clay.

For homogenous clays we use empirical relations which enable us to estimate some strength parameters of soil directly from the value of cone resistance. In case of Pliocene clays it is worth noting that the coefficient "α", which defines dependence between cone resistance and oedometric modulus, changed from 0.96 to 2.61. This modulus was calculated within the range of stresses σ_0 + 98.06kPa. It should be added that Sanglerat/1972,1977/ suggested the identical value of coefficient for speed of penetration of 0.02m/sec with Delf Penetrometer for clay having high content of clay fraction and cone resistances exceeding 2000kPa. Index $FR=q_c/f_s$ similarily supported classificational division of cohesion soils suggested by Sanglerat/1972,1979/. Due to the reasons presented earlier the empirical relation between cone resistance and cohesion has not been found. This index changed from 23 to 123, not exhibiting any functional relation. The mean value of this index was 50.

CONCLUSIONS

Examination of Pliocene clays with attenuation surfaces showed, that the results of shear parameters obtained from laboratory investigations are not satisfactory for interpretation of strength of the whole layer of clay in the subsoil. Even through there is a increase in numbers of soil samples for laboratory test, we should expect only small reduction of coefficient of site variation of shear parameters .Taking into account the high cost of such project, this method of achieving greater accuracy of estimation of shear parameters should be considered unsatisfactory. The useful method of estimating the strength of clay is static sounding. The knowledge of coefficient of friction ratio and cone resistance in determining speed of penetration, may be useful in classifying and determining the oedometric modulus.

The adopted bearing capacity equation enables us to foresee shear parameters for homogenous clays, but in case of investigated clays gives the results, which are of little practical importance. It seems necessary to carry out detailed investigation of evaluation of bearing capacity factors for clays with attenuation surfaces, because as Williams and Jennings/1977/ showed, even some field methods may give unsatisfactory results.

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