Composition and Strength of Middle Pleistocene till in Lithuania

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ABSTRACT

In Lithuania, the upper part of the Earth's crust was formed during the Pleistocene. Only a small part of Lithuania is a relic of the previous Medininkai stage (Lonian) glaciation in the Middle Pleistocene (Chibanian Age), which occur on the surface only in the southeastern area. Medininkai glacial period till soils are an almost unstudied soil type in Lithuania. Due to geotechnical investigations on new construction sites, an opportunity appeared to provide experimental investigations with Medininkai glacial period till soils. One of the main challenges of this research is to collect a perfectly undisturbed sample that would reflect the in-situ conditions. The Medininkai glaciation till soil is a mixture of different portions of clay, sand, and gravel and are different from other detectable till soils globally and unique. The main purpose of this study is to explore and review the strength and deformation properties of till soils of the Medininkai glacial period. Triaxial testing and oedometer tests were used for soil investigation in order to achieve the aim of the study. During the in-situ tests, cone penetration tests were performed as well as the borehole data was described. In this paper, the most important researches were achieved due to comparison single-stage triaxial (SST) and multi-stage triaxial (MST) test methods applying different soil testing conditions. It was concluded that there are no significant differences, only small due to moisture content and drainage conditions. Also, based on different calculate method for OCR evaluation was determined that this till soil is over consolidated. In all cases, tested specimens must preserve similar composition and state to the in situ soil to obtain representative index and mechanical parameters to be used in geotechnical design

Keywords: Middle Pleistocene Till, Triaxial Test, Overconsolidation Ratio (OCR)

1 Introduction

1.1 Geology and Investigated Site

The upper part of the Earth's crust in Lithuania, as well as the geological environment, which is interesting from an engineering point of view, was formed during the Pleistocene glacial period (2588–12 thousand years BP) – i.e., during the longest Quaternary glacial period. Glacial and interglacial periods make the Quaternary period exceptional, as, during that time, Lithuania was covered several times by glaciers whose deposits cover the entire surface of Lithuania today. The average thickness of these deposits amounts to approximately 100 m, and the maximum thickness reaches more than 315 m (Bičkauskas, et al., 2011).

Generally, glacial deposits are very diverse, and their characteristics depend on the conditions of formation. Usually, most glaciogenic environments are mainly occupied by till deposits formed at the edges of sliding glaciers and beneath them. Available data shows that till soils formed throughout glacial periods are the most predominant across Lithuania; they make up 70% by volume and 41.3% of the prevalent Qaternary stratum (Putys, et al., 2010)

According to the stratigraphic scheme of the Lithuanian Quaternary period (Satkūnas, 2009), which is based on the age of soil formation, the largest geomorphological complexes of the country's relief

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were formed during the Upper Pleistocene Nemunas stage (Tarantian) glacial period. Only a small area of the Lithuanian relief is formed during the Middle Pleistocene Medininkai glacial period (Lonian; Figure 1).

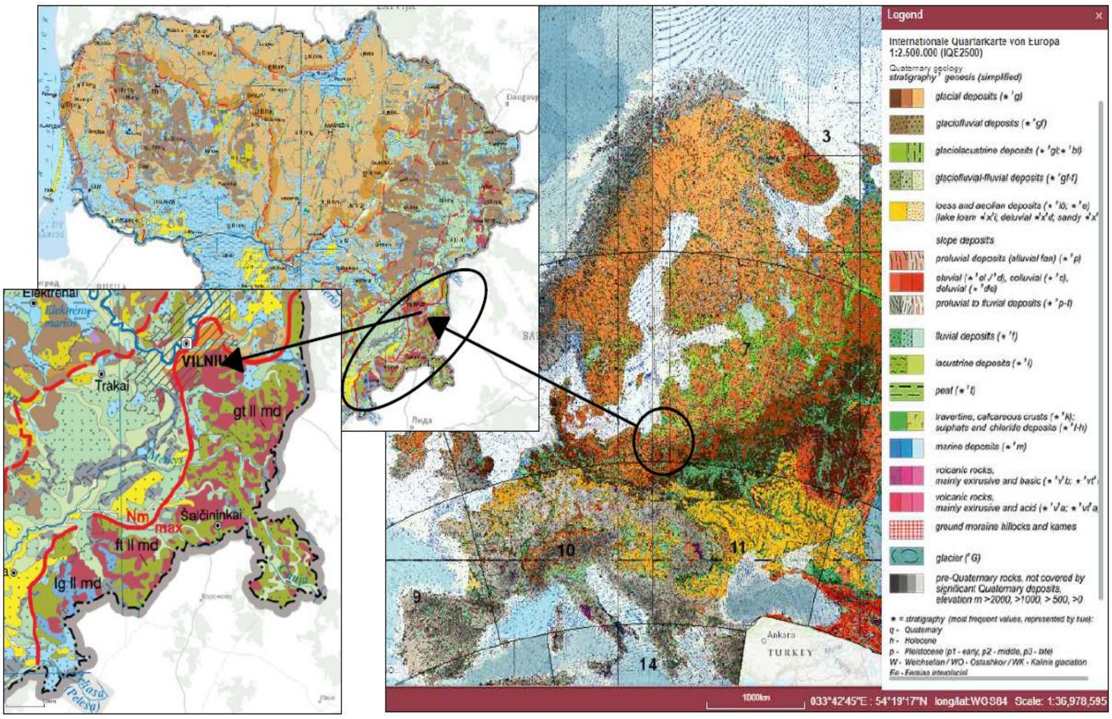

Figure 1: Middle Pleistocene Medininkai glacial (Lonian) in Lithuania and the site of the investigated soil (Guobytė, 1999; Geoviewer at https://geoviewer.bgr.de)

The Medininkai glacial period (195–128 thousand years BP) formed deposits with an average thickness of 30–40 m. The maximum thickness amounts to 50–100 m (Kavoliutė, 2012); however, the predominant layer is about 10–30 m thick (Grigelis, et al., 1994). In Lithuania, deposits from the Medininkai glacial period are widespread throughout the territory; but only in the southeastern part of the country, they outcrop across about 1459.6 km² – i.e., across 2.25% of the territory of Lithuania (Satkūnas, et al., 2007).

In this region, the glacial till soils of the Medininkai glacial period consist mainly of sandy clay and sandy silt. The mineral content shows that these deposits contain a high amount of cluster elements (Zr, Mn, Ti, Y, Yb, Pb), which are associated with weathering-resistant minerals. In contrast, the till deposits in other regions of Lithuania mainly contain lithogenic elements (Ga, Cr, Co, V, Ni), which are related to clay minerals (Bitinas, 2011).

The deformation and strength properties of the glacial till soils of the Medininkai glacial period have not been studied extensively, and, therefore, they are often characterized by properties of soils of different genesis. As these glaciogenic soils in question do not only cover a large area throughout Lithuania but also are often used as a medium for buildings, their structural parts, commercial deposits, etc., the study of the properties of these soils is essential for the country's economy. Also, on an international level, the results of this study are significant as glacial soils are difficult to study all over the world due to their property particularities.

1.2 Engineering Challenges

One of the major challenges of this study is to collect high-quality undisturbed samples which would reflect the real in-situ conditions. Usually, many problems arise at collecting high-quality samples from stiff and overconsolidated soil. The size of the sample, the sampling method, sample storage, and the transport methods have a major influence on the physical and mechanical soil properties, which are to be determined. Due to the distinctive structure of the till soil of interest in this study and the effect of sampling methods, it may be difficult to determine the exact properties of the soil. In geotechnical studies involving very stiff and overconsolidated cohesive soils, the most common problem relating to result accuracy is the occurrence of cracks in the soil sample and its potential for swelling. Geotechnical literature provides a variety of references on optimized soil sampling practices (Gaoshan, et al., 2019), but the impact of sampling on measured soil properties is still an issue that remains to be addressed.

1.3 Problem Statements

A typical problem with sampling is that the sampling itself disturbs the soil. Deep penetration into layers during soil sampling distorts the surrounding soil and engenders shear deformations. This disturbance can be so tremendous that the soil behavior in the laboratory differs significantly from in situ. Disturbances due to sampling in very stiff and overconsolidated cohesive soils are also thought to be due to microstructural damage (i.e., due to composition and bonding) and a result of effective stress variation compared to geostatic conditions (Tanaka, et al., 2006). As geostatic stresses are reduced to zero during sampling in situ, soil samples could potentially swell, resulting in a weaker soil structure. Moreover, the sample tends to lose a significant amount of its residual effective stresses instantly, so – as mentioned above – the sample may swell (Amundsen, et al., 2017; Berre, 2014). These processes start during drilling and continue while penetrating the soil and collecting samples as well as during the transportation of the sample to the laboratory, storing, preparing, and placing into testing apparatus (Tanaka, et al., 2006; Rocchi, et al., 2013). Therefore, disturbing the soil due to the entire sampling process is a considerable problem, resulting in difficulties while obtaining parameters of the soil reflecting reality. Many researchers have attempted to evaluate the effect of sampling disturbance on the mechanical properties of both intact soils in situ and laboratory-prepared normal and overconsolidated soils (Georgiannou, et al., 1994; Rahman, et al., 2010).

There are several studies for very stiff overconsolidated clays (Rahman, et al., 2010). Their results show that, due to disturbances created by sampling, initial effective stress (σ'_i), undrained shear strength (c_u), initial tangent modulus (E_i), and secant modulus (E_{50}) all decrease. Also, in the disturbed soil samples, lower pore pressure (u_0) and a higher value of axial strain (ε) are observed at the peak value of the deviator (Rahman, et al., 2010), changes related to the formed stresses are noticed in the overconsolidation ratio (OCR). Rahman, et al. (2010) and Krage, et al. (2016) suggest that in poorly collected soils, effective stress and the Skempton Pore Pressure Parameter at the peak deviator value (A_p) decreases proportionally with the increase of the OCR.

The correct results of the tested soil depend not only on the factors engendered during the sampling but also on the sensitivity, strength, porosity of the soil, its mesostructure (i.e., its cracks), the soil location environment, depth, aquifer, soil composition, amount of trace elements and their type.

The till soils of the Medininkai glacial period in the southeastern part of Lithuania are very stiff, overconsolidated, and widely known for their heterogeneous properties and complex soil structure. Sand inclusions, gravel, pebbles, and occasional larger gravel interlayers can affect the strength of the entire soil mass. Also, worth to mention that this soil due to its strength, low porosity and overconsolidation has low sensitive to pore pressure.

Therefore, when evaluating results, it is crucial to consider the influence of disturbances caused by soil sampling. Laboratory testing of soil samples requires a quantitative evaluation of the sample quality in order to evaluate the effectiveness of the test results in representing the in-situ soil properties. Empirical corrections, models, and simulations are proposed that "adjust" obtained results (Rocchi, et al., 2013; Nagaraj, et al., 2003). In this study, no corrections to results were applied yet. However, distortions in the results obtained from a potentially low-quality sample were taken into account, and the inaccurate results were eliminated.

The main purpose of this study is to explore, review and compare the strength and deformation properties of an almost unstudied till soil type in Lithuania. Taking into account main challenges like-complex till soil structure and composition as well as to collect a high-quality undisturbed sample that would reflect the in-situ conditions.

2 Investigation Site

The investigated soil is located in eastern Lithuania (Figure 2). Medininkai glacial period deposits are found superficially only in this part of the country; these deposits consist solely of glacial (g II md) and fluvioglacial (f II md) formations.

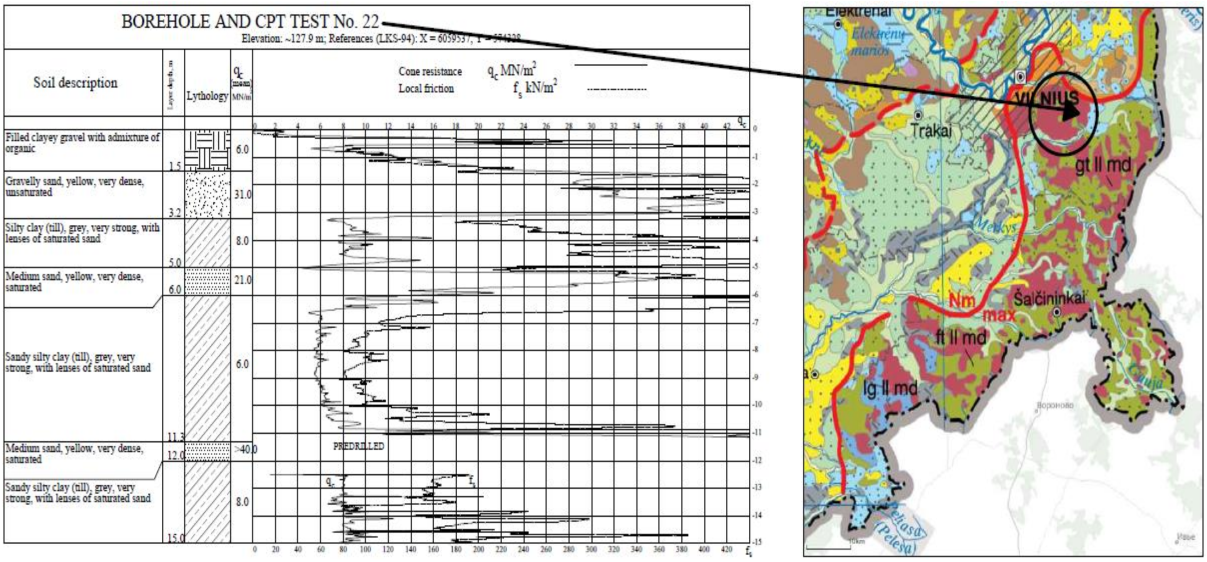

Figure 2: The site of the investigated soil and borehole and CPT test of sampling place (Gadeikis, et al., 2017)

Several field tests were performed in the research area: borehole drilling and cone penetration tests (Figures 2 and 3). Disturbed and undisturbed soil samples were taken for laboratory testing of physical and mechanical properties during drilling. The maximum depth reached with these surveys was 15 m. Hence, almost the entire depth of the Medininkai stratum layer was covered at this study site.

In order to obtain high-quality, undisturbed samples were used-Shelby tube sampling. This technic was used in order to recover intact samples that represent the in-situ soil density and moisture content. These two factors are obligatory to evaluate the most important soil engineering properties-strength, compressibility and density. The sample was subsequently extruded from the Shelby tube using an appropriately-sized hydraulic extruder and extrusion platen. There was sealed the top and bottom of the tube to prevent moisture loss, by spooning wax over the ends. Samples were kept in a container to avoid impacts of jarring or vibration until ready for testing.

Following the borehole information (Figure 3), glacial till (g II md) deposits predominate in the area under the fluvioglacial sediment (gravely sand; f II md). Most common are till-low plasticity sandy clays with medium sand interlayers and inclusions.

3 Methodology

Based on borehole drilling and cone penetration tests (CPT; Figures 2 and 3), the investigated glacial till soil from the Medininkai glacial period is encountered below 6.0 m of depth. Thus, samples with Shelby tubes for laboratory tests were taken from depths between 8.0 m and 15.0 m. According to CPT data, the studied soil is classified as very strong soil with cone resistance (q_c) > 4 MPa (LST EN 1997-2:2007).

Samples collected from depths between 8.0 m and 10.7 m have a q_c of about 6.0 MPa, whereas samples collected from deeper depths up to 15.0 m have a q_c ~ 8.0 MPa. The physical and mechanical properties of the soil were investigated during the research.

It is very important to mention that investigated Lithuanian till soil is poorly permeable and considered as partially saturated soil. In this case the principles of unsaturated soil mechanics are not studied. After the test B, it was obtained that the samples were not completely saturated. B value was from 0.65 to 0.70. This is the maximum values for these soil types. These values were achieved after three weeks of saturation process.

3.1 Investigation of Physical Properties

The following physical properties of the soil were determined for the studied soil: natural density, moisture content, plasticity, and liquid limits. Also, a grain size distribution analysis was conducted (Figure 4, Table 1). Laboratory tests were performed according to predefined standards (CEN ISO/TS 17892-12:2004; CEN ISO/TS 17892-4:2004).

3.2 Investigation of Mechanical Properties

Soil strength properties were determined by triaxial testing (LST EN ISO 17892-9:2018) using single-stage triaxial (SST) and multi-stage triaxial (MST) setups (Hormdee, et al., 2012; Shahin, et al., 2011) (Figure 3). The main differences in SST and MST tests are that in the SST test a constant confining (cell) pressure is applied for difference soil samples and the axial stress is increased until the sample achieve a failure. Considering the multistage triaxial test, where procedure uses one soil specimen that is consolidated under different confining pressures. Test include consolidation of the soil specimen and then stress deviator increase with constant vertical strain ramp until the soil specimen deforms plastically. This procedure is repeated for a second and third time under increased confining pressure. (Alsalman, 2015; Shahin, et al., 2011). In the SST test we can measure peak and residual values in the meantime, in MST test only peak shear values.

Within the SST test, all samples had a height-to-diameter-ratio of 2 (H = 100 mm, D = 50 mm). Here, two test series were performed by applying different testing methodologies. The first test series was performed in unsaturated consolidated undrained (UCU) triaxial conditions the confining pressure in this test were 160 kPa, 260 kPa, and 360 kPa. The second test series was performed in saturated consolidated drained (SCD) triaxial conditions, and consolidation stress was 200 kPa, 300 kPa, and 400 kPa. In the unsaturated condition test pore pressure was measured. Also, in both unsaturated and saturated test conditions water content was captured before test and after test in the dry soil. The test loads were selected to reflect the natural soil location conditions, considering that the soil can be loaded or unloaded by 100 kPa. The vertical axial deformation velocity in both SST test series was 0.002%/min (reaching a maximum of 15% vertical axial deformation).

Within the MST tests, samples of different height-to-diameter-ratio were analyzed using different test methodologies. The first test series was performed on samples with ratios H/D = 2 (H = 100 mm, D = 50 mm) and H/D = 2 (H = 200 mm, D = 100 mm) in unsaturated consolidated drained (UCD) triaxial conditions; the confining pressure in this test were 150 kPa, 250 kPa, and 350 kPa. The second test series was performed as well for samples with rations H/D = 2 (H = 100 mm, D = 50 mm) and H/D = 2 (H = 200 mm, D = 100 mm), but in unsaturated consolidated undrained (UCU) triaxial conditions. The loads in the UCU tests were 200 kPa, 300 kPa, and 400 kPa. Again, the test loads were selected to reflect the natural soil location conditions, considering that the soil can be loaded or unloaded by 100 kPa. The vertical axial deformation velocity in both MST test series was 0.027%/min (reaching a maximum of 15% vertical axial deformation).

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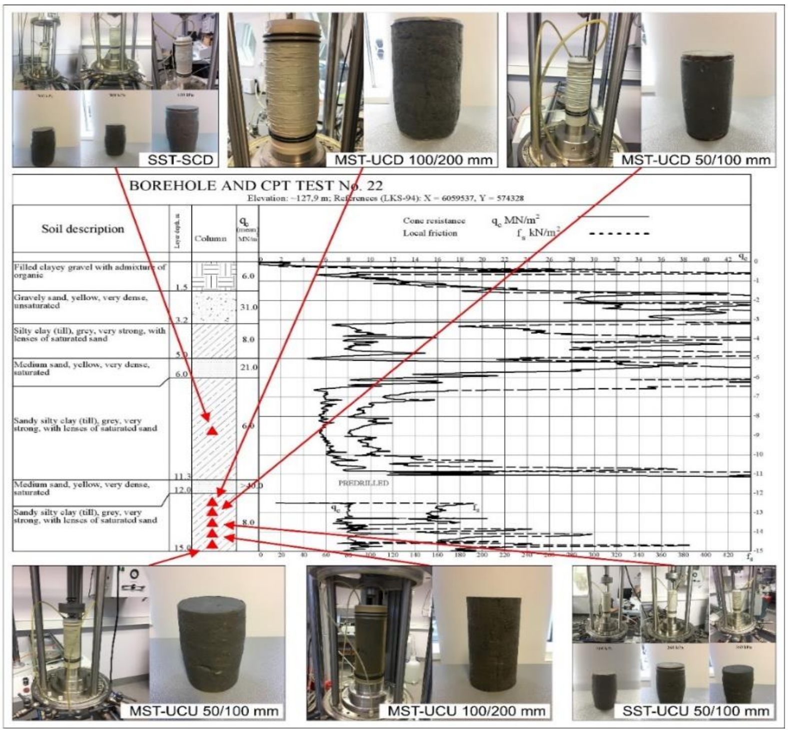

Figure 3: Borehole and CPT test of sampling place and triaxial test samples

Soil strength properties were estimated and calculated by applying several methodologies (LST EN ISO 17892-9:2018; Šimkus, 1987; Dirgėlienė, 2013; Dirgėlienė, 2007; Ho Chi Minh City University of Technology, 2016) using corrections of these methodologies as suggested Amšiejus, et al. (2010). Soil deformation properties were estimated by performing oedometer (OED) tests applying additional compression on the samples (CEN ISO/TS 17892-5:2004). The tests were performed on undisturbed soil samples with heights of 20 mm and diameters of 70 mm. Loads used in the tests were 50 kPa, 160 kPa, 360 kPa, 780 kPa, and 1610 kPa. The additional load was added every 24 hours (Table 3).

Based on the obtained results, OCR and secant moduli E50 were calculated by examining the deformation and strength properties. The secant modulus E50 is given by the ratio of the peak normal stress deviator to the corresponding deformation (Varga, et al., 2004; Hatanaka, et al., 2003); it was calculated from stress dependence on strain obtained from the single-stage triaxial and multi-stage triaxial tests. OCR were calculated from the OED tests implying gradual soil loading. Overconsolidation pressures were identified using the Casagrande graphical procedure (Jozsa, 2013; L'Heureux, et al., 2016) as well as results obtained from the oedometer moduli (E_{oed}) under different stresses (Józsa, 2016). Additionally, the OCR were calculated from the results obtained from SST and MST tests. The OCR calculation was based on the calculated secant modulus E50 values (Józsa, 2016) and applying the SHANSEP methodology with different coefficient values to evaluate the minimum and maximum values (Mayne, 1988; StroLyk, et al., 2014; Tankiewicz, et al., 2021). The OCR was also calculated based on CPT data by applying a calculation methodology for cohesive soils (Lunne, et al., 1997). This calculation methodology is based on the soil type and the plasticity index. To adapt it to Lithuanian till soils, the calculation corrections were introduced (Urbaitis, et al., 2016).

The results for strength properties calculated from SST and MST tests are compared with those values given in the literature (Bucevičiūtė, et al., 1997) that are often used and treated as appropriate. Deformation properties (E_{oed}) obtained from tests gradually loading the soil were compared with those calculated according to defined standards (CEN EN 1977-1:2004) and those given in the literature (Bucevičiūtė, et al., 1997).

4 Results and Discussion

4.1 Physical Properties from Laboratory Tests

Based on grain size distributions (Figure 4) and the results from the consistency limit identification in the laboratory (Table 1), the investigated soil is sandy low plasticity clay (saClL) (EN ISO 14688-1:2018; EN ISO 14688-2:2018).

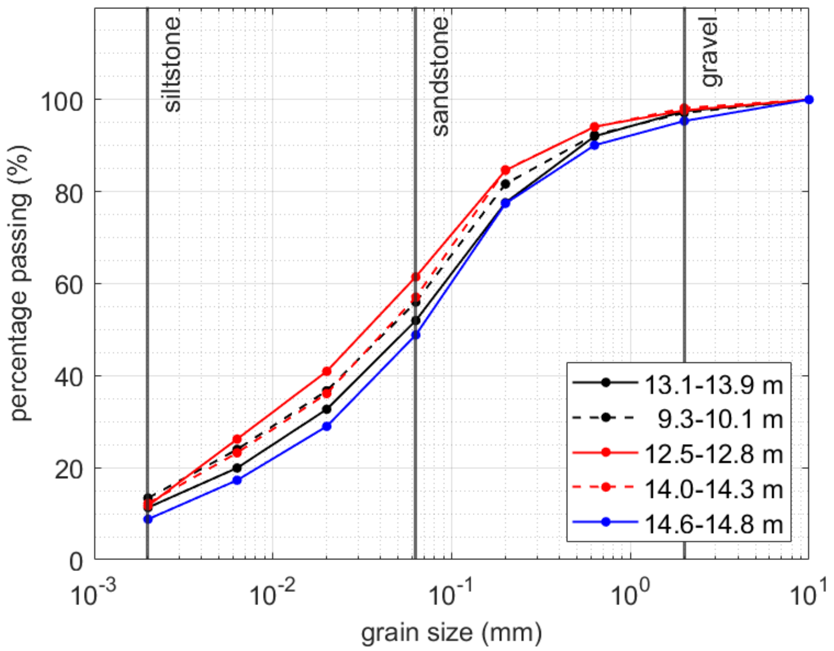

Figure 4: Grain size distributions for five samples from different depths

Table 1: Minimum/ maximum values of the different physical properties.

Bulk density (min/max)	Particle density	Moisture content	Void ratio	Plasticity index			
ϱ, g/cm^3	ϱ$_s$, g/cm^3	w (min/max) -	e (min/max) -	w_L (min/max) -	w_P (min/max) -	I_P (min/max) -	I_L (min/max) -
2.27/2.40	2.72	0.09/0.11	0.24/0.33	0.211/0.245	0.125/0.139	0.111/0.100	-0.118/-0.215

4.2 Mechanical properties from laboratory tests

Soil strength properties were analyzed by single (conventional) stage triaxial (S(C)ST) tests on samples with a depth-to-height-ratio of 50/100 mm and multi-stage triaxial (MST) tests on samples with different sizes (D/H = 50/100 mm and D/H = 100/200 mm) (Hormdee, et al., 2012; Shahin, et al., 2011).

Unsaturated consolidated drained (UCD) (Trinh, et al., 2006), unsaturated consolidated undrained (UCU) (Ding, et al., 2018) and saturated consolidated drained (SCD) (Lipinski, et al., 2010) test conditions were applied (Table 2).

Table 2: Comparison of shear strength in terms of peak friction angle (φ) and peak cohesion (c) obtained from multi-stage triaxial (MST) tests and single-stage triaxial (SST) tests.

	Triaxial test method	Sample size D/H, mm	ϕ_{peak}, °	c_{peak}, kPa
MST	UCD	50/100	25.53–28.40	36.57–37.03
		100/200	23.00–29.25	48.24–53.20
	UCU	50/100	19.87–21.29	35.40–35.88
		100/200	22.11–25.33	42.47–53.84
SST	UCU	50/100	23.50–25.81*	27.52–30.59*
	SCD	50/100	23.58–25.88*	22.67–28.03*
From literature	(Bucevičiūtė, et al., 1997)		35.00**	26.00**

*In this work and average values (**) obtained by Bucevičiūtė, et al. (1997). Data marked with one asterisk (*) are taken from Lekstutytė, et al., (2019) with added calculation method from (LST EN ISO 17892-9:2018).

Comparing the results of the strength properties obtained from the SST and MST tests of the uniform scale samples (i.e., D/H = 50/100 mm; Table 2), it can be seen that the mean cohesion values obtained from the MST test are higher by about 9 kPa (i.e., ~ 15%). However, the difference between the mean values of the internal friction is very small and, therefore, evaluated. Similar differences between the results obtained from the SST and MST tests were found in other works (Shahin, et al., 2011). The reasons for these higher values are the stresses and strains acting on the sample during the previous load steps (Choi, et al., 2018). The transformed Kondner's Hypothesis is proposed to be used to correct the difference (Sridharan, et al., 1972). As well as influence had different vertical axial deformation velocity (Barahona, et al., 2021). There in SST test velocity is 0.002%/min and in MST 0.027%/min. The peak deviatoric stress in MST is higher than in SST test. Consequently, and strength properties obtained in MST is slightly higher. Differences between values obtained in this work are not corrected because they are not significant. Therefore, after analyzing more samples and comparing values obtained from uniform triaxial tests, it can be stated that SST and MST test results are correlated. It can also be concluded that the MST testing method is suitable because it requires only one sample, which reduces the risk of errors and inaccurate results when collecting and preparing samples at different loads (Hormdee, et al., 2012). In addition, the effect of excess pore water pressure is worth a relevant consideration. Elevated excess pore water pressure records a higher soil deformation that is mean that reducing overall soil shear strength properties. (Thu, et al., 2006). The amount of generated excess pore pressure increases as the degree of saturation increases. Excess pore pressure is very similar between the sample, which has a degree of saturation of 0.12–0.40. Strength parameters decreases as the degree of saturation increases. That decreas is noticeable within the range of degree of saturation between 0.80 and 1.0. (Kuwano, et al., 1988). As was mentioned before investigated soil is considered as partially saturated and that mean there is no significant influence for soil strength properties due to saturation.

Within the MST test series, samples in UCD and UCU conditions were analyzed, having ratios of D/H = 50/100 mm and D/H = 100/200 mm were analyzed (Table 2). A general review of the results between the UCD and the UCU shows that the results obtained during the UCD test are higher than those of the UCU. It must be emphasized that shear strength parameters are slightly influenced by moisture conditions. In referenced literature (Bláhová, et al., 2013) it can be found that overall shear strength decreases with increasing water content. In this study, the water content of the UCD samples is higher about 2%. Either the differences can be described due to different test methodologies - i.e., due to drained and undrained conditions. When in a drained test condition, pore pressure does not occur and does not reduce values. In SST test series soils moisture is almost the same.

The results from the UCD tests (Table 2) show that the mean values (φ°) studied in the soil samples of different sizes by different methods are very similar, and the differences are small. Comparing to the ϕ° values obtained from the 50/100 mm samples analyzed in UCD conditions, it can be seen that they are slightly higher by about 0.8° than in the 100/200 mm samples. This indicates that the effective peak values used to determine the friction angle decrease with increasing sample size (Skuodis, et al., 2019). However, the opposite result was obtained from the UCU tests, where the mean value (φ°) in the 100/200 mm samples is higher by 3.2° than in the 50/100 mm samples. This difference can be explained due to moisture content. In UCU 100/200 mm sample moisture is slightly higher than in 50/100 mm.

The evaluation of the cohesion values for samples of different dimensions (Table 2) shows the same regularity of results as the values increase for samples with higher specimen ratio. In the 50/100 mm samples, this value is smaller by 3–4 kPa than in 100/200 mm samples, regardless of the drainage conditions. Here, the effective peak values decrease with increasing sample size.Comparing values from literature (Table 2) with the values obtained in our tests, the strength values of the Medininkai glacial period till soil were taken from the Engineering Geological Map of Lithuania (Bucevičiūtė, et al., 1997). Evaluating these results, we see that the mean value of the cohesion falls within the mean values obtained in the laboratory tests when evaluating the SST test applying SCD and UCU conditions, and the difference varies only about 3 kPa. Compared to the results obtained from the MST triaxial tests in UCD and UCU conditions, the cohesion in literature is smaller by 9.0–27.0 kPa than in our laboratory tests. However, the internal friction angle is greater than those obtained from the triaxial tests. Theoretical values are higher by 6°–16°. From the comparison with the results reported in the literature, it can be concluded that the results presented are not reliable and do not always correlate with the values determined in the laboratory. Soil deformation properties were investigated via the oedometer (OED) test, where samples were gradually subjected to additional loads (CEN ISO/TS 17892-5:2004) (Table 3). As was mention before due to complex soil structure and composition this soil is considered as partially saturated.

After analyzing the data obtained from the OED tests, only the results from those samples are given that underwent similar pressures during triaxial testing. To assess the deformation properties of the soil, E_{oed} values (tangent modulus values from primary oedometer loading) are compared with E_{50} (elastic modulus - secant modulus values calculated from the triaxial tests) (Table 3). On a global view, the larger differences are only visible when comparing E_{oed} with values of E_{50} calculated from MST tests and 100/200 mm samples. When a sample experience 150 kPa and 200 kPa cell pressures, the E50 values are 3–4 times higher than E_{oed} values at similar pressures. When analyzing E_{50} and E_{oed} values at different pressures and test conditions, the results are close without significant differences. Small differences can be explained by heterogeneous soil composition – i.e., when more prominent sand inclusions, gravel, and pebbles are present in the samples. Therefore, the results are distorted.

When comparing E_{oed} values obtained from OED tests with E_{oed} values calculated from CPT, geostatic pressure, and plasticity index (LST EN 1997-2:2007). Large differences in the results is seen with values which are calculated from CPT. Result from CPT are more than 5–10 times greater than the Eoed values. A similar difference between results is noticed likewise while comparing them with the theoretical results presented in the literature (Bucevičiūtė, et al., 1997). We emphasize that (LST EN 1997-2:2007) the presented calculation is not adapted to till soil genesis and soil property characterization. Therefore, when evaluating the deformation properties of such soil based on theoretical charts or by calculation from the cone resistance (qc) alone, it is necessary to do so with great care and without relying solely on obtained results.

Focusing only on the E50 values (Table 3), it can be seen that the higher values are found in the 100/200mm samples (differences ranging from ~ 1.5–2.0 times). This tendency for higher results is seen in all analyzed data when evaluating the deformation and strength properties of the soil. Only with increasing acting stresses the results become more uniform. This trend can also be seen in other works (Ranjan, et al., 2000), which explains that E50 values decrease with increasing stresses (Figure 5) because the values are determined from the stress-strain curve, whose curvature goes down.

Table 3: Comparison of deformation properties in terms of secant modulus (E_{50}) calculated from multi-stage triaxial (MST) tests and single-stage triaxial (SST) tests.

	Triaxial test method	Sample size, D/H, mm	E_{50}, MPa		
MST	UCD	50/100	13.37 ^{150*}	8.94 ^{250*}	6.48 ^{350*}
		100/200	29.89 ^{150*}	15.75 ^{250*}	9.20 ^{350*}
	UCU	50/100	8.94 ^{200*}	5.87 ^{300*}	5.20 ^{400*}
		100/200	20.73 ^{200*}	11.21 ^{300*}	6.75 ^{400*}
SST	UCU	50/100	7.75 ^{160*}	7.56 ^{260*}	10.78 ^{360*}
	SCD	50/100	10.92 ^{200*}	8.37 ^{300*}	13.25 ^{400*}
E_{oed}, MPa					
OED test			5.20 - 7.95 ^{160*}		
			11.48 - 15.53 ^{360*}		
CPT test (LST EN 1997-2:2007)			63.08 - 94.60		
CPT test (LST EN 1997-2:2007) (TAR, 2015-11-16, Nr. 18162)			60.00 - 80.00		
From literature (Bucevičiūtė, et al., 1997)			49.00**		

*Deformation modulus (E_{oed}) obtained from oedometer (OED) tests in this work and average values (**) obtained by Bucevičiūtė, et al. (1997). Values marked with one asterisk (*) indicate the pressure in kPa loaded on each sample.

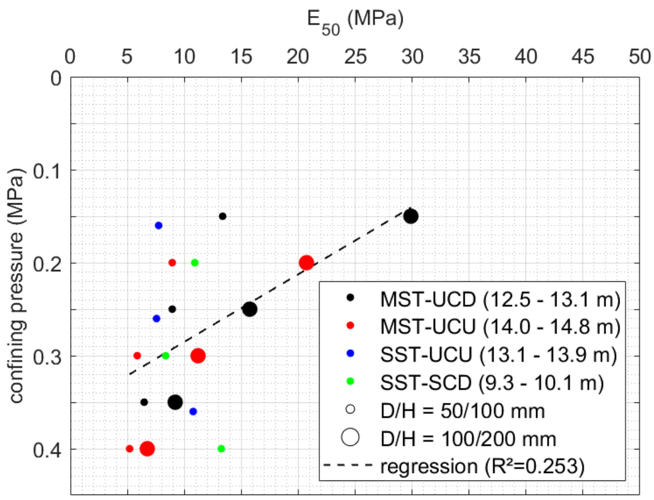

Figure 5: Secant modulus (E_{50}) versus confining pressure

When evaluating the OCR, it should be emphasized that it is particularly important for Lithuanian soils to understand their degree of consolidation, cracks, formation, and resistance to shear stresses. Typically, the OCR is calculated from the qc values obtained from CPT (Figure 6). Here, the OCR values, excluding the peaks, ranging from 20 to 9. This indicates that the soil is overconsolidated, and the degree of overconsolidation decreases with depth.

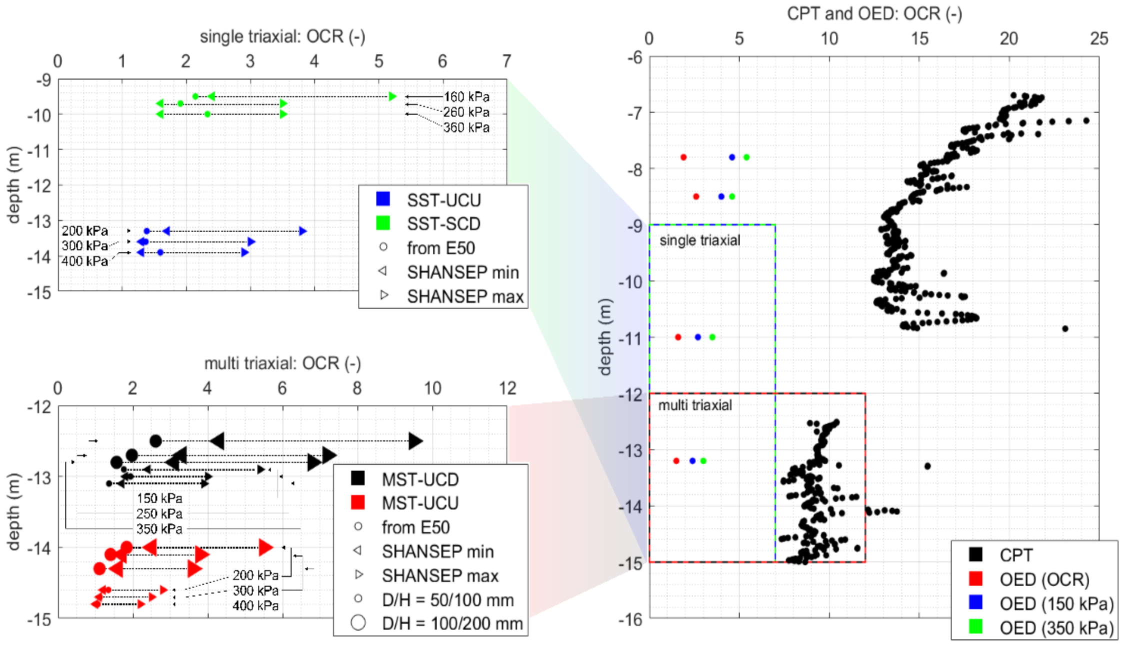

Figure 6: Overconsolidation ratio (OCR) values versus depth

The most accurate method of estimating OCR is considered to be (Urbaitis, et al., 2016) the overconsolidation pressure ratio with effective geostatic stresses σ'_p/σ'_{vo} calculated or estimated from the results of OED testing. It is seen (Figure 6, Table 4) that OCR values range from 1.5 to 2.6. OCR values from OED testing show that the soil is overconsolidated. These values are significantly lower than the values calculated from CPT. OCR values obtained from CPT are much larger (about 3 times) due to improper calculation formulas to Lithuanian till soil. Due to the exceptional properties of these soil, we cannot directly apply these formulas as submitted Urbaitis, el. al. (2016) and properly compare with other OCR results.

Table 4: Overconsolidation ratio (OCR) values calculated with different methodologies.

Triaxial test			OCR (E_{50})			OCR (SHANSEP)		
						Min-Max	Min-Max	Min-Max
MST	UCD (12.5–13.1m)	50/100	1.8 ^{150*}	1.9 ^{250*}	1.4 ^{350*}	2.4-5.4 ^{150*}	1.8-4.0 ^{250*}	1.7-3.9 ^{350*}
		100/200	2.6 ^{150*}	2.0 ^{250*}	1.6 ^{350*}	4.3-9.5 ^{150**}	3.3-7.2 ^{250}	3.1-6.8 ^{350*}
	UCU (14.0–14.8m)	50/100	1.3 ^{200*}	1.1 ^{300*}	1.1 ^{400*}	1.2-2.8 ^{200*}	1.1-2.5 ^{300*}	1.0-2.2 ^{400*}
		100/200	1.8 ^{200*}	1.4 ^{300*}	1.1 ^{400*}	2.5-5.5 ^{200}	1.7-3.8 ^{300*}	1.6-3.6 ^{400*}
SST	UCU (13.1–13.9m)	50/100	1.4 ^{160*}	1.4 ^{260*}	1.6 ^{360*}	1.7-3.8 ^{160*}	1.3-3.0 ^{260*}	1.3-2.9 ^{360*}
	SCD (9.3–10.1)	50/100	2.1 ^{200*}	1.9 ^{300*}	2.3 ^{400*}	2.4-5.2 ^{200*}	1.6-3.5 ^{300*}	16-3.5 ^{400*}

OCR (OED test)		
Depth, m	OCR = σ'_p/σ'_{vo}	OCR (E_{oed})
7.7–7.8	1.9	4.6^{150} - 5.4^{350**}
8.4–8.6	2.6	4.0^{160} - 4.6^{360**}
11.0–11.3	1.6	2.7^{150} - 3.5^{350**}
13.2–13.4	1.5	2.4^{150} - 3.0^{350**}

*Values marked with one asterisk (*) indicate the pressure in kPa loaded on each sample. Values marked with two asterisks (**) show pressure chosen from OED tests regarding pressures during the triaxial tests.

E_{oed} values were used to estimate OCR values from OED tests (Figure 6, Table 4). When comparing these E_{oed} results with OCR= σ'_p/σ'_{vo} (Figure 6), similar results of increasing-decreasing tendencies are seen (i.e., decrease with depth). Here, OCR values are about 1.5–2 times higher than in the laboratory tests, but the difference is as significant as it was with values from CPT. Here, results are closer to the laboratory tests and show that this soil is also overconsolidated. In this calculation method, OCR values were calculated from selected E_{oed} results with the same pressures as for the triaxial tests.

OCR were calculated and evaluated from the results obtained from triaxial tests using various testing methodologies (Table 4) mentioned before. OCR (E_{50}) values calculated from the secant modulus are very close to the values obtained from OCR= σ'_p/σ'_{vo} (Figure 6). Although values are close, but using UCU test conditions to investigate 50/100 mm sample it can be seen that the soil is normally consolidated according to the OCR value (OCR > 1.5). However, when evaluating the obtained mean OCR value from different pressures, the soil is nonetheless overconsolidated-except for MST and SST 50/100 mm tests in UCU conditions, in which it stays normally consolidated. However, we emphasize that this normally consolidated sample was collected from a different depth, where no OED test was performed.

Also, an OCR (SHANSEP) calculation (Figure 6, Table 4) was performed for the data from the triaxial tests. The minimum and maximum values calculated with the SHANSEP formula according to different coefficients at specific pressures show that the soil is overconsolidated. OCR obtained using this method are close to the OCR (E_{50}), OCR= σ'_p/σ'_{vo} and OCR(E_{oed}) values. It is also observed that minimum values obtained in the MST and SST triaxial test by UCU conditions and in 50/100 mm sample sizes show that the soil is normally consolidated.

5 Conclusions

Deformation and strength properties of Middle Pleistocene Medininkai glacial period till soils are very poorly investigated. However, the soils of this period and composition do not only cover the surface of Lithuania but are often subjected to human economic activity (i.e., as medium for buildings or commercial deposits, etc.).

Strength properties of the soil were investigated via single (conventional) stage triaxial (S(C)ST) tests with 50/100 mm size samples and via multi-stage triaxial (MST) tests having soil samples of different sizes (50/100 mm, 100/200 mm) by applying unsaturated consolidated drained (UCD), unsaturated consolidated undrained (UCU) and saturated consolidated drained (UCD) conditions. Deformation properties of the soil were investigated in oedometer (OED) tests, during which the soil gradually underwent additional load cycles.

The analysis of the results obtained from the different SST and MST test methods suggests that the MST test method is suitable as it requires only one sample, which reduces the risk of errors and inaccurate results when collecting and preparing samples at different loads.

When comparing the strength and deformation properties of the soil determined by different test methods, no significant differences in the results were observed. However, a large gap in the results has been observed compared to those widely published in the literature and used in our study.

The calculation of the OCR for Medininkai glacial period till soils shows that these soils are overconsolidated (OCR > 1.5).

6 Publisher's Note

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