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INDIRECT DETERMINATION OF SOIL YOUNG'S MODULUS IN LITHUANIA USING CONE PENETRATION TEST DATA

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Abstract. Simplified methods based on cone penetration test results are commonly used to determine soil deformation modulus, depending on the engineering geological and geotechnical conditions and the complexity of the computational approach. This paper reviews some empirical equations based on the results of the cone penetration test and gives recommendations for the assessment of Young's modulus, oedometric modulus and residual modulus from the cone penetration test result, according to the Lithuanian technical requirements and other standards. Theoretical interpretations of results are presented together with practical examples for coarse and fine soils, limits of empirical equations application are explained.

Keywords: cone penetration test (CPT), constrained modulus, residual modulus, soil deformation modulus, Young's modulus.

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Introduction

From a geological point of view, most of the Lithuanian territory consists of Holocene and Upper Pleistocene deposits. More specifically, in the southeastern part and the deep erosive valleys of Nemunas, Neris, and Šventoji rivers, Middle Pleistocene deposits are commonly found. Pre–Quaternary bedrocks fall into the sphere of the active impact of building construction only in the northeastern part of the territory of Lithuania, where under the thin cover of Quaternary deposits Ketleriai, Žagarė, Švėtė, Muriai, Akmena, Kuršiai, Joniškis, Šiauliai, Kruoja, Pakruojis, Stipinai, Pamūšis, Įstras, and Tatula formations rocks are present from Upper Devon period. The gypsum rocks of Tatula formation affected by karst phenomena have special properties. This bedrock is covered by a Quaternary cover (up to 20 m), and its geological engineering conditions depend on the intensity of karst phenomena and processes. The geological engineering conditions of Lithuania territory are reflected in the engineering geological map with a scale 1:500 000 (Bucevičiūtė et al., 1997). This map provides generalised information on geotechnical constructional properties of soil and modern geological phenomena. Thirty geological engineering districts are singled out on the map. The districts represent different complex geological engineering conditions. The values of physical and mechanical properties of soils are statistically summarised in the compiled summary of engineering geological cross-sections.

Risk and uncertainty are characteristics of the ground that always exists. The appropriate level of sophistication for site characterisation and analyses have to be based on (Robertson & Cabal, 2015):

- precedent and local experience;
- design objectives;
- level of geotechnical risk (Urbanavičienė & Skuodis, 2019);
- potential cost savings.

The design and building of all constructions begin from engineering geological and geotechnical investigations. The cone penetration test (CPT) is the most common in engineering geological investigations in Lithuania. It is suitable for various strengths and grain size distribution (Žaržojus & Kelevišius, 2016).

This paper presents investigations of soil Young's modulus variation when it is calculated based on cone penetration data. Particular attention is paid to the dependencies and properties presented in the literature, explaining the possibilities of applying the data presented in the literature. This explanation is relevant to engineering geology and geotechnical engineers. Engineering geologists have to pay particular attention to the analysis of archival material or literature material because one of the goals of geological engineering research is a brief analysis and evaluation of the engineering geological cartography map and previous research data (*STR 1.04.02:2011 Inžineriniai geologiniai ir geotechniniai tyrimai*). It is hard to conduct the proper analysis and evaluation of the archival material because the different regulations and standards for engineering geological investigations were valid in different periods of investigations.

1. Calculation of Young's modulus in Lithuania practice

In engineering geological and geotechnical research in Lithuania, the soil Young's modulus E for more than 30 years is calculated from the results of CPTs (Žaržojus & Dundulis, 2010). The calculations are based on empirical Equations (1)–(4) proposed by Brilingas (1988) and still are used in their original form. The equations mentioned above were formed after analysing the results of more than 250 plate load tests and CPT conducted while investigating the conditions of Lithuanian soils (Brilingas, 1988). Brilingas (1988) studied the glacial, glaciolacustrine, and glaciofluvial deposits of different lithology from Upper Pleistocene Nemunas glaciation Baltija glacial stage. Due to glaciation, all of the mentioned soils are over-consolidated. The correlation coefficients of regression equations demonstrated the relation between q_c and E. The correlation coefficient of fine soils was 0.75–0.84 and for sands – 0.86 (Brilingas, 1988). Equations (1)–(5) for calculating Young's modulus based on cone penetration data are presented separately:

– glacial loam (till) (gIIIbl):

$$E = 7.4q_c + 7.2; \tag{1}$$

– glaciolacustrine clay (lgIIIbl):

$$E = 8.2q_c - 3.1; \tag{2}$$

– glaciolacustrine loam (lgIIIbl):

$$E = 4.8q_c + 4.9; \tag{3}$$

– glaciofluvial and glacial sands (fIIIbl and gIIIbl):

$$E = 7.8q_c^{0.71}; \tag{4}$$

– generalised linear regression:

$$E = \alpha q_c. \tag{5}$$

In Equation (5), the coefficient α varies depending on soil genesis, lithological composition, and cone resistance q_c values. The coefficient α values of upper Pleistocene Nemunas glaciation Baltija stage till loam range from 14 when $q_c = 1.0$ MPa to 8 when $q_c = 9.0$ MPa. The glaciolacustrine loam of Baltija stage α values vary from 10 when

q_c = 1.0 MPa to 5.8 when q_c = 5.0 MPa, in the clays from 5 when q_c = 1.0 MPa to 7.5 when q_c = 5.0 MPa. In sands, α values vary from 6.5 when q_c = 1.0 MPa to 3.3 when q_c = 20.0 MPa.

All Equations (1)–(5) are applied for soils classified by the standard in force at the time *GOST 25100–82 Grunty. Klassifikacija.* This standard specified that sand is classified based on grain size distribution, and fine soils are classified based on their plasticity index (IP). After Lithuania regained its independence, the *GOST* classification was still in force later. It was replaced by *LST 1445:1996*. These classifications were replaced by *LST EN ISO 14688–1:2007 Geotechniniai tyrinėjimai ir bandymai. Gruntų atpažintis ir klasifikacimas. 1 dalis. Atpažintis ir aprašymas* and *LST EN ISO 14688–2:2007 Geotechniniai tyrinėjimai ir bandymai. Gruntų atpažintis ir klasifikacimas. 2 dalis. Klasifikavimo principai* and from 2019 by *LST EN ISO 14688–1:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikacimas. 1 dalis. Indetifikavimas ir aprašymas* and *LST EN ISO 14688–2:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikacimas. 2 dalis. Klasifikavimo principai* came into force. The latter standard in Lithuania is adapted to the characteristics of soils in the region, issued by the Lithuanian Geological Survey "Classification of Engineering Geological and Geotechnical (EGG) Survey Soils" (Lietuvos geologijos tarnyba, 2019).

All classifications changed the names of the soil, the composition of the soil matrix changed because the principles of classification differed substantially, i.e., interpretation of grain size distribution based on the soil grain size. In the *GOST 25100–82* classification, it was stated that the sand particles are from 2.00 to 0.05 mm. The classifications used later assumed that the sand particles are between 2.00 mm and 0.06 mm. The principles of sand classification also differed. For example, by *LST 1445:1996* sand was classified as containing more than 50% of particles larger than 0.063 mm. As reported by the later changed classification *LST EN ISO 14688–2:2007* sand contained more than 40% of particles larger than 0.06 mm. Under the current EGG classification in Lithuania, coarse soil will be when the number of fines (< 0.063 mm) is less than 35%, i.e., the amount of sand and coarser particles is more than 65%.

Another critical difference is the classification of fine soil. Under *GOST 25100–82*, fine soil (clay, silt or mixtures thereof and mixtures with sand) was classified based on the plasticity index (IP), where clay, loam and sandy loam were attributed to different categories. The Vasilyev cone determined the liquid limit. In the subsequent classifications *LST 1445:1996* and *LST EN ISO 14688–2:2007*, the fine soil was attributed to categories conforming to the grain size distribution, taking into account the grain size (< 0.063 mm) and classified as sandy silty clay, sandy clayey silt, taking into account the quantity of clay particles (< 0.002 mm) in the soil. In 2019 the classification of fine soils had changed again. They were classified by the Cassagrande graph based on the liquid limit (w_L) and plasticity index (IP). Clay and silt are classified concerning line A and are further classified by the plasticity index into low, medium, high, and very high plasticity soils.

Usually, the liquid limit is estimated, confirming the falling cone method *LST EN ISO 17892–12:2018* (Table 1). This method is slightly different from the Vasiliev cone (*GOST 5180–84 Grunty. Metody laboratornogo opredelenija fizičeskih harakteristik*). The test procedures and depth of cone penetration differ as well. In the first case, the liquid limit is estimated when the cone penetration depth is 20 mm, in the second case – 10 mm. The tests showed that the difference in cone design has a negligible impact on test results, a similar result is obtained, and a difference is less than 1% (Di Matteo, 2012; Spagnoli, 2012).

However, there is a difference in the soil matrix investigated by the falling cone and Vasiliev methods. The fine soil is sieved through a sieve of 0.4 mm for the falling cone test procedures. The soil is sieved through a 1 mm sieve during the Vasiliev test procedure. As a result, different soil mixtures are tested, where in the first case, the soil is finer, in the second – coarser, which is very important in determining the name of the soil. In this case, the difference among results is more significant, reaching 8%. A comparison of the Casagrande plate method (*ASTM D3418 – 17e1 Standard Test Methods for Liquid Limit, Plastic Limit,*

Table 1. The main peculiarities of soil classifications standards

Standard	Determination method of W_L (or *LL*)	Fines, mm	Fine – coarse soil limit, %	Main fine soil classification index
GOST 25100–82	Vasilyev cone (*GOST 5180–84*)	< 0.05 (< 0.1)*	fines > 50%	I_P
LST 1445:1996	Falling cone method (*LST EN ISO 17892–12:2018*)	< 0.063	fines > 50%	I_P
LST EN ISO 14688–2:2007		< 0.063	fines > 40%	grain size
LST EN ISO 14688–2:2018		< 0.063	fines > 50%	I_P, w_L
EGG soil classification (Lietuvos geologijos tarnyba, 2019)		< 0.063	fines > 35%	I_P, w_L
ASTM D2487–17e1	Casagrande plate (*ASTM D3418–17e1*)	< 0.075	fines > 50%	PI, LL

Note: *based on *GOST 25100–82*, fine soil particles are particles smaller than 0.05 mm, but in the description of silty sand, the size of fine soil particles is indicated from 0.1 mm.

and Plasticity Index of Soils) with the falling cone method (*LST EN ISO 17892–12:2018*) gives minor differences in the liquid limit (Table 1), as it uses the same soil sieved through a 0.4 mm sieve. In general, the w_L value obtained by the falling cone method is up to 5% higher than the liquid limit determined by the Cassagrande method. There is a close linear relationship among these values, where the correlation coefficient is close to 1.0 (Canelas et al., 2018). It is known that Young's modulus is a ratio of the variation of principal stress by the linear strain obtained in the same direction, with the other principal stresses remaining unchanged. During the calculations of Young's modulus from CPT data confirming correlations (Equations (1)–(5)), it is necessary to know in which load range the deformation modulus was evaluated. The magnitude of the vertical load is another important factor in estimating Young's modulus (Tamošiūnas et al., 2020). Knowing that the correlations were made with the modulus of deformation obtained by calculating the results of the static plate load tests, a statement is drawn that the limits of the load range are from 0.05 MPa to 0.3 MPa. These loads were typically applied for tests under *GOST 12374–77 Grunty. Metod polevogo ispytanija statičeskimi nagruzkami procedure*. The result of the plate load test is interpreted as a general modulus of deformation. Alternatively, it is considered the strain level (the average ratio of settlement over plate diameter) or the applied pressure over the limit pressure.

2. Methodology for calculating the soil Young's modulus in the USA and Europe

This paragraph presents Young's modulus calculation methods based on cone penetration data used in the USA and Europe. It is essential to know that to apply the formulas for the calculation of Young's modulus confirming the CPT of moraine clays. It is necessary to check whether the formulas presented in the literature are suitable for conventional (normally-consolidated) clays or not (Radaszewski & Wierzbicki, 2019). The physical and mechanical properties of normally-reconsolidated or low-reconsolidated soils (Bagheri & Rezania, 2021; Emmanuel et al., 2019; Gundersen et al., 2019; Khan et al., 2019) directly are not applied to moraine soils (Lekstutytė et al., 2019; Peri et al., 2019) are primarily reported in the literature.

2.1. Young's modulus

The most common modulus in geological engineering reports is Young's modulus E (in some literature – general Young's modulus). Young's modulus E is directly related to the constrained (oedometric) modulus D or E_{oed} by Poisson's ratio ν. The relationship among these moduli is expressed as follows (Di Matteo, 2012) (Equations (6)–(7)):

$$E = \beta \cdot E_{oed}, \tag{6}$$

$$\beta = 1 - \frac{2\nu^2}{1-\nu}. \tag{7}$$

Using the results of the CPT, the Minnesota Department of Transportation (Dagger et al., 2018) for coarse soils proposes to determine Young's Modulus E (Equation (8)):

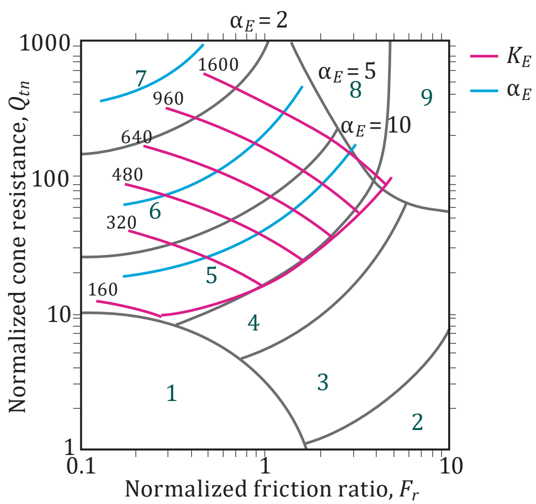
$$E = \frac{E_{oed}}{1.1}, \tag{8}$$

where E_{oed} – constrained (oedometric) modulus, MPa.

Robertson (2009; 2012) and Robertson & Cabal (2015) propose the Equation (9) for Young's modulus E for coarse soils:

$$E' = K_E p_a \left(\frac{\sigma'_{v_0}}{p_a} \right)^n, \tag{9}$$

where p_a – atmospheric pressure, kPa; K_E – Young's modulus number from the graph (Figure 1); σ'_{v_0} – effective vertical stresses, kPa; n – stress exponent, for coarse soils $n = 0.5$; F_r and Q_{tn} are determined by Equations (16) and (17); α_E – by Equation (11).



Note: 1 – sensitive, fine-grained;
2 – organic soils – clay;
3 – clay – silty clay to clay;
4 – silt mixtures – clayey silt to silty clay;
5 – sand mixtures – silty sand to sandy silt;
6 – sands – clean sand to silty sand;
7 – gravelly sand to dense sand;
8 – very stiff sand to clayey sand;
9 – very stiff, fine-grained.

Figure 1. Young's modulus number K_E setting graph

Robertson (2009, 2012) proposed other Equations (10)–(11) for coarse soils:

$$E' = \alpha_E \left(q_t - \sigma_{v_0}\right), \tag{10}$$

$$\alpha_E = 0.015\left[10^{(0.55 I_c + 1.68)}\right], \tag{11}$$

where α_E – modulus factor determines Young's modulus; q_t – total cone tip resistance, MPa; σ'_{v_0} – effective vertical stresses, MPa; I_c – modulus factor in determining Young's modulus is obtained using Equation (15).

2.2. Oedometric (constrained) deformation modulus

Young's modulus of one of the soil that is calculated using the results of the CPT is the constrained modulus D, commonly referred to in Lithuania and Europe as the Oedometric modulus E_{oed}. The most commonly found formula for calculating the constrained modulus E_{oed} in literature (Abu-Farsakh et al., 2007; Dagger et al., 2018; Kulhawy & Mayne, 1990) consists of the modulus factor α, the total cone tip resistance q_t, the vertical stresses in the soil. Moreover, it is calculated as follows (Equation (12)):

$$E_{oed} = \alpha\left(q_t - \sigma_{v_0}\right). \tag{12}$$

The vertical stresses in soil depend on the unit weight of the soil and the height of the soil column above the test point. The total cone tip resistance is calculated (Equation (13)):

$$q_t = q_c + u_2 \cdot (1 - a), \tag{13}$$

where q_t – total cone tip resistance, MPa; q_c – cone tip resistance, MPa; u_2 – porewater pressure acting behind the cone tip, MPa; a – cone area ratio, recommended value $a = 0.8$.

The modulus factor α determines the type of soil and the calculated situation to which the formula is applied. The Minnesota Department of Transportation (Dagger et al., 2018) suggests using $\alpha = 5.0$ for all soil types. In overconsolidated clays, Kulkhawy & Mayne (1990) proposed the use of $\alpha = 8.25$. In saturated clays, for the calculation of sediment due to consolidation, Abu-Farasakh et al. (2007) suggested the use of $\alpha = 3.6$. For all soil types, Robertson (2009) and Robertson & Cabal (2010) proposed the following method for determining the modulus factor (Equations (14)–(17)):

– if $I_c > 2.2$ use the value:
  $\alpha = Q_{tn}$ when $Q_{tn} \le 14$,
  $\alpha = 14$ when $Q_{tn} > 14$;
– if $I_c < 2.2$ use $\alpha = 0.03\left[10^{(0.55 I_c + 1.68)}\right]$, (14)

where I_c – index of soil behaviour type; Q_{tn} – first normalised cone parameter (Robertson, 1990).

$$I_c = \left[\left(3.47 - \log Q_{tn}\right)^2 + \left(\log F_r + 1.22\right)^2 \right]^{0.5}, \tag{15}$$

$$Q_{tn} = \frac{q_t - \sigma_{v_0}}{\sigma'_{v_0}}, \tag{16}$$

$$F_r = \left[f_s \left(q_t - \sigma_{v_0} \right) \right] \cdot 100\%, \tag{17}$$

where F_r – second normalised cone parameter; σ'_{v_0} – effective vertical stresses in the soil, kPa; f_s – sleeve friction, kPa.

The report of the US National Cooperative Highway Research Program (Mayne, 2007) for various types of soils suggested the use of:

- $\alpha = 5.0$ for soft to firm "vanilla clays" and normally consolidated "hourglass sands";
- $\alpha = 1...2$ for organic plastic clays of Sweden;
- $\alpha = 10...20$ for cemented (Fucino) clays.

The values proposed in the report of the Wisconsin Highways Research Program (Schneider & Hotstream, 2011):

- $\alpha = 1...2$ for soft high plasticity clays;
- $\alpha \leq 10$ for cemented clays;
- $\alpha = 2.3$ for lightly overconsolidated and silty clays (Tonni & Gottardi, 2011).

Although the imperial measurement system is used in the USA, units of the SI measurement system often being used for the above expressions.

In Europe, and more specifically in the European Union, Eurocodes are applied to design buildings and structures. *LST EN 1997–2:2009 Eurokodas 7. Geotechninis projektavimas. 2 dalis. Pagrindo tyrinėjimai ir bandymai* (Table 2) provides a calculation of the constrained, so-called oedometric modulus, which, unlike the above expression, uses cone tip resistance q_c instead of total cone tip resistance q_t. The Equation (18) used is as follows:

$$E_{oed} = \alpha \cdot q_c, \tag{18}$$

where α – correlation coefficient for different soils based on local experience.

The Swedish Geotechnical Institute presented a report (Nhuan, 1981) proposing a method for determining the correlation coefficient (Equation (19)):

$$\alpha = \frac{2.3(1+e)}{C_c} \cdot \frac{\delta_0}{q_c}, \tag{19}$$

where q_c – cone tip resistance, MPa; C_c – soil compressibility index; δ_0 – effective soil vertical pressure, kPa; e – pore volume.

2.3. Residual deformation modulus

The residual modulus is one of the key parameters in evaluating and designing a basis subjected to cyclic loads (Buchanan, 2007). In foreign literature, this module is often marked M_R, in Lithuania and Europe – E_R by *LST EN 13286–7:2004 Birieji ir hidrauliniais rišikliais sujungti mišiniai. 7 dalis. Biriųjų mišinių periodinės apkrovos triašis bandymas*. This parameter is commonly determined using the results of the CPT.

The Minnesota Department of Transportation (Dagger et al., 2018) for all soil types proposes to determine the residual modulus M_R (Equation (20)):

$$M_R = \left(1.46 q_c^{0.53} + 113.55 f_s^{1.4} + 2.36\right)^{2.44}, \tag{20}$$

where f_s – sleeve friction, kPa.

The report of the Wisconsin Highways Research Program (Puppala, 2008; Schneider & Hotstream, 2011) proposed the determination of the residual modulus M_R for all soil types:

$$\frac{M_R}{\sigma_3^{0.55}} = \frac{1}{\sigma_1}\left(47 q_c + 170.4 \frac{f_s}{w}\right) + 1.7 \frac{\gamma_d}{\gamma_w}, \tag{21}$$

Table 2. Correlation coefficient values*

Soil	q_c	Correlation coefficient α
Low–plasticity clay	$q_c \le 0.7$ MPa	$3.0 < \alpha < 8.0$
	$0.7 < q_c \le 2.0$ MPa	$2.0 < \alpha < 5.0$
	$q_c \ge 2.0$ MPa	$1.0 < \alpha < 2.5$
Low–plasticity silt	$q_c < 2.0$ MPa	$3.0 < \alpha < 6.0$
	$q_c \ge 2.0$ MPa	$2.0 < \alpha < 5.0$
Very plastic clay	$q_c < 2.0$ MPa	$2.0 < \alpha < 6.0$
Very plastic silt	$q_c \ge 2.0$ MPa	$1.0 < \alpha < 2.0$
Very organic silt	$q_c \le 1.2$ MPa	$2.0 < \alpha < 8.0$
Peat and very organic clay	$q_c \le 0.7$ MPa	$1.5 < \alpha < 4.0$
	$50 < w \le 100$	$1.0 < \alpha < 1.5$
	$100 < w \le 200$	$\alpha < 0.4$
	$w > 300$	
Chalks	$2.0 \text{ MPa} < q_c \le 3.0$ MPa	$2.0 < \alpha < 4.0$
	$q_c \ge 3.0$ MPa	$1.5 < \alpha < 3.0$
Sands	$q_c < 5.0$ MPa	$\alpha = 2.0$
	$q_c > 10.0$ MPa	$\alpha \le 1.5$

Note: *by LST EN 1997–2:2009

where M_R – residual deformation modulus, MPa, σ_1 – vertical normal stress, kPa; σ_3 – horizontal normal stress, kPa; f_s – sleeve friction, MPa; w – soil moisture, γ_d – dry unit weight, kN/m^3; γ_w – water density, kN/m^3; q_c – cone tip resistance, MPa.

The Minnesota Department of Transportation back in 2011 (Dehler & Labuz, 2007) provided a way to determine the residual deformation modulus M_R for fine and coarse soils found in Louisiana (Mohammad et al., 1999):

– for fine soils (Equation (22)):

$$\frac{M_R}{\sigma_3^{0.55}} = \frac{1}{\sigma_1}\left(31.79q_c + 74.81\frac{f_s}{w}\right) + 4.08\frac{\gamma_d}{\gamma_w}, \tag{22}$$

– for fine soils exposed to transport loads (Equation (23)):

$$\frac{M_R}{\sigma_3^{0.55}} = \frac{1}{\sigma_1}\left(47.03q_c + 170.40\frac{f_s}{w}\right) + 1.67\frac{\gamma_d}{\gamma_w}, \tag{23}$$

– for coarse soils (Equation (24)):

$$\frac{M_R}{\sigma_3^{0.55}} = 6.66\frac{q_c\sigma_b}{\sigma_v^2} - 32.09\frac{f_s}{q_c} + 0.52\frac{\gamma_d}{w\gamma_w}, \tag{24}$$

where σ_b – volume stress, the sum of essential stresses, kPa.

3. Interpretation of results

Equations (1)–(5) are applied for soils that have been classified to the previously valid *GOST 25100–82* standard. In all the soil classification standards discussed in this paper, coarse soils – sand and gravel, are classified based on the particle size distribution (Figure 2). The basic name of the coarse soil (in this case, sand and gravel) is used when the coarse soil fraction is no less than 50% of the total weight of the sample. Therefore, the application of the Equations proposed by Brillingas (1988) to coarse soils – sand and gravel, is appropriate only if the particle size of the dominant fraction of the studied soil, which determines the main name of the soil, is not smaller and not bigger than particle sizes of the same name by *GOST 25100–82* (Figure 2). The definition of gravel particles for all discussed standards in this paper (*ASTM D2487–17 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System*; *LST EN ISO 14688–1:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikacimas. 1 dalis. Indetifikavimas ir aprašymas*; *LST EN ISO 14688–2:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikacimas. 2 dalis. Klasifikavimo principai*; Lietuvos geologijos tarnyba, 2019; *LST 1445:1996*

Geotechnika. Gruntų klasifikacija ir identifikacija) corresponds to that defined in *GOST 25100–82* (Figure 2). Therefore, Equations (4) and (5) proposed by Brillingas (1988) are applicable if the main name of the soil is gravel, according to the standards mentioned above. Equations (4) and (5) are also applicable to the main soil sand if the particle size (no less than 50%) of the predominant fraction is between 0.1 mm and 2.0 mm, according to the standards mentioned above. *GOST 25100–82* specifies that fine soil particles are particles smaller than 0.05 mm, but the size of fine soil particles in silty sand is specified from 0.1 mm.

Given the differences in classification, it is clear that using Equations (1)–(5) in the recommendations EGG surveys (Lietuvos geologijos tarnyba, 2015) for fine soils in current geological engineering and geotechnical surveys is problematic and may be erroneous. It is necessary to perform an additional correlation analysis of the classifications to establish the connections between the previously used names (clay, loam, sandy loam) and the current names of the fine soil.

Despite the mentioned discrepancies, the above Equations (1)–(5) continue to be used in modern EGG surveys, and their recommendations (Lietuvos geologijos tarnyba, 2015) are applied together with *SN 448–72 Ukazanija po zondirovaniju gruntov dlja stroitel'stva*, which is no longer valid in Lithuania. At the time of issuing the recommendations (Lietuvos geologijos tarnyba, 2015), *LST EN ISO 14688–2:2007* was valid, where all soils were classified by the particle size distribution and possible discrepancies in soil composition was not taken into account when classifying them according to the *GOST 25100–82* methodology.

In order to solve the problems that have arisen due to the change in classifications, it is necessary to set an indicator that has little change over time and is independent of the classification of soils. One such

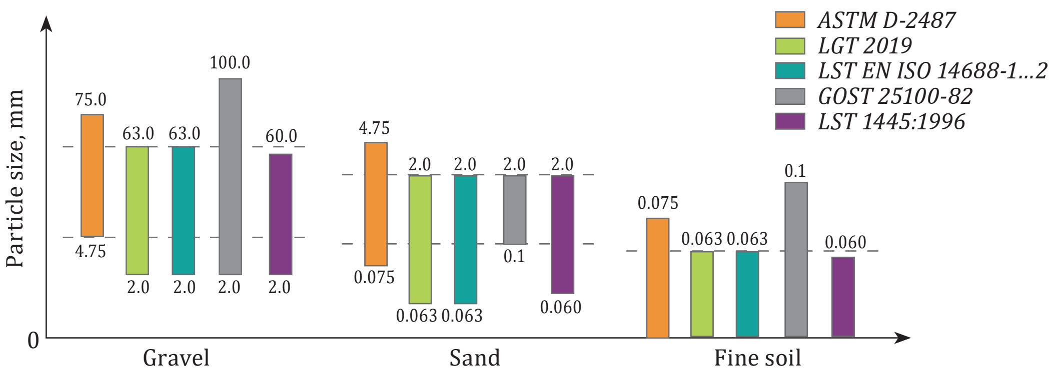


Figure 2. Comparison of gravel, sand, and fine soil fractions confirming different normative documents

indicator is the plasticity index (IP), which determination is practically the same for all methodologies. Only the test soil composition differs due to the use of different sieves. The primers identified by *GOST* standards is coarser because of a coarser sieve (0.1 mm). As already mentioned, the difference in plasticity index is up to 8% on average. Primers identified according to *GOST* standards are larger due to the larger sieve (0.1 mm). As already mentioned, the difference in the plasticity index is up to 8% on average. After determining the IP indicator, the soil is assessed according to the *GOST* methodology (loam, sandy loam or clay). It is also important to determine the genesis of the soil. Equations (1)–(5) proposed by Brillingas (1988) can be used to calculate Young's modulus with a small error. It is also important to determine the genesis of the soil. Then, with a slight error, Equations (1)–(5) proposed by Brillingas (1988) are possible to apply to calculate Young's modulus.

Another independent indicator is the ratio between cone tip resistance (q_c) and sleeve friction (f_s) – the friction index (R_f) measured during the CPT. This indicator shows the behaviour of the soil during mechanical action and is widely used for soil identification. Brillingas (1988) presented the q_c/f_s ratio values obtained with the CPT probes PIKA–9 and PIKA–II used at that time, correlating with *GOST 25100–82* and classified soil names (Table 3). Brilingas (1988) calls this classification lithological.

Correlation analysis would better assess possible discrepancies with the currently used soil classification and would help refine the use of equations range. In cases where EGG surveys are performed at the site and no data from previous studies are available, the correlation dependences given in the EGG survey recommendations (Lietuvos geologijos tarnyba, 2015) (Equations (1)–(5)) with the limitations

Table 3. Lithological classification of Lithuanian soils confirming cone penetration data*

Soil name by *GOST 25100–82*	$\frac{q_c}{f_s}$	$\frac{f_s}{q_c}$	*Ip* *GOST 25100–82*
Sand	>100	<0.01 (1%)	non-plastic
Sandy loam	100–71	0.01–0.014 (1.0–1.4%)	I_P < 7%
Loam	71–22	0.014–0.045 (1.4–4.5%)	7 < I_P < 17%
Clay	<22	>0.045 (>4.5%)	I_P > 17%

Note: *based on Brilingas (1988).

mentioned in this section can be used for simple and rapid estimation of soil Young's modulus. However, suppose the Young's modulus is to be properly assessed. In that case, the intact cores must be taken, and soil compressibility tests with a compression apparatus (oedometer) or triaxial pressure tests must be performed. It is possible to determine the history of stresses and their dependence on load only in this case. In modern geotechnical problem calculation and modelling programs, it is possible to estimate the change of the soil deformation modulus due to load. This method makes it possible to estimate the influence of the current or future structure on the soil base and the change of its properties with sufficient accuracy and with a small error. The same statements are made in the descriptions of generally accepted and widely used geotechnical software using three–dimensional (Plaxis 3D Foundation, 2007) or two-dimensional (GEO5, 2020) soil modelling. The soil 3D models are a qualitative representation of soil behaviour, whereas the model parameters qualify the soil characteristics. Moreover, the accuracy at which reality is approximated depends highly on the expertise of the user regarding the modelling of the problem, the understanding of the soil models and their limitations, the selection of model parameters, and the ability to judge the reliability of the computational results (Plaxis 3D Foundation, 2007). Using simplified programs (GEO5, 2020), which allow the assessment of soil only based on CPT data, calculations are performed by *LST EN 1997–2:2009* confirming to Robertson & Cabal (2010), i.e., conforming to the values of cone tip resistance and sleeve friction. In this case, regardless of the name of the soil, the calculations based on the CPT's data are made for the soil, which behaves like the soil whose name is determined, confirmed in Figure 2. For example, sometimes fine sands with a small amount of clay are classified as silt. In this case, Young's modulus of soil is determined by the silt and undetermined by the sand with a small admixture of clay and silt Equations.

Thus, depending on the EGG conditions and the complexity of the geotechnical calculations performed, the simplified methods based on CPT data are used to determine soil Young's modulus, provided that the current situation is uncomplex. In the case of a complex geological situation or a complex geotechnical calculation, it is proposed to apply complex methods that directly determine soil Young's modulus.

4. Case study

This section presents coarse and fine soils' main problems, determining the soil name and interpreting Young's modulus, oedometric modulus, and residual modulus. Soils presented in Tables 4–7 are taken from the different construction sites in the territory of the Republic of Lithuania. Based on the genesis, these soils are classified as glaciofluvial and glacial Upper and Middle Pleistocene deposits. The deposits consist of sands and fine till (moraine) soils (fine soils are much more common than coarse soils). These deposits are primarily met in construction areas in Lithuania (Satkūnas, 2009).

It is seen that there are almost no difficulties to determine the name of coarse soil based on different standards analysing results presented in Table 4. For coarse soils, confusing results appear when it is necessary to obtain deformation modulus confirming cone penetration data (Table 5). Here Young's modulus variations are considerable, and if Young's modulus of the soil is determined by value for Lithuanian coarse soil types, it is recommended to use Equation (4) (Table 5). Equations (9) and (10) (Table 5) are unsuitable for Lithuanian coarse soil types mainly due to different geological soil formation and over consolidation ratios.

Determining oedometric modulus (Table 5) by q_c value for Lithuanian coarse soil types, it was observed that only Equation (12) gives higher oedometric modulus values than Young's modulus obtained from Equation (4). Usually, Young's modulus E is directly related to the constrained (oedometric) modulus E_{oed} by the Poisson's ratio ν, as described in Equations (6) and (7). So, based on existing investigations experience for Lithuanian coarse soil types, it is suggested to use

Table 4. Results of determining the name of coarse soils based on different standards

No.	Depth, m	Soil fraction, %								
		<0.06	**0.06–0.106**	**0.106–0.212**	**0.212–0.300**	**0.300–0.6**	**0.6–1.0**	**1.0–2.0**	**2.0–4.75**	**>4.75**
1	3.7	0.7	1.4	3.0	0.4	43.8	36.0	2.9	11.9	0.0
2	2.4	0.1	0.1	1.2	0.9	53.1	5.8	4.0	2.6	32.3
3	1.5	4.1	2.1	16.2	42.2	35.3	0.1	0.0	0.0	0.0
4	3.0	1.7	1.2	5.0	6.4	26.5	24.7	19.5	11.4	3.5
5	4.5	2.9	1.3	2.7	3.3	24.8	28.1	22.3	10.2	4.5
6	8.3	2.1	0.7	2.1	5.2	33.1	15.3	11.1	9.4	21.0
7	1.8	3.6	2.1	5.6	3.8	11.6	1.2	7.6	7.9	56.7
8	4.8	7.3	7.4	29.9	14.6	22.2	6.3	4.4	2.9	4.9
9	10.0	1.3	1.1	4.9	6.4	34.4	30.0	16.9	3.7	1.4
10	4.4	4.3	1.5	2.9	3.0	14.2	10.1	16.2	21.9	25.9

Note: * confirming to Table 3.

End of Table 4. Results of determining the names of coarse soil based on different standards

No.	q_c, MPa	f_s, kPa	$\frac{q_c}{f_s}$	Soil name					
				Brilingas (1988)*	***GOST 25100–82***	***LST 1445:1996***	***ASTM D2487–17***	***LST EN ISO 14688–1:2018 LST EN ISO 14688–2:2018***	**Lietuvos geologijos tarnyba (2019)**
1	8.0	101.3	79.0	Loam	Medium coarse sand	Gravelly sand of uniform formation	Poorly graded sand	Poorly sorted sand	Evenly sorted sand
2	23.0	153.1	150.2	Sand	Gravely sand	Very gravelly sand of uniform formation	Poorly graded sand with gravel	Poorly sorted gravelly sand	Evenly sorted gravelly sand
3	8.2	127.0	64.6	Loam	Medium coarse sand	Sand of uniform formation	Poorly graded sand	Poorly sorted sand	Evenly sorted sand
4	12.0	95.0	126.3	Sand	Coarse sand	Gravelly sand of uniform formation	Poorly graded sand	Poorly sorted sand	Poorly sorted sand
5	3.0	14.0	214.3	Sand	Coarse sand	Gravelly sand of uniform formation	Poorly graded sand	Poorly sorted sand	Poorly sorted sand
6	17.0	146.0	116.4	Sand	Gravely sand	Very gravelly sand of uniform formation	Poorly graded sand with gravel	Poorly sorted gravelly sand	Poorly sorted gravelly sand
7	33.0	385.0	85.7	Sandy loam	Sandy gravel	Very sandy gravel of stepped formation	Well graded gravel with sand	Well graded sandy gravel	Well graded sandy gravel
8	29.0	465.0	62.4	Loam	Fine sand	Silty gravelly of uniform formation	Poorly graded sand with silt	Poorly graded silty sand	Poorly sorted sand with low fine fraction impurity
9	44.0	600.0	73.3	Sandy loam	Coarse sand	Gravelly sand of uniform formation	Poorly graded sand	Poorly sorted sand	Evenly sorted sand
10	75.0	622.0	120.6	Sand	Gravely sand	Very gravelly silty sand of uniform formation	Well graded sand with gravel	Medium graded gravelly sand	Medium graded gravelly sand

Note: * confirming to Table 3.

Equation (12), if the oedometric modulus is determined according to q_c value with the indirect method. Also, confirming experience related to soil residual modulus investigation (Skuodis et al., 2018), it is suggested to use Equation (24) (Table 5) for indirect residual modulus determination from q_c. Equation (24) (Table 5) gives reliable and conservative results in comparison with results obtained from Equation (20). Residual modulus determined confirming to Equation (21) (Table 5) has a higher variation compared to results obtained from Equation (20).

Analysis of fine soils results presented in Tables 6 and 7 showed that determining soil name based on different standards is more complicated than calculating deformation modulus. The principles of fine soil classification have changed over time. At the time when Brilingas (1988) was investigating and proposing Equations (1)–(4) in Lithuania, the *GOST 25100–82* standard was valid. Since then, the standards have changed four times, and the fine soil name and group changed as well. In some cases, fine soil became coarse soil – silty sand (in the case of *LST EN ISO 14688–2:2018* or *ASTM D2487–17*). Equation (4) is used to calculate silty sand Young's modulus and gives twice lower results than Equation (1). At present Lithuanian geological survey (LGS) proposed soil classifications methodology, which correlates with *GOST 25100–82* and Brilingas (1988) q_c/f_s (Tables 3 and 6) classifications are used in Lithuania.

Table 5. Results of determination of coarse soil deformation modulus

No.	Young's modulus E, MPa			Oedometric modulus E_{oed}, MPa			Residual modulus M_R, MPa		
	Equation (4)	Equation (9)	Equation (10)	Equation (12)	Equation (14)	Equation (18)	Equation (20)	Equation (21)	Equation (24)
1	34.1	130.6	118.8	39.7	110.6	13.6	375.6	87.5	124.7
2	72.3	106.6	405.9	114.8	319.9	34.5	>1000	186.8	119.6
3	34.7	83.1	68.5	40.9	136.9	13.8	535.0	127.9	84.2
4	45.5	117.6	158.5	59.7	166.7	18.0	431.0	109.5	119.1
5	17.0	28.0	27.2	14.6	54.4	6.0	57.5	39.8	134.0
6	58.3	195.6	636.1	84.3	236.0	25.5	947.0	111.1	187.4
7	93.4	93.6	930.3	164.8	456.6	49.5	>1000.0	351.5	123.6
8	85.2	152.8	>1000.0	144.5	399.5	43.5	>1000.0	246.5	159.0
9	114.5	223.4	>1000.0	219.0	607.6	66.0	>1000.0	251.8	231.3
10	114.5	268.1	>1000.0	218.6	598.9	66.0	>1000.0	321.7	267.1
11	167.3	148.2	>1000.0	374.6	>1000.0	112.5	>1000.0	454.0	197.0

Determined oedometric modulus confirming to q_c for fine soils is less than Young's modulus (Table 7). Since the determination of oedometric modulus from q_c is unreliable, it is not recommended to make such an indirect calculation of oedometric modulus for fine soils. Obtained Young's modulus from Equation (1) is more reliable than one obtained from Equation (9). Analysing residual modulus obtained values (Table 7), Equations (21) and (22) for fine soils indirect results interpretation from q_c are used.

Table 6. Results of determining the name of fine soils based on different standards

No.	Depth, m	W_L, %	I_P	Soil composition*, %				q_c, MPa	f_s, kPa
				Clay	Silt	Sand	Gravel		
1	1.4	20.2	7.7	10.7	25.8	60.7	2.8	2.0	43.0
2	6.2	18.0	6.9	8.1	34.1	54.4	3.4	2.5	65.0
3	5.0	17.9	6.8	7.9	34.3	53.5	4.4	1.4	30.0
4	4.5	20.1	9.0	9.2	27.2	61.4	2.2	2.0	52.0
5	18.5	19.3	8.2	9.3	28.5	56.9	5.3	2.4	44.0
6	9.7	18.6	8.0	8.2	31.0	58.9	1.9	2.8	75.0
7	12.6	19.4	8.4	9.9	26.6	59.7	3.8	2.5	54.0
8	5.4	27.6	15.0	28.1	32.7	33.6	5.6	2.8	107.0
9	7.2	28.6	15.0	20.2	40.3	33.1	6.4	2.5	78.0
10	5.2	26.2	13.0	20.0	40.5	35.1	4.4	1.6	57.0
11	5.0	27.3	15.0	17.6	27.3	36.0	19.1	1.6	47.0
12	6.8	19.3	8.6	9.3	29.4	58.5	2.8	6.2	178.0
13	14.0	18.4	7.8	8.6	36.1	52.8	2.6	2.4	43.0
14	14.3	18.5	7.0	7.6	27.6	53.1	11.7	3.3	65.0
15	11.3	19.1	7.7	9.7	28.7	58.8	2.9	2.9	50.0
16	3.7	21.8	9.7	9.2	26.8	61.2	2.9	2.4	56.0
17	5.8	19.4	8.1	10.2	27.7	59.0	3.1	4.6	116.0
18	2.8	22.6	10.6	12.7	38.1	47.9	1.4	5.7	200.0

Note: *soil description based on LST EN ISO 14688–1:2018, LST EN ISO 14688–2:2018.

End of Table 6. Results of determining the name of fine soils based on different standards

No.	Soil name					
	Brilingas (1988)**	***GOST 25100–82***	***LST 1445:1996***	***ASTM D2487–17***	***LST EN ISO 14688–1:2018 LST EN ISO 14688–2:2018***	**Lietuvos geologijos tarnyba (2019)**
1	Loam	Loam	Very silty sand	Silty sand	Silty sand	Sandy clay
2	Loam	Sandy loam	Very silty sand	Silty sand	Silty sand	Sandy clay or silt
3	Loam	Sandy loam	Very silty sand	Silty sand	Silty sand	Sandy clay or silt
4	Loam	Loam	Very silty sand	Silty sand	Silty sand	Sandy clay
5	Loam	Loam	Very silty gravelly sand	Silty sand	Silty sand	Sandy clay
6	Loam	Loam	Very silty sand	Silty sand	Silty sand	Sandy clay
7	Loam	Loam	Very silty sand	Silty sand	Silty sand	Sandy clay
8	Loam	Loam	Sandy clay	Sandy clay	Clay	Sandy clay
9	Loam	Loam	Sandy clay	Sandy clay	Clay	Sandy clay
10	Loam	Loam	Sandy clay	Sandy clay	Clay	Sandy clay
11	Loam	Loam	Sandy clay	Silty sand	Silty sand	Sandy clay
12	Loam	Loam	Very silty sand	Silty sand	Silty sand	Sandy clay
13	Loam	Loam	Very silty sand	Sandy clay	Silty sand	Sandy clay
14	Loam	Sandy loam	Very silty gravelly sand	Silty sand	Silty sand	Sandy clay or silt
15	Loam	Loam	Very silty sand	Silty sand	Silty sand	Sandy clay
16	Loam	Loam	Very silty sand	Silty sand	Silty sand	Sandy clay
17	Loam	Loam	Very silty sand	Silty sand	Silty sand	Sandy clay
18	Loam	Loam	Silty sandy clay	Sandy clay	Clay	Sandy clay

Note: ** confirming to Table 3.

Suppose a low geotechnical category is provided for engineering geological and geotechnical investigations on the construction site. In that case, it is possible to use an indirect method to interpret Young's modulus, oedometric and residual modulus. For high geotechnical category investigations, it is not recommended to use the only indirect method. The results must be validated with experimental investigations, such as the oedometer and triaxial tests. Only validation of indirect methods with experimental results shows which indirect method described in this paper the best correlates investigated construction sites. Classification of soil and selection of indirect method application for deformation modulus evaluation must be based on the soil age, genesis, and over consolidation ratio. These factors have a significant influence on the results obtained.

Conclusions and recommendations

Summarising the possibilities of determining soil Young's modulus presented in this paper from the cone penetration data, the following conclusions are drawn.

1. In simple geological situations where complex geotechnical calculations are not required (low geotechnical category), it

Table 7. Results of determination of fine soil deformation modulus

No.	Young's modulus E, MPa		Oedometric modulus E_{oed}, MPa			Residual modulus M_R, MPa		
	Equation (1)	Equation (9)	Equation (12) $\alpha = 5.0$	Equation (12) $\alpha = 8.25$	Equation (12) $\alpha = 3.6$	Equation (20)	Equation (21)	Equation (22)
1	22.0	25.4	10.0	16.5	7.2	74.6	47.4	46.5
2	25.7	53.5	12.5	20.6	9.0	123.8	66.0	99.1
3	17.6	16.0	7.0	11.6	5.0	49.3	49.4	87.7
4	22.0	30.4	10.0	16.5	7.2	88.4	52.9	83.2
5	25.0	30.8	12.0	19.8	8.6	82.9	85.5	179.1
6	27.9	22.3	14.0	23.1	10.1	155.0	72.6	126.1
7	25.7	25.4	12.5	20.6	9.0	101.4	75.2	145.2
8	27.9	16.6	14.0	23.1	10.1	265.3	65.1	92.2
9	25.7	19.2	12.5	20.6	9.0	156.1	62.9	107.2
10	19.0	16.3	8.0	13.2	5.8	88.9	52.1	89.7
11	19.0	16.0	8.0	13.2	5.8	73.0	50.6	87.8
12	53.1	18.7	31.0	51.2	22.3	911.3	98.5	105.5
13	25.0	26.8	12.0	19.8	8.6	81.4	75.6	153.7
14	31.6	27.1	16.5	27.2	11.9	140.1	81.3	155.7
15	28.7	24.1	14.5	23.9	10.4	101.4	73.0	136.9
16	25.0	41.3	12.0	19.8	8.6	103.2	54.7	75.3
17	41.2	17.2	23.0	38.0	16.6	363.2	81.3	96.5
18	49.4	12.0	28.5	47.0	20.5	>1000.0	107.2	69.0

is proposed to use the calculation formulas of the soil Young's module based on static cone penetration data, taking into account the exceptions and limitations described in this paper.

2. In case of complex geological conditions or necessary to perform complex geotechnical calculations (high geotechnical category), it is proposed to determine soil Young's modulus directly, i.e., perform compression tests on soil compressibility with an oedometer and triaxial pressure tests.
3. For coarse soils, almost no difficulties arise when it is necessary to determine soil names based on different standards presented in this paper. Confirming cone penetration data, the main uncertainty (variation of results) appears when evaluating Young's modulus, oedometric modulus, and residual modulus with indirect methods.
4. It is not very easy to determine the soil name for fine soils based on different standards presented in this paper. For example, granulometric soil composition is the same, but the soil names are different, i.e., loam, very silty sand, silty sand, and sandy clay. Determination of incorrect soil name leads to incorrect and improper cone penetration data usage to evaluate indirect soil Young's modulus.
5. It is proposed to carry out additional studies to clarify the original Equations that are used. Therefore, these additional studies would eliminate confusing interpretations of regulations, standards, and recommendations. Updated original Equations (and possibly the supplemented ones) would no longer be subject to the restrictions set out in this paper.

REFERENCES

Abu–Farsakh, M. Y., Zhang, Z., & Gautreau, G. (2007). Evaluating deformation modulus of cohesive soils from piezocone penetration test for consolidation settlement. *Transportation Research Record*, *2004*(1), 49–59. https://doi.org/10.3141/2004-06

ASTM D2487–17 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

ASTM D3418 – 17e1 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

Bagheri, M., & Rezania, M. (2021). Geological and geotechnical characteristics of London clay from the Isle of Sheppey. *Geotechnical and Geological Engineering*, *39*(2), 1701–1713. https://doi.org/10.1007/s10706-020-01572-3

Brilingas, A. (1988). Metodika inzhenerno – geologicheskih izyskanij dlja promyshlennogo i grazhdanskogo stroitel'stva v rajonah raspostranenija lednikovyh otlozhenij (na primere territorii Litovskoj SSR). Dissertacija na soiskanie uchenoj stepeni kandidata geologo–minearalogicheskih nauk. PNIIIS. Moskva (in Russian)

Bucevičiūtė, S., Marcinkevičius, D., & Dansevičienė, D. (1997). *Lietuvos inžinerinis geologinis žemėlapis*. Lietuvos geologijos tarnyba (in Lithuanian).

Buchanan, S. (2007). *Resilient modulus: what, why, and how*. Vulcan Materials Company. www.vulcaninnovations.com%2Fpublic%2Fpdf%2F2-Resilient-Modulus-Buchanan.pdf&clen=205345

Canelas, D., Fernandes, I., & da Graça Lopes, M. (2018). Use of Fall Cone Test for the determination of undrained shear strength of cohesive soils. *MATEC Web of Conferences, 251*, Article 04067. EDP Sciences. https://doi.org/10.1051/matecconf/201825104067

Dagger, R., Saftner, D., & Mayne, P. (2018). *Cone penetration test design guide for state geotechnical engineers* (Report No. 2018-32). Minnesota Department of Transportation.

Dehler, W., & Labuz, J. (2007). *Cone penetration testing in pavement design* (Report No. MN/RC 2007-36). Minnesota Department of Transportation Research Services Section.

Di Matteo, L. (2012). Liquid limit of low- to medium-plasticity soils: comparison between Casagrande cup and cone penetrometer test. *Bulletin of Engineering Geology and the Environment*, *71*(1), 79–85. https://doi.org/10.1007/s10064-011-0412-5

Emmanuel, E., Anggraini, V., Raghunandan, M. E., Asadi, A., & Bouazza, A. (2019). Improving the engineering properties of a soft marine clay with forsteritic olivine. *European Journal of Environmental and Civil Engineering*, *26*(2), 519–546. https://doi.org/10.1080/19648189.2019.1665593

GEO5. (2020). *Analysis of vertical load–bearing capacity and settlement of piles investigated on the basis of CPT tests*. Engineering manual No. 15.

GOST 12374–77 Grunty. Metod polevogo ispytanija statičeskimi nagruzkami (in Russian)

GOST 25100–82 Grunty. Klassifikacija (in Russian)

GOST 5180–84 Grunty. Metody laboratornogo opredelenija fizičeskih harakteristik (in Russian)

Gundersen, A., Hansen, R., Lunne, T., L'Heureux, J. S., & Strandvik, S. O. (2019). Characterisation and engineering properties of the NGTS Onsøy soft clay site. *AIMS Geosciences. 2019, 5*(3), 665–703. https://doi.org/10.3934/geosci.2019.3.665

Khan, M. S., Ivoke, J. A., & Nobahar, M. (2019). Progressive change in shear strength of Yazoo clay. *Geo–Congress 2019: Geotechnical Materials, Modeling, and Testing* (pp. 560–569). Reston, VA: American Society of Civil Engineers. https://doi.org/10.1061/9780784482124.057

Kulhawy, F. H., & Mayne, P. W. (1990). *Manual on estimating soil properties for foundation design* (No. EPRI-EL-6800). Electric Power Research Inst., Palo Alto, CA (USA); Cornell Univ., Ithaca, NY (USA). Geotechnical Engineering Group.

Lekstutytė, I., Gadeikis, S., Žaržojus, G., & Skuodis, Š. (2019). Engineering geological and geotechnical properties of till soil of the Middle Pleistocene glacial period. *Estonian Journal of Earth Sciences*, *68*(2), 101–111. https://doi.org/10.3176/earth.2019.09

Lietuvos geologijos tarnyba (LGT). (2019). *Inžinerinių geologinių ir geotechninių tyrimų gruntų klasifikacija*. Lietuvos Respublikos Aplinkos Ministerija (in Lithuanian).

Lietuvos geologijos tarnyba. (2015). *Projektinių inžinerinių geologinių ir geotechninių tyrimų rekomendacijos, I priedas*. Lietuvos Respublikos Aplinkos Ministerija (in Lithuanian).

LST 1445:1996 Geotechnika. Gruntų klasifikacija ir identifikacija (in Lithuanian).

LST EN 13286–7:2004 Birieji ir hidrauliniais rišikliais sujungti mišiniai. 7 dalis. Biriųjų mišinių periodinės apkrovos triašis bandymas (in Lithuanian).

LST EN 1997–2:2009 Eurokodas 7. Geotechninis projektavimas. 2 dalis. Pagrindo tyrinėjimai ir bandymai (in Lithuanian).

LST EN ISO 14688–1:2007 Geotechniniai tyrinėjimai ir bandymai. Gruntų atpažintis ir klasifikacimas. 1 dalis. Atpažintis ir aprašymas (in Lithuanian).

LST EN ISO 14688–1:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikacimas. 1 dalis. Indetifikavimas ir aprašymas (in Lithuanian).

LST EN ISO 14688–1:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikacimas. 1 dalis. Indetifikavimas ir aprašymas (in Lithuanian).

LST EN ISO 14688–2:2007 Geotechniniai tyrinėjimai ir bandymai. Gruntų atpažintis ir klasifikacimas. 2 dalis. Klasifikavimo principai (in Lithuanian).

LST EN ISO 14688–2:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikacimas. 2 dalis. Klasifikavimo principai (in Lithuanian).

LST EN ISO 14688–2:2018 Geotechniniai tyrinėjimai ir bandymai. Gruntų identifikavimas ir klasifikacimas. 2 dalis. Klasifikavimo principai (in Lithuanian).

LST EN ISO 17892–12:2018 Geotechniniai tyrinėjimai ir bandymai. Laboratoriniai grunto bandymai. 12 dalis. Takumo ir plastiškumo ribų nustatymas (in Lithuanian).

Mayne, P. W. (2007). *Cone penetration testing*. Transportation Research Board.

Mohammad, L. N., Titi, H. H., & Herath, A. (1999). Evaluation of resilient modulus of subgrade soil by cone penetration test. *Transportation Research Record*, *1652*(1), 236–245. https://doi.org/10.3141/1652-30

Nhuan, B. D. (1981). *Field testing – equipment, test methods and interpretation of test results*. Swedish Geotechnical Institute.

Peri, E., Nielsen, S. D., Nielsen, B. N., & Ibsen, L. B. (2019). Consequences of slenderness and boundary conditions in triaxial testing on the reliability of design parameters. *7th International Symposium on Geotechnical Safety and Risk* (pp. 370–375). Taipei, Taiwan. https://doi.org/10.3850/978-981-11-2725-0-IS12-13-cd

Plaxis 3D Foundation. (2007). *Material models. Manual. Version 2*.

Puppala, A. J. (2008). *Estimating stiffness of subgrade and unbound materials for pavement design*. Washington, DC: The National Academies Press.

Radaszewski, R., & Wierzbicki, J. (2019). Characterisation and engineering properties of AMU Morasko soft clay. *AIMS Geosci*, *5*(2), 235–264. https://doi.org/10.3934/geosci.2019.2.235

Robertson, P. K. (1990). Soil classification using the cone penetration test. *Canadian Geotechnical Journal*, *27*(1), 151–158. https://doi.org/10.1139/t90-014

Robertson, P. K. (2009). Interpretation of cone penetration tests—a unified approach. *Canadian Geotechnical Journal*, *46*(11), 1337–1355. https://doi.org/10.1139/T09-065

Robertson, P. K. (2012). Interpretation of in-situ tests – some insights. In *Proc. 4th Int. Conf. on Geotechnical and Geophysical Site Characterization–ISC'4*, *4*, 3–24. Recife, Brazil.

Robertson, P. K., & Cabal, K. L. (2010). *Guide to cone penetration testing for geotechnical engineering* (5th ed.). Gregg Drilling & Testing.

Robertson, P. K., & Cabal, K. L. (2015). *Guide to cone penetration testing for geotechnical engineering*. Gregg Drilling & Testing.

Satkūnas, J. (2009). *Lietuvos Kvartero stratigrafijos schema*. Vilnius, 13 pp. (in Lithuanian).

Schneider, J. A., & Hotstream, J. N. (2011). *Cone penetrometer comparison testing*. Wisconsin Highway Research Program.

Skuodis, Š., Karpis, R., Zakarka, M., Gedvilas, M., Raginis, V., Orlova, K., & Katauskas, M. (2018). Grunto, veikiamo periodinėmis apkrovomis, elgsenos tyrimai. *Geologija. Geografija*, *4*(4). (in Lithuanian) https://doi.org/10.6001/geol-geogr.v4i4.3888

SN 448–72 Ukazanija po zondirovaniju gruntov dlja stroitel'stva (in Russian)

Spagnoli, G. (2012). Comparison between Casagrande and drop-cone methods to calculate liquid limit for pure clay. *Canadian Journal of Soil Science*, *92*(6), 859–864. https://doi.org/10.4141/cjss2012-011

STR 1.04.02:2011 Inžineriniai geologiniai ir geotechniniai tyrimai (in Lithuanian).

Tamošiūnas, T., Skuodis, Š., & Žaržojus, G. (2020). Overview of Quaternary sediments deformation modulus dependency on the testing methodology. *Baltica*, *33*(2), 191–199. https://doi.org/10.5200/baltica.2020.2.6

Tonni, L., & Gottardi, G. (2011). Analysis and interpretation of piezocone data on the silty soils of the Venetian lagoon (Treporti test site). *Canadian Geotechnical Journal*, *48*(4), 616–633. https://doi.org/10.1139/t10-085

Urbanavičienė, V., & Skuodis, S. (2019). Lack of attention to geological conditions investing in land plot for construction. *Architecture, Civil Engineering, Environment*, *12*(4), 87–95. https://doi.org/10.21307/ACEE-2019-054

Žaržojus, G., & Dundulis, D. (2010). Problems of correlation between dynamic probing test (DPSH) and cone penetration test (CPT) for cohesive soils of Lithuania. *The Baltic Journal of Road and Bridge Engineering*, *5*(2), 69–75. https://doi.org/10.3846/bjrbe.2010.10

Žaržojus, G., & Kelevišius, K. (2016). Smėlio tyrimai patobulintu dinaminiu penetrometru. *Geologija. Geografija*, *2*(2), 84–91 (in Lithuanian). https://doi.org/10.6001/geol-geogr.v2i2.3320