Some mechanical properties of Medininkai glacial period overconsolidated moraine clay

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1 Introduction

According to the stratigraphic scheme of Lithuanian Quaternary (Satkūnas 1994), which is based on the genesis of soil, the analyzed soil belongs to the glacial formations of the Middle Pleistocene Medininkai glacial period (gt II md). The tested soil is laying in the eastern part of Lithuanian territory (Guobytė 2014) (Fig. 1).

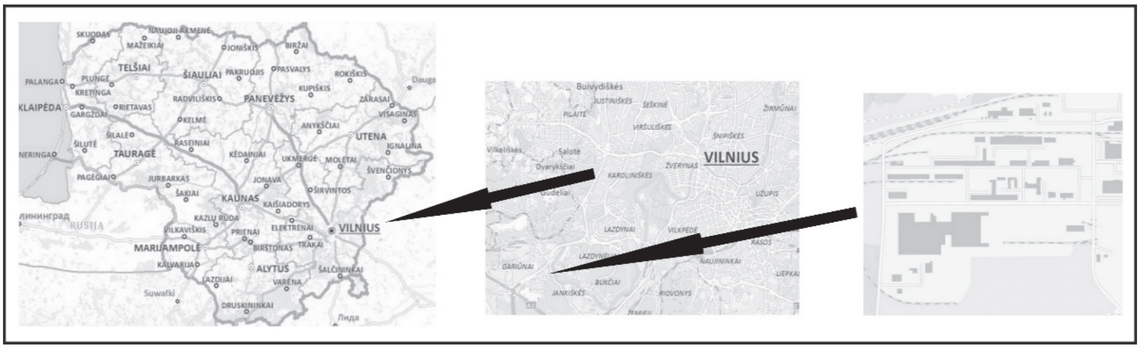

Fig. 1: Investigated site

Medininkai glacial period (195–128 thousand years) left a cover of 30–40 m thick. Maximum thickness is about 50–100 m. (Kavoliutė 2012). In Lithuania Medininkai glacial period is prevalent only in the south eastern territory of the country and covers about 1459.6 km^2 (i.e. 2.25 %) of Lithuanian territory. Prevalent thickness is 10–30 m (Grigelis et. al. 1994).

Research of strength properties on the glacial till soils of the Medininkai glacial period is almost not investigated. Majority of the focus was concentrated at the determination of the physical properties. Based on the information and calculations of the Lithuanian geology survey (Gaigalas et. al. 2001), glacial soil (till) in

Lithuania, comprises about 70 % (i.e. about 45000 km^2). Soils of this period and composition are an object of human economic activity, base or bearing strata for engineering structures, etc. (Dundulis 2004).

This research is not focused only at the properties of the till soils of Medininkai glacial period. The main aim is to investigate mechanical properties of the till soils in general and to compare obtained results with analyzed literature, regardless of its genesis.

2 Investigation methodology

2.1 In-situ tests

In total 29 cone penetration tests were performed at the test site. Cone penetration test (CPT) was performed till 6,0 – 20,0 m depth (Geotestus 2017). In this research only one test point (Chart 22) (Fig. 2) was used. Based on the information obtained during the performed test (q_c, fs), the tested soil according its strength is classified as a very strong soil, when q_c> 4 MN/m^2 (according soil strength classification from cone penetration test) (Gadeikis et al. 2012).

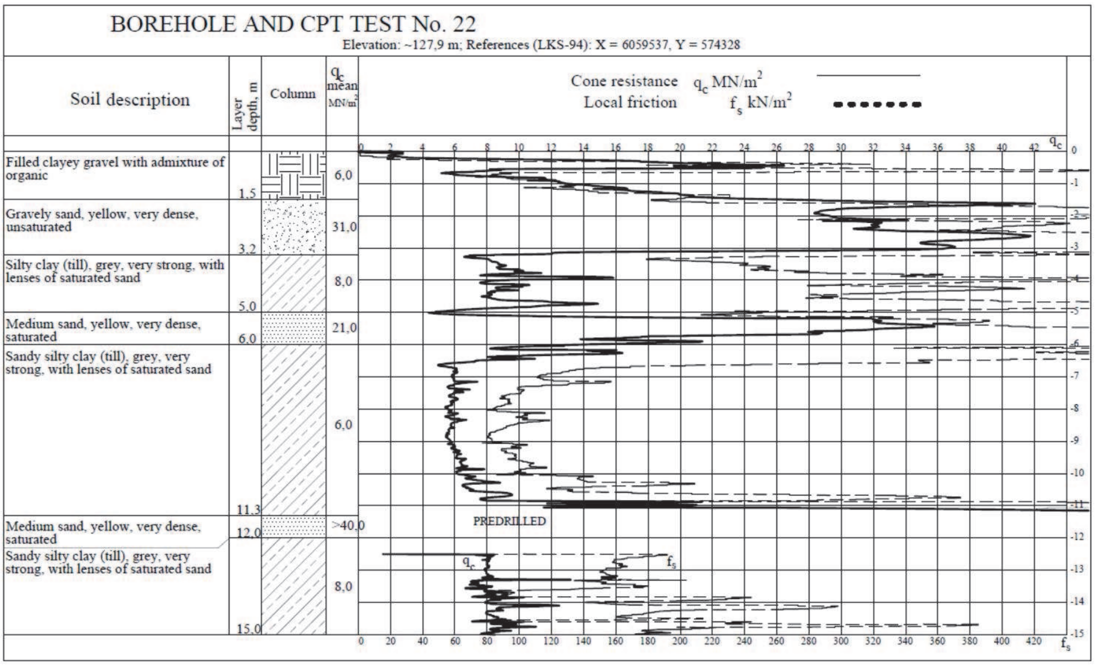

Fig. 2. Borehole and CPT test example. (Geotestus 2017)

2.2 Laboratory tests

2.2.1 Physical soil properties

In addition, physical properties of the soil were determined. The determination of the natural density, moisture and Atterberg limits was performed, as well as the grain size analysis of the soil. Laboratory testing was performed base on the standards (CEN ISO/TS 17892-12:2004, CEN ISO/TS 17892-4:2004). Obtained results are presented in the tables (Tab. 1 and 2).

Tab. 1. Grain size distribution of taken samples

Grain size distribution			
Clay	Silt	Sand	Gravel
<0.002	0.002 - 0.063	0.063 - 2.00	>2.00
9,3 – 10,1 (SCD) sandy silty clay			
11.32	40.66	45.49	2.53
13,1 – 13,9 (UCU) sandy silty clay			
13.4	42.58	41.14	2.88

Tab. 2. Physical properties of taken samples

Density	Particle density	Moisture	Plasticity index			
ρ, g/cm^3	ρ$_s$, g/cm^3	w, part.u.	w_L, part.u.	w_P, part.u.	I_P, part.u.	I_L, part.u.
Sampling depth 9.3–10.1 m						
2.27	2.72	0.11	0.222	0.125	0.111	-0.185 (very stiff)
Sampling depth 13.1–13.9 m						
2.29	2.72	0.119	0.245	0.139	0.1	-0.215 (very stiff)

2.2.2 Mechanical soil properties

Two sample series were tested with the triaxial apparatus (height H=100 mm, diameter D=50 mm), with the application of different testing methodologies. First series was unsaturated consolidated undrained (UCU) triaxial tests for soil sampled from 13.1–13.9 m depth. In this test applied cell pressures were, namely: 160 kPa, 260 kPa, and 360 kPa. Second series of samples – saturated consolidated drained (SCD) triaxial tests for soil sampled from 9.3–10.1 m depth. Applied cell pressures were, namely: 200 kPa, 300 kPa, and 400 kPa. During both testing methodologies, the velocity of vertical deformations was 0.002 %/min (up to 15 % of a vertical deformation).

Several techniques were chosen for the investigated soil strength parameters analysis:

1. According to the law of Coulomb, marginal condition of tensions is defined by the tangent of Mohr's circles, which inclines at the angle ϕ' and shears of the line segment at the vertical axis c'. Average indicators of the shear strength and the angle of the internal friction are calculated with the method of the least squares. (Šimkus 1987, Amšiejus, et al. 2006, CEN EN 1977-1:2004)
2. According to the N and M (СНиП 2.02.02–85 1986, Dirgėlienė 2013, Dirgėlienė 2007).

3. According to the $t'–s'$ (Massachusetts) coordinate system. (Dirgėlienė, 2013, Amšiejus, et al. 2010, Ho Chi Minh City University of Technology, 2016).
4. According to the p – q (Cambridge) coordinate system. (Dirgėlienė 2007, Ho Chi Minh City University of Technology 2016).

3 Analysis of the results

3.1 Results of field test and physical soil properties

According to the description of the borehole (see Fig. 2), the analyzed glacial till soil of Medininkai glacial period is found from 6.0 m and deeper. For laboratory testing, samples were collected from the depth which ranges: 9.3– 10.1 m and 13.1–13.9 m. Soil according to the cone resistance (q_c) is classified as very strong soil (when q_c > 4 MN/m^2). For the sample from the depth of 9.3–10.1 m, q_c = 6.0 MN/m^2, and for the sample from the depth of 13.1-13.9 m, q_c = 8.0 MN/m^2.

Soil for laboratory testing was collected from two depth ranges, but as it can be seen from the performed testing of soil physical properties (Tab. 1 and 2), these soils are very similar. After evaluation of the grain size analysis (see Tab. 1), the investigated soil is identified as a sandy silty clay (till). The differences of the obtained results are minimal.

3.2 Results of mechanical soil properties

Simulated tests with triaxial cell was intended to determine the strength properties of the till soil, when it is saturated and unsaturated conditions in its natural laying environment. During the triaxial testing, soils were analyzed by two methodologies (Lade 2016):

1. By saturating, consolidating, and draining (**SCD**). During the test consolidation pressures were: 200 kPa, 300 kPa, and 400 kPa. (Fig. 3).
2. By consolidating without saturation and without drainage (**UCU**). During the test consolidation pressures were: 160 kPa, 260 kPa, and 360 kPa. (Fig. 4).

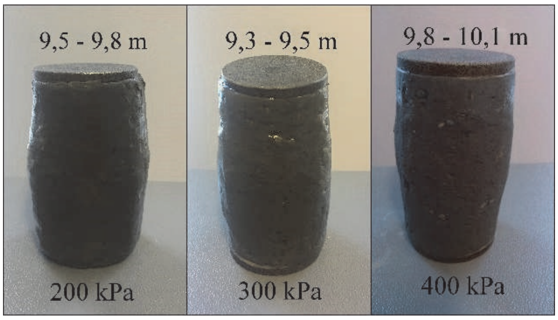

Fig. 3. Samples after SDC tests

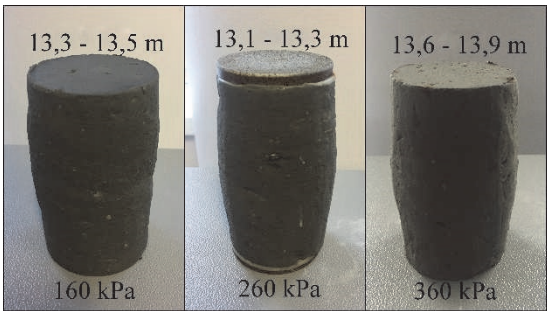

Fig. 4. Samples after UCU tests

Consolidation pressures applied for two different methodologies was chosen according to sample depth. Soil samples was saturated (Fig. 5), which are collected from the depth of 9.3–10.1 m. After of the test B, it was obtained that the samples were not completely saturating. B value was from 0.65 to 0.77. Highest value of the B test was in the sample, which later was applied cell pressure 200 kPa (Fig. 5).

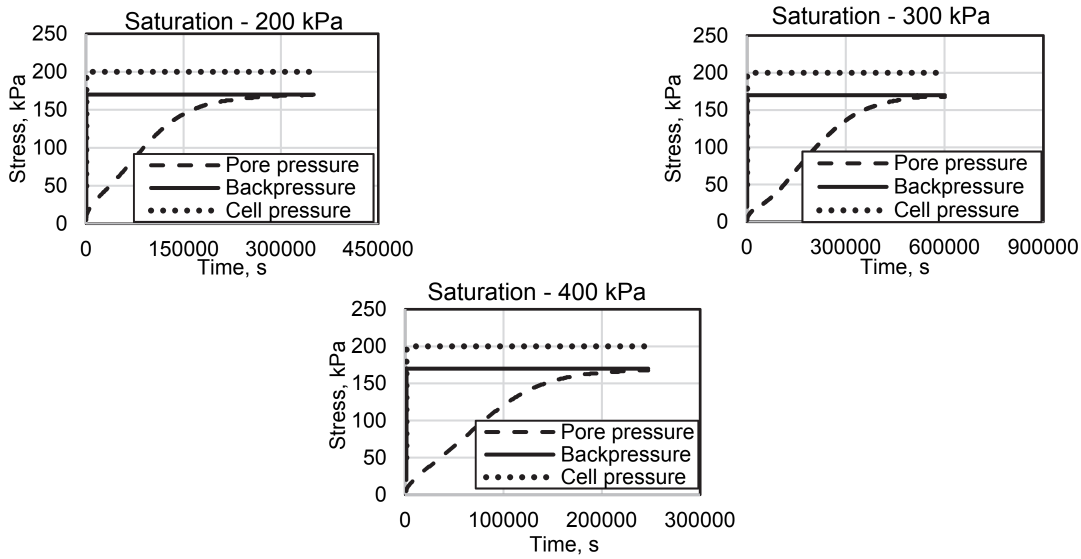

Fig. 5. Specimen saturation by increasing back pressure (SCD method)

Next stage of tests was consolidation of samples (Figs. 6–7), where the change of the pore pressure (*u*) was obtained. Taking into consideration the time scale, it can be stated, that samples were consolidating at the similar speed (pore pressure dissipated at the similar time intervals), but the interval of the pore pressure change was different. The range of pore pressure change (from 2 to 12 kPa) for the UCU sample (Fig. 7) is lower than in case of the SCD (from 100 to 200 kPa) (Fig. 6). The difference of pore pressure scales is caused by the fact that SCD sample was saturated.

During the loading stage the samples stress paths are quite similar for SCD and UCU tests (Fig. 8–9). Stress paths of UCU test (Fig. 9) are slightly steeper, but in total characteristics are identical as for SCD tests (Fig. 8). During the analysis of the dependency between stress and deformations, the curve of SCD with the load of 200 kPa (Fig. 10) stands out the most. It shows that this sample was sheared very suddenly and quickly. That can be explained by the fact, that sample contained pebble particles of larger diameter in the shearing plane. During the analysis of dependencies of other samples unique differences or uncommon curves are not observed.

The results of the dependency of pore pressure and deformations (Fig. 12–13) for the samples tested with different methodologies are not the same. As it was mentioned before, during the discussion about the consolidation stage (Figs. 6–7), difference in the scales of pore pressure is obtained. Analyzing figures 12–13, obt-

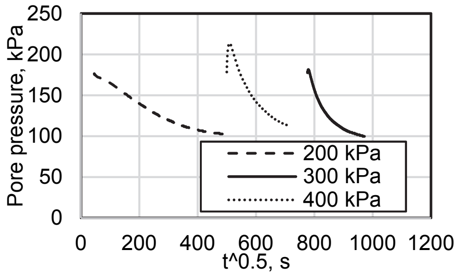

Fig. 6 SCD Pore pressure versus time

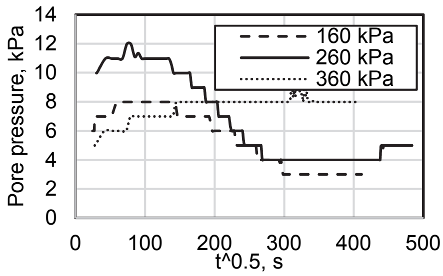

Fig. 7 UCU Pore pressure versus time

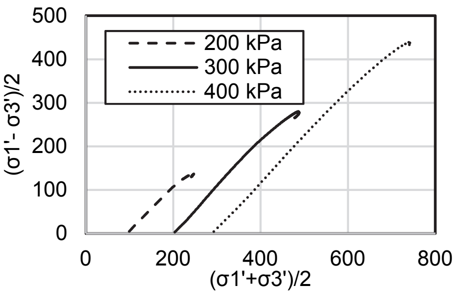

Fig. 8 SCD Stress path

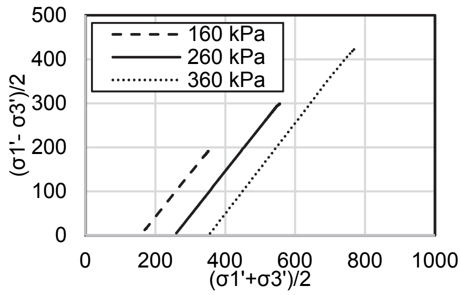

Fig. 9 UCU Stress path

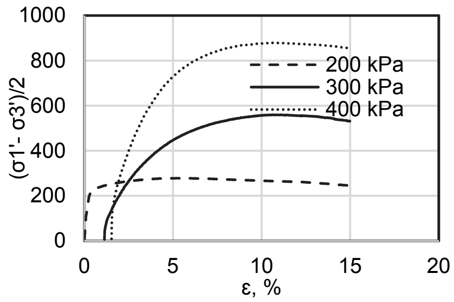

Fig. 10 SCD Stress versus strain

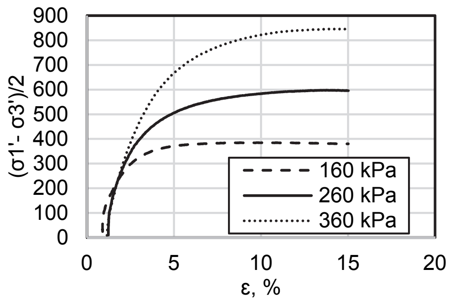

Fig. 11 UCU Stress versus strain

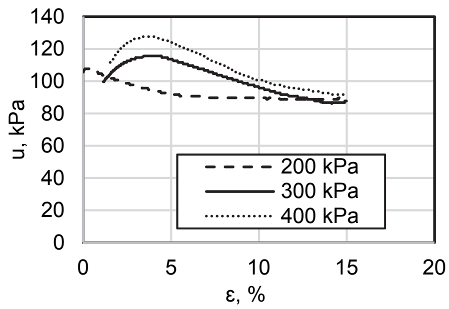

Fig. 12 SCD Pore pressure versus strain

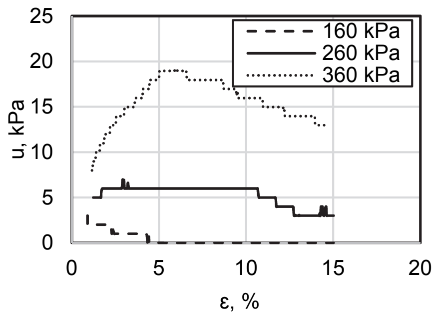

Fig. 13 UCU Pore pressure versus strain

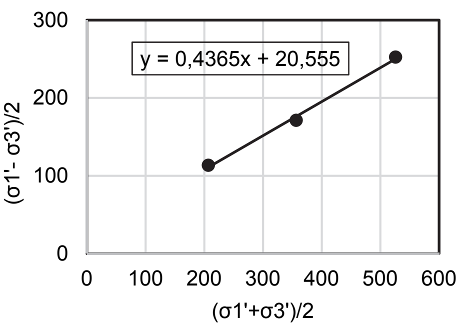

Fig. 14 SCD Peak strength values

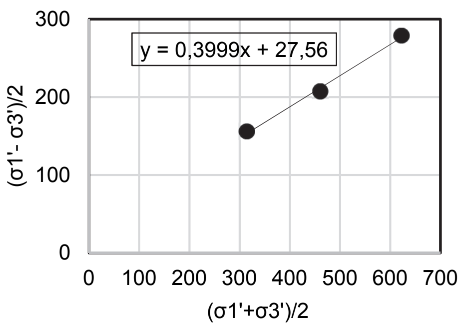

Fig. 15 UCU Peak strength values

ained quite similar dissipation of the pore pressure. In the SCD tests, the pore pressure (u) in the sample with the load of 200 kPa (Fig. 12) dissipated at the deformation of 5 %, also the sample of UCU tests with the load of 160 kPa (Fig. 13) obtained, that u dissipated at the deformation (ε) of about 4.5–5.0 %.

Based on the provided investigations (in–situ and laboratory), it can be stated, that properties of the soils have no dependence from depth. UCU sample is slightly stronger, its q_c is 2.0 MN/m^2 higher, density and plasticity indexes are also slightly higher, than SCD samples. When comparing values of the properties of both soils depths, differences among them are not high, usually very minimal, and both are assigned to the class of very strong soils. The angle of internal friction ϕ and cohesion c was evaluated according to several techniques (Tab. 3 lines a–d).

Tab. 3 Comparison of shearing strength

	Internal friction angle ϕ°		Cohesion c, kPa	
	SCD	UCU	SCD	UCU
a. Least squares method (Šimkus 1987, ГОСТ 20522–96)	23.58	21.75	20.55	27.56
b. M, N (СНиП 2.02.02–85 1986)	25.79	25.79	23.35	30.59
c. t – s (Massachusetts) coordinate system	25.88	23.57	28.03	30.06
d. p – q (Cambridge) coordinate system	25.84	23.5	22.95	30.16
e. Literature analysis (Sližytė et. al 2012)	28.00		82.00	
f. Literature analysis (Šimkus et. al. 1973)	27.00		66.00	

Shearing strength results presented in Tab. 3 lines "a–d" are similar. The most conservative results are obtained using least squares method (Tab. 3 lines "a"), where angle of internal friction is smaller ~2–4° (respectively SCD and UCU) and cohesion is smaller ~8.0–3.0 kPa (respectively SCD and UCU) than comparing with results presented in Tab. 3 lines "b–d".

Analyzing results presented in the row "e" (Tab. 3), that are given for the silt and clay soils based on the values of their cone resistance (q_c). These results should be applied only to drained soils. After analyzed values given in literature, it was obtained that suggested shearing strength parameters (Sližytė, et al. 2012, Šimkus et. al. 1973) are overestimated when comparing with obtained results. Angle of internal friction given in literature is higher ~5–3° (for SCD tests) and ~6–2° (for UCU tests). Meanwhile, cohesion given in literature is higher ~54.0–61.0 kPa (for SCD tests) and ~52.0–55.0 kPa (for UCU tests). Therefore, after the results are evaluated, the big discrepancies are noted when comparing cohesion – c, which, in some cases, is even 4 times higher. Highest differences are among the results obtained through the SCD tests (difference from 2.9 to 3.9 times), slightly smaller differences are in the UCU tests (difference from 2.6 to 2.9 times). When evaluating the angle of internal friction $\phi°$, values differ only from 6 to 2° and that is relatively not a big difference. However, it is worth to mention, that this table is intended only for till soil, that is why such differences are obtained.

4 Conclusions

Based on the obtained laboratory results of physical properties, the tested soil is sandy silty clay (till), which according to the results of the field test is assigned to the class of very strong clays.

After the tests with triaxial apparatus according to two different testing methodologies, the obtained shearing strength properties c' and ϕ' did not differ a lot (angle of internal friction by 2°, and shear strength about 7 kPa).

The most conservative shearing strength values are obtained using least squares method for results interpretation. After analyzed values given in literature, it was obtained that suggested shearing strength parameters are overestimated. Angle of internal friction given in literature is higher ~5–3° (for SCD tests) and ~6–2° (for UCU tests). Meanwhile, cohesion given in literature is higher ~54.0–61.0 kPa (for SCD tests) and ~52.0–55.0 kPa (for UCU tests).

Shearing strength which is given in literature should be evaluated very carefully and based on local soil properties knowledge and geotechnical engineer experience.

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