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Temperature effect on the mechanical parameters of a clayey compacted soil

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ABSTRACT: Energetic geostructures consist in establishing a thermal exchange with the ground directly through building foundations or structures. Adapting the whole or part of foundation piles into energy piles brings questions on the impact of the temperature variations on the geotechnical parameters of the sensitive soils (illite, smectite ...). The estimation of bearing capacity of deep foundations is actually based on pressuremeter or penetrometer test results. In this study, these tests, adapted to the laboratory, were carried out on compacted materials submitted to different thermal variations. Firstly, a set of mini-penetration tests were performed at different temperature. The results showed that the measured parameters namely the cone resistance (q_c) and the sleeve friction resistance (f_s) are affected by temperature at low water contents. Secondly, mini-pressuremeter tests were conducted on an illitic soil compacted in a cylindrical container, submitted to 20 and 40 °C. An impact of the temperature variation on the limit pressure (p_l) and the creep pressure (p_f) was detected.

Keywords: energetic geostructures, mini-penetrometer, mini- pressuremeter, illite, compacted soils, laboratory testing

1 INTRODUCTION

Geothermal energy is an energy source with cost, reliability and environmental advantages over conventional energy sources. Thermo-active geostructures are sustainable structures that enable thermal exchange between buildings and the ground through liquids that flow through closed-loop circulation systems integrated into the geostructures. These systems use the subsurface soil as a heating or cooling source for buildings (Fromentin et al. 1999, Laloui et al. 2003, Brandl 2006). Thermoactive geostructures may be diaphragm walls, tunnel lining (Zhang et al. 2013) or energy piles (Pahud et al. 1999, Moel et al. 2010). The classical way in which thermo-active foundations are used results in changes in the surrounding ground temperature (approximately 12 °C) over a scale from 4 to 30 °C (Peron et al. 2011). Some questions remain on the impact of this temperature variation on the soil geotechnical parameters especially for clayey soils.

In this work we consider the piles, which are deep foundations designed to carry the imposed charges to depth. Foundation piles function through a combination of two effects: the tip resistance and the shaft resistance along the pile. These two parameters are

related to the shear strength of the soil and the mechanical properties of the soil–structure interface respectively. In the literature, there are few papers on the impact of temperature variations on the friction strength in rocks under high temperatures (Stesky 1978). However, studies on the effect of temperature on the mechanical properties of the soil–structure interface are missing. The impact of temperature on the shear strength is complex. Some authors such as Mitchell (1964), Lingnau et al. (1996), Wiebe et al. (1998) and Dumont et al. (2010) have reported that heating decreases the shear strength of soil, whereas others, such as Tanaka et al. (1996), Abuel-Naga et al. (2006) and Laloui & Cekerevac (2008) have obtained the opposite results. All of the studies concluded on a contraction of the elastic domain with heating (Lingnau et al. 1996, Graham et al. 2001, Cekerevac & Laloui 2004 and Marques et al. 2004).

The objectives of this work were to improve the understanding and quantify the impact of temperature variation on soil bearing capacity. Calculation of the bearing capacity of foundations is based on the results of pressuremeter tests or penetrometer tests. Experimental methods have been developed to enable the conduction of these tests in the laboratory conditions.

In this paper, the tested material is presented, the proposed mini-penetration test and associated parameters are introduced and the obtained results are discussed. Afterwards, the mini-pressuremeter test is explained and the results are given and commented. Finally, conclusions on the impact of temperature variations on the bearing capacity of the compacted material are discussed.

2 THE ILLITIC SOIL

The tested material is an illitic soil named Arginotech© from the east of Germany. It contains 77% illite, 10% kaolinite, 12% calcite and traces of quartz and feldspar. Illite is a non-swelling clay present in variable proportions in soils due to mineral illitization (Lynch 1997). It seems to be sensitive to temperature according to Tanaka et al. (1996), who observed a decrease of the elastic limit of illite with increasing temperature (from 28 to 65 and 100 °C).

The liquid limit (LL), the plastic limit (PL) and the plasticity index (PI) of the material were 65, 34 and 31%, respectively (AFNOR 1993) and the optimum water content and the maximum dry density were 31.3% and 1.41 Mg/m³, respectively at room temperature (approximately 20 °C) (Fig. 1) (AFNOR 1999).

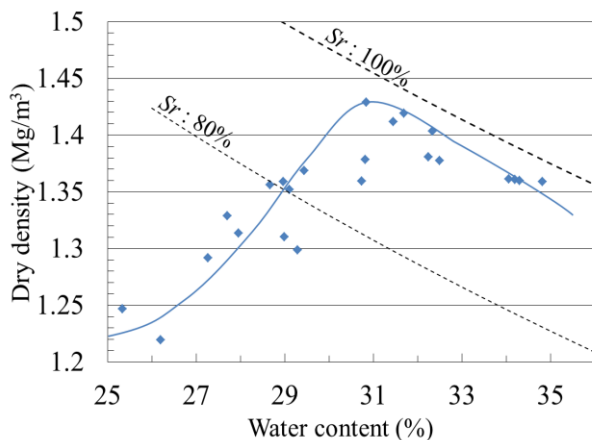


Figure 1. Specific compaction curve of the material. Sr: Degree of saturation.

According to the particle size distribution of the soil, almost 85% of the particles are smaller than 0.002 mm (clay particles), and the remaining 15% are smaller than 0.02 mm (silt particles). In the French standard for soil classification (GTR 2000), the material is classified as an A3 and according to the Unified soil classification system (ASTM 2006) the material is classified as a flat clay, CH.

3 MINI-PENETRATION TEST

3.1 Material preparation and compaction

The material was mixed with water to reach desired water contents and then packed in hermetic bags and

left to homogenize for at least 24 hours. Samples 117 mm in height were compacted into 6 layers in a CBR mold. Dynamic compaction was applied to ensure the homogeneity of the material. The compaction level was equivalent to the standard Proctor compaction energy. The initial temperature of the samples was 20 °C. The samples were thermally cured at 1, 20, 40 and 70 °C for at least 24 h in a climatic chamber while their water contents were kept constant.

3.2 Experimental device

The mini-penetration test is a laboratory test adapted from the standard static penetration test (AFNOR, 1996). A probe consisting of a cone with a 60° apex penetrates the sample at a constant rate of 30 mm/s using a tension-compression system with an accuracy of ± 1 N (Fig. 2). Two stainless steel probes named P1 and P2 were used. The lengths (250 mm), angles (60°) and maximal external diameters (12 mm) of the two probes were the same. However, the diameter of the rod of P1 was 10 mm and that of P2 was 12 mm (Fig. 2). The maximum penetration length of both probes was 100 mm. The force on cone was measured with P1, whereas P2 allowed the measurement of the combined force on cone and the force on friction sleeve.

Three mini-penetration tests were carried out successively in each mold after temperature equilibrium was reached. To minimize edge effects and influence between tests, the points are positioned on a concentric circle twice smaller than the radius of the CBR mold. The tests were performed 38 mm from the edge of the mold, and the distance between the test points was 66 mm.

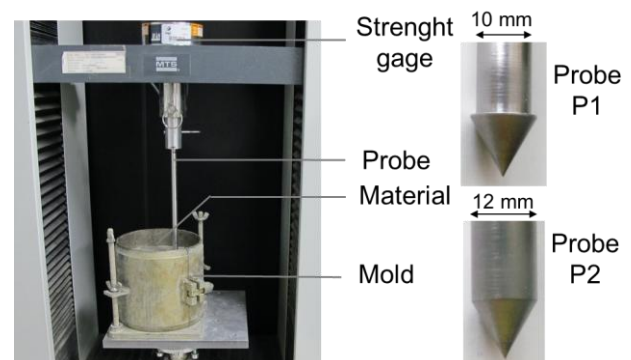


Figure 2. Mini-penetration test with tension-compression press and probes for mini-penetration test.

3.3 Mini-penetration curves typology

Figure 3 presents the typology of the two types of tests: the penetration of P1 and the penetration of P2. The force on cone (Q_c) was then calculated by averaging the force obtained in the linear part (BC) for a penetration depth of 50 mm to 100 mm in the P1 test. The cone resistance (q_c) was calculated according to equation 1.

$$q_c = Q_c / (\pi R_c^2) \quad (1)$$

where R_c is the maximum radius of the conical apex.

Part BD of the curve is specific to the P2 test, in which the measured force (Q_t) is the sum of the force on cone and the force on sleeve friction. Q_t increased as the probe came into contact with the soil. The sleeve friction resistance (f_s) is the quotient of the difference between Q_t and Q_c and the lateral surface of the probe in contact with the sample (Equation 2).

$$f_s = (Q_t - Q_c) / (2\pi R_c(L - L_c)) \quad (2)$$

where L is the penetration length and L_c is the height of the cone apex.

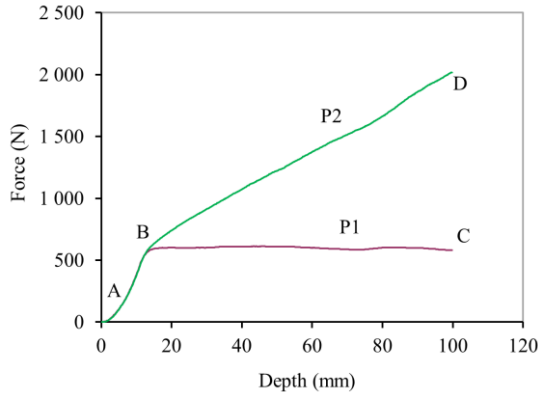


Figure 3. Total force versus depth measurements using probes P1 and P2 for penetration test.

3.4 Results of mini-penetration tests

In this section, the effects of the water content and dry density on the determined strengths at 1, 20, 40 and 70 °C are studied. First, the cone resistance results are presented. Then, the sleeve friction resistance results in penetration are discussed.

3.4.1 Cone resistance

Penetration tests were performed on samples compacted at various water contents and densities. Figure 4 superimposes the cone resistance (q_c) onto the compaction curve of the material at 1, 20, 40 and 70 °C. The cone resistance varies more rapidly on the dry side than on the wet side of the compaction curve.

Globally the maximum values were obtained for the material on the dry side ($q_c = 11,000$ kPa), and q_c decreased linearly when approaching the optimum (1,800 kPa). On the wet side of the optimum, q_c changed slowly, tending towards approximately 1,000 kPa.

Concerning the effect of temperature, the cone resistance decreased slightly with increasing temperature for the samples prepared on the dry side of the optimum but did not vary significantly and it is not affected on the wet side of the optimum.

3.4.1 Sleeve friction resistance in penetration

For each test and each penetration depth, the sleeve friction resistance (f_s) was calculated (Equation 2). Then, the mean values of f_s for penetration depths of 50 mm to 100 mm were chosen to obtain the mean sleeve friction resistance of the test for each temperature (Fig. 5).

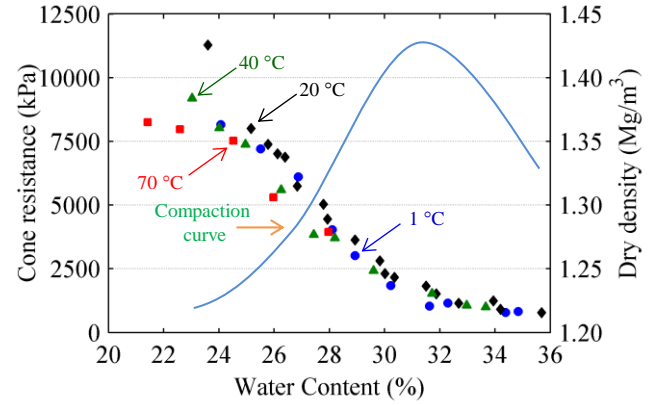


Figure 4. Mean cone resistance at 1, 20, 40 and 70 °C.

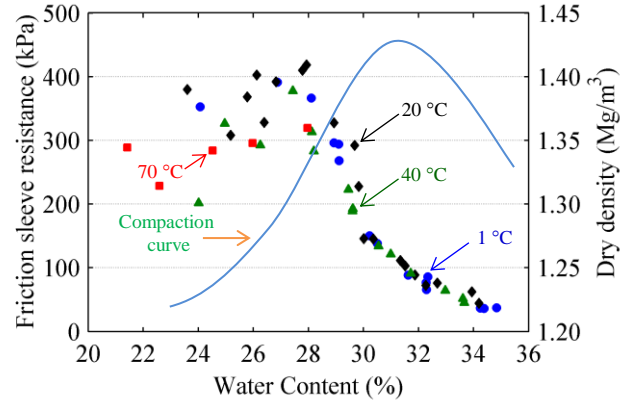


Figure 5. Mean sleeve friction resistance at 1, 20, 40 and 70 °C.

Globally, the evolution of the sleeve friction resistance was not monotonic on the dry side. This could be due to the compacted soil micro-structure, which is irregularly aggregated on the dry side. At lower water content, the sleeve friction resistance increased with decreasing water content. This evolution could be related to the increase in the matric suction associated with the rise of the soil shear strength.

A maximum value was reached on the dry side of the optimum. For example, at 20°C, the sleeve friction resistance reached 420 kPa for the sample prepared at a 28% water content (3.3% drier than the optimum). After that point, f_s decreased quickly, reaching 70 kPa for samples prepared at a 32.3% water content (1% wetter than the optimum). On the wet side of the optimum, the decrease in f_s was smaller than that on the dry side (Fig. 5).

For samples prepared at water contents below 30%, the results were scattered. However, for the samples prepared on the dry side of the optimum, increasing temperature tended to decrease the sleeve

friction resistance. On the contrary, on the wet side, the variation of temperature did not affect the sleeve friction resistance.

These results are consistent with those obtained by Wiebe et al. (1998). Their results showed that the shear strength decreases with increasing temperature and that the effect of temperature is greater when the degree of saturation is low, i.e., on the dry side of the optimum. They concluded that the decrease in shear strength may be caused by the decrease in suction due to heating at a constant degree of saturation.

According to Uchaipichat and Khalili (2009), increasing temperature caused retraction of the yield locus due to the thermal softening. This softening of the soil response was shown in their tests with a shift to the left of the loading collapse (LC) curve and a shift in normal consolidation lines toward a lower stress.

The decrease in the studied parameters with increasing temperature could be due to the reorganization of the soil micro-structure and the decreased viscosity of water. Indeed, a decrease in pore-water viscosity can facilitate sliding between grains and the soil-probe interface, thereby reducing the measured strength. This could explain the temperature effect on the dry side of the optimum.

4 PRESSUREMETER TEST

The pressuremeter test is widely used for designing the foundations of a building (AFNOR, 2000). The principle is to introduce a cylindrical probe with a flexible cover membrane which can expand radially in a borehole in order to measure the pressuremeter modulus E_M , the creep pressure P_f and the limit pressure P_l . To investigate the effect of temperature variation on a compacted material, a container connected to a heating/cooling system (thermoregulator) has been made.

4.1 Experimental device

A cylindrical stainless steel container 800 mm in height and 600 mm in diameter has been made. Stainless steel tubes welded to the outside of the container were connected to a Vulcatherm® thermoregulator. Ethylene-glycol water solution circulates in the tubes for imposing the temperature (-10 to 90 °C). Insulating sleeves were placed on the tubes in order to reduce heat exchange with the surrounding atmosphere (Fig. 6). Finally, the assembly was placed in a box made of extruded polystyrene plates of 40 mm thick which reinforced the insulation.

4.2 Sample preparation and compaction

Before compaction, water was added to the material in a large capacity, fixed-speed mixer MIX120®. The wet material was stored in 8 canisters for a minimum period of 5 days to ensure good homogenization. A pneumatic compactor was used to compact the material at room temperature (approximately 20 °C). It created dynamic forces applied on a metallic plate of 4 mm in thick and 600 mm in diameter, to distribute the compaction energy over the entire surface of the material. The volume of each layer was controlled by three rulers fixed to the inside wall of the container.

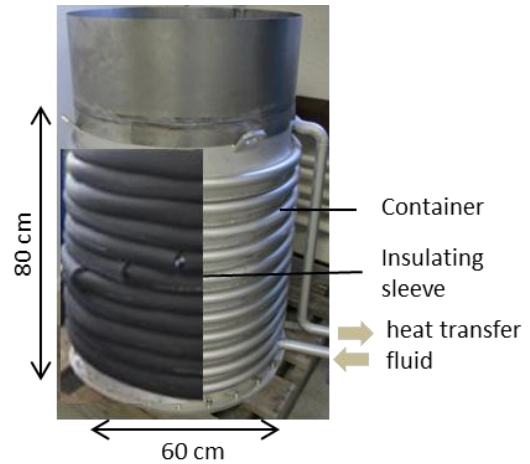


Figure 6. Container with insulating sleeve.

Seven thermal sensors were positioned within the sample to monitor temperature evolution inside the compacted soil during the variation of the imposed temperature. The soil was covered by a plastic cap to avoid drying. Finally, the thermal insulation box was set and the temperature variation was applied by the thermoregulator to reach the target temperature.

In this study the illitic material, prepared at a water content of 31.3% was compacted at a dry density of 1.29 Mg/m³, which is 90% of the Proctor optimum dry density. The massif was compacted in 11 layers of 70 mm.

4.3 Pressuremeter tests

Before each pressuremeter test, a core drilling with a diameter equal to the pressuremeter probe was made using a hollow core drill. The 630 mm core is divided into pieces of 25 mm height to determine the water content and the density of the material as a function of the depth. The pressuremeter tests were conducted with an APAGEO® mini-pressuremeter probe of 340 mm in height and 22 mm in diameter at rest. The probe was connected to a pressure-volume controller brand GDS®. The pressure controlled test consists in applying increasing pressure with equal increments of 25kPa for at least 1 minute. The equilibrium volume is measured for each increment and the volume versus pressure curve was plotted. The

test was stopped when the imposed volume reached $140,000 \text{ mm}^3$.

Four pressuremeter tests were carried out at 20°C (20a, b, c, d) and two at 40°C (40a, b). Average water content and dry density profiles at each step are presented in figure 7. All measured water contents were between 29.5% and 32% while the dry densities ranged from 1.21 to 1.42 Mg/m^3 . These profiles showed acceptable deviations compared to the target values and denoted the satisfactory overall homogeneity of the sample.

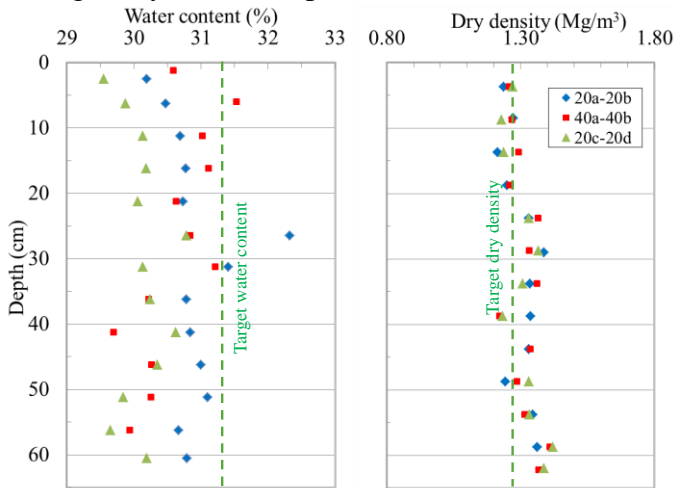


Figure 7: Water content and dry density versus depth

4.4 Results

The test 20a failed due to the membrane leakage. The results of the five other tests are presented in figure 8. The pressuremeter curve consists in 3 steps: (i) the probe inflates to obtain the contact with the wall of the hole, (ii) the volume increases linearly with the increasing pressure allowing for the calculation of the pressuremeter modulus E_M (the soil pseudo-elastic reaction against the probe pressure) and (iii) the large displacements take place and the soil becomes plastic. The creep pressure, p_f is the boundary between the second and the third steps of the test. The limit pressure p_l is the measured pressure when the injected volume reaches twice the original volume of the cavity. Pressuremeter modulus, E_M , and limit pressure, p_l and creep pressure, p_f , were determined for each test (Table 1).

The results showed no significant variation of E_M with temperature while p_f , p_l values decreased with heating to 40°C . One can notice that for the tests at 40°C , the plastic deformations started earlier than at 20°C reflecting a reduction of the elastic domain.

The decrease of the studied parameters could be related to the decrease of suction and the thermal softening due to increasing temperature as discussed for the penetration test.

The decrease of the yield limit with heating was also observed during triaxial tests (Lingnau et al. 1996, Graham et al. 2001, Cekerevac & Laloui 2004

and Uchaipichat and Khalili (2009)) and vane tests (Marques et al. 2004).

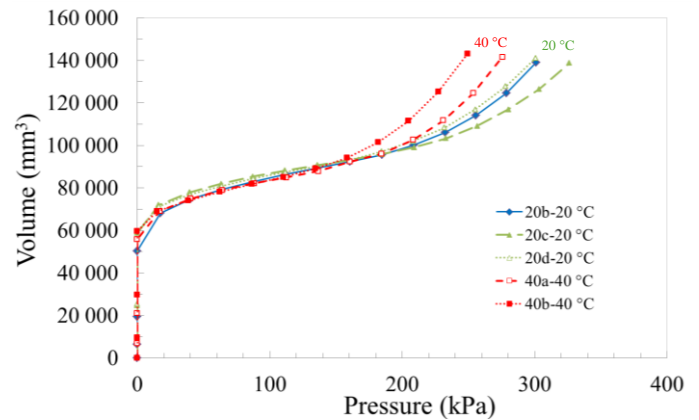


Figure 8: Pressuremeter curves at 20 and 40°C .

Table 1. Pressuremeter modulus, E_M , creep pressure, p_f and limit pressure, p_l at 20 and 40°C

Test	E_M (MPa)	p_f (kPa)	p_l (kPa)
Test 1, 20°C	3.55	198	357
Test 2, 20°C	4.09	200	380
Test 3, 20°C	3.52	180	353
Test 4, 40°C	3.52	163	317
Test 5, 40°C	2.94	162	283

5 CONCLUSIONS

The objective of this study was to develop laboratory repeatable and reliable tests to define the effects of temperature variations on the compacted soil pile-design parameters. Two laboratory tests were performed: the mini-penetration test and the pressuremeter test.

For the mini-penetration, we showed that decreasing the initial water content of the samples increased both the cone resistance and the sleeve friction resistance. Globally, the initial dry density of the sample seems to have less impact on the penetration parameters than the water content. Both the cone resistance and the sleeve friction resistance vary more sharply on the dry side of the compaction curve than on the wet side. For the soil compacted on the dry side of the optimum and then heated, a slight decrease in the cone resistance and in the sleeve friction resistance was observed. On the wet side, the changes in cone resistance and sleeve friction resistance with temperature variations were not significant.

The first results obtained with the mini-pressuremeter tests showed a decrease of the limit pressure and the creep pressure with temperature while pressuremeter modulus remains almost constant. These results tend to reflect a decrease of the elastic domain with temperature. Cyclic temperature variation tests will be carried out by this method to study the pressuremeter parameters evolution.

Ongoing work is dealing with measuring the impact of the obtained soil parameters variations on the soil-pile system behavior.

6 ACKNOWLEDGEMENTS

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