BULETINUL INSTITUTULUI POLITEHNIC DIN IAŞI
Publicat de
Universitatea Tehnică „Gheorghe Asachi” din Iaşi
Tomul LVIII (LXII), Fasc. 3, 2012
Secţia
CONSTRUCŢII. ARHITECTURĂ

USE OF CONE PENETRATION TESTS AND CONE PENETRATION TESTS WITH POREWATER PRESSURE MEASUREMENT FOR DIFFICULT SOILS PROFILING

BY

ADRIAN-TRAIAN ILIESI[1,*], ANA-LUCIANA TOFAN[1] and DIEGO LO PRESTI[2]

[1] "Gheorghe Asachi" Technical University of Iaşi
Faculty of Civil Engineering and Building Services
[2] University of Pisa, Department of Civil Engineering, Italy

Received: July 9, 2012
Accepted for publication: September 28, 2012

Abstract. As a support for a future construction, geotechnical soil characterization is necessary by laboratory testing or *in situ* probing methods for optimal choice of foundation system, and for avoiding that the soil foundation will not reach the ultimate limit states or the serviceability limit states.

Most of *in situ* methods of probing the soil foundation are made discontinuously by meter to meter, having the disadvantage that cannot provide geotechnical information for the entire depth of the investigated area. Cone Penetration Tests (CPT) and cone penetration tests with pore water pressure measurement (CPTu), are among the only tests that provide accurate information about the stratigraphic limits, lithologic anomalies and soils type as a result of continuous records of the geotechnical parameters of the foundation soil for the entire study area.

Key words: CPT; CPTu; *in situ* tests; stratigraphic limits.

---

[*]Corresponding author: *e-mail*: iliesi_adrian_traian@yahoo.com

1. Introduction

Cone Penetration Tests (CPT) have been used for more than 40 years. The equipment was designed by the Dutch Department of Public Works in 1930, and in 1932 P. Barentsen used it for the first time to verify a 4 m thick filling (Mayne, 2007). The original machine had two sets of pipes, external protecting inner cone connected to a manometer, where readings were made visually by an operator at each 20 cm advance / push.

Thereafter, in 1948 Delft Soil Mechanics Laboratory (DSML) introduced the electric cone, which was operated by a continuous push at a steady speed, allowing continuous measurements of pressure. A year later the system was tested by measuring both cone tip resistance and sleeve friction, but it only became commercial in 1960. In 1962 DSML introduced the piezocone, which initially was used only in sands, and after 1970, various authors used piezocone successfully in cohesive rocks, soft and layered soils, particularly those that are highly stratified.

An inclinometer was incorporated to detect deviations from verticality and thus offer a warning to the user against excessive slope and/or buckling problems (Van De Graaf & Jekel, 1982).

Over the past three decades, a number of other sensors or devices have been installed within the penetrometers (Fig. 1), including: temperature, electrodes, geophones, stress cells, fulldisplacement pressuremeters, vibrators, radio-isotope detectors for density and water content determination, microphones for monitoring acoustical sounds, and dielectric and permittivity measurements (Jamiolkowski, 1995).

More recently, electronic systems have become available that contain the signal conditioning, amplification, and digital output directly within the penetrometer downhole. With digital cone penetrometers, only four wires are needed to transmit the data uphole in series (instead of the parallel signals sent by cable). Other developments include a number of wireless CPT systems and special designs for deployment in the offshore environment (Lunne, 2001).

Land survey tests using cone have major advantages to traditional working methods (soil drilling and laboratory sampling) regarding subsoil investigation because they are rapid, repeatable and economic.

These advantages have led to a steady increase in the use and application methods of CPT in many places around the world. One of the major application of the CPT is providing real time information on the layered soil distribution and soil type.

Mechanical and piezocone tests began to be used increasingly often in practice because they are economic methodologies to analyse soil foundation and provide accurate information on some geotechnical parameters (the tip/cone resistance, q_c, sleeve friction, f_s, and in case of CPTu the pore water pressure, u).

Repetitive measurement and the possibility of analysing a large volume of soil, greater than with laboratory tests, plus continuous records regarding soil parameters make the use of CPT and CPTu methods very important in identifying lithology changes and stratigraphic profile reconstruction of the site (Lo Presti *et al.*, 2009).

Currently identifying stratigraphic limits of the subsoils can only be reconstructed based on empirical correlations (classification charts), such as Begemann (1965), Schmertmann (1978) and Searle (1979) for CPT and Robertson *et al.* (1986, 1990), Senneset *et al.* (1989), Eslami & Fellenius (1997, 2000) for the piezocone (CPTu). More recently, accurate methods for soil classification based on CPT have been proposed, such as those based on the so-called *fuzzy logic* (Zhang & Tumay, 1999), the principles of artificial neural networks (Kurup & Griffin, 2006) or probabilistic approaches (Jung *et al.*, 2008).

The empirical correlations (classification charts) available in technical literature resulted from analysis of multiple databases and intrinsic limitations of these correlations were the extrapolations made in different contexts. Thus the methods of identification of lithologic limits across a volume of soil based on neural networks or other similar methods require specific training *prior* to use. CPT and CPTu can be used to determine several geotechnical parameters, but in this paper are based only on defining stratigraphic limits of the subsoil, tip resistance, sleeve friction and neutral pressure.

2. Testing Procedure for CPT and CPTu

Cone penetration test (CPT) is currently an *in situ* geotechnical investigation which is used more and more often for civil, industrial, roads and bridges, due to its low cost, reduced execution time (1 … 1½ h to about 30 m linear probing) and large volume of information they provide, compared with conventional geotechnical investigation. Standard test procedure described in 1242/2-76, C159-89 and ASTM D 5778, uses a hydraulic system which ensures continuous penetration of a rods system (of 1.00 m or 1.50 m) supporting the cone, with a speed of 20 mm/s (Fig. 2).

At every 1…5 cm intervals the following geotechnical parameters are measured:

a) tip resistance (q_c);

b) sleeve friction (f_s);

c) porewater pressure (u_{12});

d) inclination (*i*).

The standard cone penetrometers are available in two models, both with a front end of a 60˚ apex conical tip as follows (Stănciucu, 2010):

a) *standard version*, with the base cone diameter of 35.7 mm, the corresponding cross-sectional area A_c = 10 cm^2 and the sleeve area A_s = 150 cm^2;

b) *commercial version*, with the base cone diameter of 43.7 mm, the corresponding cross-sectional area A_c = 15 cm^2 and the sleeve area A_s = 200 cm^2 or A_s = 300 cm^2; this type has the advantage that allows vigorous penetration field and the arrangement of a large number of sensors on the lateral surface.

An internal load cell is used to register the axial force at the front of the penetrometer (F_c). A second load cell is used to record the axial force either along the sleeve (F_s) within a "tension-type cone" design, or else located in the back and records the total tip force plus sleeve (F_c + F_s). In the latter case (termed *subtraction-type cone*), the combined force minus the separately-measured front force provides the sleeve force.

Porewater pressure measurement is made by its admission to the sensors through porous filter, having permeabilities around 0.01 cm/s. They are made of high density (granules of 120 μ), ceramic or sintered metal poly-propylene.

The penetrometer's pushing systems is currently very diverse in terms of construction. Thus it can be either a classical rod seal mounted on a heavy truck or a specially developed system that can be fitted on many types of vehicles. The most frequently heavy systems used to push rods have a capacity between 100 and 200 kN reaching up to 350 kN in exceptional circumstances. The maximum penetration depth of the subsoil depends on the geological condition of the site, however the most common penetration systems can easily reach depths of 30 m.

Depth logger system consists of a depth wheel, displacement transducer (either LVDT or DCDT), potentiometer (spooled wire), gear box, ultrasonics sensor, and optical reader. As each parameter measurement device is placed in various positions on the penetrometer tube, standard procedure involves correcting the depth readings on a common basis, namely the penetrometer tip.

In terms of data acquisition systems, the advantage is that old analogue systems are adaptable to any type of penetrometer, while new digital systems can only be used by the penetrometers they were designed for. *Modern data acquisition units incorporate global positioning systems (GPS), which allows the test results to be integrated in a GIS environment, thus regional databases can be created.*

3. Measured Parameters

3.1 Cone Resistance (q_c)

Represents an effort of which value is closely related to the soil bearing capacity and is defined as the force acting on the cone, F_c, divided by the projected area of the cone, A_c

$$q_c = \frac{F_c}{A_c}. \tag{1}$$

For sands cone resistance q_c depends on effective friction angle (Φ'), relative density (D_r) and the effective geostatic lateral stress (σ'_{h0}).

In the case of clayey rocks q_c depends on the undrained shear strength, (S_u) and the effective preconsolidation stress ($\sigma'p$)[1]. Most times in piezocone penetration tests conducted in cohesive rocks, q_c measured value should be corrected, taking into account the porewater pressure recorded at the cone tip. According to Lunne *et al.*, (1997), the corrected cone resistance is calculated with the relation

$$q_t = q_c + (1 - a_n)u_2, \tag{2}$$

where: $a_n = d^2/D^2$ is the ratio between shoulder area (cone base) unaffected by the porewater pressure to total shoulder area; u_2 – pore pressure measured at cone shoulder.

Net areas ratio, a_n, is defined as the unequal areas ratio at the ends of the device (Fig. 1), which is a constant of the equipment, obtained by the uniform compression of cone in the triaxial cell, or is provided by the device manufacturer. In practical applications, devices with $a_n \geq 0.80$ are preferred because they provide a minimal correction (NCHRP, 2007).

However, most cones with 10 cm^2 have $0.75 \leq a_n \leq 0.82$, and those with 15 cm^2 have $0.65 \leq a_n \leq 0.80$ (Mayne, *et al.*, 2001). It is clearly specified in the literature (Lunne *et al.*, 1986; Robertson & Campanella, 1988) that under a correct calibration between measured values (q_c) and corrected (q_t) differences can be 20% to 70%. For sands, the corrected values are generally $q_t \geq 5$ MPa, for clays and silts $q_t \leq 2$ MPa, except for overconsolidated clayey rocks where $q_t \geq 5$ MPa (Sabatini *et al.*, 2002).

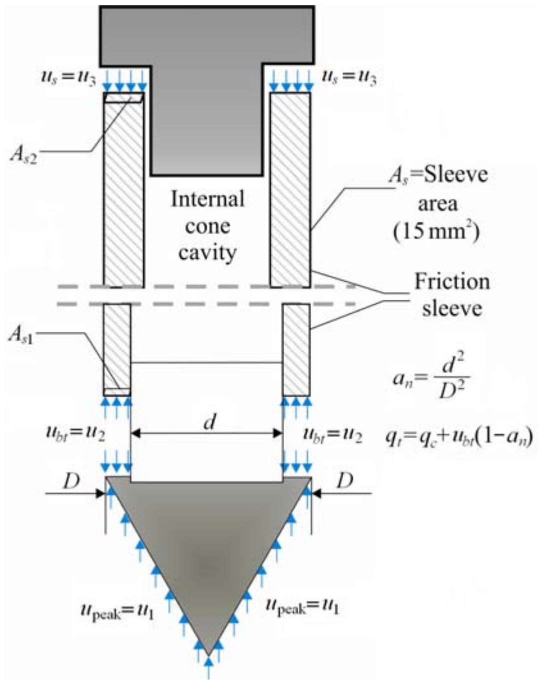


Fig. 1 – Unequal piezocone areas.

3.2. Sleeve Friction (f_s)

The ratio between the measured axial force over the sleeve (F_s) and the sleeve area (A_s) is

$$f_s = \frac{F_s}{A_s}. \tag{3}$$

The same as the tip resistance, the sleeve friction is corrected taking into account the values of the measured porewater pressure

$$f_t = f_s - b_n u_2, \tag{4}$$

where b_n is a device constant obtained by uniform compression of the cone in the triaxial cell and is called the *net ratio of the sleeve area*. Sleeve resistance is often expressed in terms of corrected cone resistance (Lunne *et al.*, 1997) by a friction ratio of which value is related to rock granularity

$$\mathrm{FR} = \frac{f_s}{q_t} \cdot 100, \ [\%]. \tag{5}$$

High values (3…4% < FR <10%) are associated to clayey and dusty rocks because of their high cohesion and reduced friction, while lower values (FR < 1…1.5%) are specific to sandy rocks or dry clays.

3.3. Porewater Pressure (u)

The pressure that water develops between the rock grains over penetration is measured through porous filters by specific transducers placed in different positions. Thus, standard position of porous filters is on the cone shoulders (u_2) because it enables the tip resistance correction. With porous filters placed on cone shoulders in sandy rocks the recorded pressure value is very close to hydrostatic pressure ($u_2 \approx u_0$), while for the clays, regardless of their consistency, is much higher ($u_2 > u_0$).

Based on stress values transmitted by the geological load, (σ_{v0}) (Stanciu & Lungu, 2006) and hydrostatic pressure (u_0), it is obtained the B_q parameter value of the interstitial pressure,

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} . \tag{6}$$

This relation is used for the normalization of the test values having as result the identification and classification of crossed layers.

4. Case Study

To reflect the applicability of these *in situ* investigation methods, a case study of Piano di Conca village in the province of Lucca, Italy, was performed through five piezocone tests in the area of the village, as shown in Fig. 2.

Through continuous records of three measured independent parameters, cone penetration tests provide an effective tool for lithological layers identification, unlike the laboratory sampling. The results interpretation require the use of a specialized computer program named *GeoLogismiki* to automatically calculate and plot the measured geotechnical parameters: q_t, f_s, u and FR (Fig. 4).

The results interpretation is based on the following generally valid inequalities:

$$q_t > f_s \text{ and } q_t > u_1 > u_2 > u_3 . \tag{7}$$

The interpretation charts known in the literature for basic soils classification that can be used for cone and piezocone penetration tests are over 20 in number. In this case the identification of lithological layers is based on the profiling charts established by Robertson *et al.* (1986) (Fig. 3).

Fig. 2 – The approximate locations of the piezocone tests.

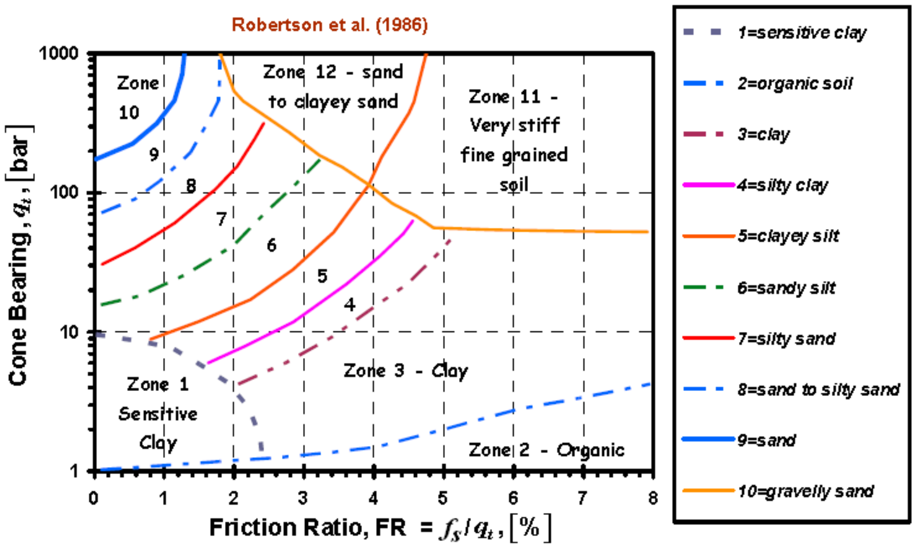
According to Robertson *et al.* (1986) profiling charts, it can be easily observed that the whole inspected depth is a succession of lithological layers which consists of sensitive fine grained up to clay & silty clay for the first four test points, but in the test fifth point were identified also sandy silt & clayey silt, silty sand & sandy silt.

As shown in Fig. 4 the resistance on the tip of the cone (q_c), has an approximately uniform aspect except the first two meters where $q_c \approx 0.75$ MPa and then remains constant at approximately 0.25 MPa up to the final rate of the survey.

In the case of test point five, from the depth of 3.5 m to 6.0 m the tip resistance reaches up to 2.0 MPa because of the intrusion of a sand layer.

The friction ratio does not have a constant aspect for all five point tests, but a variable one that provides information about the existence of a succession of distinct lithological deposits.

The porewater pressure value (u_2) reaches from almost 25 kPa up to 400 kpa because of the presence of clayey soils which develop big neutral forces between particles.



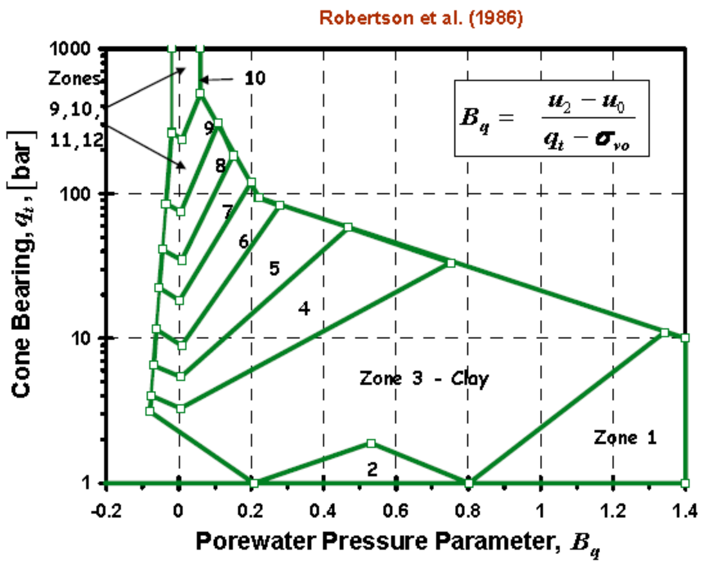


Fig. 3 – CPTu Soil Behavioral Type (SBT) for Layer Classification (Robertson *et al.*, 1986).

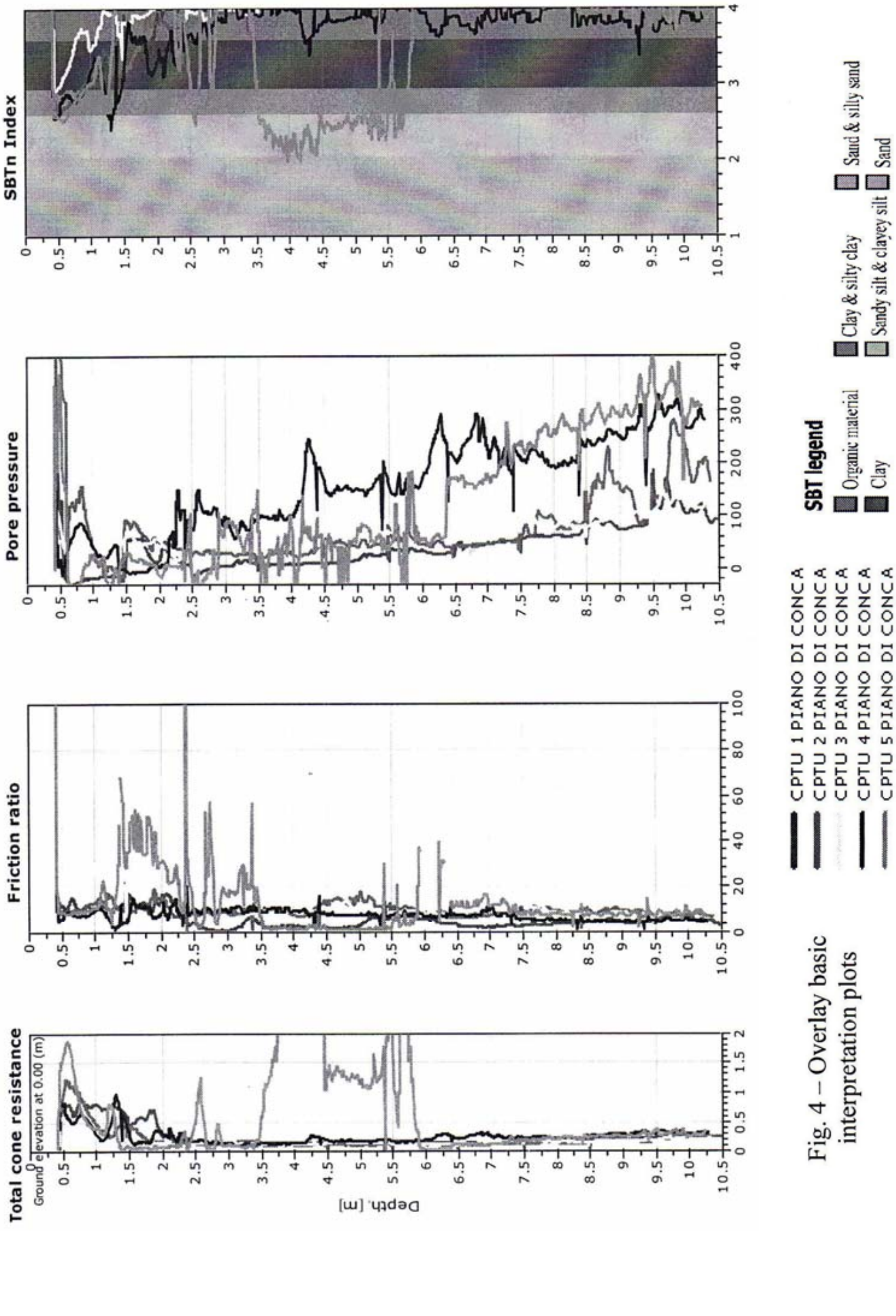


Fig. 4 – Overlay basic interpretation plots

5. Conclusions

Cone penetration test is an *in situ* investigation technique, commonly used to identify near surface unconsolidated soils, offering the possibility to create a continuous geostratigraphic profile, soils property evaluation and also various physical parameters are recorded during the entire test.

Using the *in situ* cone and piezocone tests as a geotechnical study method has the advantage that on the entire ongoing period of the test, geotechnical data are received quickly and continuously, requiring a reduced time with high productivity, the collected results are not influenced by the technical operator, involves the use of a very solid theoretical basis for interpretation and is particularly recommended for researching the soft soils with low consistency.

The disadvantages of these tests consist of: high purchasing costs for the equipment; the operators need to be highly qualified; involves a difficult calibration process of the device; the lack of soil sampling analysis.

CPTu provides more accurate information than CPT regarding the exact location and identification of the soil layers, but in some cases even piezocone method implies difficulties in lithological classification of certain thin soil lenses.

Acknowledgement. This paper was realized with the support of EURODOC „Doctoral Scholarships for Research Performance at European Level" project, financed by the European Social Found and Romanian Government. Also, special thanks are given to professor Diego Lo Presti, for the support and guidance during the research stage at the University of Pisa, Italy, Department of Civil Engineering.

REFERENCES

Begemann H.K.S., *The Friction Jacket Cone as an Aid in Determining the Soil Profile*. Proc. 6th ICS-MEF, **1**, 1965, 17-20.

Eslami A., Fellenius B.H., *Pile Capacity by Direst Cone Penetration Tests and Cone Penetration Tests with Porewater Pressure Measurement Methods Applied to 102 Case Histories*. Can. Geotech. J., **34**, 886-904 (1997).

Fellenius H.B., Eslami A., *Soil Profile Interpreted from Cone Penetration Tests with Porewater Pressure Measurement Data*. "Year 2000 Geotechnics", Geotech. Engng. Conf. Bangkok: Asian Inst. of Technol., 2000.

Jamiolkowski M., *Opening Address: CPT'95*. Proc. Internat. Symp. on Cone Penetration Testing, **1**, Linköping: Swedish Geotechn. Soc. Report 3:95, October 4-5, 1995, 7-15.

Jung B. C., Gardoni P., Biscontin G., *Probabilistic Soil Identification Based on Cone Penetration Tests*. Geotechnique, **58**, 591-603 (2008).

Kurup U.E., Griffin E.P., *Prediction of Soil Composition from Cone Penetration Tests Data Using General Regression Neural Network*. J. of Comp. in Civil Engng., **20**, 281-289 (2006).

Lo Presti D., Squeglia N, *Soil Profiling from Cone Penetration Tests. Do we Really Need Classification Charts? Capability of Penetration Tests in Geotechnical Research and Practice*. Proc. of the 1st Europe-China Workshop, Pisa: Plus-Pisa Univ. Press, 2010, 85-92.

Lo Presti D., Meisina C., Squeglia N., *Applicabilita di prove penetrometriche statiche nella riconstruzione del profilo stratigrafico*. Rivista italiana di geotecnica, **2**, 9-33 (2009).

Lunne T., *In-Situ Testing in Offshore Geotechnical Investigations*. Proc., Internat. Conf. on *In-Situ* Meas. of Soil Properties a. Case Histories, Bali, 2001, 61-81.

Lunne T., Robertson P.K., Powell J.J.M., *Cone Penetration Testing in Geotechnical Practice*. Blackie, London, 1997, 312.

Mayne P.W., Cone *Penetration Testing State-of-Practice*. State-of-Practice Transp. Res. Board. Synth. Study, NCHRP Project 20-05, Topic 37-14, 2007.

Robertson P.K., *Soil Classification Using the Cone Penetration Test*. Can. Geotech. J., **27**, 151-158 (1990).

Robertson P.K., Campanella R.G., Gillespie D., Grieg J., *Use of Piezometer Cone Data*. Proc. of Amer. Soc. of Civil Eng, Blacksburg: S. Clemence, June 23-25, Geotech. Spec. Publ. GSP, **6**, 1986, 1263-1280.

Sabatini P.J., Bachus R.C., Mayne P.J., Scheneider J.A., Zettler T.E., *Geotehnical Engineering*, Circular no. *5*, *Evaluation of Soil and Rock Properties*, Report no. FHWA-IF-02-034. US Dept. of Transp., Washington D.C., 2002.

Schmertmann J.H., *Guidelines for Cone Penetration Test, Performance and Design*. Report No. FHWA-TS-78-209, U.S. Dept. of Transp., Washington, D.C., 1978, 145.

Searle I. W., *The Interpretation of Begemann Friction Jacket Cone Results to Give Soil Type and Design Parameters*. Design Param. in Geotechn. Engng., **2**, BCS London, 1979, 265-270.

Senneset K., Sandven R., Jambu N., *Evaluation of Soil Parameters from Piezocone Test*. *In Situ Testing of Soil Properties for Transportation*. Transp. Res. Record No. *1235*, Washington D.C., 1989, 24-37.

Stanciu A., Lungu I., *Fundații*. Vol. **1**, Edit. Tehnică, Bucureşti, 2006.

Stănciucu M., *Investigații geotehnice in situ*. Bucureşti, 2010.

Van De Graaf H.C., Jekel J.W.A., *New Guidelines for the Use of Inclinometer with the Cone Penetration Tests. Penetration Testing*. Proc. ESOPT-1, Amsterdam, The Netherlands, **2**, Balkema, Rotterdam, 1982, 581-584.

Zhang Z., Tumay M. T., *Statistical to Fuzzy Approach Toward Cone Penetration Tests Soil Classification*. J. Geotech. Geoenviron. Engng., **125**, 179-186 (1999).

* * * *Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils*. ASTM D, 5778-07.

* * * *Instrucțiuni tehnice pentru cercetarea terenului de fundare prin metoda penetrării cu con, penetrare statică, penetrare dinamică, vibropenetrare*. C 159-89.

* * * *Cone Penetration Testing*. A Synthesis of Highway Practice, *368*, NCHRP, 2007.

UTILIZAREA TESTELOR DE PENETRARE STATICĂ PE CON ŞI A TESTELOR DE PENETRARE STATICĂ PE PIEZOCON ÎN SCOPUL IDENTIFICĂRII LIMITELOR STRATIGRAFICE ALE TERENURILOR DIFICILE DE FUNDARE

(Rezumat)

Utilizarea unei suprafețe de teren ca suport pentru o ulterioară construcție presupune identificarea proprietăților fizico-mecanice ale terenului respectiv prin prelevarea de probe de laborator sau utilizarea unor metode de sondare *in situ* cu scopul alegerii sistemului de fundare optim, evitându-se apariția unei stări limită ultime sau a unei stări limită de serviciu.

Cele mai multe metode de sondare ale terenului de fundare se realizează discontinuu, din metru în metru, având dezavantajul că nu pot oferi informații geotehnice pe toată adâncimea zonei de teren analizată. Testele de penetrare statică pe con (CPT) şi testele de penetrare statică pe con cu determinarea presiunii apei din pori (CPTu) sunt printre singurele care oferă informații exacte despre limitele stratigrafice, diferitele anomalii litologice şi tipul de teren ca urmare a înregistrărilor continue a parametrilor geotehnici ai terenului de fundare în toată zona analizată.