

# Interpretation of CPT and SDMT tests for Lublin loess soils exemplified by Cyprysowa research site

Krzysztof Nepelski

*Department of Geotechnical Engineering, Faculty of Civil Engineering and Architecture, Lublin University of Technology, e-mail: k.nepelski@pollub.pl, ORCID: 0000-0001-9495-6457*

**Abstract:** This paper presents an example of interpretation of in situ tests, CPT static sounding and seismic Marchetti dilatometer tests (SDMT). The studies were carried out on loess soils in Lublin. Four CPT tests and four SDMT seismic tests were performed. The article describes the method of deriving geotechnical parameters from in situ testing. In particular, the formulas for calculating the constrained modulus based on the cone resistance  $q_c$  were analysed. Some of the parameters were interpreted using the proposed formulas. Values of deformation parameters determined with various methods for different strain ranges were compared.

**Keywords:** CPT, SDMT, loess, constrained modulus

## 1. Introduction

To analyse the building-subsoil interaction, soil conditions as well as their variability and parameters must be identified. Boreholes are the basic and most widely used research method in the world, since they provide information about the type of soil found in the ground and its main characteristics. It allows for identifying stratigraphic and lithological divisions, which is the basis for developing a sampling plan for laboratory tests. Soil behaviour is complex and depends on numerous factors; therefore, different types of tests should be used when evaluating a geotechnical model and determining strength and deformation characteristics. A geotechnical profile should show separate layers with similar mechanical properties. For this reason, in-situ sounding has an important role in the identification process.

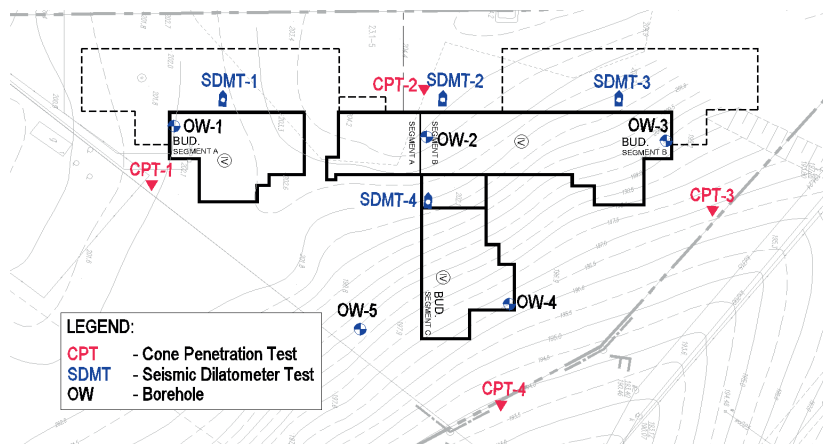


Fig. 1. Location of test points

This paper presents an example of interpretation of in situ tests, CPT static sounding and seismic Marchetti dilatometer tests (SDMT). The research was carried out on loess soils in the area of Cyprykowa Street in Lublin. The location of test points is shown in Fig. 1. The study involved four CPT tests and four SDMT dilatometer tests with seismic shear wave velocity measurements.

## 2. Cone Penetration Tests (CPT)

CPT tests were performed using the Pagani T63-150 with a maximum pull-down force of 150 kN. A Begemann mechanical cone was used during the tests. It was pressed at a speed of 2 cm/s, with penetration characteristics recorded every 20 cm. Although a mechanical cone provides less information than a cone with electrical sensors, studies [1] have shown that in the case of typical loess soils, the differences in cone resistance  $q_c$  are low. The values taken during the test are cone resistance and friction on the friction sleeve. The cone that was used had standard geometry: base surface of 10 cm<sup>2</sup>, friction sleeve surface of 150 cm<sup>2</sup>, and cone tip angle of 60°. All test parameters were in accordance with the standards defining the conditions of static sounding [2]–[4].

The readings recorded during the tests provided the basis for their subsequent interpretation. In order to interpret the data and determine the geotechnical parameters of the soil layers distinguished in the subsoil, the data was presented with the use of standard parameters:  $q_c$  – cone resistance,  $f_s$  – friction on the friction sleeve, and  $R_f$  – friction ratio, used for classifying soil by soil behaviour type.

To identify the subsoil structure, the Robertson nomogram was used, modified and adapted to the Polish conditions by Młynarek et al. [5], as well as information obtained from the test boreholes. The data from drilled boreholes were used as the leading data for the identification of soil type. The interpretation and analysis of soundings is widely described in the literature and has been compiled by Sikora [6]. The results of the sounding were used to determine soil parameters. The liquidity index  $I_L$  was derived using the formula by Nepelski et al. [7]:

$$I_L = 0.76 - 0.17q_c \quad (1)$$

Undrained shear strength  $c_u$  was determined in accordance with Eurocode 7 standards [3] and PN-B-04452 [4], using the following formula:

$$c_u = \frac{(q_c - \sigma_{v0})}{N_{kt}} \quad (2)$$

where  $\sigma_{v0}$  is the geostatic stress at the measurement level  $q_c$ , and  $N_{kt}$  is the empirical coefficient taken depending on the type of soil. A coefficient of  $N_{kt}=40$  was adopted for loess soils based on Frankowski's research [8].

The constrained modulus  $M$  was determined in accordance with the Eurocode 7 [3]. According to the aforesaid standard, the constrained modulus is determined by Sanglerat's relationship [9] using the following formula:

$$M = \alpha_m q_c \quad (3)$$

The essence of the correct estimation of constrained modulus is the adoption of an appropriate empirical coefficient  $\alpha_m$ . Sanglerat [9] proposes using the  $\alpha_m$  coefficient in the range of 1÷8, depending on the type of soil and cone resistance. For stiffer soils, lower values are assumed. A similar formula is put forward by Ciloglu [10], according to whom  $\alpha_m$  should be

within the range of  $3.1 \div 13.5$ , depending on the plasticity index and the content of fine-grained fractions. A slightly adjusted dependency in the following form:

$$M = \alpha_m (q_t - \sigma_{v0}) \quad (4)$$

is put forward by Senneset [11], where  $\alpha_m$  is assumed within the range of  $5 \div 15$  for overconsolidated soils and  $4 \div 8$  for normally consolidated soils. A different correlation that assumes a constant  $\alpha_m$  coefficient is put forward by Kulhawy and Mayne [12]:

$$M = 8.25(q_t - \sigma_{v0}) \quad (5)$$

Młynarek and Wierzbicki used this relationship for loess soils from the Łańcut area ([13], [14]). Frankowski [15] determined a coefficient of  $\alpha_m = 2.5$  for formula (3) for loess soils from the Kazimierz Dolny area. This result was obtained from calculations in which the constrained modulus was determined from oedometer tests. According to the stiffness degradation curve [16], oedometer tests correspond to high plastic deformations, i.e. conditions that differ significantly from those found under typical foundations. The work of subsoil under typical foundations is far from the critical state, therefore oedometric constrained moduli are not suitable for the calculation of building settlements. Taking into account the aforesaid data taken from the literature and the results of the performed analyses, it was decided to adopt the formula (3) with the  $\alpha_m = 6$  coefficient for loess soils. In [17] it was proved with numerical analyses and geodetic measurements that it is a correct value.

The original substrate deformation modulus  $E_{0,CPT}$  was determined according to Pisarczyk's recommendations [18] using the following formula:

$$E_{0,CPT} = 3.8q_c + 2.5 \text{ MPa} \quad (6)$$

A short comparative analysis of the deformation parameters interpreted from static soundings was carried out. The constrained modulus calculated using Sanglerat's formula (3) with the coefficient  $\alpha_m = 6$ , denoted as  $M_{CPT[1]}$ , was compared with the deformation modulus  $E_{0,CPT}$  found using formula (6) and recalculated to constrained modulus marked as  $M_{CPT[2]}$  using the following formula:

$$E = M \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \quad (7)$$

where  $M$  is the constrained modulus, and  $\nu$  is the Poisson ratio. The comparison is illustrated in Fig. 2.

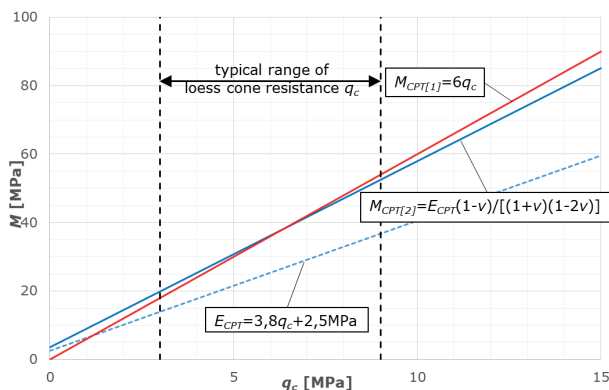


Fig. 2. Comparison of constrained moduli interpreted from CPT static tests using different formulas

It shows that the constrained moduli found with both formulas obtain very similar results for the cone resistance range of  $q_c$ , which is typical for Lublin loess soils. For the mean cone resistance value for loess soils, i.e.  $q_c$  of approx. 6.5 MPa [19], the constrained modulus  $M_{CPT}$  found with both formulas is about 39 MPa. For  $q_c < 6.5$  MPa, the modulus determined using Sanglerat's relationship has lower values than the one determined on the basis of the  $E_{0,CPT}$  modulus, whereas for  $q_c > 6.5$  MPa it is the other way around. For a typical range of cone resistance for loess soils of  $q_c = 3 \div 9$  MPa, the  $M_{CPT[1]}/M_{CPT[2]}$  ratio is from 0.9 for  $q_c = 3$  MPa to 1.03 for  $q_c = 9$  MPa, which means that in the case of stiffer loess soils the differences are so low that the choice of formula used is insignificant. For softer loess soils, however, the difference is greater and increases as the  $q_c$  value decreases. More unfavourable parameters will be obtained with the use of Sanglerat's formula (3).

The basic parameters of the selected CPT-1 test are shown in the diagrams in Fig. 3. The division into subsoil layers was made on the basis of soil classification and sounding characteristics. In a continuous profile, with characteristics described every 20 cm, layers with representative parameters determined on the basis of cone resistance were distinguished. The mean value of cone resistance for a given layer was taken as the representative value. Extremely high values were rejected. The division into layers is shown only in the basic parameter charts.

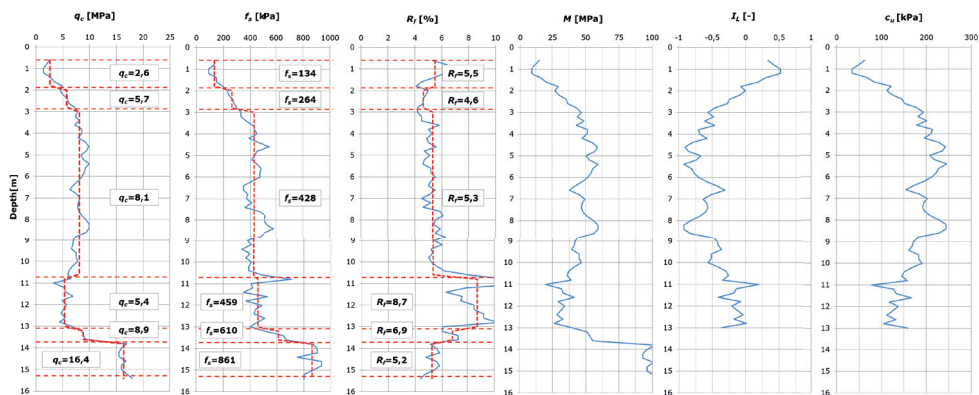


Fig. 3. CPT-1 test parameters

To summarise the CPT profiles, it was found that for the investigated area, loess soils under building foundations are characterised by cone resistances  $q_c$  in the range of 4.6 ÷ 8.1 MPa, with a mean value of 5.3 MPa. The obtained values show that loess soils in this area can be considered solid load-bearing subsoil that is representative for the Lublin area. Furthermore, for the obtained  $q_c$  range, the differences in the constrained moduli determined using the adopted formulas as presented in Fig. 2. are negligible.

### 3. Seismic Marchetti Dilatometer Tests (SDMT)

Seismic Marchetti dilatometer tests SDMT were carried out in cooperation with the Department of Geotechnical Engineering of the Warsaw University of Life Sciences and using the equipment provided by that unit. The tests were carried out using a Van der Berg Hysom 200 kN probe. A standard Marchetti dilatometer consists of a flat, steel blade with a circular, flexible membrane, and a measuring/control unit with pressure readout. During the test, the blade is pressed vertically into the ground, and then measurements are taken at intervals of

0.2 m (sometimes 0.1 or 0.5 m). During the measurements, gas pressure is applied to the membrane from the ground level by means of a pneumatic line. During this operation, the membrane deforms towards the ground and readings A and B are taken. Reading A is the gas pressure value obtained during the initial phase of membrane movement (displacement of the membrane centre by 0.05 mm), which causes it to come into contact with the surrounding soil. Reading B is the pressure value obtained with an additional displacement of the membrane centre towards the ground by approx. 1.05 mm, for a total of 1.1 mm. A third reading C is sometimes taken, corresponding to the gas pressure after the return of the membrane to its initial position. The readings are adjusted by corrections  $\Delta A$  and  $\Delta B$ , which result from the rigidity of the membrane. The dilatometer used in the study was additionally equipped with a seismic module to measure the shear wave velocity. The seismic module is placed on a rod directly behind the DMT measuring blade and consists of two geophones positioned 0.5 m apart, which are used as receivers for measuring the shear wave generated during the test. Shear wave velocity is usually measured at depth intervals of 0.5 m. The wave, generated by a hammer hitting an anvil pressed against the ground, first reaches the upper receiver and then the lower one. The movement of ground particles which occurs with shear wave propagation corresponds to very small deformations of the subsoil. The shear wave propagation velocity is the basis for determining the initial shear modulus [20]–[22].

The interpretation of dilatometer tests is based on three basic indexes: material index  $I_D$ , horizontal stress index  $K_D$  and dilatometer modulus  $E_D$ . These indexes are determined as follows:

$$I_D = \frac{(p_1 - p_0)}{(p_1 - u_0)} \quad (7)$$

$$K_D = \frac{(p_0 - u_0)}{\sigma'_{v0}} \quad (9)$$

$$E_D = 34.7(p_1 - p_0) \quad (10)$$

where:

- $p_0$  – the pressure of membrane's contact with the ground,
- $p_1$  – the pressure of membrane displacement by 1.1 mm,
- $u_0$  – hydrostatic pore water pressure,
- $\sigma'_{v0}$  – effective vertical stress in situ.

The material index is primarily used to determine the type of soil. Generally speaking,  $I_D=1.8$  is the boundary between cohesive and non-cohesive soils. Fine-grained (cohesive) soils have a lower  $I_D$ , while coarse-grained (non-cohesive) soils have a higher  $I_D$ . Just like the  $R_f$  index in the case of CPT tests interpretation, the material index  $I_D$  from DMT tests determines the behaviour of the soil and does not classify it on the basis of grain size as assumed in the standards. For the studied loess soils, the  $I_D$  index showed the same behaviour as in the case of sandy soil.

The horizontal stress coefficient  $K_D$  is used to determine undrained shear strength  $c_u$  and to determine the lateral earth pressure coefficient  $K_0$ . In accordance with the Marchetti's relationship, the undrained shear strength  $c_{u,DMT}$  and the lateral earth pressure coefficient  $K_0$  were determined from the following formulas:

$$c_{u,DMT} = 0.22\sigma'_{v0} (0.5 K_D)^{1.25} \quad (11)$$

$$K_{0,DMT} = (K_D / 1.5)^{0.47} - 0.6 \quad (12)$$

The dilatometer modulus  $E_D$  determines the relationship between the stress acting on the membrane and its displacement. This module, however, cannot be used for direct calculations of settlements, but only reflects the stiffness of the ground and can be used for calculations after taking into account the history of lateral stress, denoted by the  $K_D$  index. For determining the settlement, the dilatometer constrained modulus  $M_{DMT}$  calculated using the Marchetti formula [21] is used:

$$M_{DMT} = R_M E_D \quad (13)$$

Another important parameter is the overconsolidation ratio  $OCR$ , which was determined using the Marchetti [21] formula, later expanded by Mayne and Martin [23] into the following form:

$$OCR_{DMT} = (0,5K_D)^{1.56} \quad \text{for soils with a material index of } I_D < 1.2, \quad (14)$$

$$OCR_{DMT} = (mK_D)^n \quad \text{for soils with a material index of } 1.2 < I_D < 2.0, \quad (15)$$

where:

$$m = 0.5 + 0.17p,$$

$$n = 1.56 + 0.35p,$$

$$p = (I_D - 1.2) / 0.8,$$

$$OCR_{DMT} = (0.67K_D)^{1.91} \quad \text{for soils with a material index of } I_D > 2.0. \quad (16)$$

Currently, for clay soils, i.e. those with a material index of  $I_D < 1.2$ , there are also numerous other expanded versions of the original Marchetti formula, which primarily take into account the type of soil and regional conditions. On the other hand, for silty and, in particular, sandy soils, i.e. soils with  $I_D > 1.2$ , the determination of OCR is much more complicated and most often it is connected with cone resistance  $q_c$  [24], therefore it additionally requires CPT static sounding.

The friction angle for non-cohesive soils with a material index of  $I_D > 1.8$  was determined using the following formula:

$$\varphi_{DMT} = 28 + 14.6 \log K_D - 2.1 \log^2 K_D \quad (17)$$

The initial shear modulus  $G_0$ , derived from the formula, was also determined from seismic tests:

$$G_0 = \rho V_s^2 \quad (18)$$

where:

$\rho$  – density of the soil,

$V_s$  – shear wave velocity measured during the SDMT test.

These interpretations are mainly based on formulas originally developed and recommended by Marchetti, first published in 1980 and updated from time to time [21], [24], [25]. Currently, in addition to the basic interpretations, there are a number of relationships derived by other researchers for soils from various parts of the globe. In Poland, Lechowicz et al. [20], [26] Rabarijoely [27], and Młynarek and Wierzbicki [14], [28] conducted large-scale analyses of the interpretation of dilatometer test results.

The basic parameters for the selected SDMT-2 tests are shown in Fig. 4, while Fig. 5 shows a representative diagram of the shear wave velocity recorded at a depth of 4.5 m during the

SDMT-2 test. The signal reaching the upper receiver is marked in blue and the signal reaching the lower receiver is marked in red. The graphs on the left show the signals recorded directly, while the re-phased signals are shown on the right.

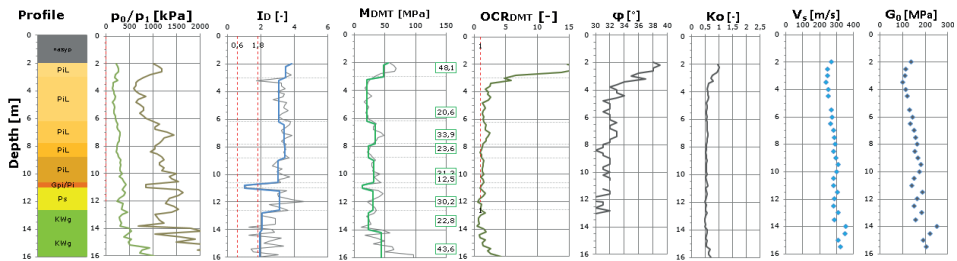


Fig. 4. Results of SDMT-2 tests at the Cyprysowa site

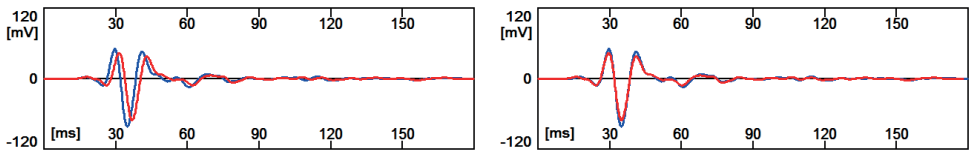


Fig. 5. Shear wave seismogram during SDMT-2 tests at a depth of 4.5 m

Dilatometer tests, like the results of static soundings, are the basis for distinguishing geotechnical layers and describing their parameters. The main parameter used in the analyses is the dilatometer modulus  $M_{DMT}$ . For the area in question, this parameter varies in the most of the profile within the limits of 30÷70 MPa, which should be considered a relatively high value. The constrained moduli determined in DMT tests were slightly higher than those determined during CPT static probing. The initial shear modulus  $G_0$  determined in SDMT seismic tests ranged from 150 to 200 MPa.

#### 4. Conclusions

In-situ tests provide a lot of information with regard to subsoil parameters and the distribution of soil stiffness at depth. In field tests, several selected parameters are usually measured and then converted into geotechnical parameters, e.g. internal friction angle, constrained modulus, undrained shear strength, etc., using empirical formulas. Since the most important parameters are not determined directly, it is extremely important to properly interpret the results measured directly on-site. The paper presents selected results and methods of interpretation of a CPT static test and SDMT tests performed on Lublin loess soils. The determined parameters can be used to analyse the building-subsoil interaction and to construct a computational model, e.g. in the form of geotechnical cross-sections or computational regions. The analysed loess silts are intermediate soils with features of both cohesive and non-cohesive soils. They have low cohesiveness and the appearance of cohesive soils in macroscopic terms, but both the  $R_f$  index from CPT static probing and the  $I_D$  index from DMT tests indicate behaviour that is characteristic of non-cohesive soils.

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