Determination of Pile Bearing Capacity By "In Situ" Tests

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Abstract - **Jablanica is located in the western part of Kosovo along the Drini river. For the purposes of population for this part, the government planned construction of the bridge over the river Drini river to allow movement of the population of the surrounding villages across the bridge on the main road Prishtina-Peja. For the foundations of the bridge over the Drini river were applied reinforced bored concrete piles. Bridge consists of two lateral spread footings on piles. Since the terrain where the bridge is supposed consist of layer of soft clay gray colour up to 15m depth, with variable characteristics. For this purpose were performed eight concrete pile length of 15m under foundations on both sides of the river. Piles are adopting the driven pile system Ø80mm. Based on the geotechnical soil parameters obtained from laboratory and field investigations, it is determined the load bearing capacity of two concrete driven piles and also were performed load-deformation charts for tests piles. Modulus of stiffness of clay layer is determined by field load test of pile. For this purpose, are used static penetration test. In order to compare the results, on the ground near the foundations of bridges are made two field pile load test, whereby are obtained results of bearing capacity for the field load test of piles.**

Key words: bridge abutment, pile, static penetration, bearing capacity, skin friction, field load test.

1. INTRODUCTION

For the foundations of the bridge over the Drini river were applied reinforced bored concrete piles. Bridge consists of two lateral spread footings on piles. Since the terrain where the bridge is supposed consist of layer of soft clay gray colour up to 15m depth, with variable characteristics. For this purpose were performed eight concrete pile length of 15m under foundations on both sides of the river. Piles are adopting the driven pilsistem Ø80mm. Based on the geotechnical soil parameters obtained from laboratory and field investigations, it is determined the load bearing capacity of two concrete driven piles and also were performed load-deformation charts for tests piles. Several "*In-situ"* tests can be use to obtain direct measurements of soil properties and geotechnical parameters. The common tests include: static penetration test, standard penetration (SPT), cone penetration test (CPT), flat dilatometer (DMT), pressuremeter (PMT), and vane shear (VST). Each test applies different loading schemes to measure the corresponding soil response in an attempt to evaluate material characteristics, such as strength and stiffness.
Modulus of stiffness of clay layer is determined by field pile load test. For this purpose, are used static penetration test. In order to compare the results, on the ground near the foundations of bridges are made two field load test of piles, whereby are obtained results of bearing capacity for the field load test of piles.

2. GEOLOGICAL DESCRIPTION OF INVESTIGATED AREA

Pliocene sediments - dominate in the region of Jablanica and presented in contact with a diabase-chert formation from Radavc to Vrella. The basic This sediments are represent by conglomerates, sands, sandy gravel sediments, sand and clay with lignite interlayer's. From drilling conducted in this locality are mostly silty clay dark gray colour with water content over 30%. Dark gray colour appears due to the presence of organic matters (fossils), some of which are predominant Viviparus and Dreissensia, under which was determined their age.
Quaternary deposits represent deposits updates on this region and have greater spread along the river terraces and are represented by sand and gravel.

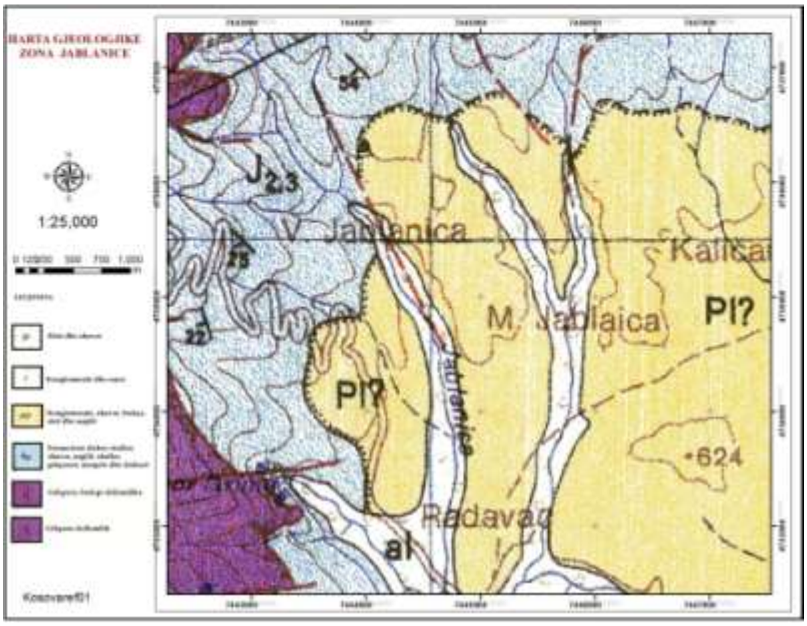


Fig. 1 Geology map of terrain

3. GEOTECHNICAL INVESTIGATION BY "IN SITU" TESTS

Static penetration test

In the field static penetration tests measure the resistance to penetration of the cone at its penetration in this under the influence of static force. Indentation force is applied by means of hydraulic presses, as well as the cons of cargo using more anchors. Through the hollow tube diameter 36mm freely moving steel rod diameter 15mm at the top of which there is a steel cone. The most common is 36mm diameter cone but is also used by 45mm. Figure 2 shows a cross section through the tube and cone penetrometer.

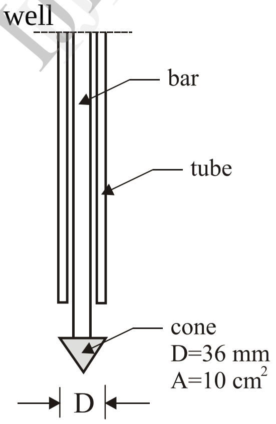


Figure 2. Detail of static penetrometer

Static penetration tests carried out in such a way that in the first stage using rods pressed only cone at a depth of 10cm. On this occasion, is measured the cone penetration resistance.

The second phase is injected just pipe to connect with cone and that when measured by lateral friction. The third phase is pressed cone and tube at the same time by a further 10 cm, so that the total sinking is about 20 cm. At this stage is measured the total penetration resistance. Figure 3 shows the phase of field tests during static penetration tests.

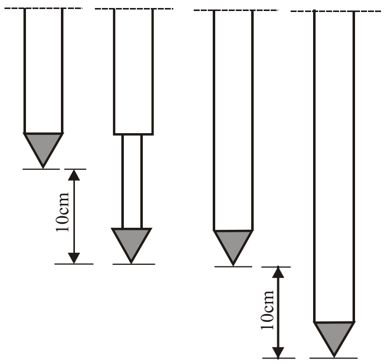


Figure 3. Phase of testing in the static penetration tests

Resistance to penetration cone R_p is given by the following expression:

$$R_p = \frac{P}{A} \qquad (kN/m^2) \tag{1}$$

where: P – impress force, A – area of cross section of cone.

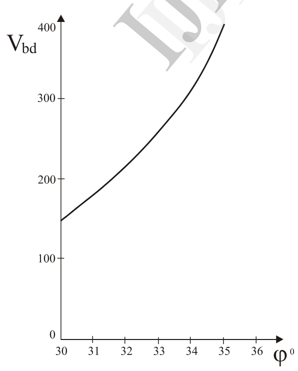
When the cone penetrates the soil around it remains in plastic state (rupture), and resistance to penetration of the cone can be written in the following form:

$$R_p = V_{bd} \cdot \sigma_o \tag{2}$$

$$V_{bd} = 1.2e^{2\pi tg\varphi} tg^2(45 + \varphi/2) \tag{3}$$

Where: σ_o is effective stress on the observed depth of the upper soil layers, φ - internal friction angle of soil. From these relationships can be through resistance to penetration cone determine the angle of internal friction φ. However, due to the influence of pore pressure that occur during penetration through coherent materials, this method can't be determined by the angle of friction φ for coherent layers.

Figure 4 shows the relationship between the coefficients V_{bd} and angle of internal friction φ.



V_{bd}	φ°
146.70	30
260.90	33
390.50	35
595.30	37
116.50	40

Figure 4. Relationship between the coefficients V_{bd} and angle of internal friction φ

4. PILE IN COHESION SOIL LAYER

Common procedure to calculate the limit load of piles consists in determining the capacity of pile base and bearing friction layer, Fig.5.

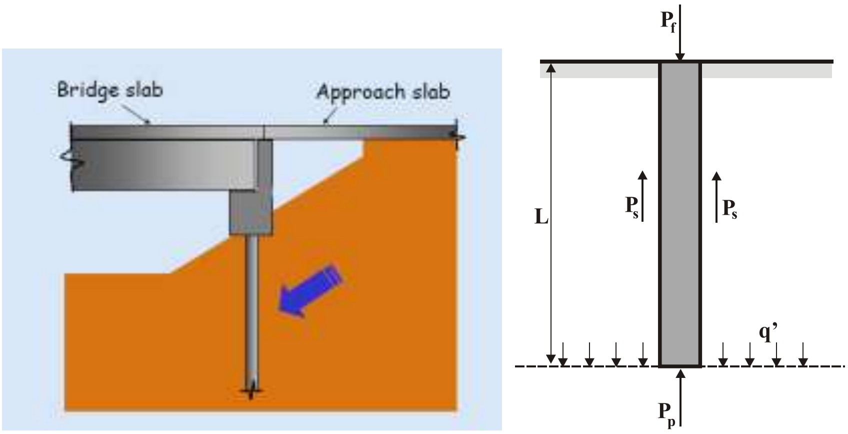


Fig. 5 Ultimate load-capacity of a pile

Static methods for the calculation of limit load can be written in the following form:

$$P_f = P_p + P_s = p \cdot A_p + f_s \cdot A_s \tag{4}$$

where: P - ultimate bearing capacity of the base of pile, A_p - sectional area base of pile, f_s - unite friction layer of pile,
A_{sk} – surface layer of pile.

The calculation of settlement of axial loaded the individual pile that the soil below the base of pile assumed as elastic, isotropic and homogeneous material. Function of linear variable displacement is given by expression:

$$\{u\} = |A| \cdot \{\alpha\} \tag{5}$$

After differentiation is obtained:

$$\{\varepsilon\} = |B| \cdot \{\alpha\} \tag{6}$$

General equation that expresses the relationship between stress and strain is given in the following form:

$$\{\sigma\} = |D| \cdot \{\varepsilon\} \tag{7}$$

Where D is matrix of stiffness.

Since the coefficients of influence without dimensions are calculated, settlement can be expressed by following simple expression:

$$s = \frac{p_o \cdot D}{E} \cdot \rho_o \cdot \rho_1 \cdot I_w \tag{8}$$

where: Po - contact stresses under the base of pile, D - diameter of pile base, E - stiffness of soil, ρ_o - factor that depends on the form of the load area, ρ_1 - factor that depends on the depth below the ground surface, I_w - coefficients of influence without dimensions which depends from thickness of deformable layer under loaded area.

To determine the stresses and strains under the base of the pile, the field load test of piles were carried load. The paper presents the results of field experiments load test on piles whose bases were performed in non-cohesive soils, as well as the results of field static penetration tests, performed in close examination of test piles. Magnitude of deformation of the soil were determined using the registered settlement of piles during load tests and the results of theoretical solutions for stresses and displacements of circular foundations, obtained by finite element method. Also, the paper presents the results of field experiments load test on piles, whose bases were carried out in a coherent materials. On the basis of registered resistance in the static penetration established the connection between specific skin friction and resistance to penetration of the cone.

5. LATERAL FRICTION IN CLAYS

Magnitude of specific skin friction in clay materials is often determined by parameters of shear resistance. Zaavaert (1960), Eide (1961), Chandler (1968) have suggested that the lateral friction calculated by using the effective stresses which prevailing in the soil:

$$f_s = K \cdot tg\varphi \cdot p_v^{'} \tag{9}$$

where:
K – coefficient of active earth pressure,

φ – effective internal friction of soil,

$p_v^{'}$ - vertical effective stress.

Broms and Hellman (1968) recommended the following expression for the calculation of skin friction of piles compressed in soil:

$$f_s = \alpha \cdot c_u \tag{10}$$

where:

c_u – undrained shear resistance of soil,

α – coefficient that depends on the undrained shear resistance of soil,

Vijayvergiya and Ficht (1972) also include undained shear resistance of soil in expression for determining lateral skin friction:

$$f_s = \lambda(p_v^{'} + 2s_u) \tag{11}$$

where λ - length of pile.

Burlan (1973) has suggested following expression for calculation of lateral skin friction:

$$f_s = \beta \cdot p_v^{'} \tag{12}$$

where β=0.25-0.40 - dimensionless coefficient determined from field load tests of piles.

Lateral skin friction of pile at clay deposit, according to Meyerhof's (1956, 1976) results of a survey, can approximately determine the empirical relationship between soil resistance measured by static or dynamic penetration experiments and analysis of pile load tests. On the basis of these studying Meyerhof recommends the following terms of the size of skin friction:

$$f_s = \frac{\overline{N}}{50}; \qquad f_s = \frac{R_p}{100} \tag{13}$$

where:

$\overline{N}$ - the number of blows for one foot penetration in the dynamic penetration,

R_p - resistance to penetration of the cone at the static penetration.

Meyerhof (1976) also indicated that the average unit frictional resistance, f_{sv}, for high-displacement driven piles may be obtained from average standard penetration resistance values as:

$$f_{sv} = 2\overline{N}_{60} \tag{14}$$

Where $\overline{N}_{60}$ average value of standard penetration resistance. For low-displacement driven pile

$$f_{sv} = \overline{N}_{60} \tag{15}$$

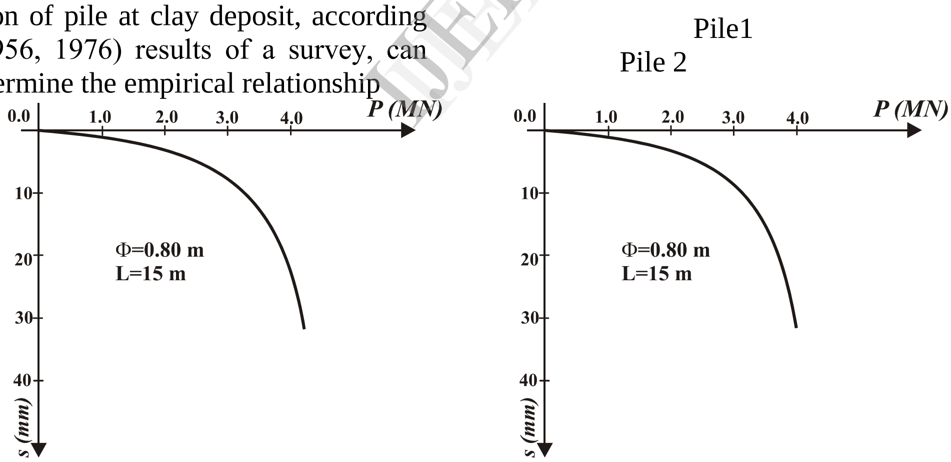


Fig. 6 Loading – unloading curve for pile No.1 and 2

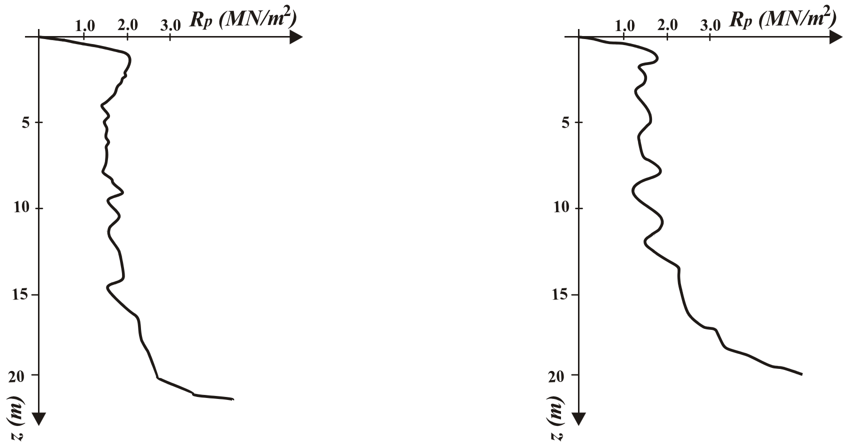


Fig. 7 Diagram of static penetration for piles No. 1 & 2

Fig.6 shows the results of field experiments driven concrete pile load test diameter Ø800mm. Results of field experiments of static penetration, performed close examination of test piles, are shown in Fig.7. Size of skin friction is determined based on the results of laboratory tests (12) and the results of field test of penetration (13, 14). Separation of the total capacity of pile on point load capacity and load capacity by friction along the skin of pile was carried out according to the Van Weele procedure. From the results, the size of the friction force calculated by equations (14) best agrees with the values of friction force obtained through the Van Weele procedure.

6. MODULUS OF DEFORMATION OF COHESIVE SOIL

Considering that to non-cohesive materials can't be obtained undisturbed samples modulus of deformation of these layers can be determined by field pile load test. For this purpose, it is often used static penetration test. Most authors show linear dependence of deformation and resistance of cone penetration. So De Beer (1956) recommends the following expression:

$$E = 1.5R_p \tag{16}$$

Meyerhof and Schmertmann (1970) in a similar way determine the size of deformation:

$$E = 2.0R_p \tag{17}$$

Based on the experiment of load tests, performed on several piles, Van Welle (1956) came to the following dependence of deformation and resistance to penetration cone:

$$\frac{E}{(1-\mu)^2} = 60R_p \tag{18}$$

Thomas (1968), had concluded that there is a connection between the modulus of deformation and penetration resistance:

$$E = (3-12)R_p \tag{19}$$

while Trofimenkov (1974) recommends the following expression for the calculation of modulus of deformation for clayey materials, respectively:

$$E = 4.9R_p + 123 \tag{20}$$

Based on the analysis of results of field load tests on piles Poulos (1979) suggests following limits for the calculation of modulus of deformation for clayey materials, respectively:

$$E = (10-40)R_p \tag{21}$$

All of the expressions assume a linear relationship of modulus of deformation and resistance of cone penetration.

The relationship between the resistance to penetration of the cone R_p and compressibility index C_c is established by Buisman (1948) and expressed as follows:

$$C_c = 1.5\frac{R_p}{\sigma_o} = 1.5 \cdot V_{bd} \tag{22}$$

It should be noted that this particular size of coefficient Cc should be applied with caution.

Based on the results of two field load test on piles, whose bases were performed in cohesive soils, as well as on the basis of data from field tests of static penetration, is analyzed the dependencies between modulus of deformation and penetration resistance.

Results shows the magnitude of modulus of deformation E, determined by means of the above expression and using data obtained from field test load on piles are in good agreement.

7. CONCLUSION

Based on obtained results we can conclude that the size of the unit skin friction can be determined using equation (12). The values of f_s determined by the expression of Burland are significantly higher than those determined using data from a field load test of pile, while the values determined by the expression of Meyerhof are on the side of safety.

It can also be concluded that the procedures that are often used in practice (equations 16, 17, 19, 20), we get too low values of modulus of deformation, and their realistic value can be determined according to the expression (21).

8. REFERENCE

- Bowels, J.E. (1997): Foundation Analysis and Design, The McGraw-Hill Companies, Inc.
- Broms, B. And Hellman L. (1968): „End bearing and skin friction resistance of piles“. Journal of the Soil Mech.and Found. Div. 94, pp.421-429.
- Burland, J.F. (1973) „Shaft friction of piles in Clay, a simple foundamental approach“ Ground
- Eng. 6, pp.30-42.
- Das, B.M. (2010): Principles of Geotechnical Engineering, Seventh Edition.
- Murthy, V.N.S. (2000): Geotechnical Engineering-Principles and practices of soil mechanics and foundation engineering,
- Meyerhof, G.G. (1956): „Penetration tests and bearing capacity of cohesive soils“. Ame. Soc.
- Civ. Eng. Proc. 82.
- Meyerhof, G.G. (1956): „Bearing capacity and settlement of pile foundation“. Journal of Geot.
- Eng. Div. 102, pp. 197-228.
- Poulos, H.G., Davis, E.H., (1980): Pile foundation analysis and design, Jon Wiley & Sons, New York
- Poulos, H.G., (1979): „Settlement of single pile in nonhomogenious soil“, Journal of Geot. Eng. Div. 195.
- Schmertmann, J.H. (1970): „Static cone to copute static settlement over sand“. Journal of Soil
- Mech. Found. Div. 96, pp. 1011-1043.