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Determination of Soil Properties Using 'In Situ" Tests for Tank Design

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Abstract

Prizren Municipality for their purposes plans to build WWPT at the location Vlasnje 5km away from the Prizren Town. The complex consists of eight opened and closed reservoirs diameter D = 9.5m. Reservoir R1 and R6 should fund on the sandy gravel, while reservoirs R7 and R8 on sandy soil.
Considering that to non-cohesive materials cannot be obtained undisturbed samples modulus of deformation of these layers can be determined by field load test such as penetration test, plate load test, SPT. In order for these purposes have been carried out field tests on the level of the foundation. For the foundations R1 to R6, whose base lies on the gravelly sand have been carried out at the foundation depth the plate load tests, while at every 1.5 to 2.0m under all the above aforementioned foundations have been carried out SPT tests. For calculation of settlement of foundations R7 and R8, which lying on the sandy soil, are used the results from the static penetration tests.
Results of all tests are shown in diagrams and tables.
Key words: *Static penetration, SPT test, Plate load test, coefficient of sub grade reaction, bearing capacity*

1. Introduction

Several in-situ tests define the geostratigraphy and obtain direct measurements of soil properties and geotechnical parameters. The common tests include: standard penetration (SPT), cone penetration test (CPT), piezocone (CPTu), 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/or stiffness.
Boreholes are required for conducting the SPT and normal versions of the PMT and VST. A rotary drilling rig and crew are essential for these tests. In the case of the CPT, CPTU, and DMT, no boreholes are needed, thus termed "direct-push" technologies. Specialized versions of the PMT (i.e., full-displacement type) and VST can be conducted without boreholes. As such, these may be conducted using either standard drill rigs or mobile hydraulic systems (cone trucks) in order to directly push the probes to the required test depths. Figure 5-2 shows examples of the truck-mounted and track-mounted systems used for production penetration testing. The enclosed cabins permit the on-time scheduling of in-situ testing during any type of weather. A disadvantage of direct-push methods is that hard cemented layers and bedrock will prevent further penetration. In such cases, borehole methods prevail as they may advance by coring or noncoring techniques. An advantage of direct-push soundings is that no cuttings or spoil are generated.

The modulus of subgrade reaction is a conceptual relationship between soil pressure and deflection that is widely used in the structural analysis of foundation members. It is used for continues footings, mats and various types of piling.

The standard penetration test is currently the most popular and economical means to obtain subsurface information. The SPT results have been used in correlations for unit weight (γ), relative density (Dr), angle of internal friction φ, and undrained compressive strength qu. It has also been used to estimate the bearing capacity of foundations and for estimating the stress-strain modulus 'Es'. In this paper the results of SPT and PLT tests on gravely soils of Prizeren alluvium are evaluated. Due to the results obtained the correlation between SPT results $(N_1)_{60}$ and modulus of subgrade reaction (Ks) are also presented for gravely soils. But results od penetration tests are evaluated for sandy soil.

2. Geological description of investigated area

In the administrative aspect, the investigated area belongs to the Municipality of Prizren. It is bounded on the north by Vllashnje in the west with Gradishte, southwest with Bezhaninë and in the east with Rikavec. Geological location of investigated area is mainly composed by new Quaternary deposits, represented by: proluvium, delluvium, alluvium and river terraces (Fig. 1. Geological map of investigated area).

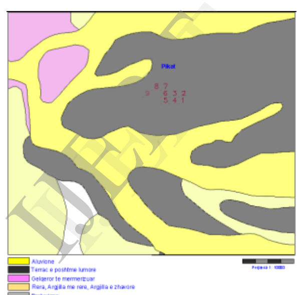

Figure 1. Geological map of investigated area

Quaternary (Q): Quaternary deposits have spread considerably, especially in the southern part of the basin of Prizren. Quaternary deposits are represented by different genetic types, generated in Pleistocene and Holocen.

Pleistocene

Pleistocene formations are represented by lacustrine deposits (clastics sediments): conglomerates, breccias, red sand and river terraces deposits.

Lacustrine sediments (clastic sediments)

In the basin of Prizren, Quaternary deposits, thickness of 20-60 m, represented by: gravel-sandy clayey yellowish brown color, sand and gravels which alternated in horizontal and vertical direction. Gravels are composed by different types of rocks which build the basin region. They have greater spread along the banks of the Drini river between Prizren and Vlashnje where they meet isolated in the river terraces represented by: sandy conglomerates (second river terrace) and conglomeratic sandstone (first river terrace).

Age of Quaternary sediments in the basin of Prizren is mainly determined on the base of their position in the cutting and relationships with older deposits with known age. In terrain these sediments lie over upper Pliocene shown paleontologically in the context of isolated areas, at an altitude of 350-400 m. Quaternary deposits are represented with colors from yellow - brown to red. Candona ex gr. Rostrata are found by fauna.

River terraces **(t_1, t_2).** In the Prizren plains are distinguished two river terraces which are composed by sand and gravels.

First River terrace (t_1)

First river terrace**,** thickness from 10 – 20 m, is located in the Toplluha plains (north-easter part of Prizren) around the Lumbardhi River and Drini i Bardhe River near Rogova village.

Second River terrace (t_2).

Second river terrace, thickness from 20 – 30 m, is located in the both side of Drini i Bardhe river.

Holocene

Holocene formations, also occupy considerable area, which meet in the southeastern part of Prizren region near to Vermica village. They are represented by: proluvium, deluvium, alluvium and “terra rosa”.

Proluvium sediments – lies in the western part of Prizren municipality around Vlashnje locality.

Alluvium **–** lies along the Drini i Bardhe river, which are represented by: mud, sand and gravel.

3. Geotecnical investigation by *“In Situ”* tests

3.1 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.

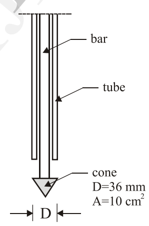

Figure 2. Detail of static penetrometer

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.

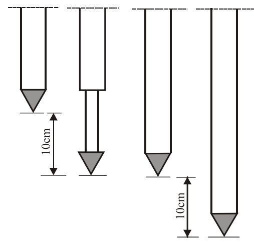

Figure 3. Phase of testing in the static penetration tests

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:

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

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

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 not 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.

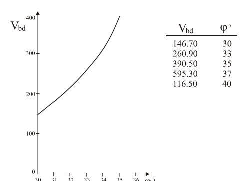

Figure 4. Relationship between the coefficients V_{bd} and angle of internal friction φ

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:

$$C_c = 1.5 \frac{R_p}{\sigma_o} = 1.5 \cdot V_{bd} \tag{4}$$

It should be noted that this particular size of coefficient Cc should be applied with caution.
In the literature can be found tables showing the relationship between the resistance to penetration of the cone R_p and relative density D_r, index konsistencije I_c and uniaxial strength q_u.

According to Buisman (1935) there is a certain relationship between V_{bd} cone penetration resistance and compression modulus M_v noncohesive material:

$$M_v = \frac{3}{2} V_{bd}, \quad or \quad M_v = 1.5 V_{bd} \tag{5}$$

De Beer (1948), studying the calculated and actual settlement of the bridges on the sand, proposed the following expression:

$$M_v = 3V_{bd} \tag{6}$$

Vesić (1970) found that the modulus of compressibility depends on the relative density I_D, and has established a relationship:

$$M_v = 2(1 + I_D) \cdot V_{bd} \tag{7}$$

whereby for completely loose sand stems $M_v = 2V_{bd}$ and for compacted $M_v = 4V_{bd}$.

Parallel examination of static penetration test and SPT was established in Belgium following correlations for sands materials:

$$\begin{aligned} V_{bd} &= 400N(kN/m^2) \text{ in noncohesive material,} \\ V_{bd} &= 200N(kN/m^2) \text{ in cohesive material,} \\ V_{bd} &= (800-1000)N(kN/m^2) \text{ in gravely material.} \end{aligned} \tag{8}$$

where: N – number of blows at SPT.

On the basis of data in the literature Šuklje proposed expression:

$$M_v = C_1 + C_2 N, \tag{9}$$

where: $2000 < C_1 < 4000$ and $400 < C_2 < 800$.

3.2 Standard Penetration Test (SPT)

The standard penetration test (SPT) is performed during the advancement of a soil boring to obtain an approximate measure of the dynamic soil resistance, as well as a disturbed drive sample (split barrel type). The test was introduced by the Raymond Pile Company in 1902 and remains today as the most common in-situ test worldwide. The procedures for the SPT are detailed in ASTM D 1586 and AASHTO T-206.

The SPT involves the driving of a hollow thick-walled tube into the ground and measuring the number of blows to advance the split-barrel sampler a vertical distance of 300 mm (1 foot). A drop weight system is used for the pounding where a 63.5 kg hammer repeatedly falls from 0.76 m (30 inches) to achieve three successive increments of 150 mm (6-inches) each. The first increment is recorded as a “seating”, while the number of blows to advance the second and third increments are summed to give the N-value ("blow count") or SPT-resistance (reported in blows/0.3 m). If the sampler cannot be driven 450 mm, the number of blows per each 150 mm increment and per each partial increment is recorded on the boring log. For partial increments, the depth of penetration is recorded in addition to the number of blows. The test can be performed in a wide variety of soil types, as well as weak rocks, yet is not particularly useful in the characterization of gravel deposits nor soft clays. The fact that the test provides both a sample and a number is useful, yet problematic, as one cannot do two things well at the same time.

The SPT is conducted at the bottom of a soil boring that has been prepared using either flight augers or rotary wash drilling methods. At regular depth intervals, the drilling process is interrupted to perform the SPT. Generally, tests are taken every 1.5 m at depths shallower than 7 meters and at intervals of 2.0 m thereafter. The head of water in the borehole must be maintained at or above the ambient groundwater level to avoid inflow of water and borehole instability.

Various factors like effective overburden pressure (σ'_v), length of drilling rod, diameter of borehole, method of sampling and type of hammer used, influence the SPT below counts. Thus in order to determine a suitable index for evaluating soil density and resistance, researchers have suggested different corrections on the basic number of blows obtained in SPT tests. The most comprehensive relations are presented in the NCEER-97 report for granular soils. In this procedure the corrected SPT below count for the 60% energy level is obtained from the following relation:

$$(N_1)_{60} = N * C_N * C_E * C_B * C_R * C_S \qquad (10)$$

where:

N: measured SPT below counts

C_E: Energy effect coefficient depending on hammer type

C_B: Correction factor for borehole diameter

C_R: Correction factor for rod length

C_S: Correction factor for type of samplers (with or without liners)

C_N: Effective overburden pressure coefficient obtained from the following relation:

$$C_N = (Pa / \sigma'_{v0})^{0.5} \qquad (11)$$

where:

Pa: Atmospheric pressure

σ'_{v0}: Effective vertical pressure at considered depth

The preferred correction factors are presented in Table 1 due to NCEER recommendation.

Table 1. Correction to SPT results (NCEER-97)

Factor	Equipment variable	Term	Correction
Overburden pressure		C_N	(Pa / σ'_{v0})
			$C_N < 2$
Energy ratio	Donut Hammer	C_E	0.5 to 1.0
	Safety Hammer		0.7 to 1.2
	Automatic Hammer		0.8 to 1.3
Borehole diameter	65 mm to 115 mm	C_B	1
	150 mm		1.05
	200 mm		1.15
Rod length	3 m to 4 m	C_R	0.75
	4 m to 6 m		0.85
	6 m to 10 m		0.95
	10 to 30 m		1
	> 30 m		<1.0
Sampling method	Standard sampler	C_s	1
	Sampler without liners		1.1 to 1.3

3.3 Plate load test

Foundation investigation in the northern part of Prizren town is often conducted by digging narrow test pits (up to 100 cm across). Trained personnel are lowered in to such openings to describe and test the soil profile, this includes relative density. The large diameter boreholes make routine on site testing of gravels practicable even at depth. Therefore, equipment was developed that could be set up inside an 100cm diameter borehole to perform vertical plate load tests at the bottom of shallow boreholes. These tests can be performed during routine foundation

investigations for little additional cost. Because of the variability of gravel properties, frequent testing at each site is desirable. For that reason, and because of the confined working space in the borehole, the dimensions and mass of the equipment developed was kept small. The equipment consists of a light hydraulic jack with attachments to connect circular plate with 305 mm diameter, Figure 6. The test procedure is coincident with ASTM D1194. The plate is placed at the centre of the hole. Load is applied to the plate in steps-about one-fourth to one-fifth of the estimated ultimate load by means of a jack. During each step load application, the settlement of the plate is observed on dial gauges. At least one hour is allowed to elapse between each load application step. The test have been conducted until failure, or at least until the plate has gone through 25 mm of settlement.

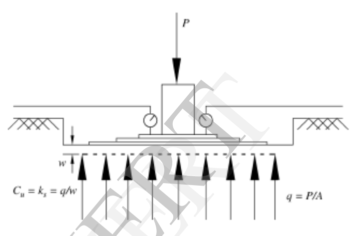

Figure 6. Plate bearing test apparatuses

The idea of modeling soil as an elastic medium was first introduced by Winkler and, not surprisingly, this principle is now referred to as the Winkler soil model. Several application of this principle are considered and illustrated by means of worked examples such as laterally loaded pile, and settlement analysis of shallow flexible foundations. Perhaps the best known are is that of a continuous, horizontal beam or footing resting on an elastic sub grade. The sub grade reaction at any point along the beam is assumed to be proportional to the vertical displacement of the beam at that point. In other word, the soil is assumed to obey Hook's law. Hence, the modulus of sub grade reaction (Ks) for the soil is given by $Ks=P/\delta$ where P is the ground bearing pressure at a point along the beam, and δ is the vertical displacement of the beam at that point.

The main difficulty in applying the Winkler soil model is that of quantifying the modulus of sub grade reaction (Ks) to be used in the analysis, as soil can be a very variable material. In practical and realistic terms, Ks, can be found only by carrying out in-situ plate bearing tests or relating it in some way to intrinsic deformation characteristics of the soil. The plate bearing test is widely used and is fully described in ASTM D1194. In foundation design, as distinct from pavement design, the value of Ks is the secant modulus of the graph over the estimated working range of bearing pressure (p') as indicated in Figure 7.

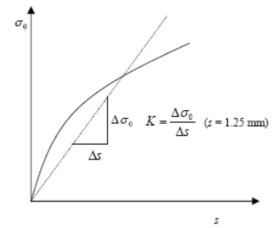

Figure 7. Ks measurement of plate load test results

Due to lack of enough data about plate load test, only empirical and theoretical relations are taken into consideration. Various relations of K_s have been proposed by researches and some of them are represented in Table 1; wherein, Es = modulus of elasticity, υ_s = Poisson's ratio, B = width of footing, EI = flexural rigidity of footing, k_{s1} = the coefficient of subgrade reaction for a plate 30.5cm wide, μ = non-dimensional soil mass per unit length, B' = least lateral dimension of footing, I_S and I_F = influence factors which depend on the shape of footing and parameter m takes 1, 2 and 4 for edges, sides and center of footing, respectively. Among these methods, approaches 1 and 5 are utilized more than the others.

Table 1. Common relations suggested for K_s

No.	Investigator	Suggested expression
1	Biot	$k_s = \frac{0.95 E_s}{B(1-\nu_s^2)}[\frac{B^4 E_s}{(1-\nu_s^2)EI}]^{0.108}$
2	Terzaghi	For sands $k_s = k_{s1}(\frac{B+1}{2B})^2$ For clays $k_s = k_{s1}\frac{1}{B}$
3	Vlassov	$k_s = \frac{E_s(1-\nu_s)}{(1+\nu_s)(1-2\nu_s)}(\frac{\mu}{2B})$
4	Vesic	$k_s = \frac{0.65 E_s}{B(1-\nu_s^2)}\sqrt[12]{\frac{E_s B^4}{EI}}$
5	Meyerhof and Baike	$k_s = \frac{E_s}{B(1-\nu_s^2)}$
6	Klopple and Glock	$k_s = \frac{2E_s}{B(1+\nu_s)}$
7	Selvadurai	$k_s = \frac{0.65}{B}.\frac{E_s}{1-\nu_s^2}$
8	---	$k_s = \frac{E_s}{B'(1-\nu_s^2)mI_sI_F}$

Calculation of modulus of subgrade reaction: The modulus of subgrade reaction, K_s, in MN/m^3, shall be calculated using the following equation:

$$K_s = \frac{\sigma_0}{s}, \quad s = 1.25mm \tag{12}$$

where

σ_0 - is the average normal stress, in MN/m^2;

s - is the settlement of the loading plate, in m.

The modulus of subgrade reaction for use in the design of road and airfield pavements shall be calculated from the normal stress, σ_0 , corresponding to an average settlement of 1,25 mm, figure 7

4. Test result from site investigation

4.1 Static penetration test

Field static penetration tests were carried out investigations on the sandy terrain and obtained diagram R_p - z. The circular foundation of diameter D = 10m is founded in the layer of sand considerable thickness at a depth of D_f = 1.0m. Field research was detected groundwater level at a depth of 4.0 m Bulk density of sand above the groundwater level is $\gamma = 17.15 kN/m^3$, and below the water level is $\gamma^{'} = 10.0 kN/m^3$. Results of field experiments of static penetration, performed close examination of test piles, are shown in figure 8.

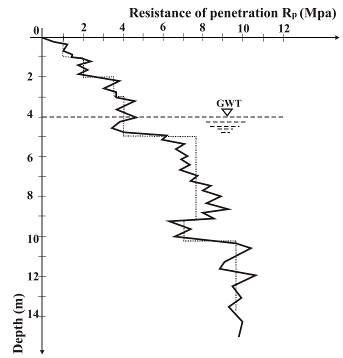

Figure 8. Diagram of static penetration for foundation No. 7 & 8

According to value of static penetration is determined the settlement of the circular foundation.
Settlement of the central point of foundation were determined by Busman - De Beer's expression:

$$s = \int_0^H \frac{1}{C} \cdot \ln\left(\frac{p_0^{'} + \Delta\sigma_z}{p_0^{'}}\right) dz = \sum_0^H \left(\frac{1}{C} \cdot \ln\frac{p_0^{'} + \Delta\sigma_z}{p_0^{'}}\right) \cdot \Delta z \qquad (13)$$

where C is constant of compressibility,

$$C = 1.5 \cdot \frac{R_p}{p_0^{'}} \qquad (14)$$

Based on the above equation is derived subsidence s = 7.82 cm.

4.2 Standard Penetration Test (SPT)

The standard penetration tests are also performed in each site and the SPT below counts are measured in corresponding depths. Then the corrected SPT values $(N_1)_{60}$ are evaluated based on NCCER-97 (1997) procedure. Based on most applicability of standard penetration test in geotechnical projects in Kosovo, the presentation of relationship between SPT results and other geotechnical properties of gravel such as Ks would be very useful. The obtained corrected SPT results $(N_1)_{60}$ and modulus of subgrade reaction (Ks) data are presented in Table 2.

Table 2. Obtained $(N_1)_{60}$ and Ks data

Foundation No.	*USCS Classification*	$(N_1)_{60}$	K_s $(x10^4\ kN/m^3)$

1	GW-GC	38	20
2	GW-GC	36	18
3	GW	44	22
4	GW-GC	39	21
5	GW-GC	35	18
6	GP	33	17

The allowable bearing capacity is etimated by means of modulus of subgrade reaction Ks from relationship reported by Bowles:

$$K_s = 40 \cdot F_s \cdot q_a \tag{15}$$

where q_a is allowable bearing capacity in kN/m^2 and F_s is the safety factor taken as 3.
The allowable bearing capacity is shown in table 3.

Table 3. Bearing Capacity Values by Field Tests

Foundation No.	*Field plate load test*	
	K_s ($x10^3$ kN/m^3)	*q_a (kN/m^2)*
1	20	416.66
3	22	458.33
6	17	354.17

where: Ks =160 × 3× q_{all}. (Field) for max. ΔH= 6 mm (According equation 15)

4.3 Plate load test

10 vertical plate load tests are performed on medium to dense gravely soils in Prizren alluvium. The vertical settlement (δ) and contact pressure (p) for each test were measured and plotted in figures such as Figure 9. Then the secant modulus of each graph (K_s) is determined.

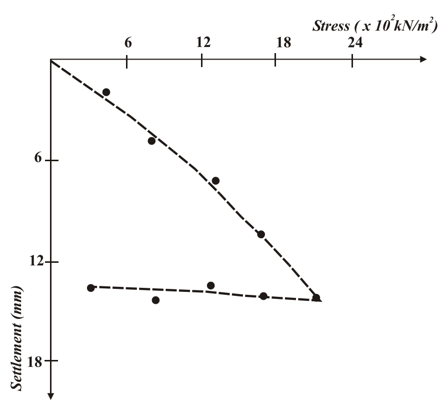

Figure 9. Typical (p-δ) curve obtained from plate load test

5. Conclusion

The standard penetration test and plate load test results could be correlated in medium to dense gravely soils.
The modulus of subgrade reaction (Ks) of medium to dense gravely soil are correlated with corrected SPT below counts (N1)60.
The modulus of subgrade reaction (Ks) of medium to dense gravely soils are increased with increasing the corrected SPT below counts (N1)60.
Plate bearing test can be use to predict bearing capacity of soil layer for shallow layer depth to avoid using comprehensive soil investigation and economizing both cost and time.
Plate bearing test could be used as alternative methods when the full investigations unavailable.

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